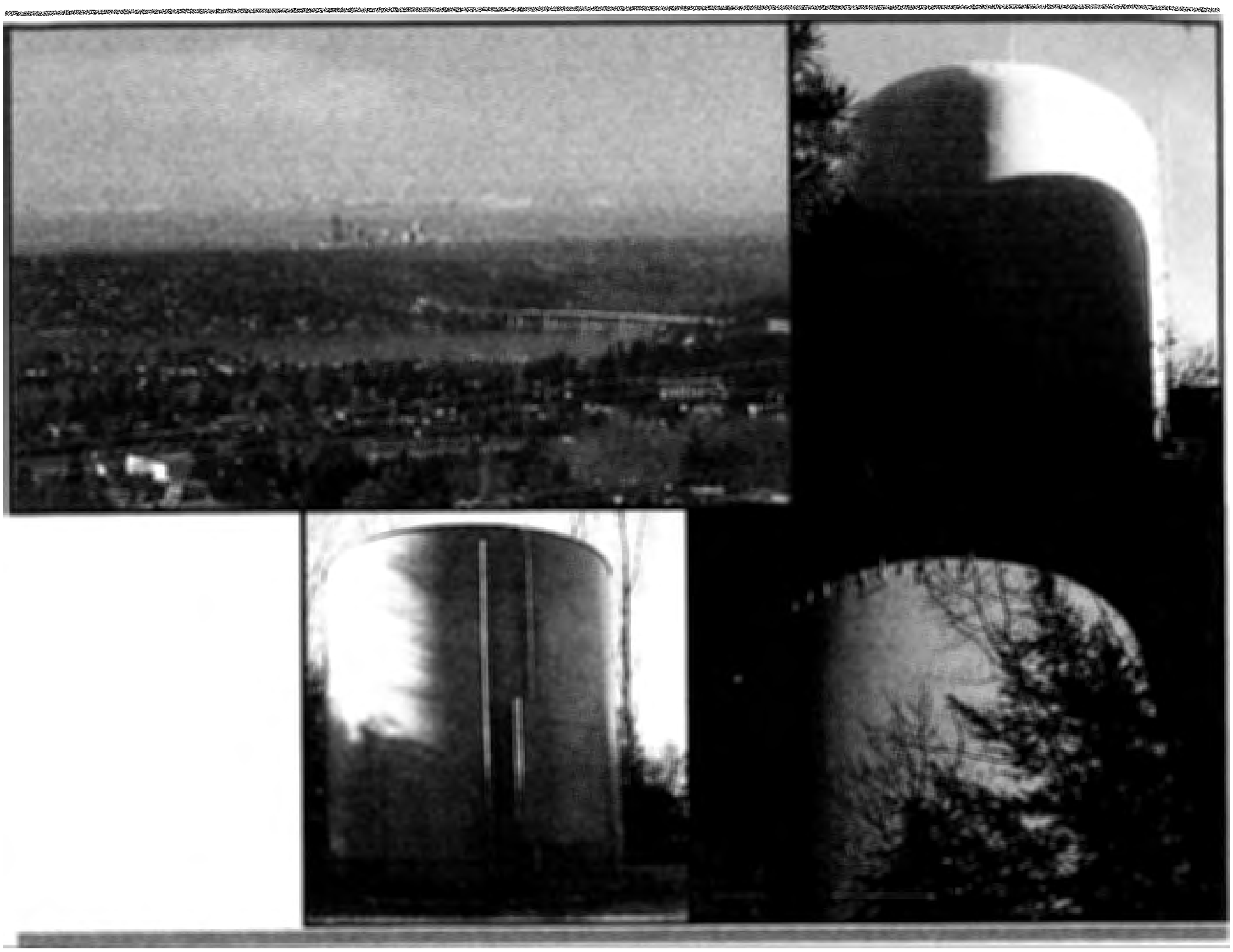

City of Bellevue

Preliminary Engineering Design Report for

Reservoir Structural/Seismic Evaluation

March, 1999

MONTGOMERY WATSON

MONTGOMERY WATSON

March 16, 1999

Mr. Regan Sidie
Engineering Division
Utilities Department
City of Bellevue
301 – 116th Ave SE, Suite 300
Bellevue, WA 98009-9012

Subject: Reservoir Structural/Seismic Evaluation
Final Report

Dear Mr. Sidie:

We are please to submit the final report for our Reservoir Structural/Seismic Evaluation. We have included three copies for your use and will submit the remaining copies in accordance with our agreement as soon as possible.

The final report incorporates both your review comments and our internal review comments. We have also included a list of our responses to your comments.

We have enjoyed working with you and your staff in the preparation of this report and look forward to opportunities to serve you in the future.

If you have any questions concerning the final report or desire any other assistance, do not hesitate to call.

Sincerely,

Thomas P. Delaney

Tomas P. Delaney, P.E., S.E.
Project Engineer

cc: Harry Dunham, MW

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RESPONSES TO QUESTIONS AND COMMENTS
Draft Report for Reservoir Structural Seismic Evaluation

March 16, 1999

GENERAL COMMENTS

COMMENT 1 What is expected remaining useful life for these reservoir? This can help us decide how much rehabilitation to do. For instance, if there is only ten to fifteen years of use left in a structure, we might design to less than Zone 4 level and save money for reconstruction.

What damage will occur at a reservoir designed for Zone 3, when a Zone 4-level event occurs? Possibly create a table to aid in decision making (e.g., including life of tank without any upgrade, will tank remain functional without damage, will tank remain functional but have some damage, etc.)

For the sake of comparison, what is cost to reconstruct new reservoirs at each site?

RESPONSE: The expected useful life of the reservoirs will be shown in Table 11-1. The remaining part of this question has been addressed in "Seismic Event Evaluation" subsection of Section 2.

The estimated cost to reconstruct new reservoirs at each site has been shown in Table 11-1.

COMMENT 2 Does Montgomery Watson have an opinion, as to which seismic zone should be used for design? What are other jurisdictions using?

RESPONSE: The response to this comment has been included in the clarifications for the "Design Ground Motions" subsection of Section 2.

COMMENT 3 Is it possible to do a design alternative that allows some damage following the design-level earthquake, but leaves the reservoir operational?

RESPONSE: The response to this comment has been included in the clarifications for the "Design Ground Motions" subsection of Section 2.

COMMENT 4 The report leaves the impression that the cathodic protection systems have not been in use. They actually have been in use, but of course were not energized while the tanks were empty. The sections discussing cathodic protection need to be revised.

Since cathodic protection systems have been in use, does your observation of "trace blisters" and "pinhole coating failures" indicate that we need to readjust the cathodic protection levels?

RESPONSE: It was not the intention of the report to leave the impression that the cathodic protection systems have not been in use. This will be clarified in the "Cathodic Protection Conditions" subsection of Sections 4 through 8.

Yes, adjustment of cathodic protection levels will be clarified in the "Cathodic Protection Condition" subsection of Sections 4 through 8.

COMMENT 5 In general, the seismic upgrade costs are lower than what we expected. What is included in the cost estimates? Do they include engineering, permits, contingencies? The basis for each estimate should be shown in the report.

RESPONSE: The basis for each estimate is included with Table 11-1.

COMMENT 6 Where pipe connections are allowed movement, why are some pipes and not others provided flexible connections? Does it have to do with distance from the edge of the tank? If so, what is the threshold point where flexible connections are required on a pipe?

RESPONSE: Pipe connections to the outside of the shell plate of the reservoir or within the bottom annulus on the inside of the reservoir require flexible connections when there is an uplift condition. This is described in detail and has been clarified in the "Seismic Forces" subsection for each steel reservoir. The width of the annular ring is generally 2 to 3 feet for these reservoirs.

COMMENT 7 What type of flexible connection is recommended for pipe flexibility?

RESPONSE: The "Comparison of Alternatives" subsection of each reservoir has been clarified with recommendations for piping connections.

COMMENT 8 On several reservoirs, the exterior paint coatings may have excessive lead levels and poor adhesion. What further testing is required to confirm these findings? Is this confirmation worth the extra cost? Is removal of high-lead coatings recommended, or should they be encapsulated by a new coating?

RESPONSE: No further testing is required. The lead based coatings may be encapsulated with a new coating after minor surface preparation which includes chipping away areas where coating has delaminated. The "Interior and Exterior Coatings" subsection and has been clarified for each reservoir.

COMMENT 9 Why is allowable bearing pressure shown in ksi units, shouldn't this be in the same units as the actual bearing pressure (ksf)? At what level of seismic loading does AWWA D-100 require the reservoir foundation to be increased to reduce bearing pressure?

RESPONSE: The units for the allowable bearing pressure was changed to ksf. AWWA D-100 does not require the foundation to be increased to reduce bearing pressure. This statement was removed where it was shown in the "Seismic Forces" subsection for each reservoir.

COMMENT 10 Each reservoir assessment should include general statements that improvements for water quality, operations, and safety should be addressed during design for seismic improvements, such as need for sampling devices, new valves, or non-seismic repairs.

RESPONSE: A general note was added to Table 11-1.

COMMENT 11 Constructibility should be considered in evaluations of alternatives. For instance at Lake Hills North and Woodridge the proximity of other buildings may not allow installation of anchors at desired intervals.

RESPONSE: Constructability was considered for the evaluation alternatives and has been clarified specifically for the Lake Hills North and Woodridge reservoirs in the "Comparison of Alternatives" subsections of Section 4 and 6.

COMMENT12 Table 11-1 is missing from report.

RESPONSE: Table 11-1 has been included in the report.

SPECIFIC COMMENTS

Lake Hills North

COMMENT 1 Note that cupola ponding and corrosion should be corrected.

RESPONSE: This was clarified in Section 11.

COMMENT 2 Is there enough room to install anchors where the building is very close to the structure?

RESPONSE: See response to General Comment No. 11.

COMMENT 3 We will want to install a new altitude valve along with seismic upgrade.

RESPONSE: This was added to Section 11.

COMMENT 4 Should piping outside of the pump station receive flexible connections?

RESPONSE: The piping for the pump station should be reviewed as part of the final design for this reservoir. This has been added to Section 11 and the cost added to Table 11-1.

Lake Hills South

COMMENT 1 Why are upgrade costs at Lake Hills South lower than costs for Lake Hills North?

RESPONSE: The difference in cost comes from repairing corrosion in the roof plate and adding drain holes to the cupola at Lake Hill North. This was clarified by adding a note to Table 4-4 and 5-4.

COMMENT 2 The gouges in the tank picture look significant, but were not addressed in the text of the report. Are they significant? Were they measured? Should bollards be installed to prevent future damage?

RESPONSE: These gouges are significant and should be repaired. Bollards should be installed to prevent future damage. This was clarified in the "General Reservoir Condition" subsection of Section 8 and noted in Section 11.

COMMENT 3 Presentation of roof shell thickness is different in Table 5-2 and 4-2. Are the roofs constructed differently?

RESPONSE: The structure for the Lake Hills North and Lake Hills South reservoirs, including the roof structure, is slightly different. This was clarified in the "General Reservoir Conditions" sections for both reservoirs.

Woodridge

COMMENT 1 The Woodridge site is fairly confined. Is there enough room to install anchors?

RESPONSE: See response to General Comment No. 11.

COMMENT 2 A basement area is located under the Woodridge tank (adjacent to the old altitude valve vault). This should be identified in the report. Does this have an impact on the seismic resistance?

RESPONSE: See response to General Comment No. 11.

Horizon View No. 1

COMMENT 1 The shell thickness is much thinner on this reservoir. It was found to be sufficient to resist seismic forces but how close was it to the allowable level?

RESPONSE: The allowable static hoop stress is 15,000 psi and the allowable seismic hoop stress is 17,000. The allowable compression stress varies with each reservoir. The actual stresses are shown in Table 7-3. These allowable stresses were added to the "Seismic Forces" subsection of each steel reservoir.

Parksite

COMMENT 1 The photos show ponding around the reservoir. Note that drainage improvements should be performed to remove this condition.

RESPONSE: Required drainage improvements are noted in the General Reservoir Conditions in Section 8 and Section 11 of the report.

COMMENT 2 Exterior paint coating appears to be delaminated exterior coating, however the report indicates that the coating has excellent adhesion strength. The exterior was coated about four years ago.

RESPONSE: There were a couple of minor instances where the exterior coating was delaminated and some locations on the west side of the reservoir where the coating system was damaged. This damage appeared to be from vandalism and not from coating failure. The damage was added to the "Interior and Exterior Coatings" subsection of Section 8.

Pikes Peak

COMMENT 1 Note that cathodic protection should be added during seismic upgrade work.

RESPONSE: This was added to Section 11.

Somerset No. 1

COMMENT 1 If the reservoir is supporting the road and shoulder, what is supporting the road past the ends of the reservoir (is there a retaining wall or is the road farther from the edge of the slope)?

RESPONSE: The road past the ends of the reservoir is supported by the slope. The reservoir supports the lateral loads resulting from the road because it is benched into the slope as noted in the first paragraph of the "Site Characteristics" subsection of Section 10 of the report.

COMMENT 2 The concrete in the precast panels was noted to be in poor condition "the fine aggregate can be hand rubbed away". If this is considered typical wear for a 35-year old reservoir, what is the life expectancy for a concrete reservoir?

RESPONSE: The life expectancy of the reservoir was addressed by the changes associated with General Comment No. 1. The statement that "fine aggregate can be rubbed away is considered typical wear" for a 35-year old reservoir was removed.

COMMENT 3 Based on poor condition of Somerset No. 1, should the Utility be concerned about Somerset 2 and 3 (they have similar construction, but do not have the unbalanced lateral loads like No. 1)?

RESPONSE: If Somerset No. 2 and No. 3 do not have unbalanced lateral loads similar to Somerset No. 1, it is unlikely there should be as much concern related to seismic retrofit of these reservoirs. However, since these reservoirs were not inspected as part of this report, it is impossible to make that judgement at this time.

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Section 1

MONTGOMERY WATSON

Section 1
Introduction

The City of Bellevue Reservoir Structural/Seismic Evaluation, *Preliminary Engineering Design Report,* presents the structural and seismic assessment of seven of the City's distribution reservoirs. The seven reservoirs include six steel tanks and one concrete tank. The steel tanks evaluated in this study include: Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, and Horizon View No. 1. Somerset No. 1 is the concrete tank evaluated in this assessment. Section I of the Preliminary Engineering Design Report provides a summary of the authorization and scope of work, the general report format, and a list of commonly used abbreviations.

AUTHORIZATION

This Preliminary Engineering Design Report has been prepared in accordance with an agreement between the City of Bellevue (City) and Montgomery Watson (MW). Montgomery Watson received authorization from the City on April 9, 1998 to commence work on the Phase I Evaluation and Preliminary Engineering Design of the City's seven reservoirs.

OBJECTIVE

The objective of this Preliminary Engineering Design Report is to summarize the work completed for Phase I - Evaluation and Preliminary Engineering Design of the City's Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, Horizon View No. 1 and Somerset No. 1 Reservoirs. The locations of these reservoirs are shown on Figure 1-1. In addition, this Preliminary Engineering Design Report establishes the seismic design criteria and presents alternative upgrades for meeting the seismic criteria for each of the seven reservoirs. Cost estimates are provided for the City's use for capital budgeting of the structural/seismic upgrade of the City's reservoirs. This Preliminary Engineering Design Report will also provide the foundation for the Phase 2 Supplemental Services for the design and construction of recommended improvements.

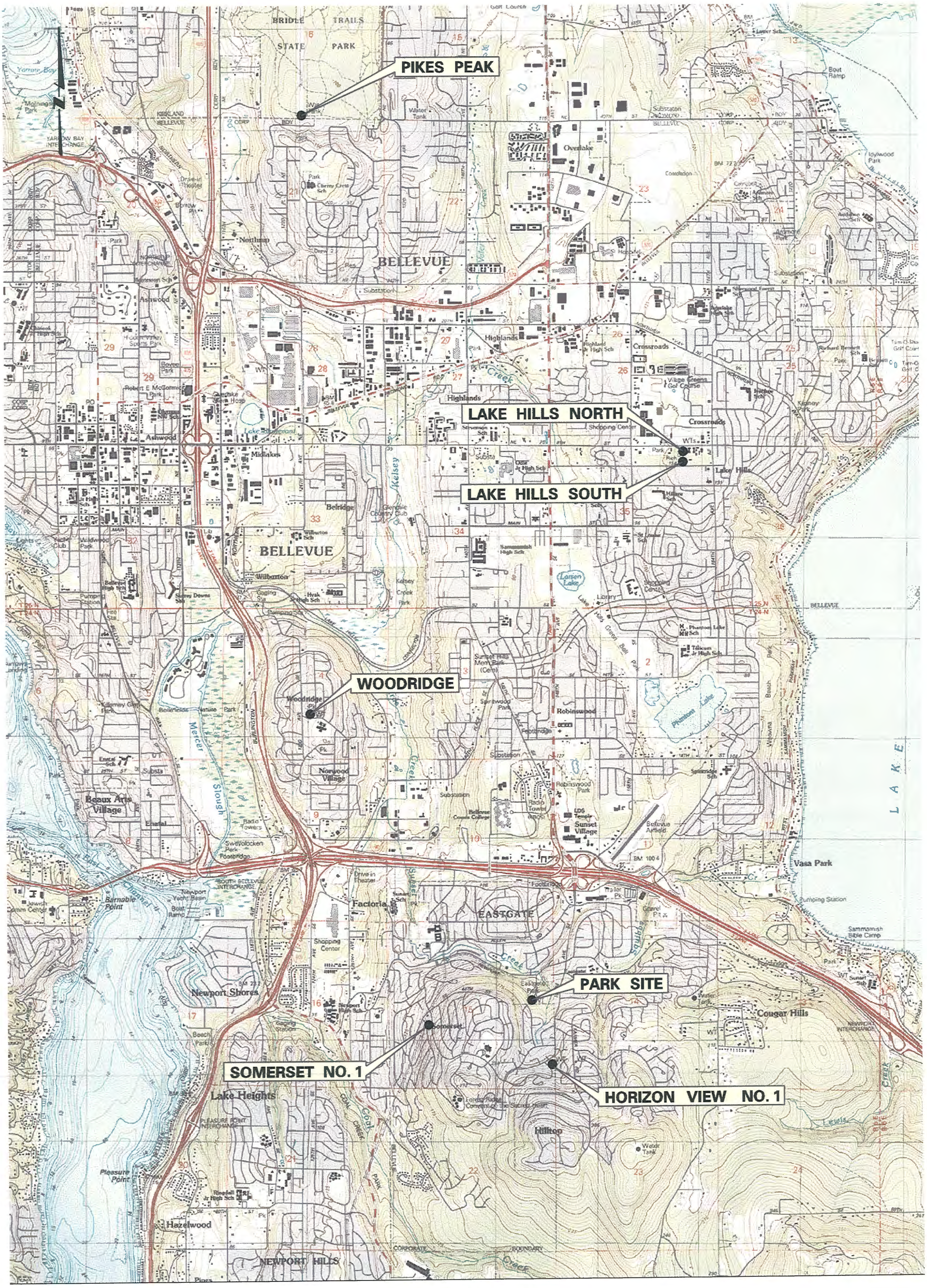
SCOPE OF WORK

The scope of work for the structural/seismic evaluation of the seven City of Bellevue reservoirs was developed in concert with the City. The following paragraphs provide a summary of the Tasks that comprise the Scope of Work for Phase 1 - Evaluation and Preliminary Engineering Design.

Task I - Review Existing Information

Collect and review data relevant to the design and construction of the City of Bellevue's six steel reservoirs and one concrete reservoir. Data to be provided by the City includes:

1. Design/construction drawings and specifications.
2. Shop drawings.



City of Bellevue
Reservoir Location Map
Figure 1–1

3. Geotechnical data and reports for the reservoir sites and foundation systems, if available.
4. Notes/reports of tank inspections.
5. Cathodic Protection monitoring records.
6. Protective Coatings maintenance history including most recent recoating documentation.
7. Other information which may be beneficial in the evaluation of the steel reservoirs.

Task 2 - Conduct Site Investigation

Conduct an on-site investigation of each reservoir. The purpose of the on-site investigation is to obtain supplemental information, not contained in the City records, for each reservoir and to conduct a condition assessment of the reservoir structure and site conditions.

Task 3 - Determine Appropriate Seismic Event

Prepare for and conduct a workshop with City personnel and project team members for the purpose of formulating the seismic event that represents the level of reliability the City desires for their reservoir facilities. In addition, the methodology, which will be utilized to estimate the site specific parameters during the structural evaluation, will be established. Results of the workshop will be summarized in a Technical Memorandum.

Task 4 - Geotechnical Investigation

Review information contained in the City records and published geologic maps. Conduct individual site investigations at each of the seven reservoir sites. Site investigations include subsurface exploration consisting of borings and test pits. The data obtained from records and the field/laboratory investigation will provide the basis for the allowable static and dynamic soil bearing pressures and modulus of subgrade reaction for each site. Allowable capacities for deep foundation systems to increase resistance to overturning and uplift will be determined. Based on the selected seismic design event, site-specific response spectra for each site will be developed. Peak ground acceleration representative of the risk levels will also be identified.

Task 5 - Corrosion Investigation

Interior surfaces of the roof, shell, and floor will be inspected for condition of protective coatings, corrosion and other damage to the steel. Examination of all steel connections and pipe penetrations will be completed. Cathodic protection tests will be performed to determine coating efficiency, cathodic protection design, and for development of cost comparisons.

Inspect interior coating systems to determine if they qualify for repair versus replacement. Exterior coatings will be tested for adhesion, disbondment between layers of coatings and steel surfaces, chalking that may effect over-coat adhesion, compatibility with current coating formulas, and coating thickness. Corrosion condition assessment of welded connections will also be completed.

Task 6 - Detailed Structural Investigation

Using the information and data gathered from review of historic data and the reservoir site investigations, a seismic evaluation of the current condition of the City's Reservoirs will be completed. The modes of potential failure during a seismic event for the steel tanks include: sliding, overturning, and uplift. The seismic evaluation will consist of the following elements:

1. Review and summarize the design criteria from the construction drawings and specifications, and geotechnical reports.

2. Analyze tanks for seismic capacity in accordance with the AWWA D100, *Standard for Welded Steel Tanks for Water Storage,* American Concrete Institute 350, and the 1997 Uniform Building Code standards to identify seismic deficiencies. Elements to be evaluated for the steel tanks include the tank walls (buckling), uplift of the base plate, sloshing height (roof), hoop stresses, overturning, sliding, soil bearing capacity, piping connections, and column supports. The concrete tank will be evaluated for overall structural stresses induced during contents sloshing, unbalanced lateral earth loads during a seismic event, and partial loss of foundation support.

3. Assess risk for tank failure relative to original tank design criteria versus AWWA D100 Standards and the 1997 UBC Standards.

Task 7 - Reservoir Piping Vulnerability

Assess the inlet/outlet and overflow piping configuration at each reservoir. Determine the degree of flexibility of each existing piping configuration and locations of potential points of failure at each reservoir, and develop alternatives that will limit the potential points of failure.

Task 8 - Develop Retrofit Alternatives and Cost Estimates

Develop a minimum of two earthquake mitigation measures that will provide the desired level of seismic resistance assigned to each of the seven reservoirs. Recommended seismic upgrades may include: anchorage improvements, shell improvements, or provisions for flexibility in piping connections. Conceptual level strengthening details will be presented, as will preliminary construction cost estimates for comparison and budgeting purposes.

Task 9 - Prepare Preliminary Engineering Design Report

Prepare a report summarizing the work completed for Tasks 1 through 8. The report will include a summary of original tank design criteria, findings of the site investigation, seismic assessment of the current tank conditions, development of alternative seismic retrofits, and an evaluation of the available alternatives. The report will also include recommendations and budget estimates for construction costs as warranted for seismic upgrades. Conceptual details for recommended seismic upgrades will also be included in the report.

REPORT FORMAT

The Preliminary Engineering Design Report for the City of Bellevue Structural/Seismic Evaluation of Reservoirs consists of the following two volumes:

Volume I - Preliminary Engineering Design Report
Volume II - Preliminary Design Memoranda and Notes

Volume I of this Preliminary Design Report consists of eleven sections. The first three sections of the report provide an introduction to the project, project design criteria, and a general description of the seven reservoirs. Sections 4 through 10 include the individual analysis of each of the seven reservoirs, and Section 11 provides a summary of conclusions and recommendations for the reservoirs. Each section has a number of subsections indicated in bold type as follows:

FIRST-LEVEL SUBSECTION HEADING
Second-Level Subsection Heading
Third-Level Subsection Heading

A list of references, reservoir photographs, and structural calculations are included in the appendices of Volume I. In addition, the field notes from the site investigations, the seismic design criteria technical memorandum, the geotechnical memorandums, and the corrosion assessment technical memorandums have been included in Volume II of this Preliminary Engineering Design Report.

ABBREVIATIONS

To conserve space and improve readability the following abbreviations have been used in this report.

Abbreviation	Meaning
AISC	American Institute of Steel Construction, Inc.
AWWA	American Water Works Association
AWWA D-100	American Water Works Association - Standard for Welded Steel Tanks for Water Storage
City	City of Bellevue
DBE	Design Basis Earthquake
ft	feet
g	acceleration of gravity
k-ft	kip-foot (moment)
Kips	1000 pounds
Ksf	kips per square foot
ksi	kips per square inch
km	kilometer
lbs	pounds
M	magnitude, as measured for earthquakes
Mw	moment magnitude
MCE	Maximum Credible Earthquake
MG	Million gallon
mgal	million gallon

MM	Modified Mercalli
MW	Montgomery Watson
NACE	National Association of Corrosion Engineers
OBE	Operating Basis Earthquake
pcf	pounds per cubic foot
psf	pounds per square foot
psi	pounds per square inch
UBC	Uniform Building Code

Section 2

MONTGOMERY WATSON

Section 2
Design Standards and Seismic Criteria

The criteria to be used in the analysis of the City's seven reservoirs will be consistent with current industry design standards and seismic criteria. A summary of terms and definitions used in the structural/seismic evaluation of the reservoirs, the applicable reservoir design standards, and a discussion of the recommended seismic design criteria is presented in this section.

DEFINITION OF STRUCTURAL SEISMIC TERMS

There are several terms presented in this report that are used in the discussion of the structural/seismic characterization and evaluation of the reservoirs. Some of these terms and their definitions are summarized below.

Base Shear

During an earthquake, the ground moves horizontally, causing the tank and its contents to accelerate. This acceleration acts on the entire tank and its contents. The amount of horizontal force that occurs at the bottom of the tank caused by this acceleration is called base shear.

Overturning Moment

Reservoirs are supported at their base, however, the center of gravity of the tank and its contents is well above the ground level. During an earthquake, the horizontal force acting on the tank's base accelerates the tank and its contents through the tank's center of gravity. The acceleration through the tank's center of gravity creates an overturning moment that causes uplift on one side of the tank and a significant downloading on the opposite side.

The ratio of a tank's diameter to its height will determine the effect of the overturning moment. The weight of the tank wall, and to some small degree the weight of water, of a large diameter tank will resist the uplift caused by the overturning moment. In contrast the overturning moment of small diameter, tall tanks may be great enough to topple the tank, if the tank is not properly held in position.

Anchorage

In order to hold down or anchor the tank, bolts are added around the perimeter of the base of tanks, and are embedded into a reinforced concrete foundation. The bolts around the perimeter of the tank are referred to as anchorage.

Convective and Impulsive Forces

During an earthquake, water within a reservoir will slosh back and forth due to the ground motion. In a tall, small diameter tank, only the water near the upper surface will be involved in this wave action. In a large diameter, short tank most of the water depth will be involved in this wave action. An analogy would be the ocean near the shore where the wave action affects the water all the way to the bottom, but in the middle of the ocean the wave action only affects the surface.

In a reservoir that is undergoing an earthquake, the water that is involved in wave action causes a convective force. Water below the wave action will still be subject to acceleration from the earthquake, and causes an impulsive force. Taller, small diameter tanks will have a greater depth of water resulting in an impulsive force.

In designing a reservoir for seismic loads, the convective and impulsive forces and the height above the base where they occur are determined. These forces are the majority of the base shear. The height of these forces above the base of the tank result in the majority of the overturning moment. The weight of the reservoir walls and roof also contribute to the base shear and overturning moment.

INDUSTRY DESIGN STANDARDS

The American Water Works Association has developed industry design standards for both welded steel and pre-stressed concrete water storage tanks. These standards are evolving and are updated regularly to reflect new data and technologies within the industry. The AWWA standards in turn reference other building and design standards. These other standards include the American Concrete Institute and the Uniform Building Code. The applicable standards for the structural evaluation of the City's seven reservoirs include the following.

- AWWA D-100 Standard for Welded Steel Tanks for Water Storage
- American Concrete Institute 350
- Uniform Building Code 1997

The primary standard AWWA D-100 Standard for Welded Steel Tanks is the primary standard and its applicability to the structural evaluation of the City's reservoirs is summarized in the following paragraphs.

AWWA D-100 Standard for Welded Steel Tanks for Water Storage

The AWWA D-100 Standard for Welded Steel Tanks for Water Storage is appropriate for the Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge and Horizon View No. 1 reservoirs. Analysis and design standards for steel tanks have evolved considerably since the construction of the City's reservoirs. In 1965, Appendix C was "added to provide for the alternative use of higher strength steels for standpipes and reservoirs." The rules for welding, weld qualification, and weld testing were also completely rewritten in 1965. Riveting was not eliminated from D-100 until 1973. In 1979, the first seismic provisions were added, but were non-mandatory.

Since the City's reservoirs were constructed prior to 1979, AWWA D-100 has undergone two major revisions that have affected the seismic provisions. The 1984 revision of AWWA D-100 contained two approaches for designing steel tanks for seismic loads: fixed percentage and pseudodynamic. The pseudodynamic approach was required in Seismic Zone 4 and either approach was allowed for tanks in Seismic Zones 1, 2, and 3. The 1996 AWWA D-100 standard requires that the pseudodynamic approach be used in Seismic Zones 1, 2A, 2B, 3 and 4. The pseudodynamic approach accounts for the effects of water sloshing due to ground motion. Because of this provision, the pseudodynamic approach is a more accurate representation of how the tank will respond during a seismic event. In general, the pseudodynamic approach will result in larger seismic forces for tanks with a diameter to height ratio of approximately 2.0 or less.

SEISMIC EVENT EVALUATION

The seismic criteria are in a state of flux within the Puget Sound area, as ongoing seismologic and geological studies are constantly providing new data that revises characterization of earthquake sources and the level of hazard within the region. A workshop presenting the current understanding of seismic hazards was held in mid-June, during which site-specific seismic safety criteria to be used in the evaluation of the City's reservoirs was determined. Those present at the workshop included City of Bellevue personnel and representatives from Montgomery Watson, HWA GeoSciences Inc., and Woodward-Clyde. The summary of this workshop and the resulting criteria follows.

Mr. Ivan Wong of Woodward-Clyde prepared a technical memorandum, Seismic Hazards Evaluation for the City of Bellevue Distribution Reservoirs. This technical memorandum summarizes the conclusions reached during the workshop, and the recommendations for site-specific seismic safety criteria to be used in the evaluation of Lake Hills North and South, Horizon View No. 1, Parksite, Pikes Peak, Somerset No. 1, and Woodridge Reservoirs. The following excerpts from the technical memorandum include a discussion of the seismic environment, design earthquakes, soil profile types, design ground motions, and surface faulting. The Technical Memorandum is included in its entirety in Volume II of this report.

Seismic Environment

The City of Bellevue is located in the Puget Sound region, which has been recognized for its relatively high level of seismic hazards. Two categories of potential seismic sources are significant to the City: (1) the Cascadia subduction. zone and (2) shallow crustal faults. These potential seismic sources are shown on Figure 2-1. Within the subduction zone, both interplate earthquakes (those events which rupture the interface (megathrust) between the subducting Juan de Fuca plate and the overriding North American plate) and intraplate earthquakes (within the Juan de Fuca plate) can occur and generate damaging ground motions and other associated hazards.

Based on a model proposed by Hyndman and Wang (1995) and modified by Wong and Silva (1998), Bellevue is at a distance of about 150 km from the eastern edge of the megathrust rupture. The crustal fault of greatest significance to the City is the Seattle fault because of its proximity and capability to generate large earthquakes ($M > 7$). Based principally on geophysical and seismic data, the Seattle fault is believed to trend east-west across the southern portion of the City (Sam Johnson, USGS, written communication, 1997). The fault consists of at least four splays in the near surface, three of which cut across the City. These potentially active faults are shown on Figure 2-2.

The earthquakes of possibly greatest significance to the Bellevue area are crustal events (M ~: 7) similar to the one that shook the Seattle area about 1,100 years ago. This earthquake, which may have resulted from rupture of the Seattle fault, caused landslides into Lake Washington, rock avalanches in the Olympic Mountains, liquefaction, and a tsunami in the Puget Sound (Adams, 1992).

The most recent megathrust earthquake along the Cascadia subduction zone is believed to have occurred on 26 January 1700. Based on historical accounts and computer modeling of the resulting tsunami, Satake et al. (1996) suggested that the earthquake was as large as moment magnitude (K) 9, and that it may have ruptured the full I 100-km length of the Cascadia subduction. zone. Such a large interplate earthquake could generate peak horizontal accelerations greater than 0. 15g in the Seattle area (Wong and Silva, 1998).

The largest earthquake to occur in historical times in the Puget Sound region was the 13 April 1949, M 7.1 Olympia event that had its source at a depth of 54 km within the Juan de Fuca plate (Figure 2-1). A peak horizontal acceleration of 0.07g was recorded at the Federal Office Building in downtown Seattle. Modified Mercalli (MM) intensity VEII was reported in Seattle from this earthquake. The second largest historical event was the 29 April 1965 M 6.5 Seattle-Tacoma earthquake, which occurred about 20-km north of Tacoma at a depth of approximately 60-km. The strong motion instrument in the Seattle Federal Office Building recorded a peak horizontal acceleration of 0.079. The maximum intensity of this event was MM VIII in localized areas of Seattle. A MM VI was reported in Bellevue. Both the 1949 and 1965 earthquakes were intraplate events.

Design Earthquakes

In order to evaluate the seismic vulnerability of the City's reservoirs, it is recommended that three design earthquakes defined by American Water Works Association (AWWA) be adopted:

Operating Basis Earthquake (OBE) - Minor repairable damage but facility/structure will remain functional; 50% probability of exceedance in 50 years (72-year return period).

Design Basis Earthquake (DBE) - Facility/structure remain standing; 10% probability of exceedance in 50 years (475-year return period).

Maximum Credible Earthquake (MCE) - Facility/structure must remain in operation in order to avoid catastrophic consequences.

The DBE and OBE are probabilistic-based earthquakes in contrast to the MCE, which is deterministic in nature. That is, the probability of occurrence of the MCE is not considered, but rather that it is a rational event that has a reasonable chance of occurring. The DBE is defined assuming a probability level or return period, which is identical to that of the 1997 Uniform Building Code (UBC). The national probabilistic seismic hazard map for a 475-year return period recently produced by the United States Geological Survey (USGS) (Frankel et al., 1996) indicates that the peak horizontal acceleration in the Bellevue area, assuming rock site conditions, is between 0.3 to 0.4g. For the 72-year return period OBE, the probabilistic peak horizontal acceleration for the Seattle area is about 0. 12g.

It should be noted that some controversy exists in the engineering community in Seattle regarding the new USGS maps because of the increased hazard compared to the previous national hazard maps. The hazard increase is due to the inclusion of the Seattle fault whose earthquake potential was not clearly understood until 1991. Despite this controversy, it is recommended that the 1996 USGS maps provide the basis for selecting the DBE ground motions (on rock) for the reservoirs.

None of the reservoirs is considered to be a high enough risk by the City to warrant a MCE design event. If design for a MCE event is required, the ground motions should be calculated based on a Mw 7 ½ occurring on the Seattle fault.

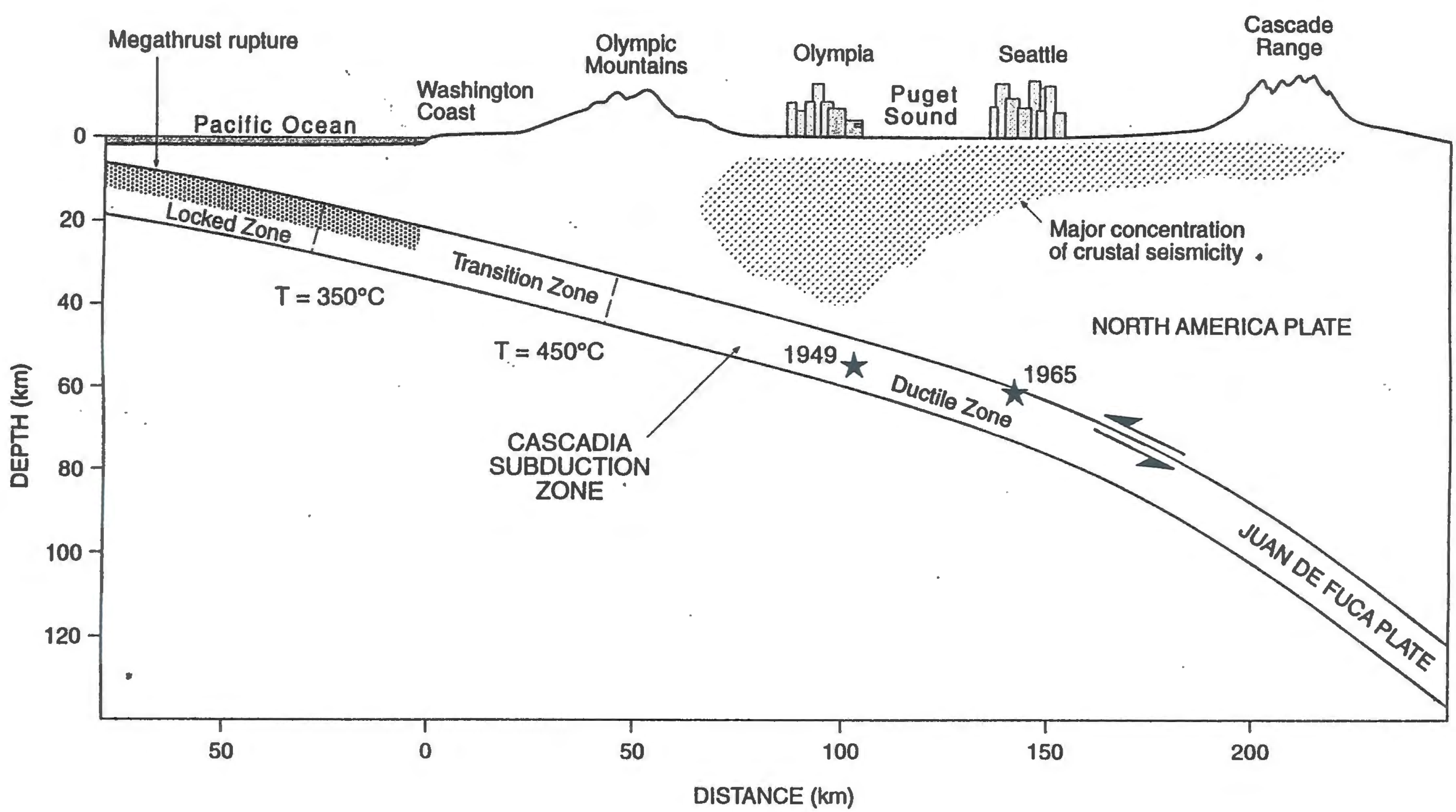
Soil Profile Types

The USGS maps only provide ground motions on rock, and do not include ground motions on soil. Thus design ground motions for sites located on soil need to be adjusted for site response effects. Based on the geotechnical exploratory investigations by HWA GeoSciences (Technical Memorandum dated 8 October 1998), all seven reservoirs are located on Uniform Building Code soil type S_c Classification of soil types was based on Standard Penetration Test blow counts per foot within the top 100 feet of the soil profile. Blow counts for the Lake Hills North and South, Pikes Peak, Parksite, Horizon. View No. 1 and Somerset No. 1 reservoirs were generally in excess of 50, consistent with a S_c soil type. Given the lack of site-specific shear velocity data, this Uniform Building Code approach is considered acceptable although the Woodridge reservoir site, which is situated on advanced outwash, may be close to a S_d soil category.

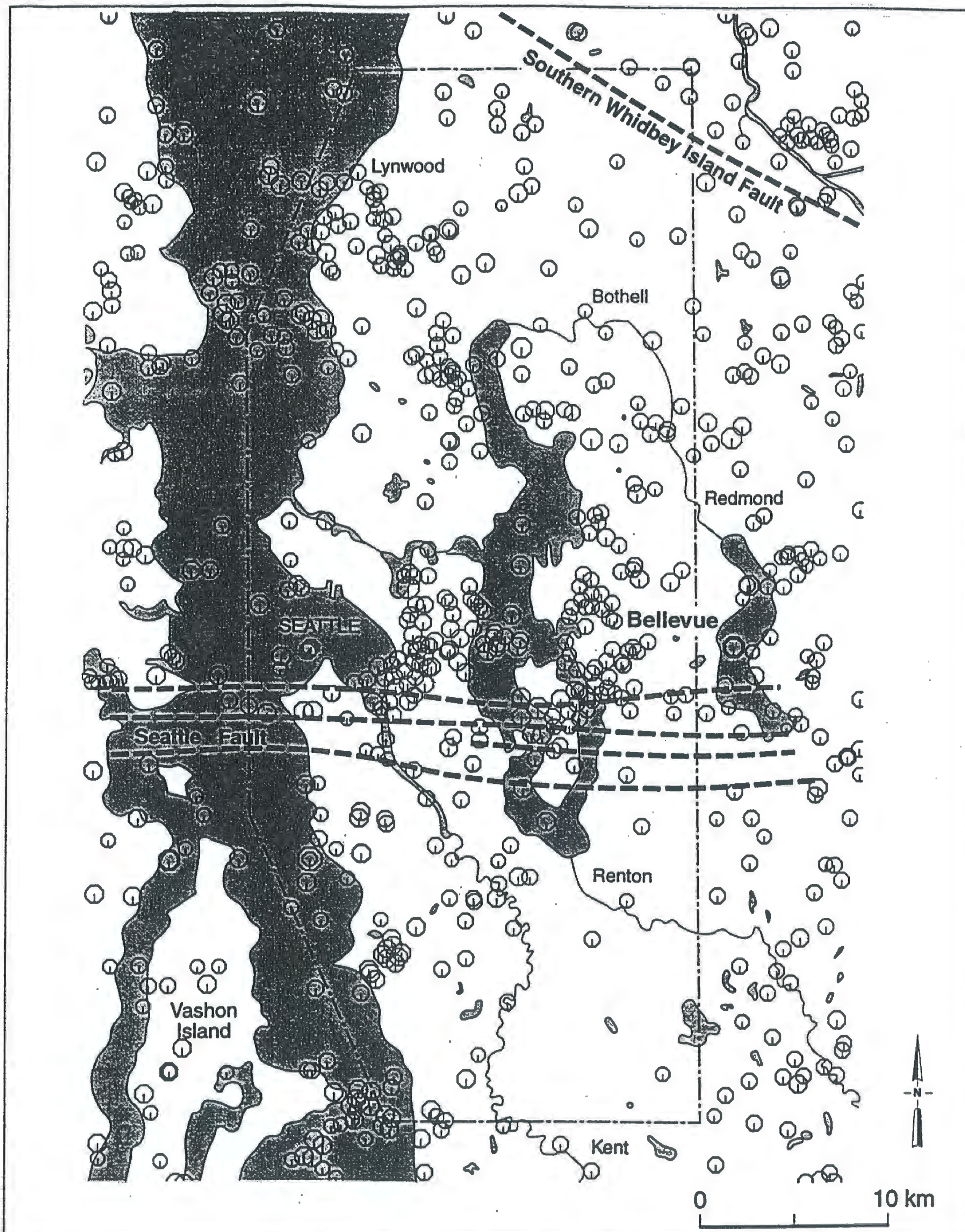
Design Ground Motions

The workshop participants concluded that the City adopt two levels of Design Basis Earthquake ground motions for the seismic safety evaluation of the City's reservoirs. These two levels include criteria presently specified for Uniform Building Code Seismic Zones 3 and 4. As described earlier, the 1996 USGS national hazard maps indicate a 475-year return period probabilistic ground motion range from 0.3 to 0.4g for the Seattle area. Such values would be appropriate for Uniform Building Code Seismic Zone 4 although the process of officially going from Zone 3 to 4 is lengthy and always controversial. At this time, no official process has been initiated to move to Zone 4.

As described in the HWA GeoSciences Technical Memorandum, the seismic coefficients for Seismic Zones 3 and 4 are 0.3g and 0.4g, respectively. Because of the proximity of the reservoirs to the Seattle fault (classified as a UBC seismic source type B, i.e., maximum earthquake Magnitude of 7.0 and slip rate < 5 mm/yr.), the near-source factors for Zone 4 need to be applied. These factors are distance-dependent, thus the distances between the reservoir and the Seattle fault were measured by HWA GeoSciences and used in the selection of the appropriate factors. Their distances range from less than 2-km to 7.5-km, as shown on Figure 2-3.

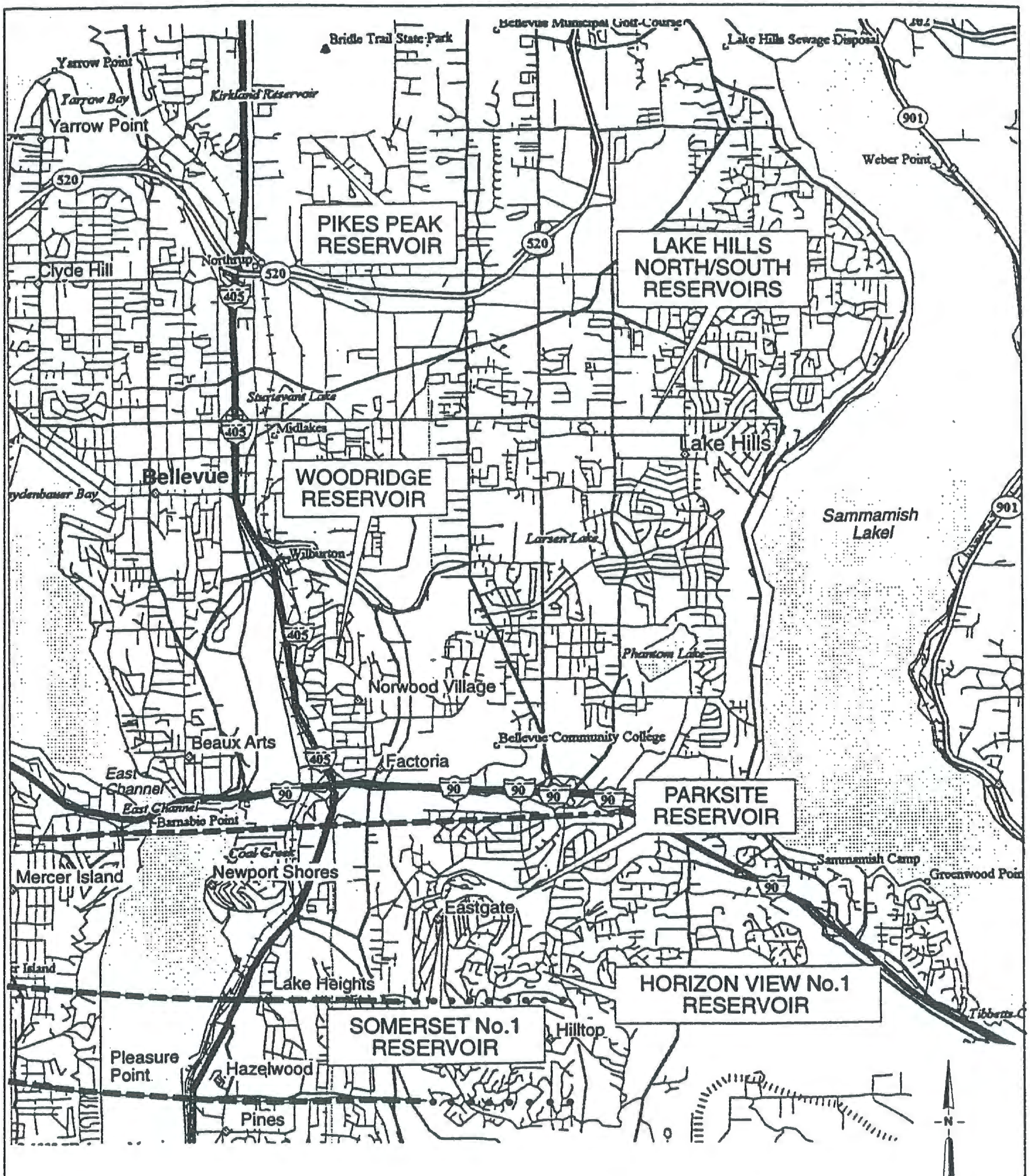


Project No. SK9830	Bellevue Reservoirs	CROSS-SECTION THROUGH THE CASCADIA SUBDUCTION ZONE AT LATITUDE OF SEATTLE	FIGURE 2-1
Woodward-Clyde Federal Services			



Project No. SK9830	Bellevue Reservoirs	POTENTIALLY ACTIVE FAULTS AND HISTORICAL SEISMICITY (1871 to 1997) OF THE SEATTLE METROPOLITAN AREA	FIGURE 2-2
Woodward-Clyde Federal Services			

SK9830-1000/101298/gos



LEGEND

Fault; dashed where approximately located, dotted where inferred.

(Figure modified from HWA Geosciences)

Project No. SK9830	Bellevue Reservoirs	APPROXIMATE LOCATIONS OF THE SEATTLE FAULT AND RESERVOIR SITES	FIGURE 2-3
Woodward-Clyde Federal Services			

SK9830-1000/101298/gos

If the reservoirs are designed to meet the criteria for a Zone 3 Design Basis Earthquake (DBE) and one occurs, it is anticipated that the reservoirs will sustain some damage but will remain functional. However, some repairs will be required to get them back to their pre-event condition. If the reservoirs are designed for a Zone 3 DBE, and a Zone 4 event occurs, the reservoirs will sustain a significant amount of damage, will likely no longer hold water, and will require complete replacement. If the reservoirs are designed for a Zone 4 DBE and a Zone 4 event occurs, it is anticipated that the resulting damage will be similar, if not identical to, the damage that will result with Zone 3 DBE design and the occurrence of a Zone 3 event (e.g., they will sustain some damage but will remain functional).

Designing the reservoirs for a Zone 4 DBE would result in an increase of approximately 25% in capital improvement cost. With this increased cost comes the advantage of having a much more reliable reservoir. An additional advantage to retrofit the reservoirs for a Zone 4 DBE is that, although there has been no official process initiated to move to Zone 4 in the UBC at this time, based on recent research by the USGS it is possible that UBC will require design for a Zone 4 earthquake in the future.

There is another issue worth noting that could affect the design level for seismic retrofit of the reservoirs. That is the City's future use of water from the Seattle pipeline. The City of Seattle currently uses site specific seismic hazard risk analysis to determine the appropriate seismic criteria to design new improvements. The results of these analysis at times sets criteria which exceeds the UBC Zone 3 criteria. However, the existing pipeline is relatively old and much of it has not been retrofitted for seismic considerations. Therefore, it is possible that some of the water supply from this pipeline will be lost subsequent to a seismic event and the affect on the City of Bellevue's water supply could be significant. The integrity and storage capacity of the City's reservoirs, in the event of loss of supply from the Seattle pipeline, would thus be of critical importance. It should also be noted that other utilities similar to the City of Bellevue that use water from the Seattle pipeline use criteria similar to this. The actual criteria set typically depends on the importance of each reservoir to the water supply of a given area.

Surface Faulting

As described in the HWA GeoSciences Technical Memorandum, no surface faulting of the Seattle fault had been previously observed. However, a recent investigation by the USGS suggests the potential for surface rupture or near-surface deformation may exist at least along some localized portions of the fault zone. The Parksite, Horizon View No. 1, and Somerset No. 1 reservoirs are located within the Seattle fault zone between the two northern traces of the fault, as shown on Figure 2-3. Although the potential for surface rupture is still considered to be very low, the hazard for surface rupture may require further investigations.

Summary of Seismic Design Criteria

Summarizing the recommendations resulting from the seismic event hazard workshop, the following criteria are to be considered in the structural/seismic evaluation of the City's reservoirs.

- Design Earthquakes:
  - -- Design Basis Earthquake (DBE) - peak horizontal acceleration, 0.3g to 0.4g.
  - -- Operating Basis Earthquake (OBE) - peak horizontal acceleration, 0. 12g.
  - -- Maximum Credible Earthquake (MCE) - not applicable for the City's reservoirs.
- 1996 USGS Map form the basis for selecting DBE ground motions on rock.
- Design Basis Earthquake Ground Motions - Seismic Zones 3 and 4 are both to be considered in the evaluation of the City's Reservoirs.
- Applicable near-source factors - Seismic Zone 4.
- Surface Faulting - very low potential.
- Uniform Building Code Soil Classification Profile Type S_c.
- Uniform Building Code Importance Factor I=1.25

The design criteria summarized above provide the general guidelines for the evaluation of the City's reservoirs. The specific design criteria, safety factors, and seismic evaluation criteria appropriate for the individual reservoirs are presented with the discussion of the individual reservoirs.

Section 3

MONTGOMERY WATSON

Section 3
Existing Reservoir Conditions

Previous studies and design drawings, provide general background data for the Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, Horizon View No.1, and Somerset No. 1 reservoirs. From this available data the general descriptions and physical characteristics of the Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, Horizon View No. 1, and Somerset No. 1 reservoirs has been summarized. Summaries of the reservoirs' general descriptions and physical characteristics are presented in this section.

GENERAL DESCRIPTION

As shown on Figure 1-1, the seven reservoirs included in this structural/seismic evaluation are located throughout the City of Bellevue. Six of the seven reservoirs, Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, and Horizon View No. I reservoirs are of welded steel construction. The seventh reservoir, Somerset No. 1 is precast reinforced concrete construction. The general description of each of the reservoirs includes the location of the reservoir, the type of construction, the design date, the Engineer, and where known, the designer/fabricator of the reservoir. These general characteristics are summarized in Table 3-1. With the exception of the Pikes Peak reservoir, the design engineer of record is Harstad & Associates.

The City's seven reservoirs that are included in this evaluation were all designed and constructed prior to 1970. The Woodridge reservoir is the oldest of the reservoirs with a design date of 1955; Pikes Peak is the youngest reservoir with a design date of 1968. As noted in Section 2 of this report, the AWWA Design Standard for Welded Steel Tanks D-100 has been updated on numerous occasions since the design of these reservoirs. The most significant change to the AWWA design standards is the inclusion of seismic provisions.

Seismic design provisions were first introduced to the AWWA design standards in 1979, and were considered voluntary. It is assumed that seismic design provisions were not included in the design of the Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, and Horizon View No. 1 reservoirs, since each of these reservoirs was constructed prior to 1979.

PHYSICAL CHARACTERISTICS

The physical characteristics of the Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, and Horizon View No. 1, and Somerset No. 1 reservoirs are presented in Table 3-2. All reservoirs are closed and are circular, with the exception of Somerset No. 1, which is rectangular in shape. The capacity of the reservoirs ranges from 0. 1 million gallons to 2.0 million gallons.

TABLE 3-1
City of Bellevue
General Description of Reservoirs

Reservoir	Location	Construction Type	Design Date	Engineer	Fabricator
Lake Hills North	16049 NE 8th Street	Welded Steel	1958	Harstad & Associates	Unknown
Lake Hills South	16049 NE 8th Street	Welded Steel	1962	Harstad & Associates	Unknown
Pikes Peak	124th NE and NE 40th	Welded Steel	1968	Roy L. Gardner & Associates	General American Transportation Corp.
Parksite	14501 Newport Way	Welded Steel	1962	Harstad & Associates	Unknown
Woodridge	125th SE and SE 20th Place	Welded Steel	1955	Harstad & Associates	Chicago Bridge & Iron
Horizon View No. 1	4825 148th SE	Welded Steel	1963	Harstad & Associates	Unknown
Somerset No. 1	4454 Somerset Boulevard	Reinforced Concrete	1961	Harstad & Associates	N/A

TABLE 3-2
City of Bellevue
Physical Characteristics of Reservoirs

Reservoir	Shape	Capacity (MG)	Dimensions (Feet)	Maximum Water Depth (Feet)
Lake Hills North	Cylindrical w/dome roof	2.0	Radius=34.00	75.00
Lake Hills South	Cylindrical w/dome roof	2.0	Radius=34.75	75.00
Pikes Peak	Cylindrical w/sloped roof (1:12)	1.0	Radius=42.50	24.00
Parksite	Cylindrical w/sloped roof (0.75:12)	2.0	Radius=46.50	40.50
Woodridge	Cylindrical w/dome roof	2.0	Radius=35.50	71.00
Horizon View No. 1	Cylindrical w/sloped roof	0.2	Radius=15.583	35.00
Somerset No. 1	Rectangular w/curved precast shell panels	0.1	80 X 16	10.32

As noted in Section 2, a reservoir will be more likely to experience an overturning moment the smaller the diameter and the taller the tank. Hence the greater the diameter to height ratio the less likely the tank will experience overturning. The ratio of the diameter to the height of each of the cylindrical tanks is summarized in Table 3-3, as is the existing anchorage system.

The diameter to height ratio is also an indicator for the seismic forces on a tank when using the pseudodynamic approach for earthquake design. In general, large seismic forces will be realized with a diameter to height ratio of approximately 2 or less. With the exception of the Parksite and Pikes Peak reservoirs, the impact of wave action in the reservoir during an earthquake will result in significant seismic forces on the tank.

TABLE 3-3
City of Bellevue
Diameter to Height Ratio and Existing Anchorage

Reservoir	Diameter/ Height Ratio	Existing Anchorage[(a),(b)]
Lake Hills North	0.91	12 equally spaced anchors. 3-inch wide by ½-inch thick plate overlain on 4-inch wide by ½-inch thick bar that is welded to tank. 3-inch plate is embedded 15.5 inches into concrete ringwall. Lower part of plate is bent 90' and has another plate welded to it opposite the bend to form an inverted 'T'.
Lake Hills South	0.91	12 equally spaced anchors. 3-inch wide by ½-inch thick plate overlain on 4-inch wide by ½-inch thick bar that is welded to tank. 3-inch plate is embedded 15.5 inches into concrete ringwall. Lower part of plate is bent 90' and has another plate welded to it opposite the bend to form an inverted 'T'.
Pikes Peak	3.54	Unanchored
Parksite	2.30	12 equally spaced anchors. 3-inch wide by ½-inch thick bar overlaps a 4-inch wide by ½-inch thick bar that is welded to the tank wall. The 3-inch bar is embedded in the ringwall.
Woodridge	1.00	12 equally spaced anchors. 3-inch wide by ½-inch thick plate overlain on 4-inch wide by ½-inch thick that is welded to tank. 3-inch plate is embedded approximately 2-feet into concrete ringwall. Lower part of plate is bent 900 and has another plate welded to it opposite the bend to form an inverted 'T'.
Horizon View No. 1	0.89	4 equally spaced 2-inch by 3/8-inch flat bar anchors. Tops of anchors are welded to tank wall. into the ringwall. Lower part of plate is bent 900 and has another plate welded to it opposite the bend to form an inverted "T'.

(a) Anchorage information is from Kennedy-Jenks Consultants, Inc. October 1993.
(b) Existing anchorage is deemed to be below AWWA D-100 Standards.

The data presented in Tables 3-1 through 3-3 were confirmed where possible during conversations with the City of Bellevue personnel, by reviewing available construction drawings and during the site investigations of each of the reservoirs. These data, along with that gathered during the site investigations, geotechnical investigations, and corrosion investigations, form the basis for the structural/seismic evaluation of the reservoirs.

Section 4

MONTGOMERY WATSON

Section 4
Lake Hills North

The Lake Hills North reservoir is a welded steel tank designed in 1958. It is located adjacent to and on the same parcel of property as the City's Lake Hills South reservoir. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition, alternative seismic retrofits, including a cost comparison, have been developed for the Lake Hills North reservoir. The evaluation of the alternatives and a recommended seismic retrofit alternative are also presented in this section. Photographs of the Lake Hills north reservoir are included in Appendix B.

SITE INVESTIGATION

Site investigations for the Lake Hills North reservoir have included on-site inspections when the reservoir was full and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and reports resulting from these investigations are included in Volume II of this preliminary design report.

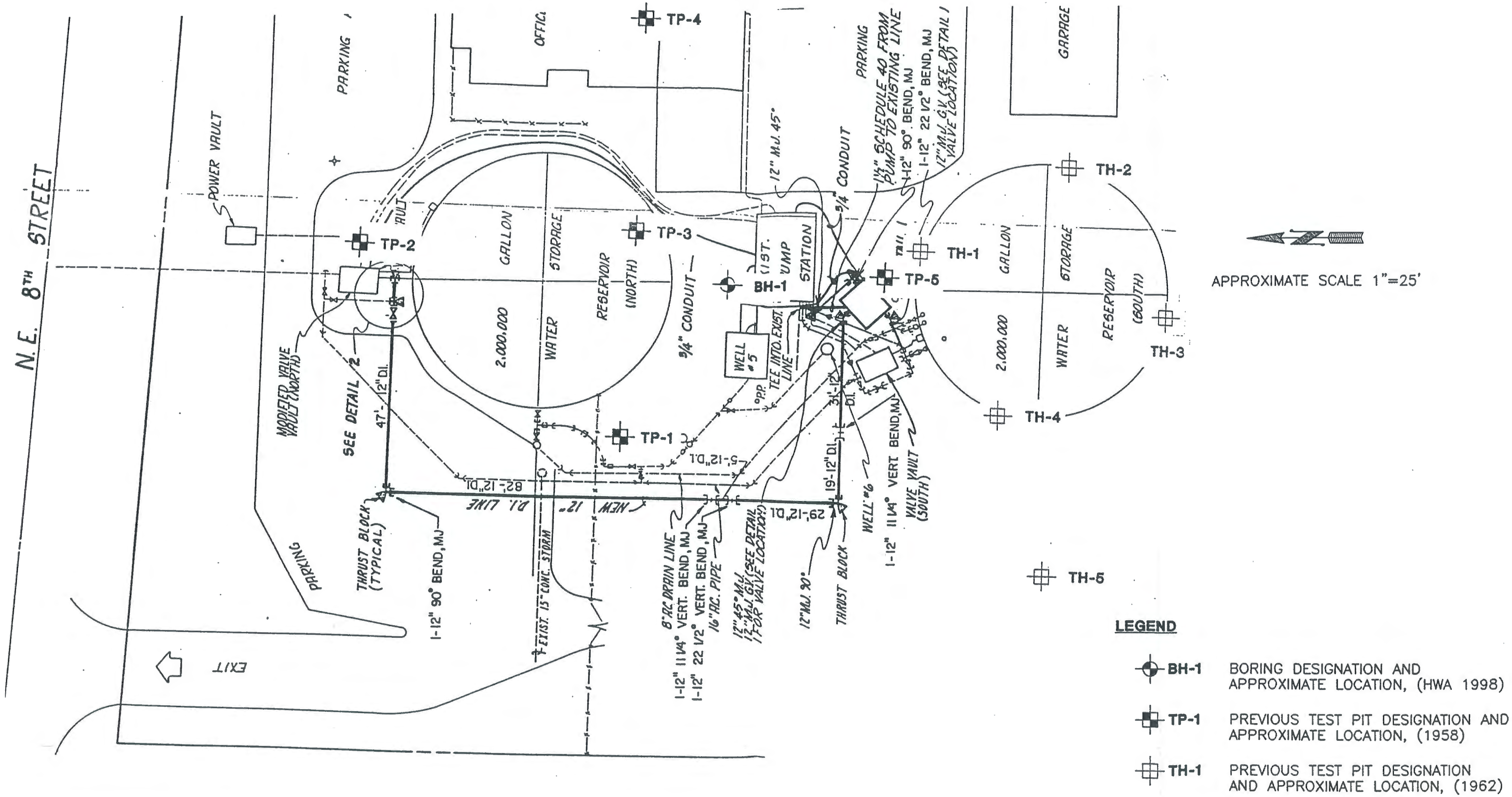
Site Characteristics

The Lake Hills North reservoir site plan is shown on Figure 4-1. Based on the data reviewed, and the subsurface conditions observed from the HWA GeoSciences exploratory boring, the Lake Hills North reservoir site appears to be underlain by glacial till. Glacial till is typically a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders that was deposited beneath an advancing glacier and subsequently overridden and compacted by glacial ice. TM is often relatively dense and is locally referred to as "hardpan". The boring shows that the unweathered glacial till. was overlain by 4 feet of either fill or weathered till. The upper soil unit consists of very dense, gravelly, silty sand and extends to a minimum depth of 40.5 feet.

Based on the 1958 plans for the Lake Hills North reservoir, the existing reservoir ring wall footings appear to be founded on dense glacial till. The ring wall is 3.5 feet wide by 4.5 feet deep. The interior of the ring wall foundation was backfilled with compacted pit run gravel, with the exception of the five inches immediately below the bottom of the tank, which consists of oiled sand.

General Reservoir Condition

General visual inspection of the Lake Hills North reservoir indicates that interior and exterior ladders, safety cages, and catwalks are in good condition with little apparent deterioration or corrosion. The interior roof structure also appears to be in good condition. The interior piping, floor drain, overflow and support, and the outlet/inlet piping, all appear to be in good condition with no apparent corrosion or deterioration. The access manway also appears to be in good condition.



REFERENCE: As-built plans provided by the City of Bellevue titled "NE 8th Street Pump Station System Modifications". Prepared by Horton Dennis, dated March 1982.

MONTGOMERY WATSON

City of Bellevue
Lake Hills North/South Reservoirs
Site Plan and Subsurface Exploration Plan
FIGURE 4-1

At the time of the inspection, floor and shell plate thickness were acquired for use in the seismic analysis structural calculations. The data was primarily collected for characterization of existing steel thickness but also suggests that no significant general corrosion has occurred in the shell over the service life of the tank. The tank plate thickness results are presented in Table 4-1. It should be noted that, although the Lake Hills North and Lake Hills South reservoirs appear identical from the exterior, their structure (including the roofs) is different. This accounts for the difference in shell plate and roof plate thicknesses.

TABLE 4-1
Lake Hills North Reservoir
Tank Shell Thickness

Shell Course	Thickness, in.
Floor	0.490
1	1.065
2	0.930
3	0.830
4	0.710
5	0.620
6	0.500
7	0.385
8	0.275
Dome Base	0.260
Roof 1	0.255
Roof 2	0.260
Roof 3	0.260

Some moss build-up was noted on the exterior two lower courses of the tank. As would be expected, the build-up was more severe in those areas that remain in the shade. Grout between the exterior floor plate and ring wall foundation is in fair condition, whereas the exposed concrete on top of the ring wall foundation is weathered. The exterior roof plate above the roof hatch is badly corroded and in need of repair. It appears that rainwater has been ponding in this area contributing significantly to the deterioration of this area.

There is an existing underdrain system below the tank. No problems have been identified with the drainage system. Further, no groundwater was encountered during the subsurface exploration, and no standing water was apparent on the site.

Weld Condition

Based on the visual inspection of the Lake Hills North reservoir, the interior floor plate and shell plate welds are in good condition. The floor plate is lapped and fillet welded. Exterior shell plate welds also appear to be in good condition. The roof plate is lapped and fillet welded, and the welds appear to be in good condition.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Lake Hills North reservoir. Data collected included visual inspection of interior and exterior coatings, steel substrate, appurtenances and cathodic protection system components. Physical data collected on the tank included generic classification of coating, Toxicity Characteristic Leaching Procedure lead content, coating thickness and coating adhesion. The findings of the corrosion investigation are summarized in the following paragraphs.

Interior and Exterior Coatings

Visual inspection coupled with physical measurements formed the basis of the interior and exterior coatings investigation of the Lake Hills North reservoir. Visual inspection included qualitative evaluation of overall coating condition (e.g., blisters, underfilm corrosion, pinhole failure, cathodic disbondment), characterization of any floor, shell or appurtenance corrosion failure and close inspection of weld lanes and sharp edges. Physical measurements included interior and exterior coating thickness measurements on the wall and floor, and generic coating typing for both the interior and exterior coating systems. In addition, the exterior reservoir coating was tested for coating adhesion quality.

The coating thickness data collected for the Lake Hills North reservoir are typical for internal tank linings. In general, the internal visual inspection showed the tank coating was in good condition. Small trace blisters and pinhole coating failures occur throughout the surface of the tank floor and shell. While spread throughout the tank, the overall percent area of coating failed remains very small. These types of failures could indicate that the existing cathodic protection should be adjusted.

In 1991, pit depth and coating thickness measurements were taken at the Lake Hills North reservoir. The differences between the same data collected in 1998 and that collected in 1991 are within the tolerance of the measurement technique and suggest no corrosion has occurred during the service interval between inspections. The interior coating, in conjunction with an operative cathodic protection system, is sufficient to protect the interior surfaces from corrosion. The coatings on the tank appurtenances, such as the access ladder and other structural steel remain in good condition with no signs of accelerated failure.

The coating adhesion tests conducted on the exterior coating system for the Lake Hills North reservoir showed failure at the primer/topcoat interface and failure between the primer and substrate. The significance of these test results would require extensive testing beyond the scope of this work to remove the inherent statistical variation. Therefore, the coating adhesion tests are considered inconclusive. However, the exterior inspection revealed no visible -signs of coating failure.

Fourier Transform Infrared analysis suggest that the generic type of both the interior and exterior coatings is vinyl based. The data on the coating samples also show that the interior sample was well below the 5.0 ppm maximum concentration for leachable lead. However, the exterior sample was well above the limit. However, based on the findings for the exterior coating system it is recommended that additional testing confirming the leachable lead content need not be performed prior to any maintenance coating operations. In addition, the existing coating may be encapsulated with minor surface preparation that includes removal of areas of delaminated coating.

Cathodic Protection Condition

The condition of cathodic protection was evaluated simultaneously with the interior and exterior coatings evaluation. Visual inspection of the Lake Hills North reservoir shows that a cathodic protection system exists at the reservoir site, and that the components of the system (e.g. anodes, reference electrode) remain in good condition. The cathodic protection system, if properly adjusted and maintained, should prevent significant corrosion in the areas where trace blisters and pinhole coating failures have occurred.

It is recommended that the output of the cathodic protection system be adjusted to maintain protected tank-to-electrolyte potential in accordance with the National Association of Corrosion Engineers (NACE) criteria. In addition, it is recommended that the City monitors the rectifier output (i.e., voltage and current) quarterly, and that the City performs checkout of the cathodic protection system annually.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Lake Hills North reservoir consist of the following:

- 8-inch overflow piping rigidly attached to the tank base, tank wall, and knuckle. The center of the overflow line is 1.75 feet from the tank wall.
- 8-inch drain piping that penetrates the tank base 1.75 feet from the tank wall.
- 16-inch inlet piping that penetrates the tank base 5 feet from the tank wall.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Lake Hills North reservoir site were determined as described in Section 2. Table 4-2 summarizes the seismic coefficients to be used for the evaluations consistent with Seismic Zones 3 and 4.

Table 4-2
Lake Hills North Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficient C_a	Seismic Coefficient C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.2	1.5	0.48	0.84	0.14	0.70

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing ring wall foundations be evaluated using an allowable soil bearing pressure of 8 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces. Hoop tensile forces in the ring wall foundations resulting from the fluid weight may be evaluated using an earth pressure coefficient of 0.4, assuming the ring walls are relatively rigid.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated that the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 800 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Lake Hills North reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

Seismic Forces

Table 4-3 summarizes the seismic forces derived from analyzing the reservoir by the pseudodynamic method per AWWA D-100 for a seismic event with accelerations of 0.4g and 0.3g.

Table 4-3
Lake Hills North Reservoir
Seismic Forces

Seismic Forces	0.4g	0.3g
Overturning Moment (k-ft)	153,300	114,975
Soil Bearing Pressure (ksf)	14.1	10.9
Static Hoop Stress (psi)	14,973	14,973
Seismic Hoop Stress (psi)	14,767	14,257
Compression Stress (psi)	3,340	2,549
Anchor Force (k)	694	510

The analysis shows that the shell plating is adequate for both seismic overturning compressive stress and tensile hoop stresses. This assumes the allowable stresses to be used for the analysis are from Chapter 3 of AWWA D-100, which assumes that high strength steels have not been used in the construction of the Lake Hills North Reservoir. The allowable static hoop stress is

15,000 psi and the allowable seismic hoop stress is 17,000 psi. The allowable compressive stress is 4,229 psi.

As shown in Table 3-3 of this report, the reservoir is anchored to the ringwall footing by 12 equally spaced anchors. The capacity of each of these anchors, assuming a material strength of 30 ksi, is 36k. The forces in the anchors shown in Table 4-3 due to overturning in a 0.4g and 0.3g seismic event are 694k and 510k, respectively. Therefore, the existing anchors are inadequate to resist overturning. Additional anchorage should be provided between the reservoir and the ring footing to prevent overturning during an earthquake.

The allowable soil bearing pressure based on the results of the soil investigation for transient loading conditions such as seismic forces is 12 ksf. The actual bearing pressure for a 0.4g seismic event is 14.1 ksf and for a 0.3g seismic event is 10.9 ksf.

In accordance with AWWA D-100, resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, the portion of the contents that may be used to resist overturning is dependent on the width of the bottom annulus. The annulus may be thought of as a portion of the bottom plate, which lifts off the foundation and supports the weight of the tank contents.

There are three criteria to determine the degree of overturning and consequently the amount of anchorage required for the reservoir. If the ratio of the overturning moment divided by the product of the square of the diameter of the reservoir and the weight of the tank shell, roof and contents is less than or equal to 0.785, then there is no uplift and no anchorage is required. If this ratio is between 0.785 and 1.54, then there is uplift but no anchorage is required. However, since there is uplift, anything that is within the annular ring or outside the reservoir and connected to the reservoir must be allowed to move, either with the reservoir or relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the inside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than 1.54 the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Lake Hills North reservoir, this ratio is 3.92 and 2.94 respectively, for a 0.4g and 0.3g earthquake. Because these ratios are so high, the annular ring cannot be sufficiently thickened to resist overturning and the tank must be anchored to the ring footing.

ALTERNATIVE SEISMIC RETROFITS

As noted in the "Seismic Forces" subsection, the Lake Hills North Reservoir must be anchored to the existing ring footing to resist overturning for a 0.4g and a 0.3g earthquake. In addition, the existing ring footing must be able to resist overturning. Three alternatives have been considered to seismically retrofit the reservoir to resist overturning.

Alternative 1 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. Adding more weight to the footing would then increase the resistance of the ring footing to overturning. This would be accomplished by adding a new ring footing on the outside of the existing footing and doweling them together as shown in Figure 4-2.

Alternative 2 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. A portion of the bottom plate on the inside of the reservoir would be removed and a new ring footing would be added. The new ring footing would then be doweled into the existing footing as shown in Figure 4-3. The combination of the two footings would serve to support a portion of the tank contents sufficient to resist overturning.

Alternative 3 is to use small diameter high strength earth anchors to resist overturning. The earth anchors would be drilled through the existing ring footing at equal spaces on the outside of the reservoir as shown in Figure 4-4. The anchors would then be connected to the shell plate by use of anchor chairs.

Also noted in the "Seismic Forces" subsection, the size of the ring footing must be increased for the Lake Hills North Reservoir for a 0.4g earthquake. In Alternatives 1 and 2, increasing the width of the footing to resist overturning would be adequate to decrease the soil bearing pressure below the allowable. Finally, in accordance with AWWA D-100 Section 13.6, foundations under flat-bottom tanks have fared well under seismic loading and the seismic loading does not provide justification for increased foundations. Therefore, in Alternative 3, the size of the ring footing does not need to be increased.

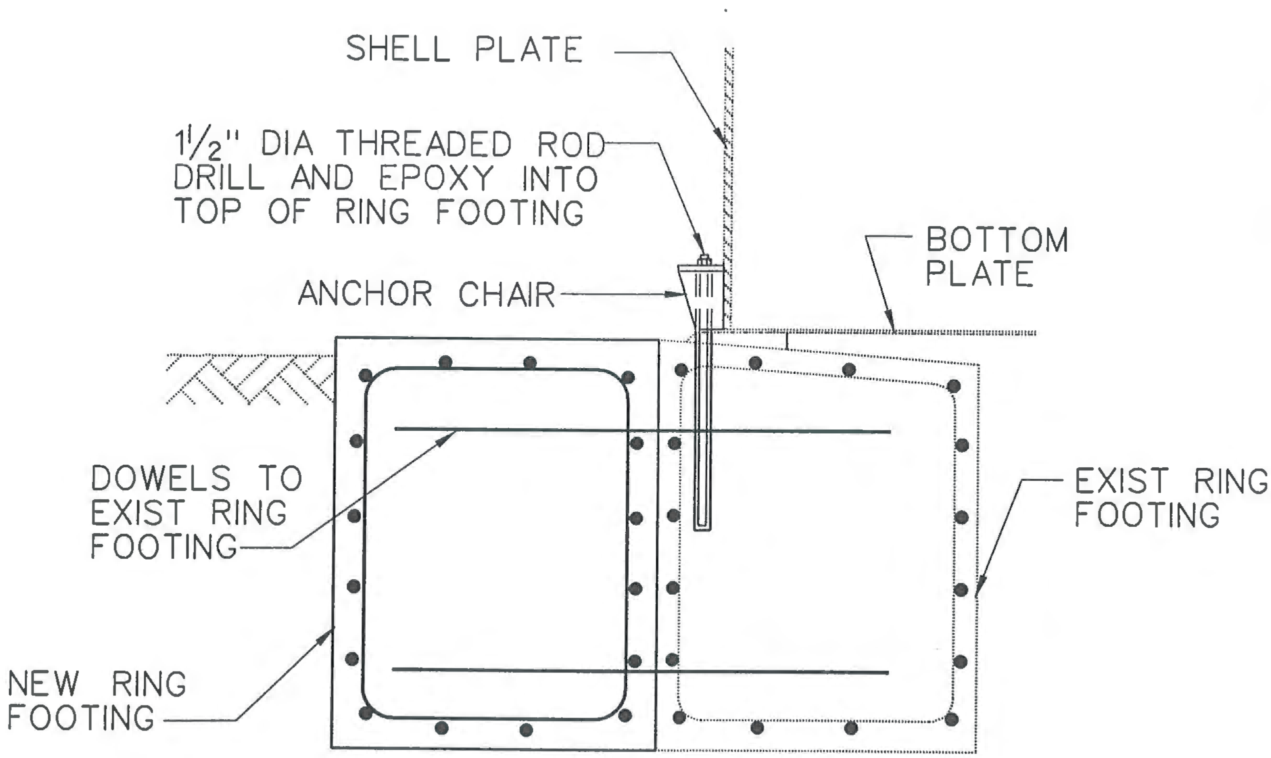
COMPARISON OF ALTERNATIVES

For the Lake Hills North Reservoir, Alternative 1 does not appear to be an adequate solution for a number of reasons. First, since the weight of the footing which resists overturning is relatively small compared to the weight of the reservoir plus its contents which cause overturning, simply adding weight to the footing is an inefficient method for increasing the overturning resistance. The width that would have to be added to the footing is greater than the width of the existing footing, making this alternative economically impractical.

Second, the existing footing is not adequately reinforced to support the weight of additional concrete from the new footing during uplift. Finally, the adhesive anchors connecting the tank shell to the footing are relatively low capacity anchors and the number required would cause them to be so close together that they would be essentially ineffective.

Alternative 2 does not appear to be an effective solution for two reasons. The first is that the required number and spacing of adhesive anchors would cause them to be essentially ineffective, similar to Alternative 1. The second reason is that the existing footing is not adequately reinforced to support the additional weight of the tank contents, required to resist overturning and the additional weight of the footing concrete.

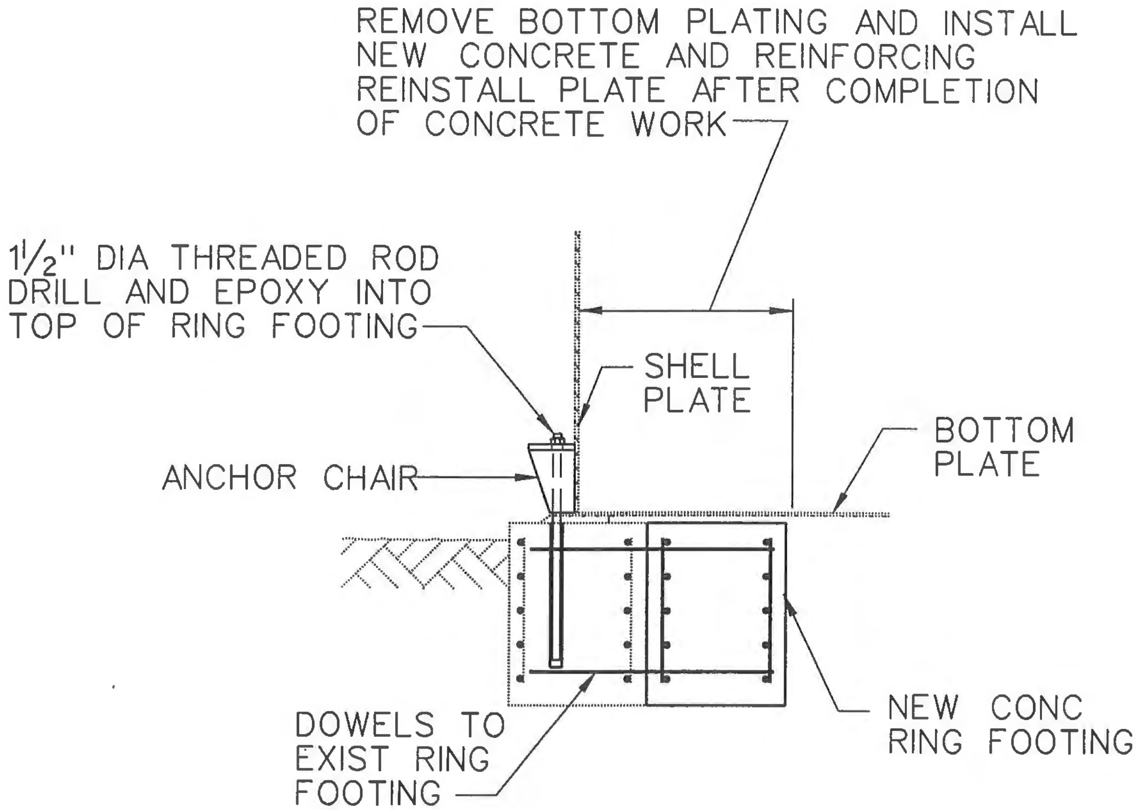
Alternative 3 appears to be an adequate seismic retrofit solution. Alternative 3 will meet the seismic requirements for both a Seismic Zone 3 and 4 earthquake. There are two different overturning cases that were considered for this alternative. In the first case, the earth anchors would be designed to resist the entire overturning moment. In the second case, the earth anchors



SECTION THROUGH RING FOOTING

½" = 1'-0"

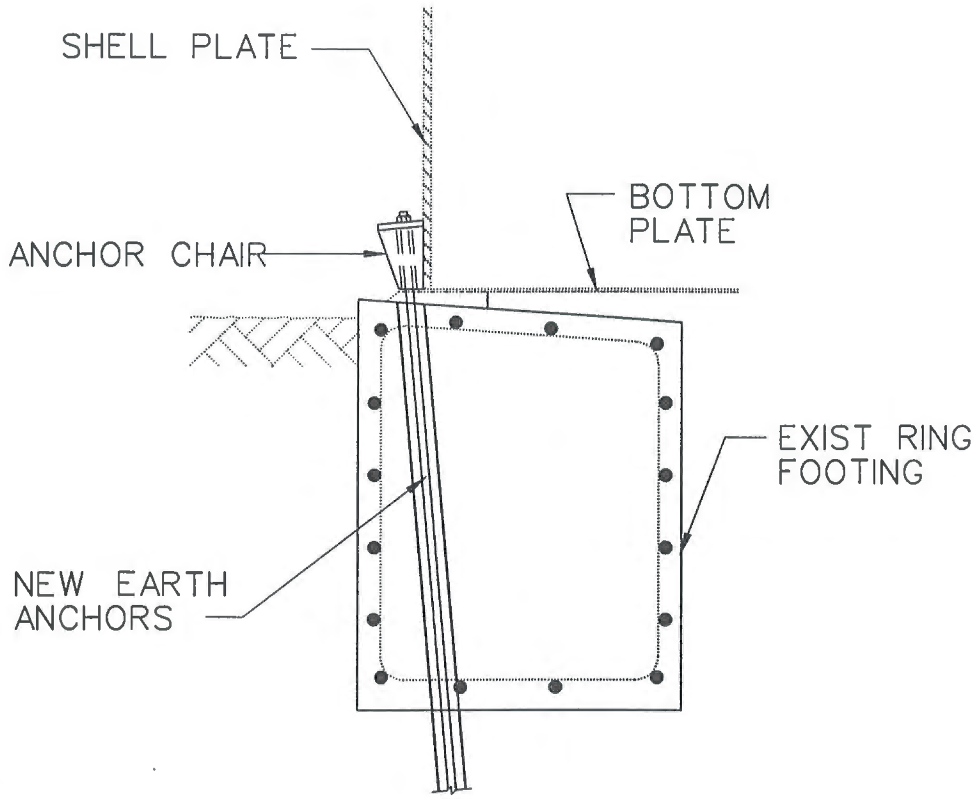
MONTGOMERY WATSON Bellevue, Washington	CITY OF BELLEVUE RESERVOIR ANCHORING ALTERNATIVE 1	FIGURE 4-2



SECTION THROUGH RING FOOTING

½" = 1'-0"

MONTGOMERY WATSON Bellevue, Washington	CITY OF BELLEVUE RESERVOIR ANCHORING ALTERNATIVE 2	FIGURE 4-3



SECTION THROUGH RING FOOTING

1/2" = 1'-0"

MONTGOMERY WATSON Bellevue, Washington	CITY OF BELLEVUE RESERVOIR ANCHORING ALTERNATIVE 3	FIGURE 4-4

would be designed to resist an overturning moment that is slightly reduced. The overturning moment would be reduced by an amount that is equal to the square of the diameter of the reservoir multiplied by the sum of the weights of the reservoir shell plate and the roof structure multiplied by the ratio 1.54. This allows the roof structure and shell plate to resist a portion of the overturning moment up to a point where the reservoir would require anchorage. In this case piping connections would require modifications so that they allow movement.

Alternative 3, case one, requires 40-foot long earth anchors and anchor chairs at about a 4-foot spacing for a 0.4g earthquake and a 40-foot long earth anchors and anchor chairs at about a 5.5-foot spacing for a 0.3g earthquake. Alternative 3, case two, requires the same earth anchors at about a 4.5-foot spacing for a 0.4g earthquake and at about a 6.5-foot spacing for a 0.3g earthquake. In addition, for Alternative 3, case two, flexible couplings would be added to pipe connections for the 8 inch overflow pipe and the 8 inch drain pipe. However, it should be noted that because these pipes are embedded in the foundation and welded to the bottom plate of the reservoir, adding sufficient flexibility to the connections would require a significant amount of additional work and cost. Finally, it should be noted that for the purposes of the computations in this preliminary design report, the spacing of soil anchors has been assumed to be uniform. Respacing of anchors to avoid below-grade obstructions and above grade structures may be required. However, this should not significantly impact the overall seismic capacity of the anchor system or the cost of construction.

Estimated costs for both cases of Alternative 3 are summarized in Table 4-6 below for both a 0.4 g and a 0.3g seismic event. For a description of what is included in estimated costs, see Table 11-1.

Table 4-4
Lake Hills North Reservoir
Summary of Estimated Costs

	0.4g	0.3g
Alternative 3 – Case One	$470,000	$360,000
Alternative 3 – Case Two	$455,000	$345,000

Note: The difference in cost between the Lake Hills North and Lake Hills South reservoirs is due primarily to the fact that the roof plate adjacent to the roof hatch is badly corroded and is in need of repair (see Section 11).

would be designed to resist an overturning moment that is slightly reduced. The overturning moment would be reduced by an amount that is equal to the square of the diameter of the reservoir multiplied by the sum of the weights of the reservoir shell plate and the roof structure multiplied by the ratio 1.54. This allows the roof structure and shell plate to resist a portion of the overturning moment up to a point where the reservoir would require anchorage. In this case piping connections would require modifications so that they allow movement.

Alternative 3, case one, requires 40-foot long earth anchors and anchor chairs at about a 4-foot spacing for a 0.4g earthquake and a 40-foot long earth anchors and anchor chairs at about a 5.5-foot spacing for a 0.3g earthquake. Alternative 3, case two, requires the same earth anchors at about a 4.5-foot spacing for a 0.4g earthquake and at about a 6.5-foot spacing for a 0.3g earthquake. In addition, for Alternative 3, case two, flexible couplings would be added to pipe connections for the 8 inch overflow pipe and the 8 inch drain pipe. However, it should be noted that, because these pipes are embedded in the foundation and welded to the bottom plate of the reservoir, adding sufficient flexibility to the connections would require a significant amount of additional work and cost. Finally, it should be noted that for the purposes of the computations in this preliminary design report, the spacing of soil anchors has been assumed to be uniform. Respacing of anchors to avoid below grade obstructions and above grade structures may be required. However, this should not significantly impact the overall seismic capacity of the anchor system or the cost of construction.

Estimated costs for both cases of Alternative 3 are summarized in Table 4-6 below for both a 0.4 g and a 0.3g seismic event. For a description of what is included in estimated costs, see Table 11-1.

Table 4-4
Lake Hills North Reservoir
Summary of Estimated Costs

	0.4G	0.3G
Alternative 3 – Case One	$370,000	$293,000
Alternative 3 – Case Two	$365,000	$288,000

Note: The difference in cost between the Lake Hills North and Lake Hills South reservoirs is due primarily to the fact that the roof plate adjacent to the roof hatch at Lake Hills South is badly corroded and is in need of repair (see Section 11).

Section 5

MONTGOMERY WATSON

Section 5
Lake Hills South

The Lake Hills South reservoir is a welded steel tank designed in 1962 by Harstad & Associates. It is located adjacent to and on the same parcel of property as the City's Lake Hills North reservoir. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition, alternative seismic retrofits, including a cost comparison, have been developed for the Lake Hills South reservoir. The evaluation of the alternatives and a recommended seismic retrofit alternative are also presented in this section. Photographs of the Lake Hills South reservoir are included in Appendix B.

SITE INVESTIGATION

Site investigations for the Lake Hills South reservoir have included on-site inspections when the reservoir was full and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and reports resulting from these investigations are included in Volume II of this preliminary design report.

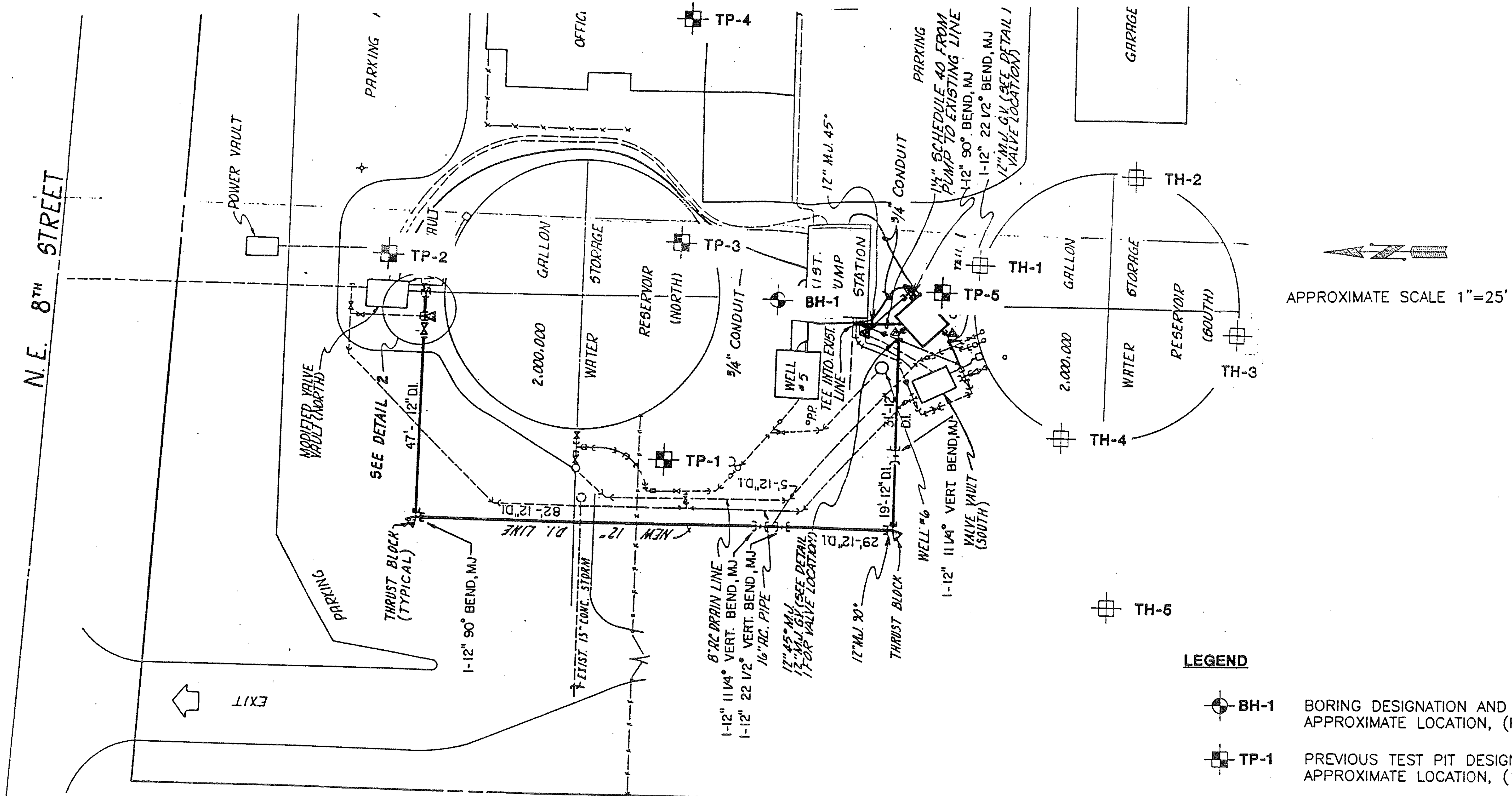
Site Characteristics

The Lake Hills South reservoir site plan is shown on Figure 5-1. The Lake Hills South reservoir site characteristics are the same as those of the Lake Hills North reservoir due to their proximity to one another. Based on the data reviewed and the subsurface conditions observed from the HWA GeoSciences exploratory boring, the Lake Hills South reservoir site appears to be underlain by glacial till. Glacial till is typically a heterogeneous mixture of clay, silt, sand, gravel, cobbles and boulders that was deposited beneath an advancing glacier and subsequently overridden and compacted by the glacial ice. . Till is often relatively dense and is locally referred to as "hardpan"'. The boring shows that the unweathered glacial till. was overlain by 4 feet of either fill or weathered till. The upper soil unit consists of very dense, gravelly, silty sand and extends to a minimum depth of 40.5 feet.

Based on the 1962 plans for the Lake Hills South reservoir, the existing reservoir ring wall footings appear to be founded on dense glacial till. The ring wall is 3.5-feet wide by 4.5-feet deep. The interior of the ring wall foundation was backfilled with compacted pit run gravel, with the exception of the five inches immediately below the bottom of the tank, which consists of oiled sand.

General Reservoir Condition

General visual inspection of the Lake Hills South reservoir indicates that interior and exterior ladders, safety cages, and catwalks are in good condition with little apparent deterioration or corrosion. The interior roofing structure also appears to be in good condition. The interior piping, floor drain, overflow and support, and the outlet/inlet piping, all appear to be in good condition



REFERENCE: As-built plans provided by the City of Bellevue titled "NE 8th Street Pump Station System Modifications". Prepared by Horton Dennis, dated March 1982.

LEGEND

BH-1	BORING DESIGNATION AND APPROXIMATE LOCATION, (HWA 1998)
TP-1	PREVIOUS TEST PIT DESIGNATION AND APPROXIMATE LOCATION, (1958)
TH-1	PREVIOUS TEST PIT DESIGNATION AND APPROXIMATE LOCATION, (1962)

MONTGOMERY WATSON

City of Bellevue
Lake Hills North/South Reservoirs
Site Plan and Subsurface Exploration Plan
FIGURE 5-1

with no apparent corrosion or deterioration. The access manway also appears to be in good condition.

At the time of the inspection, floor and shell plate thickness were acquired for use in the seismic analysis structural calculations. The data was primarily collected for characterization of existing steel thickness but also suggests that no significant general corrosion has occurred in the shell over the service life of the tank. The tank plate thickness results are presented in Table 5-1. It should be noted that, although the Lake Hills North and Lake Hills South reservoirs appear identical from the exterior, their structure (including the roofs) is different. This accounts for the difference in shell plate and roof plate thicknesses. There is some damage to the first course of shell plate on the east side that appears to be from truck traffic. This damage should be repaired and bollards placed adjacent to the repaired areas to prevent further damage.

TABLE 5-1
Lake Hills South Reservoir
Tank Shell Thickness

Shell Course	Thickness, in.
Floor	0.490
1	1.115
2	1.010
3	0.900
4	0.805
5	0.690
6	0.590
7	0.500
8	0.395
9	0.290
Dome Base	0.265
Roof 1	0.255

Grout between the exterior floor plate and ring wall foundation is in fair condition, whereas the exposed concrete on top of the ring wall foundation is weathered. There is no apparent corrosion at the exterior roof hatch.

There is an existing underdrain system below the tank. No problems have been identified with the drainage system. Further, no groundwater was encountered during the subsurface exploration, and standing water was not apparent on the site.

Weld Condition

Based on the visual inspection of the Lake Hills South reservoir, the interior floor plate and shell plate welds are in good condition. The floor plate is lapped and fillet welded. Exterior shell plate welds also appear to be in good condition. The roof plate is lapped and fillet welded, and the welds appear to be in good condition.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Lake Hills South reservoir. Data collected included visual inspection of interior and exterior coatings, steel substrate, appurtenances and cathodic protection system components. Physical data collected on the tank included generic classification of coating, Toxicity Characteristic Leaching Procedure lead content, coating thickness and coating adhesion. The findings of the corrosion investigation are summarized in the following paragraphs.

Interior and Exterior Coatings

Visual inspection coupled with physical measurements formed the basis of the interior and exterior coatings investigation of the Lake Hills South reservoir. Visual inspection included qualitative evaluation of overall coating condition (e.g., blisters, underfilm corrosion, pinhole failure, cathodic disbondment), characterization of any floor, shell or appurtenance corrosion failure and close inspection of weld lanes and sharp edges. Physical measurements included interior and exterior coating thickness measurements on the wall and floor, and generic coating typing for both the interior and exterior coating systems. In addition, the exterior reservoir coating was tested for coating adhesion quality.

The coating thickness data collected for the Lake Hills South reservoir are typical for internal tank linings. In general, the internal visual inspection showed that the tank coating was in good condition. Small trace blisters were noted on the interior shell approximately 6 feet from the floor. No corrosion was noted on the substrate beneath the blisters suggesting that the cathodic protection system is mitigating corrosion, though the presence of these blisters indicates that the cathodic protection system may require adjustment

Coating thickness on the floor was somewhat lower and varied by about 8 mils. Variation of this magnitude across the floor is not desirable. Blistering has also occurred on the tank floor, primarily along weld seams. Stripe coating with a brush prior to spraying a primer coat can provide extra protection against corrosion along these seams.

The interior coating, in conjunction with an operative cathodic protection system, is sufficient to protect the interior surfaces from corrosion. The coatings on the tank appurtenances, such as the access ladder and other structural steel remain in good condition with no signs of accelerated failure.

The exterior inspection revealed no visible signs of coating failure, with the bottom four shell courses showing signs of maintenance coating. The coating adhesion tests conducted on the exterior coating system for the Lake Hills South reservoir showed failure at the primer/topcoat interface and failure between the primer and substrate. The significance of these test results would require extensive testing beyond the scope of this work to remove the inherent statistical variation. Therefore, the coating adhesion tests are considered inconclusive.

Fourier Transform Infrared analysis suggests that the generic type of the interior coating is in the epoxy category, and that the exterior coating is vinyl based. The data on the coating samples also show that the interior samples were well below the 5.0 ppm. maximum concentration for leachable lead and the exterior sample was well above the EPA limit at 12 ppm. However, based on the findings for the exterior coating system it is recommended that additional testing confirming the leachable lead content need not be performed prior to any maintenance coating operations and that the existing coating may be encapsulated with minor surface preparation which includes removal of areas of delaminated coating.

Cathodic Protection Condition

The condition of cathodic protection was evaluated simultaneously with the interior and exterior coatings evaluation. Visual inspection of the Lake Hills South reservoir shows that a cathodic protection system exists at the reservoir site, and that the components of the system (e.g. anodes, reference electrode) remain in good condition. The cathodic protection system, if properly adjusted and maintained, should prevent significant corrosion in the areas where trace blisters have occurred.

It is recommended that the output of the cathodic protection system be adjusted to maintain protected tank-to-electrolyte potential in accordance with National Association of Corrosion Engineers criteria. In addition, it is recommended that the City monitors the rectifier output (i.e., voltage and current) quarterly, and that the City performs checkout of the cathodic protection system annually.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Lake Hills South reservoir consist of the following:

- 8-inch overflow piping rigidly attached to the tank base, tank wall, and knuckle. The center of the overflow line is 1 foot from the tank wall.
- 8-inch drain piping that penetrates the tank base 1 foot from the tank wall.
- 16-inch inlet piping that penetrates the tank base 4.25 feet from the tank wall.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Lake Hills South reservoir site were determined as described in Section 2. Table 5-2 summarizes the seismic coefficients to be used for the evaluations consistent with Seismic Zones 3 and 4.

Table 5-2
Lake Hills South Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficient C_a	Seismic Coefficient C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.2	1.5	0.48	0.84	0.14	0.70

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing ring wall foundations be evaluated using an allowable soil bearing pressure of 8 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces. Hoop tensile forces in the ring wall foundations resulting from the fluid weight may be evaluated using an earth pressure coefficient of 0.4, assuming the ring walls are relatively rigid.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated that the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 800 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Lake Hills South reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

Seismic Forces

Table 5-3 summarizes the seismic forces derived from analyzing the reservoir by the pseudodynamic method per AWWA D-100 for a seismic event with accelerations of 0.4g and 0.3g.

Table 5-3
Lake Hills South Reservoir
Seismic Forces

Seismic Forces	0.4g	0.3g
Overturning Moment (k-ft)	148,634	111,475
Soil Bearing Pressure (ksf)	14.3	11.1
Static Hoop Stress (psi)	13,991	13,991
Seismic Hoop Stress (psi)	13,438	12,989
Compression Stress (psi)	3,237	2,473
Anchor Force (k)	686	504

The analysis shows that the shell plating is adequate for both seismic overturning compressive stress and tensile hoop stresses. This assumes that the allowable stresses to be used for the analysis are from Chapter 3 of AWWA D-100 that assumes that high strength steels have not been used in the construction of the Lake Hill South Reservoir. The allowable static hoop stress is 15,000 psi and the allowable seismic hoop stress is 17,000 psi. The allowable compressive stress is 4,650 psi.

As shown in Table 3-3 of this report, the reservoir is anchored to the ringwall footing by 12 equally spaced anchors. The capacity of each of these anchors, assuming a material strength of 30 ksi, is 36k. The forces in the anchors shown in Table 5-3 due to overturning in a 0.4g and 0.3g seismic event are 686 and 504k, respectively. Therefore, the existing anchors are inadequate to resist overturning and additional anchorage should be provided between the reservoir and the ring footing.

The allowable soil bearing pressure based on the results of the soil investigation for transient loading conditions such as seismic forces is 12 ksf. The actual bearing pressure for a 0.4g seismic event is 14.3 ksf and for a 0.3g seismic event is 11.1 ksf.

In accordance with AWWA D-100, resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, the portion of the contents that may be used to resist overturning is dependent on the width of the bottom annulus. The annulus may be thought of as a portion of the bottom plate that lifts off the foundation and supports the weight of the tank contents.

There are three criteria to determine the degree of overturning and consequently the amount of anchorage required for the reservoir. If the ratio of the overturning moment divided by the product of the square of the diameter of the reservoir and the weight of the tank shell, roof and contents is less than or equal to 0.785, then there is no uplift and no anchorage is required. If this ratio is between 0.785 and 1.54, then there is uplift but no anchorage is required. However, since there is uplift, anything that is within the annular ring or outside the reservoir and connected to the reservoir must be allowed to move, either with the reservoir or relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the inside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than 1.54 the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Lake Hills South reservoir, this ratio is 3.92 and 2.94 respectively for a 0.4g and 0.3g earthquake. Because these ratios are so high the annular ring cannot be sufficiently thickened to resist overturning and the tank must be anchored to the ring footing.

ALTERNATIVE SEISMIC RETROFITS

As noted in the "Seismic Forces" subsection, the Lake Hills South Reservoir must be anchored to the existing ring footing to resist overturning for a 0.4g and a 0.3g earthquake. In addition, the existing ring footing must be able to resist overturning. Three alternatives have been considered to seismically retrofit the reservoir to resist the overturning.

Alternative 1 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. Adding more weight to the footing would then increase the resistance of the ring footing to overturning. This would be accomplished by adding a new ring footing on the outside of the existing footing and doweling them together as previously shown in Figure 4-2.

Alternative 2 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. A portion of the bottom plate on the inside of the reservoir would be removed and a new ring footing would be added. The new ring footing would then be doweled into the existing footing as shown in Figure 4-3. The combination of the two footings would serve to support a portion of the tank contents sufficient to resist the overturning.

Alternative 3 is to use small diameter high strength earth anchors to resist overturning. The earth anchors would be drilled through the existing ring footing at equal spaces on the outside of the reservoir as previously shown in Figure 4-4. The anchors would then be connected to the shell plate by the use of anchor chairs.

Also noted in the "Seismic Forces" subsection, the size of the ring footing must be increased for the Lake Hills South Reservoir for a 0.4g earthquake. In Alternatives 1 and 2, increasing the width of the footing to resist overturning would be adequate to decrease the soil bearing pressure below the allowable. Finally, in accordance with AWWA D-100 Section 13.6, foundations under flat-bottom tanks have fared well under seismic loading and the seismic loading does not provide justification for increased foundations. Therefore, in Alternative 3 the size of the ring footing does not need to be increased.

COMPARISON OF ALTERNATIVES

For the Lake Hills South Reservoir, Alternative 1 does not appear to be an adequate solution for a number of reasons. First, since the weight of the footing which resists overturning is relatively small compared to the weight of the reservoir plus its contents which cause overturning, simply adding weight to the footing is an inefficient method for increasing the overturning resistance. The width that would have to be added to the footing is greater than the width of the existing footing, making this alternative economically impractical.

Second, the existing footing is not adequately reinforced to support the weight of additional concrete from the new footing during uplift. Finally, the adhesive anchors connecting the tank shell to the footing are relatively low capacity anchors and the number required would cause them to be so close together that they would be essentially ineffective.

Alternative 2 does not appear to be an effective solution for two reasons. The first is that the required number and spacing of adhesive anchors would cause them to be essentially ineffective, similar to Alternative 1. The second reason is that the existing footing is not adequately reinforced to support the additional weight of the tank contents, required to resist overturning and the additional weight of the footing concrete.

Alternative 3 appears to be an adequate seismic retrofit solution. Alternative 3 will meet the seismic retrofit requirement for both a Seismic Zone 3 and 4 earthquake. There are two different overturning cases that were considered for this alternative. In the first case, the earth anchors would be designed to resist the entire overturning moment. In the second case, the earth anchors would be designed to resist an overturning moment that is slightly reduced. The overturning moment would be reduced by an amount that is equal to the square of the diameter of the reservoir multiplied by the sum of the weights of the reservoir shell plate and the roof structure multiplied by the ratio 1.54. This allows the roof structure and shell plate to resist a portion of the overturning moment up to a point where the reservoir would require anchorage. In this case piping connections would require modifications so that they allow movement.

Alternative 3, case one, requires 40-foot long earth anchors and anchor chairs at about a 4-foot spacing for a 0.4g earthquake and a 40-foot long earth anchor and anchor chairs at about a 5.5-foot spacing for a 0.3g earthquake. Alternative 3, case two, requires the same earth anchors at about a 4.5-foot spacing for a 0.4g earthquake and at about a 6.5-foot spacing for a 0.3g earthquake. In addition, for Alternative 3, case two, flexible couplings would be added to the pipe connections for the 8 inch overflow pipe and the 8 inch drain pipe. However, it should be noted that, because these pipes are embedded in the foundation and welded to the bottom plate of the reservoir, adding sufficient flexibility to their connections would require a significant amount of additional work and cost.

Estimated costs for both conditions of Alternative 3 are summarized in Table 5-4 for both a 0.4 g and a 0.3g seismic event. For a description of what is included in estimated costs see Table 11-1.

Table 5-4
Lake Hills South Reservoir
Summary of Estimated Costs

	0.4g	0.3g
Alternative 3 – Case One	$435,000	$320,000
Alternative 3 – Case Two	$420,000	$305,000

Note: The difference in cost between the Lake Hills North and Lake Hills South reservoirs is due primarily to the fact that the roof plate adjacent to the roof hatch is badly corroded and is in need of repair (see Section 11).

Table 5-4
Lake Hills South Reservoir
Summary of Estimated Costs

	0.4G	0.3G
Alternative 3 – Case One	$316,000	$238,000
Alternative 3 – Case Two	$311,000	$233,000

Note: The difference in cost between the Lake Hills North and Lake Hills South reservoirs is due primarily to the fact that the roof plate adjacent to the roof hatch is badly corroded and is in need of repair (see Section 11).

Section 6

MONTGOMERY WATSON

Section 6
Woodridge

The Woodridge reservoir, designed in 1955 by Harstad & Associates, is the oldest of the seven reservoirs included in the structural/seismic evaluation. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition, alternative seismic retrofits, including a cost comparison, have been developed for the Woodridge reservoir. The evaluation of the alternatives and a recommended seismic retrofit alternative are also presented in this section. Photographs of the Woodridge reservoir are included in Appendix B.

SITE INVESTIGATION

Site investigations for the Woodridge reservoir have included on-site inspections when the reservoir was fall and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and reports resulting from these investigations are included in Volume II of this preliminary design report.

Site Characteristics

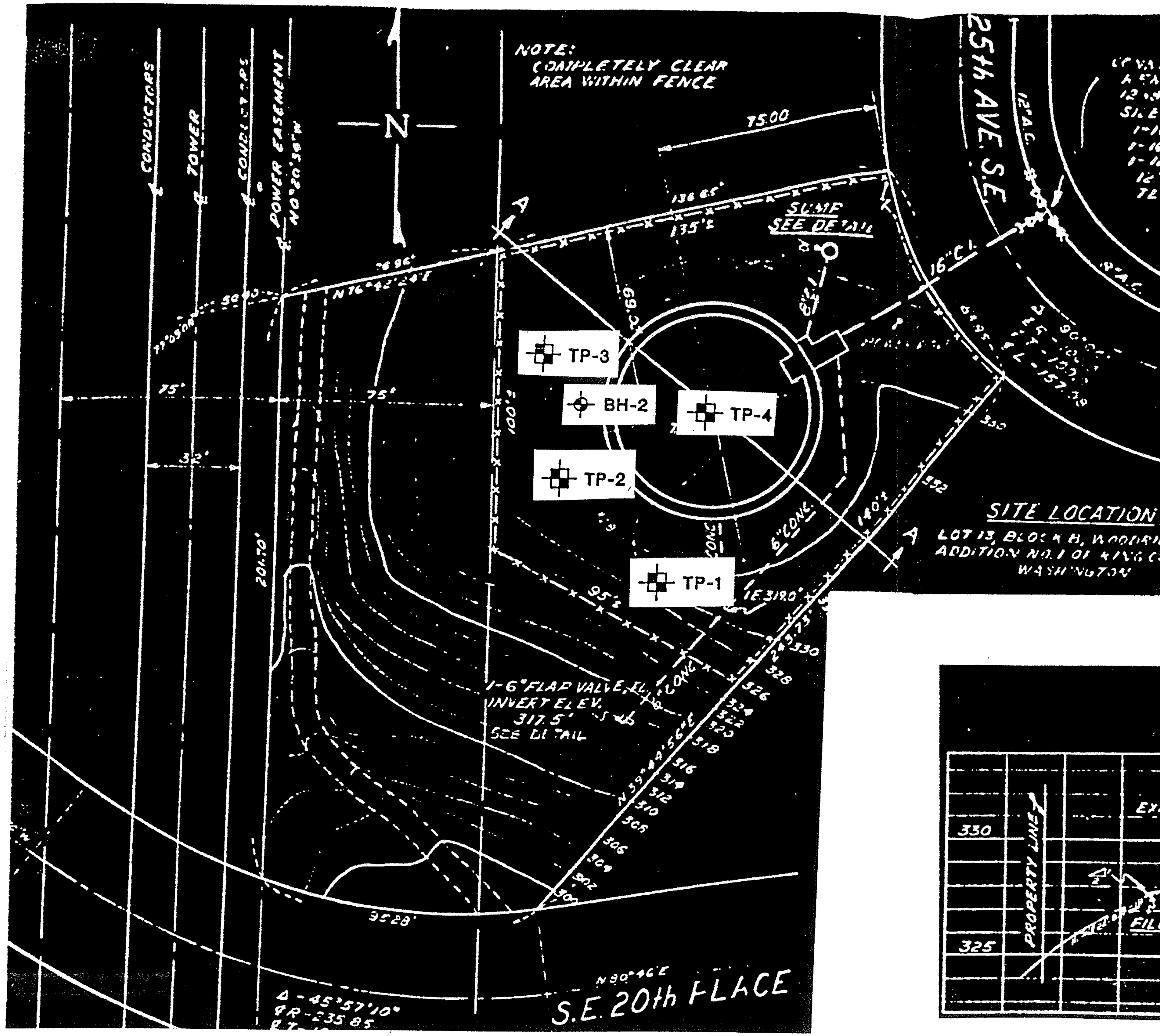
The Woodridge reservoir site plan is shown on Figure 6-1. Based on the 1955 plans for the Woodridge reservoir, it appears that original site grades were zero to three feet higher than current grades adjacent to the tank. Therefore, it is concluded that the reservoir is located in a cut area. Based on the data reviewed and subsurface investigations, the Woodridge reservoir site appears to be underlain by glacial till and advance outwash. The fill is approximately 5 feet in depth, and consists of medium dense sand, with varying silt and gravel contents. A 2.5-foot layer of glacial till is found beneath the fill. Extending at least to 48.5 feet beneath the fill is a layer of advance outwash consisting of medium dense to very dense sand with varying amounts of silt. Based on the 1955 plans, the existing reservoir footings are anticipated to be founded on the glacially deposited soil. Groundwater was not encountered during the subsurface explorations.

The ring wall foundation is 3.5 feet wide and 4 feet deep. The interior of the ring wall foundation was backfilled with compacted pit run gravel, with the exception of the four inches immediately below the bottom of the tank, which consists of oiled sand.

General Reservoir Condition

The interior of the Woodridge reservoir was completely painted towards the end of 1997, and is therefore in very good condition. The access manway, piping connections, and steel ladder located within the tank are all in good condition. Some pitting was observed in the floor plate beneath the most recent painting layer.

At the time of the inspection, floor and shell plate thickness were acquired for use in the seismic analysis structural calculations. The data was primarily collected for characterization of existing

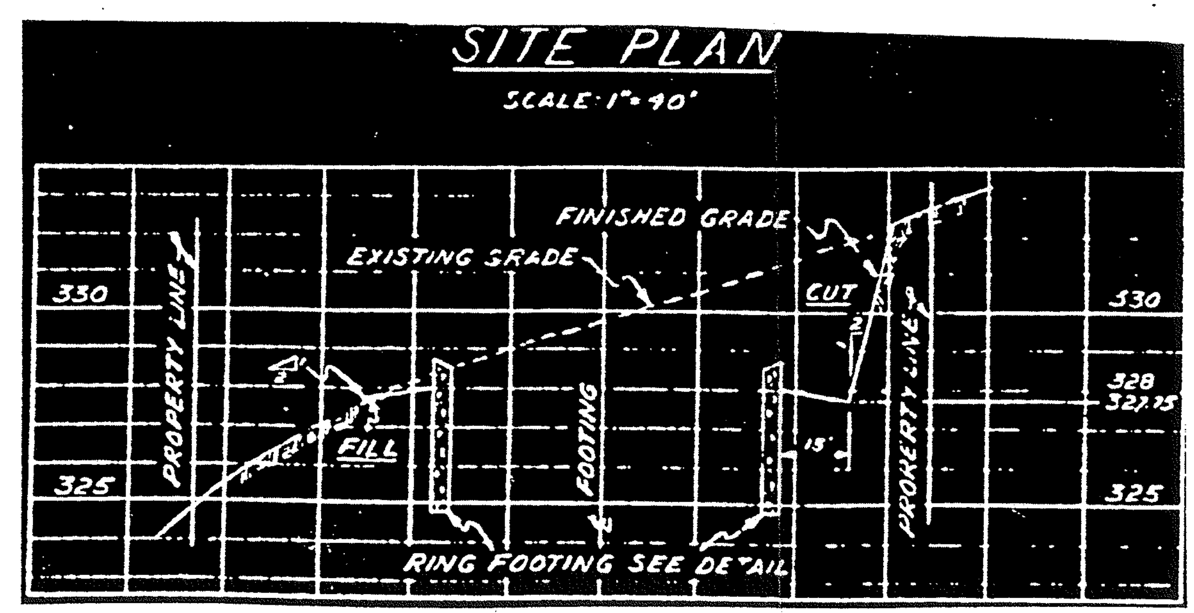


LEGEND

BH-1 BORING DESIGNATION AND APPROXIMATE LOCATION, (HWA, 1998)

TP-1 TEST PIT DESIGNATION AND APPROXIMATE LOCATION, (1955)

APPROXIMATE SCALE 1"=40'



REFERENCE: As-built plans provided by the City of Bellevue titled "General Facilities Standpipe Foundation and Piping Details". Prepared by Harstad and Associates, dated Sept. 1955.

MONTGOMERY WATSON

City of Bellevue
Woodridge Reservoir,
Site Plan and Subsurface Exploration Plan
FIGURE 6-1

steel thickness but also suggests that no significant general corrosion has occurred in the shell over the service life of the tank. The tank plate thickness results are presented in Table 6-1.

TABLE 6-1
Woodridge Reservoir
Tank Shell Thickness

Shell Course	Thickness, in.
Floor	0.490
1	1.125
2	1.035
3	0.855
4	0.775
5	0.650
6	0.530
7	0.405
8	0.295
9	0.255
Roof 1	0.260

Grout between the exterior floor plate and ring wall foundation is in poor condition. The exposed concrete on top of the ring wall foundation is weathered with minor spalling. The steel ladder, safety cage and safety rail located at the tank exterior is all in good condition, as is the electrical conduit raceway adjacent to the ladder. The exterior of the access manway is also in good condition.

There is an existing underdrain system below the tank and a perimeter drainage system located approximately 15 feet beyond the tank. No problems have been identified with the drainage system.

Weld Condition

Based on the visual inspection of the Woodridge reservoir, the interior floor plate and shell plate welds are in good condition. The floor plate is lapped and fillet welded. Exterior shell plate welds also appear to be in good condition. The roof plate is lapped and fillet welded, and the welds appear to be in good condition.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Woodridge reservoir. Data collected included visual inspection of interior and exterior coatings, steel substrate, appurtenances and cathodic protection system components. Physical data collected on the tank included generic classification of coating, Toxicity Characteristic Leaching Procedure lead content, coating thickness and coating adhesion. The findings of the corrosion investigation are summarized in the following paragraphs.

Interior and Exterior Coatings

Visual inspection coupled with physical measurements formed the basis of the interior and exterior coatings investigation of the Woodridge reservoir. Visual inspection included qualitative evaluation of overall coating condition (e.g., blisters, underfilm corrosion, pinhole failure, cathodic disbondment), characterization of any floor, shell or appurtenance corrosion failure and close inspection of weld lanes and sharp edges. Physical measurements included interior and exterior coating thickness measurements on the wall and floor, and generic coating typing for both the interior and exterior coating systems. In addition, the exterior reservoir coating was tested for coating adhesion quality.

In general, the internal visual inspection showed the tank coating was in good condition. No blisters were noted in the shell or floor coating. However, some pin hole failures with iron oxide corrosion product were visible on the floor. Pitting was also observed beneath the most recent painting coat. These areas would be protected by a properly operating cathodic protection system. The weld seams and plate material laminations showed some corrosion and iron oxide staining. Stripe coating with a brush prior to spray applying a primer coat can provide the extra protection needed for these areas. Coatings on the interior tank appurtenances, ladder and structural steel, showed signs of accelerated failure. In particular, the ladder suffered a through-hole corrosion penetration.

Visual inspection of the roof and upper shell courses revealed no significant corrosion or coating failures. Some small spot rusting at pin hole coating failures was evident as well as minor surface corrosion of structural members in difficult to coat areas. These failure modes should not effect the operation or integrity of the tank. However, existence of such failures indicates that the cathodic protection system may require adjustment.

The lower four courses of the tank exterior have been overcastted with a maintenance coating. There is no visible sign of coating failure. Adhesion tests completed for the Woodridge reservoir showed failure at various coating interfaces. The significance of these results would require extensive testing to remove the inherent statistical variation of the methodology. Based on the findings for the exterior coating system, it is recommended that the existing coating may be encapsulated with minor surface preparation which includes removal of areas of delaminated coating.

Cathodic Protection Condition

The condition of cathodic protection was evaluated simultaneously with the interior and exterior coatings evaluation. Visual inspection of the Woodridge reservoir shows that a cathodic protection system exists at the reservoir site, and that the components of the system (e.g. anodes, reference electrode) remain in good condition. While present at the reservoir site, the cathodic protection system was not operating at the time of the site investigation. The potential control circuit card had been removed. This card monitors tank potential and adjusts rectifier current output as required to maintain the set point. The card should be reinstalled, the system tested and adjusted to achieve the accepted level of cathodic protection.

It is recommended that the output of the cathodic protection system be adjusted to maintain protected tank-to-electrolyte potential in accordance with National Association of Corrosion Engineers criteria. In addition, it is recommended that the City monitors the rectifier output (i.e., voltage and current) quarterly, and that the City performs checkout of the cathodic protection system annually.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Woodridge reservoir consist of the following:

- 8-inch overflow piping is rigidly attached to the tank wall and base. The overflow pipe penetration is 1 foot from the tank wall.
- 8-inch drain piping penetrates the tank floor plate 1 foot from the tank wall.
- 16-inch inlet/outlet pipe penetrates the tank floor plate 5.25 feet from the tank wall.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Woodridge reservoir site were determined as described in Section 2. Table 6-2 summarizes the seismic coefficients to be used for the evaluations consistent with Seismic Zones 3 and 4.

Table 6-2
Woodridge Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficient C_a	Seismic Coefficient C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.3	1.6	0.52	0.90	0.14	0.69

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing ring wall foundations be evaluated using an allowable soil bearing pressure of 5 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces. Hoop tensile forces in the ring wall foundations resulting from the fluid weight may be evaluated using an earth pressure coefficient of 0.4, assuming the ring walls are relatively rigid.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated that the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 600 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Woodridge reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

Seismic Forces

Table 6-3 summarizes the seismic forces derived from analyzing the reservoir by the pseudodynamic method per AWWA D-100 for a seismic event with accelerations of 0.4g and 0.3g.

Table 6-3
Woodridge Reservoir
Seismic Forces

Seismic Forces	0.4g	0.3g
Overturning Moment (k-ft)	138,896	104,172
Soil Bearing Pressure (ksf)	12.4	9.6
Static Hoop Stress (psi)	13,706	13,706
Seismic Hoop Stress (psi)	14,104	13,451
Compression Stress (psi)	2,761	2,111
Anchor Force (k)	611	448

The analysis shows that the shell plating is adequate for both seismic overturning compressive stress and tensile hoop stresses. This assumes the allowable stresses to be used for the analysis are from Chapter 3 of AWWA D-100 that assumes that high strength steels were not used in the construction of the Woodridge Reservoir. The allowable static hoop stress is 15,000 psi and the allowable static hoop stress in 17,000 psi. The allowable compression stress is 4,167 psi.

As shown in Table 3-3 of this report, the reservoir is anchored to the ringwall footing by 12 equally spaced anchors. The capacity of each of these anchors, assuming a material strength of 30 ksi, is 36k. The forces in the anchors shown in Table 6-3 due to overturning in a 0.4g and 0-3g seismic event are 611 and 448, respectively. Therefore, the existing anchors are inadequate to resist overturning and additional anchorage should be provided between the reservoir and the ring footing.

The allowable soil bearing pressure based on the results of the soil investigation for transient loading conditions such as seismic forces is 7.5 ksf. The actual bearing pressure for a 0.4g seismic event is 12.4 ksf and for a 0.3g seismic event is 9.6 ksf.

In accordance with AWWA D-100, resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, the portion of the contents that may be used to resist overturning is dependent on the width of the bottom annulus. The annulus may be thought of as a portion of the bottom plate that lifts off the foundation and supports the weight of the tank contents.

There are three criteria to determine the degree of overturning and consequently the amount of anchorage required for the reservoir. If the ratio of the overturning moment divided by the product of the square of the diameter of the reservoir and the weight of the tank shell, roof and contents is less than or equal to 0.785, then there is no uplift and no anchorage is required. If this ratio is between 0.785 and 1.54, then there is uplift but no anchorage is required. However, since there is uplift, anything that is with the annular ring outside the reservoir and connected to the reservoir must be allowed to move, relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the inside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than 1.54 the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Woodridge reservoir, this ratio is 3.46 and 2.60 for a 0.4g and 0.3g earthquake respectively. Because these ratios are so high the annular ring cannot be sufficiently thickened to resist overturning and the tank must be anchored to the ring footing.

ALTERNATIVE SEISMIC RETROFITS

As noted in the previous section, the Woodridge Reservoir must be anchored to the existing ring footing to resist overturning for a 0.4g and a 0.3g earthquake. In addition, the existing ring footing must be able to resist the same overturning moment. Three alternatives have been considered to seismically retrofit the reservoir to resist the overturning moment.

Alternative 1 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. Adding more weight to the footing would then increase the resistance of the ring footing to overturning. This would be accomplished by adding a new ring footing on the outside of the existing footing and doweling them together as previously shown in Figure 4-2.

Alternative 2 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. A portion of the bottom plate on the inside of the reservoir would be removed and a new ring footing would be added. The new ring footing would then be doweled into the existing footing as previously shown in Figure 4-3. The combination of the two footings would serve to support a portion of the tank contents sufficient to resist the overturning.

Alternative 3 is to use small diameter high strength earth anchors to resist overturning. The earth anchors would be drilled through the existing ring footing at equal spaces on the outside of the reservoir as previously shown in Figure 4-4. The anchors would then be connected to the shell plate by the use of anchor chairs.

Also noted in the "Seismic Forces" paragraph of this section, the size of the ring footing must be increased for the Woodridge Reservoir for a 0.4g and a 0.3g earthquake. In Alternatives 1 and 2 increasing the width of the footing to resist overturning would be adequate to decrease the soil bearing pressure below the allowable. Finally, in accordance with AWWA D-100 Section 13.6, foundations under flat-bottom tanks have fared well under seismic loading and the seismic loading does not provide justification for increased foundations. Therefore, in Alternative 3 the size of the ring footing does not need to be increased.

COMPARISON OF ALTERNATIVES

For the Woodridge Reservoir, Alternative 1 does not appear to be an adequate solution for a number of reasons. First, since the weight of the footing which resists overturning is relatively small compared to the weight of the reservoir plus its contents which cause overturning, simply adding weight to the footing is an inefficient method for increasing the overturning resistance. The width that would have to be added to the footing is greater than the width of the existing footing, making this alternative economically impractical.

Second, the existing footing is not adequately reinforced to support the weight of additional concrete from the new footing during uplift. Finally, the adhesive anchors connecting the tank shell to the footing are relatively low capacity anchors and the number required would cause them to be so close together that they would be essentially ineffective.

Alternative 2 does not appear to be an effective solution for two reasons. The first is that the required number and spacing of adhesive anchors would cause them to be essentially ineffective, similar to Alternative 1. The second reason is that the existing footing is not adequately reinforced to support the additional weight of the tank contents, required to resist overturning and the additional weight of the footing concrete.

Alternative 3 appears to be an adequate seismic retrofit solution. However, there are two different overturning cases that were considered for this alternative. In the first case, the earth anchors would be designed to resist the entire overturning moment. In the second case the earth anchors would be designed to resist an overturning moment which is slightly reduced. The overturning moment would be reduced by an amount that is equal to the square of the diameter of the reservoir multiplied by the sum of the weights of the reservoir shell plate and the roof structure multiplied by the ratio 1.54. This allows the roof structure and shell plate to resist a portion of the overturning moment up to a point where the reservoir would require anchorage. In this case piping connections would require modifications such that they allow movement.

Alternative 3, case one, requires 45-foot long earth anchors and anchor chairs at about a 4.25-foot spacing for a 0.4g earthquake and a 45-foot long earth anchor and anchor chairs at about a 5.75-foot spacing for a 0.3g earthquake. Alternative 3, case two, requires the same earth anchors at about a 5.25-foot spacing for a 0.4g earthquake and at about a 7.5-foot spacing for a 0.3g earthquake. In addition, for Alternative 3, condition two, the pipe connections for the 8 inch overflow pipe and the 8 inch drain pipe would be modified to be flexible connections. However, it should be noted that, because these pipes are embedded in the foundation and welded to the bottom plate of the reservoir, adding sufficient flexibility to their connection would require a significant amount of additional work and cost.

Finally, it should be noted that for the purposes of the computations in this preliminary design report, the spacing of soil anchors has been assumed to be uniform and it is understood that there is not much room on the site for drilling equipment. Respacing of anchors to avoid below-grade obstruction and above grade structures may be required. However, these items should not significantly impact the overall seismic capacity of the anchor system or the cost of construction.

Estimated costs for both conditions of Alternative 3 are summarized in Table 6-4 below for both a 0.4 g and a 0.3g seismic event. For a description of what is included in estimated costs see Table 11-1.

Table 6-4
Woodridge Reservoir
Summary of Estimated Costs

	0.4g	0.3g
Alternative 3 – Case One	$435,000	$320,000
Alternative 3 – Case Two	$420,000	$305,000

Section 7

MONTGOMERY WATSON

Section 7
Horizon View No. 1

The Horizon View No. I reservoir is located in a residential area near the intersection of 148th Avenue Southeast and Southeast 47th Place. Harstad & Associates designed it in 1963. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition, alternative seismic retrofits, including a cost comparison, have been developed for the Horizon View No. I reservoir. The evaluation of the alternatives and a recommended seismic retrofit alternative are also presented in this section. Photographs of the Horizon View No. 1 reservoir are included in Appendix B.

SITE INVESTIGATION

Site investigations for the Horizon View No. I reservoir have included on-site inspections when the reservoir was full and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and reports resulting from these investigations are included in Volume 11 of this preliminary design report.

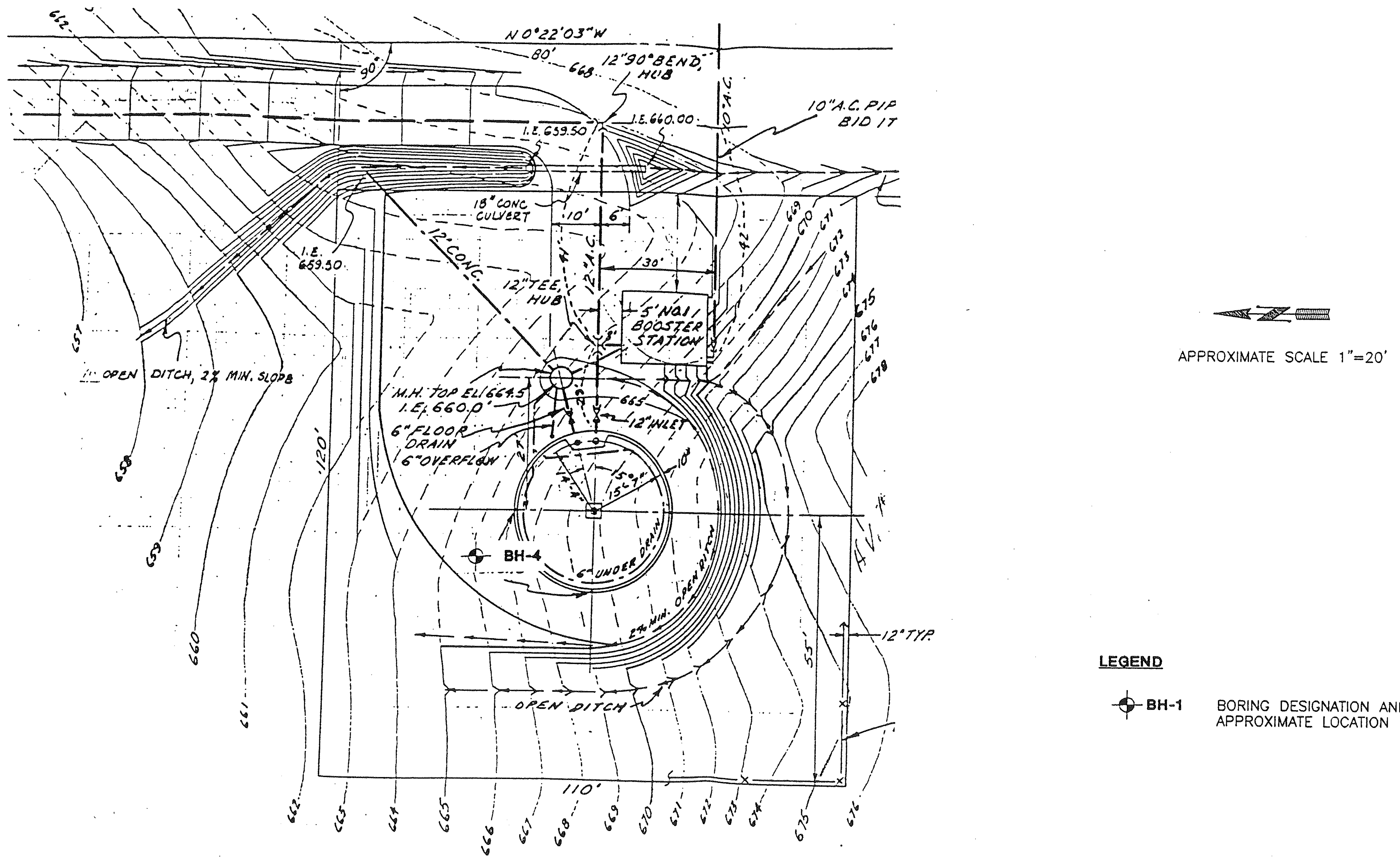
Site Characteristics

The Horizon View No. I reservoir site plan is shown on Figure 7-1. Based on the 1963 plans for the Horizon View No. 1 reservoir, it appears that original site grades were zero to six feet higher than current grades adjacent to the tank. Therefore, it is concluded that the reservoir is located in a cut area. Based on the data reviewed and subsurface investigations, the Horizon View No. I reservoir site appears to be underlain by glacial till. Approximately 4.5 feet of till overlay the glacial till. The fill consists of medium dense gravelly silty, sand extending to the depth of the subsurface exploration of 28 feet. Based on the 1963 plans, the existing reservoir footings are anticipated to be founded on the glacially deposited soil. Groundwater was not encountered during the subsurface explorations.

The ring wall foundation is 10-inches wide and extends from 2 feet to an unknown depth below the base of the tank. The interior of the ring wall foundation was backfilled with compacted pit run gravel, with the exception of the four inches immediately below the bottom of the tank, which consists of oiled sand. A center column bearing on a 3-foot square spread footing provides support for the roof structure.

General Reservoir Condition

In general the interior and-exterior paint is in good condition. However, some corrosion of the interior shell plate near the high water level was observed. The access manway, piping connections, and steel ladder located within the tank are all in good condition. Some pitting was observed in the floor plate beneath the most recent painting layer.



LEGEND

BH-1 BORING DESIGNATION AND APPROXIMATE LOCATION

REFERENCE: As-built plans provided by the City of Bellevue titled "Schedule A, Eastgate No. 1 and 2 Reservoir Foundation and Site Development". Prepared by Harstad and Associates, dated Feb. 1963.

MONTGOMERY WATSON

City of Bellevue
Horizon View No. 1 Reservoir
Site Plan and Subsurface Exploration Plan
FIGURE 7-1

At the time of the inspection, floor and shell plate thickness were acquired for use in the seismic analysis structural calculations. The data was primarily collected for characterization of existing steel thickness but also suggests that no significant general corrosion has occurred in the shell over the service life of the tank. The tank plate thickness results are presented in Table 7- 1.

TABLE 7-1
Horizon View No. 1 Reservoir
Tank Shell Thickness

Shell Course	Thickness, in.
Floor Plate	0.250
1	0.260
2	0.260
3	0.255
4	0.250
5	0.250
Roof 1	0.195

Grout between the exterior floor plate and ring wall foundation had recently been painted making it difficult to evaluate its condition. However, it appears be in fair condition. The exposed concrete on top of the ring wall foundation is weathered. The steel ladder, safety post, overflow and access manway located at the tank exterior are all in good condition. The roof hatch is also in good condition. The paint on the exterior of the roof is in fair condition, and could benefit from maintenance.

There is an existing underdrain system below the tank and a shallow drainage ditch located at the toe and at the top of the adjacent slope. During the site visit ponding was observed at the toe of the slope. No problems have been identified with the drainage system.

Weld Condition

Based on the visual inspection of the Horizon View No. 1 reservoir, the interior floor plate and shell plate welds are in good condition. The floor plate is lapped and fillet welded. Exterior shell plate welds also appear to be in good condition. The roof plate is lapped and fillet welded, and the welds appear to be in good condition.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Horizon View No. I reservoir. Data collected included visual inspection of interior and exterior coatings, steel substrate, appurtenances and cathodic protection system components. Physical data collected on the tank included generic classification of coating, Toxicity Characteristic Leaching Procedure lead content, coating thickness and coating adhesion. The findings of the corrosion investigation are summarized in the following paragraphs.

Interior and Exterior Coatings

Visual inspection coupled with physical measurements formed the basis of the interior and exterior coatings investigation of the Horizon View No. 1 reservoir. Visual inspection included qualitative evaluation of overall coating condition (e.g., blisters, underfilm corrosion, pinhole failure, cathodic disbondment), characterization of any floor, shell or appurtenance corrosion failure and close inspection of weld lanes and sharp edges. Physical measurements included interior and exterior coating thickness measurements on the wall and floor, and generic coating typing for both the interior and exterior coating systems. In addition, the exterior reservoir coating was tested for coating adhesion quality.

The thicknesses of the internal shell coating are typical, and are sufficient to provide protection. Coating thicknesses on the floor were also reasonable. However, variations of thickness across the floor were observed. Corrosion was heavy on the floor directly beneath the center column support. Sections of the floor show fairly extensive pitting that occurred prior to the last maintenance coating application. No coating failures were visible near these pit sites suggesting that the corrosion occurred prior to the most recent coating. These areas should be monitored during subsequent inspections.

In general, the internal visual inspection showed the tank coating was in good condition. No blisters were noted in the shell or floor coating. However, some pin hole failures with iron oxide corrosion product were visible on the floor. Pitting was also observed beneath the most recent painting coat. Existence of these types of failures indicates that the cathodic protection system may require adjustment. The weld seams and plate material laminations showed some corrosion and iron oxide staining. Stripe coating with a brush prior to spray applying a primer coat can provide the extra protection needed for these areas. Coatings on the interior tank appurtenances, ladder and structural steel, remained in good condition.

Visual inspection of the roof and upper shell courses revealed no significant corrosion or coating failures. Some small spot rusting at pin hole coating failures was evident as well as minor surface corrosion of structural members in difficult to coat areas. These failure modes should not have an impact on the operation or integrity of the tank.

The lower four courses of the tank exterior have been overcoated with a maintenance coating. There is no visible sign of coating failure. Adhesion tests completed for the Horizon View No. 1 reservoir showed mixed failure at various coating interfaces. The significance of these results would require extensive testing to remove the inherent statistical variation of the methodology.

Fourier Transform Infrared analysis suggest that the generic type of the interior and exterior coating systems is epoxy based. The data on the coating samples also show that the interior and exterior coating samples were well below the 5.0 ppm maximum concentration for leachable lead. Based on findings for the exterior coating system it is recommended that the existing coating system may be encapsulated with minor surface preparation which includes removal of areas of delaminated coating.

Cathodic Protection Condition

The condition of cathodic protection was evaluated simultaneously with the interior and exterior coatings evaluation. Visual inspection of the Horizon View No. 1 reservoir shows that a cathodic protection system exists at the reservoir site, and that the components of the system (e.g. anodes, reference electrode) remain in good condition. While present at the reservoir site, the cathodic protection system was not operating at the time of the site investigation. The set potential for the tank was well below the National Association of Corrosion Engineers criteria for adequate cathodic protection. Cathodic protection current at the existing control point setting would not prevent most steel corrosion. This low set point may account for the presence of red iron oxide corrosion product noted during the internal inspection.

It is recommended that the output of the cathodic protection system be adjusted to maintain protected tank-to-electrolyte potential in accordance with National Association of Corrosion Engineers criteria. In addition, it is recommended that the City monitors the rectifier output (i.e., voltage and current) quarterly, and that the City performs checkout of the cathodic protection system annually.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Horizon View No. I reservoir consist of the following:

- 6-inch overflow piping is rigidly attached to the outside of the tank wall and penetrates the tank wall near the top of the tank wall.
- 6-inch drain piping penetrates the tank floor 1.25 feet from the tank wall.
- 12-inch inlet/outlet pipe penetrates the tank floor 1.25 feet from the tank wall.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Horizon View No. I reservoir site were determined as described in Section 2. Table 7-2 summarizes the seismic coefficients to be used for the evaluations consistent with Seismic Zones 3 and 4.

Table 7-2
Horizon View Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficient C_a	Seismic Coefficient C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.3	1.6	0.52	0.90	0.14	0.69

The slope of the site is relatively stable and poses no significant threat to the reservoir during an earthquake. Some continued raveling of the slope and deposition of sloughing soil at the toe of the slope should be anticipated. Measures to improve and control the site drainage will increase the general stability of the slope and will reduce surficial erosion. Grading and regarding the drainage ditches along the tank perimeter and the top of the slope to prevent watering from ponding around the tank would be appropriate measures to increase general stability.

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing ring wall foundations be evaluated using an allowable soil bearing pressure of 4 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces. Hoop tensile forces in the ring wall foundations resulting from the fluid weight may be evaluated using an earth pressure coefficient of 0.4, assuming the ring walls are relatively rigid.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 800 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Horizon View No. I reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

Seismic Forces

Table 7-3 summarizes the seismic forces derived from analyzing the reservoir by the pseudodynamic method per AWWA D-100 for a seismic event with accelerations of 0.4g and 0.3g.

Table 7-3
Horizon View No. 1 Reservoir
Seismic Forces

Seismic Forces	0.4g	0.3g
Overturning Moment (k-ft)	7,390	5,542
Soil Bearing Pressure (ksf)	12.7	9.7
Static Hoop Stress (psi)	12,835	12,385
Seismic Hoop Stress (psi)	12,381	12,013
Compression Stress (psi)	3,294	2,517
Anchor Force (k)	223	163

The analysis shows that the shell plating is adequate for both seismic overturning compressive stress and tensile hoop stresses. This assumes the allowable stresses to be used for the analysis are from Chapter 3 of AWWA D-100 that assumes that high strength steels were not used in the construction of the Horizon View No. 1 Reservoir. The allowable static hoop stress is 15,000 psi and the allowable seismic hoop stress is 17,000 psi. The allowable compression stress is 3,465 psi.

As shown in Table 3-3 of this report, the reservoir is anchored to the ringwall footing by 4 equally spaced anchors. The capacity of each of these anchors, assuming a material strength of 30 ksi, is 18k. The forces in the anchors shown in Table 7-3 due to overturning in a 0.4g and 0.3g seismic event are 223 and 163k respectively. Therefore, the existing anchors are inadequate to resist overturning and additional anchorage should be provided between the reservoir and the ring footing. .

The allowable soil bearing pressure based on the results of the soil investigation for transient loading conditions such as seismic forces is 6 ksf. The actual bearing pressure for a 0.4g seismic event is 12.7 ksf and for a 0.3g seismic event is 9.7 ksf.

In accordance with AWWA D-100, resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, the portion of the contents that may be used to resist overturning is dependent on the width of the bottom annulus. The annulus may be thought of as a portion of the bottom plate that lifts off the foundation and supports the weight of the tank contents.

There are three criteria to determine the degree of overturning and consequently the amount of anchorage required for the reservoir. If the ratio of the overturning moment divided by the product of the square of the diameter of the reservoir and the weight of the tank shell, roof and contents is less than or equal to 0.785, then there is no uplift and no anchorage is required. If this ratio is between 0.785 and 1.54, then there is uplift but no anchorage is required. However, since there is uplift, anything that is within the annular ring or outside the reservoir and connected to the reservoir must be allowed to move, either with the reservoir or relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the inside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than 1.54 the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Horizon View No. 1 reservoir, this ratio is 3.82 and 2.87, respectively, for a 0.4g and 0.3g earthquake. Because these ratios are so high the annular ring cannot be sufficiently thickened to resist overturning and the tank must be anchored to the ring footing.

ALTERNATIVE SEISMIC RETROFITS

As noted in the "Seismic Forces" subsection, the Horizon View No. 1 Reservoir must be anchored to the existing ring footing to resist overturning for a 0.4g and a 0.3g earthquake. In addition, the existing ring footing must be able to resist overturning. Three alternatives have been considered to seismically retrofit the reservoir to resist the overturning.

Alternative 1 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. Adding more weight to the footing would then increase the resistance of the ring footing to overturning. This would be accomplished by adding a new ring footing on the outside of the existing footing and doweling them together as previously shown in Figure 4-2.

Alternative 2 is to first connect the shell plate of the reservoir directly to the ring footing using closely spaced adhesive anchors. A portion of the bottom plate on the inside of the reservoir would be removed and a new ring footing would be added. The new ring footing would then be doweled into the existing footing as previously shown in Figure 4-3. The combination of the two footings would serve to support a portion of the tank contents sufficient to resist the overturning.

Alternative 3 is to connect the shell plate of the reservoir directly to the ring footing with closely spaced adhesive anchors. Then a new ring footing would be added to the outside of the existing footing and the two footings would be doweled together. Finally, a series of equally spaced small diameter, high-strength earth anchors would be drilled through the new footing as shown in Figure 7-2.

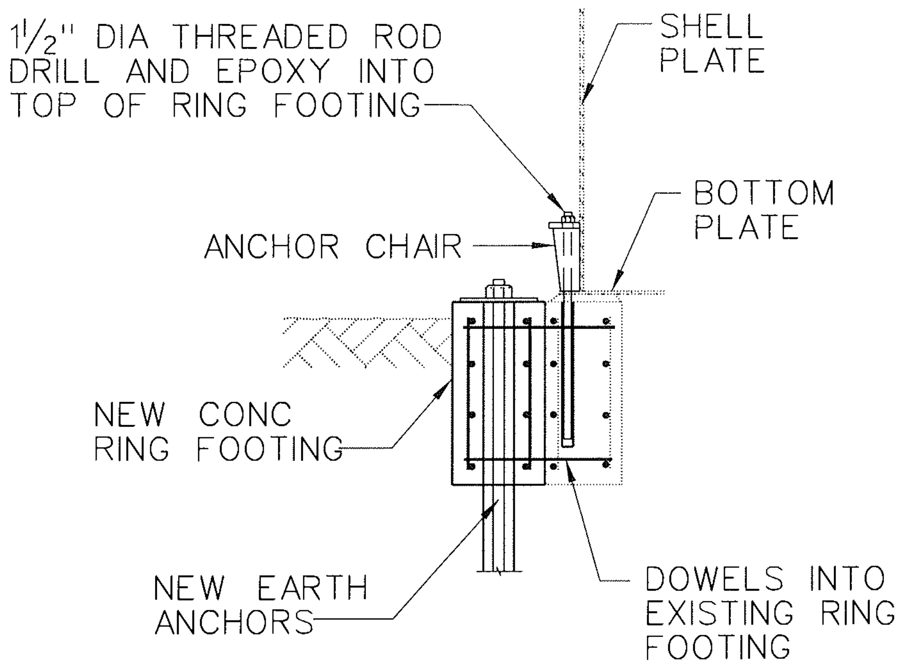
Also noted in the "Seismic Forces" subsection, the size of the ring footing must be increased for the Horizon View No. 1 Reservoir for a 0.4g and a 0.3g earthquake. In Alternatives 1 and 2 increasing the width of the footing to resist overturning would be adequate to decrease the soil bearing pressure below the allowable. Finally, in accordance with AWWA D-100 Section 13.6, foundations under flat-bottom tanks have fared well under seismic loading and the seismic loading does not provide justification for increased foundations. Therefore, in Alternative 3 the size of the ring footing does not need to be increased.

COMPARISON OF ALTERNATIVES

For the Horizon View No. 1 Reservoir, Alternative 1 does not appear to be an adequate solution for two reasons. First, since the weight of the footing which resists overturning is relatively small compared to the weight of the reservoir plus its contents which cause overturning, simply adding weight to the footing is an inefficient method for increasing the overturning resistance. The width that would have to be added to the footing is greater than the width of the existing footing, making this alternative economically impractical. Second, the existing footing is not adequately reinforced to support the weight of additional concrete from the new footing during uplift.

Alternative 2 does not appear to be an effective solution because the existing footing is not adequately reinforced to support the additional weight of the tank contents required to resist overturning and support the additional weight of the footing concrete.

Alternative 3 appears to be an adequate seismic retrofit solution. However, there are two different overturning cases that were considered for this alternative. In the first case, the earth anchors would be designed to resist the entire overturning moment. In the second case the earth



SECTION THROUGH RING FOOTING

½" = 1'-0"

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Bellevue, Washington

CITY OF BELLEVUE
RESERVOIR ANCHORING ALTERNATIVE 1

FIGURE
7-2

anchors would be designed to resist an overturning moment which is slightly reduced. The overturning moment would be reduced by an amount that is equal to the square of the diameter of the reservoir multiplied by the sum of the weights of the reservoir shell plate and the roof structure multiplied by the ratio 1.54. This allows the roof structure and shell plate to resist a portion of the overturning moment up to a point where the reservoir would require anchorage. In this case piping connections would require modifications so that they allow movement.

Alternative 3, case one, requires twelve, 30-foot long earth anchors and adhesive anchors with anchor chairs at about a 2-foot spacing for a 0.4g earthquake and twelve 25-foot long earth anchor and adhesive anchors with anchor chairs at about a 2.5-foot spacing for a 0.3g earthquake. Alternative 3, case two requires twelve, 25-foot long earth anchors and adhesive anchors with anchor chairs at about a 2.25-foot spacing for a 0.4g earthquake and twelve, 20-foot long earth anchors and adhesive anchors with anchor chairs at about a 3.25-foot spacing for a 0.3g earthquake. In addition, for Alternative 3, condition two, flexible couplings would be added to the pipe connections for the 6-inch overflow pipe, 6-inch drain pipe and 12-inch inlet/outlet pipe. It should be noted that because the pipes are embedded in the foundation and connected to the bottom plate of the reservoir, adding sufficient flexibility to their connection would require a significant amount of additional work and cost. Finally, the width of the existing footing for both conditions should be increased by adding a new 1-foot by 2.5-foot ring to the outside of the footing.

Estimated costs for both conditions of Alternative 3 are summarized in Table 7-4 below for both a 0.4 g and a 0.3g seismic event. For a description of what is included in estimated costs see Table 11-1.

Table 7-4
Horizon View No. 1 Reservoir
Summary of Estimated Costs

	0.4g	0.3g
Alternative 3 – Case One	$150,000	$130,000
Alternative 3 – Case Two	$170,000	$150,000

anchors would be designed to resist an overturning moment which is slightly reduced. The overturning moment would be reduced by an amount that is equal to the square of the diameter of the reservoir multiplied by the sum of the weights of the reservoir shell plate and the roof structure multiplied by the ratio 1.54. This allows the roof structure and shell plate to resist a portion of the overturning moment up to a point where the reservoir would require anchorage. In this case piping connections would require modifications so that they allow movement.

Alternative 3, case one, requires twelve, 30-foot long earth anchors and adhesive anchors with anchor chairs at about a 2-foot spacing for a 0.4g earthquake and twelve 25-foot long earth anchor and adhesive anchors with anchor chairs at about a 2.5-foot spacing for a 0.3g earthquake. Alternative 3, case two requires twelve, 25-foot long earth anchors and adhesive anchors with anchor chairs at about a 2.25-foot spacing for a 0.4g earthquake and twelve, 20-foot long earth anchors and adhesive anchors with anchor chairs at about a 3.25-foot spacing for a 0.3g earthquake. In addition, for Alternative 3, condition two, flexible couplings would be added to the pipe connections for the 6-inch overflow pipe, 6-inch drain pipe and 12-inch inlet/outlet pipe. It should be noted that because the pipes are embedded in the foundation and connected to the bottom plate of the reservoir, adding sufficient flexibility to their connection would require a significant amount of additional work and cost. Finally, the width of the existing footing for both conditions should be increased by adding a new 1-foot by 2.5-foot ring to the outside of the footing.

Estimated costs for both conditions of Alternative 3 are summarized in Table 7-4 below for both a 0.4 g and a 0.3g seismic event. For a description of what is included in estimated costs see Table 11-1.

Table 7-4
Horizon View No. 1 Reservoir
Summary of Estimated Costs

	0.4g	0.3g
Alternative 3 – Case One	$131,000	$113,000
Alternative 3 – Case Two	$145,000	$125,000

Section 8

MONTGOMERY WATSON

Section 8
Parkside

Harstad & Associates designed the Parksite reservoir in 1962. The Parksite reservoir is a large diameter, relatively short, 2.0 million gallon welded steel tank. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition, alternative seismic retrofits, including a cost comparison, have been developed for the Parksite reservoir. The evaluation of the alternatives and a recommended seismic retrofit alternative are also presented in this section. Photographs of the Parksite Reservoir are included in Appendix B.

SITE INVESTIGATION

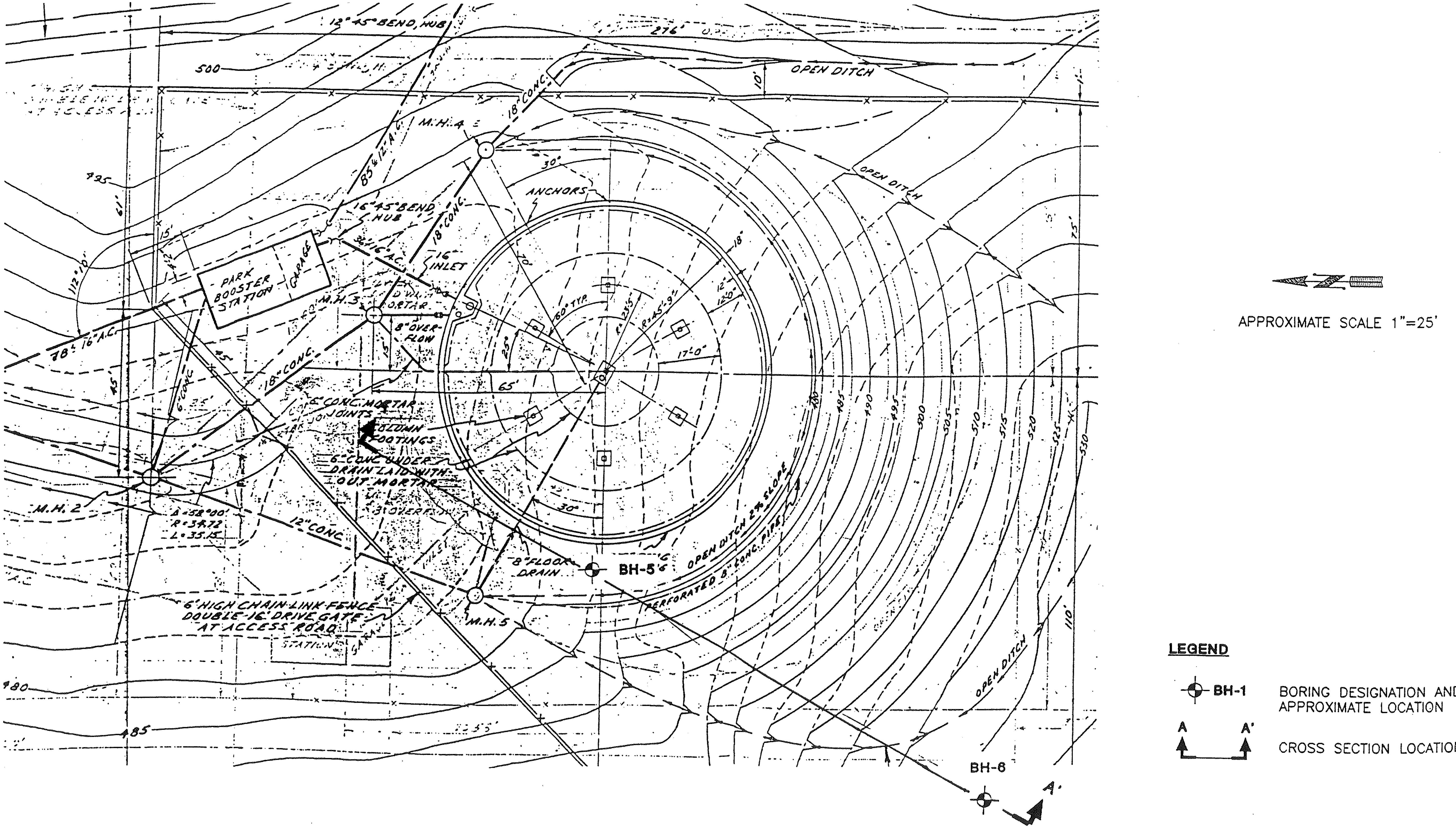
Site investigations for the Parksite reservoir have included on-site inspections when the reservoir was full and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and reports resulting from these investigations are included in Volume 11 of this preliminary design report.

Site Characteristics

The Parksite reservoir site plan is shown on Figure 8-1. Based on the 1962 plans for the Parksite reservoir, it appears that the original site sloped at an angle as much as 2.5-foot horizontal to 1-foot vertical. When the site was developed, the slope along the south side of the site was cut back to accommodate the tank. The lower 3 to 6 feet of the slope was cut to near vertical and is supported by a rockery. The current condition of the rockery appears to be stable. It was also observed during the site visit that the mature tree trunks along the cut slope have remained vertical over the years, further confirming the stability of the slope. Although the slope appears stable, the steepness of the slope will result in some raveling and deposition of sloughing soil at the toe of the slope.

Based on the data reviewed and the subsurface explorations, the Parksite reservoir site appears to be underlain by the Blakely Formation, glacial till and fill soils. The Blakely Formation at the Parksite reservoir is weathered and consists of very stiff to hard sandy silt, interbedded with very dense silty sand. The Blakely Formation was overlain by glacial till at some locations on the site. In addition to the Blakely Formation and the glacial till, fill is anticipated to underlie the northern portion of the site. Based on the 1962 reservoir plans, approximately 25 feet of the northern portion of the reservoir may be underlain by fill. However, the plans suggest that the existing ring wall is founded on native soils, likely glacial till or weathered Blakely Formation soils. Groundwater seepage was observed within the Blakely Formation soils at an approximate depth of 19 feet.

The ring wall foundation is 1.5-feet wide with a varying depth. In general the ring wall is shallower along the south side of the tank and deeper along the north side of the tank. The interior of the ring wall foundation was backfilled with compacted pit run gravel, with the



REFERENCE: As-built plans provided by the City of Bellevue titled "Schedule A, Parksite Reservoir Foundation and Site Development." Prepared by Harstad and Associates, dated Feb. 1963.

MONTGOMERY WATSON

City of Bellevue
Parksite Reservoir ,
Site Plan and Subsurface Exploration Plan
FIGURE 8-1

exception of the five inches immediately below the bottom of the tank, which consists of oiled sand.

General Reservoir Condition

General visual inspection of the Parksite reservoir indicates that interior and exterior paint is in good condition. However, there is evidence of minor corrosion at the top of the fourth course and bottom of the fifth course of the shell plate. The interior roof structure and surface paint appear to be in good condition.

The ladder located within the tank is painted steel with a safety cage and an aluminum safety post. All appear to be in good condition. The interior piping, floor drain, overflow and support, and the outlet/inlet piping, all appear to be in good condition with no apparent corrosion or deterioration. The access manway also appears to be in good condition. The exterior appurtenances, ladder, safety rail, and safety cage are in good condition relative to corrosion and deterioration.

At the time of the inspection, floor and shell plate thickness were acquired for use in the seismic analysis structural calculations. The data was primarily collected for characterization of existing steel thickness but also suggests that no significant general corrosion has occurred in the shell over the service life of the tank. The tank plate thickness results are presented in Table 8-1.

TABLE 8-1
Parksite Reservoir
Tank Shell Thickness

Shell Course	Thickness, in.
Floor Plate	0.500
1	0.820
2	0.630
3	0.455
4	0.260
5	0.255
Roof 1	0.250

Grout between the exterior floor plate and ring wall foundation is in poor condition, and absent in some places along the interface. The exposed concrete on top of the ring wall foundation is weathered with minor spalling.

There is an existing underdrain system below the tank and a 5-foot deep drainage system located around the tank perimeter at the toe of the existing slope. During several site visits water was observed to have ponded between the tank and the cut slope, indicating the perimeter interceptor drain is not functioning properly. Ponded water against the side of the steel tank could lead to corrosion problems. The existing interceptor drainage system along the toe of the slope should be replaced.

Weld Condition

Based on the visual inspection of the Parksite reservoir, the interior floor plate and shell plate welds are in good condition. The floor plate is lapped and fillet welded. Exterior shell plate welds also appear to be in good condition. The roof plate is lapped and fillet welded, and the welds appear to be in good condition.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Parksite reservoir. Data collected included visual inspection of interior and exterior coatings, steel substrate, appurtenances and cathodic protection system components. Physical data collected on the tank included generic classification of coating, Toxicity Characteristic Leaching Procedure lead content, coating thickness and coating adhesion. The findings of the corrosion investigation are summarized in the following paragraphs.

Interior and Exterior Coatings

Visual inspection coupled with physical measurements formed the basis of the interior and exterior coatings investigation of the Parksite reservoir. Visual inspection included qualitative evaluation of overall coating condition (e.g., blisters, underfilm corrosion, pinhole failure, cathodic disbondment), characterization of any floor, shell or appurtenance corrosion failure and close inspection of weld lanes and sharp edges. Physical measurements included interior and exterior coating thickness measurements on the wall and floor, and generic coating typing for both the interior and exterior coating systems. In addition, the exterior reservoir coating was tested for coating adhesion quality.

The coating thickness data collected for the Parksite reservoir for the interior shell are typical for tank linings and are sufficient to provide protection. Coating thicknesses on the floor were also reasonable for epoxy systems. Both the shell walls and floor show good consistency of applied thickness. In general, the internal visual inspection showed the tank coating was in good condition. No blisters were noted in the shell or floor coating. However, some pin hole failures with iron oxide corrosion product were visible on the floor. The existence of these types of failures indicates that the cathodic protection system may require adjustment.

Visual inspection of the roof and upper shell courses revealed no significant corrosion or coating failures. Some small spot rusting at pin hole coating failures was evident as well as minor surface corrosion of structural members in difficult to coat areas. These failure modes should not effect the operation or integrity of the tank.

The lower four courses of the tank exterior have been overcoated with a maintenance coating. There is no visible sign of coating failure. However, there were a couple of minor locations where coating was delaminated. Some damage to the coating on the west side of the reservoir was observed which appeared to be the result of vandalism. Further, results of the coating adhesion tests indicate excellent adhesion strength.

Fourier Transform Infrared analysis suggest that the generic type of the interior is epoxy based. The generic type for the exterior coating can be categorized as vinyl. The data on the coating samples also show that the interior samples were well below the 5.0 ppm maximum concentration for leachable lead. The exterior coating samples were above the limit at 8.2 ppm. However, based on the findings of the exterior coating system evaluation, it is recommended that the existing coating may be encapsulated with minor surface preparation which includes removal of areas of delaminated coating.

Cathodic Protection Condition

The condition of cathodic protection was evaluated simultaneously with the interior and exterior coatings evaluation. Visual inspection of the Parksite reservoir shows that a cathodic protection system exists at the reservoir site, and that the components of the system (e.g. anodes, reference electrode) remain in good condition. The cathodic protection system, if properly adjusted and maintained, should prevent significant corrosion in the areas where trace blisters have occurred.

It is recommended that the output of the cathodic protection system be adjusted to maintain protected tank-to-electrolyte potential in accordance with National Association of Corrosion Engineers criteria. In addition, it is recommended that the City monitors the rectifier output (i.e., voltage and current) quarterly, and that the City performs checkout of the cathodic protection system annually.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Parksite reservoir consist of the following:

- 8-inch overflow piping is rigidly attached to the tank wall and base. The overflow pipe penetration is 1.25 feet from the tank wall.
- 8-inch drain piping penetrates the tank floor plate adjacent to the center column.
- 16-inch inlet/outlet penetrates the tank floor plate, 4.25 feet from the tank wall.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Parksite reservoir site were determined as described in Section 2. Table 8-2 summarizes the seismic coefficients to be used for the evaluations consistent with Seismic Zones 3 and 4.

Table 8-2
Parksite Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficient C_a	Seismic Coefficient C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.3	1.6	0.52	0.90	0.14	0.69

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing ring wall foundations be evaluated using an allowable soil bearing pressure of 5 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces. Hoop tensile forces in the ring wall foundations resulting from the fluid weight may be evaluated using an earth pressure coefficient of 0.4, assuming the ring walls are relatively rigid.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated that the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 600 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Parksite reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

Seismic Forces

Table 8-3 summarizes the seismic forces derived from analyzing the reservoir by the pseudodynamic method per AWWA D-100 for a seismic event with accelerations of 0.4g and 0.3g.

Table 8-3
Parksite Reservoir
Seismic Forces

Seismic Forces	0.4g	0.3g
Overturning Moment (k-ft)	50,484	37,963
Soil Bearing Pressure (ksf)	6.0	4.7
Static Hoop Stress (psi)	14,692	14,692
Seismic Hoop Stress (psi)	17,223	16,039
Compression Stress (psi)	851	662
Anchor Force (k)	158	113

The analysis shows that the shell plating is adequate for both seismic overturning compressive stress and tensile hoop stresses. This assumes the allowable stresses to be used for the analysis are from Chapter 3 of AWWA D-100 that assumes that high strength steels were not used in the construction of the Parkside Reservoir. The allowable static hoop stress is 15,000 psi and the allowable seismic hoop stress is 17,000 psi. The allowable compressive stress is 2,256 psi.

As shown in Table 3-3 of this report, the reservoir is anchored to the ringwall footing by 4 equally spaced anchors. The capacity of each of these anchors, assuming a material strength of 30 ksi, is 18k. The forces in the anchors shown in Table 8-3 due to overturning in a 0.4g and 0.3g seismic event are 158 and 113k respectively. Therefore, the existing anchors are inadequate to resist overturning and additional anchorage should be provided between the reservoir and the ring footing.

The allowable soil bearing pressure based on the results of the soil investigation for transient loading conditions such as seismic forces is 7.5 ksf. The actual bearing pressure for a 0.4g seismic event is 6.0 ksf and for a 0.3g seismic event is 4.7 ksf.

In accordance with AWWA D-100, resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, the portion of the contents that may be used to resist overturning is dependent on the width of the bottom annulus. The annulus may be thought of as a portion of the bottom plate that lifts off the foundation and supports the weight of the tank contents.

There are three criteria to determine the degree of overturning and consequently the amount of anchorage required for the reservoir. If the ratio of the overturning moment divided by the product of the square of the diameter of the reservoir and the weight of the tank shell, roof and contents is less than or equal to 0.785, then there is no uplift and no anchorage is required. If this ratio is between 0.785 and 1.54 then there is uplift but no anchorage is required. However, since there is uplift, anything that is within the annular ring or outside the reservoir that is connected to the reservoir must be allowed to move, either with the reservoir or relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the outside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than 1.54 the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Parksite reservoir, this ratio is 1.10 and 0.83, respectively, for a 0.4g and 0.3g earthquake. These ratios are less than 1.54 so the reservoir is not required to be anchored to the ring footing or have the annular ring thickened. However, the flexibility of piping connections to the reservoir must be addressed.

ALTERNATIVE SEISMIC RETROFITS

As noted in the "Seismic Forces" subsection the Parksite Reservoir does not require anchorage to resist overturning for a 0.4g or a 0.3g earthquake and the actual soil bearing pressure is less than the allowable soil bearing pressure. However, according to the psuedodynamic: analysis, there is uplift for both a 0.4g and a 0.3g earthquake. Therefore, two alternatives were considered to seismically retrofit the reservoir.

Alternative 1 is to use small diameter high strength earth anchors to prevent uplift. The anchors would be drilled through the existing ring footing on equal spaces on the outside of the reservoir as previously shown in Figure 4-4. The anchors would then be connected to the shell plate by anchor chairs.

Alternative 2 would be to modify the piping connections so they allow movement relative to the reservoir.

COMPARISON OF ALTERNATIVES

Both alternatives appear to be adequate to seismically retrofit the Parksite Reservoir. Alternative 1 requires twelve, 45-foot long earth anchors and anchor chairs for a 0.4g earthquake and twelve, 35-foot long earth anchors and anchor chairs for a 0.3g earthquake. Alternative 2 requires that flexible couplings be added to the 8-inch overflow pipe. This would require installation of a 10-inch spool at the base of the overflow inside the reservoir, removal of a section of the overflow, and installation of a 10-inch transition type sleeve coupling between the remaining overflow and spool. In addition, the existing 16-inch A.C. inlet pipe between the 16-inch C.I. adapter and the park booster station should be removed and replaced with ductile iron pipe. The approximate construction cost for Alternative 1 is $125,000 for a 0.4g earthquake and $110,000 for a 0.3g earthquake. The cost for Alternative 2 is approximately $60,000. For a description of what is included in estimated costs see Table 11 - 1.

the reservoir must be allowed to move, either with the reservoir or relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the outside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than 1.54 the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Parksite reservoir, this ratio is 1.10 and 0.83, respectively, for a 0.4g and 0.3g earthquake. These ratios are less than 1.54 so the reservoir is not required to be anchored to the ring footing or have the annular ring thickened. However, the flexibility of piping connections to the reservoir must be addressed.

ALTERNATIVE SEISMIC RETROFITS

As noted in the "Seismic Forces" subsection the Parksite Reservoir does not require anchorage to resist overturning for a 0.4g or a 0.3g earthquake and the actual soil bearing pressure is less than the allowable soil bearing pressure. However, according to the psuedodynamic: analysis, there is uplift for both a 0.4g and a 0.3g earthquake. Therefore, two alternatives were considered to seismically retrofit the reservoir.

Alternative 1 is to use small diameter high strength earth anchors to prevent uplift. The anchors would be drilled through the existing ring footing on equal spaces on the outside of the reservoir as previously shown in Figure 4-4. The anchors would then be connected to the shell plate by anchor chairs.

Alternative 2 would be to modify the piping connections so they allow movement relative to the reservoir.

COMPARISON OF ALTERNATIVES

Both alternatives appear to be adequate to seismically retrofit the Parksite Reservoir. Alternative 1 requires twelve, 45-foot long earth anchors and anchor chairs for a 0.4g earthquake and twelve, 35-foot long earth anchors and anchor chairs for a 0.3g earthquake. Alternative 2 requires that flexible couplings be added to the 8-inch overflow pipe. This would require installation of a 10-inch spool at the base of the overflow inside the reservoir, removal of a section of the overflow, and installation of a 8-inch transition type sleeve coupling between the remaining overflow and spool. The approximate construction cost for Alternative 1 is $104,000 for a 0.4g earthquake and $92,000 for a 0.3g earthquake. The cost for Alternative 2 is approximately $45,000. For a description of what is included in estimated costs see Table 11 - 1.

Section 9

MONTGOMERY WATSON

Section 9
Pikes Peak

The Pikes Peak is the newest reservoir of the seven included in the structural/seismic evaluation. It was designed by Roy L. Gardner & Associates in 1968, and fabricated by General American Transportation Corporation. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition alternative seismic retrofits, including a cost comparison, have been developed for the Pikes Peak reservoir. The evaluation of the alternatives and a recommended in this section. Photographs of the Pikes Peak reservoir are included in Appendix B.

SITE INVESTIGATION

Site investigations for the Pikes Peak reservoir have included on-site inspections when the reservoir was full and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and reports resulting from these investigations are included in Volume II of this preliminary design report.

Site Characteristics

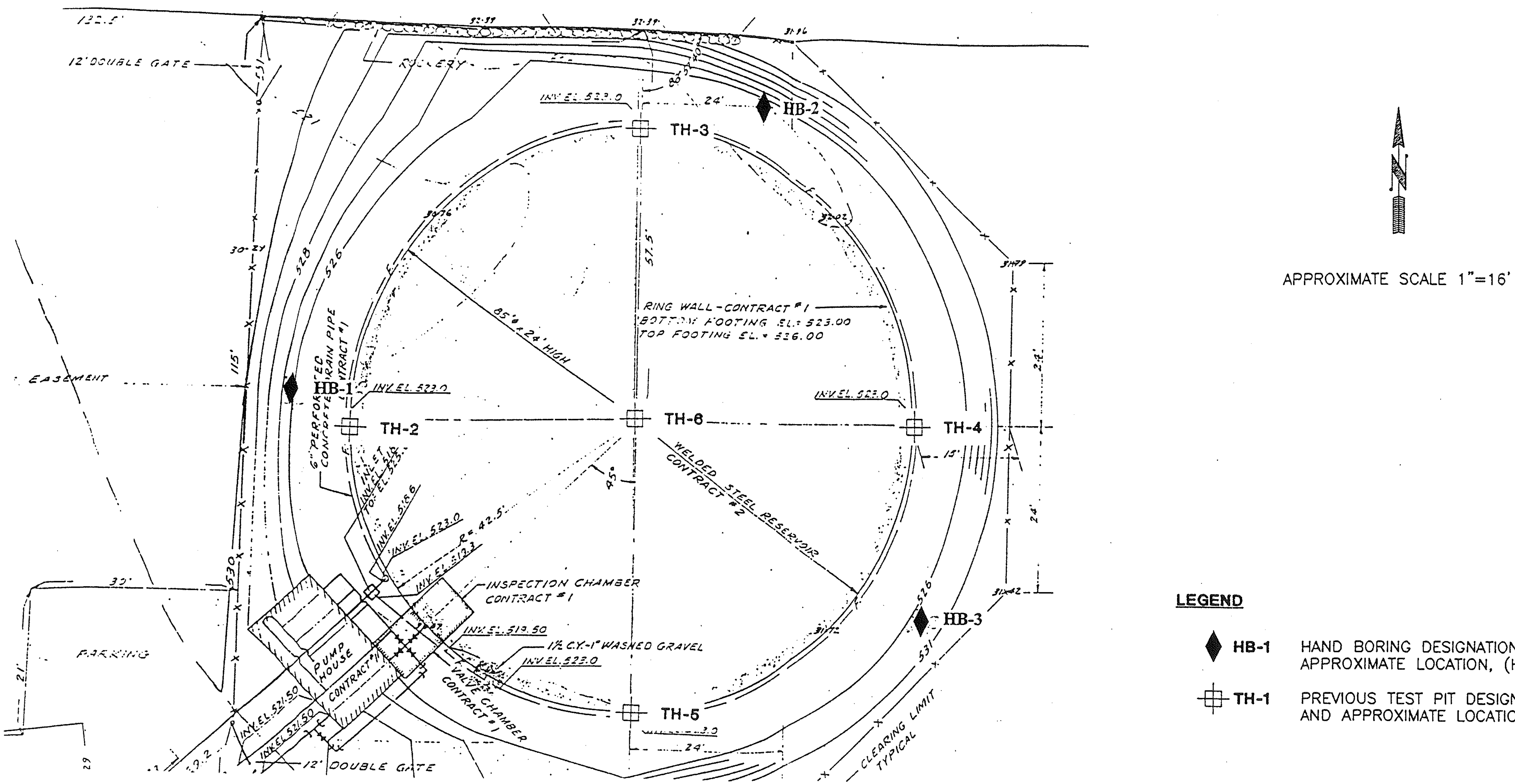
The Pikes Peak reservoir site plan is shown on Figure 9-1. Based on the 1968 plans for the Pikes Peak reservoir, it appears that original site grades were 0 to 5 feet higher than current grades adjacent to the existing reservoir. Therefore, it is concluded that the reservoir is located in a cut area. Based on the subsurface conditions observed from the HWA GeoSciences exploratory boring, the Pikes Peak reservoir site appears to be underlain by advance outwash, which is typically relatively dense. The data further indicates that the advance outwash consists of dense sand with varying amount of silt and gravel.

The ring wall foundation is 1-foot wide by 3-feet deep. The interior of the ring wall foundation was backfilled with compacted pit run gravel, with the exception of the four inches immediately below the bottom of the tank, which consists of oiled sand.

General Reservoir Condition

General visual inspection of the Pikes Peak reservoir indicates that interior and exterior paint is in good condition. However, the shell plate just below the roof plate is showing some deterioration, with concentrations of corrosion directly under the roof support beams. The interior roof structure has some minor corrosion. The overflow piping shows sign of corrosion, and is in need of cleaning and painting.

The ladder located within the tank is painted steel with what appears to be stainless steel connections and climbing device. All appear to be in good condition. The exterior appurtenances, piping, ladder, safety rail, and access manhole are in good condition relative to corrosion and deterioration, but should be painted.



LEGEND

♦ HB-1	HAND BORING DESIGNATION AND APPROXIMATE LOCATION, (HWA 1998)
⊞ TH-1	PREVIOUS TEST PIT DESIGNATION AND APPROXIMATE LOCATION, (1968)

REFERENCE: As-built plans provided by the City of Bellevue titled "Pikes Peak Reservoir Site Plan" Prepared by Gardner and Associates, dated Jan. 1968.

MONTGOMERY WATSON

City of Bellevue
Pikes Peak Reservoir,
Site Plan and Subsurface Exploration Plan
FIGURE 9-1

At the time of the inspection, floor and shell plate thickness were acquired for use in the seismic analysis structural calculations. The data was primarily collected for characterization of existing steel thickness but also suggests that no significant general corrosion has occurred in the shell over the service life of the tank. The tank plate thickness results are presented in Table 9- 1.

TABLE 9-1
Pikes Peak Reservoir
Tank Shell Thickness

Shell Course	Thickness, in.
Floor	0.260
1	0.490
2	0.385
3	0.260
Roof	0.250

Grout between the exterior floor plate and ring wall foundation is in poor condition, and absent in some places along the interface. The exposed concrete on top of the ring wall foundation is weathered with minor spalling. The roof vent at the center of the roof exterior is in good condition, as is the paint on the exterior of the roof.

There is an existing underdrain system below the tank, as well as a perimeter drainage system. No problems have been identified with the drainage system. Further, no groundwater was encountered during the subsurface exploration, and no standing water was apparent on the site.

Weld Condition

Based on the visual inspection of the Pikes Peak reservoir, the interior floor plate and shell plate welds are in good condition. The floor plate is lapped and fillet welded. Exterior shell plate welds also appear to be in good condition. The roof plate is lapped and fillet welded, and the welds appear to be in good condition.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Pikes Peak reservoir. Data collected included visual inspection of interior and exterior coatings, steel substrate, appurtenances and cathodic protection system components. Physical data collected on the tank included generic classification of coating, Toxicity Characteristic Leaching Procedure lead content, coating thickness and coating adhesion. The findings of the corrosion investigation are summarized in the following paragraphs.

Interior and Exterior Coatings

Visual inspection coupled with physical measurements formed the basis of the interior and exterior coatings investigation of the Pikes Peak reservoir. Visual inspection included qualitative evaluation of overall coating condition (e.g., blisters, underfilm corrosion, pinhole failure, cathodic disbondment), characterization of any floor, shell or appurtenance corrosion failure and close inspection of weld lanes and sharp edges. Physical measurements included interior and exterior coating thickness measurements on the wall and floor, and generic coating typing for both the interior and exterior coating systems. In addition, the exterior reservoir coating was tested for coating adhesion quality.

The coating thickness data collected for the Pikes Peak reservoir are somewhat lower than is typically found for internal tank linings, but should be sufficient to provide protection. In general, the internal visual inspection showed the tank coating was in good condition. Small trace blisters were visible on the tank shell. Deposits beneath the blisters indicate active corrosion. Corrosion and iron oxide staining were also apparent in most of the weld lanes and on the ladder and other appurtenances.

Visual inspection of the roof and upper shell courses revealed no significant corrosion or coating failures. Some small spot rusting at pin hole coating failures was evident as well as minor surface corrosion of structural members in difficult to coat areas. These failure modes should not effect the operation or integrity of the tank.

The tank exterior has been overcoated with a maintenance coating. This system is blistering and shows very poor intercoat adhesion. The outer most coating can be peeled away easily. The coating beneath this overcoat remains well adhered to the tank with little evidence of pin hole failure or blisters.

Fourier Transform Infrared analysis suggest that the generic type of the interior is epoxy based. The generic type for the exterior coating could not be identified using the Fourier Transform Infrared analysis. The data on the coating samples also show that the interior and exterior samples were well below the 5.0 ppm maximum concentration for leachable lead. Based on the findings for the exterior coating system, it is recommended that the existing coating system may be encapsulated with minor surface preparation that includes removal or areas of delaminated coating.

Cathodic Protection Condition

Currently there is not a cathodic protection system at the Pikes Peak reservoir. The installation of a cathodic protection system would mitigate internal corrosion as coating ages.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Pikes Peak reservoir consist of the following:

- 12-inch overflow piping is secured to tank wall in two places and penetrates tank wall near the base before going underground.

- 8-inch drain piping penetrates the tank floor plate.
- 16-inch inlet/outlet piping penetrates the tank floor plate.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Pikes Peak reservoir site were determined as described in Section 2. Table 9-2 summarizes the seismic coefficients to be used for the evaluations and are consistent with Seismic Zones 3 and 4.

Table 9-2
Pikes Peak Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficien t C_a	Seismic Coefficien t C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.3	1.6	0.40	0.62	0.12	0.62

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing ring wall foundations be evaluated using an allowable soil bearing pressure of 5 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces. Hoop tensile forces in the ring wall foundations resulting from the fluid weight may be evaluated using an earth pressure coefficient of 0.4, assuming the ring walls are relatively rigid.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated that the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 800 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Pikes Peak reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

Seismic Forces

Table 9-3 summarizes the seismic forces . derived from analyzing the reservoir by the pseudodynamic method per AWWA D-100 for a seismic event with accelerations of 0.4g and 0.3g.

Table 9-3
Pikes Peak Reservoir
Seismic Forces

Seismic Forces	0.4g	0.3g
Overturning Moment (k-ft)	15,462	11,597
Soil Bearing Pressure (ksf)	5.3	3.9
Static Hoop Stress (psi)	12,735	12,735
Seismic Hoop Stress (psi)	14,903	13,884
Compression Stress (psi)	571	451
Anchor Force (k)	0 (unanchored)	0 (unanchored)

The analysis shows that the shell plating is adequate for both seismic overturning compressive stress and tensile hoop stresses. This assumes the allowable stresses to be used for the analysis are from Chapter 3 of AWWA D-100 that assumes that high strength steels were not used in the construction of the Pikes Peak Reservoir. The allowable static hoop stress is 15,000 psi and the allowable seismic hoop stress is 17,000 psi. The allowable compressive stress is 1,778 psi.

As shown in Table 3-3 of this report, the reservoir is not anchored to the ringwall footing. However, some anchorage may be required between the reservoir and the ring footing to resist overturning.

The allowable soil bearing pressure based on the results of the soil investigation for transient loading conditions such as seismic forces is 7.5ksf. The actual bearing pressure for a 0.4g seismic event is 5.3 ksf and for a 0.3g seismic event is 3.9 ksf.

In accordance with AWWA D-100, resistance to the overturning moment at the bottom of the shell may be provided by the weight of the tank shell, weight of roof reaction on the shell and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks or by anchorage of the tank shell. For unanchored tanks, such as the Pikes Peak Reservoir, the portion of the contents that may be used to resist overturning is dependent on the width of the bottom annulus. The annulus may be thought of as a portion of the bottom plate that lifts off the foundation and supports the weight of the tank contents.

There are three criteria to determine the degree of overturning and consequently the amount of anchorage required for the reservoir. If the ratio of the overturning moment divided by the product of the diameter of the reservoir and the weight of the tank shell, roof and contents is less than or equal to 0.785, then there is no uplift and no anchorage is required. If this ratio is between 0.785 and 1.54 then there is uplift but no anchorage is required. However, since there is uplift, anything that is within the annular ring or outside the reservoir and connected to the reservoir must be allowed to move, either with the reservoir or relative to the reservoir. For example, piping which is connected to the outside of the shell plate or within the annular ring on the outside of the reservoir must be flexible enough to withstand the movement. If the ratio is greater than of the reservoir must either be anchored or the bottom annulus must be thickened to support more of the tank contents. For the Pikes Peak reservoir, this ratio is 0.96 and 0.72 for a 0.4g and 0.3g earthquake respectively. These ratios are less than 1.54 so the reservoir is not required to be anchored to the ring footing or have the annular ring thickened. However, the flexibility of piping connections to the reservoir must be addressed for a 0.4g earthquake.

ALTERNATIVE SEISMIC RETROFITS

As noted in the "Seismic Forces" subsection the Pikes Peak Reservoir does not require anchorage to resist overturning for a 0.4g or a 0.3g earthquake and the actual soil bearing pressure is less than the allowable soil bearing pressure. However, according to the psuedodynamic analysis, there is uplift for a 0.4g earthquake. Therefore, two alternatives were considered to seismically retrofit the reservoir for a 0.4g seismic event.

Alternative 1 is to use of small diameter to high strength earth anchors to prevent uplift. The anchors would be drilled through the existing ring footing on equal spaces on the curbside of the reservoir as previously shown in Figure 4-4. The anchors would then be connected to the shell plate by anchor chairs.

Alternative 2 would be to modify the piping connections so they allow movement relative to the reservoir. The 12-inch overflow pipe and the 8-inch drain pipe appear to have sufficient flexibility already built in to allow sufficient movement and no modification would be required. The 16-inch inlet/outlet would require installation of a vertical sleeve coupling in the vertical section of pipe on the outside of the reservoir just above the ground surface.

COMPARISON OF ALTERNATIVES

Alternative 1 does not appear to be an adequate solution to retrofit the reservoir. The width of the ring footing is only 12 inches and the reservoir shell plate is centered on the footing. Therefore, it does not appear that there is adequate space to drill even a small-diameter earth anchor through the footing on the outside of the tank wall. For Alternative 2, add flexible couplings to the connection of the 12-inch overflow piping where the pipe goes underground on the outside of the tank. The approximate cost for Alternative 2 is estimated to be $35,000. For a description of what is included in the estimated costs. See Table 11-1.

Section 10

MONTGOMERY WATSON

Section 10
Somerset No. 1

The Somerset No. 1 reservoir is a 100,000 gallon reinforced concrete reservoir located in a residential area near the Somerset Recreation Club. It was designed by Harstad & Associates in 1961, and is in part constructed of precast concrete panels. This section presents the findings of the site investigation, seismic assessment of the tank's current condition, and an evaluation of the piping connections to the tank. In addition, alternative seismic retrofits, including a cost comparison, have been developed for the Somerset No. 1 reservoir. The evaluation of the alternatives and a recommended seismic retrofit alternative are also presented in this section. Photographs of the Somerset No. 1 reservoir are included in Appendix B.

SITE INVESTIGATION

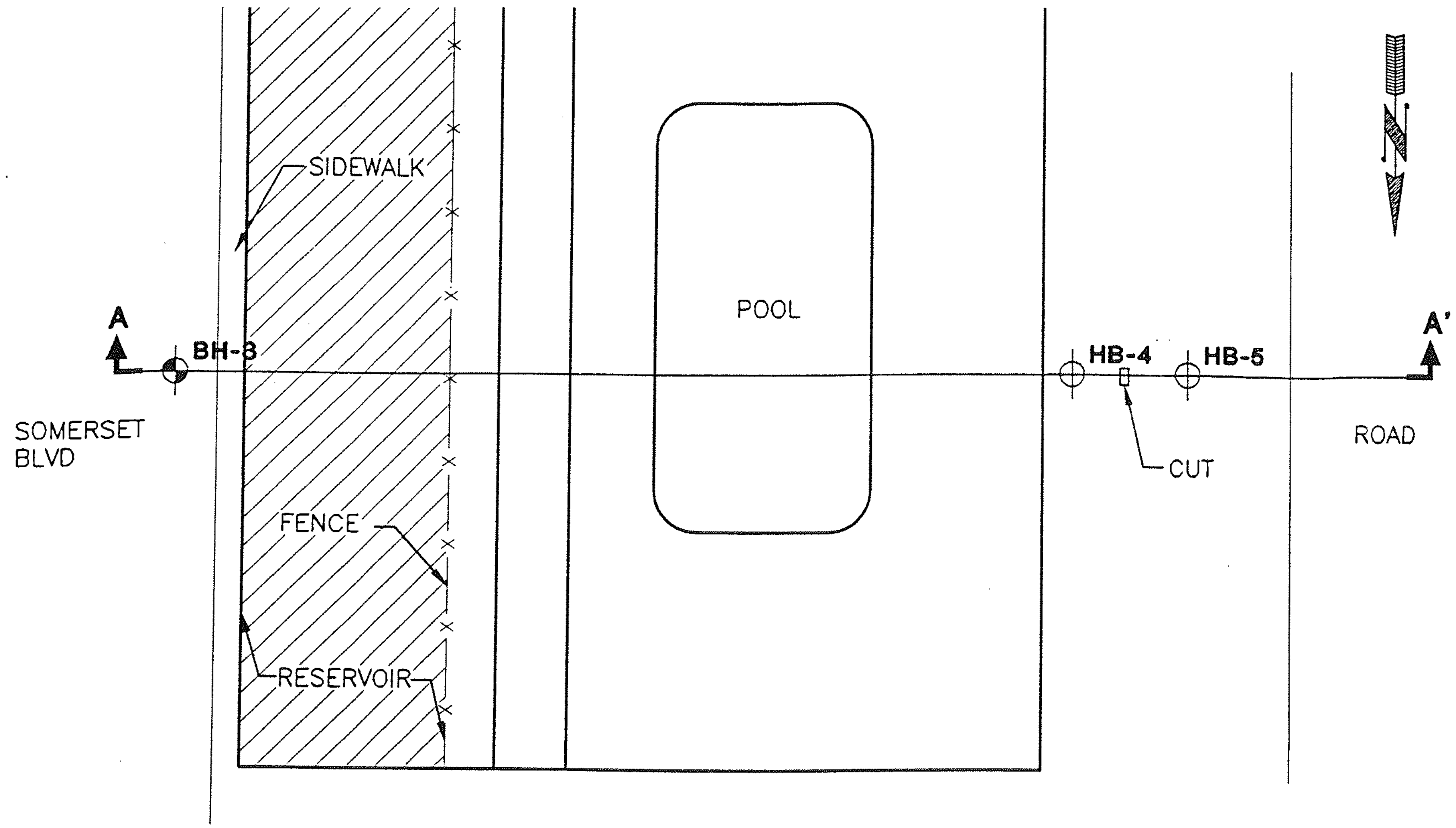
Site investigations for the Somerset No. 1 reservoir have included on-site inspections when the reservoir was full. and empty. Site inspections have been completed by the structural engineer (Montgomery Watson), the geotechnical engineer (HWA GeoSciences), and the corrosion engineer (Corrpro Companies, Inc.). The following paragraphs summarize the findings of these site investigations. Detailed field notes and the geotechnical report resulting from these investigations is included in Volume II of this preliminary design report.

Site Characteristics

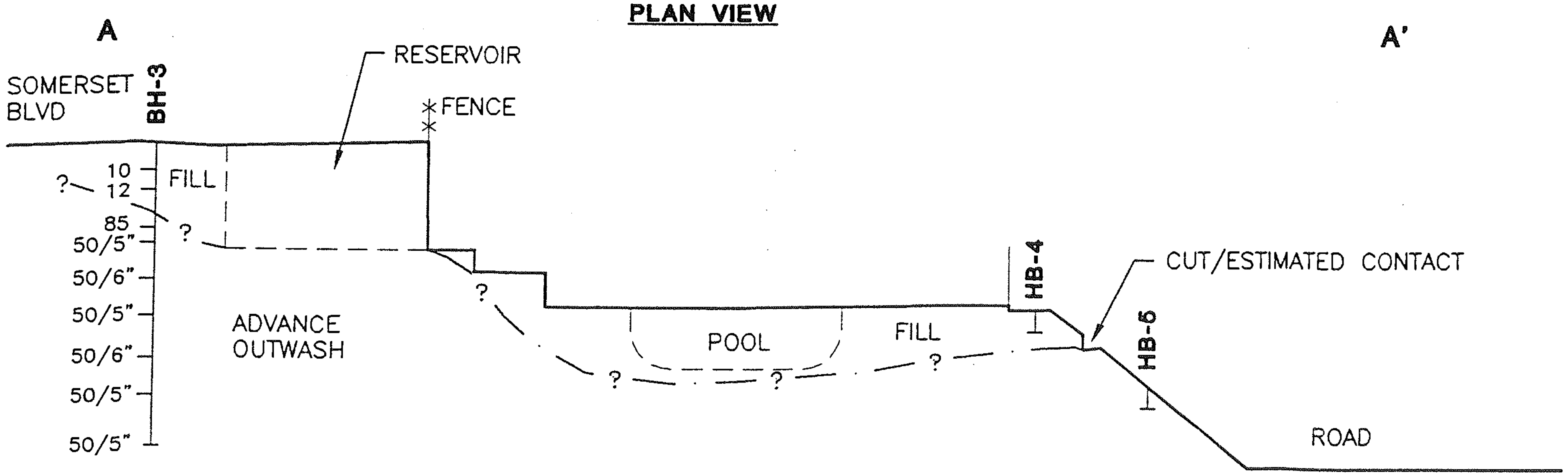
The Somerset No. 1 reservoir site profile and plan are shown on Figure 10-1. The reservoir's interior is approximately 10-feet high by, 16-feet wide and 80-feet long. The structure is founded on an 8-inch thick, reinforced concrete mat. A pump station occupies the northern-most section of the concrete vault. Based on the surrounding site topography, the original Somerset No. 1 reservoir site sloped downward from east to west. The existing concrete tank and pump station were constructed by excavating into the original slope and creating a bench. The east side of the tank is tucked into the slope, and the west side daylights.

Based on the data reviewed and subsurface investigations, the Somerset No. 1 reservoir site appears to be underlain by advance outwash, consisting of medium dense to very dense silty sand that is partially cemented. The advance outwash is overlain by approximately 6 feet of fill consisting of loose to dense, silty sand with varying gravel content. Based on the 1961 plans, the existing reservoir is founded on the advance outwash, with fill behind the below-grade walls of the reservoir.

Groundwater was not encountered during the subsurface explorations. However, groundwater seepage outcropping from the slope along the east side of the Somerset Recreation Club parking lot was observed during the site visit. Discussions with the Recreation Club personnel confirm that seepage is fairly continual at the toe of slope adjacent to the reservoir.



PLAN VIEW



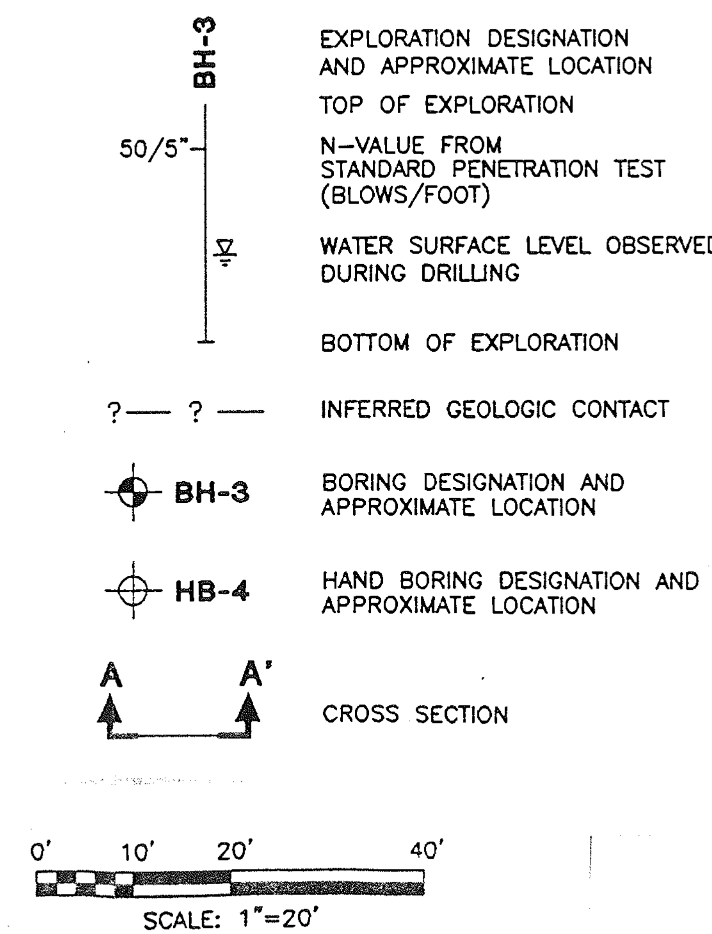
PROFILE VIEW - A-A'

REFERENCE: Plan and Profile prepared by Hong HWA GeoSciences, Inc.

NOTE

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate.
2. The profile view A–A' was developed using a hand–level, rod and tape and should be considered approximate.

LEGEND



MONTGOMERY WATSON

City of Bellevue
Somerset No. 1 Reservoir
Site Plan and Subsurface Exploration Plan
FIGURE 10-1

General Reservoir Condition

The investigation of the interior of the reservoir revealed the concrete in the precast wall panels is in poor condition. In particular, fine aggregate can be hand rubbed away. There is extensive cracking throughout the floor slab that appears to have been repaired over the life of the reservoir. It also appears that all the joints between the precast wall panels have recently been repaired. The perimeter joint between the floor slab and foundation is in fairly good condition.

The reservoir outlet, overflow, and drain piping are all badly corroded, and should, at minimum, be cleaned and painted. The concrete adjacent to most of the diaphragm connections has spalled. The galvanized steel ladder located in the reservoirs interior is in poor condition and should be replaced.

Corrosion Inspection

Corrpro Companies, Inc. completed a corrosion investigation of the Somerset No. 1 concrete water tank. Data collected included visual inspection of interior and exterior and steel appurtenances.

Visual inspection formed the basis of this investigation. City of Bellevue personnel drained the tank to allow access to the floor and walls during the internal inspection. The inspection included qualitative evaluation of concrete for cracks, spalling and other failures, evaluation of tank appurtenances and sounding of concrete. Sounding is a technique that involves light tapping of the concrete in discreet areas. The operator listens for hollow areas or areas of dead sound. These areas locate possible concrete delamination.

The visual inspection revealed no signs of corrosion of the tank's steel reinforcing bars. Such attack, if advanced, would manifest itself as spalled concrete or red iron oxide staining coming from concrete cracks.

Stress cracks were visible across the floor. Vertical cracks were apparent in the stressed sections of the prefabricated concrete panels. The cracks were discerned because of a phenomenon known as effervescence. Effervescence occurs when calcium compounds migrate from, the bulk concrete material to surface at cracks. A field of stalactites form along the length of the crack. Observation of effervescence allows identification of microcracking that would be otherwise too small to see. Effervescence was noted at the roof near station 6E and near the tank entryway.

Cursory sounding testing revealed no delamination. However, the procedure, which requires light tapping of the concrete with a hammer, did dislodge small concrete chips. This test method does not typically generate enough force to destructively remove pieces of concrete. The test results suggest that the concrete has softened somewhat over its service life. This is not surprising given the age of the tank.

These degradations of the concrete could be slowed through the use of protective coatings. The tank would need to be drained and cleaned. Surface preparation would include a brush blast to loosen concrete and the application of two coats of epoxy or epoxy novolac system.

The tank appurtenances were heavily corroded. Steel pipe fittings show thick build up of iron oxide corrosion product. In addition the stainless steel ladder is accelerating the corrosion of carbon steel bolts used to attach it. Some crevice type corrosion is also evident on the stainless steel.

RESERVOIR PIPING CONNECTIONS

The piping connections at the Somerset No. 1 reservoir consist of the following:

- 8-inch overflow piping penetrates the tank wall near the top of the tank wall. The overflow line is rigidly attached to the reservoir wall and floor below the adjoining pump station.
- 6-inch drain piping penetrates the reservoir floor.
- 10-inch inlet piping penetrates the reservoir wall just above the reservoir floor. The inlet piping also passes through the reservoir floor of the adjoining pump station.

SEISMIC ASSESSMENT OF CURRENT TANK CONDITIONS

Seismic design parameters for the Somerset No. 1 reservoir site were determined as described in Section 2. Table 10-1 summarizes the seismic coefficients to be used for the evaluations consistent with Seismic Zones 3 and 4.

Table 10-1
Somerset No. 1 Reservoir
Seismic Coefficients

Seismic Zone	Soil Profile Type	Near Source Factor, N_v	Near Source Factor, N_a	Seismic Coefficient C_a	Seismic Coefficient C_v	Control Period T_o	Control Period T_s
3	S_c	n/a	n/a	0.33	0.45	0.11	0.55
4	S_c	1.3	1.6	0.52	0.90	0.14	0.69

Based on results of the soil investigation performed by HWA GeoSciences, it is recommended that the existing mat foundation be evaluated using an allowable soil bearing pressure of 6 ksf (kips per square foot) for static conditions. This value may be increased by 50 percent for evaluating transient loading conditions, such as seismic forces.

The existing reservoir retaining wall may be evaluated using either an at-rest or an active earth pressure. The at-rest earth pressures generally provide a conservative structural design such that temporary seismic overloading conditions can readily be accommodated in the structure considering the factors of safety that are normally used in the structural design for static conditions. Active earth pressures against below-grade walls can be modeled using an equivalent, horizontal fluid pressure of 33 pcf (pounds per cubic foot) for static conditions. For

Zone 3 analysis, the incremental earthquake induced loading should be modeled with a uniform, rectangularly-distributed, horizontal pressure of 12H pounds per square foot, where H is the height of the below-grade wall. For Zone 4 analysis, the earthquake component of the active earth pressure should be increased to 16H pounds per square foot.

If at-rest earth pressures are used for the analyses, an equivalent horizontal fluid pressure of 51 pounds per cubic foot is recommended. It is the opinion of HWA GeoSciences that the increased lateral earth pressures resulting from an earthquake can be neglected if the below-grade walls are evaluated using the at-rest earth pressures.

Frictional resistance along the base of the footings may be evaluated using 0.60 for the coefficient of base friction. It is estimated the full base friction force will be mobilized within about 0.25 inch of lateral movement. Passive resistance against the sides of the footings may be evaluated using an equivalent fluid density of 800 pcf (pounds per cubic foot). Suitable factors of safety should be incorporated in evaluating lateral resistance of the ring wall footing.

Soil liquefaction can occur when saturated, loose sands and silty sands lose strength and behave as a liquid in response to earthquake shaking. Because the soils at the Somerset No. 1 reservoir site are neither saturated or loose, the potential for soil liquefaction is low. Further, the anticipated infrequent recurrence coupled with no previous evidence of surface rupture associated with the Seattle Fault indicates that the risk of ground rupture at the site is low.

The slope of the site is relatively stable and poses no significant threat to the reservoir during an earthquake. Some continued raveling of the slope and deposition of sloughing soil at the toe of the slope should be anticipated. Control of surface water at the top of the slope can limit the extent of surficial erosion.

SEISMIC FORCES

At the time of this investigation construction drawings were not available for review to determine the structure of the reservoir. Therefore, little is known about the specifics of the actual construction or original design criteria for the reservoir and it was determined that seismic retrofit alternatives should be investigated which would resist most of the anticipated seismic loads and do not rely on the strength of the existing structure. The basic structure, as determined by field observation, is a cast-in-place concrete floor slab, precast concrete wall panels, precast tee roof beams and a cast-in-place concrete roof slab.

As noted in the "Site Characteristics" paragraph of this Section, the reservoir was constructed by excavating into the original slope of the site creating a bench where the east side of the tank is tucked into the slope and the west side of the tank daylights. Lateral earth pressures during a seismic event resulting from having higher soil on the east side of the reservoir consist of the active earth pressure, surcharge from traffic on the road on the east of the reservoir, and an incremental earthquake loading. These anticipated HWA GeoSciences, Inc summarizes loads in the geotechnical reports for Somerset No. 1. Other anticipated lateral forces during a seismic event come from the weight of the reservoir structure, the weight of its contents and the effect of sloshing of the tank contents.

During preliminary seismic analysis it was discovered that the lateral forces during a seismic event resulting from lateral earth pressures are larger than the lateral forces resulting from the weight of the reservoir, its contents and sloshing. The original design criteria for the reservoir should have included lateral forces due to lateral earth pressure. Therefore, it is reasonable to assume that a seismic retrofit alternative that reinforces the existing structure to resist lateral earth pressures should be adequate and that the reservoir structure can resist the weight of the structure, its contents and sloshing. Therefore, the retrofit alternatives considered, were assumed to resist only lateral earth pressure during a seismic event.

ALTERNATIVE SEISMIC RETROFITS

Alternative 1 is to build a reinforced concrete cantilever retaining wall on the east side of the reservoir. As previously noted, the retaining wall would be designed to resist active earth pressure, surcharge due to vehicular traffic from the road on the east side of the reservoir and earthquake induced loading for a both 0.4g and 0.3 earthquake.

Alternative 2 is to use a system of tiebacks to reinforce the precast concrete wall panels on the east side of the reservoir for a 0.4g and a 0.3g earthquake. The system would be a series of small diameter, pressure grouted tiebacks designed to resist all of the lateral earth pressures. The tiebacks would be drilled horizontally through the wall into loose fill and dense advance outwash soils that are under the road on the east side of the wall. Each tieback would then be anchored to a block of reinforced concrete that is doweled into the precast wall panel.

Alternative 3 is to empty the reservoir and abandon its use for water storage. In Alternative 3 a system of tiebacks would still be required, even though the reservoir would be empty, to reinforce the wall panels on the east side of the reservoir for a 0.4g and a 0.3g. Therefore, the tiebacks would be designed to resist only the lateral forces due to soil pressure from an incremental earthquake loading. The reservoir structure would be assumed to resist the weight of the reservoir and lateral earth pressure due to active pressure and surcharge.

COMPARISON OF ALTERNATIVES

As shown in Table 3-2, the storage capacity of the Somerset No. 1 Reservoir is only about 0.1 million gallons. In addition, since there are no construction drawings or shop drawings available for the reservoir, very little is known about the actual construction or the capacity of the existing reservoir to resist forced imposed by a seismic event.

Alternative 1 requires an approximately 16 foot high, 1.5-foot thick retaining wall with a 14-foot wide, 1.5-foot footing. The retaining wall would be continuous along the length of the reservoir. Alternative 2 requires approximately 20 tiebacks, 25- to 30-feet long, located vertically at the quarter points of each panel. In addition, each tieback would be anchored to 3-foot by 3-foot cast-in-place concrete block that is doweled in to the precast panel. Alternative 3 requires approximately 10 tiebacks, 25- to 30-feet long, located vertically at the mid-point of the precast panel and similar concrete anchor blocks.

The estimated costs for Alternative 1, 2 and 3 are shown in Table 10-4 below. Since the difference in total lateral earth pressure is small for a 0.4 and a 0.3 earthquake, the estimate cost is assumed to be approximately the same. Therefore, only one cost is shown for each Alternative. For the description of what is included in the estimated costs see Table 11 - 1.

Table 10-4
Somerset No.1 Reservoir
Summary of Estimated Costs

Alternative	Cost
Alternative 1	$190,000
Alternative 2	$130,000
Alternative 3	$80,000

As can be seen in Table 10-4, Alternatives 1 and 2, which keep the reservoir in service, are very expensive retrofit alternatives 'when the small storage capacity of the reservoir is taken into consideration. Alternative 3, which takes the reservoir out of service, is also rather expensive. However, the reservoir is located adjacent to a roadway and above the pool at Somerset Recreation Club. Therefore, if the reservoir is left in place, it should be retrofitted even if taken out of service. It should also be noted that, even if the reservoir is abandoned and removed, a retaining wall similar to the one in Alternative 1 will still be required to support the roadway east of the reservoir.

Therefore, if the storage capacity of this reservoir is not essential to the City, it is recommended that the reservoir be taken out of service and retrofitted in accordance with Alternative 3. However, it is also recommended that the City review other distribution improvements, where this storage could be combined with other planned facilities. Finally, it is recommended that the City review other possible new sites for this storage.

The estimated costs for Alternative 1, 2 and 3 are shown in Table 10-4 below. Since the difference in total lateral earth pressure is small for a 0.4 and a 0.3 earthquake, the estimate cost is assumed to be approximately the same. Therefore, only one cost is shown for each Alternative. For the description of what is included in the estimated costs see Table 11 - 1.

Table 10-4
Somerset No.1 Reservoir
Summary of Estimated Costs

Alternative	Cost
Alternative 1	$185,000
Alternative 2	$115,000
Alternative 3	$70,000

As can be seen in Table 10-4, Alternatives 1 and 2, which keep the reservoir in service, are very expensive retrofit alternatives 'when the small storage capacity of the reservoir is taken into consideration. Alternative 3, which takes the reservoir out of service, is also rather expensive. However, the reservoir is located adjacent to a roadway and above the pool at Somerset Recreation Club. Therefore, if the reservoir is left in place, it should be retrofitted even if taken out of service. It should also be noted that, even if the reservoir is abandoned and removed, a retaining wall similar to the one in Alternative 1 will still be required to support the roadway east of the reservoir.

Therefore, if the storage capacity of this reservoir is not essential to the City, it is recommended that the reservoir be taken out of service and retrofitted in accordance with Alternative 3. However, it is also recommended that the City review other distribution improvements, where this storage could be combined with other planned facilities. Finally, it is recommended that the City review other possible new sites for this storage.

Section 11

MONTGOMERY WATSON

Section 11
Conclusions and Recommendations

This section of the report summarizes the conclusions and recommendations for the seismic/structural evaluation of seven reservoirs located in the City of Bellevue. The seven reservoirs include six steel tanks and one concrete tank. The steel tanks evaluated in this study include: Lake Hills North, Lake Hills South, Pikes Peak, Parksite, Woodridge, and Horizon View No. 1. Somerset No. 1 is the concrete tank evaluated as part of this study.

CONCLUSIONS

The structures are for the most part founded on glacial till which is referred to as hard pan. In general, the structure of the reservoirs, including piping and appurtenances, appears to be in good condition with little corrosion. The interior and exterior coating systems of the reservoirs show little visual signs of failure although adhesion tests were sometimes inconclusive. In some cases, the exterior coating systems showed significant levels of leachable leads. Cathodic protection systems, where they exist remain in good condition and should prevent significant future corrosion.

Seismic analysis and evaluation revealed that the steel reservoirs in general do not meet current requirements shown in AWWA D100, particularly as it relates to overturning of the reservoirs. The reservoirs fell into three basic categories of upgrade requirements to meet AWWA D-100 Standards.

First, because they have small diameter to width ratios, the Lake Hills North, Lake Hills South, Woodridge, and Horizon View No. I reservoirs have significant overturning problems. Therefore, they require extensive seismic improvements, which include additional anchorage.

Second, since the Parksite and Pikes Peak reservoirs have relatively large diameter to width ratios, they have few problems with overturning. Improvements for these reservoirs involve either minimal anchorage or modification to piping connections to allow them to move relative to the reservoir during a seismic event.

The third category is the Somerset No. 1 reservoir. This reservoir requires extensive improvements to reinforce it to withstand a seismic event. Since the storage of this reservoir is very small, the cost to make such improvements could be nearly equal to the cost to replace the storage elsewhere in the City's water distribution system. The City should review other district system improvements where this storage could be combined with other planned facilities. In addition, the City should review other possible new sites for this storage.

RECOMMENDATIONS

The following recommendations are organized by reservoir. More detailed information for each reservoir is presented in the previous sections of this report. The estimated costs for the recommended improvements are summarized in Table 11 - 1.

Lake Hills North

- Repair the exterior roof plate in the cupola adjacent to the roof hatch and add drain holes at the bottom of the cupola to prevent ponding.
- Adjust the output of the cathodic protection system in accordance with the National Association of Corrosion Engineers criteria.
- Use earth anchors (Alternative 3, Case 1) to fully anchor the reservoir as described in Section 4.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.
- Install a new altitude valve for the reservoir.
- Review and make recommendations for piping modifications at the pump station south of the reservoir.

Lake Hills South

- Adjust the output of the cathodic protection system in accordance with the National Association of Corrosion Engineers criteria.
- Use earth anchors (Alternative 3, Case 1) to fully anchor the reservoir as described in Section 5.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.
- Repair the gouged shell plate on the east side of the reservoir and place bollards adjacent to the repaired area to prevent future damage.

Woodridge

- Adjust the output of the cathodic protection system in accordance with the National Association of Corrosion Engineers criteria.
- Use earth anchors (Alternative 3, Case 1) to fully anchor the reservoir as described in Section 6.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.

Horizon View

- Adjust the output of the cathodic protection system in accordance with the National Association of Corrosion Engineers criteria.
- Use earth anchors (Alternative 3, Case 1) to fully anchor the reservoir as described in Section 7.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.

Parksite

- Replace the existing interceptor drainage system along the toe of the existing slope.

- Adjust the output if the cathodic protection system in accordance with the National Association of Corrosion Engineers criteria.
- Use earth anchors (Alternative 3, Case I or Case 2) as described in Section 8.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.
- Remove existing 16-inch A.C. inlet pipe between 16-inch C.I. adapter and the park booster station and replace with ductile iron pipe.

Pikes Peak

- Add flexible piping connections as described in Section 9 to seismically retrofit the reservoir.
- Add cathodic protection to the reservoir.

Somerset

- If the storage capacity is not essential to the overall system it is recommended that the reservoir be abandoned and that the reservoir be retrofitted as described in Section 10 (Alternative 3) using tiebacks to reinforce the wall panels on the east side of the reservoir.
- The City should review other district system improvements where this storage could be combined with other planned facilities.
- The City should also review other possible new sites for the storage.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.
- If the reservoir is abandoned, add a pressure reducing valve station at another point in the system to replace the one at the reservoir.

- Adjust the output of the cathodic protection system in accordance with the National Association of Corrosion Engineers criteria.
- Use earth anchors (Alternative 3, Case I or Case 2) as described in Section 8.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.

Pikes Peak

- Add flexible piping connections as described in Section 9 to seismically retrofit the reservoir.
- Add cathodic protection to the reservoir.

Somerset

- If the storage capacity is not essential to the overall system it is recommended that the reservoir be abandoned and that the reservoir be retrofitted as described in Section 10 (Alternative 3) using tiebacks to reinforce the wall panels on the east side of the reservoir.
- The City should review other district system improvements where this storage could be combined with other planned facilities.
- The City should also review other possible new sites for the storage.
- Based upon the estimated costs for structural/seismic improvements, the City should determine whether to design for Zone 3 or Zone 4.

Horizon View #2

Table 11-1
Summary of Estimated Costs
Structural/Seismic Upgrades

Reservoir	Alternative Description	Estimated Cost			
		Zone 3-Retrofit	Zone 4-Retrofit	New Reservoir	Expected Useful Life
Lake Hills North	Earth anchors with full overturning moment Earth anchors with reduced overturning moment	$360,000 $345,000	$470,000 $455,000	$1,000,000	30-40 years
Lake Hills South	Earth anchors with full overturning moment Earth anchors with reduced overturning moment	$320,000 $305,000	$435,000 $420,000	$1,000,000	30-40 years
Woodridge	Earth anchors with full overturning moment Earth anchors with reduced overturning moment	$320,000 $305,000	$435,000 $420,000	$1,000,000	25-35 years
Horizon View No. 1	Earth anchors with full overturning moment Earth anchors with reduced overturning moment	$130,000 $150,000	$150,000 $170,000	$275,000	30-40 years
#1 becuz Parksite Park is being improved	Earth anchors Modify pipe connections only	$110,000 $60,000	$125,000 $60,000	$1,000,000	30-40 years
Pikes Peak	Modify pipe connections only	$35,000	$35,000	$700,000	35-45 years
Somerset No. 1	Add retaining wall Add tieback system Abandon reservoir + PRV station	$190,000 $130,000 $80,000 +35K	$190,000 $130,000 $80,000 +35K	$225,000	10-15 years

(a) Estimated costs include the cost of construction, sales tax at 8.6 percent, contingencies at 25 percent, and engineering and administration at 25 percent.
(b) Estimated costs are presented for design criteria to meet either Zone 3 or Zone 4 of the UBC.
(c) Estimated costs were developed using the November 1998 ENR for Seattle (No. 6561).
(d) Improvements for non-seismic repairs for items such as water quality, operations and safety should be addressed for each reservoir during the final design for seismic retrofit. These improvements should recognize that the Washington Department of Heath utilized the 10 State Standards as a basis for review and approval of water works projects. They should also include any recent changes to the new 1996 AWWA D-100 and the current OSHA standards for such items as access hatches, ladders, safety posts, etc. Finally improvements to circulation and the reduction of short circuiting should also be considered. The costs for such improvements has not been included in Table 11-1 but can be expected to be approximately $50,000 to $75,000 per reservoir.
(e) Estimated construction costs for new reservoirs are taken from median values of past bids for steel reservoirs. They are for the reservoir structure and include such items as removal of the existing reservoirs, reservoir foundation, excavation, piping, etc.

Table 11-1
Summary of Estimated Costs
Structural/Seismic Upgrades

the Tolt Supply Pipe is only designed to Zone 3 → if have Zone 4 earthquake need storage because won't have supply

Reservoir	Alternative Description	Estimated Cost			
		Zone 3-Retrofit	Zone 4-Retrofit	New Reservoir	Expected Useful Life w/out Repairs
Lake Hills North	Earth anchors with full overturning moment	$293,000	$370,000	$750,000	25 years
	Earth anchors with reduced overturning moment	$288,000	$365,000		
Lake Hills South	Earth anchors with full overturning moment	$238,000	$316,000	$750,000	25 years
	Earth anchors with reduced overturning moment	$233,000	$311,000		
Woodridge	Earth anchors with full overturning moment	$245,000	$324,000	$750,000	20 years
	Earth anchors with reduced overturning moment	$240,000	$319,000		
Horizon View No. 1	Earth anchors with full overturning moment	$113,000	$131,000	$200,000	25 years
	Earth anchors with reduced overturning moment	$125,000	$145,000		
Parksite	Earth anchors	$92,000	$104,000	$750,000	25 years
	Modify pipe connections only	$45,000	$45,000		
Pikes Peak	Modify pipe connections only	$35,000	$35,000	$500,000	30 years
Somerset No. 1	Add retaining wall	$185,000	$185,000	$150,000	<10 years
	Add tieback system	$115,000	$115,000		
	Abandon reservoir	$70,000	$70,000		

(a) Estimated costs include the cost of construction, sales tax at 8.6 percent, contingencies at 25 percent, and engineering and administration at 25 percent.
(b) Estimated costs are presented for design criteria to meet either Zone 3 or Zone 4 of the UBC.
(c) Estimated costs were developed using the November 1998 ENR for Seattle (No. 6961).
(d) Improvements for non-seismic repairs for items such as water quality, operations and safety should be addressed for each reservoir during the final design for seismic retrofit. These improvements should recognize that the Washington Department of Heath utilized the 10 State Standards as a basis for review and approval of water works projects. They should also include any recent changes to the new 1996 AWWA D-100 and the current OSHA standards for such items as access hatches, ladders, safety posts, etc. Finally, improvements to circulation and the reduction of short circuiting should also be considered. The costs for such improvements have not been included in Table 11-1 but can be expected to be approximately $50,000 to $75,000 per reservoir.
(e) Estimated construction costs for new reservoirs are taken from median values of past bids for steel reservoirs. They are for the reservoir structure only and do not include such items as removal of the existing reservoirs, excavation, piping, etc.

Appendix A

MONTGOMERY WATSON

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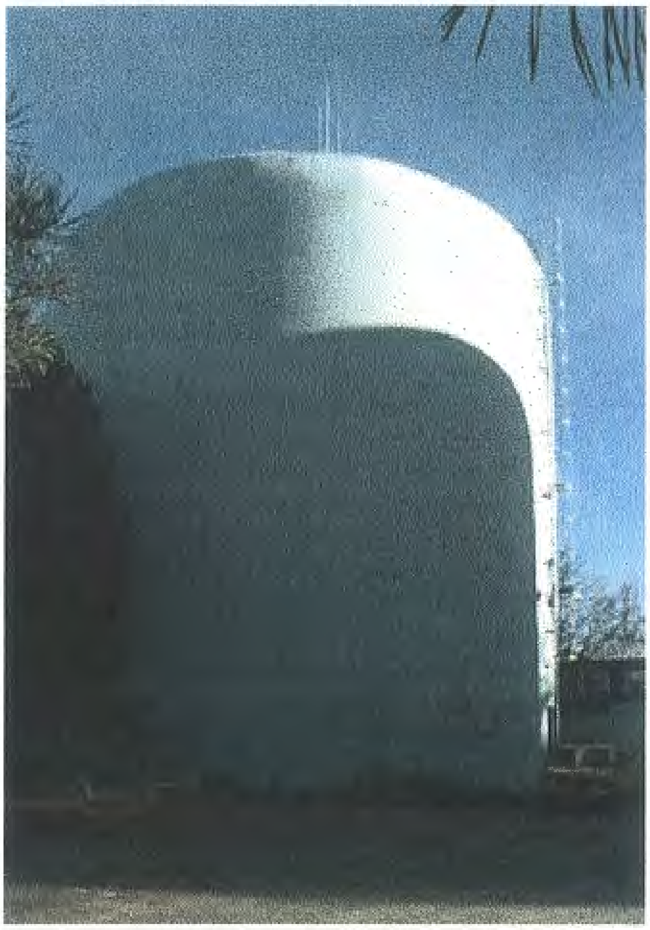
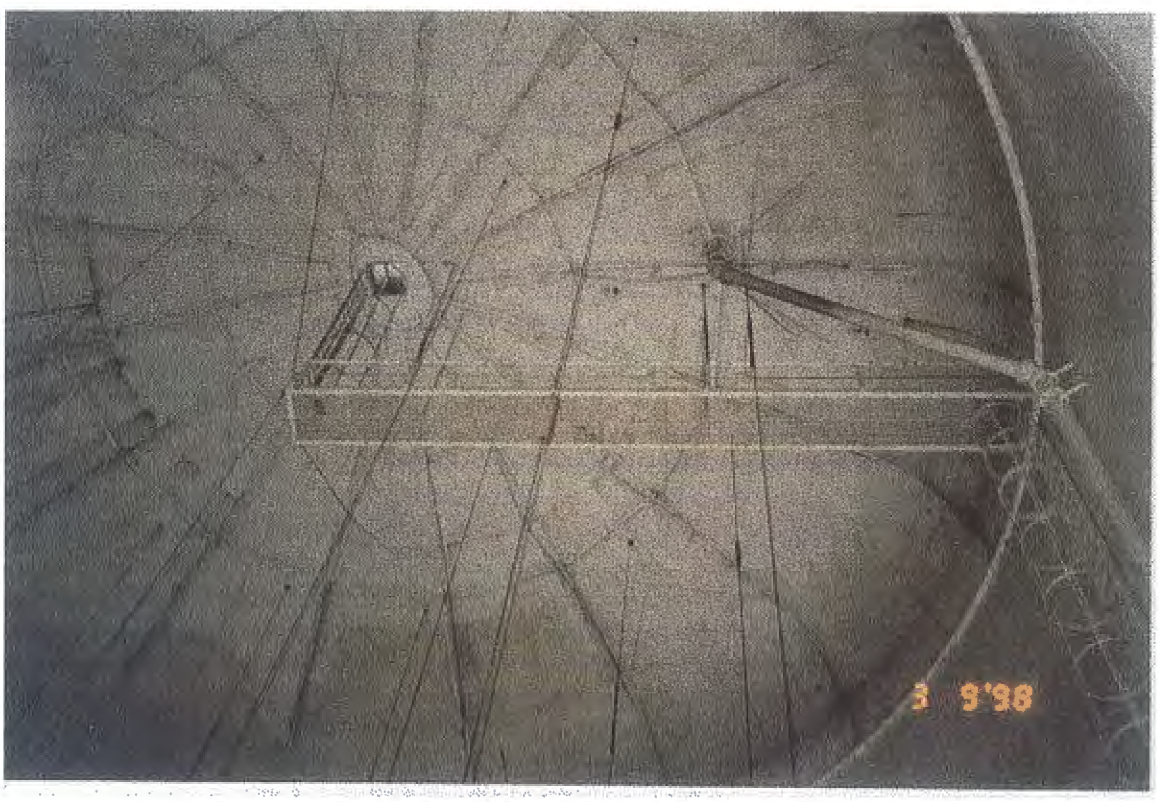
Appendix B

MONTGOMERY WATSON

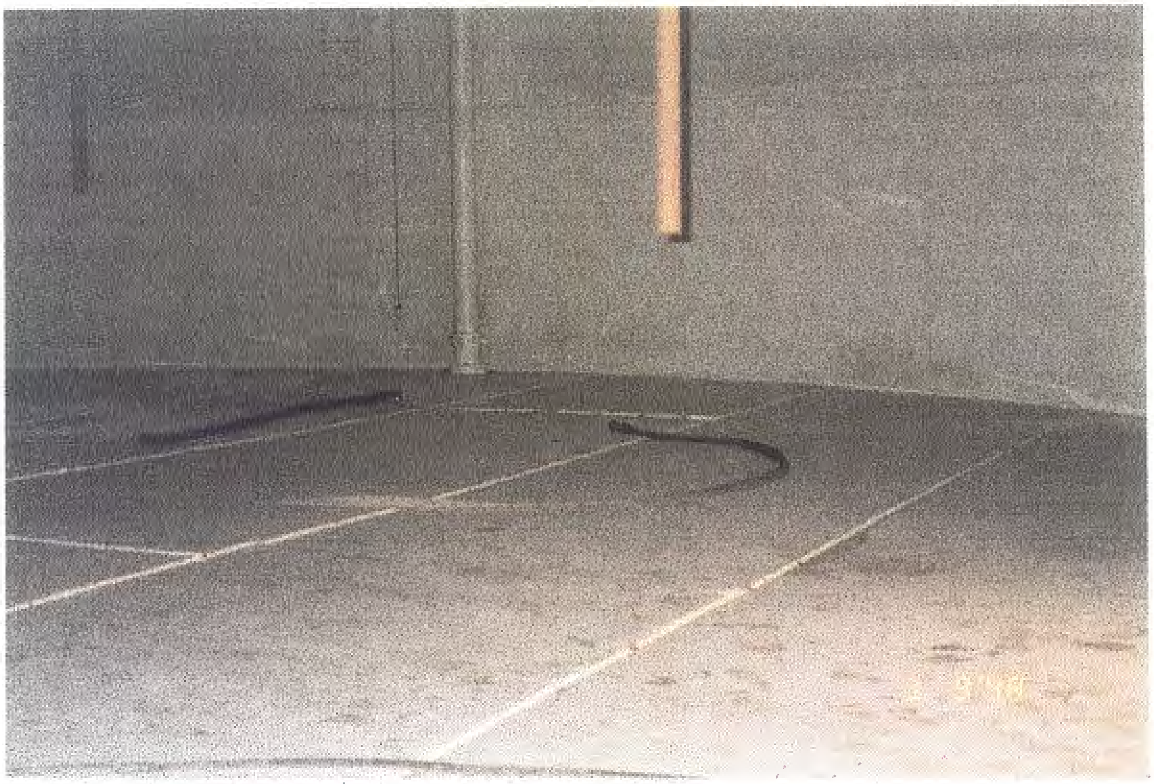


Lake Hills North - Reservoir

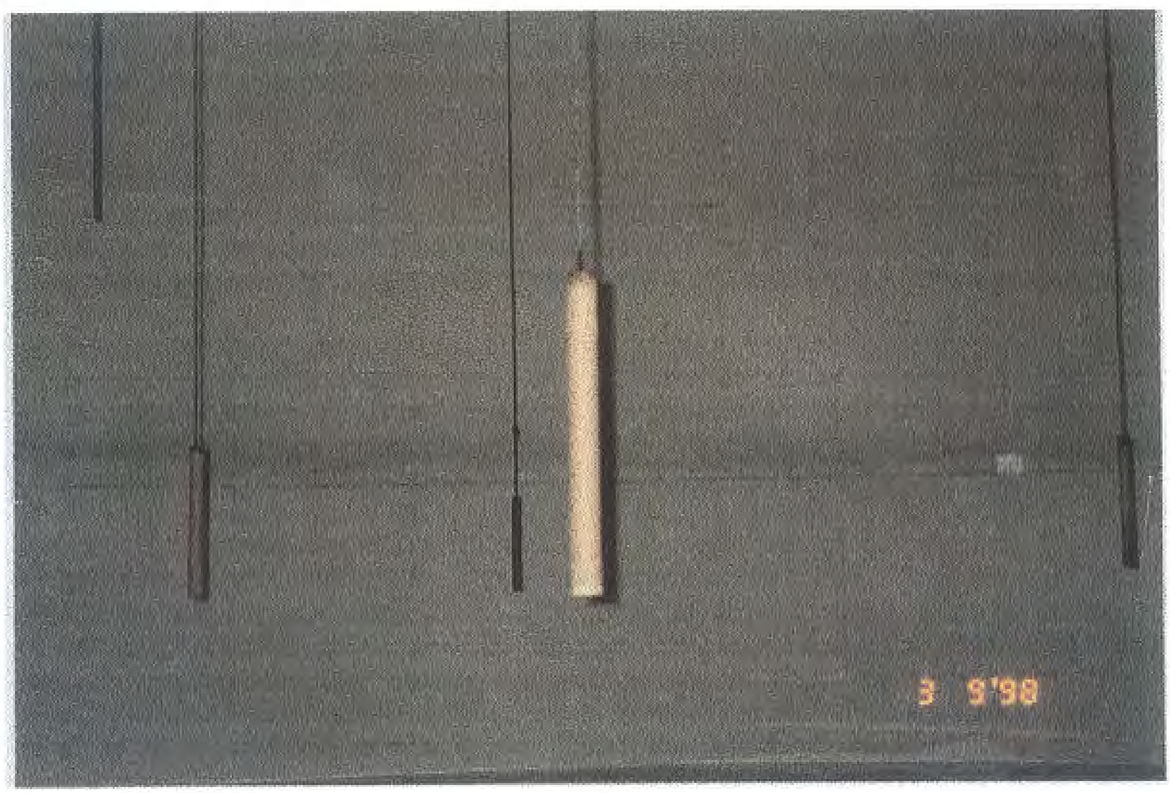
Lake Hills North - Wind Anchor



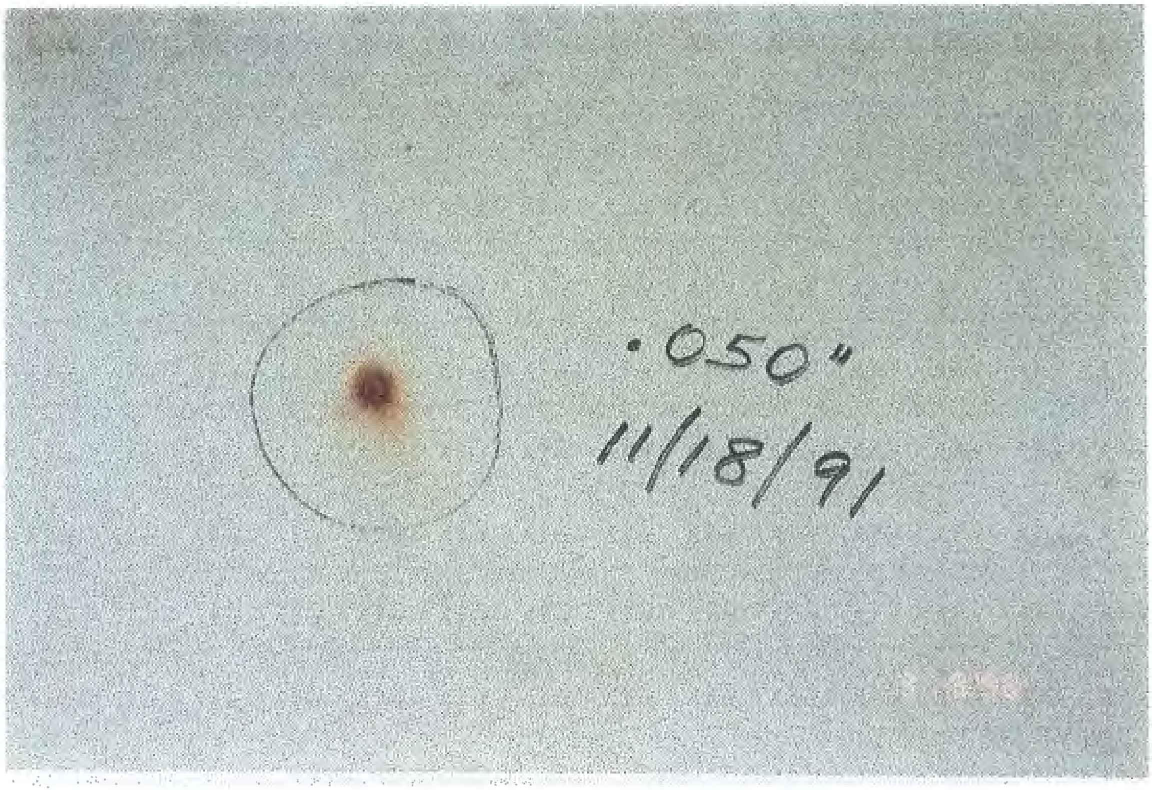
Lake Hills North - Catwalk



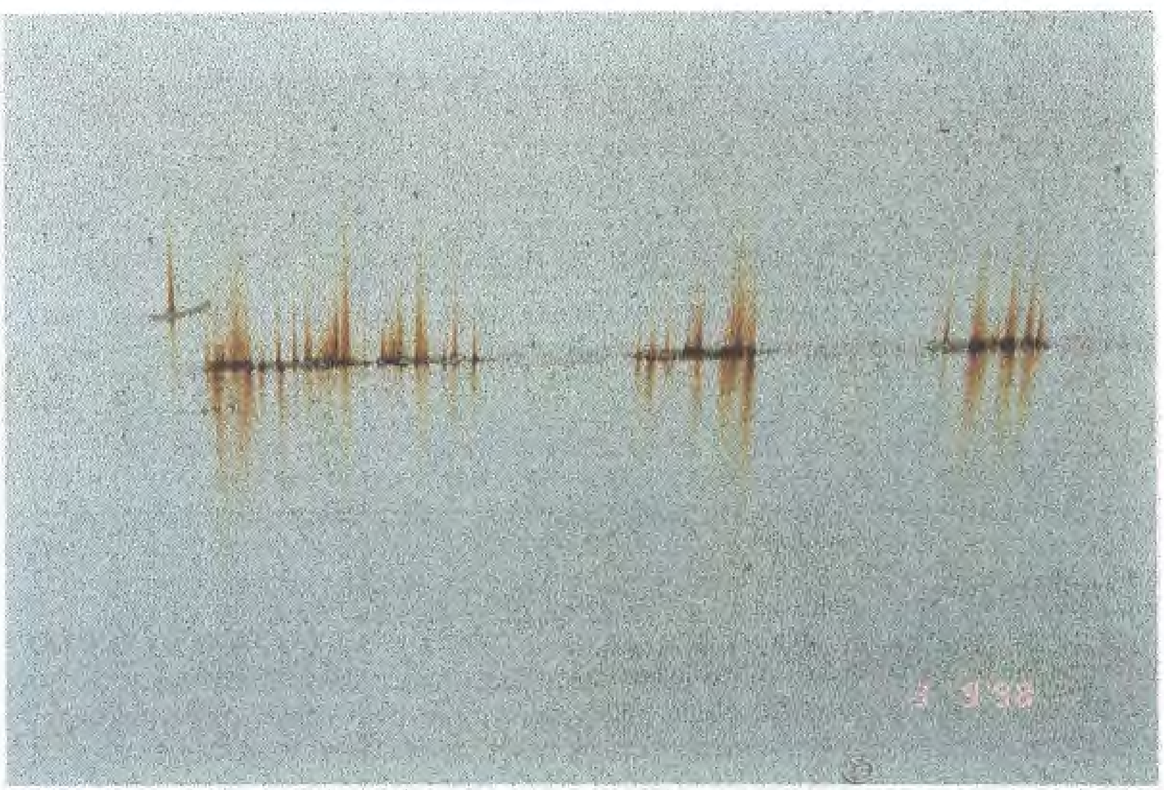
Lake Hills North - Overflow



Lake Hills North - Cathodic Protection



Lake Hills North - Pitting in Floor Plate



Lake Hills North - Pitting in Floor Plate

Lake Hills North - Corrosion Adjacent to Access Hatch

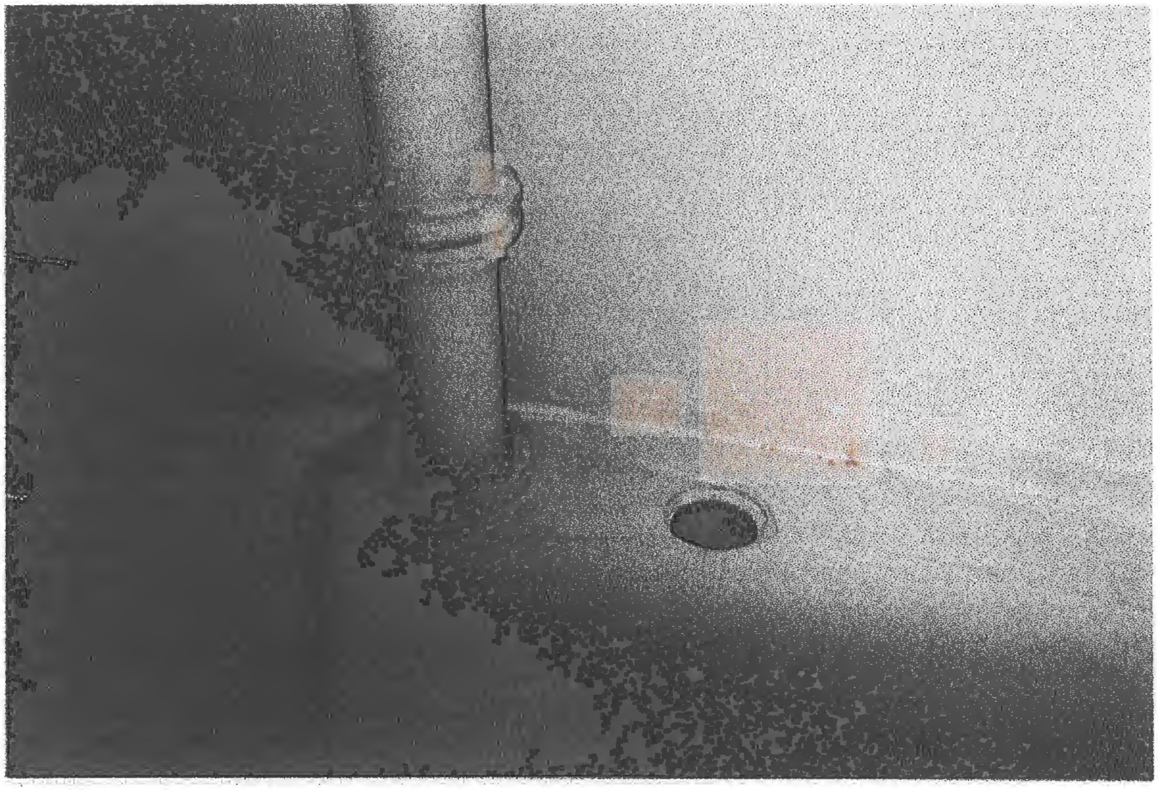
Woodridge Reservoir

Lake Hills South - Access Hatch

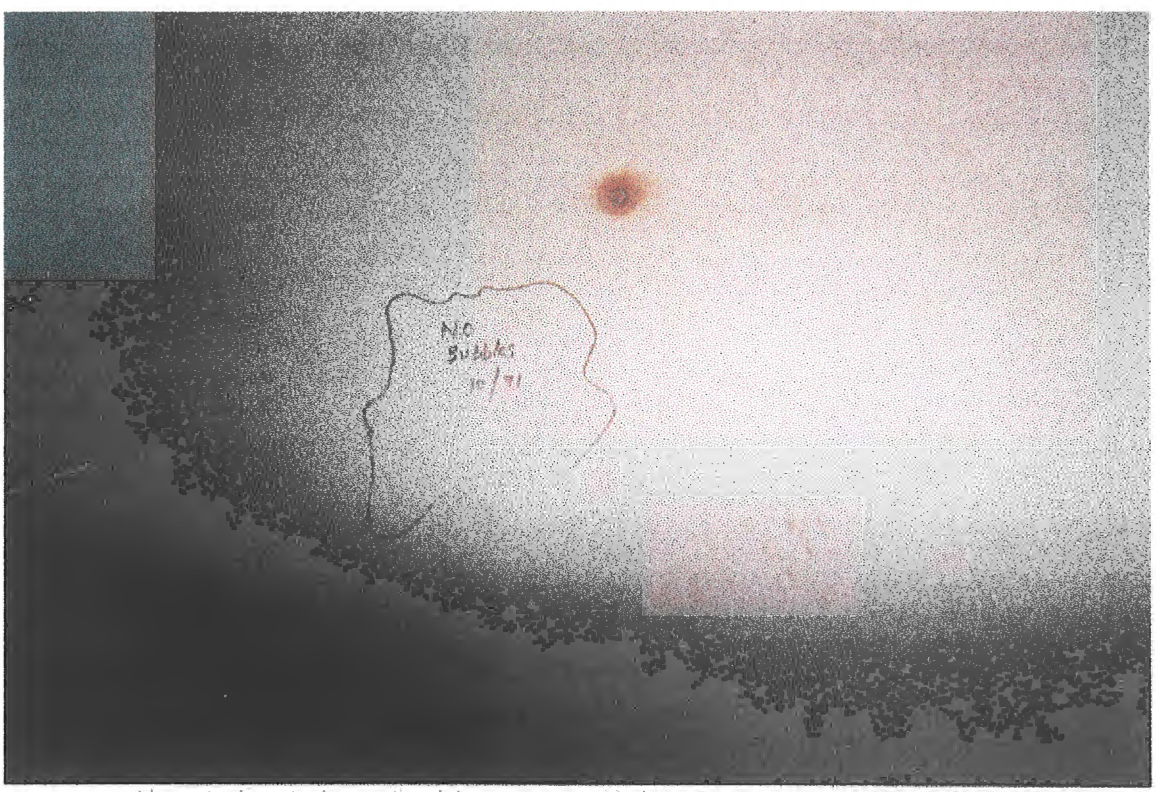
Lake Hills South - Access Ladder

Lake Hills South - Gouged Shell Plate

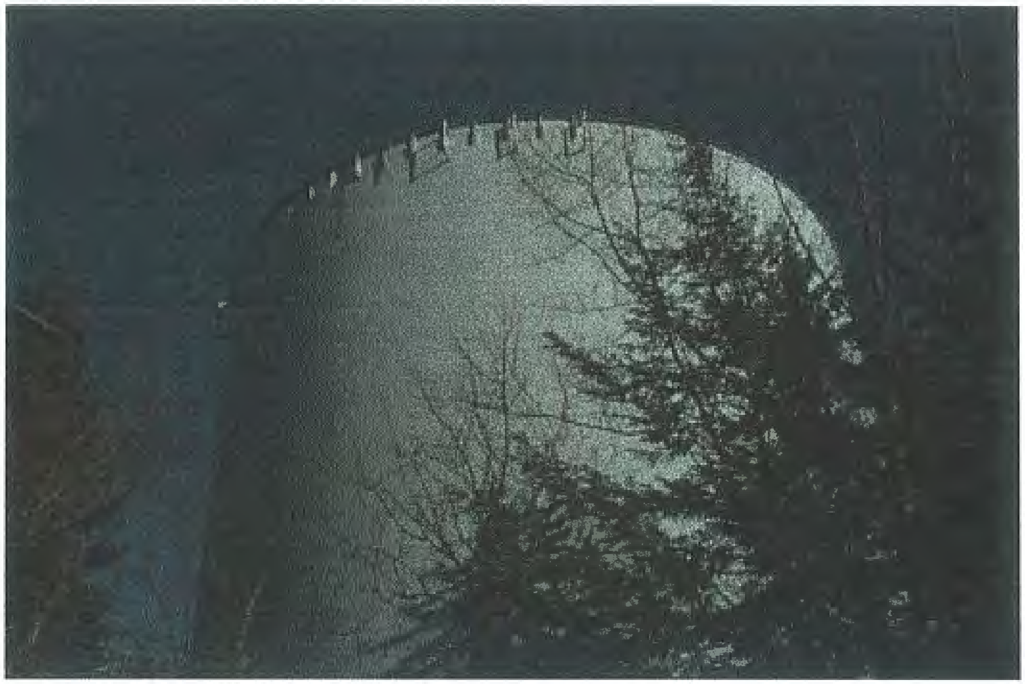
Lake Hills South - Access Manway



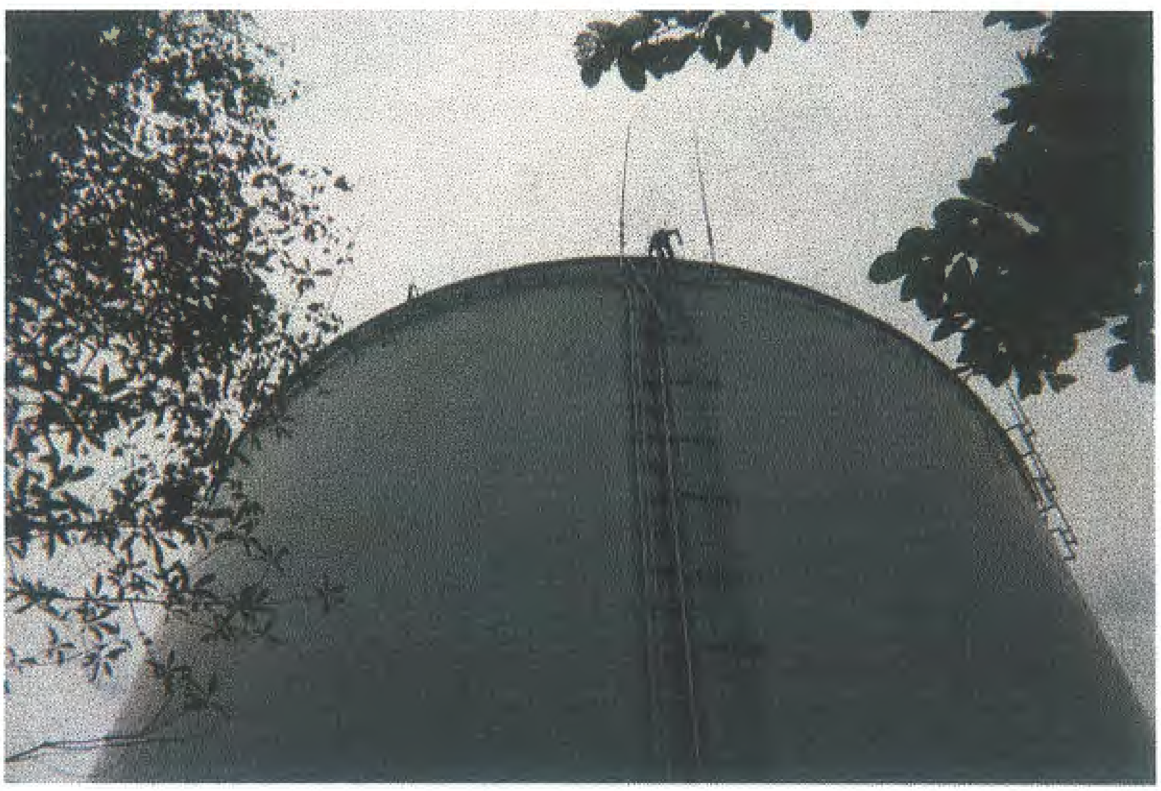
Lake Hills South - Overflow



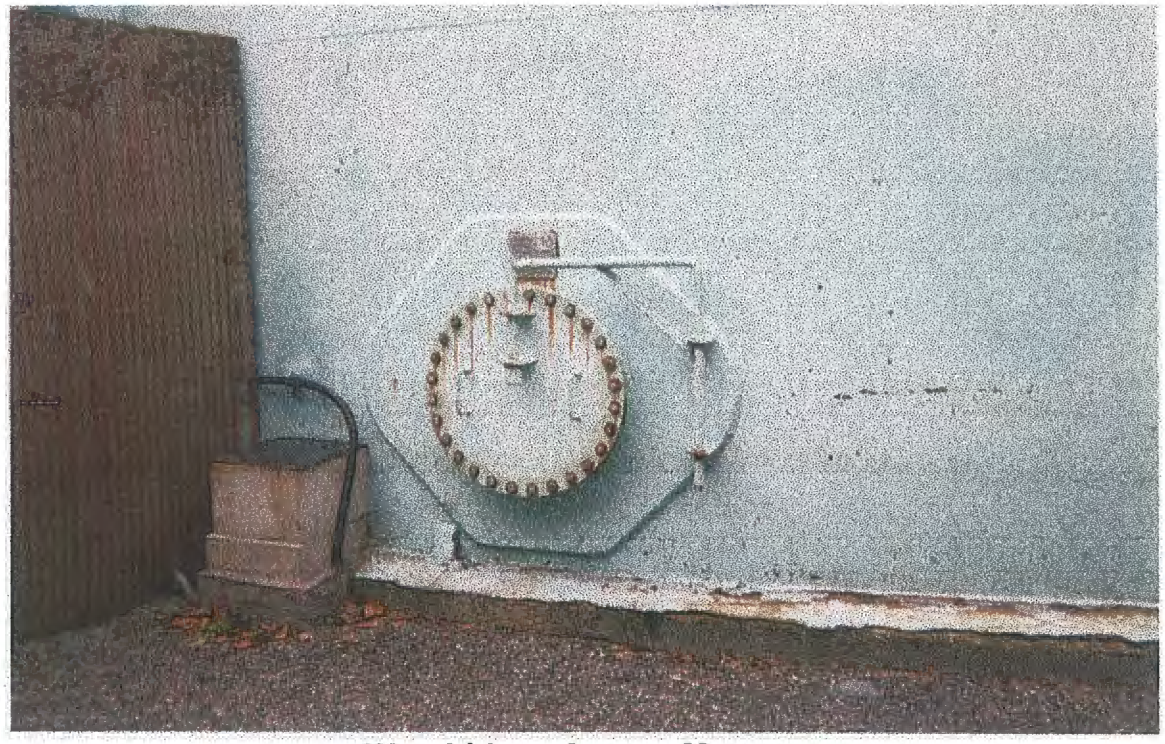
Lake Hills South - Pitting in Floor Plate

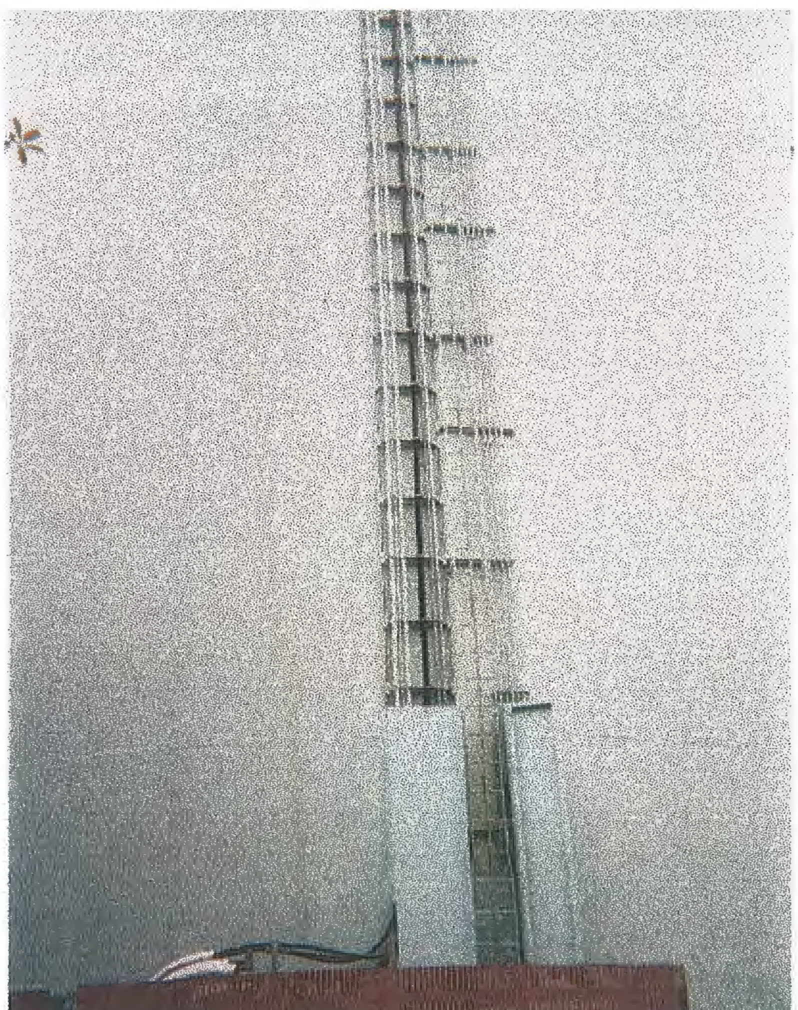


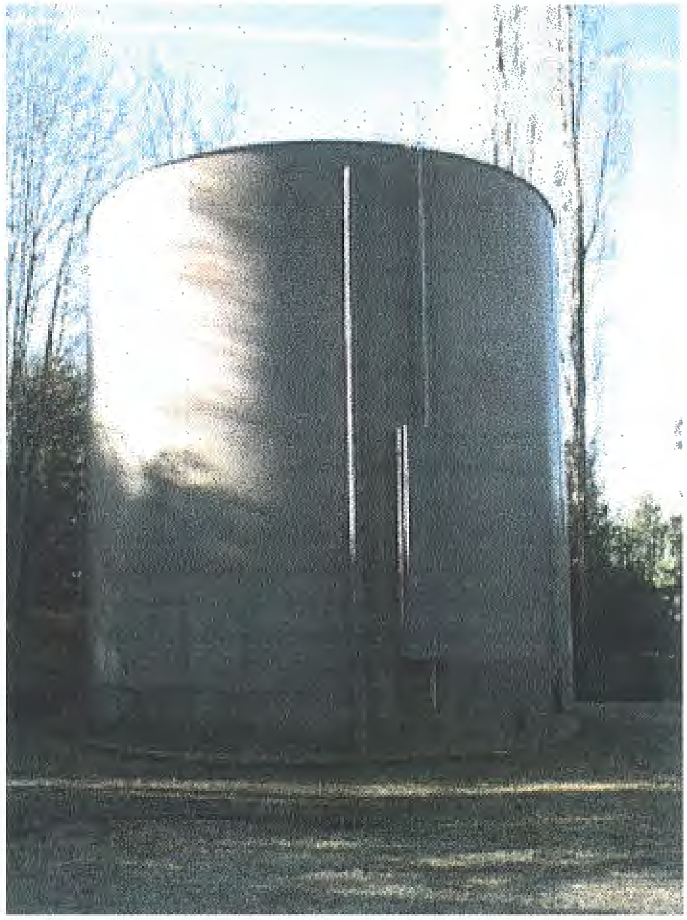
Woodridge Reservoir



Woodridge Reservoir


Woodridge - Access Manway


Woodridge - Access Ladder



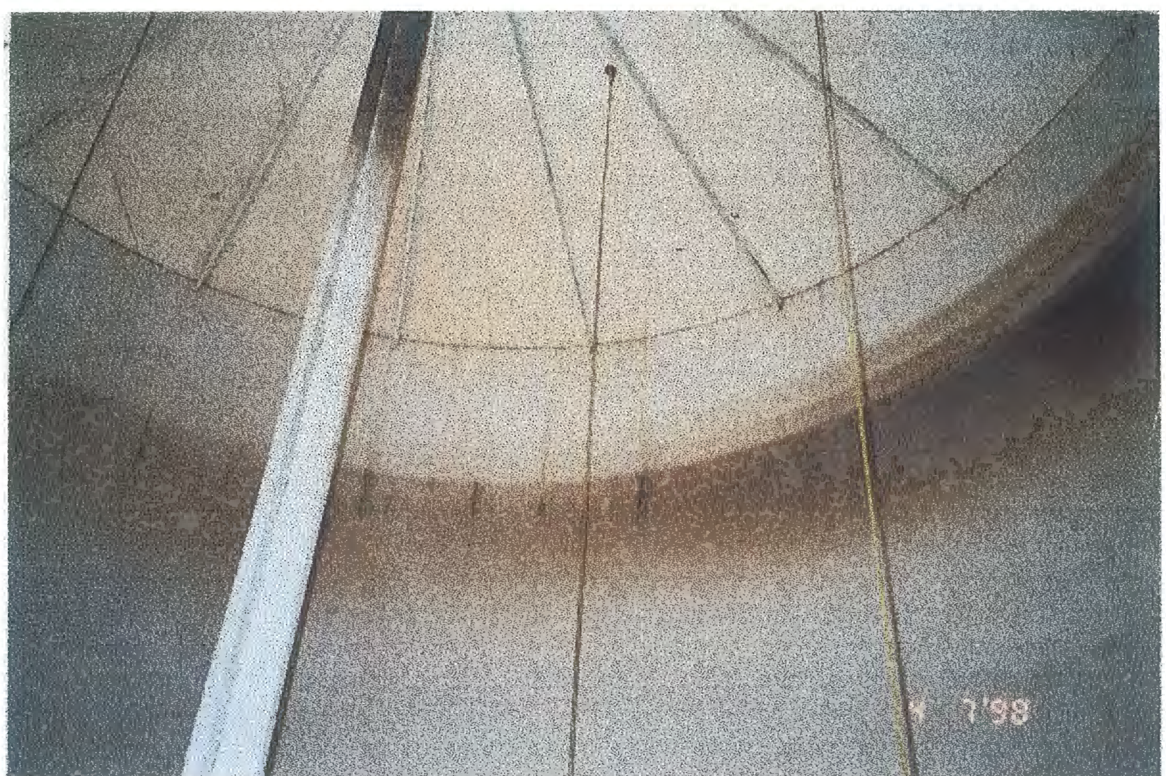
Horizon View No. 1 - Reservoir

Horizon View No. 1 - Wind Anchor

Horizon View No. 1 - Access Ladder and Overflow

Horizon View No. 1 - Access Manway

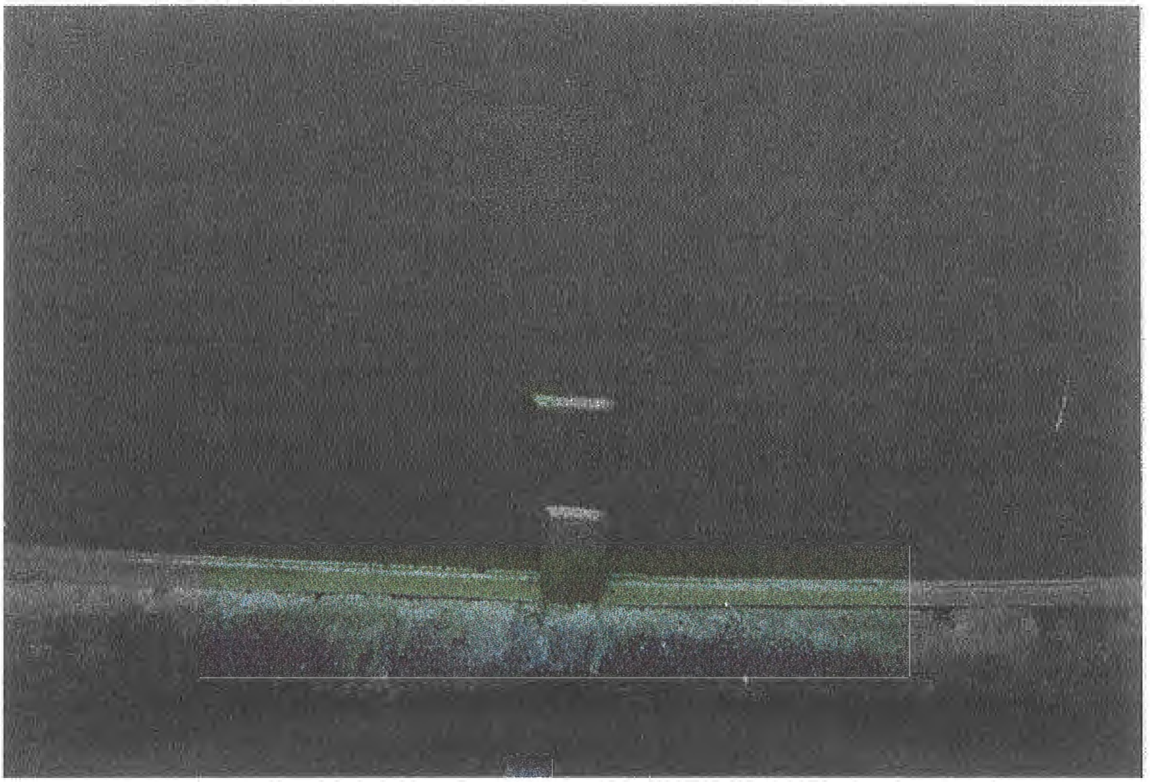
Horizon View No. 1 - Access Hatch



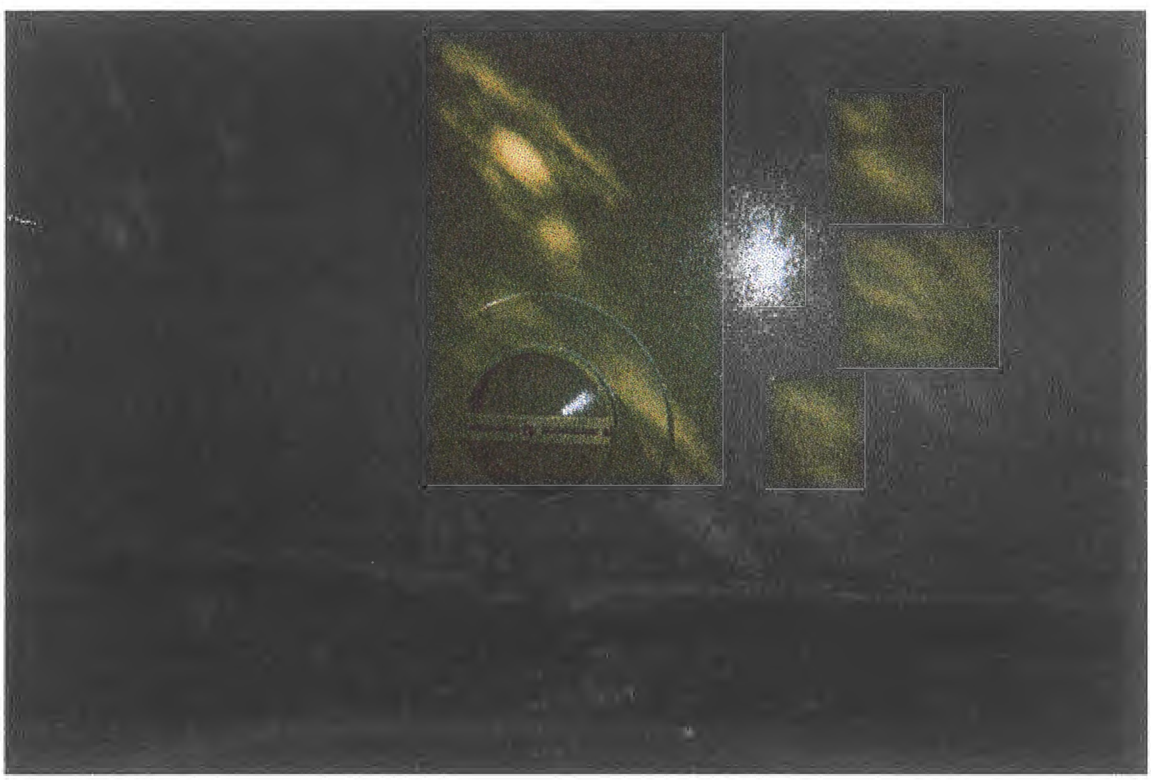
Horizon View No. 1 - Corroded Shell Plate

Horizon View No. 1 - Inlet/Outlet

Horizon View No. 1 - Cathodic Protection



Parksite - Wind Anchors



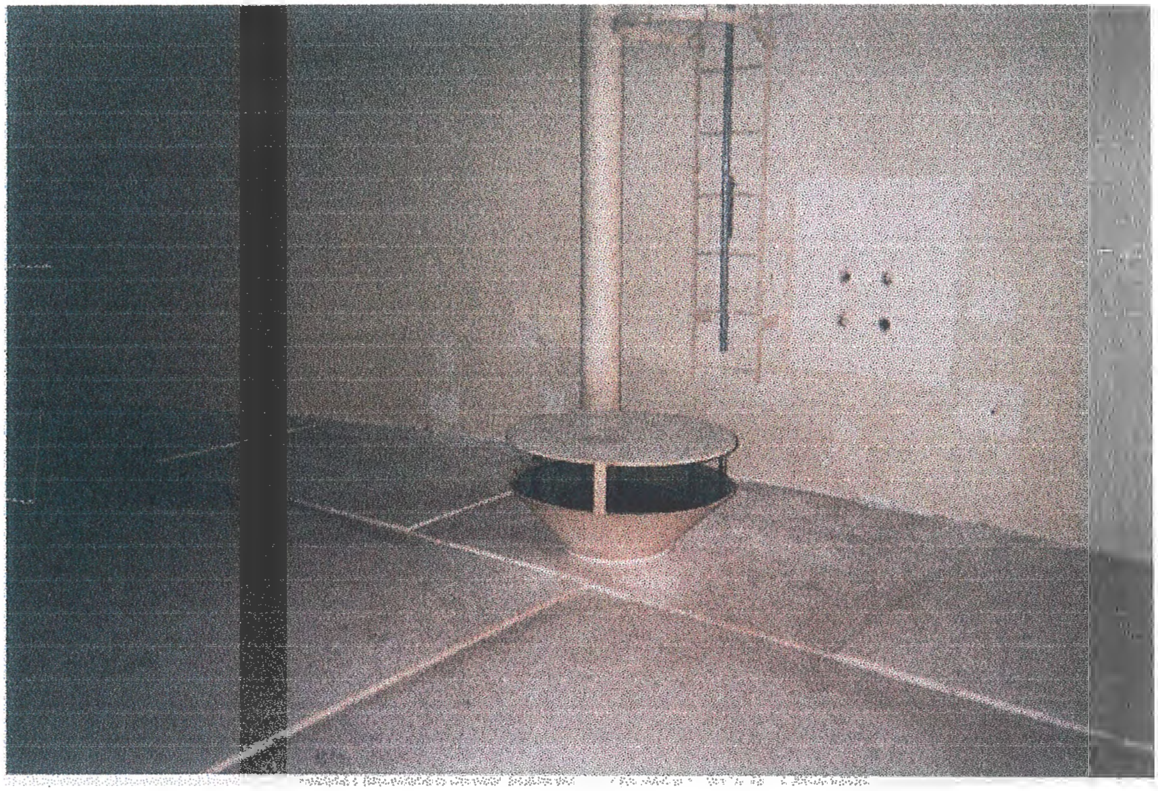
Parksite - Access Manway

Parksite - Delaminated Exterior Coating

Parksite - Site Drainage Problem

Parksite - Roof Vent and Safety Cables

Parksite - Existing Cut Slope

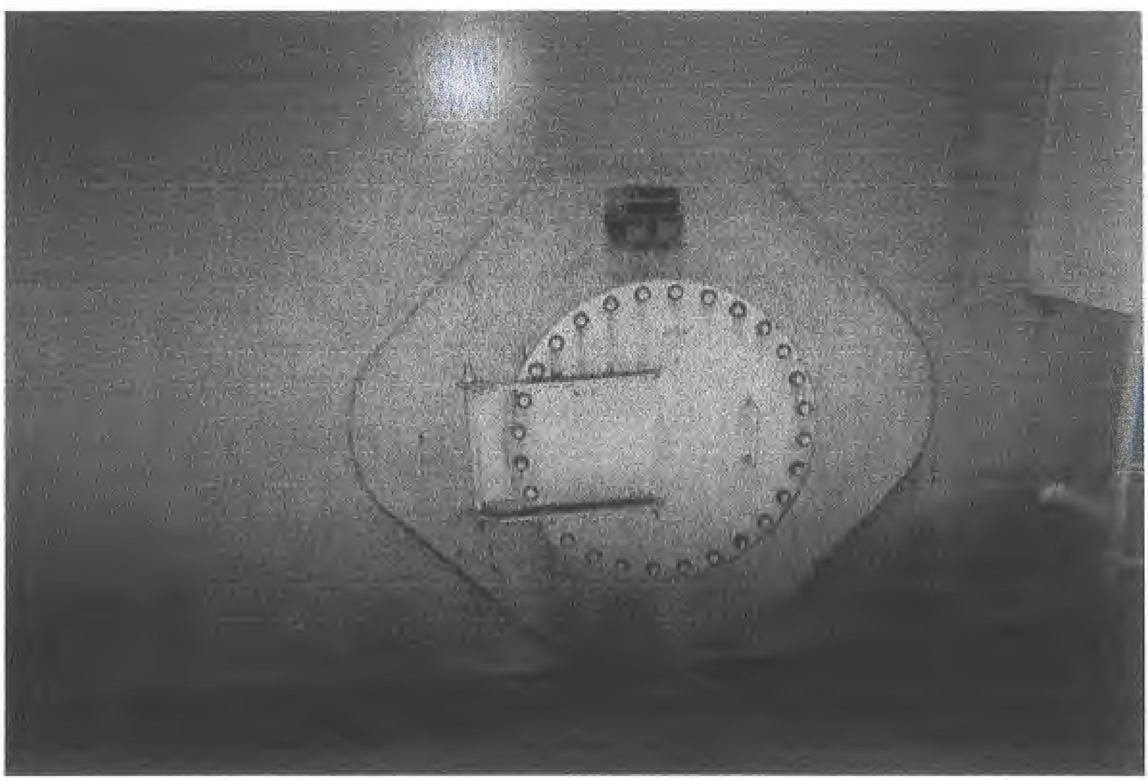


Parksite - Overflow and Inlet/Outlet

Pikes Peak - Wind Anchor

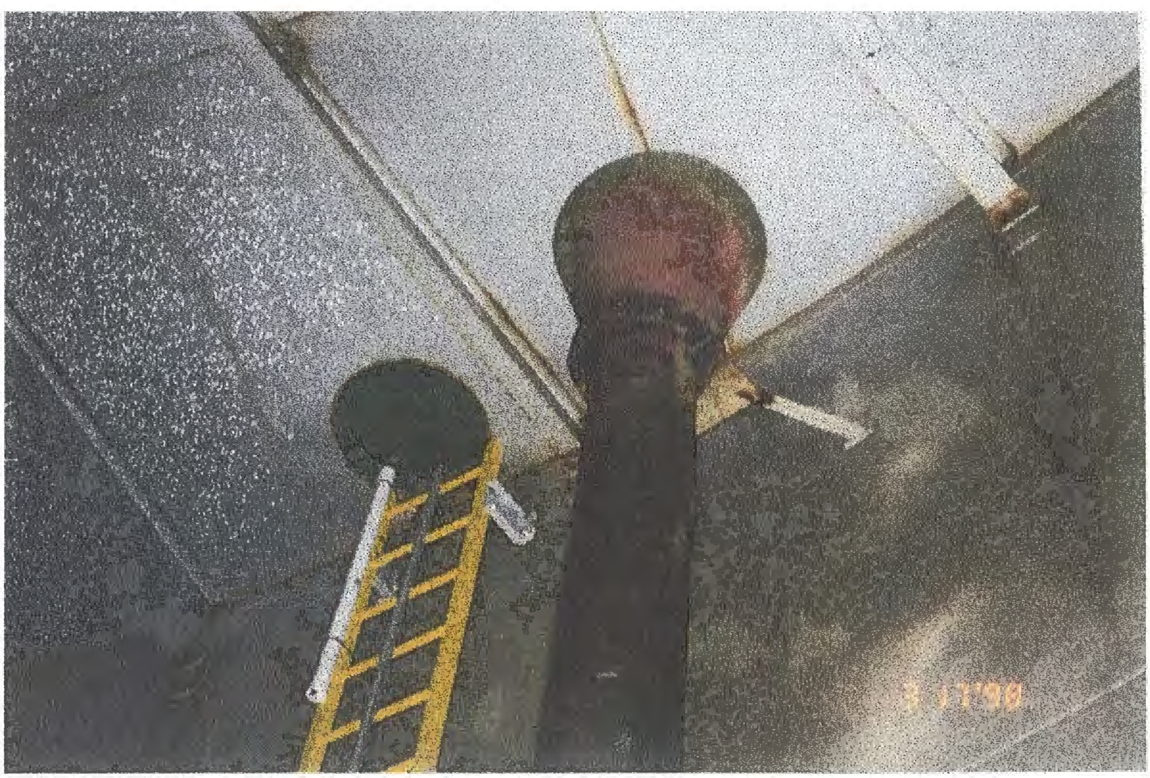
Pikes Peak - Bottom of Shell Plate at Ring Footing

Pikes Peak - Access Hatch



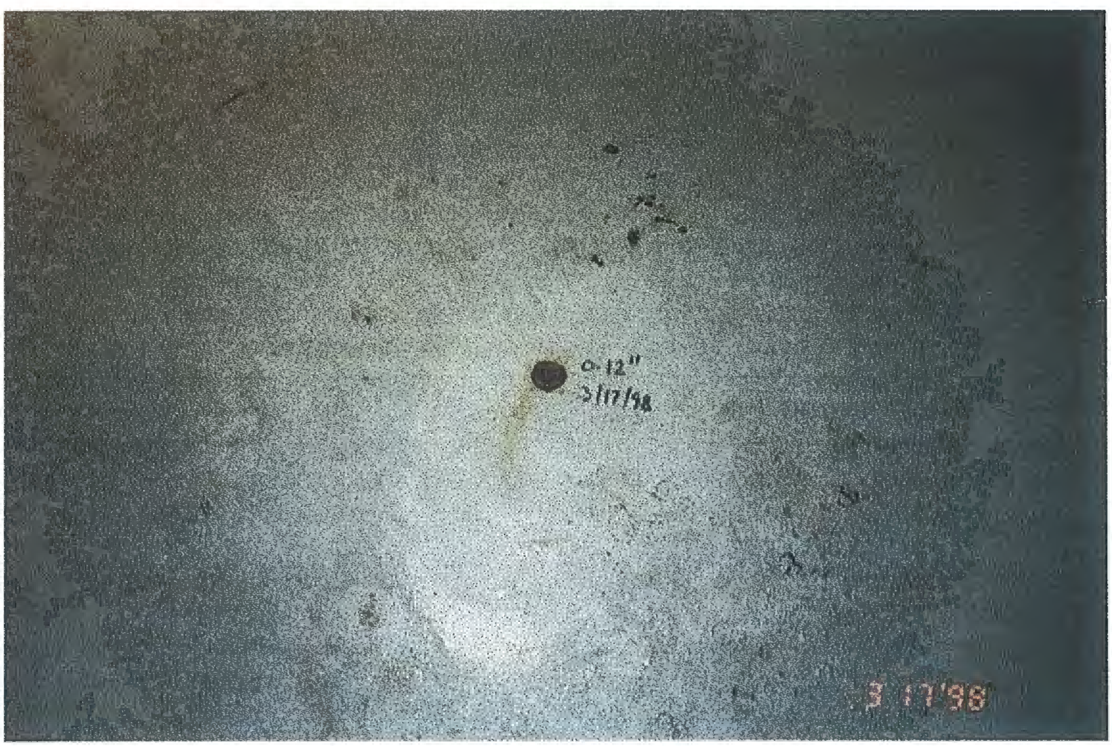
Pikes Peak - Access Manway

Pikes Peak - Overflow



Pikes Peak - Overflow

Pikes Peak - Inlet/Outlet



Pikes Peak - Pitting in Floor Plate

Somerset No. 1 - Reservoir

Somerset No. 1 - Cracks in Floor Slab

Somerset No. 1 - Vertical Wall Crack

Somerset No. 1 - Deteriorated Wall

Somerset No. 1 - Spalled Concrete at Diaphram Connection

Somerset No. 1 - Corroded Access Ladder

Somerset No. 1 - Corroded Overflow

Appendix C

MONTGOMERY WATSON

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:		TPD
DESCRIPTION:	Lake Hills North Tank using a 40%g seismic		CHECKER:		

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.128
tank diameter, D (ft) = **69.5**
max fluid depth, H (ft) = **75** Therefore, D/H = 0.927
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level
D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	75.00				
					75.00			0.00	
					75.00			0.00	
	ring # 8	8.50	0.275	8.50	66.50	0.85	6571	95.45	**OK**
	ring # 7	9.50	0.385	18.00	57.00	0.85	9939	149.35	**OK**
	ring # 6	9.50	0.500	27.50	47.50	0.85	11692	193.96	**OK**
	ring # 5	9.50	0.620	37.00	38.00	0.85	12687	240.51	**OK**
	ring # 4	9.50	0.710	46.50	28.50	0.85	13923	275.42	**OK**
	ring # 3	9.50	0.830	56.00	19.00	0.85	14343	321.97	**OK**
	ring # 2	9.50	0.930	65.50	9.50	0.85	14973	360.76	**OK**
Bottom	ring # 1	9.50	1.065	75.00	0.00	0.85	14971	413.13	**OK**

Weight of Wall per Foot = 2050.55 plf of circumference
Center of Gravity of the Wall = 29.41 feet above base

Weighted average wall thickness, ta = 0.670 inches
Bottom shell ring thickness, ts = 1.065 inches

Bottom Annular Plate Thickness, tb = **0.49** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 40%g seismic			CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	2.128	
tank diameter, D (ft) =	69.50	
max fluid depth, H (ft) =	75.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	2050.55	(see Previous Page)
C.G. of wall, (ft from base)=	29.41	(see Previous Page)
Bottom shell course thickness, ts (in) =	1.065	(see Previous Page)
ratio of D/H =	0.927	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)

Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — D / H = 0.927
- K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.577$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1 / W_T = 0.829$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1 / H = 0.413$
- W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2 / W_T = 0.213$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2 / H = 0.758$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 17754.37 kip

VALUES:
- Zone = 4
- Z = 0.40 (Table 24)
- I = 1.25 (Table 26)
- Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
- Ws = pi * 69.5 * 2050.55 / 1000 = 447.72 kip
- Xs = 29.41 ft
- Wr = 50.00 kip (engineer estimate, incl wt of shell above HW level)
- Ht = 76.00 ft
- W_1 = 0.829 * W_T = 0.829 * 17754.37 = 14,718.38 kip
- X_1 = 0.413 * H = 0.413 * 75 = 30.975 ft
- Soil Profile type = C (Table 27)
- S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 40%g seismic			CHECKER:	

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.213 * W_T = 0.213 * 17754.37 = 3,781.68 kip

$X_2 =$ 0.758 * H = 0.758 * 75 = 56.850 ft

Tw = K_p * Sqrt(D) = 4.810 ,(Eq 13-7)

C_1 = 0.0324 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 153,299.8 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 2,050.55 lbs/ft

wrs = 230.00 lbs/ft

wt = ws + wrs = 2,280.55 lbs/ft ,(Eq 13-18)

w_L, lbs = 5,806.50 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.490

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.117 ,L <= 0.035D, OK

0.035D (ft) = 2.433

CHECK UPLIFT: If M/(D^2*(wt+w_L))<0.785, then no uplift

If M/(D^2*(wt+w_L))>0.785 and <=1.54, then uplift is OK

If M/(D^2*(wt+w_L))>1.54, then bottom annular ring must be thickened or anchor the tank.

M/(D^2*(wt+w_L)) = 3.924 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 1.065

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 3339.77 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.670 ,(From Page 1)

ta/R = 0.00161

allowable compressive stress, F_L (psi) = 3,172.2 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 4,228.5 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 40%g seismic			CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 12725 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 0.927

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring # 8	66.50	0.275	292.0	561.1	1536.0	3102	5585	8687	**OK**
ring # 7	57.00	0.385	176.8	1070.3	3252.6	3239	8448	11688	**OK**
ring # 6	47.50	0.500	107.4	1455.0	4969.3	3125	9939	13063	**OK**
ring # 5	38.00	0.620	65.7	1715.4	6685.9	2873	10784	13656	**OK**
ring # 4	28.50	0.710	40.9	1851.3	8402.6	2665	11835	14500	**OK**
ring # 3	19.00	0.830	26.7	1878.4	10119.2	2295	12192	14487	**OK**
ring # 2	9.50	0.930	19.5	1878.4	11835.9	2041	12727	14767	**OK**
ring # 1	0.00	1.065	17.3	1878.4	13552.5	1780	12725	14505	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w) =$ 2.826 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:		TPD
DESCRIPTION:	Lake Hills North Tank using a 40%g seismic		CHECKER:		

Evaluate Tank Anchors:

M =	153,299.8	ft-kip
wt =	2,280.55	lbs/ft
D =	69.5	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 18.20 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 693618 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !

(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Oct-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 30%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.128
tank diameter, D (ft) = **69.5**
max fluid depth, H (ft) = **75** Therefore, D/H = 0.927
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level

D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	75.00				
					75.00			0.00	
					75.00			0.00	
	ring #8	8.50	0.275	8.50	66.50	0.85	6571	95.45	**OK**
	ring #7	9.50	0.385	18.00	57.00	0.85	9939	149.35	**OK**
	ring #6	9.50	0.500	27.50	47.50	0.85	11692	193.96	**OK**
	ring #5	9.50	0.620	37.00	38.00	0.85	12687	240.51	**OK**
	ring #4	9.50	0.710	46.50	28.50	0.85	13923	275.42	**OK**
	ring # 3	9.50	0.830	56.00	19.00	0.85	14343	321.97	**OK**
	ring # 2	9.50	0.930	65.50	9.50	0.85	14973	360.76	**OK**
Bottom	ring # 1	9.50	1.065	75.00	0.00	0.85	14971	413.13	**OK**

Weight of Wall per Foot = 2050.55 plf of circumference
Center of Gravity of the Wall = 29.41 feet above base

Weighted average wall thickness, ta = 0.670 inches
Bottom shell ring thickness, ts = 1.065 inches

Bottom Annular Plate Thickness, tb = **0.49** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Oct-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 30%g seismic	CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	2.128	
tank diameter, D (ft) =	69.50	
max fluid depth, H (ft) =	75.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	2050.55	(see Previous Page)
C.G. of wall, (ft from base)=	29.41	(see Previous Page)
Bottom shell course thickness, ts (in) =	1.065	(see Previous Page)
ratio of D/H =	0.927	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[\,0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1\,]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — $\boxed{D / H = 0.927}$
- K_p = Coefficient Relating Tank Size to Period . . . . (Figure 8) $K_p = 0.577$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs . . . . . . (Figure 9) $W_1 / W_T = 0.829$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 . . . . (Figure 10) $X_1 / H = 0.413$
- W_2 = Weight of Sloshing Contents of Tank, Lbs . . . . (Figure 9) $W_2 / W_T = 0.213$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 . . . . (Figure 10) $X_2 / H = 0.758$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi*D^2/4$ *H*G*62.4 = 17754.37 kip

VALUES:
- Zone = 3
- Z = 0.30 (Table 24)
- I = 1.25 (Table 26)
- Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
- Ws = pi * 69.5 * 2050.55 / 1000 = 447.72 kip
- Xs = 29.41 ft
- Wr = 50.00 kip (engineer estimate, incl wt of shell above HW level)
- Ht = 76.00 ft
- W_1 = 0.829 * W_T = 0.829 * 17754.37 = 14,718.38 kip
- X_1 = 0.413 * H = 0.413 * 75 = 30.975 ft
- Soil Profile type = C (Table 27)
- S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Oct-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 30%g seismic			CHECKER:	

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.213 * W_T = 0.213 * 17754.37 = 3,781.68 kip

$X_2 =$ 0.758 * H = 0.758 * 75 = 56.850 ft

Tw = K_p * Sqrt(D) = 4.810 ,(Eq 13-7)

C_1 = 0.0324 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 114,974.8 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 2,050.55 lbs/ft

wrs = 230.00 lbs/ft

wt = ws + wrs = 2,280.55 lbs/ft ,(Eq 13-18)

w_L, lbs = 5,806.50 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.490

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.117 ,L <= 0.035D, OK

0.035D (ft) = 2.433

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 2.943 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 1.065

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 2549.44 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.670 ,(From Page 1)

ta/R = 0.00161

allowable compressive stress, F_L (psi) = 3,172.2 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 4,228.5 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Oct-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills North Tank using a 30%g seismic			CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 12725 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 0.927

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)

Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	Forces: H-dynamic convective force, N_c (lb/in)	Forces: H-dynamic impulsive force, N_i (lb/in)	Forces: H-static force, N_h (lb/in)	Stresses: σ_s = Sum H-dynamic Stress** (psi)	Stresses: H-static Stress (psi)	Stresses: Total Stress (psi)	
ring #8	66.50	0.275	219.0	420.8	1536.0	2327	5585	7912	**OK**
ring #7	57.00	0.385	132.6	802.7	3252.6	2429	8448	10878	**OK**
ring #6	47.50	0.500	80.5	1091.3	4969.3	2344	9939	12282	**OK**
ring #5	38.00	0.620	49.2	1286.5	6685.9	2154	10784	12938	**OK**
ring #4	28.50	0.710	30.7	1388.4	8402.6	1999	11835	13833	**OK**
ring # 3	19.00	0.830	20.1	1408.8	10119.2	1722	12192	13913	**OK**
ring # 2	9.50	0.930	14.6	1408.8	11835.9	1531	12727	14257	**OK**
ring # 1	0.00	1.065	12.9	1408.8	13552.5	1335	12725	14060	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w)$ = 2.120 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Oct-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Lake Hills North Tank using a 30%g seismic		CHECKER:		

Evaluate Tank Anchors:

M =	114,974.8	ft-kip
wt =	2,280.55	lbs/ft
D =	69.5	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 18.20 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt)$ = 509840 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b)$ = 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 40%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.038
tank diameter, D (ft) = **68**
max fluid depth, H (ft) = **75** Therefore, D/H = 0.907
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level
D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	75.00				
					75.00			0.00	
	ring # 9	7.00	0.290	7.00	68.00	0.85	5021	82.89	**OK**
	ring # 8	8.50	0.390	15.50	59.50	0.85	8267	135.36	**OK**
	ring # 7	8.50	0.500	24.00	51.00	0.85	9984	173.54	**OK**
	ring # 6	8.50	0.590	32.50	42.50	0.85	11458	204.78	**OK**
	ring # 5	8.50	0.690	41.00	34.00	0.85	12359	239.49	**OK**
	ring # 4	8.50	0.805	49.50	25.50	0.85	12790	279.40	**OK**
	ring # 3	8.50	0.900	58.00	17.00	0.85	13404	312.38	**OK**
	ring # 2	8.50	1.010	66.50	8.50	0.85	13695	350.55	**OK**
Bottom	ring # 1	8.50	1.115	75.00	0.00	0.85	13991	387.00	**OK**

Weight of Wall per Foot = 2165.39 plf of circumference
Center of Gravity of the Wall = 29.53 feet above base

Weighted average wall thickness, ta = 0.707 inches
Bottom shell ring thickness, ts = 1.115 inches

Bottom Annular Plate Thickness, tb = **0.49** inches

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 40%g seismic	CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	2.038	
tank diameter, D (ft) =	68.00	
max fluid depth, H (ft) =	75.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	2165.39	(see Previous Page)
C.G. of wall, (ft from base)=	29.53	(see Previous Page)
Bottom shell course thickness, ts (in) =	1.115	(see Previous Page)
ratio of D/H =	0.907	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
Z = Seismic Zone Coefficient
I = Importance Factor (use 1.25 UNO)
Rw = Force Reduction Coefficient
Ws = Total Weight of Shell, Lbs
Xs = Height, FT, from bottom of tank to C.G. of shell
Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
Ht = Height of Tank Shell, FT — $\boxed{D/H = 0.907}$
K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.577$
W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1/W_T = 0.835$
X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1/H = 0.415$
W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2/W_T = 0.208$
X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2/H = 0.762$
C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
S = Site Amplification Factor
Tw = First Mode Sloshing Wave Period
W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 16996.27 kip

VALUES:
Zone = 4
Z = 0.40 (Table 24)
I = 1.25 (Table 26)
Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
Ws = pi * 68 * 2165.39 / 1000 = 462.59 kip
Xs = 29.53 ft
Wr = 50.00 kip (engineer estimate, incl wt of shell above HW level)
Ht = 76.00 ft
W_1 = 0.835 * W_T = 0.835 * 16996.27 = 14,191.88 kip
X_1 = 0.415 * H = 0.415 * 75 = 31.125 ft
Soil Profile type = C (Table 27)
S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 40%g seismic			CHECKER:	

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.208 * W_T = 0.208 * 16996.27 = 3,535.22 kip

$X_2 =$ 0.762 * H = 0.762 * 75 = 57.150 ft

Tw = K_p * Sqrt(D) = 4.758 ,(Eq 13-7)

C_1 = 0.0331 (Eq 13-5 or 13-6), Tw <4.5, C_1 = 1/(6Tw)

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 148,633.4 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 2,165.39 lbs/ft

wrs = 230.00 lbs/ft

wt = ws + wrs = 2,395.39 lbs/ft ,(Eq 13-18)

w_L, lbs = 5,806.50 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.490

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.117 ,L <= 0.035D, OK

0.035D (ft) = 2.380

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 3.919 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 1.115

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 3237.26 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.707 ,(From Page 1)

ta/R = 0.00173

allowable compressive stress, F_L (psi) = 3,488.2 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 4,649.7 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 40%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 11892 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 0.907

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring # 9	68.00	0.290	306.6	458.4	1237.6	2638	4268	6906	**OK**
ring # 8	59.50	0.390	193.7	924.3	2740.4	2867	7027	9893	**OK**
ring # 7	51.00	0.500	122.6	1290.5	4243.2	2826	8486	11313	**OK**
ring # 6	42.50	0.590	77.9	1557.1	5746.0	2771	9739	12510	**OK**
ring # 5	34.00	0.690	49.9	1724.1	7248.8	2571	10506	13077	**OK**
ring # 4	25.50	0.805	32.7	1791.5	8751.6	2266	10872	13138	**OK**
ring # 3	17.00	0.900	22.5	1798.2	10254.4	2023	11394	13417	**OK**
ring # 2	8.50	1.010	17.1	1798.2	11757.2	1797	11641	13438	**OK**
ring # 1	0.00	1.115	15.5	1798.2	13260.0	1627	11892	13519	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D*(ZIC_1S/R_w) =$ 2.825 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 40%g seismic	CHECKER:	

Evaluate Tank Anchors:

M =	148,633.4	ft-kip
wt =	2,395.39	lbs/ft
D =	68	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 17.80 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 685815 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 30%g seismic			CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.038
tank diameter, D (ft) = **68**
max fluid depth, H (ft) = **75** Therefore, D/H = 0.907
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level

D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	75.00				
					75.00			0.00	
	ring # 9	7.00	0.290	7.00	68.00	0.85	5021	82.89	OK
	ring # 8	8.50	0.390	15.50	59.50	0.85	8267	135.36	OK
	ring # 7	8.50	0.500	24.00	51.00	0.85	9984	173.54	OK
	ring # 6	8.50	0.590	32.50	42.50	0.85	11458	204.78	OK
	ring # 5	8.50	0.690	41.00	34.00	0.85	12359	239.49	OK
	ring # 4	8.50	0.805	49.50	25.50	0.85	12790	279.40	OK
	ring # 3	8.50	0.900	58.00	17.00	0.85	13404	312.38	OK
	ring # 2	8.50	1.010	66.50	8.50	0.85	13695	350.55	OK
Bottom	ring # 1	8.50	1.115	75.00	0.00	0.85	13991	387.00	OK

Weight of Wall per Foot = 2165.39 plf of circumference
Center of Gravity of the Wall = 29.53 feet above base

Weighted average wall thickness, ta = 0.707 inches
Bottom shell ring thickness, ts = 1.115 inches

Bottom Annular Plate Thickness, tb = **0.49** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 30%g seismic			CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	2.038	
tank diameter, D (ft) =	68.00	
max fluid depth, H (ft) =	75.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	2165.39	(see Previous Page)
C.G. of wall, (ft from base)=	29.53	(see Previous Page)
Bottom shell course thickness, ts (in) =	1.115	(see Previous Page)
ratio of D/H =	0.907	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — D / H = 0.907
- K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.577$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1 / W_T = 0.835$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1 / H = 0.415$
- W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2 / W_T = 0.208$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2 / H = 0.762$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi*D^2/4 \;*H*G*62.4$ = 16996.27 kip

VALUES:
- Zone = 3
- Z = 0.30 (Table 24)
- I = 1.25 (Table 26)
- Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
- Ws = pi * 68 * 2165.39 / 1000 = 462.59 kip
- Xs = 29.53 ft
- Wr = 50.00 kip (engineer estimate, incl wt of shell above HW level)
- Ht = 76.00 ft
- W_1 = 0.835 * W_T = 0.835 * 16996.27 = 14,191.88 kip
- X_1 = 0.415 * H = 0.415 * 75 = 31.125 ft
- Soil Profile type = C (Table 27)
- S = 1.50 (Table 27)

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.208 * W_T = 0.208 * 16996.27 = 3,535.22 kip

$X_2 =$ 0.762 * H = 0.762 * 75 = 57.150 ft

Tw = K_p * Sqrt(D) = 4.758 ,(Eq 13-7)

C_1 = 0.0331 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

M = 111,475.1 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 2,165.39 lbs/ft

wrs = 230.00 lbs/ft

wt = ws + wrs = 2,395.39 lbs/ft ,(Eq 13-18)

w_L, lbs = 5,806.50 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.490

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.117 ,L <= 0.035D, OK

0.035D (ft) = 2.380

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 2.939 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = yes

bottom shell ring#1 thickness, ts (in) = 1.115

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 2472.70 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.707 ,(From Page 1)

ta/R = 0.00173

allowable compressive stress, F_L (psi) = 3,488.2 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 4,649.7 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

<u>SHELL COMPRESSION FORCE, OKAY</u>

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Lake Hills South Tank using a 30%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 11892 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 0.907

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)

Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	Forces: H-dynamic convective force, N_c (lb/in)	Forces: H-dynamic impulsive force, N_i (lb/in)	Forces: H-static force, N_h (lb/in)	Stresses: σ_s = Sum H-dynamic Stress** (psi)	Stresses: H-static Stress (psi)	Stresses: Total Stress (psi)	
ring # 9	68.00	0.290	230.0	343.8	1237.6	1979	4268	6246	**OK**
ring # 8	59.50	0.390	145.3	693.2	2740.4	2150	7027	9177	**OK**
ring # 7	51.00	0.500	92.0	967.9	4243.2	2120	8486	10606	**OK**
ring # 6	42.50	0.590	58.4	1167.9	5746.0	2078	9739	11817	**OK**
ring # 5	34.00	0.690	37.4	1293.1	7248.8	1928	10506	12434	**OK**
ring # 4	25.50	0.805	24.5	1343.7	8751.6	1700	10872	12571	**OK**
ring # 3	17.00	0.900	16.9	1348.7	10254.4	1517	11394	12911	**OK**
ring # 2	8.50	1.010	12.8	1348.7	11757.2	1348	11641	12989	**OK**
ring # 1	0.00	1.115	11.6	1348.7	13260.0	1220	11892	13112	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w) =$ 2.119 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Lake Hills South Tank using a 30%g seismic		CHECKER:		

Evaluate Tank Anchors:

M =	111,475.1	ft-kip
wt =	2,395.39	lbs/ft
D =	68	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 17.80 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt)$ = 503700 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b)$ = 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Woodridge Tank using a 40%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.103
tank diameter, D (ft) = **71**
max fluid depth, H (ft) = **71** Therefore, D/H = 1.000
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level

D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = 2.60*h_p*D*G / (t*E)

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	71.00				
					71.00			0.00	
	ring # 9	7.00	0.255	7.00	64.00	0.85	5962	72.89	**OK**
	ring # 8	8.00	0.390	15.00	56.00	0.85	8353	127.40	**OK**
	ring # 7	8.00	0.405	23.00	48.00	0.85	12333	132.30	**OK**
	ring # 6	8.00	0.530	31.00	40.00	0.85	12703	173.13	**OK**
	ring # 5	8.00	0.650	39.00	32.00	0.85	13031	212.33	**OK**
	ring # 4	8.00	0.755	47.00	24.00	0.85	13520	246.63	**OK**
	ring # 3	8.00	0.885	55.00	16.00	0.85	13497	289.10	**OK**
	ring # 2	8.00	1.035	63.00	8.00	0.85	13219	338.10	**OK**
Bottom	ring # 1	8.00	1.125	71.00	0.00	0.85	13706	367.50	**OK**

Weight of Wall per Foot = 1959.39 plf of circumference
Center of Gravity of the Wall = 27.01 feet above base

Weighted average wall thickness, ta = 0.676 inches
Bottom shell ring thickness, ts = 1.125 inches

Bottom Annular Plate Thickness, tb = **0.5** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Woodridge Tank using a 40%g seismic	CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	2.103	
tank diameter, D (ft) =	71.00	
max fluid depth, H (ft) =	71.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	1959.39	(see Previous Page)
C.G. of wall, (ft from base)=	27.01	(see Previous Page)
Bottom shell course thickness, ts (in) =	1.125	(see Previous Page)
ratio of D/H =	1.000	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[\,0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1\,]$,(Equation 13-8)

where:
Z = Seismic Zone Coefficient
I = Importance Factor (use 1.25 UNO)
Rw = Force Reduction Coefficient
Ws = Total Weight of Shell, Lbs
Xs = Height, FT, from bottom of tank to C.G. of shell
Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
Ht = Height of Tank Shell, FT — $\boxed{D/H = 1.000}$
K_p = Coefficient Relating Tank Size to Period . . . . (Figure 8) $K_p = 0.578$
W_1 = Weight of Tank Contents that Moves with the Shell, Lbs . . . . . . (Figure 9) $W_1/W_T = 0.808$
X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 . . . . (Figure 10) $X_1/H = 0.406$
W_2 = Weight of Sloshing Contents of Tank, Lbs . . . . (Figure 9) $W_2/W_T = 0.230$
X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 . . . . (Figure 10) $X_2/H = 0.742$
C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
S = Site Amplification Factor
Tw = First Mode Sloshing Wave Period
W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 17540.80 kip

VALUES:
Zone = 4
Z = 0.40 (Table 24)
I = 1.25 (Table 26)
Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
Ws = pi * 71 * 1959.39 / 1000 = 437.05 kip
Xs = 27.01 ft
Wr = 52.00 kip (engineer estimate, incl wt of shell above HW level)
Ht = 72.00 ft
W_1 = 0.808 * W_T = 0.808 * 17540.8 = 14,172.97 kip
X_1 = 0.406 * H = 0.406 * 71 = 28.826 ft
Soil Profile type = C (Table 27)
S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Woodridge Tank using a 40%g seismic		CHECKER:		

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.230 * W_T = 0.23 * 17540.8 = 4,034.39 kip

$X_2 =$ 0.742 * H = 0.742 * 71 = 52.682 ft

Tw = K_p * Sqrt(D) = 4.870 ,(Eq 13-7)

C_1 = 0.0316 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 138,896.2 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 1,959.39 lbs/ft

wrs = 233.00 lbs/ft

wt = ws + wrs = 2,192.39 lbs/ft ,(Eq 13-18)

w_L, lbs = 5,764.84 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.500

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.220 ,L <= 0.035D, OK

0.035D (ft) = 2.485

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 3.463 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 1.125

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 2760.57 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.676 ,(From Page 1)

ta/R = 0.00159

allowable compressive stress, F_L (psi) = 3,125.8 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 4,166.6 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Woodridge Tank using a 40%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 11650 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 1.000

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)									
			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring # 9	64.00	0.255	324.4	480.2	1292.2	3155	5067	8222	**OK**
ring # 8	56.00	0.390	214.6	946.2	2769.0	2976	7100	10076	**OK**
ring # 7	48.00	0.405	142.3	1324.0	4245.8	3620	10483	14104	**OK**
ring # 6	40.00	0.530	94.9	1613.5	5722.6	3223	10797	14021	**OK**
ring # 5	32.00	0.650	63.9	1814.8	7199.4	2890	11076	13966	**OK**
ring # 4	24.00	0.755	44.1	1927.9	8676.2	2612	11492	14104	**OK**
ring # 3	16.00	0.885	32.0	1960.4	10153.0	2251	11472	13724	**OK**
ring # 2	8.00	1.035	25.5	1960.4	11629.8	1919	11237	13155	**OK**
ring # 1	0.00	1.125	23.5	1960.4	13106.6	1763	11650	13414	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

d = 7.53 D*(ZIC_1S/R_w) = 2.816 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Woodridge Tank using a 40%g seismic			CHECKER:	

Evaluate Tank Anchors:

M =	138,896.2	ft-kip
wt =	2,192.39	lbs/ft
D =	71	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 18.59 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 611221 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Woodridge Tank using a 30%g seismic		CHECKER:		

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.103
tank diameter, D (ft) = **71**
max fluid depth, H (ft) = **71** Therefore, D/H = 1.000
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level
D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60 * h_p * D * G / (t * E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	71.00				
					71.00			0.00	
	ring # 9	7.00	0.255	7.00	64.00	0.85	5962	72.89	OK
	ring # 8	8.00	0.390	15.00	56.00	0.85	8353	127.40	OK
	ring # 7	8.00	0.405	23.00	48.00	0.85	12333	132.30	OK
	ring # 6	8.00	0.530	31.00	40.00	0.85	12703	173.13	OK
	ring # 5	8.00	0.650	39.00	32.00	0.85	13031	212.33	OK
	ring # 4	8.00	0.755	47.00	24.00	0.85	13520	246.63	OK
	ring # 3	8.00	0.885	55.00	16.00	0.85	13497	289.10	OK
	ring # 2	8.00	1.035	63.00	8.00	0.85	13219	338.10	OK
Bottom	ring # 1	8.00	1.125	71.00	0.00	0.85	13706	367.50	OK

Weight of Wall per Foot = 1959.39 plf of circumference
Center of Gravity of the Wall = 27.01 feet above base

Weighted average wall thickness, ta = 0.676 inches
Bottom shell ring thickness, ts = 1.125 inches

Bottom Annular Plate Thickness, tb = **0.5** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Woodridge Tank using a 30%g seismic		CHECKER:		

TANK INFORMATION:

CAPACITY (Million gal)=	2.103	
tank diameter, D (ft) =	71.00	
max fluid depth, H (ft) =	71.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	1959.39	(see Previous Page)
C.G. of wall, (ft from base)=	27.01	(see Previous Page)
Bottom shell course thickness, ts (in) =	1.125	(see Previous Page)
ratio of D/H =	1.000	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
Z = Seismic Zone Coefficient
I = Importance Factor (use 1.25 UNO)
Rw = Force Reduction Coefficient
Ws = Total Weight of Shell, Lbs
Xs = Height, FT, from bottom of tank to C.G. of shell
Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
Ht = Height of Tank Shell, FT D / H = 1.000
K_p = Coefficient Relating Tank Size to Period . . . . (Figure 8) $K_p = 0.578$
W_1 = Weight of Tank Contents that Moves with the Shell, Lbs . . . . (Figure 9) $W_1 / W_T = 0.808$
X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 . . . . (Figure 10) $X_1 / H = 0.406$
W_2 = Weight of Sloshing Contents of Tank, Lbs . . . . (Figure 9) $W_2 / W_T = 0.230$
X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 . . . . (Figure 10) $X_2 / H = 0.742$
C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
S = Site Amplification Factor
Tw = First Mode Sloshing Wave Period
W_T = Total weight of tank contents = $\pi * D^2/4 * H*G*62.4$ = 17540.80 kip

VALUES:
Zone = 3
Z = 0.30 (Table 24)
I = 1.25 (Table 26)
Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
Ws = pi * 71 * 1959.39 / 1000 = 437.05 kip
Xs = 27.01 ft
Wr = 52.00 kip (engineer estimate, incl wt of shell above HW level)
Ht = 72.00 ft
W_1 = 0.808 * W_T = 0.808 * 17540.8 = 14,172.97 kip
X_1 = 0.406 * H = 0.406 * 71 = 28.826 ft
Soil Profile type = C (Table 27)
S = 1.50 (Table 27)

OVERTURNING MOMENT (cont)...

$W_2 =$ 0.230 * W_T = 0.23 * 17540.8 = 4,034.39 kip

$X_2 =$ 0.742 * H = 0.742 * 71 = 52.682 ft

Tw = K_p * Sqrt(D) = 4.870 ,(Eq 13-7)

C_1 = 0.0316 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 104,172.2 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 1,959.39 lbs/ft

wrs = 233.00 lbs/ft

wt = ws + wrs = 2,192.39 lbs/ft ,(Eq 13-18)

w_L, lbs = 5,764.84 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.500

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.220 ,L <= 0.035D, OK

0.035D (ft) = 2.485

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 2.597 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = yes

bottom shell ring#1 thickness, ts (in) = 1.125

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 2111.03 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.676 ,(From Page 1)

ta/R = 0.00159

allowable compressive stress, F_L (psi) = 3,125.8 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 4,166.6 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Woodridge Tank using a 30%g seismic			CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 11650 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 1.000

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)									
			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring # 9	64.00	0.255	243.3	360.1	1292.2	2366	5067	7434	**OK**
ring # 8	56.00	0.390	161.0	709.6	2769.0	2232	7100	9332	**OK**
ring # 7	48.00	0.405	106.8	993.0	4245.8	2715	10483	13199	**OK**
ring # 6	40.00	0.530	71.1	1210.1	5722.6	2417	10797	13215	**OK**
ring # 5	32.00	0.650	47.9	1361.1	7199.4	2168	11076	13244	**OK**
ring # 4	24.00	0.755	33.1	1445.9	8676.2	1959	11492	13451	**OK**
ring # 3	16.00	0.885	24.0	1470.3	10153.0	1688	11472	13161	**OK**
ring # 2	8.00	1.035	19.2	1470.3	11629.8	1439	11237	12676	**OK**
ring # 1	0.00	1.125	17.6	1470.3	13106.6	1323	11650	12973	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w)$ = 2.112 ft ,(Eq 13-26)

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Woodridge Tank using a 30%g seismic	CHECKER:	

Evaluate Tank Anchors:

M =	104,172.2	ft-kip
wt =	2,192.39	lbs/ft
D =	71	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 18.59 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 448228 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !

(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 40%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 0.200
tank diameter, D (ft) = **31.17**
max fluid depth, H (ft) = **35** Therefore, D/H = 0.891
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level
D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60 * h_p * D * G / (t * E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	35.00				
					35.00			0.00	
					35.00			0.00	
					35.00			0.00	
					35.00			0.00	
	ring # 6	5.00	0.250	5.00	30.00	0.85	1907	51.04	**OK**
	ring # 5	6.00	0.250	11.00	24.00	0.85	4195	61.25	**OK**
	ring #4	6.00	0.250	17.00	18.00	0.85	6483	61.25	**OK**
	ring # 3	6.00	0.255	23.00	12.00	0.85	8600	62.48	**OK**
	ring # 2	6.00	0.260	29.00	6.00	0.85	10634	63.70	**OK**
Bottom	ring # 1	6.00	0.260	35.00	0.00	0.85	12835	63.70	**OK**

Weight of Wall per Foot = 363.42 plf of circumference
Center of Gravity of the Wall = 17.34 feet above base

Weighted average wall thickness, ta = 0.254 inches
Bottom shell ring thickness, ts = 0.260 inches

Bottom Annular Plate Thickness, tb = **0.25** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 40%g seismic	CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	0.200	
tank diameter, D (ft) =	31.17	
max fluid depth, H (ft) =	35.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	363.42	(see Previous Page)
C.G. of wall, (ft from base)=	17.34	(see Previous Page)
Bottom shell course thickness, ts (in) =	0.260	(see Previous Page)
ratio of D/H =	0.891	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)

Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — D / H = 0.891
- K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.577$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1 / W_T = 0.840$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1 / H = 0.417$
- W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2 / W_T = 0.205$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2 / H = 0.766$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 1666.54 kip

VALUES:

Zone =	4	
Z =	0.40	(Table 24)
I =	1.25	(Table 26)
Rw =	4.50	(Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
Ws =	pi * 31.17 * 363.42 / 1000 = 35.59 kip	
Xs =	17.34	ft
Wr =	7.50	kip (engineer estimate, incl wt of shell above HW level)
Ht =	36.00	ft
W_1 =	0.840 * W_T = 0.84 * 1666.54 = 1,399.89 kip	
X_1 =	0.417 * H = 0.417 * 35 = 14.595 ft	
Soil Profile type =	C	(Table 27)
S =	1.50	(Table 27)

OVERTURNING MOMENT (cont)...

$W_2 =$ 0.205 * W_T = 0.205 * 1666.54 = 341.64 kip

$X_2 =$ 0.766 * H = 0.766 * 35 = 26.810 ft

Tw = K_p * Sqrt(D) = 3.221 ,(Eq 13-7)

C_1 = 0.0517 (Eq 13-5 or 13-6), Tw <4.5, C_1 = 1/(6Tw)

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 7,389.8 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 363.42 lbs/ft

wrs = 230.00 lbs/ft

wt = ws + wrs = 593.42 lbs/ft ,(Eq 13-18)

w_L, lbs = 1,396.42 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.250

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 1.581 ,L > 0.035D... **TANK MU**

0.035D (ft) = 1.091

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 3.822 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = yes

bottom shell ring#1 thickness, ts (in) = 0.260

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 3293.55 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.254 ,(From Page 1)

ta/R = 0.00136

allowable compressive stress, F_L (psi) = 2,599.4 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 3,465.0 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 40%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 10910 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 0.891

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring # 6	30.00	0.250	81.5	143.9	405.2	902	1621	2522	**OK**
ring # 5	24.00	0.250	40.2	271.1	891.5	1245	3566	4811	**OK**
ring #4	18.00	0.250	20.0	348.7	1377.7	1475	5511	6986	**OK**
ring # 3	12.00	0.255	10.3	376.7	1864.0	1517	7310	8827	**OK**
ring # 2	6.00	0.260	5.9	377.8	2350.2	1476	9039	10515	**OK**
ring # 1	0.00	0.260	4.7	377.8	2836.5	1471	10910	12381	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w)$ = 2.022 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:		TPD
DESCRIPTION:	Horizon View Tank using a 40%g seismic		CHECKER:		

Evaluate Tank Anchors:

M =	7,389.8	ft-kip
wt =	593.42	lbs/ft
D =	31.17	ft

number of anchors to be used = **4**

A_b = anchor area = **0.75** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 24.48 ft

anchor tension, $T_B = S_L$ * ((1.273*M / D^2) - wt) = 222508 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = 4/3*(0.6*Fy*A_b) = 18000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 30%g seismic			CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 0.200
tank diameter, D (ft) = 31.17
max fluid depth, H (ft) = 35 Therefore, D/H = 0.891
joint efficiency factor, E = 0.85
fluid specific gravity, G = 1

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level

D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = 15000 psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	35.00				
					35.00			0.00	
					35.00			0.00	
					35.00			0.00	
					35.00			0.00	
	ring # 6	5.00	0.250	5.00	30.00	0.85	1907	51.04	OK
	ring # 5	6.00	0.250	11.00	24.00	0.85	4195	61.25	OK
	ring #4	6.00	0.250	17.00	18.00	0.85	6483	61.25	OK
	ring # 3	6.00	0.255	23.00	12.00	0.85	8600	62.48	OK
	ring # 2	6.00	0.260	29.00	6.00	0.85	10634	63.70	OK
Bottom	ring # 1	6.00	0.260	35.00	0.00	0.85	12835	63.70	OK

Weight of Wall per Foot = 363.42 plf of circumference
Center of Gravity of the Wall = 17.34 feet above base

Weighted average wall thickness, ta = 0.254 inches
Bottom shell ring thickness, ts = 0.260 inches

Bottom Annular Plate Thickness, tb = 0.25 inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 30%g seismic	CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	0.200	
tank diameter, D (ft) =	31.17	
max fluid depth, H (ft) =	35.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	363.42	(see Previous Page)
C.G. of wall, (ft from base)=	17.34	(see Previous Page)
Bottom shell course thickness, ts (in) =	0.260	(see Previous Page)
ratio of D/H =	0.891	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)

Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — $\boxed{D / H = 0.891}$
- K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.577$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1 / W_T = 0.840$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1 / H = 0.417$
- W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2 / W_T = 0.205$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2 / H = 0.766$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 1666.54 kip

VALUES:
- Zone = 3
- Z = 0.30 (Table 24)
- I = 1.25 (Table 26)
- Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
- Ws = pi * 31.17 * 363.42 / 1000 = 35.59 kip
- Xs = 17.34 ft
- Wr = 7.50 kip (engineer estimate, incl wt of shell above HW level)
- Ht = 36.00 ft
- W_1 = 0.840 * W_T = 0.84 * 1666.54 = 1,399.89 kip
- X_1 = 0.417 * H = 0.417 * 35 = 14.595 ft
- Soil Profile type = C (Table 27)
- S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 30%g seismic			CHECKER:	

OVERTURNING MOMENT (cont)...

$W_2 =$ 0.205 * W_T = 0.205 * 1666.54 = 341.64 kip

$X_2 =$ 0.766 * H = 0.766 * 35 = 26.810 ft

Tw = K_p * Sqrt(D) = 3.221 ,(Eq 13-7)

C_1 = 0.0517 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 5,542.3 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 363.42 lbs/ft

wrs = 230.00 lbs/ft

wt = ws + wrs = 593.42 lbs/ft ,(Eq 13-18)

w_L, lbs = 1,396.42 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.250

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 1.581 ,L > 0.035D... **TANK MU**

0.035D (ft) = 1.091

CHECK UPLIFT: If M/(D^2*(wt+w_L))<0.785, then no uplift

If M/(D^2*(wt+w_L))>0.785 and <=1.54, then uplift is OK

If M/(D^2*(wt+w_L))>1.54, then bottom annular ring must be thickened or anchor the tank.

M/(D^2*(wt+w_L)) = 2.867 **UPLIFT OCCURS** ,(Eq 13-15)

THICKEN THE BOTTOM ANNULAR RING or ANCHOR THE TANK !!

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 0.260

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 2517.72 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.254 ,(From Page 1)

ta/R = 0.00136

allowable compressive stress, F_L (psi) = 2,599.4 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 3,465.0 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Horizon View Tank using a 30%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 10910 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 0.891

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)

Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	Forces: H-dynamic convective force, N_c (lb/in)	Forces: H-dynamic impulsive force, N_i (lb/in)	Forces: H-static force, N_h (lb/in)	Stresses: σ_s = Sum H-dynamic Stress** (psi)	Stresses: H-static Stress (psi)	Stresses: Total Stress (psi)	
ring # 6	30.00	0.250	61.1	107.9	405.2	676	1621	2297	**OK**
ring # 5	24.00	0.250	30.2	203.4	891.5	934	3566	4500	**OK**
ring #4	18.00	0.250	15.0	261.5	1377.7	1106	5511	6617	**OK**
ring # 3	12.00	0.255	7.7	282.5	1864.0	1138	7310	8448	**OK**
ring # 2	6.00	0.260	4.5	283.4	2350.2	1107	9039	10146	**OK**
ring # 1	0.00	0.260	3.5	283.4	2836.5	1104	10910	12013	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D*(ZIC_1S/R_w) =$ 1.517 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Horizon View Tank using a 30%g seismic		CHECKER:		

Evaluate Tank Anchors:

M =	5,542.3	ft-kip
wt =	593.42	lbs/ft
D =	31.17	ft

number of anchors to be used = **4**

A_b = anchor area = **0.75** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 24.48 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 163249 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 18000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Nov-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Parksite Tank using a 40%g seismic			CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.058
tank diameter, D (ft) = **93**
max fluid depth, H (ft) = **40.5** Therefore, D/H = 2.296
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)
t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level
D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	40.50				
					40.50			0.00	
					40.50			0.00	
					40.50			0.00	
					40.50			0.00	
					40.50			0.00	
	ring #5	6.50	0.255	6.50	34.00	0.85	7251	67.68	**OK**
	ring #4	8.50	0.355	15.00	25.50	0.85	12020	123.21	**OK**
	ring # 3	8.50	0.465	23.50	17.00	0.85	14376	161.39	**OK**
	ring # 2	8.50	0.630	32.00	8.50	0.85	14449	218.66	**OK**
Bottom	ring # 1	8.50	0.820	40.50	0.00	0.85	14050	284.61	**OK**

Weight of Wall per Foot = 855.56 plf of circumference
Center of Gravity of the Wall = 15.91 feet above base

Weighted average wall thickness, ta = 0.517 inches
Bottom shell ring thickness, ts = 0.820 inches

Bottom Annular Plate Thickness, tb = **0.5** inches

MONTGOMERY WATSON

CLIENT: City of Bellevue	JOB NO: 1060123.021602	DATE: Nov-98	
PROJECT: Seismic Evaluation of Reservoirs	ENGINEER:	TPD	
DESCRIPTION: Parksite Tank using a 40%g seismic	CHECKER:		

TANK INFORMATION:

CAPACITY (Million gal)=	2.058	
tank diameter, D (ft) =	93.00	
max fluid depth, H (ft) =	40.50	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	855.56	(see Previous Page)
C.G. of wall, (ft from base)=	15.91	(see Previous Page)
Bottom shell course thickness, ts (in) =	0.820	(see Previous Page)
ratio of D/H =	2.296	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)

Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:

Z = Seismic Zone Coefficient

I = Importance Factor (use 1.25 UNO)

Rw = Force Reduction Coefficient

Ws = Total Weight of Shell, Lbs

Xs = Height, FT, from bottom of tank to C.G. of shell

Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs

Ht = Height of Tank Shell, FT — $\boxed{D / H = 2.296}$

K_p = Coefficient Relating Tank Size to Period . . . . (Figure 8) $K_p = 0.601$

W_1 = Weight of Tank Contents that Moves with the Shell, Lbs . . . . (Figure 9) $W_1 / W_T = 0.484$

X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 . . . . (Figure 10) $X_1 / H = 0.375$

W_2 = Weight of Sloshing Contents of Tank, Lbs . . . . (Figure 9) $W_2 / W_T = 0.487$

X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 . . . . (Figure 10) $X_2 / H = 0.585$

C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)

S = Site Amplification Factor

Tw = First Mode Sloshing Wave Period

W_T = Total weight of tank contents = $\pi * D^2/4 * H * G * 62.4$ = 17167.04 kip

VALUES: Zone = 4

Z = 0.40 (Table 24)

I = 1.25 (Table 26)

Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5

Ws = pi * 93 * 855.56 / 1000 = 249.97 kip

Xs = 15.91 ft

Wr = 75.00 kip (engineer estimate, incl wt of shell above HW level)

Ht = 41.50 ft

W_1 = 0.484 * W_T = 0.484 * 17167.04 = 8,308.85 kip

X_1 = 0.375 * H = 0.375 * 40.5 = 15.188 ft

Soil Profile type = C (Table 27)

S = 1.50 (Table 27)

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.487 * W_T = 0.487 * 17167.04 = 8,360.35 kip

$X_2 =$ 0.585 * H = 0.585 * 40.5 = 23.693 ft

Tw = K_p * Sqrt(D) = 5.796 ,(Eq 13-7)

C_1 = 0.0223 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 50,570.0 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 855.56 lbs/ft

wrs = 120.00 lbs/ft

wt = ws + wrs = 975.56 lbs/ft ,(Eq 13-18)

w_L, lbs = 4,353.97 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.500

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.939 ,L <= 0.035D, OK

0.035D (ft) = 3.255

CHECK UPLIFT: If M/(D^2*(wt+w_L))<0.785, then no uplift

If M/(D^2*(wt+w_L))>0.785 and <=1.54, then uplift is OK

If M/(D^2*(wt+w_L))>1.54, then bottom annular ring must be thickened or anchor the tank.

M/(D^2*(wt+w_L)) = 1.097 **UPLIFT OCCURS** ,(Eq 13-15)

BOTTOM ANNULAR RING OK (no anchorage required)

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 0.820

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 855.56 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.517 ,(From Page 1)

ta/R = 0.00093

allowable compressive stress, F_L (psi) = 1,692.2 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 2,255.7 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Nov-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Parksite Tank using a 40%g seismic			CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 11943 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 2.296

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)									
			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring #5	34.00	0.255	447.6	675.4	1571.7	4404	6164	10568	**OK**
ring #4	25.50	0.355	339.3	1380.7	3627.0	4845	10217	15062	**OK**
ring # 3	17.00	0.465	269.7	1884.6	5682.3	4633	12220	16853	**OK**
ring # 2	8.50	0.630	230.8	2186.9	7737.6	3838	12282	16119	**OK**
ring # 1	0.00	0.820	218.4	2287.6	9792.9	3056	11943	14999	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

d = 7.53 D*(ZIC_1S/R_w) = 2.603 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Nov-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Parksite Tank using a 40%g seismic			CHECKER:	

Evaluate Tank Anchors:

M =	50,570.0	ft-kip
wt =	975.56	lbs/ft
D =	93	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 24.35 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 157468 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Nov-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Parksite Tank using a 30%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 2.058
tank diameter, D (ft) = **93**
max fluid depth, H (ft) = **40.5** Therefore, D/H = 2.296
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level

D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = 2.60*h_p*D*G / (t*E)

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	40.50				
					40.50			0.00	
					40.50			0.00	
					40.50			0.00	
					40.50			0.00	
					40.50			0.00	
	ring #5	6.50	0.255	6.50	34.00	0.85	7251	67.68	**OK**
	ring #4	8.50	0.355	15.00	25.50	0.85	12020	123.21	**OK**
	ring # 3	8.50	0.465	23.50	17.00	0.85	14376	161.39	**OK**
	ring # 2	8.50	0.630	32.00	8.50	0.85	14449	218.66	**OK**
Bottom	ring # 1	8.50	0.820	40.50	0.00	0.85	14050	284.61	**OK**

Weight of Wall per Foot = 855.56 plf of circumference
Center of Gravity of the Wall = 15.91 feet above base

Weighted average wall thickness, ta = 0.517 inches
Bottom shell ring thickness, ts = 0.820 inches

Bottom Annular Plate Thickness, tb = **0.5** inches

MONTGOMERY WATSON

CLIENT: City of Bellevue	JOB NO: 1060123.021602	DATE: Nov-98	
PROJECT: Seismic Evaluation of Reservoirs	ENGINEER: TPD		
DESCRIPTION: Parksite Tank using a 30%g seismic	CHECKER:		

TANK INFORMATION:

CAPACITY (Million gal)= 2.058
tank diameter, D (ft) = 93.00
max fluid depth, H (ft) = 40.50
fluid specific gravity, G = 1
shell weight (lbs/ft)= 855.56 (see Previous Page)
C.G. of wall, (ft from base)= 15.91 (see Previous Page)
Bottom shell course thickness, ts (in) = 0.820 (see Previous Page)
ratio of D/H = 2.296

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
Z = Seismic Zone Coefficient
I = Importance Factor (use 1.25 UNO)
Rw = Force Reduction Coefficient
Ws = Total Weight of Shell, Lbs
Xs = Height, FT, from bottom of tank to C.G. of shell
Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
Ht = Height of Tank Shell, FT — D / H = 2.296
K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.601$
W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1 / W_T = 0.484$
X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1 / H = 0.375$
W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2 / W_T = 0.487$
X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2 / H = 0.585$
C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
S = Site Amplification Factor
Tw = First Mode Sloshing Wave Period
W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 17167.04 kip

VALUES:
Zone = 3
Z = 0.30 (Table 24)
I = 1.25 (Table 26)
Rw = 4.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
Ws = pi * 93 * 855.56 / 1000 = 249.97 kip
Xs = 15.91 ft
Wr = 75.00 kip (engineer estimate, incl wt of shell above HW level)
Ht = 41.50 ft
W_1 = 0.484 * W_T = 0.484 * 17167.04 = 8,308.85 kip
X_1 = 0.375 * H = 0.375 * 40.5 = 15.188 ft
Soil Profile type = C (Table 27)
S = 1.50 (Table 27)

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Nov-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Parksite Tank using a 30%g seismic	CHECKER:	

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.487 * W_T = 0.487 * 17167.04 = 8,360.35 kip

$X_2 =$ 0.585 * H = 0.585 * 40.5 = 23.693 ft

Tw = K_p * Sqrt(D) = 5.796 ,(Eq 13-7)

C_1 = 0.0223 (Eq 13-5 or 13-6), Tw <4.5, C_1 = 1/(6Tw)

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

M = 37,927.5 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 855.56 lbs/ft

wrs = 120.00 lbs/ft

wt = ws + wrs = 975.56 lbs/ft ,(Eq 13-18)

w_L, lbs = 4,353.97 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.500

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 2.939 ,L <= 0.035D, OK

0.035D (ft) = 3.255

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 0.823 **UPLIFT OCCURS** ,(Eq 13-15)

BOTTOM ANNULAR RING OK (no anchorage required)

Will the tank be anchored ? (yes or no) = **yes**

bottom shell ring#1 thickness, ts (in) = 0.820

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 666.45 ,(Eq 13-14)

Weighted average wall thickness, ta = 0.517 ,(From Page 1)

ta/R = 0.00093

allowable compressive stress, F_L (psi) = 1,692.2 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 2,255.7 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

<u>SHELL COMPRESSION FORCE, OKAY</u>

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Nov-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Parksite Tank using a 30%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 11943 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 2.296

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)

			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring #5	34.00	0.255	335.7	506.5	1571.7	3303	6164	9467	**OK**
ring #4	25.50	0.355	254.5	1035.5	3627.0	3634	10217	13851	**OK**
ring # 3	17.00	0.465	202.2	1413.4	5682.3	3475	12220	15695	**OK**
ring # 2	8.50	0.630	173.1	1640.1	7737.6	2878	12282	15160	**OK**
ring # 1	0.00	0.820	163.8	1715.7	9792.9	2292	11943	14235	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w) =$ 1.952 ft ,(Eq 13-26)

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Nov-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Parksite Tank using a 30%g seismic		CHECKER:		

Evaluate Tank Anchors:

M =	37,927.5	ft-kip
wt =	975.56	lbs/ft
D =	93	ft

number of anchors to be used = **12**

A_b = anchor area = **1.5** in^2

anchor material yield strength = **30000** psi

S_L = anchor spacing = π * D / (number of anchors) = 24.35 ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ 112163 lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 36000 lbs.

ANCHOR TENSION EXCEEDS THE ALLOWABLE, NO GOOD !
(Note: concrete embedment stresses also need to be checked)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Pikes Peak Tank using a 40%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 1.019
tank diameter, D (ft) = **85**
max fluid depth, H (ft) = **24** Therefore, D/H = 3.542
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level

D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	24.00				
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
	ring # 3	8.00	0.260	8.00	16.00	0.85	8000	84.93	OK
	ring # 2	8.00	0.385	16.00	8.00	0.85	10805	125.77	OK
Bottom	ring # 1	8.00	0.490	24.00	0.00	0.85	12735	160.07	OK

Weight of Wall per Foot = 370.77 plf of circumference
Center of Gravity of the Wall = 10.38 feet above base

Weighted average wall thickness, ta = 0.378 inches
Bottom shell ring thickness, ts = 0.490 inches

Bottom Annular Plate Thickness, tb = **0.26** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Pikes Peak Tank using a 40%g seismic	CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	1.019	
tank diameter, D (ft) =	85.00	
max fluid depth, H (ft) =	24.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	370.77	(see Previous Page)
C.G. of wall, (ft from base)=	10.38	(see Previous Page)
Bottom shell course thickness, ts (in) =	0.490	(see Previous Page)
ratio of D/H =	3.542	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)

Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — $\boxed{D/H = 3.542}$
- K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.655$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1/W_T = 0.325$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1/H = 0.375$
- W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2/W_T = 0.633$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2/H = 0.541$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi*D^2/4 *H*G*62.4$ = 8498.13 kip

VALUES:

Zone = 4

Z = 0.40 (Table 24)

I = 1.25 (Table 26)

Rw = 3.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5

Ws = pi * 85 * 370.77 / 1000 = 99.01 kip

Xs = 10.38 ft

Wr = 66.00 kip (engineer estimate, incl wt of shell above HW level)

Ht = 25.00 ft

W_1 = 0.325 * W_T = 0.325 * 8498.13 = 2,761.89 kip

X_1 = 0.375 * H = 0.375 * 24 = 9.000 ft

Soil Profile type = C (Table 27)

S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Pikes Peak Tank using a 40%g seismic		CHECKER:		

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.633 * W_T = 0.633 * 8498.13 = 5,379.32 kip
$X_2 =$ 0.541 * H = 0.541 * 24 = 12.984 ft

Tw = K_p * Sqrt(D) = 6.039 ,(Eq 13-7)
C_1 = 0.0206 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$
Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)
M = 15,462.2 ft-kip

Overturning Checks (Section 13.3.3.3)
where - ws = Weight of Shell (lbs. per ft)
wrs = Weight of Roof acting on the Shell (lbs. per ft)
w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 370.77 lbs/ft
wrs = 120.00 lbs/ft
wt = ws + wrs = 490.77 lbs/ft ,(Eq 13-18)

w_L, lbs = 1,742.88 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C
tb = thickness of bottom plate, (in) = 0.260
L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)
L (ft) = 1.986 ,L <= 0.035D, OK
0.035D (ft) = 2.975

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift
If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK
If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 0.958 **UPLIFT OCCURS** ,(Eq 13-15)
BOTTOM ANNULAR RING OK (no anchorage required)
Will the tank be anchored ? (yes or no) = **no**

bottom shell ring#1 thickness, ts (in) = 0.490
σ_c = Shell Compressive Stress, (psi) = vL) / (0.607 - 0.18667*(M/(D^2*(wt+wL)))^2.3) - wL) / (12*ts) = 571.23 ,(Eq 13-16)
Weighted average wall thickness, ta = 0.378 ,(From Page 1)
ta/R = 0.00074
allowable compressive stress, F_L (psi) = 1,333.9 ,(AWWA D100, Table 10, pg 19)
Earthquake allowable stress, F_L (psi) = 1,778.1 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 10824 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 3.542

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)									
			Forces			Stresses			
Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	H-dynamic convective force, N_c (lb/in)	H-dynamic impulsive force, N_i (lb/in)	H-static force, N_h (lb/in)	σ_s = Sum H-dynamic Stress** (psi)	H-static Stress (psi)	Total Stress (psi)	
ring # 3	16.00	0.260	439.9	914.8	1768.0	5210	6800	12010	**OK**
ring # 2	8.00	0.385	373.3	1463.7	3536.0	4772	9184	13956	**OK**
ring # 1	0.00	0.490	352.0	1646.7	5304.0	4079	10824	14903	**OK**

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D*(ZIC_1S/R_w) =$ 2.825 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:	TPD	
DESCRIPTION:	Pikes Peak Tank using a 40%g seismic		CHECKER:		

Evaluate Tank Anchors: **DISREGARD CALCULATION - ANCHORS ARE NOT SPECIFIED !**

M =	15,462.2	ft-kip
wt =	490.77	lbs/ft
D =	85	ft

number of anchors to be used =

A_b = anchor area = in^2

anchor material yield strength = psi

S_L = anchor spacing = π * D / (number of anchors) = #DIV/0! ft

anchor tension, $T_B = S_L * ((1.273*M / D^2) - wt) =$ #DIV/0! lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = $4/3*(0.6*Fy*A_b) =$ 0 lbs.

#DIV/0!

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Pikes Peak Tank using a 30%g seismic	CHECKER:	

TANK DESCRIPTION: Reservoir
CAPACITY (Million gal) = 1.019
tank diameter, D (ft) = **85**
max fluid depth, H (ft) = **24** Therefore, D/H = 3.542
joint efficiency factor, E = **0.85**
fluid specific gravity, G = **1**

PRELIMINARY TANK WALL ANALYSIS (from AWWA D100-96, Sec 3.7 and Sec 14)

t(h)=2.60*hp*D*G/(s*E) t(h)=thickness (inches) @ height "hp" (feet) from top of HW level
D = diameter in feet
G = specific gravity, 1 for water
s = allowable design unit tensile stress, psi
f = actual shell tensile stress, psi
E = joint efficiency factor,
E = 0.85 ,for AWWA section 3.7
E = 1.00 ,for AWWA section 14

f (actual) = shell plate stress = $2.60*h_p*D*G / (t*E)$

allowable design unit tensile stress per AWWA, s = **15000** psi

	Shell Ring Number	Shell Pl. Ring Height (ft)	Shell wall Thickness (inches)	water h_p from top	water h' from base	Joint efficiency E	Shell Pl. Stress = f (psi)	Shell Wt. per Ring (plf)	
		top of shell wall		0.00	24.00				
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
					24.00			0.00	
	ring # 3	8.00	0.260	8.00	16.00	0.85	8000	84.93	**OK**
	ring # 2	8.00	0.385	16.00	8.00	0.85	10805	125.77	**OK**
Bottom	ring # 1	8.00	0.490	24.00	0.00	0.85	12735	160.07	**OK**

Weight of Wall per Foot = 370.77 plf of circumference
Center of Gravity of the Wall = 10.38 feet above base

Weighted average wall thickness, ta = 0.378 inches
Bottom shell ring thickness, ts = 0.490 inches

Bottom Annular Plate Thickness, tb = **0.26** inches

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs			ENGINEER:	TPD
DESCRIPTION:	Pikes Peak Tank using a 30%g seismic			CHECKER:	

TANK INFORMATION:

CAPACITY (Million gal)=	1.019	
tank diameter, D (ft) =	85.00	
max fluid depth, H (ft) =	24.00	
fluid specific gravity, G =	1	
shell weight (lbs/ft)=	370.77	(see Previous Page)
C.G. of wall, (ft from base)=	10.38	(see Previous Page)
Bottom shell course thickness, ts (in) =	0.490	(see Previous Page)
ratio of D/H =	3.542	

HYDRODYNAMIC LOADING per AWWA STD D100-96. IF RESPONSE SPECTRUM AVAILABLE USE MW SDM

OVERTURNING MOMENT (Section 13.3.3.1)
Refer to AWWA-D100; Figure 8 for determining Kp, and Figures 9 & 10 for W_1, W_2, X_1, X_2

$M = (18ZI/Rw)[\,0.14(Ws{*}Xs + Wr{*}Ht + W_1{*}X_1) + S{*}W_2{*}X_2{*}C_1\,]$,(Equation 13-8)

where:
- Z = Seismic Zone Coefficient
- I = Importance Factor (use 1.25 UNO)
- Rw = Force Reduction Coefficient
- Ws = Total Weight of Shell, Lbs
- Xs = Height, FT, from bottom of tank to C.G. of shell
- Wr = Weight of Tank Roof & Shell above HW level(Incl snow, if req'd, but no LL), Lbs
- Ht = Height of Tank Shell, FT — D / H = 3.542
- K_p = Coefficient Relating Tank Size to Period .... (Figure 8) $K_p = 0.655$
- W_1 = Weight of Tank Contents that Moves with the Shell, Lbs .... (Figure 9) $W_1 / W_T = 0.325$
- X_1 = Height, FT, from bot to centroid of seismic force applied to W_1 .... (Figure 10) $X_1 / H = 0.375$
- W_2 = Weight of Sloshing Contents of Tank, Lbs .... (Figure 9) $W_2 / W_T = 0.633$
- X_2 = Height, FT, from bot to centroid of seismic force applied to W_2 .... (Figure 10) $X_2 / H = 0.541$
- C_1 = Coefficient Relating to Period (Equations 13-5 or 13-6)
- S = Site Amplification Factor
- Tw = First Mode Sloshing Wave Period
- W_T = Total weight of tank contents = $\pi{*}D^2/4$ *H*G*62.4 = 8498.13 kip

VALUES:
- Zone = 3
- Z = 0.30 (Table 24)
- I = 1.25 (Table 26)
- Rw = 3.50 (Table 25) ,unanchored flat bottom tanks = 3.5, anchored flat bottom tanks = 4.5
- Ws = pi * 85 * 370.77 / 1000 = 99.01 kip
- Xs = 10.38 ft
- Wr = 66.00 kip (engineer estimate, incl wt of shell above HW level)
- Ht = 25.00 ft
- W_1 = 0.325 * W_T = 0.325 * 8498.13 = 2,761.89 kip
- X_1 = 0.375 * H = 0.375 * 24 = 9.000 ft
- Soil Profile type = C (Table 27)
- S = 1.50 (Table 27)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO:	1060123.021602	DATE:	Sep-98
PROJECT:	Seismic Evaluation of Reservoirs		ENGINEER:		TPD
DESCRIPTION:	Pikes Peak Tank using a 30%g seismic		CHECKER:		

OVERTURNING MOMENT (cont). . .

$W_2 =$ 0.633 * W_T = 0.633 * 8498.13 = 5,379.32 kip

$X_2 =$ 0.541 * H = 0.541 * 24 = 12.984 ft

Tw = K_p * Sqrt(D) = 6.039 ,(Eq 13-7)

C_1 = 0.0206 (Eq 13-5 or 13-6), Tw <4.5, $C_1 = 1/(6Tw)$

Tw>= 4.5, $C_1 = 0.75/(Tw^2)$

M = (18ZI/Rw)[0.14(Ws*Xs + Wr*Ht + W_1*X_1) + S*W_2*X_2*C_1] ,(Equation 13-8)

M = 11,596.6 ft-kip

Overturning Checks (Section 13.3.3.3)

where - ws = Weight of Shell (lbs. per ft)

wrs = Weight of Roof acting on the Shell (lbs. per ft)

w_L = Weight of Contents which Helps to Resist Overturning ,(Eq 13-12)

ws = 370.77 lbs/ft

wrs = 120.00 lbs/ft

wt = ws + wrs = 490.77 lbs/ft ,(Eq 13-18)

w_L, lbs = 1,742.88 (Eq 13-12): 7.9 tb*SQRT(fy HG) ,not to exceed 1.28 HDG

assumed bottom plate yield, Fy (psi) = 30,000 ,ASTM A283 grade C

tb = thickness of bottom plate, (in) = 0.260

L = Length of Annular Ring = 0.216 tb (sqrt(Fy/(HG))) ,not to exceed 0.035D ,(Eq 13-13)

L (ft) = 1.986 ,L <= 0.035D, OK

0.035D (ft) = 2.975

CHECK UPLIFT: If $M/(D^2*(wt+w_L))<0.785$, then no uplift

If $M/(D^2*(wt+w_L))>0.785$ and <=1.54, then uplift is OK

If $M/(D^2*(wt+w_L))>1.54$, then bottom annular ring must be thickened or anchor the tank.

$M/(D^2*(wt+w_L))$ = 0.719 **NO UPLIFT** ,(Eq 13-15)

BOTTOM ANNULAR RING OK (no anchorage required)

Will the tank be anchored ? (yes or no) = **no**

bottom shell ring#1 thickness, ts (in) = 0.490

σ_c = Shell Compressive Stress, (psi) = (wt + 1.273 * M / D^2) / (12 * ts) = 430.96 (Eq 13-14)

Weighted average wall thickness, ta = 0.378 ,(From Page 1)

ta/R = 0.00074

allowable compressive stress, F_L (psi) = 1,333.9 ,(AWWA D100, Table 10, pg 19)

Earthquake allowable stress, F_L (psi) = 1,778.1 Does not include the increase in the allowable buckling stress due to internal liquid pressure. (See section 13.3.3.7.4)

SHELL COMPRESSION FORCE, OKAY

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Pikes Peak Tank using a 30%g seismic	CHECKER:	

Hoop Stress Calculations

(see sections 13.3.3.6 & 13.3.3.2.3)

maximum Hydrostatic hoop stress (psi) =62.4*H*G*(D/2)/ts = 10824 psi

Hoop stress is below the allowable of s*E = 15000*0.85=12750 - Static Case is OK

Hydrodynamic Hoop Stress, σ_s = (Ni + Nc)/t ,(Eq 13-20)

Ni = Impulsive Hoop Force (lb/in) ,(Eq 13-20, 13-21, or 13-22)

Nc = Convective Hoop Force (lb/in) ,(Eq 13-24)

Nh = Hydrostatic Hoop Force (lb/in)

u''v = Vertical Acceleration

D/H = 3.542

Vertical Acceleration, (decimal) = **0**

Allowable Seismic Hoop Tensile Stress = 1.333*s*E = 1.333*15000*0.85 = 17000 psi

Hydrodynamic + Static Hoop Stress (see section 13.3.3.6)

Shell Ring Number	bottom of ring: height h' from base	Plate Thickness (in)	Forces: H-dynamic convective force, N_c (lb/in)	Forces: H-dynamic impulsive force, N_i (lb/in)	Forces: H-static force, N_h (lb/in)	Stresses: σ_s = Sum H-dynamic Stress** (psi)	Stresses: H-static Stress (psi)	Stresses: Total Stress (psi)	
ring # 3	16.00	0.260	329.9	686.1	1768.0	3908	6800	10708	OK
ring # 2	8.00	0.385	280.0	1097.8	3536.0	3579	9184	12763	OK
ring # 1	0.00	0.490	264.0	1235.0	5304.0	3059	10824	13884	OK

**note: The sum of the hydrodynamic stresses use eq 13-20 when vertical acceleration is zero, and eq 13-25 when vertical acceleration is specified.

Fluid slosh height, d:

$d = 7.53\ D^*(ZIC_1S/R_w) =$ 2.119 ft ,(Eq 13-26)

MONTGOMERY WATSON

CLIENT:	City of Bellevue	JOB NO: 1060123.021602	DATE: Sep-98
PROJECT:	Seismic Evaluation of Reservoirs	ENGINEER:	TPD
DESCRIPTION:	Pikes Peak Tank using a 30%g seismic	CHECKER:	

Evaluate Tank Anchors:

DISREGARD CALCULATION - ANCHORS ARE NOT SPECIFIED !

M =	11,596.6	ft-kip
wt =	490.77	lbs/ft
D =	85	ft

number of anchors to be used =

A_b = anchor area = in^2

anchor material yield strength = psi

S_L = anchor spacing = π * D / (number of anchors) = #DIV/0! ft

anchor tension, $T_B = S_L$ * ((1.273*M / D^2) - wt) = #DIV/0! lbs. ,(Eq 13-19)

anchor allowable tension (with 1/3 stress increase for seismic) = 4/3*(0.6*Fy*A_b) = 0 lbs.

#DIV/0!

MONTGOMERY WATSON

BY TPD DATE 8/27/98 CLIENT City of Bellevue SHEET 1 OF 29
CHKD. BY ____ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE No. 1
Lake Hills North (0.4g Acceleration)

Anchor spacing = 4' ±
No. Anchors = $2\pi(69.5/2)/4$ = 55 Anchors

Anchor Tension $T_B = S_L * ((1.273 M/D^2) - wt)$
$= 4 * ((1.273(153,300/(69.5)^2) - (2.28))$
$= 152.5^k$

Lake Hills North (0.3g Acceleration)

Anchor spacing = 5'6" ±
No. Anchors = $2\pi(69.5/2)/5.5$ = 40 Anchors

Anchor Tension $T_B = 5.5((1.273(114,975/(69.5)^2) - 2.28))$
$= 154.1^k$

Lake Hills South (0.4g Acceleration)

Anchor spacing = 4' ±
No. Anchors = $2\pi(68.0/2)/4$ = 54 Anchors

Anchor Tension $T_B = 4((1.273(148,634/(68)^2) - (2.40))$
$= 154.1^k$

Lake Hills South (0.3g Acceleration)

Anchor Spacing = 5'6" ±
No. Anchors = $2\pi(68/2)/5.5$ = 39 anchors

Anchor Tension $T_B = 5.5((1.273(111,475/(68)^2) - 2.40)$
$= 155.6^k$

Woodridge (0.4g Acceleration)

Anchor Spacing = 4'3" ±
No. Anchors = $2\pi(71/2)/4.25$ = 53 Anchors

Anchor Tension $T_B = 4.25((1.273(138,896/(71)^2) - (2.19))$
$= 139.8^k$

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MONTGOMERY WATSON

BY TPD DATE 8/27/98 CLIENT City of Bellevue SHEET 2 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE NO. 1

Woodridge (0.3g Acceleration)

Anchor Spacing = 5'9" ±

No. Anchors = $2\pi(71/2)/5.75$ = 39 Anchors

Anchor Tension = $5.75((1.273(104,172)/(91)^2) - 2.19)$

= 138.7

Horizon View (0.4g Acceleration)

Anchor Spacing = 2'0" ±

No. Anchors = $2\pi(31.17/2)/2.0$ = 49 Anchors

Anchor Tension = $2((1.273(7390)/(31.175^2) - 0.6)$

= 18.2^k

Horizon View (0.3g Acceleration)

Anchor Spacing = 2'6" ±

No. Anchors = $2\pi(3.17/2)/2.5$ = 39 Anchors

Anchor Tension = $2.5((1.273(5543)/(31.17)^2) - 10.6)$

= 16.7^k

Park Site (0.4g Acceleration)

No. Anchors = 12

Anchor Spacing = $2\pi(93/2)/12$ = 24.35'

Anchor Tension = $24.35((1.273(50,484)/(93)^2) - 0.94)$

= 158.0^k

Park Site (0.3g Acceleration)

No. Anchors = 12

Anchor Spacing = $2\pi(93/2)/12$ = 24.35'

Anchor Tension = $24.35((1.273(37,863)/(93)^2 - 0.94)$

= 112.8k

MONTGOMERY WATSON

BY TPD DATE 8/27/98 CLIENT City of Bellevue SHEET 2A OF 29
CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

Alternate No. 1

① Horizon View (0.4g Accel)

Use 12 Anchors
Anchor Spacing $= \pi(31.17)/12 = 8.16'$

Anchor Tension $= 8.16((1.273(7390)/(31.17)^2) - 0.6)$
$= 74K$

② Horizon View (0.3g Accel)

Use 12 Anchors

Anchor Tension $= 8.16((1.273(5543)/(31.17)^2) - 0.6)$
$= 54K$

Alternate No. 3

③ Horizon View (0.4g Accel)

Anchor Tension $= 8.16((1.273(6490)/(31.17)^2 - 0.6)$
$= 64K$

④ Horizon View (0.3g Accel)

Anchor Tension $= 8.16((1.273(4640)/(31.17)^2 - 0.6)$
$= 45K$

Anchor Length

① 74/5.3 + 10 + 2.5 = 26.5 say 30'
② 54/5.3 + 10 + 2.5 = 22.7 say 25'
③ 64/5.3 + 10 + 2.5 = 24.6 say 25'
④ 45/5.3 + 10 + 2.5 = 21.0 say 20'

MONTGOMERY WATSON

BY TPD DATE 8/27/98 CLIENT City of Bellevue SHEET 3 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE NO. 1

Pikes Peak (0.4 g Acceleration)

No. Anchors = 12

Anchor Spacing = $2\pi(85/2)/12 = 22.25'$

Anchor Tension = $22.25((1.273(15,462)/(85)^2) - 0.37)$

$= 52.4^k$

Pikes Peak (0.3g Acceleration)

No uplift

∴ No anchors required,

MONTGOMERY WATSON

BY TPO DATE 8/27/98 CLIENT City of Bellevue SHEET 4 OF 29
CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE NO. 1
Anchor Chairs

Lake Hills North and South

$P = 155^k$
Assume $1^{3}/_{8}$" Dywidag thread bar anchor.

Top Plate thickness:

$$C = \left[\frac{P}{Sf}\left(0.375g - 0.22d\right)\right]^{1/2}$$

$$C = \left[\frac{155}{25(4.19)}\left(0.375(3) - 0.22(1.375)\right)\right]^{1/2}$$

$C = 1.10\,in$ use $1^{1}/_{4}$"

$d = 1.375\,in$
$S = 25\,KSI$
$g = 3\,in$
$g_{min} = d/2 + 1/8 = 13/16 + 1/8 = 1^{1}/_{16}\,in$
use 8" Plate
$\therefore f = 8 - 3 - 13/16 = 4.19$"
distance to ℄ hole from tank Plate

Chair Height

$$S = \frac{Pe}{t^2}\left[\frac{1.32z}{\frac{1.43ah^2}{Rt} + (4ah^2)^{1/3}} + \frac{0.31}{\sqrt{Rt}}\right]$$

$$z = \frac{1.0}{\frac{0.177am}{\sqrt{Rt}}(m/t)^2 + 1}$$

$$25 = \frac{155(3)}{(1.065)^2}\left[\frac{1.32(0.365)}{\frac{1.43(4.5)(h^2)}{417(1.065)} + (4(4.5)(h^2))^{1/3}} + \frac{0.31}{\sqrt{417(1.065)}}\right]$$

$a = 4.5\,in$
$m = 0.49\,in$
$R = 69.5/2 = 34.75\,ft$
$= 417\,in$
$t = 1.065$

$$25 = 409.97\left[\frac{0.48}{0.0145(h^2) + (18h^2)^{1/3}} + 0.0147\right]$$

$$25 = \frac{196.78}{0.0145(h)^2 + (18h^2)^{1/3}} + 6.03$$

$$z = \frac{1.0}{\frac{0.177(4.5)(0.49)}{\sqrt{417(1.065)}}\left(\frac{0.49}{1.065}\right)^2 + 1}$$

$$18.97\left[0.0145(h)^2 + (18h^2)^{1/3}\right] = 196.8$$

$z = 0.365$

$$0.275h^2 + 18.97(18h^2)^{1/3} = 196.8$$

$e = 3\,in$

Try 8" chair height
$= 216 > 165$ ok

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BY TPD DATE 8/27/98 CLIENT City of Bellevue SHEET 5 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE NO. 1
Anchor Chairs

Vertical Side Plates

$j_{min} = \frac{1}{2}\,in$

$0.04(h-e) = 0.04(8-3) = 0.2\,in$

$\frac{P}{25k} = \frac{15k}{25(5)} = 1.25$ use $1\frac{1}{4}$ Plate

$k = \frac{1}{2}(8+2) = 5$

Woodridge

Use same chair as Lake Hills north and south

Horizon View

$P = 18^{k}$

Assume 1" Ø Adhesive anchor

Top Plate thickness:

$$C = \left[\frac{P}{Sf}\left(0.375g - 0.22d\right)\right]^{\frac{1}{2}}$$

$$C = \left[\frac{18}{25(3)}\left(0.375(3) - 0.22(1)\right)\right]^{\frac{1}{2}}$$

$C = 0.47\,in$ use $\frac{1}{2}\,in$ plate

$d = 1\,in$
$S = 25\,ksi$
$g = 3\,in$
$f_{min} = 5/8\,in$
Try 5 in Plate
$f = 5 - 1\frac{1}{2} - \frac{1}{2} = 3\,in$

Chair Height

$$S = \frac{P_e}{t^2}\left[\frac{1.32\,z}{\frac{1.43\,a h^2}{Rt} + (4ah^2)^{1/3}} + \frac{0.31}{\sqrt{RT}}\right]$$

$$z = \frac{1.0}{\frac{0.177am}{\sqrt{Rt}}(m/t)^2 + 1}$$

$$25 = \frac{18(1.5)}{(0.26)^2}\left[\frac{1.32(0.979)}{\frac{1.43(3.5)(h^2)}{187(0.26)} + (4(3.5)(h)^2)^{1/3}} + \frac{0.31}{\sqrt{187(0.26)}}\right]$$

$a = 3.5\,in$
$m = 0.25\,in$
$R = 31.17(12/2) = 187\,in$
$t = 0.26\,in$

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ALTERNATE NO. 1
Anchor Chairs

$$25 = 399.4\left[\frac{1.29}{0.103h^2 + (14h^2)^{1/3}} + 0.00445\right]$$

$$Z = \frac{1.0}{\frac{0.177(3.5)(0.25)}{\sqrt{174(0.26)}}\left(\frac{0.25}{0.26}\right)^2 + 1}$$

$$25 = \frac{515.0}{0.103h^2 + (14h^2)^{1/3}} + 1.78$$

$$Z = 0.979$$

$$23.22\left(0.103h^2 + (14h^2)^{1/3}\right) = 515.0$$

$$2.39(h)^2 + 23.22(14h^2)^{1/3} = 515.0$$

Try $h = 10.5$ in

$531.8 > 515.0$ ok

Vertical Side Plates

$$t_{min} = \frac{1}{2} \text{ in}$$

$$0.04(h-c) = 0.04(9.5-1.5) = 0.32 \text{ in}$$

$$\frac{P}{25k} = \frac{18}{25(3.5)} = 0.21 \text{ in}$$

$$k = (5+2)\tfrac{1}{2} = 3.5 \text{ in}$$

Use $\frac{1}{2}$ in

Woodridge

Use same as Lake Hills North and South

Parkrite

Use same as Lake Hills North and South

Part VII
Anchor Bolt Chairs

When anchor bolts are required at supports for a shell, chairs are necessary to distribute the load to the shell. Small tubular columns (less than 4 ft in diameter) may be an exception if the base plate is adequate to resist bending. Otherwise, chairs are always needed to minimize secondary bending in the shell.

For flat-bottom tanks, choose a bolt circle to just barely clear the bottom without notching it. For other structures, follow the minimum clearances shown in Fig. 7-1*a*. The designer must evaluate anchor bolt location for interference with base or bottom plate.

Notation

a = top-plate width, in., along shell
b = top-plate length, in., in radial direction
c = top-plate thickness, in.
d = anchor-bolt diameter, in.
e = anchor-bolt eccentricity, in.
e_{min} = $0.886d + 0.572$, based on a heavy hex nut clearing shell by 1/2 in. See Table 7-1
f = distance, in., from outside of top plate to edge of hole
f_{min} = $d/2 + 1/8$
g = distance, in., between vertical plates (preferred $g = d + 1$) [Additional distance may be required for maintenance.]
h = chair height, in.
j = vertical-plate thickness, in.
k = vertical-plate width, in. (average width for tapered plates)
L = column length, in.
m = bottom or base plate thickness, in.
P = design load, kips; or maximum allowable anchor-bolt load or 1.5 times actual bolt load, whichever is less
r = least radius of gyration, in.
R = nominal shell radius, in., either to inside or centerline of plate (radius normal to cone at bottom end for conical shells)
S = stress at point, ksi
t = shell or column thickness, in.
w = weld size (leg dimension), in.
W = total load on weld, kips per lin. in. of weld
W_H = horizontal load, kips per lin. in. of weld
W_V = vertical load, kips per lin. in. of weld
θ = cone angle, degrees, measured from axis of cone
Z = reduction factor

Top Plate

Critical stress in the top plate occurs between the hole and the free edge of the plate. For convenience we can consider this portion of the top plate as a beam with partially fixed ends, with a portion of the total anchor bolt load distributed along part of the span. See Fig. 7-2.

$$S = \frac{P}{fc^2}(0.375g - 0.22d) \qquad (7\text{-}1)$$

or

$$c = \left[\frac{P}{Sf}(0.375g - 0.22d)\right]^{1/2} \qquad (7\text{-}2)$$

Top plate may project radially beyond vertical plates as in Fig. 7-1*d*, but no more than 1/2″.

Chair Height

Chair must be high enough to distribute anchor bolt load to shell or column without overstressing it. If the anchor bolt were in line with the shell the problem would be simple — the difficulty lies in the bending caused by eccentricity of the anchor bolt with respect to the shell. Except for the case where a continuous ring is used at the top of chairs, maximum stress occurs in the vertical direction and is a combination of bending plus direct stress. Formulas which follow are approximations, based on the work of Bjilaard.

$$S = \frac{Pe}{t^2}\left[\frac{1.32\,Z}{\frac{1.43\,ah^2}{Rt} + (4ah^2)^{.333}} + \frac{.031}{\sqrt{Rt}}\right] \qquad (7\text{-}3)$$

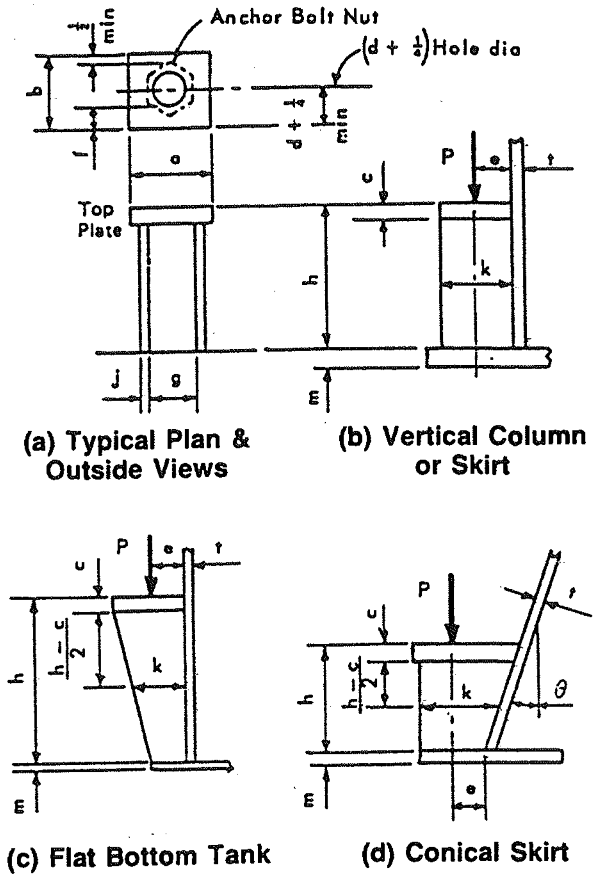

Figure 7-1. Anchor-Bolt Chairs.

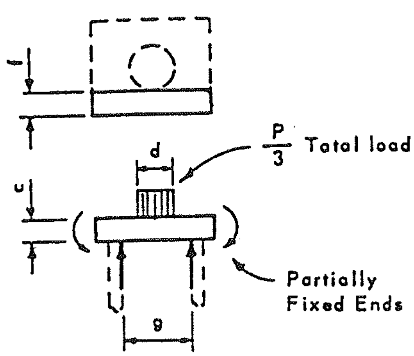

Figure 7-2. Assumed Top-Plate Beam.

$$\text{Where: } Z = \frac{1.0}{\frac{.177\,am}{\sqrt{Rt}}\left(\frac{m}{t}\right)^2 + 1.0} \qquad (7\text{-}4)$$

Maximum recommended stress is 25 ksi. This is a local stress occurring just above the top of the chair. Since it diminishes rapidly away from the chair, a higher than normal stress is justified but an increase for temporary loads, such as earthquake or wind is not recommended. The following general guidelines are recommended.

Minimum chair height $h=6''$, except use $h=12''$ when base plate or bottom plate is 3/8" or thinner

Table 7-1. Top-Plate Dimensions

Based on anchor-bolt stresses up to 12 ksi for 1½-in.-dia. bolts and 15 ksi for bolts 1¾ in. in diameter or larger; higher anchor bolt stresses may be used subject to designer's decision.

Top Plate Dimensions, in.						Bolt Load, kips
d	f	$g=d+1$	a	e_{min}	c_{min}	P
1½	⅞	2½	4½	1.87	0.734	19.4
1¾	1	2¾	4¾	2.09	0.919	32.7
2	1⅛	3	5	2.30	1.025	43.1
2¼	1¼	3¼	5¼	2.52	1.145	56.6

and where earthquake or winds over 100 mph must be considered.

Maximum recommended chair height $h=3a$.

If chair height calculated is excessive, reduce eccentricity e, if possible, or use more anchor bolts of a smaller diameter. Another solution is to use a continuous ring at top of chairs.

If continuous ring is used, check for maximum stress in circumferential direction, considering the ring as though it were loaded with equally spaced concentrated loads equal to Pe/h. Portion of shell within $16t$ either side of the attachment may be counted as part of the ring. (Refer to Fig. 7-3)

Note that the base plate or bottom is also subjected to this same horizontal force, except inward instead of outward. This is true even if a continuous ring is not used around the top of the chairs — but it should never cause any very high stresses in the base, so we do not normally check it. However, it is a good thing to keep in mind in case you have a very light base ring.

Vertical Side Plates

Be sure top plate does not overhang side plate (as in Fig. 7-1d) by more than 1/2" radially.

Vertical-plate thickness should be at least j_{min} = 1/2" or 0.04 $(h-c)$, whichever is greater.

Another requirement is $jk \geq P/25$, where k is the average width if plate is tapered.

These limits assure a maximum L/r of 86.6 and a maximum average stress in the side plates of 12.5

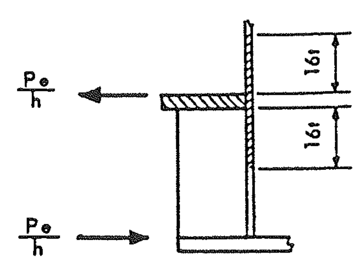

Figure 7-3. Chair with Continuous Ring at Top.

ksi, even assuming no load was transmitted into the shell through the welds.

Assembly of Chair

For field erected structures, ship either the top plate or the entire chair loose for installation after the structure is sitting over the anchor bolts.

Where base plate is welded to skirt or column in shop, attach side plates in the shop and ship top plate loose for field assembly. See Fig. 7-4.

Where base or bottom plate is not welded to shell in the shop, as for flat-bottom tanks and single pedestal tanks, shop attach side plates to top plates and then ship the assembly for field installation. When you do this, weld both sides at top of side plates so shrinkage will not pull side plate out of square. See Fig. 7-5.

Welds between chair and shell must be strong enough to transmit load to shell. ¼" minimum fillet welds as shown in Figs. 7-4 and 7-5 are nearly always adequate, but you should check them if you have a large anchor bolt with a low chair height. Seal welding may be desired for application in corrosive environments.

Assume a stress distribution as shown in Fig. 7-6 as though there were a hinge at bottom of chair. For the purpose of figuring weld size, the base or bottom plate is assumed to take horizontal thrust only, not moment.

Note that loads are in terms of kips per inch of weld length, not in terms of kips per square inch stress. Critical stress occurs across the top of the chair. The total load per inch on the weld is the resultant of the vertical and horizontal loads.

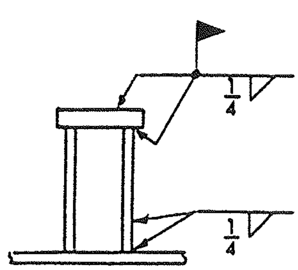

Figure 7-4. Typical Welding, Base Plate Shop Attached.

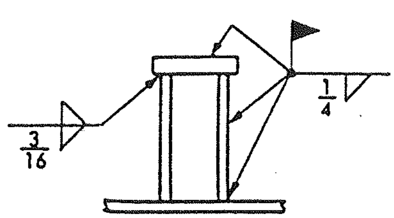

Figure 7-5. Typical Welding, Base or Bottom Field Attached.

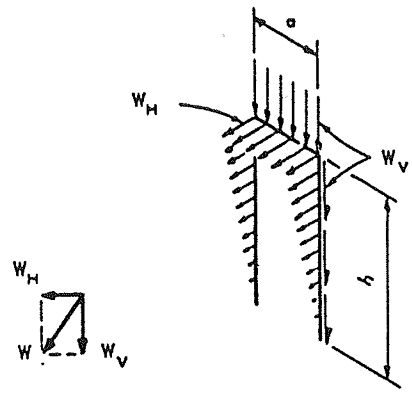

Figure 7-6. Loads on Welds.

Formulas may also be used for cones, although this underrates the vertical welds some.

$$W_v = \frac{P}{a + 2h} \tag{7-5}$$

$$W_H = \frac{Pe}{ah + 0.667h^2} \tag{7-6}$$

$$W = \sqrt{W_V^2 + W_H^2} \tag{7-7}$$

For an allowable stress of 13.6 ksi on a fillet weld, the allowable load per lin. in. is 13.6 × 0.707 = 9.6 kips per in. of weld size. For weld size *w*, in., the allowable load therefore is

$$9.6w \geq W \tag{7-8}$$

Design References

H. Bednar, "Pressure Vessel Design Handbook", 1981, pp. 72-93.

M.S. Troitsky, "Tubular Steel Structures", 1982, pp. 5-10 — 5-16.

P.P. Bjilaard, "Stresses From Local Loadings In Cylindrical Pressure Vessels," ASME Transactions, Vol. 77, No. 6, 1955.

P. Buthod, "Pressure Vessel Handbook," 7th Edition, pp. 75-82.

MONTGOMERY WATSON

BY TPD DATE 9/14/98 CLIENT City of Bellevue SHEET 7 OF 39
CHKD. BY ____ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE No. 1
Soil Anchors:

Lake Hills North and South (0.4g and 0.3g Acceleration)

Anchor Tension = 155 K
From HWA GeoSciences report, allowable load per foot
= 5.3K excluding top 10 feet below footing

∴ Length = 155/5.3 + 10 = 39.2
since footing is 4'-6" deep use total length of 40'-0"

Woodridge (For 0.4g and 0.3g Acceleration)

Anchor Tension = 140 K
From HWA Geosciences report, allowable load per foot
= 4.7 K excluding top 10 feet below footing

∴ total Length = 140/4.7 + 14.5 = 44.3' use 45'-0"

Horizon View (For 0.4g and 0.3g Acceleration)

Anchor Tension = 18 K
From HWA Geoscience report, allowable load per foot
= 5.3 K excluding top 10 feet below footing

∴ Total length = 18/5.3 + 10 + 2.5 = 15.9 use 18'-0"

Park Site (For 0.4g acceleration)

Anchor Tension = 158 K
Per Dan Campbell on 10/8/98 use 5.3 K/ft.

∴ Total length = 158/5.3 + 10 + 3 = 42.8 use 45'-0"
0.3g ∴ l = 113/5.3 + 13 = 34.3 use 35'

Pikes Peak (For 0.4g acceleration)

Anchor Tension = 53 K

use length of 53/5.3 + 10 + 2.5 = 22.5 say 25'-0"

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MONTGOMERY WATSON

BY TPD DATE 9/14/98 CLIENT City of Bellevue SHEET 8 OF 29

CHKD. BY ________ DESCRIPTION Foundation Bearing Pressure JOB NO. 1060123

ALTERNATE NO. 1

Lake Hills North (0.4g Acceleration)

$M_{max} = 153,300 \text{ K-Ft}$

$S = 0.098175 \left(\frac{d^4 - d_1^4}{d} \right)$

$= 0.098175 \left(\frac{(71.00)^4 - (64.00)^4}{71.00} \right)$

$= 11939 \text{ Ft}^3$

$fb = \frac{P}{A} + \frac{MC}{I}$ $\quad P = 2.05 \text{ KLF} + 0.15(4.5)(3.5) = 4.41 \text{ KLF}$

$A = 1(3.5) = 3.5 \text{ ft}^2$

$fb = \frac{4.41}{3.5} + \frac{153,000}{11939} = 14.08 \text{ KSF} > 8.0(1.5) = 12 \text{ KSF}$ No Good

Try adding 1'-0" ring around outside of existing footing

$S = 0.098175 \left(\frac{(73.00)^4 - (64.00)^4}{73.00} \right)$

$S = 15629 \text{ Ft}^3$

$fb = \frac{P}{A} + \frac{M}{S}$ $\quad P = 2.05 + 0.15(4.5)(4.5) = 5.09 \text{ KLF}$

$A = 1(4.5) = 4.5 \text{ Ft}^2$

$= \frac{5.09}{4.50} + \frac{153,300}{15629} = 10.94 \text{ KSF} < 12.0 \text{ KSF}$

Lake Hills North (0.3g Acceleration)

$M_{max} = 114,975 \text{ K-Ft}$

$S = 11939 \text{ ft}^3$

$fb = \frac{4.41}{3.5} + \frac{114,975}{11939} = 10.89 \text{ KSF} < 12 \text{ KSF}$ ok

MONTGOMERY WATSON

BY TPD DATE 9/14/98 CLIENT City of Bellevue SHEET 9 OF 29

CHKD. BY ______ DESCRIPTION Foundation Bearing Pressure JOB NO. 1060123

ALTERNATE NO. 1

Lake Hills South (0.4g Acceleration)

$M_{max} = 148,634$ K-FT

Try 1'-0" ring around outside of existing footing

$$S = 0.098175\left(\frac{(71.50)^4 - (62.5)^4}{71.50}\right)$$

$S = 14,934\ FT^3$

$$f_b = \frac{P}{A} + \frac{M}{S} \qquad P = 2.17 + 0.15(4.5)(4.5) = 5.21\ \text{KSF}$$

$$= \frac{5.21}{4.50} + \frac{148,634}{14934} = 11.11\ \text{KSF} \leq 12.00\ \text{KSF OK}$$

Lake Hills South (0.3g Acceleration)

$M_{max} = 111,475$ K-FT

$$S = 0.098175\left(\frac{(69.5)^4 - (62.5)^4}{69.5}\right)$$

$S = 11403\ FT^3$

$$f_b = \frac{P}{A} + \frac{M}{S} \qquad P = 2.17 + 0.15(4.5)(3.5) = 4.53\ \text{KSF}$$
$$A = 3.5(1) = 3.5\ FT^2$$

$$f_b = \frac{4.53}{3.50} + \frac{111475}{11403} = 11.07\ \text{KSF}$$

MONTGOMERY WATSON

BY TPD DATE 9/14/98 CLIENT City of Bellevue SHEET 10 OF 29
CHKD. BY ______ DESCRIPTION Foundation Bearing Pressure JOB NO. 1060123

ALTERNATE NO-1
Woodridge (0.4g acceleration)

$M_{max} = 138,896$ K-Ft

$S = 0.098175 \left(\frac{(72.50)^4 - (65.5)^4}{72.50} \right)$

$= 12488 \text{ Ft}^3$

$f_b = \frac{P}{A} + \frac{M}{S}$

$P = 1.96 + 0.15(3.5)(4.5) = 4.32$ KLF
$A = 3.5 \text{ Ft}^2$

$f_b = \frac{4.32}{3.5} + \frac{138,896}{12488} = 12.36$ KSF $\gg 5.0(1.5) = 7.5$ KSF

Try 2'-6" ring around outside of existing footing

$S = 0.098175 \left(\frac{(77.50)^4 - (65.5)^4}{77.50} \right)$

$S = 22382 \text{ Ft}^3$

$f_b = \frac{P}{A} + \frac{M}{S}$

$P = 1.96 + 0.15(5.5)(4.5) = 5.67$ KLF
$A = 5.5 \text{ Ft}^2$

$= \frac{5.67}{5.5} + \frac{138,896}{22382} = 7.23$ KSF < 7.5 KSF ok

Woodridge (0.3g acceleration)

Try 1'-6" ring around outside of existing footing

$S = 0.098175 \left(\frac{(74.50)^4 - (65.50)^4}{74.50} \right)$

$= 16339 \text{ Ft}^3$

$f_b = \frac{P}{A} + \frac{M}{S}$

$P = 1.96 + 0.15(4.5)^2 = 5.00$ KLF
$A = 4.5 \text{ Ft}^2$

$= \frac{5.0}{4.5} + \frac{104172}{16339} = 7.49$ KSF < 7.5 KSF ok

MONTGOMERY WATSON

BY TPD DATE 9/14/98 CLIENT City of Bellevue SHEET 11 OF 29
CHKD. BY ______ DESCRIPTION Foundation Bearing Pressure JOB NO. 1060123

ALTERNATE NO. 1

Horizon View (0.4g Acceleration)

$M_{max} = 7390$ K-Ft

$S = 0.098175 \left(\frac{(32.0)^4 - (30.33)^4}{32.0} \right)$

$= 621\ Ft^3$

$f_b = P/A + M/S$

$P = 0.36 + 0.83(0.15)(2.5) = 0.67$ KLF

$A = 0.83\ Ft^2$

$= \frac{0.67}{0.83} + \frac{7390}{621} = 12.70$ KSF N.G.

Try 1'-0" ring around outside of existing footing

$S = 0.098175 \left(\frac{(34.0)^4 - (30.33)^4}{34.0} \right)$

$= 1402\ Ft^3$

$f_b = P/A + M/S$

$P = 0.36 + 1.83(0.15)(2.5) = 1.05$ KLF

$A = 1.83\ Ft^2$

$= \frac{1.05}{1.83} + \frac{7390}{1402} = 5.84$ KSF < 6.0 KSF

Horizon View (0.3g Acceleration)

By inspection use 1'-0" ring around outside of exist footing

EN 1S (10/78)

MONTGOMERY WATSON

BY TPD DATE 9/14/98 CLIENT City of Bellevue SHEET 12 OF 28

CHKD. BY ______ DESCRIPTION Foundation Bearing Pressure JOB NO. 1060123

ALTERNATE NO. 1

Parksite: (0.4g Acceleration)

$M_{max} = 50,484$ K-Ft

$S = 0.098175 \left(\frac{(94.5)^4 - (91.5)^4}{94.5} \right)$

$= 10,030 \text{ in}^3$

$f_b = \frac{P}{A} + \frac{M}{S}$ $P = 0.82 + 0.15(2.5)(1.5) = 1.38$

$A = 1.5 \text{ Ft}^2$

$= \frac{1.38}{1.5} + \frac{50,484}{10,030} = 5.95 \text{ KSF} < 7.5 \text{ KSF}$

Parksite: (0.3g Acceleration)

This will be ok by inspection

Pikes Peak: (0.4g Acceleration)

$M_{max} = 15,462$ K-Ft

$S = 0.098175 \left(\frac{(93.5)^4 - (92.5)^4}{93.5} \right)$ $P = 0.37 + 0.15(1.0)(2.5) = 0.75$

$= 3378$

$f_b = \frac{0.75}{1} + \frac{15465}{3378} = 5.3 \text{ KSF} < 7.5 \text{ KSF}$

Pikes Peak (0.3g Acceleration)

$f_b = 0.75 + \frac{11,597}{3378} = 3.93 \text{ KSF} < 7.5 \text{ KSF}$

EN 1S (10/78)

MONTGOMERY WATSON

BY TPD DATE 9/17/98 CLIENT City of Bellevue SHEET 13 OF 23

CHKD. BY _______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE NO. 2

Lake Hills North (0.4g acceleration)

Anchor spacing = 1'6" ±

No. Anchors = $2\pi(69.5/2)/1.5 = 146$ Anchors

Anchor Tension $T_B = 1.5((1.273(153,300)/(69.5)^2) - (2.28))$
$= 57.2^k$

Use 2" ∅ Threaded rod. Capacity $= 20(\pi)(2/2)^2 = 62.8^k > 57.2^k$ ok

Lake Hills North (0.3g Acceleration)

Anchor spacing = 2'0" ±

No. Anchors $= 2\pi(69.5/2)/2.0 = 109$ Anchors

Anchor Tension $= T_B = 2.0((1.273(114,975/(69.5)^2) - 2.28))$
$= 56.0^k$

∴ Use 2" ∅ THREADED ROD

Lake Hills South (0.4g acceleration)

Anchor spacing = 1'6" ±

No Anchors $= 2\pi(68/2)/1.5 =$ say 144 anchors

Anchor Tension $T_B = 1.5((1.273(144,634/(68)^2) - (2.40))$
$= 57.8 < 62.8^k$

Use 2" ∅ THREADED ROD

Lake Hills South (0.4g Acceleration)

Anchor spacing = 2'0" ±

No. Anchors $= 2\pi(68/2)/2.0 = 107$ anchors

Anchor Tension $T_B = 2.0((1.273(111,475/(68)^2) - 2.40)$
$= 56.6$

Use 2" ∅ THREADED ROD

MONTGOMERY WATSON

BY TPD DATE 9/17/98 CLIENT City of Bellevue SHEET 14 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

ALTERNATE NO. 2

Woodridge (0.4g Acceleration)

Anchor spacing = 1'9" ±

No. Anchors = $\pi(71)/1.75 \approx 128$ anchors

Anchor Tension = $1.75((1.273(138,896/(71)^2) - 2.19)$
$= 57.5^k$

∴ Use 1 1/2" ⌀ THREADED ROD

Woodridge (0.3g acceleration)

Anchor spacing = 2'6"

No. Anchors = $\pi(71)/2.5 = 90$ anchors

Anchor Tension = $2.5((1.273(104,172)/(71)^2 - 2.19)$
$= 60.3^k$

∴ Use 1 1/2" ⌀ THREADED ROD

Horizon View (0.4g Acceleration)

See Sheet No. 2

Horizon View (0.3g Acceleration)

See sheet No. 2

Parksite (0.4g Acceleration)

No. Anchors = 36

Anchor Spacing = $2\pi(93/2)/36 = 8.12'$

Anchor Tension = $8.12((1.273(50,484)/(93)^2) - 0.941)$
$= 52.7^k$

Use 2" ⌀ Anchor, Capacity = $20(\pi)(2/2)^2 = 628^k$

Parksite (0.3g Acceleration)

"Use same Anchor as above"

Pikes Peak (0.4g) use same anchors as Horizon View

EN 1S (10/78)

MONTGOMERY WATSON

BY TPD DATE 9/17/98 CLIENT City of Bellevue SHEET 15 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

Alternate No. 2

Anchor Chairs:

Lake Hills North, Lake Hills South and Woodridge (0.4g and 0.3g Accel)

$P = 58^k$

Assume 2" Ø THREADED ROD

Top Plate THICKNESS:

$$C = \left[\frac{P}{Sf}\left(0.375g - 0.22d\right)\right]^{1/2}$$

d = 2.0 in
S = 25 KSI
g = 3 in
f = 6 - 3 - 1 = 2.0
using 6" Plate

$$C = \left[\frac{58}{25(2.0)}\left(0.375(3) - 0.22(2.0)\right)\right]^{1/2}$$

C = 0.891 use 1" PLATE

Chair Height

$$S = \frac{Pe}{t^2}\left[\frac{1.32\,Z}{\frac{1.43ah^2}{Rt} + (4ah^2)^{1/3}} + \frac{0.31}{\sqrt{Rt}}\right]$$

$$Z = \frac{1.0}{\frac{0.177am}{\sqrt{Rt}}\left(m/t\right)^2 + 1}$$

$$25 = \frac{58(3)}{(1.115)^2}\left[\frac{1.32(0.997)}{\frac{1.43(4)h^2}{408(1.115)} + (4(4)h^2)^{1/3}} + \frac{0.31}{\sqrt{408(1.115)}}\right]$$

a = 4 in
m = 0.49 in
R = 408 in
t = 1.115

$$25 = 140.0\left[\frac{1.32}{0.0126h^2 + (16h^2)^{1/3}} + 0.0145\right]$$

$$Z = \frac{1.0}{\frac{0.177(4)(0.49)}{\sqrt{408(1.115)}}\left(\frac{0.49}{1.115}\right)^2 + 1}$$

$$25 = \frac{184.8}{0.0126h^2 + (16h^2)^{1/3}} + 2.03$$

Z = 0.997
e = 3 in

$$0.289h^2 + 22.97(16h^2)^{1/3} = 184.8$$

Try 7" chair ht.
226.0 > 184.8 ok

EN 1S (10/78)

MONTGOMERY WATSON

BY TPD DATE 9/17/98 CLIENT City of Bellevue SHEET 16 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

Alternate No. 2

Anchor Chairs

Vertical side Plates:

$t_{min} = \frac{1}{2} in = \frac{1}{2} in$

$0.04(h-e) = 0.04(4) = 0.16 in$

$\frac{P}{25k} = \frac{58}{25(4)} = 0.58 in$ use 5/8"

$k = \frac{1}{2}(6+2) = 4$

Park Site:

Use same chair as Lake Hills N+S and Woodridge

Alternate No. 2

Find Additional Weight of Water Column plus foundation concrete to get no uplift or

$M/(D^2)(wt + wl + wc) < 0.785$ or

Lake Hills North (0.4 g Acceleration)

$$\frac{M}{D^2(wt + wl + wc)} < 0.785$$

$$\frac{M}{0.785(D)^2} < wt + wl + wc$$

$wl = 0.0625(H)(dw) = 4.69\,dw$
$wc = 0.15(4.5)(0.75 + dw)$
$= 0.675(0.75 + dw)$

$$\frac{153{,}300}{0.785(69.5)^2} < 2.28 + wl + wc)$$

$wl + wc = 38.1^k$
$4.69\,dw + 0.675(0.75 + dw) = 38.1^k$
$0.51 + 5.37\,dw = 38.1$
$dw = 7.0\ ft$

Additional Conc $= 7.0 - 3.5 = 3.5'$ say $3'6''$

Check Moment to be transfered into footing.

$$M_{ULT} = 1.4\left(0.15\frac{(4.5)}{2}(7)^2(12)\right) + 1.7\left(\frac{0.0625)(75)}{2}(7)^2(12)\right)$$

$= 278 + 2343$
$= 2621$ k-in

Capacity of #4 @ 3'6" stirrups $\qquad d = 54 - 3 - 1/4 = 50.75$

$$c = \frac{0.2(\frac{1}{3})(60)}{(0.85)^2 4(12)} = 0.12\ in$$

$0.9M_n = 0.9(0.2/3)(60)(50.75 - 0.42(0.12))$
$= 183$ k-in $<< 2621$ k-in N.G.

Note: This condition will exist for Lake Hills North (0.3g), Lake Hills South (0.4g and 0.3g) and Woodridge (0.4g and 0.3g)

MONTGOMERY WATSON

BY TPD DATE 9/17/98 CLIENT City of Bellevue SHEET 18 OF 28

CHKD. BY ______ DESCRIPTION Additional Concrete JOB NO. 1060123

Alternate No. 2

Horizon View (0.4 g Acceleration)

$$wl + wc > \frac{M}{D^2(0.785)} - wt$$

$$wl + wc > \frac{7390}{(31.17)^2(0.785)} - 0.59$$

$wl + wc > 9.1^k$

$2.19\,dw + 0.3\,dw + 0.13 > 9.1$

$2.49\,dw > 9.0$

$dw = 3.6$

$wl = 0.0625(35)(dw) = 2.19\,dw$

$wc = 0.15(2.0)(0.42 + dw) = 0.3(0.42 + dw)$

Additional Concrete = 3.6 − 0.4 = 3.2 say 3'3"

Check Moment to be transfered into footing

$$M_{ULT} = 1.4\left[0.15(2)\frac{(3.67)^2}{2}(12)\right] + 1.7\left[(0.0625)\frac{(35)}{2}(3.67)^2(12)\right]$$

$= 34 + 301$

$= 335$ k-in

Check steel required

Try #5 @12"

$d = 24 - 3 - 5/16 = 20.69$ in

$$C = \frac{0.31(60)}{(0.85)^2(4)(12)} = 0.54\text{ in}$$

$0.9 M_n = 0.9(60)(0.31)(20.69 - 0.42(0.54))$

$= 342$ k-in ok

However, you could not develop a #5 bar in a 10" wall so this is no good.

Horizon view (0.3g Acceleration)

$$wl + wc = \frac{5543}{(31.17)^2(0.785)} - 0.59 = 6.7$$

$2.49\,dw = 6.6^k$; $dw = 2.65'$ Additional conc = 2.23 say 2'3"

Check Moment to be transfered into footing

$$M_{ULT} = 1.4\left[0.15(2)\frac{(2.67)^2}{2}(12)\right] + 1.7\left[(0.0625\frac{(35)}{2}(2.67)^2(12)\right]$$

$= 18 + 159 = 177$ k-in

If correct spacing is use a #4 bar could possibly be developed in a 10" wall

MONTGOMERY WATSON

BY TPD DATE 9/17/98 CLIENT City of Bellevue SHEET 19 OF 29
CHKD. BY ______ DESCRIPTION Additional Concrete JOB NO. 1060123

Alternate No. 2

Park site (0.4g acceleration)

$$w_\ell + w_c > \frac{M}{D^2(0.785)} - w_t$$

$w_\ell = 0.0625(40.5)d_w = 2.53 d_w$
$w_c = 0.15(2)(0.75 + d_w)$
$= 0.3 d_w + 0.23$

$$w_\ell + w_c > \frac{50{,}484}{(93)^2(0.785)} - 0.94$$

$w_\ell + w_c = 6.50$ k
$2.83\ d_w = 6.27$
$d_w = 2.21$ ft
additional concrete $= 2.21 - 0.75 = 1.44'$ say 1'-6"

Check moment transfered to footing

$$M_{ucr} = 1.4\left[0.15\frac{(2)}{2}(2.25)^2 12\right] + 1.7\left[(0.0625)\frac{(40.5)}{2}(2.25)^2(12)\right]$$

$= 13 + 131$
$= 144$ k-in

By inspection #4@12" will work and can be developed in a 1'-6" wall.

Pikes Peak (0.4g acceleration)

$$\frac{M}{D^2(w_\ell + w_c + w_t)} < 0.785$$

$$\frac{15462}{(85)^2(0.49 + 1.74 + 0.45)} = 0.799 \cong 0.785$$ say ok

$w_c = 0.15(1.5)(2) = 0.45$ k

MONTGOMERY WATSON

BY TPD DATE 9/24/98 CLIENT City of Bellevue SHEET 20 OF 24
CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

Alternate No. 3

Note: This alternate will look at the required resistance to overturning at the point where the tank must be anchored according to AWWA D100-96 or where

$$M/(D^2 * (wt + wL)) = 1.54$$

In this case all piping connections must be modified to allow for flexible connections

Lake Hills North (0.3g or 0.4g acceleration)

$M_{1.54} = 1.54\,(D^2)(w_t)$
$= 1.54\,(66.1)^2(2.28) = 16{,}960$ K-Ft say 17,000 K-Ft

$M_{3.92} - M_{1.54} = 153300 - 17{,}000 = 136300$ K-Ft For 0.4g
$M_{2.94} - M_{1.54} = 114970 - 17{,}000 = 97{,}970$ K-Ft For 0.3g

Lake Hill South (0.3g or 0.4g Acceleration)

$M_{1.54} = 1.54\,(68)^2(2.40) = 17090$ say 17,100 K-Ft

$M_{3.92} - M_{1.54} = 148{,}630 - 17100 = 131530$ K-Ft For 0.4g
$M_{2.94} - M_{1.54} = 111{,}480 - 17100 = 94{,}380$ K-Ft For 0.3g

Woodridge (0.3g or 0.4g Acceleration)

$M_{1.54} = 1.54\,(71)^2(2.19) = 17001$ Say 17,000 K-Ft

$M_{3.46} - M_{1.54} = 138{,}900 - 17{,}000 = 121{,}900$ K-Ft For 0.4g
$M_{2.60} - M_{1.54} = 104{,}170 - 17{,}000 = 87{,}170$ K-Ft For 0.3g

Horizon View (0.3g or 0.4g Acceleration)

$M_{1.54} = 1.54\,(31.17)^2(0.60) = 898$ K-Ft

$M_{3.82} - M_{1.54} = 7390 - 898 = 6490$ K-Ft
$M_{2.87} - M_{1.54} = 5540 - 898 = 4640$ K-Ft

Note: Parksite and Pikes Peak are under $M_{1.54}$ for both 0.4g and 0.3g accelerations

EN 1S (10/78)

Alternate No. 3

Lake Hills North (0.4g Acceleration)

Anchor Spacing = 4'-6" ±
No. of Anchors = π (69.5 / 4.5) = 48.5 say 49 anchors

Anchor Tension T_B = 4.5 ((1.273 (136,300) / (69.5)²) − (2.28))
= 151.4k

Lake Hills North (0.3 g Acceleration)

Anchor Spacing = 6'-6" ±
No. of Anchors = π (69.5) / 6.5 = 33.6 say 34 anchors

Anchor Tension = 6.5 ((1.273 (97,970)) / (69.5)² − 2.28)
= 153.0 K

Lake Hills South (0.4 g Acceleration)

Anchor Spacing = 4'-6" ±
No. of Anchors = π (68) / 4.5 = 47.5 say 48 anchors

Anchor Tension = 4.5 ((1.273 (131530) / (68.0)² − 2.40)
= 152.1 K

Lake Hills South (0.3 g Acceleration)

Anchor Spacing = 6'-6" ±
No. of Anchors = π (68) / 6.5 = 33.1 say 34 anchors

Anchor Tension = 6.5 ((1.273 (94,380) / (68.0)² − 2.40)
= 153.2K

Woodridge (0.4 g Acceleration)

Anchor Spacing = 5'-3" ±
No. Anchors = π (71) / 5.25 = 42.5 say 43 anchors

Anchor Tension = 5.25 ((1.273 (121,900) / (71)²) − 2.19)
= 150.1K

MONTGOMERY WATSON

BY TPD DATE 9/24/98 CLIENT City of Bellevue SHEET 22 OF 29
CHKD. BY DESCRIPTION Reservoir Anchors JOB NO. 1060123

Alternate No. 3

Woodridge (0.3 g Acceleration)

Anchor Spacing = 7'-6"
No. Anchors = $\pi(71)/7.5 = 29.7$ say 30 anchors

Anchor Tension $= 7.5\left((1.273(87170)/(71)^2 - 2.19\right)$
$= 148.7^k$

Horizon View (0.4 g Acceleration)

Anchor Spacing = 2'-3" ±
No. Anchors $= 31.17(\pi)/2.25 = 43.5$ say 44 anchors

Anchor Tension $= 2.25\left((1.273(6490)/(31.17)^2 - 0.60\right)$
$= 17.8^k$

Horizon View (0.3 g Acceleration)

Anchor spa = 3.25'
No Anchors $= 31.17(\pi)/3.25 = 30.1'$ say 30

Anchor Tension $= 3.25\left((1.273(4640)/(31.17)^2 - 0.6\right)$
$= 17.8^k$

Anchor Chairs:

Note: for the purposes of this investigation use same anchor chairs as in alternate No. 1

MONTGOMERY WATSON

BY TPD DATE 9/24/94 CLIENT City of Bellevue SHEET 23 OF 29

CHKD. BY ______ DESCRIPTION Reservoir Anchors JOB NO. 1060123

Alternate No. 3

Soil Anchors:

Lake Hill North and South (0.4g and 0.3g Acceleration)

Use 40'-0" Anchor like Alternate No. 1

Woodridge (0.4g acceleration)

Use 45'-0" Anchor like alternate No. 1

Woodridge (0.3g acceleration)

Use 45'-0" Anchor like alternate No. 1

Horizon View (0.4g and 0.3g acceleration)

Use same length as alterate 1

MONTGOMERY WATSON

BY TPD DATE 9/25/97 CLIENT City of Bellevue SHEET 24 OF 29

CHKD. BY ____ DESCRIPTION Additional Concrete JOB NO. 1060123

Alternate No. 4

Find additional weight of water column plus foundation concrete at point where uplift requires anchorage:

or $\frac{M}{D^2(wt + wl + wc)} = 1.54$

Lake Hills North (0.4g Acceleration)

$wl + wc = \frac{153,700}{1.54(69.5)^2} - 2.28$ $wl = 4.69 dw$

$wc = 0.51 + 0.675 dw$

$wl + wc = 18.3^k$

$5.37 dw = 17.8$

$dw = 3.31'$ say 3'4"

Additional Concrete = 3.33' - 2.75' = 0.58' say 9"

Check Moment to be transfered to footing

$M_{ULT} = 1.4(0.15(\frac{4.5}{2})(3.33)^2(12)) + 1.7(0.0625(\frac{75}{2})(3.33)^2(12))$

$= 63 + 530$

$= 593$ k-in

Capacity of #4 @ 3" o.c. Stirrups = 183 k-in << 593 k-in N.G.

Note: This condition will exist for Lake Hills South (0.4g) and Woodridge (0.4g)

Lake Hills North (0.3g Acceleration)

$wl + wc = \frac{114,970}{1.54(69.5)^2} - 2.28$

$wl + wc = 13.2^k$

$5.37\ dw = 12.7^k$

$dw = 2.36'$ say 2'4"

Check Moment transfered to footing

$M_{ULT} = 1.4(0.15(4.5)(2.33)^2(12)\frac{1}{2}) + 1.7(0.0625(75/2)(2.33)^2(12)) = 290$ k-in < 183 k-in

Note: This condition will be true for Lake Hills South and Woodridge (0.3g)

MONTGOMERY WATSON

BY TPD DATE 9/25/98 CLIENT City of Bellevue SHEET 25 OF 29
CHKD. BY DESCRIPTION Additional Concrete JOB NO. 1060123

Alternate No. 4

Horizon View (0.4g Acceleration)

$$\text{wt rwc} = \frac{7390}{(31.17)^2\ 1.34} - 0.59 = 4.3^k$$

$2.49\ dw = 4.3^k$

$\therefore dw = 1.73'$ say $1'9''$

By inspection (See sheet 19 0.3g Accel) this will work

MONTGOMERY WATSON

BY TPD DATE 9/25/98 CLIENT City of Bellevue SHEET 26 OF 28

CHKD. BY ____ DESCRIPTION Annular Ring JOB NO. 1060123

Parksite (0.4g Acceleration)

Stiffen Annular Ring

$w_L max = 1.28 HDG = 1.28(40.5)(93) = 4821$

$\therefore M/(D^2 \times (w_t + w_L)) = \frac{50{,}484}{(93)^2(5.76)} = 1.01 > 0.785$ NG

Parksite (0.3g acceleration)

stiffen annular ring

$M/(D^2 \times (w_t + w_L)) = \frac{37862}{(93)^2(5.76)} = 0.755 < 0.785$ ok

The maximum reasonable thickness increase for the reservoir would be 1/4"

Max annular ring length = 0.035D = 3.25 ft.

$\therefore$ Use 3.25'

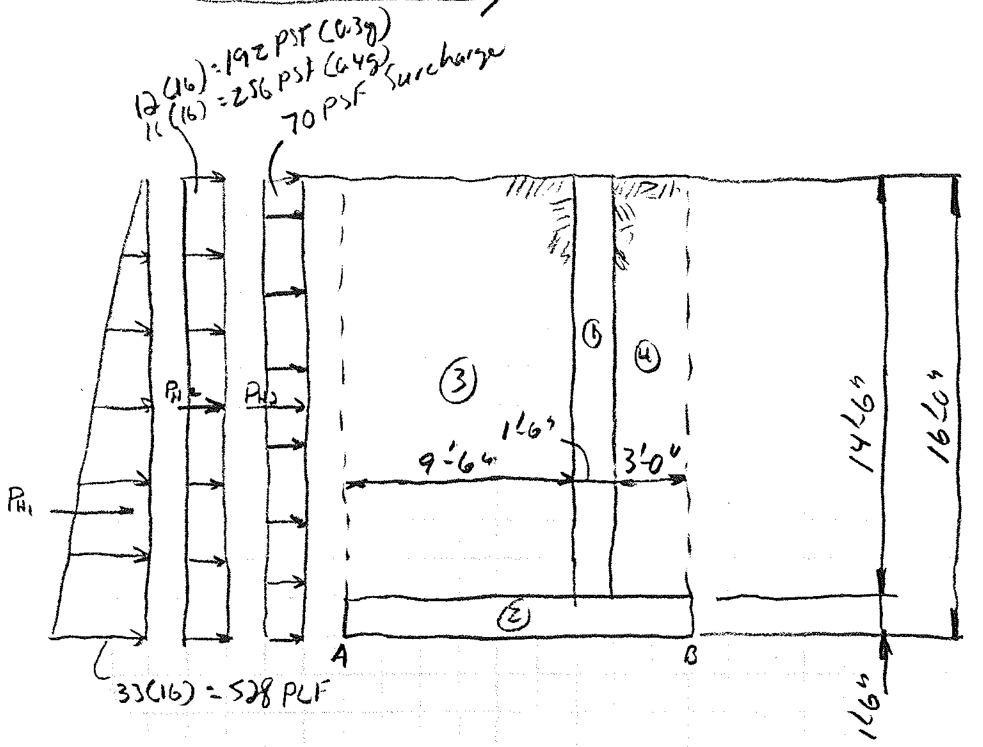
Pikes Peak (0.4 g Accel.)

Stiffen annular ring

Use same as Parksite 0.4g, stiffening annular ring will not work

Retaining wall

Check Stability



$P_{H_1} = \frac{1}{2}(0.53)(16) = 4.2^k$

$P_{H2} = 0.26(16) = 4.2^k$

$P_{H3} = 0.07(16) = 1.1^k$

Acceleration = 0.4g

Area

		Force	Arm Ⓐ	Arm Ⓑ	Mom Ⓐ	Mom Ⓑ
①	0.15 (1.5)(14.5) =	3.3^k	10.25	3.75	33.8	12.4
②	0.15 (1.5)(14.0) =	3.2	7.0	7.0	22.4	22.4
③	0.12(9.5)(14.5) =	16.5	4.75	9.25	78.4	152.6
④	0.12 (3.0)(14.5) =	5.2	12.50	1.50	65.0	7.8
		$\Sigma V = 28.2^k$				195.2 k-Ft
P_{H1}		= 4.2^k	5.33		22.4	
P_{H2}		= 4.2^k	8.0		33.6	
P_{H3}		= 1.1^k	8.0		8.8	
					$\Sigma M_A = 264.4^{k\text{-}Ft}$	

Location of resultant from A

$= 264.4/28.2 \approx 9.38'$

$\therefore e = 9.38 - 7.0 = 2.38$ →

Location of resultant without surch.

$= 255.6/28.2 \approx 9.06'$

$\therefore e = 9.06 - 7 = 2.06 < 14/6 = 2.33$

Retaining wall

Check Stability

$$q_{max} = \frac{28.2}{14.0}\left(1 + \frac{6(2.06)}{14}\right) = 3.79 \text{ KSF} < 6(1.5) = 9 \text{KSF}$$

Check Sliding:

$$FS = \frac{0.6(28.2)}{9.5} = 1.78 > 1.5 \text{ ok}$$

Check overturning

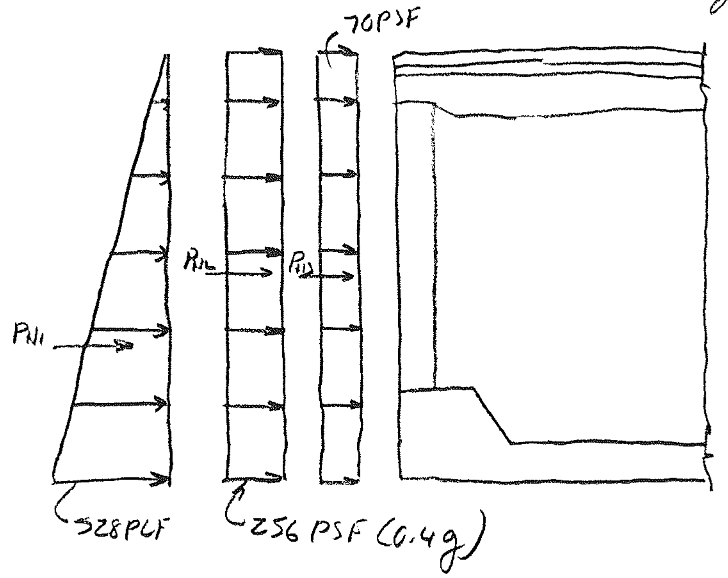
$$FS = 195.21/64.8 = 3.01 > 2.0 \text{ ok}$$

Quantities Say 85' Long wall

Concrete
$$14(85)(1.5) = 1785 \text{ ft}^3$$
$$14.5(85)(1.5) = 1849$$
$$3634 \text{ ft}^3 = 135 \text{ cy}$$

Excavation
$$14(16)(85) = 19040$$
$$\tfrac{1}{2}(16)(16)(85) = 10880$$
$$29920 \text{ ft}^3 = 1108 \text{ cy}$$

Tiebacks (look at tiebacks as if they resist seismic and other forces from soil only)



$P_{H1} = 4.2^k$
$P_{H2} = 4.2^k$] - see sheet 27
$P_{H3} = 1.1^k$

Find resultant location of horizontal forces

$$y = \frac{4.2(8) + 1.1(8) + 4.2(5.33)}{9.5} = 6.8' \text{ from bottom at tank}$$

Try two anchors per panel about 6'-0" apart with first anchor 4'-0" from bottom of tank

Total force to be resisted $= 9.5(8) = 76^k$ or 38^k per anchor

see HWA FIG 10
Length of top anchor $= 38/2 + 4 + 10\left(\frac{1}{\tan 60}\right) = 28.8$ say 30'
Length of Bott anchor $= 38/2 + 4 + 4\left(\frac{1}{\tan 60}\right) = 25.3$ say 25'

total length of anchors use $10(55) = 550'$

say need to add concrete doweled into precast panel to help transfer force to wall

Use 3'-0" square × 1'-6" block.

total Conc Quantity $= 3 \times 3 \times 1.5\,(20) = 270\ ft^2$ or 10 cy.