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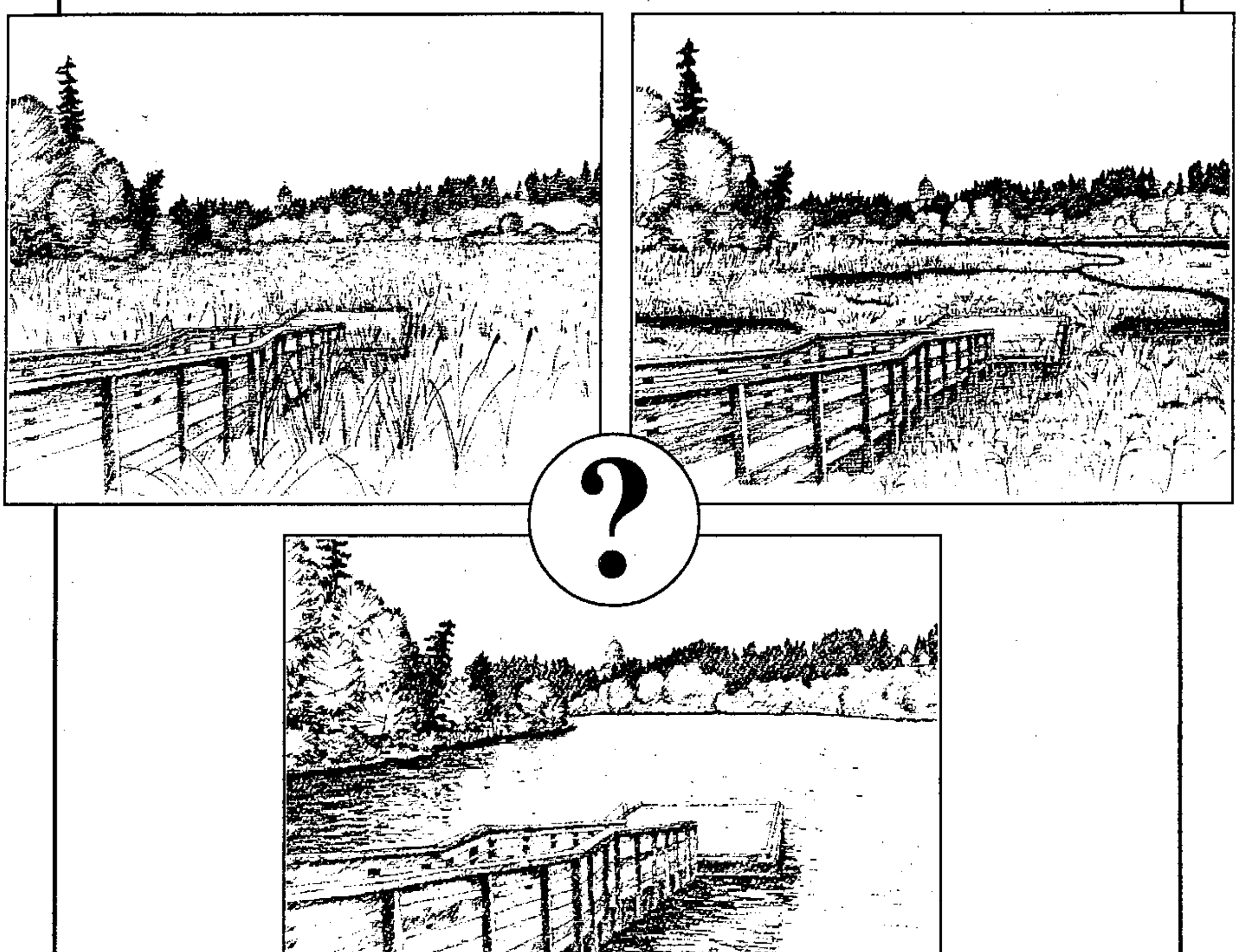
Capitol Lake

Adaptive Management Plan 1999 to 2001

Phase One - Task 3 Hydraulic Scour Analysis



April 2000

Phase One - Task 3
Hydraulic Scour Analysis

Capitol Lake Adaptive Management Plan
1999 to 2001

Olympia, Washington

Prepared For
Washington State Department of General Administration

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Summary

The Capitol Lake Adaptive Management Plan outlined several technical studies to provide information for future management decisions concerning Capitol Lake. One study was a hydraulic scour analysis to determine the impacts of opening the 5th Avenue Dam and allowing tidal exchange into the North, Middle, and South basins of Capitol Lake. The key question is: What are the predicted scour impacts to existing bridges, roadways, parkland, and the dam resulting from tidal exchange and flood events? A qualitative analysis was provided in the *Capitol Lake Adaptive Management Plan Environmental Impact Statement* (**Entranco 1998**), which concluded that scour impacts could threaten existing structures without mitigation.

The quantitative analysis documented in this report included the use of hydrologic and hydraulic computer analyses to predict river flows, critical velocities, and depth of scour at important structures. Deschutes River flood flows were updated based on recent flow data and used as input to a dynamic hydraulic model to predict velocities resulting from different flood levels as influenced by tide conditions. The lake's shape and depth and cross-section details of key structures were input into two hydraulic models, as were flows from Percival Creek, to provide scour impact results. Two different scenarios for the downstream openings for tidal exchange to Capitol Lake were assumed: 1) the existing dam with the tide gate open, and 2) the dam removed with a 500-foot opening.

The analysis results identified hydraulic scour impacts to key structures around Capitol Lake. These impacts are summarized in the following sections.

I-5 Bridge

The analyses indicate scour depths ranging from 5.7 to 10.9 feet for the conditions examined. The footings of the bridge in the main channel are at an elevation of -16.5 feet mean sea level (MSL). The maximum scour depth occurs during the 500-year flood with the dam removed. This is about 15 percent more scour than is predicted for the same flow with the existing dam in place and the tide gate maintained in the open position. The predicted scour depths represent a substantial impact; the major concern is loss of lateral support, which could be a problem, especially during earthquakes.

BN/SF Railroad Bridge

The analyses indicate scour depths ranging from 16.2 to 26.7 feet for the conditions examined. The maximum scour depth occurs during the 500-year flood with the existing dam removed. The effects of removing the dam produces scour depths that are about 6 percent greater for the 500-year flood. It is questionable whether the structure would be able to withstand these forces without mitigation.

5th Avenue Dam and Bridge

The analyses indicate scour depths ranging between 22.0 and 31.9 feet for the scenario with the tide gate open. The largest scour depth is predicted to occur during the 500-year flood. It is questionable whether the existing structure would be able to withstand these forces without mitigation. If the dam is removed, the resulting 500-foot opening produces no significant scour, unless the 5th Avenue replacement bridge has piers in the channel. The piers would have to be evaluated for scour.

4th Avenue Bridge

This summary addresses hydraulic scour impacts for both the existing and proposed 4th Avenue bridges. The analyses for the existing 4th Avenue Bridge indicate scour depths ranging between 9.1 to 15.4 feet for the two scenarios examined. The largest scour depth is predicted to occur during the 500-year flood with a 500-foot opening. This is about 15 percent more scour than is predicted for the same flow with the existing dam in place and the tide gate maintained in the open position. The predicted scour depths could undermine the existing pier foundations and compromise the integrity of the structure.

The analyses for the proposed 4th Avenue Bridge indicate scour depths ranging between 12.1 to 14.5 feet for the two scenarios examined. The largest scour depth was predicted to occur during the 500-year flood with the 500-foot opening. This is about 4 percent more scour than predicted for the same flow with the existing dam in place and the tide gate maintained in the open position. The proposed foundation for the piers located in the channel consists of an 8- or 10-foot-high footing on top of a 12-foot-high pile cap. The predicted scour depths would expose the top portion of the footings, but would not threaten the integrity of the structure.

Percival Creek Bridge

The analyses indicate a scour depth between 1.1 and 1.2 feet for the conditions examined. The scenario with a 500-foot opening results in the same scour impacts as the scenario with the existing dam and open tide gate. Based upon the available information, it appears that the structure would be able to withstand these forces.

Shoreline Area North of 5th Avenue Dam (Vicinity of KFC)

At the existing dam with the tide gate open, velocities are predicted to be 12 feet per second (fps). Shoreline erosion would be substantial and shoreline armoring would be required as mitigation.

With a 500-foot opening, velocities were predicted to be 1.5 to 2.5 fps. Shoreline erosion would be minimal and the existing shoreline armoring would be adequate.

Deschutes Parkway

Shoreline erosion would not be a problem with the maximum predicted flood velocities of 1.9 fps. However, slope stability problems, due to water level fluctuations from tidal exchange, are a major concern with the potential for local, and possibly widespread failure of the existing embankment fill slopes. Mitigation measures are recommended.

Heritage Park and Marathon Parks

Shoreline erosion does not appear to be a problem with the predicted velocities of 2.1 feet per second and 1.3 feet per second, respectively for Heritage Park and Marathon Park.

Steep Slopes East of Capitol Lake

Lake level fluctuations, due to tidal exchange, could increase the potential for slope instability along the eastern margin of the lake. This would be a problem particularly if fluctuations in water levels extended above the toe of any unstable slope areas.

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Introduction

Entranco, Inc. was contracted by the Washington State Department of General Administration (DGA) to provide technical analyses for implementing the *Capitol Lake Adaptive Management Plan – 1999 to 2001*. One task essential to plan implementation is a hydraulic scour analysis. The purpose of the analysis is to predict the hydraulic impacts to major structures in Capitol Lake, resulting from flood flows and opening the lake to tidal exchange. Two estuary conditions were assumed for this analysis: 1) opening the tide gate permanently on the existing 5th Avenue Dam, and 2) removing the dam and creating a 500-foot opening to allow tidal exchange into the lake basins.

Approach

The major components of this Hydraulic Scour Analysis are:

1. Determine the physical characteristics of the North, Middle, and South basins of Capitol Lake (based on existing topographic and bathymetric surveys).

2. Update Deschutes River flood flow estimates using new flow data.

3. Set up the dynamic hydraulic model XP-SWMM.

4. Set up the hydraulic scour model HEC-RAS

Physical Characteristics of the Capitol Lake Basins

The first major task was to understand the physical characteristics of the North, Middle, and South basins of Capitol Lake. This provided input to the hydraulic models used in the scour analysis.

Over the past several years, surveys have been conducted during lake drawdown to estimate the rate at which the lake is filling with sediment transported by the Deschutes River. Several topographic and bathymetric maps were reviewed and assessed for use in setting up the hydraulic model. Since no single map provided all the information needed, selected data from each of the following topographic maps were used for model set up:

- 1983 Capitol Lake topography by Walker & Associates, Inc. (provided specific bathymetry data around the dam)

- 1996 Capitol Lake topography by NIES Mapping Group, Inc. (most complete)

- 1999 survey basemap for the 4th Avenue Bridge design from David Goodyear Engineering Services (for Lower Budd Inlet bathymetry) (provided data downstream of dam)

It should be noted that the limited topography information available does not represent the current topography of Capitol Lake. However, after examining the limited available resources, it was concluded that the information is appropriate for a planning-level analysis.

Capitol Lake is physically separated into three distinct basins: North, Middle, and South (**figure 1**). Each basin was formed as a result of downstream structures that restrict or regulate the water flow. The shape of these basins and these structures cause flood flow expansion and contraction, both upstream and downstream of the structures. The added influence of tidal fluctuations changes flow areas, flood velocities, and the depth of scour around structures in each basin (**figure 1**).

The Interstate-5 (I-5) Bridge, the Burlington Northern Santa Fe (BNSF) railroad bridge, and the 5th Avenue Dam function as downstream controls to the South Basin, Middle Basin, and North Basin respectively (**figure 1**). Other major features of concern adjacent to Capitol Lake include the Deschutes Parkway, the Percival Creek Bridge, Heritage Park, Marathon Park, and the 4th Avenue Bridge.

Each basin's volume was estimated based on aerial photographs, survey information, as well as the modified Capitol Lake Level-pool Flood model developed by Entranco in 1996. These physical characteristics and basin volumes were used as inputs to the hydraulic models: XP-SWMM and HEC-RAS.

Updated Deschutes River Flood Flow Estimates

Additional flow data have been collected by the USGS in the vicinity of Capitol Lake. These data were combined with available historic data to create an extended flow record to use in determining flow statistics. Peak-annual flow data, or the flow rate for the largest flood during an individual year were combined from two USGS gauges located along the lower Deschutes River. The USGS flow gage station (1208000) for the Deschutes River near Olympia was maintained from 1946 through 1964 and provided flood flow data for that period. More recent flood flow data are available for gage station (1208010) for the Deschutes River at E Street Bridge in Tumwater. Flow data at this station have been collected from 1990 through 1998. Combining both stations provided a 28-year flood record with the largest recorded flood being 10,700 cubic feet per second (cfs) measured during the 1996 water year.

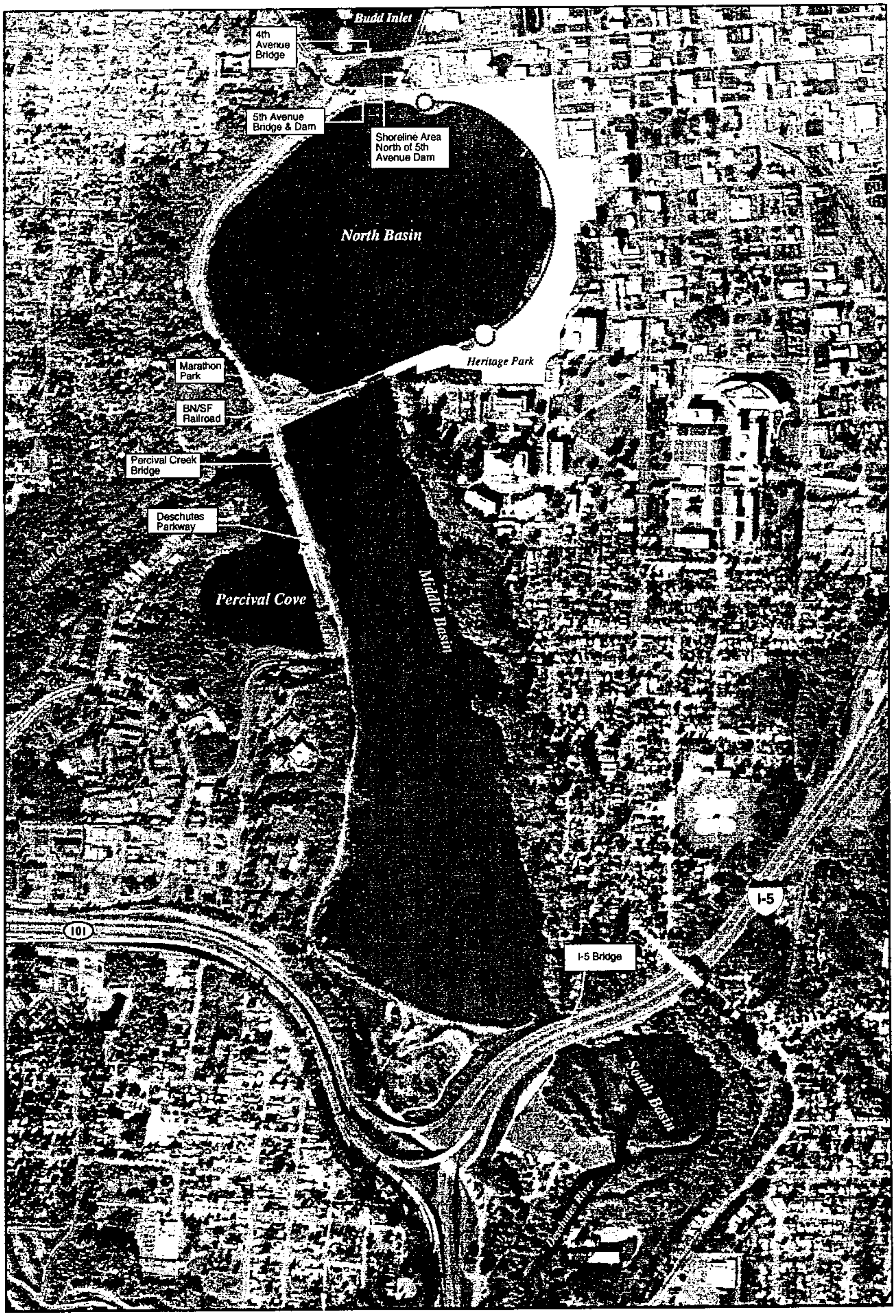

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Figure 1
Areas of Concern for Hydraulic Scour/Erosion

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A Log-Pearson Type III analysis was conducted using the 28 peak annual flows. The results of the analyses provided peak annual flow estimates corresponding to different return intervals (**table 1**). The flow rate corresponding to a specific return interval represents the flow that is expected to be equaled or exceeded on the average of once during the return interval. For example, the 100-year flow rate is the flood that is expected to be equaled or exceeded, on the average, once every 100 years.

Table 1
Updated Flood Flow Summary – Deschutes River near Capitol Lake

Peak-flow Recurrence (interval in years)	**Flow** (cfs)
1 year	1,441
2 year	3,989
5 year	5,887
10 year	7,249
25 year	9,082
50 year	10,523
100 year	12,031
500 year	15,842

For this study, flows with return periods of 500 years, 100 years, and 25 years were studied. A typical summer flow of 100 cfs was also considered to evaluate the conditions expected when tidal inflow rather than inflow from the Deschutes River dominates the lake basins.

Hydraulic Modeling

For this hydraulic analysis, tidal exchange between Capitol Lake and Lower Budd Inlet of Puget Sound had to be considered. For any estuary alternative, tidal fluctuations would have daily and seasonal effects on the lake basins' hydraulic conditions. The dynamic water interchange between the river, the lake basins, and Lower Budd Inlet has a significant impact on the water levels, flow velocities, and scour potential in the basins. These tidal effects, the Deschutes River flood flows, and the physical characteristics of the lake need to be considered to determine the hydraulic impact to structures in and adjacent to the lake.

To understand the tidal impact, the tidal elevation and its pattern as well as a typical Deschutes River hydrograph need to be incorporated in the hydraulic model. The tidal elevations and the Deschutes River hydrograph in Figure 2 and Figure 3 of Appendix C in the *Capitol Lake Adaptive Management Plan Draft Environmental Impact Statement*, respectively were used for the model. The tidal information and the hydrograph used in the previous flood analysis were input into XP-SWMM, a dynamic hydraulic model, to determine a worst-case scour condition for each major structure during different flood events.

The erosive shear force along structures within the lake is a function of the flow and velocity, which is a balance of the tidal elevation and flood flow magnitude. It becomes apparent that a dynamic hydraulic model is needed to evaluate the peak flow and peak velocity due to the interaction of the inflow, tidal elevation, as well as the basin physical characteristics. The XP-SWMM was used to estimate the peak flow and peak velocity near the major structures. However, XP-SWMMM does not evaluate scouring effects. Therefore, the peak flow and peak velocity for each basin from the XP-SWMM model were then input into the HEC-RAS model, a one-dimensional hydraulic model, to perform the scour analysis.

The basic set up of XP-SWMM starts by inputting the inflow hydrograph into the South Basin and routing the flow through the I-5 Bridge to the Middle Basin. After the flow was routed into the Middle Basin, the Percival Creek Bridge and inflow from Percival Creek were added to the basin geometry and flow set up. The flow was then routed through BNSF railroad trestle into the North Basin. From the North Basin, the flow was routed through the 5th Avenue Dam to Lower Budd Inlet. In addition, the maximum tidal elevation and pattern were incorporated into the model for the hydraulic analysis. As previously mentioned, the Deschutes River 25-year, 100-year, and 500-year flood flows as well as daily summer flows were evaluated. The flows were routed through the two assumed estuary conditions: 1) the existing dam with tide gate open and 2) the dam removed with a 500-foot opening.

Opening the tide gate completely was studied because this is a likely method for creating an estuary. Except for the potential impacts of scour and bank erosion, it would not involve any expense such as removing the dam and the 5th Avenue Bridge, or adjacent buildings.

The second scenario studied assumed a 500-foot opening. This length was chosen because it would provide the largest opening with few additional construction costs beyond removing the 5th Avenue Bridge and dam, possibly removing adjacent buildings, and replacing the 5th Avenue Bridge. Downstream of 5th Avenue, the opening can be no greater than what exists currently due to development encroaching on the estuary. There are also existing structures on either side of the 500-foot opening along the shore.

Selected flow values in **table 1** were used in the hydraulic analysis (XP-SWMM). The 500-year flow was chosen for study because it represents an extreme flood event using the largest flow predicted under the updated flow statistics developed for this study. Frequently, the 500-year flow is the worst-case scour condition due to the extraordinary amount of flow and the consequent high-water surface elevations and velocity.

The 100-year flow was chosen for study because it is often the "design flow" for scour countermeasures and bridge footing depths. This flow information will be useful if it is decided that countermeasures should be added to prevent damage to bridge footings or shoreline areas.

There are times when the 500-year flow is not the worst case for scour conditions but a smaller flow, such as the 100-year flow, causes more damage. One reason this may occur is an effect known as pressure flow. Pressure flow occurs when the flow elevation exceeds the bridge's low chord elevation (generally the bottom of the deck), but does not exceed the high chord elevation (generally the top of the handrail or top of the bridge deck). The bridge acts as an obstruction for the water and forces the water underneath the low chord. This action causes pressure in a downward direction and creates more scour than if the water was able to flow over the bridge.

The 25-year flow was examined because it has a greater frequency than the larger flows. Scour is cumulative, each flooding adds to the scour depth. If smaller floods are occurring more frequently, it may have a more detrimental effect than one storm that occurs every 100 years.

A lower flow may also create a faster velocity because the flow may be contained within the main channel section. If some of the flow is in the main channel and some is in the overbank area, the interaction between the water will cause an overall slowing effect. If the water is confined to the channel, it will be moving at a higher velocity and could cause greater local scour.

The tidal flow was studied because it affects the system in a unique way—it flows opposite to stream flow. Areas that were not affected previously by flows now could be severely affected. An example would be the area downstream of a bridge. Downstream of both the I-5 Bridge and the BNSF Bridge there are areas of vegetation and higher elevations. These areas are a result of the fact that the accelerated flow through the bridge does not spread out with a continued velocity immediately. This area has flow, but it is of low velocity, and is generally considered an ineffective flow. Also, flows at low velocity deposit material picked up while the flows accelerated through the bridge opening. When the flow is reversed, as with a tide, the flow is constricted from the entire width of the lake to the bridge opening. This causes an accelerated and turbulent flow. These actions increase the bed shear stress and may cause slope stability issues.

Hydraulic Analysis

A HEC-RAS hydraulic and scour analysis was performed for the I-5, BNSF, and Percival Creek bridges. It was also performed for the 5th Avenue Dam. The following presents the assumptions and criteria used for this analysis. **Appendix A** provides additional supporting data.

The basic input components of a HEC-RAS analysis are the geometric data and the flow data. The geometric data defines the river channel, overbank areas, sinuosity, and gradient of the river. It also defines pertinent dimensions of structures, such as bridges, located within the stream. For this study, the information was obtained from existing survey information, field measurements, and field observations.

Flow data include: peak-flow values, channel and overbank roughness, tidal elevations, and flow distribution. Some flows were developed with XP-SWMM, as described in the previous section. Other flow data were determined from historical records, field survey, and past studies.

The geometric and flow data were used to construct a computer model that represents the natural system. The HEC-RAS model performs a one-dimensional steady-flow analysis to calculate flow velocities and subsequently the magnitude of scour.

Three processes contribute to bridge scour depth:

- degradation
- contraction scour
- local scour

Degradation is the lowering of the stream bed elevation over long periods of time and over long reaches of the stream. The process is directly related to the equilibrium a stream maintains in transporting sediment. Degradation was not considered in this study. The areas of interest are located at the interface of Lower Budd Inlet and the Deschutes River, where the river gradient becomes extremely flat. This type of area typically experiences sediment deposition or aggradation, the raising of the streambed. It is unlikely it would experience degradation. An exception could be if removal of the dam allows the existing bed elevation of Capitol Lake to drop—although this could be prevented by providing a bed control structure in the vicinity of the existing dam.

Contraction scour is caused by the reduction in the flow area of a stream, and is common at bridge locations. A decrease in flow area creates an increase in the average bed shear stress throughout the constricted area. The increased shear stress causes an increase in the erosive forces, carrying away more material than is transported in, lowering the stream bed elevation. This process continues until a new equilibrium is established, generally from an increase in flow area resulting from the loss of bed material. The HEC-RAS model uses empirical formulae to calculate the maximum possible contraction scour.

Local scour is caused by the accumulation of water on the upstream surface of an obstruction, such as a bridge pier, and the subsequent acceleration of the water around the base of an obstruction. This process causes the formation of vortices, causing an increase in the shear stress which acts on the bed material at the base of the obstruction. This action removes bed material. Piers and abutments not located within the main channel may be susceptible to local scour during high flow events, when they become subject to flow. The commonly used CSU equation was used to predict the probability of scour and the extent of local scour.

Analysis Results

The following sections present the analysis results for each structure or area that would be subject to a change in flow patterns if management of Capitol Lake changes. Key assumptions used to develop each analysis are included in the tables presenting the results. It should be noted that the flow rates shown for each analysis represent the maximum instantaneous flow predicted under the conditions setup by the Deschutes River flow interfacing with the tidal effect. As a result, these flows do not match the flows presented in **table 1**. Negative flows shown in the tables reflect conditions where the incoming tide overwhelms the river flows resulting in the maximum flows occurring in the "upstream" direction.

I-5 Bridge

The I-5 Bridge crosses over Capitol Lake between the Middle and South basins, approximately 1.5 miles upstream of the confluence of the Deschutes River and Lower Budd Inlet.

Photograph 1: I-5 Bridge - Looking Downstream

The analysis of the likely scour with a narrow opening (modeled as the tide gate locked open) is similar to the current operation of Capitol Lake during a high-flow event. The water level in the lake is generally lowered as much as possible to allow for extra storage to prevent flooding of the downtown area. Also, the gate is opened as much as possible to allow the flow to continue out into Lower Budd Inlet. **Table 2** presents the analysis results for the I-5 Bridge.

The analysis found that the maximum predicted scour with the tide gate open occurred during the 500-year peak flow with 9.5 feet of scour. Approximately 1.6 feet would be contraction scour, and the remaining 7.9 feet would be pier scour. **Figure 2** illustrates the scour results for this scenario.

The analysis of the likely scour with a large channel opening (500 feet) between Capitol Lake and Lower Budd Inlet yielded results similar to the open tide gate. The worst-case scour condition occurred during the 500-year peak flow. It was found that the total scour would be 10.9 feet. Approximately 1.3 feet would result from contraction scour and 9.6 feet would result from local pier scour.

It is not surprising that the overall scour depth from both analyses would produce similar results. Although there are changes at the confluence with Lower Budd Inlet between the two scenarios, the flow is primarily influenced by tailwater effects from the BNSF Bridge.

Table 2
Scour and Velocity for I-5 Bridge

Flow Description	Open Gate 500-Year	Open Gate 100-Year	Open Gate 25-Year	Open Gate Summer[1]	Open Channel 500-Year	Open Channel 100-Year	Open Channel 25-Year
Q *(cfs)*	16,580	13,097	10,176	-2,100	16,946	13,211	10,314
Contraction Scour							
MC Approach Depth (ft)	16.7	16.6	13.4	12.5	16.6	14.8	13.2
MC Bridge Depth (ft)	17	17	13.8	12.5	16.9	15.1	13.6
Main Channel Approach Flow (cfs)	16370	12936	10128	-1690	16739	13111	10268
Main Channel Bridge Flow (cfs)	16326	12897	10119	-2096	16697	13091	10260
Approach Top Width (ft)	174	174	174	70	174	174	174
Bridge Top Width (ft)	148	148	148	150	148	148	148
Grain Size (ft)	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003
Approach Velocity (ft/s)	5.63	4.5	4.4	2.3	6.3	5.1	4.5
Critical Velocity (ft/s)	0.55	0.55	0.53	0.51	0.59	0.54	0.53
Scour Depth (ft)	1.6	1.5	1.2	0	1.3	1.4	1.2
Pier Scour							
Pier Nose Shape	Square	Square	Square	Square	Square	Square	Square
Pier Width (ft)	4	4	4	4	4	4	4
Upstream Depth (ft)	16.7	16.6	13.4	12.5	15.6	13.8	12.2
Upstream Velocity (ft/s)	5.6	4.5	4.4	2.3	6.3	3.6	3.2
Scour Depth (ft)	7.9	7.2	6.9	5.7	9.6	6.4	6
Combined Scour Depth (ft)	9.5	8.7	8	5.7	10.9	7.7	7.1

1. Summer flow represented as a reverse tidal flow.

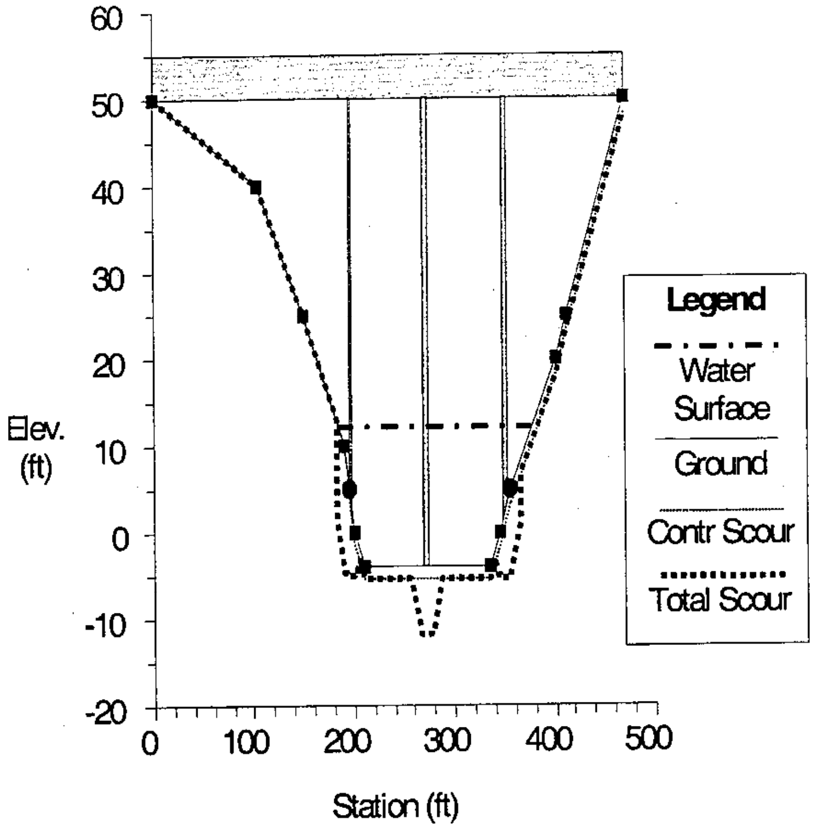


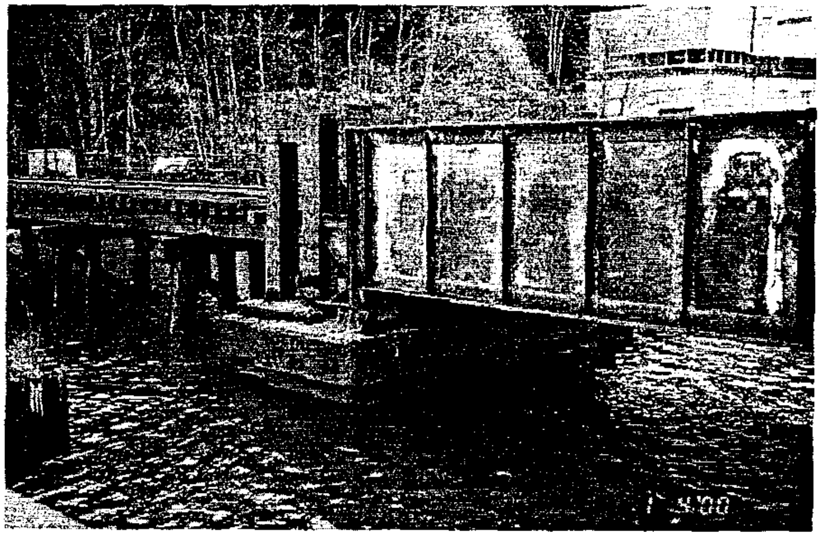
Figure 2
I-5 Bridge, 500-Year Flow with Open Tide Gate

Reverse low flow from tidal influences is predicted to be minor in comparison with flood flows. The I-5 Bridge is located approximately 1.5 miles upstream of the confluence of the Deschutes River and Lower Budd Inlet. This is far enough that the flow in that section is relatively small and does not experience a large constriction through the I-5 Bridge channel. The local pier scour would be approximately 5.7 feet.

As built drawings show that the bottoms of the footing located in the main channel is at elevation –16.5 MSL. The predicted maximum scour depth is –10.9 MSL.

BNSF Bridge

The BNSF Bridge crosses over Capitol Lake between the Middle and North basins. The bridge is located approximately 2,000 feet upstream of the confluence of the Deschutes River and Lower Budd Inlet. It is approximately a 200-foot-wide opening with approximately a 700-foot-long dike dividing the two basins.



Photograph 2: BNSF Bridge - Looking Upstream

The analysis of the likely scour with a narrow opening (modeled as the dam remaining and the tide gate locked open) is similar to the current operation of Capitol Lake during a high-flow event as described in the I-5 Bridge section. **Table 3** shows the analysis results for the BNSF Bridge.

The analysis found that the maximum predicted scour occurred during the 500-year flood with the 500-foot opening. It was found that these conditions would produce 26.7 feet of scour—10.3 feet of contraction scour and 16.4 feet from pier scour. If the existing dam is retained with the gates open, the combined scour depth would be 25.3 feet, or 6 percent less. **Figure 3** illustrates the scour results for the 500-year flood scenario with an open tide gate.

Table 3
BNSF Bridge

Flow Description	Open Gate 500-Year	Open Gate 100-Year	Open Gate 25-Year	Open Gate Summer[1]	Open Channel 500-Year	Open Channel 100-Year	Open Channel 25-Year
Q (cfs)	20,693	17,810	15,335	-6,879	22,300	18,640	15,000
Contraction Scour							
MC Approach Depth (ft)	17	14.2	12.6	7	14.9	13.5	12
MC Bridge Depth (ft)	38.5	11.5	10	7	11	10	9.3
Main Channel Approach Flow (cfs)	20,349	17,678	15,265	-6,879	22,105	18,522	14,967
Main Channel Bridge Flow (cfs)	16,934	17,795	15,335	-6,879	22,292	18,639	15,000
Approach Top Width (ft)	270	270	270	166	270	270	270
Bridge Top Width (ft)	162	162	162	138	162	162	157
Grain Size (ft)	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003
Approach Velocity (ft/s)	4.44	4.6	4.5	5.95	5.5	5	4.62
Critical Velocity (ft/s)	0.42	0.53	0.52	0.47	0.54	0.53	0.52
Scour Depth (ft)	8.7	8.7	8	0.9	10.3	9.3	8.2
Pier Scour							
Pier Nose Shape	Square	Square	Square	Square	Square	Square	Square
Pier Width (ft)	11	11	11	11	11	11	11
Upstream Depth (ft)	14.5	14.2	12.6	7	14.9	13.5	12
Upstream Velocity (ft/s)	5.73	4.6	4.5	6	5.5	5	4.62
Scour Depth (ft)	16.6	15.1	13.3	15.3	16.4	15.6	14.72
Combined Scour Depth (ft)	25.3	23.8	21.3	16.2	26.7	24.9	22.92

1. Summer flow represented as a reverse tidal flow.

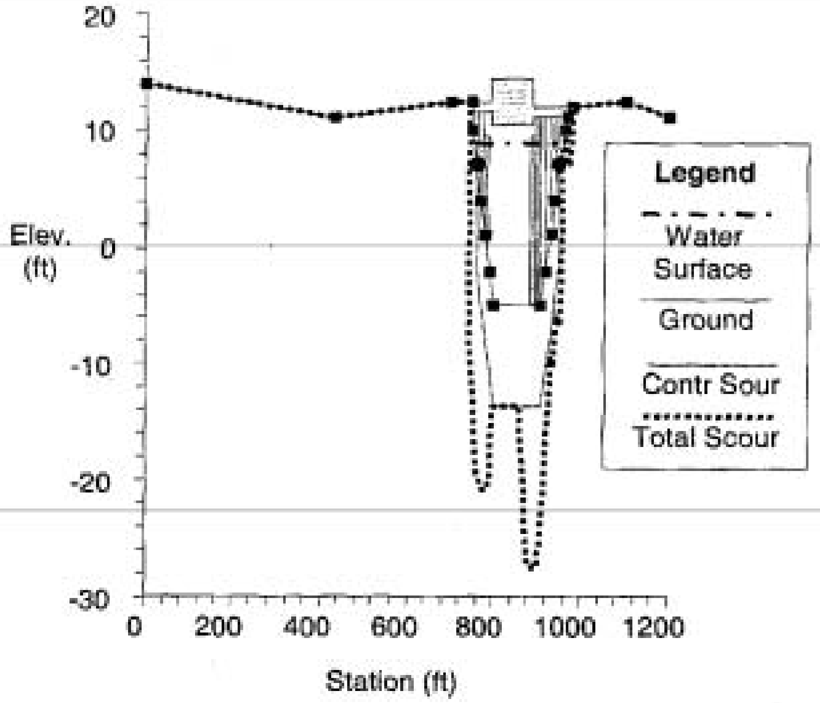


Figure 3
BNSF Bridge, 500-Year Flow with Open Tide Gate

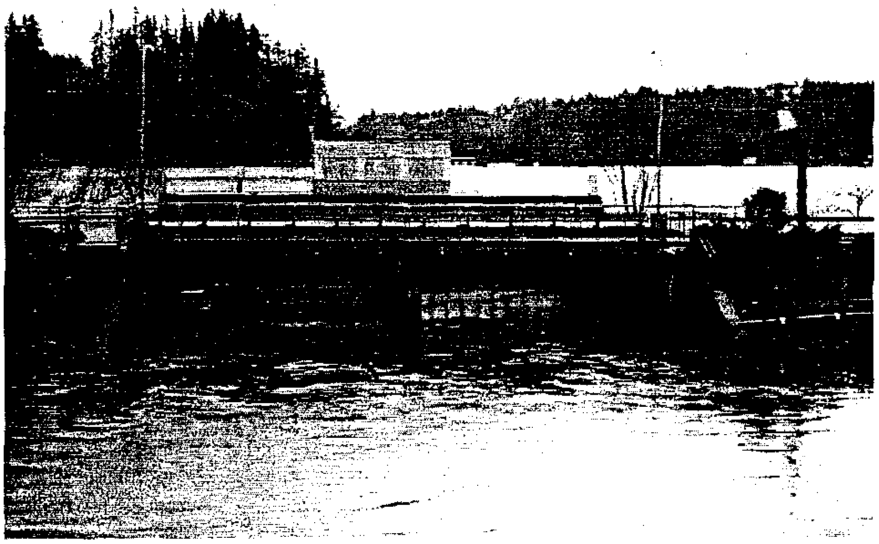
Large flows did not overtop the bridge or cause a pressure flow, but the rise in the water surface outside the normal channel, constricted the flow through the bridge section. As indicated in the I-5 Bridge analysis, the BNSF Bridge causes a significant rise in the water surface. During low flows, the majority of the flow can pass through the bridge section without being constricted between the banks. However, during higher flows all flood water is restricted to flowing through the bridge section only. This condition slows the upstream water velocity and raises the upstream water surface elevation. In addition, the constriction causes an accelerated flow through the bridge section, creating significant amounts of local pier scour.

Tidal flow will effect the BNSF Bridge daily. Whereas flood flow has a low probability and a limited duration, tidal flow occurs twice a day. If the management of Capitol Lake is changed, the BNSF Bridge will experience the full predicted scour for tidal flows.

The BNSF Bridge is expected to experience a significant amount of bridge scour. It is recommended that this bridge be studied further to determine if scour countermeasures should be installed. Timber piles support a significant portion of the bridge and the depth of timber piles can vary considerably from bridge to bridge.

5th Avenue Dam and Bridge

The 5th Avenue Dam and Bridge is at the interface of Lower Budd Inlet and the Deschutes River. The dam serves two purposes: it keeps tidal water out of the lake, and it dams the Deschutes River to form Capitol Lake.



Photograph 3: 5th Avenue Dam - Looking Upstream

On the upstream side of the dam there is a formation known as the Tide Gate Crater. Surveys conducted between the crater's inception and today indicate that the crater has a depth of –30 feet MSL[1]. This is approximately 20 to 25 feet lower than the surrounding lake bed elevations. "The crater has apparently formed as a result of the hydraulic forces generated when the lake is backfilled with marine water."[2]

It was found that for tidal flow, the resulting scour depth below the bottom of the dam (considered to have a bottom elevation of –17 MSL) is 22 feet or -39 MSL. Contraction scour accounts for 10 feet and the remaining 12 feet is

1. Tide Gate Crater Restoration Analysis, Entranco Engineers, January 18, 1983.
2. Ibid.

from the vortices formed during local scour. This corresponds well with the scour that has been found in that area to date. Note that the bottom of the dam is lined with concrete, so scour is not expected here, but it serves as an indicator of the magnitude of scour found immediately upstream or downstream of the dam. **Table 4** presents the analysis results.

Table 4
5th Avenue Dam[1]

Flow Description	Open Gate			
	500-Year	100-Year	25-Year	Summer[2]
Q (cfs)	23,120	20,980	18,618	-17,225
Contraction Scour				
MC Approach Depth (ft)	20.3	18.4	16.7	23.6
MC Bridge Depth (ft)	24.4	23	22	20.8
Main Channel Approach Flow (cfs)	16,641	16,968	15,607	-13,287
Main Channel Bridge Flow (cfs)	23,120	20,980	18,618	-17,225
Approach Top Width (ft)	135	135	135	76
Bridge Top Width (ft)	65	65	65	65
Grain Size (ft)	0.00003	0.00003	0.00003	0.00003
Approach Velocity (ft/s)	6.1	6.9	6.9	7.8
Critical Velocity (ft/s)	0.56	0.56	0.55	0.57
Scour Depth (ft)	19.9	13.3	10	9.7
Pier Scour				
Pier Nose Shape	Square	Square	Square	Square
Pier Width (ft)	4	4	4	4
Upstream Depth (ft)	20.3	18.4	16.7	23.6
Upstream Velocity (ft/s)	6.1	6.9	6.9	7.8
Scour Depth (ft)	12	12	12	12
Combined Scour Depth (ft)	31.9	25.3	22	21.7

1. The open channel scenario (500-foot opening) resulted in no contraction scour. No local pier scour could be predicted since no bridge design has been developed.
2. Summer flow represented as a reverse tidal flow.

The analysis for leaving the dam in place found that the maximum case scour occurred during the 500-year peak flow, which would produce a total scour of 31.9 feet. Approximately 20 feet would be contraction scour and the remaining 12 feet would be pier scour. **Figure 4** illustrates the analysis results with the tide gate open.

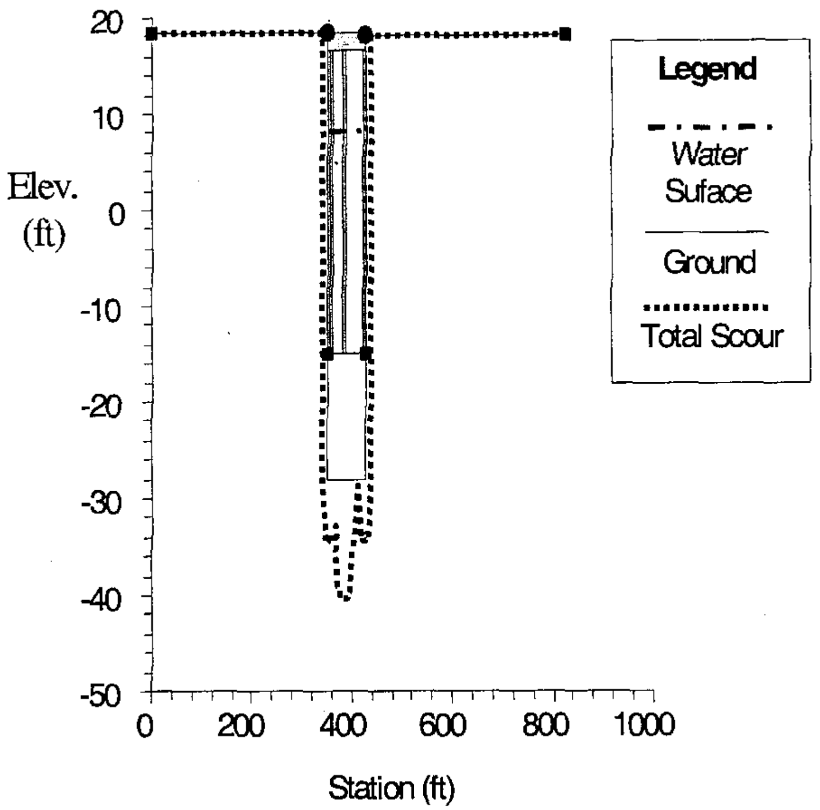
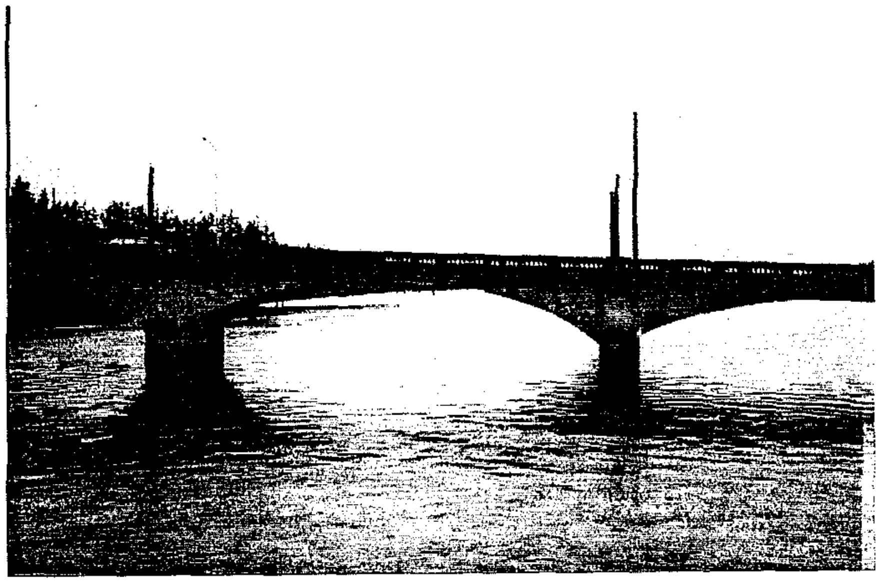


Figure 4
5th Avenue Dam, 500-Year Flow with Open Tide Gate

The second scenario assumes that the 5th Avenue Bridge and dam will be removed. It also assumes that the new 5th Avenue Bridge would have a 500-foot opening. Potential scour of the individual piers was not considered. It was found that if this 500-foot opening were created, there would not be any contraction scour. Any resulting scour would be caused by local pier or abutment scour associated with the new bridge. No local pier scour could be predicted since no bridge design has been developed.

4th Avenue Bridge

Although the 4th Avenue Bridge is not located within Capitol Lake, lake management practices have an effect on the maximum predicted scour depth at the bridge. The bridge crosses Lower Budd Inlet approximately 300 feet downstream of 5th Avenue Dam (**Photograph 4**). The 4th Avenue Bridge will be replaced and construction is scheduled to begin in 2001.



Photograph 4: Existing 4th Avenue Bridge - Looking Downstream

The following discussion addresses both the existing and proposed 4th Avenue Bridge under the two scenarios for managing Capitol Lake.

Existing 4th Avenue Bridge

The section of the existing bridge that spans Lower Budd Inlet is comprised of three luten arch structures spanning between three piers and one abutment. Two piers are located within the main channel and the remaining pier and the abutment are subject to flow during high flow/tide events.

Analysis of the existing bridge found that the maximum predicted scour occurred during the 500-year flood with the 500-foot opening. It was found that these conditions would produce 15.4 feet of scour (**table 5**). The predicted scour is a result of local bridge pier scour. If the existing dam is retained with the tide gate open, the combined scour depth would be 13.4 feet or 15 percent less for the

500-year flood. **Figure** 5 illustrates the scour results for the 500-year flood with open tide gate. Unlike the scenario for the 500-foot opening, the maximum predicted scour occurs during the 100-year flood for the open tide gate scenario. The 100-year flood produces 14.1 feet of scour. If the maximum predicted scour occurs, the footing of the eastern most pier will be undermined for either scenario.

Table 5
Existing 4th Avenue Bridge

	Open Gate				Open Channel		
Flow Description	**500-Year**	**100-Year**	**25-Year**	**Summer[1]**	**500-Year**	**100-Year**	**25-Year**
Q (cfs)	23,120	20,980	18,618	-17,225	25,569	22,406	20,000
Contraction Scour							
MC Approach Depth (ft)	27.5	27.5	27.5	23.9	27.5	27.5	27.5
Main Channel Approach Flow (cfs)	22,976	20,850	18,503	-17,369	25,409	22,267	19,877
Main Channel Bridge Flow (cfs)	23,006	20,877	18,526	-17,741	25,443	22,296	19,901
Approach Top Width (ft)	420	420	420	320	420	420	420
Bridge Top Width (ft)	117	115	115	197	116	115	115
Grain Size (ft)	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003
Approach Velocity (ft/s)	2.9	2.6	2.3	2.3	3.2	2.8	2.5
Critical Velocity (ft/s)	0.55	0.56	0.56	0.51	0.56	0.56	0.56
Scour Depth (ft)	0	0	0	2.6	0	0	0
Pier Scour							
Pier Nose Shape	Square	Square	Square	Square	Square	Square	Square
Pier Width (ft)	11	11	11	11	11	11	11
Upstream Depth (ft)	27.5	27.5	27.5	9.75	27.5	27.5	27.5
Upstream Velocity (ft/s)	3.6	3.2	2.9	0.73	4	3.5	3.1
Scour Depth (ft)	13.4	14.1	12.2	6.5	15.4	13.2	12.6
Combined Scour Depth (ft)	13.4	14.1	12.2	9.1	15.4	13.2	13.7

1. Summer flow represented as a reverse tidal flow.

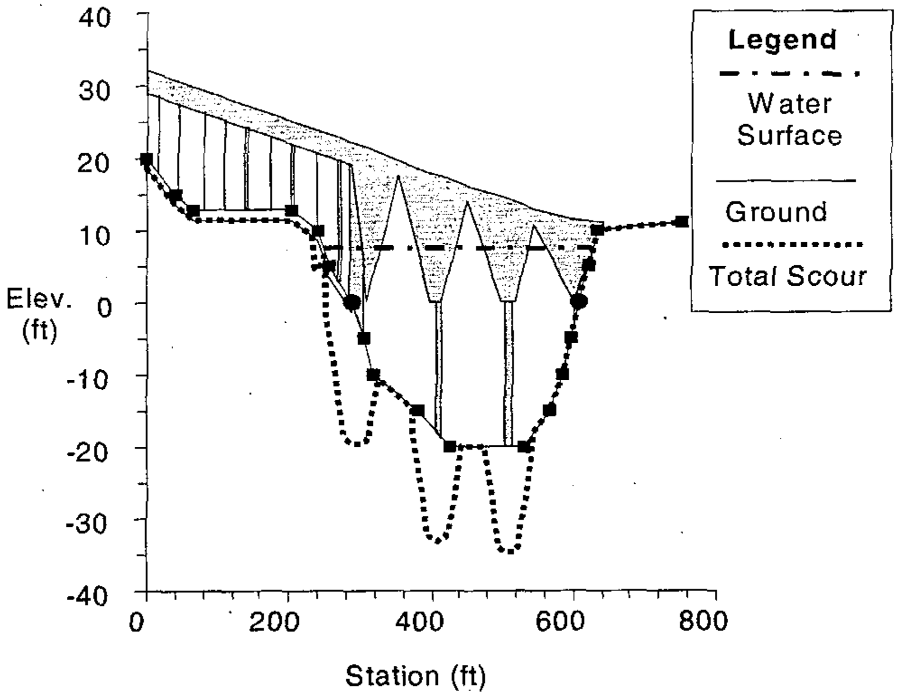


Figure 5
Existing 4th Avenue Bridge, 500-Year Flow with Open Tide Gate

The scour elevations for the 500-foot opening and the open tide gate scenario were anticipated to be similar for this bridge. The bridge is located downstream of the dam, and therefore is not subject to increased water surface elevations or decreased velocities caused by the limited capacity of the dam. The difference in the amount of scour is caused by the slight difference in flow between the two scenarios, which can be traced back to the limited capacity of the open tide gate.

Large flows did not overtop the existing bridge or cause pressure flows. However, some flows did touch the bottom chord at the lowest point on the arches.

Proposed 4th Avenue Bridge

As part of the Olympia Gateway Corridor – 4th Avenue Bridge Replacement Study for the City of Olympia, a bridge scour analysis is being performed on the proposed bridge replacement alternative. The preliminary analysis results are included in this study for planning purposes. The model is expected to be refined, as more information becomes available.

The proposed replacement bridge is a haunched girder superstructure. Two piers will be located in the channel and one additional pier may be subject to flow during high flows. The existing bridge will be demolished; however, the existing bridge foundations are proposed to be left in place below the current surface of the channel bed.

The analysis found that for the proposed bridge the worst-case bridge scour, 14.5 feet, occurred during a 500-year event with the 500-foot opening. It is predicted that there would be 13.9 feet of scour during a 500-year event with the open tide gate scenario. The scour is a result of local bridge pier scour. **Figure 6** illustrates the scour results for the 500-year flood with the open tide gate scenario. The proposed replacement bridge does not encroach in the channel, so contraction scour is not an issue as flows move out to sea. However, as tides rise and flows move toward Capitol Lake, a slight amount of contraction scour will occur, as can be seen from the summer flow results (**table 6**).

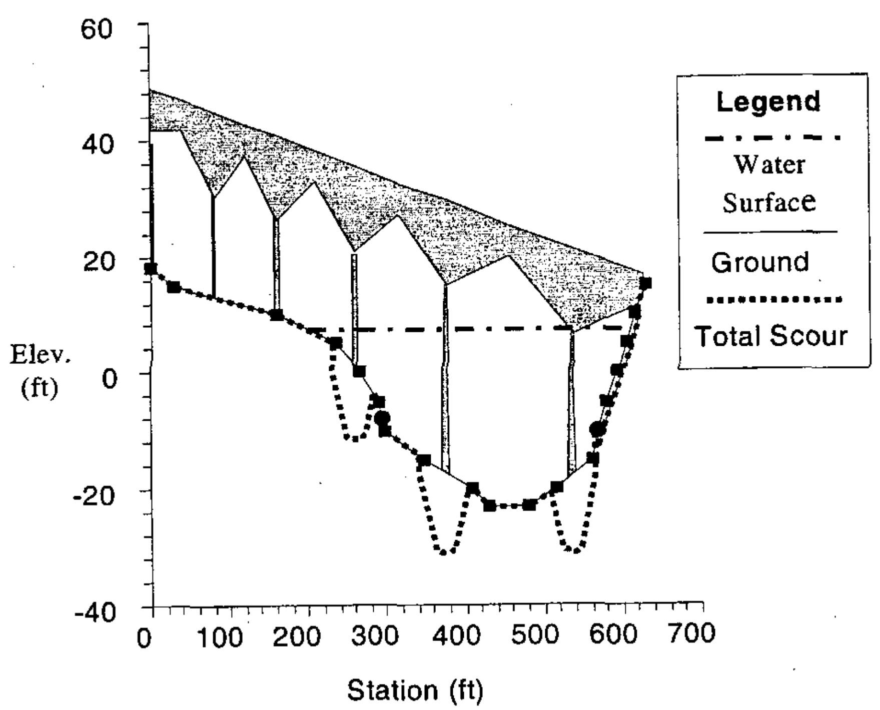


Figure 6
Proposed 4th Avenue Bridge, 500-Year *Flow with Open Tide Gate*

Table 6 Proposed 4th Avenue Bridge	Open Gate				Open Channel		
Flow Description	500-Year	100-Year	25-Year	Summer[1]	500-Year	100-Year	25-Year
Q (cfs)	23,120	20,980	18,618	-17,225	25,569	22,406	20,000
Contraction Scour							
MC Approach Depth (ft)	29.6	29.6	29.6	24.9	29.6	29.6	29.6
MC Bridge Depth (ft)	26.9	26.9	26.9	20.4	26.9	26.9	26.9
Main Channel Approach Flow (cfs)	21096	19143	16987	-16200	23332	20444	18249
Main Channel Bridge Flow (cfs)	22330	20263	17982	-16680	24696	21641	19317
Approach Top Width (ft)	265	265	265	320	265	265	265
Bridge Top Width (ft)	245	245	245	271	245	245	245
Grain Size (ft)	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003	0.00003
Approach Velocity (ft/s)	3.9	3.5	3.1	2.7	4.3	3.7	3.3
Critical Velocity (ft/s)	0.57	0.57	0.57	0.56	0.57	0.57	0.57
Scour Depth (ft)	0	0	0	1.1	0	0	0
Pier Scour							
Pier Nose Shape	Square	Square	Square	Square	Square	Square	Square
Pier Width (ft)	10	10	10	10	10	10	10
Upstream Depth (ft)	29.6	29.6	29.6	24.9	29.6	29.6	29.6
Upstream Velocity (ft/s)	3.5	3.2	2.8	3.2	3.9	3.4	3.1
Scour Depth (ft)	13.9	12.1	12.7	12.7	14.5	12.5	13.1
Combined Scour Depth (ft)	13.9	12.1	12.7	13.9	14.5	12.5	13.1

1. Summer flow represented as a reverse tidal flow.

Type, size, and location drawings of the proposed bridge indicate that if the maximum predicted scour depth for a 500-year flow with the 500-foot opening is reached, it will expose the top half of the footing. The footing has a depth of 10 feet and this is supported by the pile cap, which has a depth of 12 feet. For this reason, it is unlikely the maximum predicted scour would compromise the integrity of the structure.

Percival Creek Bridge

The Percival Creek Bridge carries Deschutes Parkway over the confluence of Percival Creek and Capitol Lake. The bridge is located approximately 200 feet upstream of the BNSF Bridge (**Photograph 5**). In addition to Percival Creek flow into Capitol Lake, tailwater from Deschutes River flow comes through the bridge opening into Percival Cove.

Photograph 5: Percival Bridge - Looking Upstream

The main factor affecting the velocities and scour beneath Percival Creek is the tailwater in the Middle Basin caused by the constriction at the BNSF Bridge. The water backing up behind the BNSF Bridge has a slow velocity and high water surface elevation. Consequently, the flow entering Capitol Lake during a large storm event is slowed down before it passes underneath the Percival Creek Bridge. Furthermore, the opening underneath the Percival Creek Bridge is sufficiently wide to prevent any contraction scour. **Table 7** presents the scour analysis results.

Table 7 Percival Creek Bridge				
	Open Gate		Open Channel	
Flow Description	100-Year	25-Year	100-Year	25-Year
Q (cfs)	678	569	678	569
Contraction Scour				
MC Approach Depth (ft)	9.8	9	12.8	8.3
MC Bridge Depth (ft)	10.6	9.8	13.6	9
Main Channel Approach Flow (cfs)	617	562	664	563
Main Channel Bridge Flow (cfs)	597	546	636	548
Approach Top Width (ft)	130	130	130	130
Bridge Top Width (ft)	55	55	55	55
Grain Size (ft)	0.0013	0.0013	0.0013	0.0013
Approach Velocity (fps)	0.48	0.48	0.4	0.52
Bridge Velocity (fps)	0.48	0.48	0.4	0.52
Critical Velocity (fps)	1.75	1.73	1.83	0.59
Scour Depth (ft)	0	0	0	0
Pier Scour				
Pier Nose Shape	Square	Square	Square	Square
Pier Width (ft)	1	1	1	1
Upstream Depth (ft)	10	9	12.8	8.3
Upstream Velocity (fps)	0.5	0.5	0.4	0.52
Scour Depth (ft)	1.2	1.2	1.1	1.2
Combined Scour Depth (ft)	1.2	1.2	1.1	1.2

The BNSF Railroad has such a significant impact on the flow underneath the Percival Creek Bridge that there is no difference in the amount of scour between opening the 5th Avenue Dam permanently or removing the dam to create a 500-foot-wide channel. **Figure 7** illustrates the scour results with the open tide gate.

The local pier scour at Percival Creek Bridge was found to be between 1 and 1.2 feet for all flows.

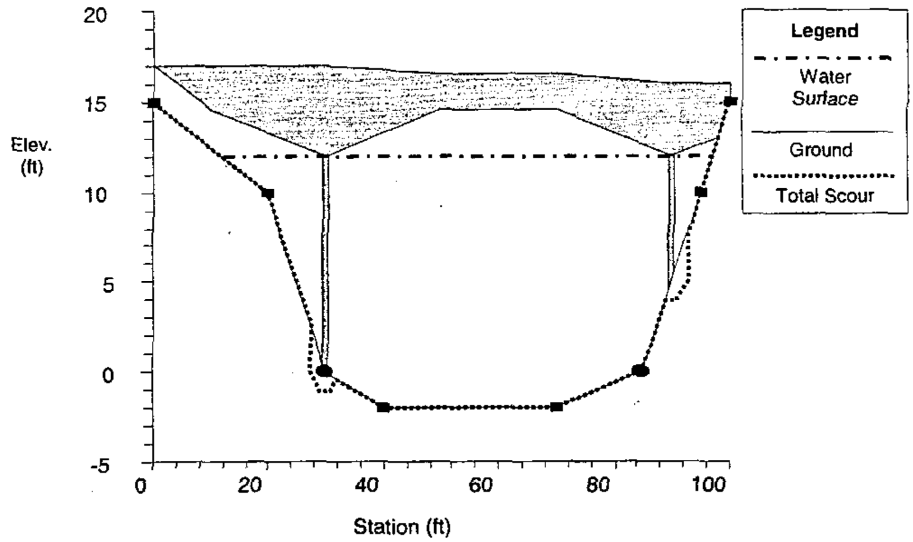


Figure 7
Percival Creek Bridge, 100-Year Flow with Open Tide Gate

Stream Bank Stability

Stream bed scour is the primary concern at bridges. However, the main concern for the Deschutes Parkway, Marathon Park, Heritage Park, and the shoreline area near the Kentucky Fried Chicken (KFC) restaurant is stream bank erosion and stability.

Similar to stream bed erosion, stream bank erosion is caused by shear stress from water interacting with stationary material. Unlike streambed erosion, which is primarily based on streambed material size and flow characteristics, stream bank stability and erosion is based on several additional factors. These factors include the angle of the bank slope, the amount and type of vegetation, and the porosity of the bank materials.

The stability of a bank with a gentle slope is less likely to be affected by stream bank erosion. One soil characteristic is angle of repose. This is the angle of internal friction of granular material at its loosest state. More simply, this is the steepest stable slope for a loosely packed soil. This is important to bank stability in a riverine environment because if the stream erodes the base of the slope, a slope near or at the angle of repose will readjust, in the form of a slide, to reestablish stability. A gentler slope will not necessarily slide, since it was not subject to a slope greater than its natural stability.

Stream banks with extensive vegetation and root systems are more stable than banks lacking vegetation. The root systems act to confine the soil and prevent sliding. Vegetation also alters the characteristics of surface water and surface erosion. A vegetated bank slows the velocity of surface runoff. The slower velocities have slower shear stresses and do not transport material down the bank.

Another factor that may affect the stability of the bank is porosity. This is the amount of water in the soil. This can affect the shear strength of the soil. Pore water cannot carry any shear stress. The strength then changes dependent upon the amount of saturation. Because the banks will become subject to fluctuating tidal elevations if the tide gate is removed, fluctuation in the saturation level also needs to be considered.

The analysis of Deschutes Parkway, Marathon Park, Heritage Park, and the area in the vicinity of the KFC considered primarily the likely response to large tidal fluctuations. The HEC-RAS and the pool analyses were used to determine the likely velocities and pool levels acting on the banks. The analyses also were used to make sure that velocities in these vicinities were relatively small.

Shoreline Area in Vicinity of KFC

The shoreline area in the vicinity of the KFC is located immediately downstream of the 5th Avenue Dam. The area is presently subject to the fluctuating tides and flow from the Deschutes River. There is some vegetation at the top of the existing bank in the area, but little or no vegetation exists beyond 5 feet below the top of the bank. Riprap has been placed on the banks, as well as a stone wall, to protect the bank from erosion.

Currently, tidal flow in this area creates velocities of approximately 1 to 2 fps. More significant flow velocities occur during flooding of the Deschutes River, when the gate is opened to diminish flooding effects in downtown Olympia. The 500-year flow in the Deschutes River causes velocities downstream of the dam of approximately 15 fps. This is an erosive force and could cause significant damage.

If Capitol Lake is operated with the dam in place and the tide gate open, it would frequently subject the shoreline to velocities on the order of 12 fps. Calculations were performed to determine if the present riprap would protect the shoreline adequately if the tide gate was open. The present riprap would not protect against forces with this velocity. It would be necessary to protect the existing banks with more substantial riprap or provide an alternative armoring method.

If the dam was removed and an open channel was created, the flows in this area would be reduced significantly. Average velocities in this area would be reduced to 1.5 to 2.5 fps. The current riprap would protect the existing bank against erosion with those velocities. If the bank were reconstructed for this alternative, it would be important to reconsider the use of countermeasures.

Finally, during lower flows in the Deschutes River, eddies are likely to form around the end of the wingwall that extends downstream of the dam. These eddies would continue to erode the area behind the wingwall and those areas adjacent to it if the scour protection measures are not maintained.

Deschutes Parkway, Heritage Park, Marathon Park

The shoreline areas of Capitol Lake were studied to determine if there would be adverse effects due to a change in Capitol Lake's management. Two primary factors were considered. The first factor was the possibility of erosional problems due to the interface of the water and soil. Opening the tide gate or creating a new channel would create flow in lake areas that were previously stagnant. The second factor involved slope stability caused by changes in the water surface elevation. At high tides or with a dam in place, the water surface elevation is at one level, but opening the dam or creating a channel would create a different water surface elevation. The strength of the drained versus the undrained soils needs to be examined to determine if the change in elevation would cause slope stability issues (see **Appendix B** for more information). Flow velocities were computed using the HEC-RAS model. Several cross-sections are shown in **table 8**.

Table 8 illustrates that flow velocities for all situations are fairly small. Without an in-depth study into the soil properties, it is difficult to say if erosion would occur. However, based on these flow velocities, erosion can be controlled. These velocities are in a range that suggests measures to armor the banks with riprap or vegetation would reduce potential erosion.

Table 8
Velocities for Deschutes Parkway, Heritage Park, and Marathon Park

	Flows						
	Open Gate				Open Channel		
	500-Year	100-Year	25-Year	Summer	500-Year	100-Year	25-Year
Heritage Park - Cross-Section 6 300' Upstream of 5th Ave. Dam	0.6	0.6	0.6	0.6	2.1	1.5	1.6
Heritage Park - Cross-Section 9 1,200' Upstream of 5th Ave. Dam	0.5	0.5	0.5	0.5	2.1	1.6	1.7
Marathon Park - Cross-Section 11	1	0.7	0.9	0.9	1.3	1.1	1.1
Deschutes Parkway - Cross-Section 13 600' Upstream of BNSF Bridge	1.7	1.3	1.5	1.5	1.9	1.7	1.6
Deschutes Parkway - Cross-Section 17 1,500' Upstream of BNSF Bridge	1.6	2	1.1	1	2	1.3	2
Deschutes Parkway - Cross-Section 21 2,000' Downstream of I-5 Bridge	1.3	1.7	0.9	0.9	1.6	1.1	1.7
Deschutes Parkway - Cross-Section 25 1,000' Downstream of I-5 Bridge	0.8	1	0.6	0.5	1.1	0.7	1

Implications to Adaptive Management of Capitol Lake – Management/ Operational Implications

In our opinion, this analysis provides sufficient quantitative information to determine that further, field-based, study of geotechnical impacts to structures of concern is required. The implications of the management changes are discussed below relative to each structure or area of concern.

I-5 Bridge

The maximum combined bridge scour depth with a 500-foot channel is predicted to be 10.9 feet. In our opinion, the estimated scour depth represents a significant potential impact to the existing I-5 Bridge. Scour adjacent to the bridge abutments, pile caps, and piles would result in the loss of lateral support, which could potentially compromise structural performance under static or seismic loading. WSDOT, and prudent engineering practice, would dictate further analysis of scour impacts of this magnitude on the integrity of the bridge foundations and superstructure. Additional geotechnical analysis would be needed to develop suitable mitigation measures, which may include protecting the bridge piers with heavy riprap, or strengthening the bridge foundations. The 500-foot estuary opening would result in about 15 percent more predicted scour than with the existing dam and the tide gate open.

BNSF Bridge

The maximum combined bridge scour depth is predicted to be 26.7 feet. In our opinion, the estimated scour depth could compromise the performance of the structure under static or seismic loading. The 500-foot estuary opening results in about 6 percent more scour than with the existing dam and the tide gate open.

5th Avenue Dam and Bridge

The maximum combined bridge scour depth is predicted to be 31.9 feet with the tide gate open. This scour depth could occur on either side of the dam structure. The present tide gate crater has not impaired performance of the dam. The bottom of the dam is concrete, so scour is not expected there. However, it is our opinion that if significant scour features are allowed to develop and deepen on both sides of the dam, a point could be reached where damage may occur to the dam/bridge structure. Additional analysis would be required to quantify the potential scour impacts on the dam structure under this alternative, and to develop appropriate mitigation measures if needed.

The second scenario assumes that the 5th Avenue Bridge and Dam would be removed to create a 500-foot opening. Scour depths were not estimated for this scenario, since any scour would likely be caused by local pier or abutment scour associated with the new bridge. In addition, flow velocities in this area are anticipated to be relatively small, as discussed below in the *Shoreline North of 5th Avenue Dam* section. Under this scenario, the new bridge foundations would need to be located outside any anticipated scour zone, or designed to withstand scour. Regraded slopes associated with creation of the new opening should be protected from erosion using riprap, bio-engineered slopes, slopes reinforced with geosynthetics, or other means.

4th Avenue Bridge

For the existing bridge, assuming that the 5th Avenue Dam remains in place, the maximum combined bridge scour depth is predicted to occur during the 100-

year flood and produce 14.1 feet of scour. In our opinion, the estimated scour depth would compromise the performance of the existing 4th Avenue Bridge. The 500-year flood with the 500-foot opening results in about 9 percent more predicted scour than the open tide gate scenario for the 100-year flood.

For the proposed replacement bridge, the maximum combined bridge scour depth is predicted to be 13.9 feet for the 500-year flood with the open tide gate scenario. The 500-foot opening results in about 4 percent more predicted scour. In our opinion, the proposed bridge will not be compromised by bridge scour.

Percival Creek Bridge

The maximum combined bridge scour depth is predicted to be 1.2 feet. In our opinion, this scour depth does not present a significant impact to the existing bridge. No further geotechnical evaluation is recommended. The 500-foot estuary opening scour results are same essentially the same as for the existing dam and the tide gate open.

Shoreline Area North of the 5th Avenue Dam (vicinity of KFC)

Currently, tidal flow in this area creates velocities of approximately 1 to 2 fps. The hydraulic scour analysis indicates the subject area could frequently experience flow rates on the order of 12 fps if Capitol Lake is operated with the dam in place and the tide gate open. Results indicate the present riprap would be inadequate to protect the existing stream bank under the estimated flow velocities. Consequently, it would be necessary to protect the existing banks with more substantial riprap or an alternative armoring method.

Under the assumed condition of a 500-foot-wide opening at the present dam location, flow velocities were estimated at 1.5 to 2.5 fps along the subject stream bank. The current riprap is considered to be adequate to protect the existing stream bank against erosion. If the existing bank is modified during removal of the existing dam and grading for the new opening, erosion protection measures would be required for the regraded stream bank. These measures might consist of riprap as is currently in place, bio-engineered slopes, or slopes reinforced with geosynthetics.

Deschutes Parkway

The hydraulic scour analyses indicate relatively small increases in flow velocities along the Deschutes Parkway roadway embankments. Estimated velocities range from 0.5 to 1.9 fps. In itself, the increase in flow velocities is not considered to represent a significant impact.

Of greater concern is the frequent fluctuation of water levels adjacent to the roadway embankments, which would occur with the 500-foot opening scenario. The water level would fluctuate between elevation 11.3 feet and –5 feet MSL along the Deschutes Parkway. The frequency of these fluctuations would vary depending on the local tides, storm events, and other factors. In an extreme case, the maximum range of water level fluctuations may occur twice per day. Existing grades along the centerline of Deschutes Parkway, in the area of concern, range generally from about elevation 12 to 16 feet. The toe of the embankment fill slope appears typically near elevation 0 feet or lower. The embankment slopes would therefore be saturated to approximately two-thirds of the slope height or more, and then drained to a level below the toe of the slope, on a cyclical basis. Under this type of "sudden drawdown" condition, the excess pore water pressures within a slope can substantially reduce slope stability, and in an extreme case, can cause slope failure. The cyclical fluctuations in water levels within the slope would also increase the tendency of fill materials within the slope to erode.

In their present state, the earth embankments along substantial sections of Deschutes Parkway are in need of rehabilitation. The roadway experienced substantial damage during the 1965 earthquake event, primarily due to liquefaction of subgrade soils. Much of the Deschutes Parkway alignment along Capitol Lake is still experiencing settlement, and localized areas of the embankment slopes are marginally stable to unstable (**JWM&A, 1997**). In general, areas of ongoing distress coincide with the areas damaged during the 1965 earthquake. The roadway would be subject to extensive damage should another strong earthquake occur in the area.

Based on results of their study, JWM&A (**1997**), supported by Grover C. Way (**1996**) recommended rehabilitation of the existing roadway. Refer to JWM&A (**1997**) for observations, conclusions, and rehabilitation alternatives recommended along specific sections of the roadway. It should be noted that these recommendations were made under the assumption that the current lake management strategy would be continued, and do not consider the effects of tidal fluctuations.

The proposed 500-foot channel opening would result in sudden drawdown conditions on embankment fill slopes two times a day. This condition would result in localized, and possibly widespread, failure of the existing embankment fill slopes unless mitigation measures are implemented. It is our opinion that implementing this scenario would require extensive rehabilitation of the Deschutes Parkway roadway embankments.

Additional geotechnical investigations and analyses would be needed to determine the most practical and cost-effective method of slope remediation. Alternatives, which may be considered, would include construction of toe drainage, slope retaining structures, regraded slopes, geogrid-reinforced embankment fill slopes, and other measures. Design of the remediated embankments must consider erosion protection, as well as internal drainage systems, which would be adequate to resist tidal fluctuations over the long term.

In addition to the above measures, additional ground improvement would be required if it were desired to mitigate the potential for soil liquefaction and related damage during earthquake shaking. Alternatives which may be considered for this purpose would include partial or full overexcavation of liquefiable soils and replacement with stable fill soils; or in-site densification using resonant compaction, vibro-flotation, or other means.

Heritage and Marathon Parks

The hydraulic scour analysis indicates relatively small increases in flow velocities along the shorelines near Marathon and Heritage Parks. Predicted velocities range from 1.3 to 2.1 fps. Without an in-depth study of soil properties at these specific locations, it is difficult to estimate the amount of erosion, which may occur. However, based on the estimated flow velocity increases, it appears that related erosion can be controlled. The estimated velocity increases are in a range that suggests armoring the banks with riprap or vegetation would be adequate to prevent erosion. A new seawall has been constructed along the lake shore at Heritage Park. In our opinion, the estimated flow velocities and the related small increase in erosion potential would not significantly impact the seawall foundations. Prior to implementation of the Estuary Alternative, the wall design and drainage systems should be reviewed to verify that wall stability will be maintained during cyclical high- to low-water conditions.

Steep Slopes East of Capitol Lake

There are steep and potentially unstable slopes above the east side of Capitol Lake. In particular, unstable slopes have been observed along the southeast edge of the North Basin, below the Capitol Building; and also on the eastern side of the South Basin, along the outside edge of a bend in the Deschutes River south of I-5. Other unstable zones may also exist on steep slopes above the lake. Detailed reconnaissance or geologic mapping of these slopes is beyond the scope of the present study.

Fluctuations in lake levels could increase the potential for slope instability along the eastern margin of the lake. This would be a problem particularly if fluctuations in water levels extended above the toe of any unstable slope area. In this event, excess pore water pressures generated during tidal fluctuations could cause slope failures, as discussed above in the *Deschutes Parkway* section.

Prior to implementation of the Estuary Alternative, a detailed geologic reconnaissance should be performed of the steep slopes east of Capitol Lake. Known and potentially unstable areas should be delineated, and the potential impacts of the estuary management strategy on slope stability evaluated. Site-specific geotechnical investigations and analyses may be required to develop mitigation measures for problematic areas identified during the geologic reconnaissance.

References

Entranco, Inc.

1998 Capitol Lake Adaptive Management Plan Draft Programmatic Environmental Impact Statement. Prepared for the Washington State Department of General Administration.

1996 Capitol Lake Restoration and Recreation Plan, Draft Supplemental Environmental Impact Statement. Prepared for the Washington State Department of General Administration.

Jerome W. Morrissette & Associates (JWM&A)

1997 Deschutes Parkway Improvement Study, Olympia Washington. Prepared for the Washington State Department of General Administration.

Way, Grover, C.

1996 Renovation of Deschutes Parkway. Prepared for Jerome W. Morrissette & Associates.

APPENDIX A

Hydraulic Analysis Supporting Information

Table A-5: Creek Flows With Existing Land Use Conditions
Percival Creek Basin
(Ref: HSPF Computer Model, 1990)

Creek Location	Flows (cfs) For 1- to 100-Year Flood Events							
	1	1.25	2	5	10	25	50	100
Percival Creek/Sapp Road	22	38	51	66	75	86	93	107
Percival Creek/Mottman Road	33	59	80	107	124	144	158	172
Black Lake Drainage Ditch/ Mottman Road	66	113	171	235	276	327	363	399
Percival Creek/Percival Cove	117	223	311	421	489	569	625	678
Grass Lake Outlet	4	7	8	10	10	11	11	11
Yauger Park Outlet	15	22	26	30	32	34	35	37
Westside Olympia Outlet	16	31	43	55	62	69	73	77
Ken Lake Outlet	9	19	27	39	46	55	61	67

FLOWS USED FOR PERCIVAL CREEK MODEL

FROM ANDY HAUB
CITY OF OLYMPIA.

PERCIVAL CREEK COMPREHENSIVE DRAINAGE BASIN PLAN.
CITY OF OLYMPIA, 1992.

ENTRANCO

SHEET NO. ____ OF ____

JOB NO. ____

PROJECT CAPITOL LAKE

CALCULATIONS FOR D_{50}

MADE BY POLLON. DATE ____ CHECKED BY ____ DATE ____

STEPS TO DETERMINE D_{50} OF SOILS.

1.) USED CAPITOL LAKE GEOTECHNICAL INVESTIGATION.
→ LOCATED HOLES CLOSE TO AREA OF INTEREST.

#34 FOR PERCIVAL CREEK
#19 FOR I-5
#12 FOR BNSF & CAPITOL DAM.

2.) USED % OF MATERIALS FROM TABLE A-2 TO PLOT CURVES.

3.) CONVERTED D_{50}(MM) TO D_{50}(FT)

Table 1
Summary of peak flow from XP-SWMM

Storm frequency	Dam peak flow (cfs)		Rail Trestle peak flow (cfs)		I5-Bridge peak flow (cfs)		4th Avenue Bridge peak flow (cfs)	
	Open Gates	Estuary	Open Gates	Estuary	Open Gates	Estuary	Open Gates	Estuary
500	23120	25569	20693	22300	16580	16946	23120	25569
100	20980	22406	17810	18640	13097	13211	20980	22406
50	19923	20000	16647	17000	11570	11728	19923	20000
25	*18618*	*20000*	*15335*	*15479*	*10176*	*10314*	*18618*	20000

Assumptions:

- Lake level starts at 5.43.
- Estuary model simulates tide gates as open channel with 500 ft wide top opening and 3H:1V slope.
- Open gates model simulates the existing tide gates as two box culverts open at all time with combined capacity at 10,000 cfs.

WEIBULL	1					
STATISTIC WITH		ZERO	FLOW	YEARS	REMOVED	
MEAN	LOG:	3.6043				
STANDARI DEVIATION:		0.1981				
SKEW:	0.1046					
YEARS:	27					
FREQUEN WITH		ZERO	FLOW	YEARS	REMOVED	
PROB=	0.999	FLOW=	1441.4			
PROB=	0.5	FLOW=	3989.3			
PROB=	0.2	FLOW=	5887			
PROB=	0.1	FLOW=	7249.2			
PROB=	0.04	FLOW=	9082.1			
PROB=	0.02	FLOW=	10523.4			
PROB=	0.01	FLOW=	12030.9			
PROB=.	0.002	FLOW=	15842.4			
THE	LOW-OUTLI	THRESHO	VALUE	IS	1274.14	

FINAL	STATISTICS		
MEAN	LOG:	3.6043	
STANDARI DEVIATION:		0.1981	
SKEW:	0.1046		
YEARS:	27		

FINAL	FREQUENCIES				
PROB=	0.999	FLOW=	1,441	1.001001	year average period when this flow is equaled or exceede
PROB=	0.5	FLOW=	3,989	2	year average period when this flow is equaled or exceede
PROB=	0.2	FLOW=	5,887	5	year average period when this flow is equaled or exceede
PROB=	0.1	FLOW=	7,249	10	year average period when this flow is equaled or exceede
PROB=	0.04	FLOW=	9,082	25	year average period when this flow is equaled or exceede
PROB=	0.02	FLOW=	10,523	50	year average period when this flow is equaled or exceede
PROB=	0.01	FLOW=	12,031	100	year average period when this flow is equaled or exceede
PROB=	0.002	FLOW=	15,842	500	year average period when this flow is equaled or exceede

WEIBULL 1

SCALE	ANN PEAK	1-P	YR	MO	DA
0.8686	10,700	0.9779	1996	9	30
0.7997	9,600	0.9412	1990	9	30
0.7613	6,760	0.9044	1997	9	30
0.731	6,650	0.8676	1964	9	30
0.7083	6,410	0.8309	1991	9	30
0.6863	6,080	0.7941	1956	9	30
0.6679	5,000	0.7574	1963	9	30
0.6503	4,920	0.7206	1961	9	30
0.6339	4,780	0.6838	1954	9	30
0.6192	4,750	0.6471	1947	9	30
0.6049	4,600	0.6103	1951	9	30
0.5903	4,210	0.5735	1957	9	30
0.5762	4,080	0.5368	1950	9	30
0.5619	3,990	0.5	1995	9	30
0.5482	3,860	0.4632	1949	9	30
0.534	3,570	0.4265	1998	9	30
0.5202	3,540	0.3897	1955	9	30
0.505	3,380	0.3529	1992	9	30
0.4896	3,340	0.3162	1960	9	30
0.4741	3,040	0.2794	1959	9	30
0.4565	2,990	0.2426	1952	9	30
0.4377	2,950	0.2059	1948	9	30
0.4172	2,870	0.1691	1953	9	30
0.3934	2,720	0.1324	1958	9	30
0.3645	1,940	0.0956	1962	9	30
0.3253	1,720	0.0588	1994	9	30
0.2562	1,700	0.0221	1993	9	30

FLOW(CF:	1-P	SCALE	CI UPP	CI LOW
1.44E+03	0.01	0.21	2.08E+03	1.14E+03
3.99E+03	0.5	0.56	4.65E+03	3.43E+03
5.89E+03	0.8	0.689		
7.25E+03	0.9	0.757	8.59E+03	5.66E+03
9.08E+03	0.96	0.827		
1.05E+04	0.98	0.874		
1.20E+04	0.99	0.916	1.52E+04	8.32E+03
1.58E+04	0.998	1		

CAP	HSPF	FILE	FOR	DRIVING	SEPARATI	PLOT	PROGRAM			
CAP	Time	Interval:		-2 mins	Last	month	In	printout	year:	9
CAP	No.	of	curves	plotted:	Point-value	8	Mean-valu		0 Total	8
CAP	Label	flag:	0	Pivl:	Year	Idelt:	15			
CAP	Plot	title:	CAPER	BEAR	CRK-EXIS'	CONDITIO	(PK-			
CAP	Y-axis	label:	FCAP	(CFS)						
CAP	Scale	Info:	Ymin:	0.00E+00	Threshold:-0.10000E+31					
CAP	Ymax:	1500								
CAP	Time:	20	Intervals/inch							
CAP	Data	for	each	curve	(Point-valu	first,	then	mean-valued):		
CAP	Label	LINTYP	INTEQ	COLCOD	TRAN	TRANCOD				
CAP	DESCHUTE	R.	AT	CAPITOL	LK	0	0	0	MAX	3
CAP	0	0	0	MAX	3					
CAP	0	0	0	MAX	3					
CAP	0	0	0	MAX	3					
CAP	0	0	0	MAX	3					
CAP	0	0	0	MAX	3					
CAP	0	0	0	MAX	3					
CAP	0	0	0	MAX	3					
CAP										
CAP										
CAP	Time	series	(pt-valued,	then	mean-valued):					
CAP										
CAP	Date/time	Values								
CAP										
CAP	1946	9	30	24	0	3270				
CAP	1947	9	30	24	0	4750				
CAP	1948	9	30	24	0	2950				
CAP	1949	9	30	24	0	3860				
CAP	1950	9	30	24	0	4080				
CAP	1951	9	30	24	0	4600				
CAP	1952	9	30	24	0	2990				
CAP	1953	9	30	24	0	2870				
CAP	1954	9	30	24	0	4780				
CAP	1955	9	30	24	0	3540				
CAP	1956	9	30	24	0	6080				
CAP	1957	9	30	24	0	4210				
CAP	1958	9	30	24	0	2720				
CAP	1959	9	30	24	0	3040				
CAP	1960	9	30	24	0	3340				
CAP	1961	9	30	24	0	4920				
CAP	1962	9	30	24	0	1940				
CAP	1963	9	30	24	0	5000				
CAP	1964	9	30	24	0	6650				
CAP	1990	9	30	24	0	9600	*Note: this value was obtained during preparation of EIS - source unknown, not reported in USGS data.*			
CAP	1991	9	30	24	0	8410				
CAP	1992	9	30	24	0	3380				
CAP	1993	9	30	24	0	1700				
CAP	1994	9	30	24	0	1720				
CAP	1995	9	30	24	0	3990				
CAP	1996	9	30	24	0	10700				
CAP	1997	9	30	24	0	6760				
CAP	1998	9	30	24	0	3570				
WEIBULL										

APPENDIX A
CAPITOL LAKE GEOTECHNICAL INVESTIGATION

A geotechnical investigation in Capitol Lake was performed to identify the physical and chemical characteristics of the sediments.

FIELD SAMPLING

Sediment samples were collected from 37 test holes ranging in depth from 1 to 22 feet below the lake bottom between 20 November 1975 and 5 December 1975. A portable drilling barge, drilling equipment, and two men were supplied by Anderson Drilling, Inc., of Bellevue, Washington, under subcontract to CH2M HILL. CH2M HILL provided jetting and coring equipment, an inflatable rubber boat, and a geologist and/or engineering technician. The drilling barge was used to investigate 15 test holes in the upper basin and 18 test holes in the middle basin. Four test holes were made in Percival Cove. The test hole locations are shown in figure A-1.

Samples were collected in all but three test holes using a 2-foot clear plastic tube (2-inch OD and 1/16-inch wall) attached to the bottom of a 3/4-inch pipe. The sample tube was jetted into the lake bottom to the desired sample depth, then hand pushed or driven approximately 2 feet using an 18-lb sliding hammer. A ballcheck valve was inserted in the 3/4-inch pipe, and the sample was withdrawn from the test hole. Jetting was performed using a pump having a capacity of 60 gpm at 100 psi.

Standard penetration tests were performed in test holes 9, 22, and 23. This test consists of a 140-lb hammer falling 30 inches to drive a 2-inch OD split-barred sampler 18 inches. The number of blows required to drive the sampler the final 12 inches was recorded as the driving resistance (N value). The standard penetration tests were not used in all test holes because sample recovery was very difficult. There was better sample recovery when the clear plastic tubes were used.

LABORATORY TESTING

Physical Properties

Physical tests on the sediments included visual classifications as well as particle size analysis by means of sieve and hydrometer analysis according to ASTM D 422.[1] Specific gravities are given in table A-1.

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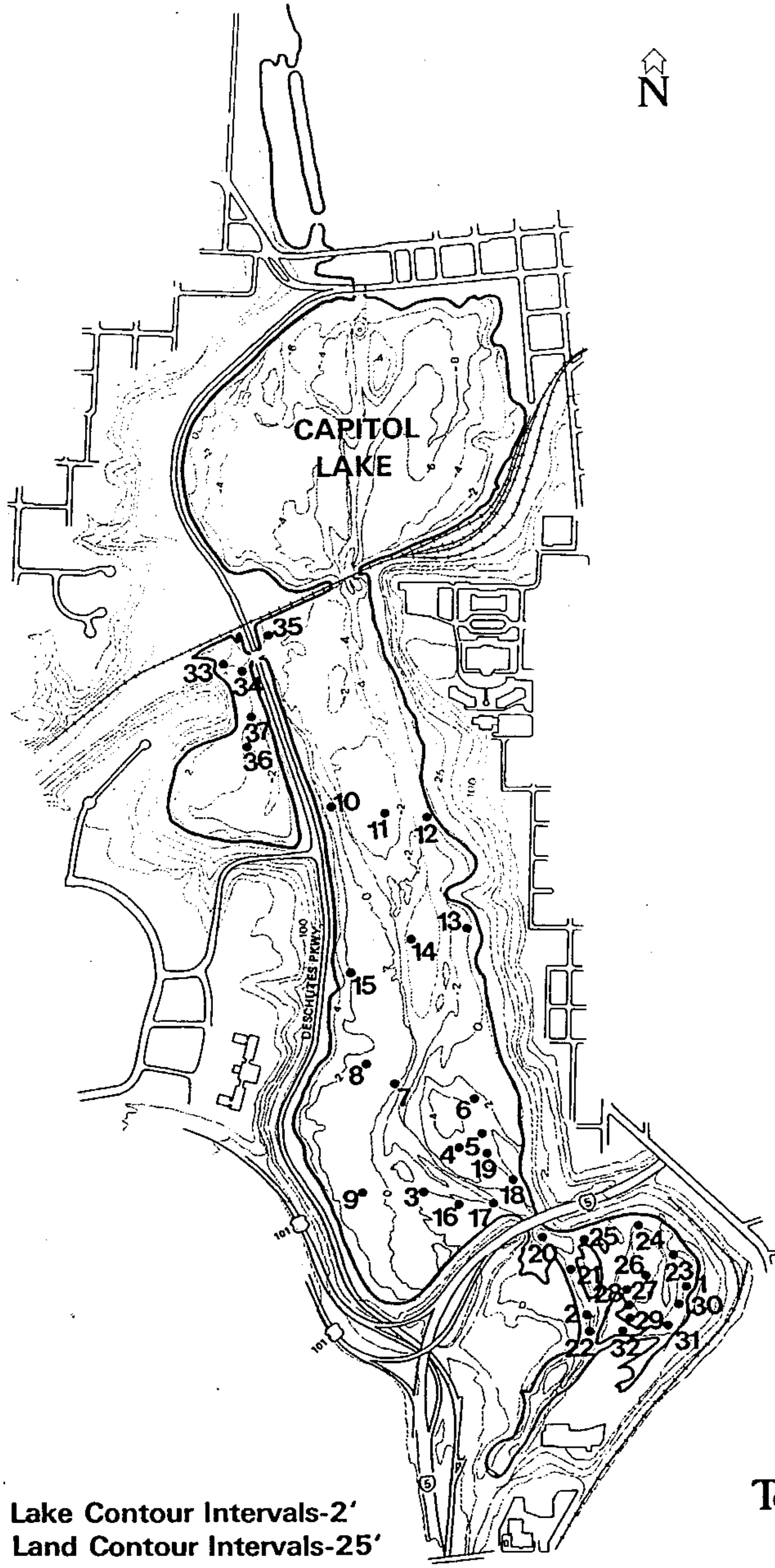
1 American Society of Testing Materials classification.



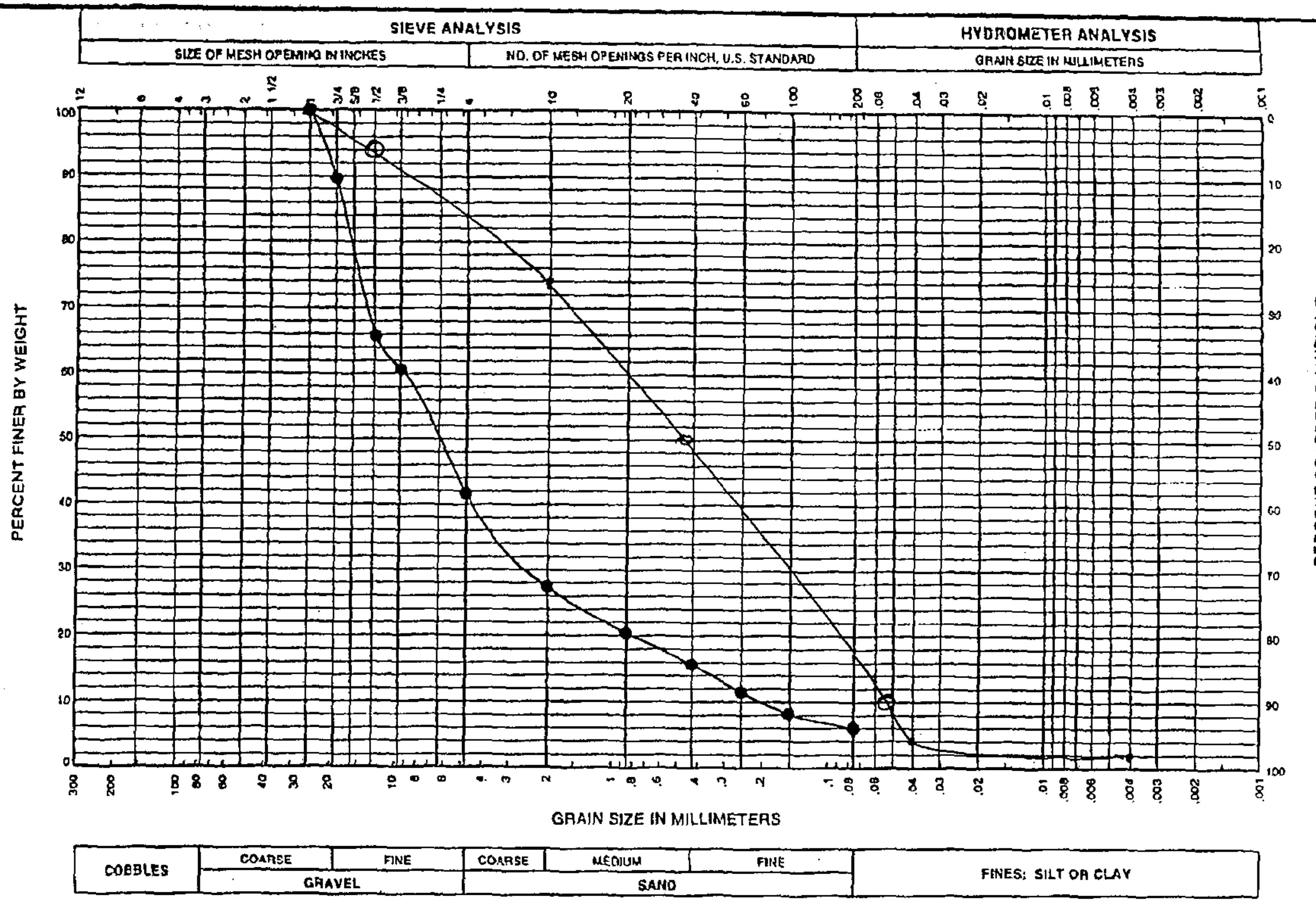
Lake Contour Intervals-2'
Land Contour Intervals-25'

Test Boring Location A-1

Table A-2. PHYSICAL PROPERTIES

Test Hole Number	Sample Depth (ft)	Percent Gravel >5mm	2.537 Percent Sand .074-5mm	0.0395 Percent Silt .005-.074mm	.004 Percent Clay <.005mm	Unified Classification	Agricultural Classification	Percent Passing No. 200 Sieve
1	0-0.2	0	60	30	10	Silty fine sand (SM)	Sandy loam	40
	1.1-1.3	3	86	6	5	Silty fine sand (SP-SM)	Gravelly sand	11
	6.0-7.7	0	52	39	9	Silty fine sand (SM)	Sandy loam	48
	10.5-12.2	45	51	3	1	Gravelly sand (SP)	Gravelly sand	3
2	1.3-3.0	2	67	18	13	Silty fine sand (SM)	Sandy loam	31
	3.4-4.7	1	87	8	4	Silty sand (SP-SM)	Gravelly sand	12
3	0-1.0	1	87	8	4	Silty sand (SW-SM)	Gravelly sand	12
	4.0-5.2	0	50	36	14	Silty fine sand (SM)	Sandy loam	50
6	1.0-1.3	0	16	63	21	Sandy silt (ML)	Silty loam	84
	4.0-4.9	1	7	69	23	Clayey silt (ML)	Gravelly silty loam	92
7	0.5-1	0	7	70	23	Clayey silt (ML)	Silty loam	93
	4.0-5.3	0	2	67	31	Clayey silt (ML)	Silty loam	98
12	0.5-0.8	0	33	57	10	Sandy silt (ML)	Sandy loam	67
	4.0-5	1	22	60	17	Fine sandy silt (ML)	Loam	77
13	0-2.0	0	25	53	22	Sandy Silt (ML)	Silty loam	75
19	2.0-4.0	0	14	63	23	Silt (ML)	Loam	86
	6.0-8.0	0	29	54	17	Sandy silt (ML)	Sandy loam	71
26	0-2.0	0	78	18	4	Silty fine sand (SM)	Loamy sand	22
	8.0-10	0	23	60	17	Fine sandy silt (ML)	Loam	77
34	0-1.5	26	70	2	2	Gravelly sand (SP)	Gravelly sand	4
37	1-1.6	0	9	66	25	Clayey silt (ML)	Silty loam	91

GSAMAIN1 W8674-05.GPJ SHAN WIL.GDT 1/3/00



BORING AND SAMPLE NO.	DEPTH (meters)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-8, S-10	13.72	GP-GM	Gray, slightly silty, sandy GRAVEL	6.0	9.2			

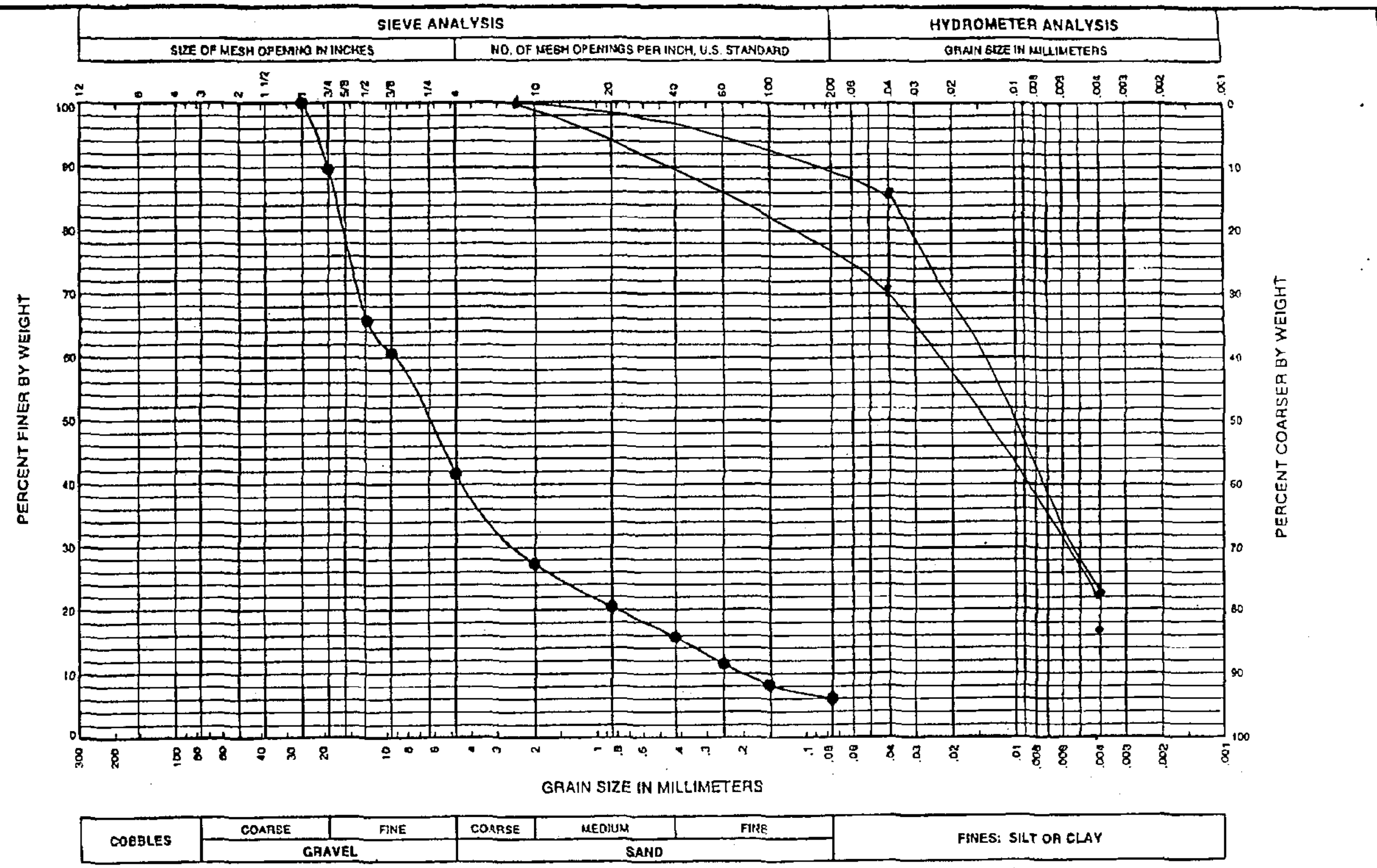
#34 PERCIVAL CR.

$D_{50} = .4$

IN FT = .0013

FIG. 3

GSAMAIN1 W6674-05.GPJ SHAN WIL.GDT 1/3/00



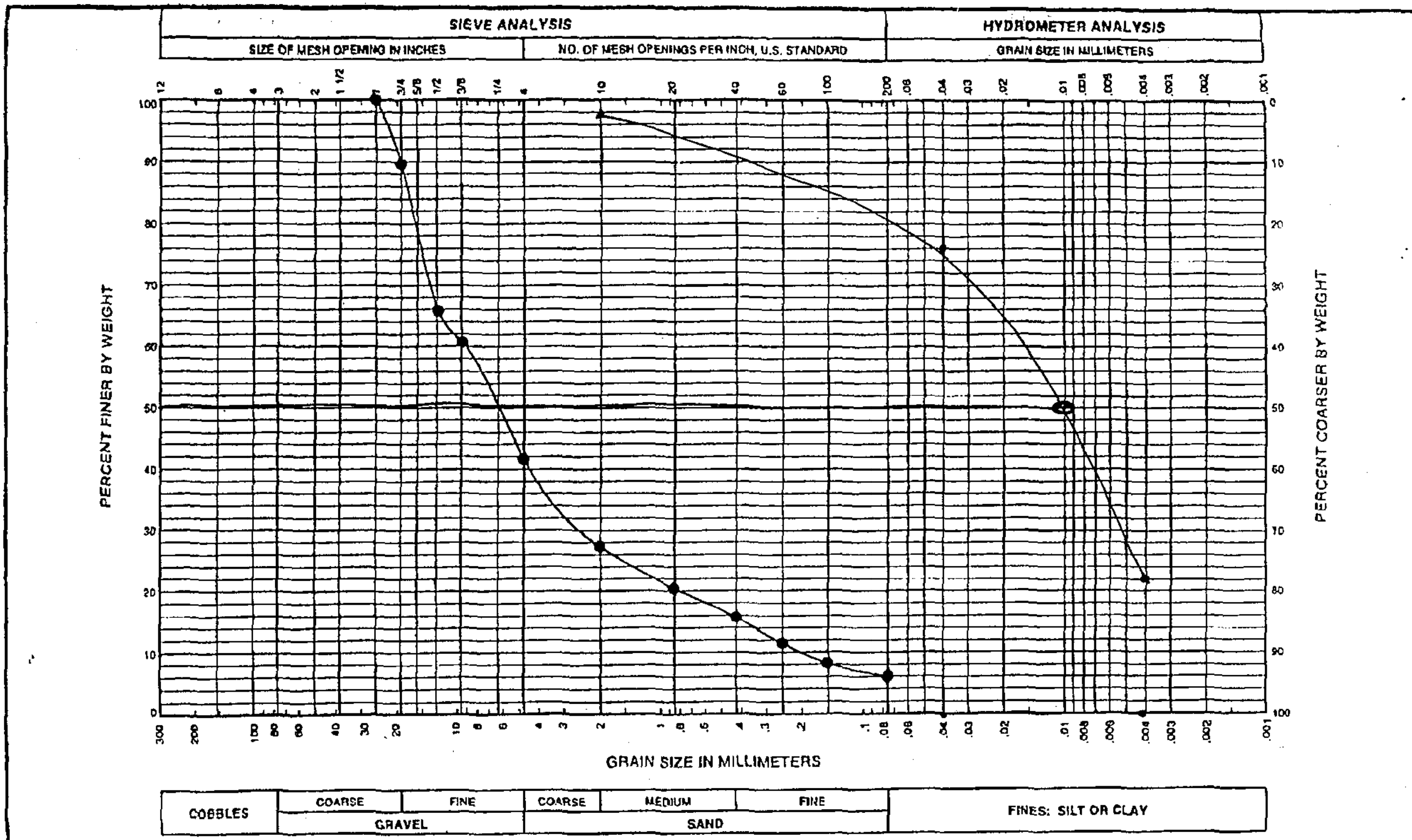
BORING AND SAMPLE NO.	DEPTH (meters)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-8, S-10	13.72	GP-GM	Gray, slightly silty, sandy GRAVEL	6.0	9.2			

TEST HOLE #19
I-5 BRIDGE
2-4' = D50 = .01
6-8' = D50 = .015

= 0.00003

FIG. 3

GSAMAIN1 W8674-05.GPJ SHAN WL.GDT 1/3/00



BORING AND SAMPLE NO.	DEPTH (meters)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● B-8, S-10	13.72	GP-GM	Gray, slightly silty, sandy GRAVEL	8.0	9.2			

TEST HOLE #12
D50 = .01mm = .0004IN
USE FOR DAM
RR TRESTLE
→ D50(FT) = .00003

FIG. 3

APPENDIX B

Geotechnical Evaluation

AGI
TECHNOLOGIES

Supplemental Geotechnical Evaluation Implementation of the Capital Lake Adaptive Management Plan Olympia, Washington

February 2, 2000

Prepared For:

Entranco Engineers, Inc.
10900 NE 8th Street, Suite 300
Bellevue, Washington 98004

AGI Project No. 14,715.011

AGI
TECHNOLOGIES
A CDM COMPANY

A Report Prepared For:

Entranco Engineers, Inc.
10900 NE 8th Street, Suite 300
Bellevue, Washington 98004

SUPPLEMENTAL GEOTECHNICAL EVALUATION
IMPLEMENTATION OF THE CAPITOL LAKE ADAPTIVE MANAGEMENT PLAN
OLYMPIA, WASHINGTON

February 2, 2000

SCOTT L. HARDMAN
STATE OF WASHINGTON
28265
REGISTERED
PROFESSIONAL ENGINEER
EXPIRES 3/18/2001

2.3.00

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AGI Project No. 14,715.011

AGI
TECHNOLOGIES
A CDM COMPANY

INTRODUCTION

Pursuant to your request, AGI Technologies (AGI) performed geotechnical evaluations for implementation of the Capitol Lake Adaptive Management Plan. This report presents the findings of our study regarding the Estuary Alternative for lake management. Specifically, AGI provided geotechnical input to Entranco's Hydraulic Scour Analysis (Task 3 of the overall scope of work).

PROJECT DESCRIPTION

The Capitol Lake Adaptive Management Plan outlined several technical studies to provide information for future management decisions concerning Capitol Lake. Included in these studies was a hydraulic scour analysis to determine the impacts of opening the 5th Avenue dam and allowing tidal exchange into the north, middle, and south basins of Capitol Lake. The key question is: What are the predicted scour impacts to existing bridges, roadways, parkland, and the dam resulting from tidal exchange and flood events? A qualitative analysis was provided in the Capitol Lake Adaptive Management Plan Environmental Impact Statement which concluded that scour impacts could threaten existing structures without mitigation.

SCOPE OF SERVICES AND AUTHORIZATION

Task 3 of the overall scope of work for this study involves a Hydraulic Scour Analysis. As part of this analysis, Entranco was to develop cross sections at up to nine key locations including:

- 5th Avenue dam.
- I-5 bridge.
- BNSF railroad trestle.
- Percival Creek bridge.
- Deschutes Parkway.
- Marathon and Heritage Parks.
- Shoreline north of 5th Avenue dam (vicinity of KFC).

As outlined in an e-mail from Entranco to AGI dated January 20, 1999, geotechnical services to support the hydraulic scour analysis consisted of evaluating potential impacts of predicted scour on the selected locations (identified above), and discussion of management/operational implications, including erosion of the subgrade of the Deschutes Parkway. In addition, potential impacts on steep slopes east of the lake are addressed. Verbal authorization for the geotechnical services was subsequently given by Mr. Dale Anderson of Entranco. The scope of work completed for this project was consistent with that described above, and included reviewing available geologic information and previous geotechnical reports for related Capitol Lake projects, performing a site reconnaissance, assessing potential geotechnical impacts, and preparing this letter report.

AGI
TECHNOLOGIES
A CDM COMPANY

SITE GEOTECHNICAL CONDITIONS

Subsurface and geologic conditions were determined based on information collected during our site reconnaissance and as part of our data review. The following section summarizes AGI's literature review and site reconnaissance. Anticipated subsurface conditions at the site are described based on the data reviewed. This study did not include subsurface exploration or quantitative geotechnical analyses.

REGIONAL GEOLOGIC CONDITIONS

Geological information for the site was obtained from the *Geologic Map of Thurston County, Washington* (Noble, 1962). The geologic map indicates that Capitol Lake and immediate vicinity are generally underlain by alluvium and recessional outwash. Alluvium, the more recent of the two geologic deposits, typically consists of fine-grained floodplain deposits. Lake bed deposits of similar composition are also included in the alluvium classification.

Recessional outwash consists of non-stratified, non-sorted sand and gravel deposits. These deposits were placed during glacial retreat and have not been consolidated by glacial ice. The geologic map denotes recessional outwash as a surficial deposit located on the hillsides around the lake.

FINDINGS FROM PREVIOUS GEOTECHNICAL INVESTIGATIONS

Previously, Milbor-Pita (1996), JWM&A (1997), HWA (1994), WSDOT (1983), RZA (1982), and CH2M Hill (1976) performed subsurface investigations at Capitol Lake and prepared reports summarizing their findings and geotechnical recommendations for related improvement projects. These reports provide information on subsurface conditions for the South, Middle, and North Basins of Capitol Lake. The WSDOT (1983) report addresses geotechnical issues associated with widening the I-5 bridge over Capitol Lake. Milbor-Pita (1996) provides information on subsurface conditions at Heritage Park, along the southeast shoreline of the North Basin. The JWM&A (1997) report includes information on subsurface conditions along Deschutes Parkway. In addition, HWA (1998) evaluated potential geotechnical issues associated with a combined lake/estuary alternative. HWA's study included an evaluation of potential impacts and construction issues associated with construction of a dam dividing the existing North Basin into separate reflecting pool and estuary zones.

The general consensus of the available reports is that the site is underlain by soft/loose silts and fine sands with organic matter deposited in stream and lake depositional environments. These deposits range from about 15 to 50 feet in thickness. In general, the thickness increases toward the north, with the deepest deposits encountered in the North Basin. Exploration logs indicate medium dense to very dense, sand and gravel deposits generally underlie the soft silts and sands. Findings of specific studies are summarized below:

- Explorations by Milbor-Pita, up to 45 feet deep along the southeast shoreline of the North Basin, did not encounter dense to very dense soil.

- Exploratory borings were performed along Deschutes Parkway west of Capitol Lake by Grover C. Way (1996), as part of the JWM&A (1997) study. These explorations indicated the presence of soft/loose silty sand, silt and clayey silt with occasional pockets of peat to depths of about 27 to 50 feet below road grade. Medium dense to very dense sand and gravel deposits were encountered at variable depths beneath the soft/loose surficial soils.

- Soil conditions similar to the JWM&A study were encountered at the I-5 bridge crossing of Capitol Lake (WSDOT, 1983), with the reported depth of soft surficial soils ranging from 20 to 40 feet below existing ground surface.

AGI SITE RECONNAISSANCE

On January 28, 2000, AGI performed a site reconnaissance in which they examined existing soil conditions and riprap protection on existing embankments at the study locations listed in the *Scope of Services and Authorization* section above. This surficial reconnaissance confirmed, in part, the findings described by the above sources. Specific observations relative to this study are discussed in more detail in the following sections.

During the site visit AGI determined that portions of the hillsides southeast of the North Basin and east of the Middle Basin consist of glacial till. Till consists of a glacially consolidated, non-sorted mixture of clay, silt, sand, and gravel and typically underlies recessional outwash. Glacial till typically exhibits high shear strength, low compressibility, and low permeability characteristics. The till appears blanketed by a relatively thin layer of recessional outwash.

GEOLOGIC HAZARDS

Geologic hazards potentially impacting alternatives for lake management include seismic, soil liquefaction potential, settlement, landslide and erosion hazards. These potential geologic hazards, are summarized below, and are discussed in more detail in the previous geotechnical studies for the project (HWA, 1994 and 1998).

SEISMIC HAZARDS

Seismic hazard areas are generally defined as areas subject to severe risk of earthquake damage as a result of seismically induced settlement or soil liquefaction. Since the 1850's, at least 25 earthquakes of Magnitude 5.0 (Richter Scale) or greater have reportedly occurred in the Puget Sound and North Cascades region. Four events may have exceeded Magnitude 6.0. These include a 1949 event near Olympia (Magnitude 7.2), and a 1965 event centered between Seattle and Tacoma (Magnitude 6.5). The subduction of the Juan De Fuca plate beneath the North American plate is believed to directly or indirectly cause most of the earthquakes in Washington (Noson et al., 1988).

The subject site lies within Seismic Zone 3 as defined by the Uniform Building Code (ICBO, 1997). Seismic Zone 3 includes western Washington, and represents an area susceptible to moderately high seismic activity. For comparison, much of California and southern Alaska are defined as Seismic Zone 4, which is an area of higher seismic risk. Consequently, moderate levels of earthquake shaking should be anticipated for earth embankments and structures associated with management of Capitol Lake.

SOIL LIQUEFACTION POTENTIAL

When shaken by an earthquake, certain soils lose strength and temporarily behave as a liquid. This phenomenon is known as soil liquefaction. The seismically induced loss of soil strength can result in failure of the ground surface that is most typically expressed as landslides or lateral spreads, surface cracks and settlement, and/or sand boils. During a large seismic event, substantial damage can occur to structures supported in or on liquefiable soils. Seismically induced liquefaction typically occurs in loose, saturated sandy materials commonly associated with recent river, lake and beach sedimentation.

Based on the information collected as part of the literature review, the surficial lakebed sediments in Capitol Lake are expected to consist of loose/soft soils. Loose, sandy zones within the lakebed deposits may be susceptible to liquefaction during a seismic event. By 1965, Deschutes Parkway had been fully developed as a motor vehicle route around Capitol Lake. The earthquake event of April 29, 1965 caused significant damage along Deschutes Parkway. Considering the type of soil in the failure areas, the probable method of construction and relatively low vertical height of the roadway section, the recorded failures were most likely a result of subgrade soil liquefaction.

SETTLEMENT HAZARDS

Based on the literature review and site reconnaissance, as well as the depositional environment of the site, it is anticipated that the near-surface soils generally consist of silts and fine sands with occasional organics. High levels of settlement can occur when additional loads are applied to these types of soils. Earth embankments or structures constructed on such compressible soils would require mitigating measures to prevent damage caused by excessive settlement.

LANDSLIDE HAZARD

Information on slope stability for the Capitol Lake area was obtained from the *Slope Stability Map of Thurston County, Washington* (Artim, 1976). A landslide hazard is shown to exist in the vicinity of the North Basin, and within other steep slope areas east of Capitol Lake.

During HWA's site reconnaissance in 1998, landslide debris was observed on the hillside south of the North Basin and partially covering the railroad tracks. At the time of our site reconnaissance for this study, the landslide debris had been removed, and it appeared that slope stabilization measures had been implemented. Evidence of instability was also noted in the steep slope area east of the South Basin, along the outside edge of a bend in the Deschutes River south of I-5.

EROSION HAZARD

Erosion hazard areas are defined as those areas containing soils that may experience severe to very severe erosion during development activities. Based on our site reconnaissance and literature review, we did not identify any extensive erosion hazard areas on upland portions of the site. Localized erosion hazard areas exist on portions of the Deschutes Parkway embankment fills, and within unstable steep slope areas. When disturbed and/or stripped of vegetation, soils will be subject to erosion. Soils most susceptible to erosion when disturbed include the granular recessional outwash unit, and sandy portions of alluvium or fill deposits. In addition, increases in flow velocities and water level fluctuations will increase the erosion potential of the affected soils. Unvegetated lake bottom sediments may represent an erosion hazard when subjected to tidal fluctuations, depending on soil type and flow velocities.

RESULTS OF GEOTECHNICAL EVALUATIONS

Based on our site reconnaissance, data review, and results of Entranco's hydraulic scour analysis, AGI evaluated potential impacts of the Estuary Alternative for lake management. Our evaluation focused on the areas specifically outlined in the scope of work. In addition, potential impacts on steep slopes ascending above the eastern side of the lake are addressed. The scope of work did not include detailed slope stability, seismic, or subgrade soil stability analyses. The conclusions drawn are based on best professional judgment, given the data presently available. Additional geotechnical investigations and analyses would be required to assess the geotechnical issues more definitively, and to develop appropriate mitigation measures where needed.

5TH AVENUE DAM

The 5th Avenue Dam is located at the north end of the North Basin. It presently serves to keep tidal water out of the lake and dams the Deschutes River to form Capitol Lake. One management strategy under consideration is to leave the tide gates in the fully open position, allowing Capitol Lake to become a natural estuary environment. On the upstream side of the dam is a feature known as the Tide Gate Crater. It has existed since the lake was first created in 1951. The bottom of the crater is approximately 20 to 25 feet lower than the surrounding lake bed elevations. This crater is believed to have formed as a result of hydraulic forces generated when the lake is backfilled with marine water.

The hydraulic scour analysis indicates that under the alternative of opening the dam's tide gates, scour depth at the dam may reach depths of 32 feet under the assumed 500-year condition. This depth of scour could occur on either side of the dam structure. The present Tide Gate Crater has not impaired performance of the dam. The bottom of the dam is concrete, so scour is not expected there. However, it is our opinion that if significant scour features are allowed to develop and deepen on both sides of the dam, damage may occur to the dam/bridge structure. Additional analysis would be required to quantify the potential impacts of scour on the dam structure under this alternative, and to develop appropriate mitigation measures if needed.

The second alternative assumes that the 5th Avenue bridge and dam will be removed, to create an opening approximately 500 feet wide. Depths of scour were not estimated for this alternative, since any scour would likely be caused by local pier or abutment scour associated with the replacement bridge. In addition, flow velocities in this area are anticipated to be relatively small, as discussed in the *Shoreline North of 5th Avenue Dam* section below. Under this alternative, the new bridge foundations would need to be located outside the zone of any anticipated scour, or designed to withstand scour. Regraded slopes associated with creation of the new opening should be protected from erosion using riprap, bio-engineered slopes, slopes reinforced with geosynthetics, or other means.

I-5 BRIDGE

Results of the hydraulic scour analysis indicate total estimated scour of 9.5 feet for the assumed 500-year event, and the alternative of opening the gates at the 5th Avenue dam. Estimated scour was j10.9 feet for the option of creating a 500-foot wide opening at the dam location. The WSDOT (1983) geotechnical study for I-5 bridge widening recommended supporting all piers of the widened bridges on pile foundations. Concrete and H-pile alternatives are discussed in the WSDOT study, and concrete piles are recommended as the preferred alternative. Piles are recommended to be driven into the underlying dense sand and gravel unit, encountered at depths of 20 to 40 feet below ground surface at the bridge pier locations. At the time of this study, "as-built" drawings were not available of the original I-5 bridge and bridge widenings. Additional research would be needed to determine the actual pile type and depth beneath the widened bridges, as well as foundation types and depths of the original bridge.

In our opinion, the estimated depths of scour may represent a significant potential impact to the existing I-5 bridge. It does not appear that bridge foundations would be undermined. However, scour adjacent to bridge abutments, pile caps, and piles would result in loss of lateral support which could potentially compromise the performance of the structure under static or seismic loading. WSDOT, and prudent engineering practice, would dictate further analysis of the impacts of this magnitude of scour on the integrity of the bridge foundations and superstructure. Additional geotechnical analysis would be needed to develop suitable mitigation measures, which may include protecting the bridge piers with heavy rip-rap, strengthening bridge foundations, or other means.

BNSF RAILROAD TRESTLE

Results of the hydraulic scour analysis indicate total estimated scour of 19.1 feet for the assumed 500-year event, and the alternative of opening the gates at the 5th Avenue dam. Estimate scour was 20.5 feet for the option of creating a 500-foot wide opening at the dam location. The existing bridge is supported on timber piles of unknown depth. Typical timber pile lengths range from 40 to 50 feet. Given the anticipated depth to the dense sand and gravel layer underlying the site, we consider it likely that existing timber piles are on the order of 50 feet in length.

In our opinion, the estimated depths of scour could compromise the performance of the structure under static or seismic loading. Additional analysis would be required to evaluate the impacts of this magnitude of scour on the integrity of the bridge foundations and superstructure. Suitable mitigation measures could include protecting the bridge piers with heavy riprap. If such countermeasures are not sufficient, it may be necessary in an extreme case to replace the bridge, using a foundation system capable of resisting the anticipated scour.

PERCIVAL CREEK BRIDGE

Estimated scour at the Percival Creek bridge is minimal. Slightly greater than 1 foot of scour is predicted for both of the dam opening alternatives evaluated. In our opinion, this depth of scour does not present a significant impact to the existing bridge. No further geotechnical evaluation is recommended.

DESCHUTES PARKWAY

Results of the hydraulic scour analysis indicate relatively small increases in flow velocities along the Deschutes Parkway roadway embankments. Estimated velocities range from about 1 to 2 feet per second (ft/s). In itself, the increase in flow velocities is not considered to represent a significant impact. Of greater concern is the frequent fluctuation of water levels adjacent to the roadway embankments, which would occur under the Estuary Alternative.

Estimates provided by Entranco indicate the water level would fluctuate between Elevation 11.3 feet and -5 feet MSL along Deschutes Parkway. The frequency of these cycles would vary depending on the local tides, storm events, and other factors. In an extreme case, the maximum range of water level fluctuations may occur twice per day. Existing grades along the centerline of Deschutes Parkway in the area of concern range generally from about Elevation 12 to 16 feet. The toe of the embankment fill slope appears typically near Elevation 0 feet or lower. The embankment slopes will therefore be saturated to approximately 2/3 the slope height or more, and then drained to a level below the toe of the slope, on a cyclical basis. Under this type of "sudden drawdown" condition, the excess pore water pressures within a slope can substantially reduce slope stability, and can in an extreme case cause slope failure. The cyclical fluctuations in water levels within the slope will also increase the tendency of fill materials within the slope to erode.

Periodically, Capitol Lake is drawn down for maintenance. Due to concerns regarding slope instability under "sudden drawdown" conditions, a staged draw down process is implemented, allowing at least a full week to lower the lake completely during dry weather. This allows excess pore water pressures to dissipate slowly, to maintain stability of the existing slopes.

In their present state, the earth embankments along substantial sections of Deschutes Parkway are in need of rehabilitation. As discussed above, the roadway experienced substantial damage during the 1965 earthquake event, primarily due to liquefaction of subgrade soils. Much of the Deschutes Parkway alignment along Capitol Lake is still experiencing settlement, and localized areas of the embankment slopes are marginally stable to unstable (JWM&A, 1997). In general, areas of ongoing distress coincide with areas damaged during the 1965 earthquake. The roadway is subject to extensive damage should another strong earthquake occur in the area.

Based on results of their study, JWM&A (1997), supported by Grover C. Way (1996) recommended rehabilitation of the existing roadway. Refer to JWM&A (1997) for observations, conclusions and rehabilitation alternatives recommended along specific sections of the roadway. It should be noted that these recommendations were made under the assumption that the current lake management strategy would be continued, and do not consider the effects of tidal fluctuations.

The proposed Estuary Alternative would result in sudden drawdown conditions on embankment fill slopes as frequently as twice daily. This condition would result in localized, and possibly widespread, failure of the existing embankment fill slopes unless mitigation measures are implemented. It is our opinion that implementing the Estuary Alternative would require extensive rehabilitation of the Deschutes Parkway roadway embankments.

Additional geotechnical investigations and analyses would be needed to determine the most practical and cost effective method of slope remediation. Alternatives which may be considered would include construction of toe drainage, slope retaining structures, regraded slopes, geogrid-reinforced embankment fill slopes, and/or other measures. Design of the remediated embankments must consider erosion protection and internal drainage systems that would be adequate to resist tidal fluctuations long-term.

In addition to the above measures, additional ground improvement would be required if it were desired to mitigate the potential for soil liquefaction and related damage during earthquake shaking. Alternatives which may be considered for this purpose would include partial or full overexcavation of liquefiable soils and replacement with stable fill soils; or in-site densification using resonant compaction, vibro-flotation, or other means.

MARATHON AND HERITAGE PARKS

Results of the hydraulic scour analysis indicate relatively small increases in flow velocities along the shorelines near Marathon and Heritage Parks. Estimated velocities range from about 1 to 2 ft/s. Without an in-depth study of soil properties at these specific locations, it is difficult to estimate the amount of erosion which may occur. However, based on the flow velocity increases estimated, it appears that related erosion can be controlled. The estimated velocity increases are in a range that suggests armoring the banks with riprap or vegetation would be adequate to prevent erosion. A new seawall has been constructed along the lake shore at Heritage Park. In our opinion, the estimated flow velocities and the related small increase in erosion potential would not significantly impact the seawall foundations. Prior to implementation of the Estuary Alternative, the wall design and drainage systems should be reviewed to verify that wall stability will be maintained during cyclical high- to low-water conditions.

SHORELINE NORTH OF 5TH AVENUE DAM (VICINITY OF KFC)

The shoreline area in the vicinity of the Kentucky Fried Chicken (KFC) establishment is located immediately downstream of the Capitol Lake Dam. There is some vegetation at the top of the bank, but little or no vegetation exists beyond 5 feet below the top of the bank. Riprap has been placed on the banks, as well as a stone wall, to protect the bank from damaging erosion.

Currently, tidal flow in this area creates velocities of approximately 1 to 2 ft/s. Results of the hydraulic scour analysis indicate the subject area could frequently be subjected to flow rates on the order of 12 ft/s if Capitol Lake is operated with the dam in place and the tide gates locked in the open position. Results of calculations by Entranco indicate the present riprap would be inadequate to protect the existing stream bank under the estimated flow velocities. Consequently, it would be necessary to protect the existing banks with more substantial riprap or an alternative armoring method.

Under the assumed condition of a 500-foot wide opening at the present dam location, flow velocities were estimated at 1.5 to 2.5 ft/s along the subject stream bank. The current riprap is considered to be adequate to protect the existing stream bank against erosion. If the existing bank is modified during removal of the existing dam and grading of the new opening, erosion protection measures would be required for the re-graded stream bank. These measures might consist of riprap as is currently in place, bio-engineered slopes, or slopes reinforced with geosynthetics.

STEEP SLOPES EAST OF CAPITOL LAKE

Steep and potentially unstable slopes ascend above the east side of Capitol Lake. In particular, unstable slopes have been observed along the southeast edge of the North Basin, below the Capitol Building; and also on the eastern side of the South Basin, along the outside edge of a bend in the Deschutes River south of I-5. Other unstable zones may also exist on steep slopes above the lake. Detailed reconnaissance or geologic mapping of these slopes is beyond the scope of the present study.

Fluctuations in lake level could increase the potential for slope instability along the eastern margin of the lake. This would particularly be a problem if fluctuations in water level extended above the toes of any unstable slope areas. In this event, excess pore water pressures generated during tidal fluctuations could cause slope failures, as discussed above in the *Deschutes Parkway* section.

Prior to implementation of the Estuary Alternative, a detailed geologic reconnaissance should be performed of the steep slopes east of Capitol Lake. Known and potentially unstable areas should be delineated, and the potential impacts of the estuary management strategy on slope stability evaluated. Site-specific geotechnical investigations and analyses may be required to develop mitigation measures for problematic areas identified during the geologic reconnaissance

We appreciate the opportunity to provide geotechnical services on this project. If you have any questions or if we can be of further service, please not hesitate to call.