Technical Memorandum

TO: Queen City Farms, Inc.

FROM: Katherine Saltanovitz, PE and Meghan Veilleux, EIT

DATE: April 4, 2018

RE: **Queen City Lake Wetland Hydroperiod Analysis**
Queen City Farms Phase III Refill
Maple Valley, WA
Project No. 0992002.050.051

Introduction and Purpose

This technical memorandum is written as an addendum to the report titled "Potential Changes to Plant Communities Resulting From Hydrologic Changes To Queen City Lake" prepared by Talasaea Consultants in 2007 (Talasaea 2007) as part of the Queen City Farms (QCF) Refill Technical Information Report (TIR; LAI 2007). The 2007 Talasaea report presented an assessment of the impacts to plant communities that would result from the proposed Phase II Refill project and the changes it would make to the Queen City Lake (Lake) hydrology.

The main proposed hydrologic change of the Phase II Refill project was to increase the stormwater storage function of Queen City Lake by adding a two-stage outlet structure, which restricted Lake outflow and allowed Lake water levels to rise during storm events. The 2007 Talasaea report included a literature review, a plant community analysis, wetland functional analysis, hydrologic modeling of Lake levels, and a qualitative assessment of the hydrologic impacts to the plant community in the Lake. Talasaea's report concluded that the proposed outlet structure, along with anticipated development changes in the offsite drainage basin (due to the closure of Cedar Hills Regional Landfill [CHRL]), would result in a slight reduction of annual and monthly mean water levels in the Lake, and potentially decrease the size of the emergent plant community in the central portion of the Lake. The total size of the wetland was not expected to change significantly, and no mitigation was proposed. King County (County) approved the Phase II TIR and Site Development application in 2008, although the Phase II Refill has not yet started (Talasaea 2007).

This current technical memorandum provides an update to the hydrologic modeling of Queen City Lake levels, taking into account the proposed QCF Phase III Refill project. In addition to placing additional soil refill, the Phase III project includes restoration of the Tributary 316A channel to its original discharge location at the Lake, which is a mapped wetland area (Wetland A). The channel restoration will increase the surface inflow to the Lake, compared to the drainage conditions that were analyzed for the purpose of permitting of the Phase II Refill.

Approach

According to the County Surface Water Design Manual (KCSWDM; King County 2016), Reference 5, Guidelines for Protection from Adverse Impacts of Modified Runoff Quantity Discharged to Wetlands

(Water Level Fluctuation Analysis), "*Protection of wetland plant and animal communities depends on controlling the wetland's hydroperiod, meaning the pattern of fluctuation of water depth and the frequency and duration of exceeding certain levels, including the length and onset of drying in the summer…*" Reference 5 goes on to explain that wetland water levels naturally fluctuate, and "*…could fluctuate more, both higher and lower after development; these greater fluctuations are termed stage excursions. The guidelines set limits on the frequency and duration of excursions, as well as on overall water level fluctuation, after development*."

The wetland hydroperiod analysis presented in this memorandum follows the KCSWDM methodology for determining existing wetland hydroperiod and forecasting future changes using a continuous simulation computer model. As noted in the document, "*These guidelines are replaced by Guide Sheet 3 in WA Ecology's 2014 edition of the SWMMWW, but are retained for the King County Surface Water Design Manual as an appropriate and possibly more stringent alternative for achieving wetland protection goals*." This methodology includes hydroperiod limits that, if exceeded, species richness is likely to decline. These limits are (King County 2016):

- *"Mean annual water level fluctuation (WLF) (and mean monthly WLF for every month of the year) does not exceed 20 cm. Vegetation species richness decrease is likely with: (1) a mean annual (and mean monthly) WLF increase of more than 5 cm (2 inches or 0.16 ft) if predevelopment mean annual (and mean monthly) WLF is greater than 15 cm, or (2) a mean annual (and mean monthly) WLF increase to 20 cm or more if pre-development mean annual (and mean monthly) WLF is 15 cm or less.*
- *The frequency of stage excursions of 15 cm above or below predevelopment stage does not exceed an annual average of six. Note: A short-term lagging or advancement of the continuous record of water levels is acceptable. The 15 cm limit applies to the temporary increase in maximum water surface elevations (hydrograph peaks) after storm events and the maximum decrease in water surface elevations (hydrograph valley bottoms) between events and during the dry season.*
- *The duration of stage excursions of 15 cm above or below predevelopment stage does not exceed 72 hours per excursion.*
- *The total dry period (when pools dry down to the soil surface everywhere in the wetland) does not increase or decrease by more than two weeks in any year.*
- *Alterations to watershed and wetland hydrology that may cause perennial wetlands to become vernal are avoided*."

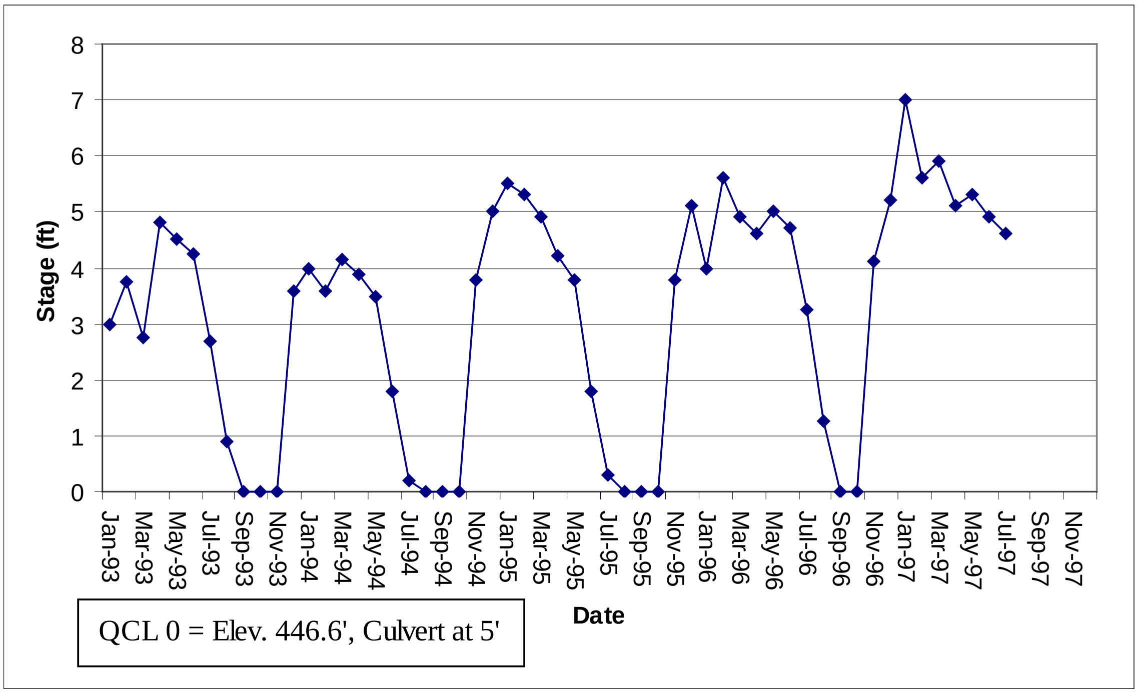
Western Washington Hydrogeologic Model (WWHM) 2012 was used to simulate water levels within Queen City Lake following the development of Phase III Refill (including the restoration of Tributary 316A to discharge to the Lake) to determine periods when the water level will increase more than 15 centimeters (cm) above or below the mean monthly water level (stage excursions). The Phase III Refill project modeled results were compared to pre-2000 (pre-developed) conditions, as well as Phase II Refill permitted final conditions, to determine the magnitude of likely changes. In addition to the 72-

hour duration of stage excursions, two longer durations (168 and 336 hours) were also tallied to provide additional comparison of the three scenarios.

Historical Lake Hydrology

The stage data presented below comes from hydrological data collected by Landau Associates, Inc. (LAI) and others. Figure 1 is reproduced from the Talasaea report and shows semi-continuous water level data for the years 1993 through 1997 (Talasaea 2007). This figure shows instantaneous measurements of Lake level varying from 446.6 ft to 453.6 ft.

Figure 1: Semi-Continuous Water Level Data: 1993 through 1997 (Talasaea 2007).



The Talasaea report includes additional historic data showing that the highest mean water levels typically occur from December to April. Lake levels drop precipitously at the end of the wet season, and the Lake typically goes dry from late August through early October. Water levels rise again rapidly in November and December (Talasaea 2007). Further discussion of historic conditions is presented in Section 4 of the Talasaea report (Talasaea 2007).

Modeled Lake Hydrology- Phase II Refill

The hydrologic analysis of the Phase II refill project was based on modeling of Queen City Lake water levels by Clear Creek Solutions (CCS), using the WWHM Version 3. The CCS analysis was based on modeling of peak stormwater inflows and outflows from the Lake that had been performed by W&H Pacific using the King County Runoff Time Series (KCRTS) hydrologic modeling program (W&H Pacific 2006). CCS used the same land use data, basin areas, and project location as W&H Pacific. The WWHM

Version 3 was used by CCS to model water levels because the KCRTS model includes only eight specific precipitation events, whereas WWHM Version 3 includes a long-term historic precipitation record. The use of the long-term record is most appropriate for a hydroperiod analysis because it includes many different seasonal conditions, not just single storm events (CCS 2006). The WWHM modeling inputs matched the drainage area, land cover, and slope assumptions that were used in KCRTS model.

As mentioned earlier, the 2007 Talasaea report (based on the 2006 CCS modeling) concluded that the proposed future condition of the completed Phase II refill project, including the new Lake outlet structure, would result in a slight reduction of annual and monthly mean water levels in the Lake.

Modeled Lake Hydrology- Phase III Refill

To assess the changes that would take place to Queen City Lake hydrology from the proposed Phase III Refill project, LAI performed updated modeling to assess the Lake water level and wetland hydroperiod. This modeling included drainage basin changes due to Phase III Refill placement and the restoration of Tributary 316A. The modeling followed the same general approach that Talasaea and CCS followed in 2007. KCRTS modeling was first conducted for the Lake, including delineation of the new contributing drainage basins, to determine an appropriate outlet structure design. This modeling is presented in Queen City Lake Basin- KCRTS Modeling technical memorandum prepared by LAI (LAI 2018b) attached to the Phase III Refill Technical Information Report (LAI 2018a). LAI then used WWHM to perform continuous modeling of Lake levels, using the new outlet structure geometry, to model Lake water level over a 60-year simulation.

Results

This section presents the results of the Lake level modeling for the Phase II and Phase III Refill projects. By comparing three different model scenarios (Scenario 1: pre-developed, Scenario 2: Phase II developed, and Scenario 3: Phase III developed), the relative magnitude of the hydrologic changes can be compared. However, the modeled wetland hydrodynamics are not calibrated to actual field measurements. Because stormwater models are generally geared toward preventing flooding and managing high peak flow events, it is likely that any error in a stormwater model such as WWHM would result in overestimating, rather than underestimating, water flow and Lake levels. Therefore, WWHM is an appropriate tool to assess the effect of additional inflows to the Lake. Scenarios 1 and 2, modeled by CCS, used 50 years of precipitation data for the analysis, whereas LAI's Scenario 3 was modeled using 61 years of data (taking advantage of more annual precipitation data since 2006).

Phase II

The 2006 CCS memorandum included several different scenarios to compare the pre- and post-developed Lake conditions, and to assess the effects of adding a two-stage outlet structure to the Lake (CCS 2006). The two scenarios that are relevant to the current analysis are Scenario 1, which represents the drainage area as it existed in the year 2000 (pre-2007 conditions, prior to Phase II Refill activities), and Scenario 4, which represents the drainage area at the completion of the Phase II Refill, with the proposed outlet structure installed in the Lake. Scenario 4 from the 2006 CCS memo is renumbered as Scenario 2 in Table 1.

Scenario 1: Pre-2007 Conditions

The existing conditions scenario represents the condition of the watershed for the year 2000. At that time, the Lake had a 36-inch diameter discharge culvert at elevation 451.62 ft.

Scenario 2: Future Conditions – Phase II

The future watershed conditions represent the future state of the basin after completion of the Phase II Refill development. This includes future stormwater control facilities at CHRLF that are planned to be installed at landfill final closure to mimic undeveloped conditions that existed in 1979. It is for this reason that the Lake basin has been modeled as mostly forested till, with 8.5 acres of impervious area set aside to represent the area of the Lake. In the Future Conditions scenario, the 36-inch diameter culvert has been replaced with a discharge configuration at the same elevation that allows for a maximum discharge of 2 cubic feet per second (cfs).

Phase III

LAI performed updated modeling for Phase III final conditions (Scenario 3) that includes changes to the drainage basin land cover type and total area entering Queen City Lake, the addition of the Tributary 316A drainage basin, a larger two-stage outlet structure installed in Queen City Lake. Model output for the updated Scenario 3 is presented in Attachment 1.

Scenario 3: Future Conditions – Phase III

This scenario represents the development conditions at the completion of Phase III, which includes the restoration of Tributary 316A. The Lake basin has been conservatively modeled as moderately sloped (5 to 15 percent) and the land types listed above in Table 1. The determination of land types can be found in Section 6.10 (LAI 2018a). A figure with the delineation of the designated land types can be found in the same section on Figure 2. The basin is routed in WWHM to a Stage-Storage-Discharge (SSD) table, which is manually input to represent the discharge out of the Lake via infiltration and the outlet structure. The SSD table can be found in the WWHM Report provided as Attachment 1. In the SSD table, the stage, area, and volume inputs were previously calculated by W&H Pacific (W&H Pacific 2006). The infiltration rates were previously estimated by LAI using the relationship of the Lake level volume, as presented in a technical memorandum dated January 29, 2007 (LAI 2007). The discharge rates out of the outlet structure were calculated based on orifice and weir questions, as presented in Section 6.10.

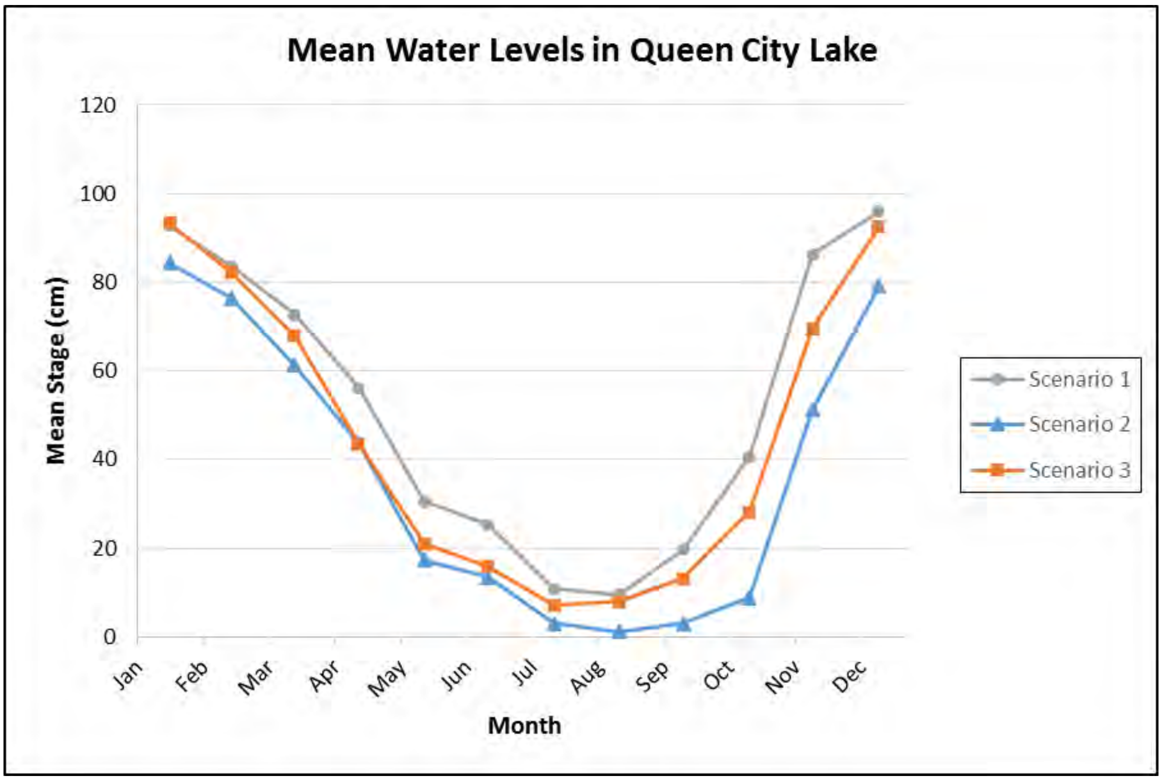
Table 1 –Summary of Modeled Drainage Areas

KCRTS Land Type	Outwash Forest (acre)	Till Forest (acre)	Till Pasture (acre)	Till Grass (acre)	Impervious (acre)	Total (acre)
Scenario 1/CSS	0	132.41	0	132.77	74.82	340
Scenario 2/CSS (a)	0	331.50	0	0	8.50	340
Scenario 3/LAI	145.00	324.00	21.00	0	65.50	555.5
WWHM Land Type/LAI	A/B, Forest	C, Forest	C, Pasture	none	Roads	---

(a) Scenario 4 from the 2006 CCS memo is renumbered as Scenario 2 in Table 1.

Figure 2 shows the mean monthly water levels in the Lake for pre-2007 conditions (Scenario 1), predicted future conditions with Phase II development (Scenario 2), and predicted future conditions with Phase III development (Scenario 3).

Figure 2: Modeled Mean Water Levels in Queen City Lake



The results of the hydroperiod analysis for Scenarios 1, 2, and 3 are presented in Tables 2 through 7.

Each scenario has two corresponding tables that display similar information in a different format. The first table displays the hydroperiod results for total hours exceeded, and the second table displays the results in terms of average annual hours of exceedance.

The column definitions for Tables 2, 4, and 6 are as follows:

1. Month: total of the results for the month over the 50 or 61 years of analysis.
2. Mean Stage (cm): the mean stage of water in the Lake over the 50 or 61 years of analysis.
3. Excursions (total hours): the total number of hours that the stage in the Lake was 15 cm above or below the mean stage of the Lake for that month.
4. Long Excursions (number of events): total number of events that the stage in the Lake remained 15 cm above or below the mean stage of the Lake for a period of longer than 72 hours. (This compares to the column labeled “total hours” in the CCS memorandum.)
5. Excursions 168 (number of events): total number of events that the stage in the Lake remained 15 cm above or below the mean stage of the Lake for a period of longer than 168 hours.
6. Excursions 336 (number of events): total number of events that the stage in the Lake remained 15 cm above or below the mean stage of the Lake for a period of longer than 336 hours.

The column definitions for Tables 3, 5, and 7 are as follows:

1. Month: averaging the results for this month over the 50 or 61 years of analysis.
2. Mean Stage (cm): the mean stage of water in the Lake over the 50 or 61 years of analysis.
3. Excursions (average hours): the average annual hours that the stage in the Lake was 15 cm above or below the mean stage of the Lake for that month.
4. Long Excursions (average number of events): the average annual number of events that the stage in the Lake remained 15 cm above or below the mean stage of the Lake for a period of longer than 72 hours.
5. 168 Hour Excursions (average number of events): the average annual number of events that the stage in the Lake remained 15 cm above or below the mean stage of the Lake for a period of longer than 168 hours.
6. 336 Hour Excursions (average number of events): the average annual number of events that the stage in the Lake remained 15 cm above or below the mean stage of the Lake for a period of longer than 336 hours.

Table 2 – Scenario 1, Total Excursions

Month	Mean Stage (cm)	Excursions (total hours)	Long Excursions (# of events) (a)	168 Hour Excursions (# of events) (a)	336 Hour Excursions (# of events) (a)
Jan	92.7	30,930	106	71	31
Feb	83.6	27,477	106	65	19
Mar	72.7	29,608	123	70	23
Apr	56.2	29,603	122	69	19
May	30.5	32,091	142	64	14
June	25.3	32,530	128	64	22
July	10.9	5,830	23	6	0
Aug	9.6	5,392	19	4	2
Sept	19.6	32,972	124	68	22
Oct	40.6	31,157	128	53	19
Nov	86.2	29,708	115	66	24
Dec	96.0	30,564	109	71	30

(a) Long Excursions, Excursions 168, and Excursions 336 were previously defined in the 2007 Talasaea analysis as the total hours the Lake water levels remained 15 cm above or below for the corresponding period, which is a typographic error. Column headings in the above table have been corrected to reflect that these are the number of events, not total hours.

Table 3 – Scenario 1: Pre-2007 Conditions, Average Annual Excursions

Month	Mean Stage (cm)	Excursions (avg. hours)	Long Excursions (avg. # of events) (a)	168 Hour Excursions (avg. # of events) (a)	336 Hour Excursions (avg. # of events) (a)
Jan	92.7	619	2.1	1.4	0.6
Feb	83.6	550	2.1	1.3	0.4
Mar	72.7	592	2.5	1.4	0.5
Apr	56.2	592	2.4	1.4	0.4
May	30.5	642	2.8	1.3	0.3
June	25.3	651	2.6	1.3	0.4
July	10.9	117	0.5	0.1	0.0
Aug	9.6	108	0.4	0.1	0.0
Sept	19.6	659	2.5	1.4	0.4
Oct	40.6	623	2.6	1.1	0.4
Nov	86.2	594	2.3	1.3	0.5
Dec	96.0	611	2.2	1.4	0.6

(a) Long Excursions, Excursions 168, and Excursions 336 were previously defined in the 2007 Talasaea analysis as the total hours the Lake water levels remained 15 cm above or below for the corresponding period, which is a typographic error. Column headings in the above table have been corrected to reflect that these are the number of events, not total hours.

Table 4 – Scenario 2: Future Conditions (Phase II), Total Excursions

Month	Mean Stage (cm)	Excursions (total hours)	Long Excursions (# of events)	168 Hour Excursions (# of events)	336 Hour Excursions (# of events)
Jan	84.4	31,457	96	67	33
Feb	76.5	28,506	102	66	20
Mar	61.2	29,622	115	67	29
Apr	43.7	30,614	105	69	29
May	17.3	26,295	96	41	10
June	13.4	5,158	21	12	3
July	3.1	1,948	7	1	0
Aug	1.2	1,168	1	0	0
Sept	3.1	2,270	6	1	0
Oct	8.8	4,921	18	8	0
Nov	51.2	31,156	93	63	26
Dec	79.2	31,052	93	62	33

Table 5 – Scenario 2: Future Conditions (Phase II), Average Annual Excursions

Month	Mean Stage (cm)	Excursions (avg. hours)	Long Excursions (avg. # of events)	168 Hour Excursions (avg. # of events)	336 Hour Excursions (avg. # of events)
Jan	84.4	629	1.9	1.3	0.7
Feb	76.5	570	2.0	1.3	0.4
Mar	61.2	592	2.3	1.3	0.6
Apr	43.7	612	2.1	1.4	0.6
May	17.3	526	1.9	0.8	0.2
June	13.4	103	0.4	0.2	0.1
July	3.1	39	0.1	0.0	0.0
Aug	1.2	23	0.0	0.0	0.0
Sept	3.1	45	0.1	0.0	0.0
Oct	8.8	98	0.4	0.2	0.0
Nov	51.2	623	1.9	1.3	0.5
Dec	79.2	621	1.9	1.2	0.7

Table 6 – Scenario 3: Future Conditions (Phase III), Total Excursions

Month	Mean Stage (cm)	Excursions (total hours)	Long Excursions (# of events)	168 Hour Excursions (# of events)	336 Hour Excursions (# of events)
Jan	93.3	39,037	124	97	38
Feb	82.2	35,367	120	80	27
Mar	68.0	37,079	141	87	33
Apr	43.5	35,434	149	78	24
May	21.0	19,356	81	32	9
June	15.9	24,068	98	49	12
July	7.1	4,115	10	0	0
Aug	7.9	5,204	22	3	0
Sept	13.2	6,954	27	3	0
Oct	28.3	28,542	116	42	16
Nov	69.5	36,349	143	76	32
Dec	92.5	39,699	127	87	35

Table 7 – Scenario 3: Future Conditions (Phase III), Average Annual Excursions

Month	Mean Stage (cm)	Excursions (avg. hours)	Long Excursions (avg. # of events)	168 Hour Excursions (avg. # of events)	336 Hour Excursions (avg. # of events)
Jan	93.3	640	2.0	1.6	0.6
Feb	82.2	580	2.0	1.3	0.4
Mar	68.0	608	2.3	1.4	0.5
Apr	43.5	581	2.4	1.3	0.4
May	21.0	317	1.3	0.5	0.1
June	15.9	395	1.6	0.8	0.2
July	7.1	67	0.2	0.0	0.0
Aug	7.9	85	0.4	0.0	0.0
Sept	13.2	114	0.4	0.0	0.0
Oct	28.3	468	1.9	0.7	0.3
Nov	69.5	596	2.3	1.2	0.5
Dec	92.5	651	2.1	1.4	0.6

Table 8 summarizes the pre-2007 and developed frequency of excursions (± 15cm) that last for 72 hrs, 168 hrs, and 336 hrs. The data in these tables are summarized in Table 8 below.

Table 8: Frequency of Excursions (±15cm)

	Scenario 1 (Pre-2007 Conditions)			Scenario 2 (Phase II Development)			Scenario 3 (Phase III Development)		
Hours	**72**	**168**	**336**	**72**	**168**	**336**	**72**	**168**	**336**
Jan	2.1	1.4	0.6	1.9	1.3	0.7	2.0	1.6	0.6
Feb	2.1	1.3	0.4	2.0	1.3	0.4	2.0	1.3	0.4
Mar	2.5	1.4	0.5	2.3	1.3	0.6	2.3	1.4	0.5
Apr	2.4	1.4	0.4	2.1	1.4	0.6	2.4	1.3	0.4
May	2.8	1.3	0.3	1.9	0.8	0.2	1.3	0.5	0.1
Jun	2.6	1.3	0.4	0.4	0.2	0.1	1.6	0.8	0.2
Jul	0.5	0.1	0.0	0.1	0.0	0.0	0.2	0.0	0.0
Aug	0.4	0.1	0.0	0.0	0.0	0.0	0.4	0.0	0.0
Sep	2.5	1.4	0.4	0.1	0.0	0.0	0.4	0.0	0.0
Oct	2.6	1.1	0.4	0.4	0.2	0.0	1.9	0.7	0.3
Nov	2.3	1.3	0.5	1.9	1.3	0.5	2.3	1.2	0.5
Dec	2.2	1.4	0.6	1.9	1.2	0.7	2.1	1.4	0.6

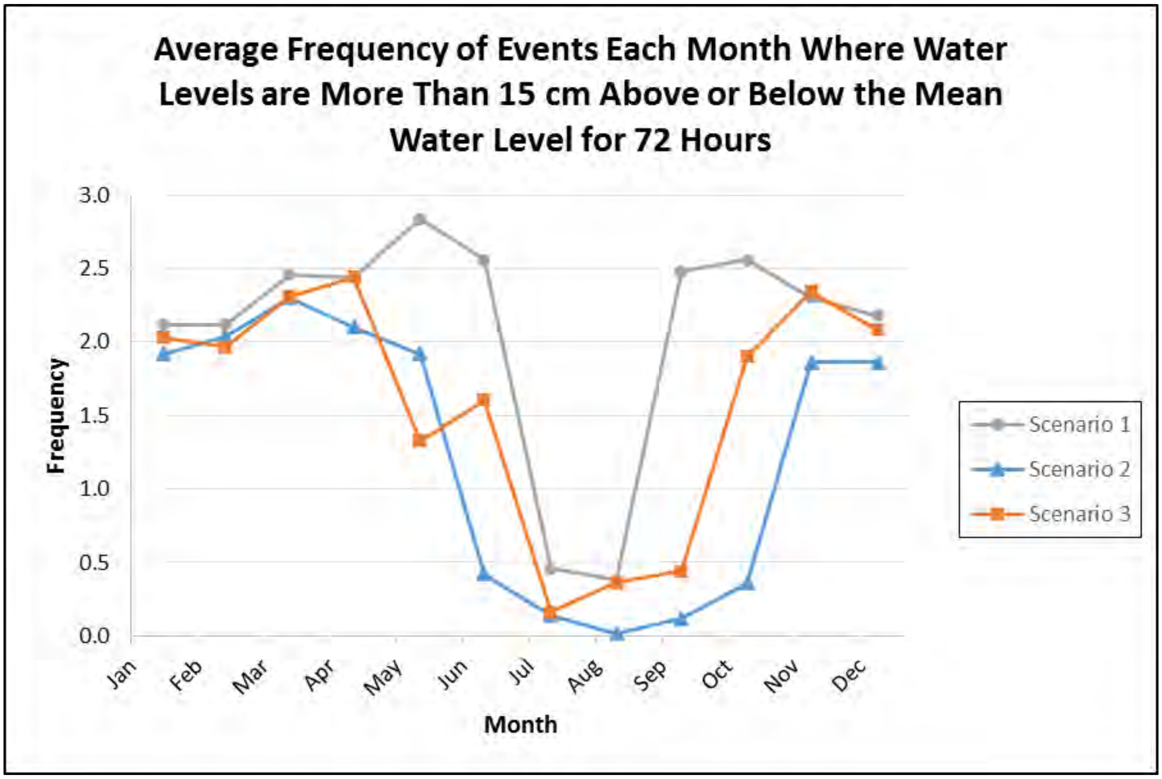
Conclusions

The conclusions of the updated hydroperiod modeling are summarized below. In general, the results indicate that water levels in Queen City Lake will be higher than the modeled Lake levels for the Phase II Refill, but lower than the modeled historical conditions.

Excursions Greater Than 72 Hours

As reported by Talasaea, there were no appreciable differences in the frequency of excursions lasting longer than 72 hours (3 days) between Scenario 1 (pre-2007) and Scenario 2 until June (Figure 3). At this point, the frequency of excursions was lower for Scenario 2 compared to Scenario 1. When comparing the future conditions for Scenarios 2 and 3, the frequencies generally increased in Scenario 3, but remained lower than the frequency of excursions for Scenario 1. This is likely attributable to the additional 200 acres added to the Lake subbasin as part of the Phase II Refill project.

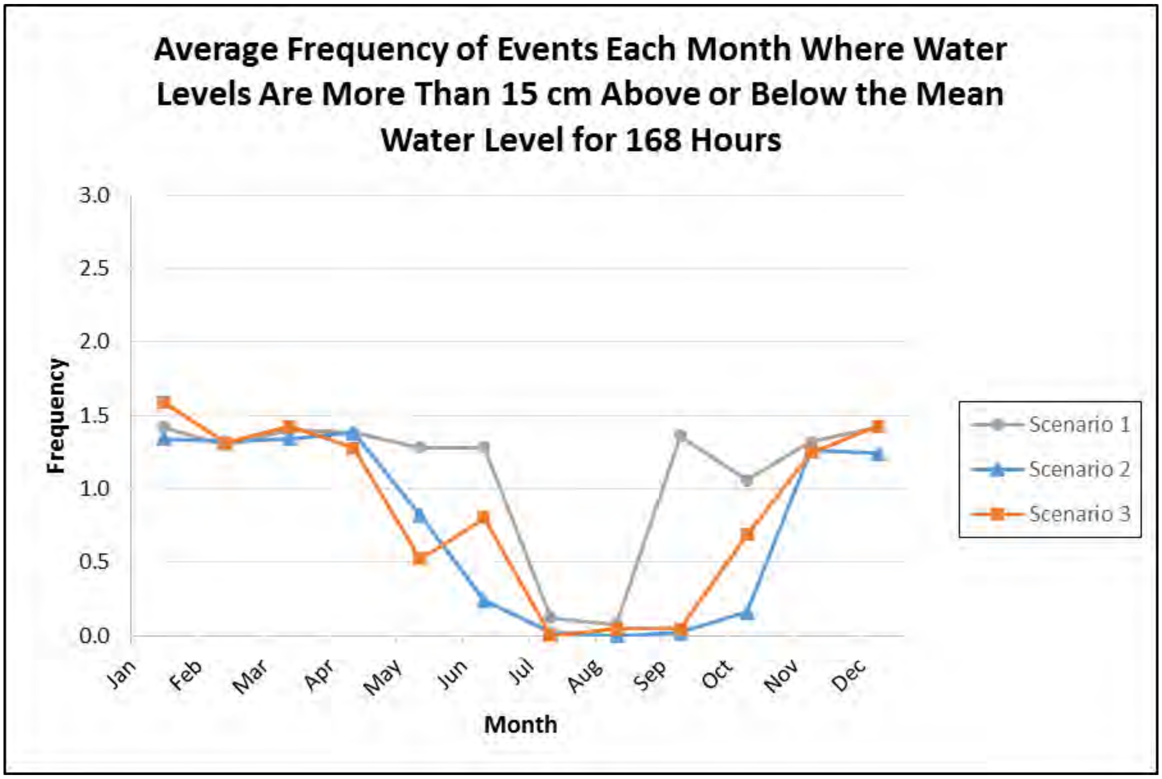
Figure 3: Average Frequency of 72-Hour Excursions



Excursions Greater Than 168 Hours

As reported by Talasaea, between the Scenario 1 and the Scenario 2 conditions, there was no appreciable difference in the frequency of excursions lasting longer than 168 hours (7 days) until April (Figure 4). At this point, the frequency of excursions was higher for Scenario 1 than for Scenario 2. This pattern lasted until November when the models predicted that the frequency of events under Scenario 2 would approximately equal the frequency of events of Scenario 1. When comparing Scenarios 2 and 3, the frequencies generally stayed the same or increased, but remained lower than the frequency of excursions modeled for Scenario 1, except for the month of January. The deviation in the frequency of events between the scenarios shown for the month of January should not adversely impact the plant community of Wetland A because the vegetation is primarily dormant during January. The typical growing season for wetland plants in the Pacific Northwest usually starts in March/April and ends in October/November.

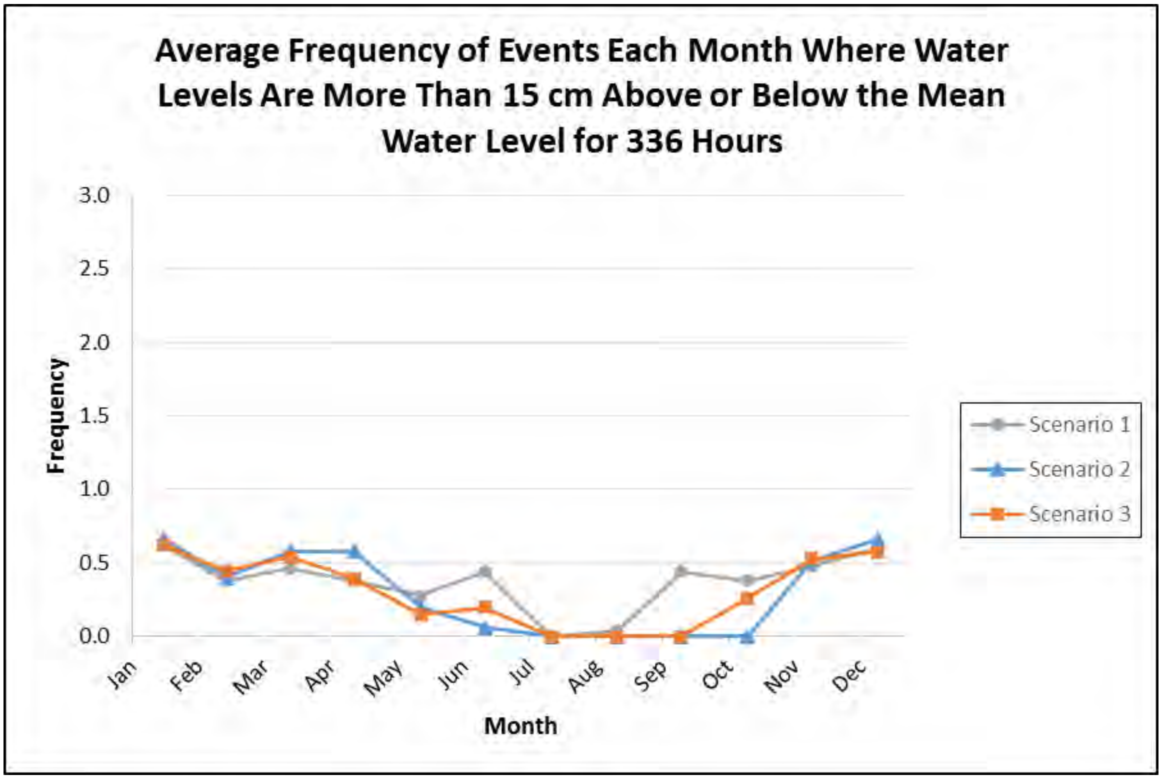
Figure 4: Average Frequency of 168-Hour Excursions.



Excursions Greater Than 336 Hours

As reported by Talasaea, the frequency of excursions lasting longer than 336 hours (14 days) was higher for Scenario 2 compared to Scenario 1 until June (Figure 5). From June to October, Scenario 1 conditions were higher than the predicted Scenario 2 conditions. Frequency of excursions for Scenario 2 was predicted to be greater than Scenario 1 from November on. When comparing Scenarios 2 and 3, the frequencies generally stayed the same or increased. Similarly, frequency of excursions for Scenario 3 is predicted to be greater for January thru April and November on. It should be noted that 336-hour events occur infrequently in all three scenarios. Because of this, and because the Lake levels during the summer months will be relatively low, corresponding with plant communities that are relatively wet-adapted, the impact of any changes to the 336-hour excursion frequency among the three scenarios is likely insignificant.

Figure 5: Average Frequency of 336-Hour Excursions.



It is important to note that these models show the frequency of excursions, but not the frequency of peak stages. The WWHM lake level modeling shows that the both Scenarios 2 and 3 will have a lower amount of variability in water level changes compared to Scenario 1, possibly indicating a more stable supply of hydrology to Wetland A.

LANDAU ASSOCIATES, INC.

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MDV/KMS/ALB/BAS/jrc
[P:\992\002\WIP\T\2017 TIR UPDATE\WETLAND HYDROPERIOD ANALYSIS (SEC 6.1)\QUEEN CITY LAKE HYDROPERIOD ANALYSIS TM_033018_FOR JRC.DOCX]

Attachment 1: Western Washington Hydrologic Model Results

References

CCS. 2006. Queen City Wetland Flux Analysis. Clear Creek Solutions, Inc. November 22.

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LAI. 2018a. Phase III Refill Technical Information Report, Queen City Farms, Maple Valley, Washington. Landau Associates, Inc. April.

LAI. 2018b. Technical Memorandum: Queen City Lake Basin - King County Runoff Time Series Modeling, Queen City Farms Phase III Refill, Maple Valley, Washington. Landau Associates, Inc. April 4.

Talasaea. 2007. Potential Changes to Plant Communities Resulting from Hydrologic Changes to Queen City Lake, Queen City Farms, King County, Washington. Talasaea Consultants, Inc. February 1.

W&H Pacific. 2006. Letter: Queen City Farms - Storage Routing and Water Level Analysis, Phase 3 Outlet Structure Sizing. From Michael Gomez, W&H Pacific, to Brian Butler, Senior Associate Geologist, Landau Associates, Inc. December 13.

Attachment 1

Western Washington Hydrologic Model Results

WWHM2012

PROJECT REPORT

General Model Information

Project Name:	QCL Hydroperiod Analysis
Site Name:	Queen City Lake
Site Address:	
City:	Maple Valley, WA
Report Date:	2/21/2018
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	0.00 (adjusted)
Version Date:	2016/02/25
Version:	4.2.12

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year
Low Flow Threshold for POC2:	50 Percent of the 2 Year
High Flow Threshold for POC2:	50 Year

Landuse Basin Data
Predeveloped Land Use

Queen City Farms

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	145
C, Forest, Mod	324
C, Pasture, Mod	21
Pervious Total	490
Impervious Land Use	acre
ROADS MOD	65.5
Impervious Total	65.5
Basin Total	555.5

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Queen City Farms

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	145
C, Forest, Mod	324
C, Pasture, Mod	21
Pervious Total	490
Impervious Land Use	acre
ROADS MOD	65.5
Impervious Total	65.5
Basin Total	555.5

Element Flows To:

Surface	Interflow	Groundwater
SSD Table 1	SSD Table 1	

Routing Elements

Predeveloped Routing

Mitigated Routing

SSD Table 1

Depth: 10.9 ft.
Element Flows To:
Outlet 1 Outlet 2

SSD Table Hydraulic Table

Stage (feet)	Area (ac.)	Volume (ac-ft.)	Manual	Infilt (cfs)	NotUsed	NotUsed	NotUsed
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.400	0.010	0.000	0.000	0.000	0.000	0.000	0.000
1.400	1.870	0.660	0.000	0.800	0.000	0.000	0.000
2.400	4.090	3.570	0.000	1.200	0.000	0.000	0.000
3.400	5.580	8.390	0.000	1.600	0.000	0.000	0.000
4.400	7.110	14.72	0.000	1.800	0.000	0.000	0.000
5.400	8.530	22.53	2.430	2.000	0.000	0.000	0.000
6.400	9.940	31.76	4.550	3.000	0.000	0.000	0.000
7.400	11.73	42.58	5.960	7.000	0.000	0.000	0.000
7.970	12.52	49.66	6.690	7.000	0.000	0.000	0.000
8.400	13.12	55.00	17.76	15.70	0.000	0.000	0.000
9.400	14.54	68.82	70.57	15.70	0.000	0.000	0.000
10.90	15.19	91.11	191.2	15.70	0.000	0.000	0.000

Analysis Results

POC 1

POC #1 was not reported because POC must exist in both scenarios and both scenarios must have been run.

POC 2
POC #2 was not reported because POC must exist in both scenarios and both scenarios must have been run.

Model Default Modifications

Total of 0 changes have been made.

PERLND Changes

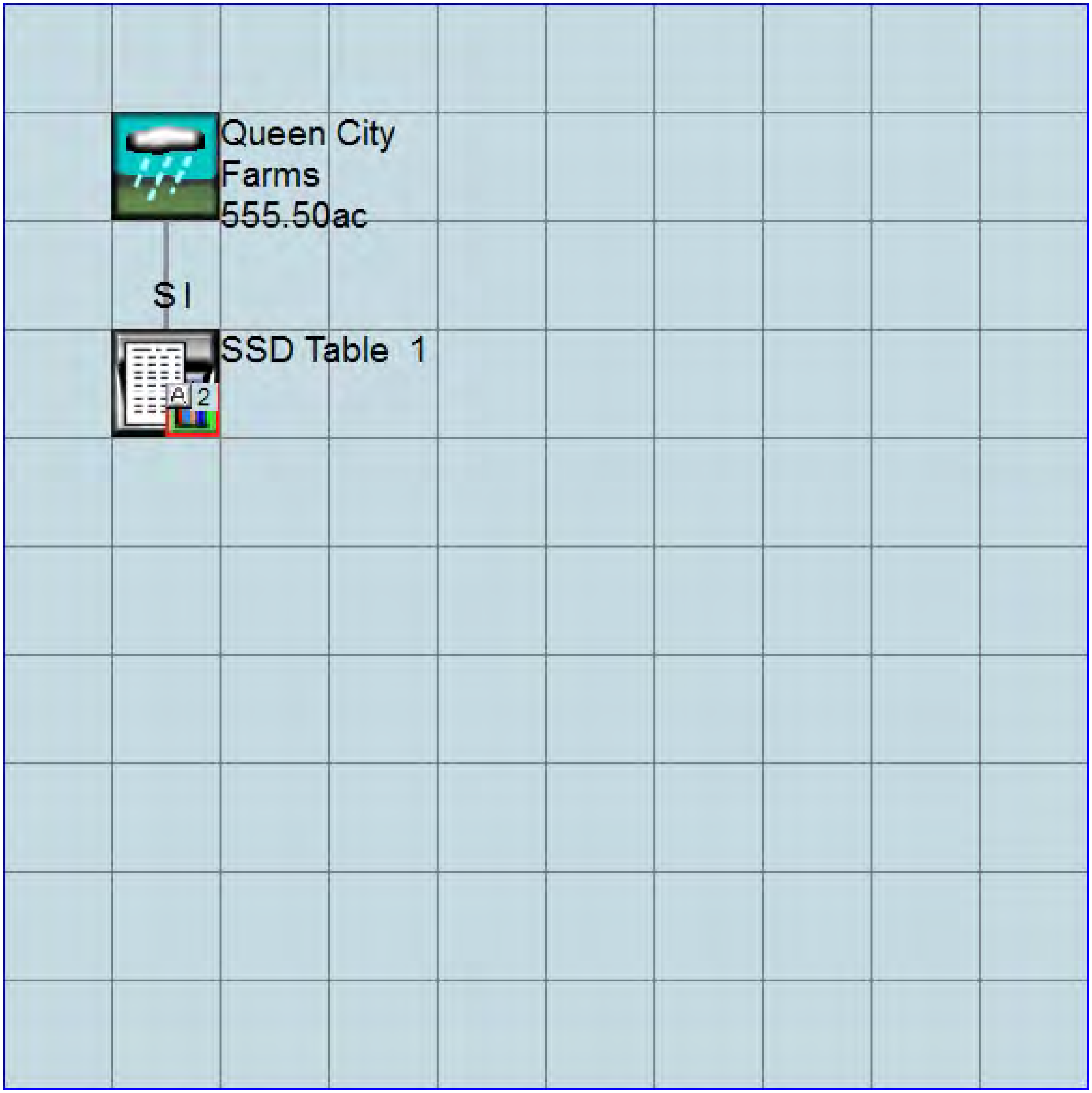
No PERLND changes have been made.

IMPLND Changes

No IMPLND changes have been made.

Appendix
Predeveloped Schematic

Mitigated Schematic



Predeveloped UCI File

Mitigated UCI File

Predeveloped HSPF Message File

Mitigated HSPF Message File

Disclaimer

Legal Notice

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Potential Changes to Plant Communities Resulting from Hydrologic Changes to Queen City Lake

Queen City Farms
King County, Washington

Prepared for:
Queen City Farms, Inc.
S9010 East Marginal Way South
Seattle, Washington, 98108

Prepared by:

Talasaea Consultants, Inc.
15020 Bear Creek Road NE
Woodinville, WA 98077

01 February 2007

EXECUTIVE SUMMARY

PROJECT NAME: Queen City Lake

SITE LOCATION: The project is located in unincorporated King County southeast of Renton Washington. It is in portions of Sections 28, 29, and 33, in Township 23N, Range 6E, WM.

CLIENT: Queen City Farms, Inc.

PROJECT STAFF: Bill Shiels, Principal; David R. Teesdale, Wetland Ecologist; Charles Repath, Wetland and Plant Ecologist.

FIELD SURVEY: A field survey was conducted on October 4 and 5, 2006, by Talasaea Consultants, Inc.

PROPOSED PROJECT: The Queen City Farms Refill Project consists of filling approximately 70 acres of former sand and gravel mine area to a maximum depth of about 120 ft. The Refill Project will be conducted in three phases. The Refill Project will not affect the Queen City Lake hydrology until the final phase, Phase 3. Phase 3 activities include eliminating the current 36-inch overflow pipe from Queen City Lake to the Main Gravel Pit Lake. This 36-inch overflow pipe will be replaced by a smaller overflow pipe at the same invert elevation (corresponding to a Queen City Lake depth of 4 feet) that will discharge to a new infiltration facility (East Retention Pond). Phase 3 is not expected to commence for at least 5 years after Phase 1 commences.

Beginning in the summer of 2007, Cedar Hills Regional Landfill (CHRL), will begin stormwater improvement projects that will reduce runoff in much of the Queen City Lake Sub-basin area. These improvements will be completed by about 2015 when the southern portion of the landfill is closed. The net effect of improvements on CHRL over this time period is to reduce runoff in the basin and reduce water levels in Queen City Lake. While Phase 3 of the refill project may tend to increase water levels in the lake slightly, the net effect of CHRL improvements and Refill Project implementation is to reduce Queen City Lake water levels. Modeled runoff in Queen City Lake Basin (Appendix 4 and TIR Section 6.10) shows Queen City Lake water will dry up almost a month earlier and refill about a month later. Modeling indicates that shorter duration hydroperiods will be appreciably less frequent under future conditions (CHRL improvements and Phase 3 Refill Project implementation) compared to current conditions. Longer duration hydroperiods will be similar in frequency.

Implementation of Phase 3 is not expected to have a significant impact on average Queen City Lake water levels or hydroperiod duration. This is because the discharge capacity of the replacement overflow pipe (i.e., 2 cfs), although greater than the average monthly winter time runoff rate from Queen City Lake Sub-basin, is substantially less than the discharge rate of the existing 36 inch overflow pipe. After large or extreme storm events, the maximum water level in Queen City Lake will likely exceed current maximums because the smaller overflow pipe capacity will be appreciably less than stormwater discharge into the lake. During these relatively infrequent and short-lived events, the future retention capacity of the lake and groundwater infiltration into Aquifer 1 will increase over current conditions.

This report focuses on the current plant communities of Queen City Lake, how these communities have developed over time to changes in hydrologic regimes of the lake, and how the wetland plant communities might respond to potential alterations of the hydrologic regime of the lake. The predicted net effect of changes in Queen City Lake hydrology will be to decrease the size of the emergent plant community in the central portion of the lake. Scrub and shrub plant communities will, over time, replace much of the emergent vegetation. While the relative percentage and distribution of existing wetland plant communities will change, the total size of the wetland is not expected to change significantly. Based on surface water runoff modeling, these wetland changes will occur as the King County Solid Waste Division gradually improves runoff conditions on CHRL over the next eight years. These changes are not attributed to implementation of the Refill Project. Consequently, mitigation for the Refill Project is not proposed.

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1.0 INTRODUCTION

The Queen City Farms (QCF) property is an approximately 393-acre area located east of SR-169 and north of Cedar Grove Road SE in King County, Washington (**Figure 1**). The Public Land Survey System location is in portions of Sections 29, 32, and 33 in Township 23N, Range 6E, WM. The King County tax parcel number is 2823069009. Queen City Lake (the lake) is a natural kettle depression, located along the north boundary of the Queen City Farm property (**Figure 2**).

The hydrology of the lake has been greatly altered over the past 70 years from its historic natural condition due to land use changes in its watershed. The biggest change in this hydrology has come from the development of the Cedar Hills Regional Landfill in the upper part of the lake's watershed (north of the QCF property boundary). Landfill development has resulted in increasing lake surface water inflow since the 1960s. About 79% of the Queen City Lake basin in located within the CHRL.

Outflow of the lake is also currently managed. In 1991, a 36" pipe was installed approximately four feet above the lakebed elevation. The discharge from this pipe eventually flows to an artificial lake (gravel pit lake) created by gravel mining operations on Queen City Farms.

1.1 Proposed Land Use Actions

Two planned projects will affect the current hydrology of Queen City Lake. These two projects are described fully in the Technical Information Report by Landau Associates, Inc.

The first project is construction of Cedar Hills Regional Landfill stormwater improvements (commencing summer 2007), and the closure of the landfill by the year 2015. This action will effectively create a very large vegetated landfill cap area requiring extensive stormwater management. Most of this stormwater will be discharged to the lake; the remainder will be routed to stormwater retention facilities that infiltrate without discharging to the lake. These changes will decrease the amount of surface water discharge to Queen City Lake. While changes at the landfill are independent of the proposed Refill Project, they are analyzed in this report because they directly impact the hydrology of Queen City Lake.

The second project is the reclamation of the gravel mine area of Queen City Farms and the filling of the main gravel pit lake. The Refill Project will be conducted in three phases. The Refill Project will not affect the Queen City Lake hydrology until the final phase, Phase 3. Phase 3 activities include eliminating the current 36-inch overflow pipe from Queen City Lake to the Main Gravel Pit Lake. This 36-inch overflow pipe will be replaced by a smaller overflow pipe at the same invert elevation (corresponding to a Queen City Lake depth of approximately 4 feet) that will discharge to a new infiltration facility (East Retention Pond). The inflow orifice for this new pipe will be sized to discharge about 2 cfs. Phase 3 is not expected to commence for at least 5 years after Phase 1 commences.

Currently, overflow from Queen City Lake during storm events is directed to the main gravel pit lake via a 36" discharge pipe. The filling of the main gravel pit lake represents a loss of stormwater detention capacity for the QCF area. To compensate for the loss in detention capacity, new stormwater retention and detention facilities will be constructed outside of the gravel pit area and the outlet structure from Queen City Lake will be modified to utilize more of the lake's natural storage and groundwater infiltration capacity during peak storm events.

Mean water levels in the lake are expected to be slightly lower in the future because of decreased surface water inputs resulting from the landfill's stormwater improvements and eventual closure. Reduction in the size of the outflow pipe will moderate the reduction in surface water inputs to the lake by allowing more water to be retained in the lake during peak storm events. However, it cannot completely compensate for decreased surface water inputs to the lake in terms of mean water levels. Considering the impact of CHRL actions and the affect of reducing the outflow pipe, the future mean water levels in the lake are still expected to be slightly lower than they are currently.

Talasaea Consultants, Inc. was requested to study the effects on the plant community of Queen City Lake with regard to these proposed changes in hydrology resulting from the closure of the landfill and the alteration of the outflow characteristics of Queen City Lake. This report specifically addresses the potential affects to the distribution of plant communities in Queen City Lake that may result from changes in hydrology.
The current plant community of the lake is the result of the site hydrology as it exists today. Different plant species have differing abilities to thrive and survive in various hydrologic regimes. Generally, upland plant species are adapted to survive in drier conditions, while wetland-adapted plants are able to survive varying periods of soil saturation or inundation.

Changes to the hydrologic regime of Queen City Lake will likely alter the composition of the plant community within the wetland. As a result of expected decreases in the mean water levels in the lake, some areas of the lake will experience a shorter period of inundation during the year. In these areas, wetland plants that are adapted to survive in saturated soil conditions will have a competitive advantage over those adapted to long periods of inundation.

In addition to changes in the mean water levels, the Queen City Lake system is expected to experience fewer extreme fluctuations in water levels throughout the year creating a more stable wetland system. Greater stability within a wetland system typically favors native plant species over invasive plant species.

1.2 Purpose of Report

This report focuses on the current plant communities of Queen City Lake, how these communities have developed over time to changes in hydrologic regimes of the lake, and how these plant communities might respond to potential alterations of the hydrologic regime of the lake.

The existing plant communities in and around the lake were identified, categorized, and mapped. This analysis identifies plant communities by hydrologic tolerances and elevation above lakebed levels. The information generated here serves as a baseline for extrapolating the distribution of plant communities resulting from the predicted changes in lake hydrology (frequency, depth, and duration of inundation). In addition, the patterns of plant communities as they existed over time were also mapped. This provides an indication of how plant communities have changed over time in response to different land use practices in the Queen City Lake basin.

Using baseline information as suggested above, this report attempts to describe the future patterns of plant community distribution based on possible hydrologic changes to the lake. These changes in hydrology are tied to studies prepared by Landau Associates, Inc, and Clear Creek Solutions, Inc. Understanding how plant community distribution has changed in the past in response to land use changes over time serves as a guide to predicting plant community distribution changes based on possible future hydrologic conditions.

2.0 GENERAL PROPERTY DESCRIPTION

2.1 Queen City Farms

As described earlier, QCF is a parcel approximately 393-acres in size. It is located north of Cedar Grove Road SE and east of SR-169 in King County, Washington. The PLSS location is in parts of Section 29, 32, and 33 of Township 23N, Range 6E, WM. The property to the north is currently used as the Cedar Hills Regional Landfill. Property to the west is either undeveloped or used as residential housing. Property to the east is mostly undeveloped. The Cedar River is located to the southwest. The property is currently used by the Stoneway Sand and Gravel Company and by Cedar Grove Composting. Most of the property is currently an unused gravel mine.

The Cedar Hills Regional Landfill is an approximately 920 acre parcel located north of the QCF property. This landfill was opened in the 1960s. It is to be closed and capped by 2015.

A relatively small area (5.2 ac) near the northeast corner of QCF was utilized as an industrial waste site. Remediation on this site was completed in the 1990s and is subject to on-going monitoring. The proposed gravel mine reclamation does not involve any direct alteration to this portion of the site and aims to avoid conditions that may affect it.

Cedar Grove Composting is located near the northwest corner of QCF. This facility occupies approximately 51.1 acres of land. Operations include receiving vegetation and other organic material for composting, and regional distribution of composted material.

2.2 Queen City Lake

Queen City Lake is a natural kettle lake that formed during the retreat of the last period of glaciation (**Figure 3**). It is believed that a large chunk of ice from the glacier remained, forming the kettle. The ice subsequently melted and the newly formed lake slowly filled in with organic material to roughly its current shape. The fine organic material in the lakebed has poor infiltration characteristics; however, the sides of the lake are composed primarily of recessional outwash material, which has a high infiltration capacity. The lake hydrology is currently supported, in large part, by stormwater discharge from the landfill, surface runoff, and groundwater seepage. Primary discharge from the lake is through the 36-inch discharge pipe and via groundwater infiltration. Queen City Lake is the primary source of groundwater input to the aquifer directly beneath the lake known as Aquifer 1.

Queen City Lake is classified as a King County Category 1 wetland. It contains elements of the following Cowardin vegetation classifications: palustrine emergent, palustrine scrub-shrub, and palustrine forested.

The lake contains three distinct vegetation types, as listed above (**Figure 4**). The lakebed is vegetated by both emergent and scrub-shrub vegetation. This corresponds to an area that is typically inundated for the longest period of time during the growing season. Species present in the emergent zone consisted primarily of spotted lady's thumb (*Polygonum persicaria*), with a small amount of skunk cabbage (*Lysichitum americanum*) and yellow pondweed (*Nuphar luteum*) present in discrete areas. Scrub-shrub vegetation near the emergent vegetation consisted mainly of Pacific willow (*Salix lasiandra*) and bittersweet nightshade (*Solanum dulcamara*). On the margins of the lakebed, the scrub-shrub vegetation included Sitka willow (*Salix sitchensis*), heartleaf willow (*S. rigida*), red-osier dogwood (*Cornus sericea*), and salmonberry *Rubus spectabilis*. Trees begin to be established above the lake bed. These consisted mainly of black cottonwood (*Populus balsamifera spp. trichocarpa*), quaking aspen (*Populus tremuloides*), western red cedar (*Thuja plicata*), and red alder (*Alnus rubra*). Upland tree and shrub species include big-leaf maple (*Acer macrophyllum*), western hemlock (*Tsuga heterophylla*), Douglas fir (*Pseudotsuga menziesii*), Pacific madrone (*Arbutus menziesii*), Indian plum (*Oemleria cerasiformis*), sword fern (*Polystichum munitum*), and bracken fern (*Pteridium aquilinum*).

The topography of the lake basin slopes down moderately from the property boundary with the landfill to the lake (**Figure 5**). A relatively low, flat area exists along the northwest edge of the lake. Topography slopes up sharply along the south boundary of the kettle. The topography south of the lake was likely fairly flat and level, as evidenced by the airfield that existed south of the lake prior to gravel mining. Currently, the topography slopes sharply downward towards Cedar Grove Road SE (corresponding to the location of the Stoneway Sand and Gravel Mine and gravel pit lake).

Queen City Lake currently receives most of its hydrology as stormwater discharge from CHRL with a smaller portion from shallow groundwater seepage. Historical hydrologic support came from entrapment of groundwater and surface runoff.

Water entering the lake typically leaves as seepage through the edges of the lake, which are composed mainly of gravelly sandy loam. In 1991, a 36" pipe was installed at four feet above lakebed elevation to control the level of the lake and prevent the lake from spilling over its bank and eroding the walls of the gravel mine to the south during peak storm events. The lakebed is composed of organic muck, which typically impedes the percolation of water. This muck is replaced by sand and gravel above the lakebed, allowing water to "leak" out. The lake supports an Aquifer identified by Landau Associates as Aquifer 1.

3.0 METHODOLOGY

The purpose of this section is to address the steps required to understand existing site conditions and how plant communities are likely to change due to potential changes in the hydrology of the lake.

3.1 Literature Review
We performed a review of available literature for information pertaining to the hydrologic requirements of Pacific Northwest plant species and how these species might respond to changes in hydrology. Sources included online resources and a literature review at the University of Washington.

Other sources of information included:
- Clear Creek Solutions, Inc. Hydrology Models;
- Washington Department of Fish and Wildlife (WDFW) Priority Habitats and Species Database;
- Washington Department of Natural Resources (WDNR) Natural Heritage Program Database;
- Previous site studies by Landau Associates, Inc.
- Previous site studies by Smayda Environmental Associates, Inc.
- Aerial photographs from 1936, 1960, 1968, 1977, 1980, 1990, 2001, and 2004.

3.2 Plant Community Analysis
The analysis of the distribution of plant communities within Queen City Lake involves an examination of existing distribution and the analysis of past distributions based on historic aerial photo-interpretation. These are described below.

3.2.1 Historical Aerial Photo-Interpretation
Historical stereo-pair aerial photographs were examined to determine land use activities around Queen City Lake and to determine how plant community distribution has changed over time. Of importance was the ability to relate changes in plant community distribution with changes in land use.

3.2.2 Existing Plant Community Analysis
The distribution of plant communities within the lake was determined by placement of seven vegetation sampling transects (**Figure 6**). Six transects were oriented north to south across the short axis of the lake. One transect was oriented east to west along the long axis. The length of each transect was walked and the plant distribution patterns were observed.

Where a clear change in plant species composition (based, in part, on the wetland indicator status of the plants in the association), a point was established and marked onsite with a wood stake. The plants in the vicinity of the point were identified. A total of 39 sample points were established, surveyed, and characterized. Vegetation was further categorized by wetland indicator status. This allowed for a rough approximation of wetland conditions around the lake.

Two series of panorama photographs were taken at a point roughly in the middle of the Queen City Lake wetland. Additional photographs were taken at random points along the vegetation transects in order to create a visual log of the transects.

The location of each sample point was mapped initially by GPS and later by professional survey. The surveyors also measured the elevation of each sampling point. Point location, elevation, and community composition data were recorded into a GIS layer. This allowed the stratification of plant communities by location and elevation.

3.3 Hydrology and Stormwater Modeling

Hydrology and stormwater modeling was performed by Landau Associates, Inc. and by Clear Creek Solutions, Inc. Information regarding these models is contained in the Technical Information Report by Landau Associates, and in the Queen City Lake Wetland Flux Report by Clear Creek Solutions.

3.4 Plant Community Response to Hydrologic Change

3.4.1 Plant Community Distribution Tied to Current Hydrologic Conditions

The current distribution of plant communities in and around Queen City Lake was analyzed with respect to the current hydrologic conditions for the lake. This information was based on current and past depth, duration, timing, and frequency of inundation. From this, a generalized "range of tolerance" to the inundation parameters listed above was created for each plant species found in the lake.

3.4.2 Plant Community Distribution in Response to Hydrologic Changes

Hydrologic models indicating the likely future condition of the lake hydrology will be used to predict the affect of changing hydrologic conditions on the existing plant communities. The "range of tolerance" to the four inundation parameters (developed in **Section 3.4.1** above) will be used to predict how plant species and plant communities will respond to the predicted hydrologic conditions.

3.5 Wetland Functional Analysis

Given the proposed changes within the contributing basin to Queen City Lake, there is a potential for changes to occur in the functions of the wetland system. A wetland functional analysis was made to compare existing conditions to future predicted conditions within the Queen City Lake wetland. The method chosen was the Washington Department of Ecology Wetland Functional Analysis, or WAFAM. WAFAM analyses were performed on existing site conditions and future conditions with a 2 cfs outlet pipe.

4.0 RESULTS

4.1 Findings from Literature Review

A literature review was performed to provide a scientific basis for understanding the effect of attempting to restore the hydrological conditions in the wetlands to pre-1979 conditions. The complete literature review is attached in **Appendix D.** Literature describing the response of native wetland vegetation to changes in hydroperiod was also reviewed. Specifically, information was looked for regarding the vegetative response to changing the seasonality, depth, frequency and duration of inundation and saturation.

Only a small fraction of the literature reviewed was specific to the Pacific Northwest. These studies are important to this report because plant species response to hydrologic conditions varies between different regions (Reed, 1988). When considering individual species, studies from the Pacific Northwest took precedence. Most of what was found was general information. The following are a few observations relevant to Queen City Lake.

Plant response to the depth, duration and seasonality of inundation and saturation, varied by community and species. Aquatic bed plant communities were adapted to the longest duration and deepest inundation, followed by emergent, scrub-shrub and then forested communities. Generally, plant species richness is negatively impacted by water depths over two feet, frequent inundation events, and inundation events lasting over six days.

Black cottonwoods were found in areas that were occasionally inundated during the dormant season, but which had little or no surface water during the growing season. Some species such as reed canarygrass (*Phalaris arundinacea*) and slough sedge (*Carex obnupta*) were found in areas with high water level fluctuations. Excessive water level fluctuation can result in emergent plant communities being dominated by non-native invasive species. Some shrub species such as hardhack (*Spiraea douglasii*), Sitka willow (*Salix sitchensis*) and Scouler's willow (*Salix scouleriana*) were found in a wide range of conditions from dry throughout the year to inundated during most of the growing season.

4.2 Findings from Historical Aerial Photo Interpretation

Aerial photographs from the years 1936, 1960, 1968, 1977, 1980, 1990, 2001 and 2004 were analyzed to determine disturbance history and also historic and present plant communities (**Appendix A**). Dr. Frank Westerlund, Professor of Urban Design & Planning at the University of Washington, analyzed the aerial photos. Dr. Westerlund has an extensive background in remote sensing applications, including analyzing aerial photos for presence/absence of wetland characteristics. **Figures A1** through **A8** in **Appendix A** illustrate the results of the photo interpretation.

Historical aerial photos show that substantial development has occurred in the Cedar Hills sub-basin. This development has altered forest cover, increased impervious surface area, and altered hydrology and wildlife habitat. The property has been logged, used as a pig farm, an airstrip, as a gravel mine, and as a landfill. The Cedar Hills Landfill began operations in the 1960s and it has grown significantly in size since its inception. A gravel mine is located south of QCL. The gravel mine was in operation from the late 1960s to 1995.

4.3 Hydrology and Stormwater Modeling

The stage data presented below comes from hydrological data collected primarily by Landau Associates, Inc. **Figure 7** shows semi-continuous water level data the years 1993 through 1997. Individual hydrographs for the years 1988 through 1993 can be found in **Appendix C**. This figure was produced from only a few years of data, three years before and six years after the installation of the 36-inch outflow pipe. However, it still may be useful. **Figure 8** shows the mean monthly water levels for QCL for existing and predicted future conditions. **Figure 9** shows two curves: the first shows the mean monthly mean stage from the years 1988 to 1990; the second shows the mean monthly mean stage from the years 1992 to 1997. **Figures 7, 8,** and **9,** and the individual hydrographs in **Appendix C,** show that the highest mean water levels typically occurred from December to April. Lake levels drop precipitously at the end of the wet season, and the lake typically goes dry from late August through early October. Water levels rise again rapidly in November and December.

Figure 7: Semi-Continuous Water Level Data: 1993 through 1997

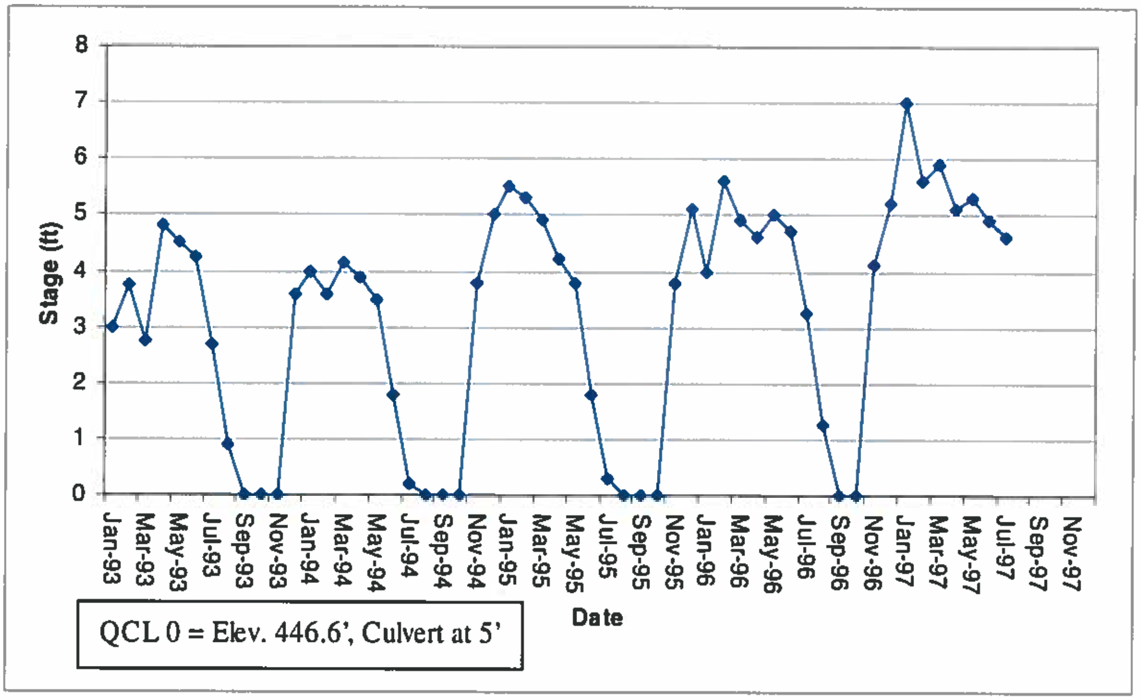


Figure 8: Mean Water Levels in Queen City Lake (Landau Associates, Inc.)

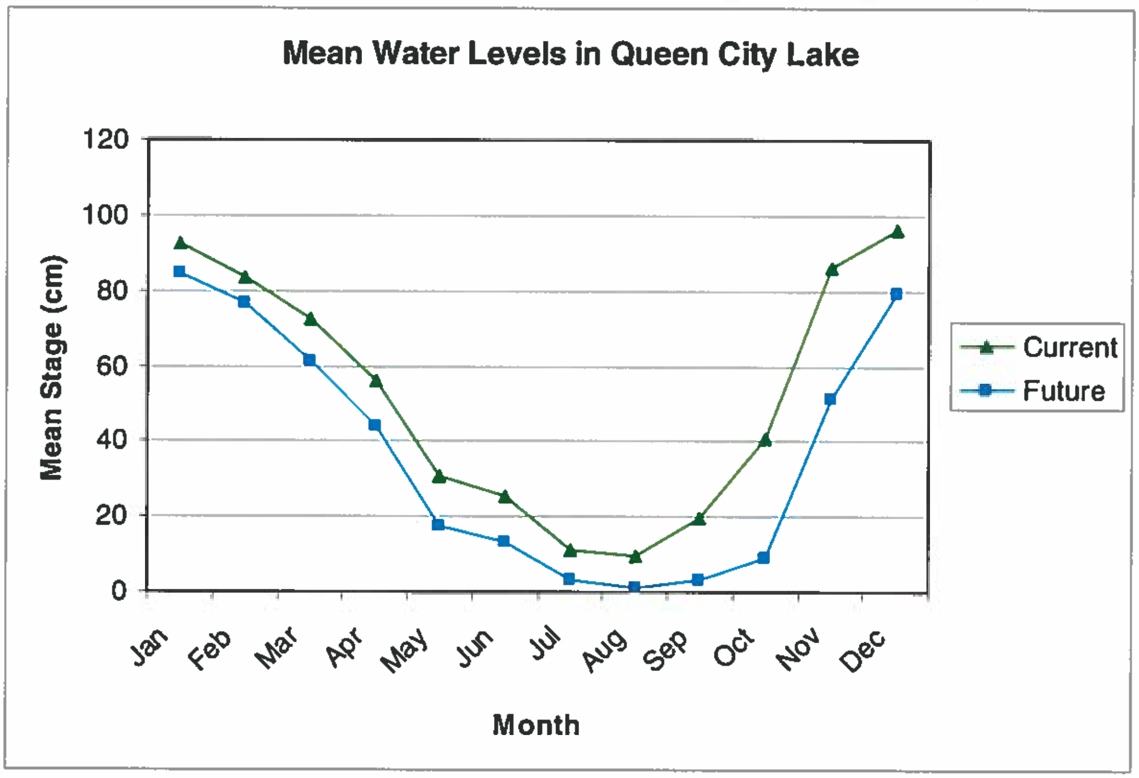


Figure 9: Queen City Lake Mean Water Surface Elevation; Pre-1991 vs. Post-1991

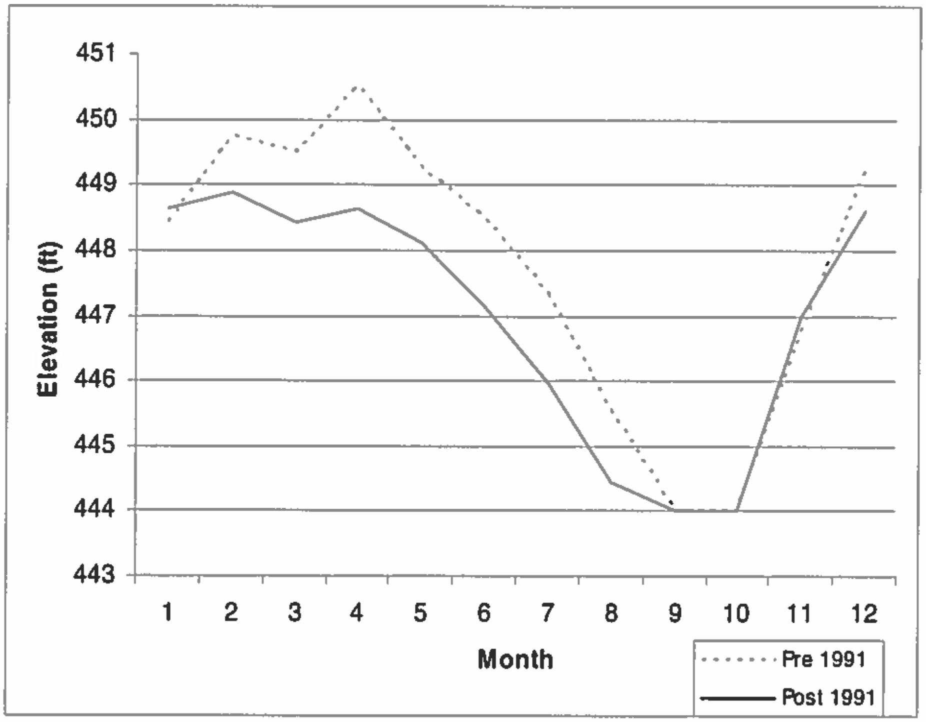


Figure 9 suggests two additional phenomena. The first phenomenon is the effect of the installation of the 36" outflow pipe in 1991. Prior to 1991, major storm events resulted in peak stages (water level in QCL) that ranged between 453 feet to over 455 feet. The mean monthly stages during the rainy season were generally over 451 feet. After the pipe was installed, peak stages were lowered and the duration of inundation was generally shorter. The drawdown of the lake occurred sooner in the growing season, in part because the mean monthly stage during the rainy season was generally less than 451 feet.

The second phenomenon is apparent when examining the slope of the curves for the hydrographs prior to and after 1991. The declining slope of the curves reflects the rate that water seeps out of QCL through the soil. The declining slopes of the hydrograph curves are remarkably similar regardless of the installation of the outfall pipe. At the very least, this shows that the outfall pipe only affects stage elevation above 451 feet, but has no effect on rates of infiltration. It may be possible to calculate other parameters based on the data behind these curves (i.e., infiltration rates, peak storage limits, groundwater recharge rates); however, these calculations are beyond the scope of this report.

Clear Creek Solutions, Inc. used the Western Washington Hydrology Model Version 3 (WWHM3) to model the Queen City Lake hydrology (see Queen City Lake Wetland Flux Analysis Report, 2006, in **Appendix E**). Two base conditions were modeled: 1) existing hydrology conditions (based on year 2000), and 2) future hydrology conditions (based on year 2015 - the year the landfill closure is to be completed).

The Clear Creek Solutions (CSS) report contains tables indicating the existing and expected frequency of excursions (± 15cm) that last for 72 hrs, 168 hrs, and 336 hrs.

The data in the CSS report are reproduced in **Table 1** below. These parameters were specified by King County.

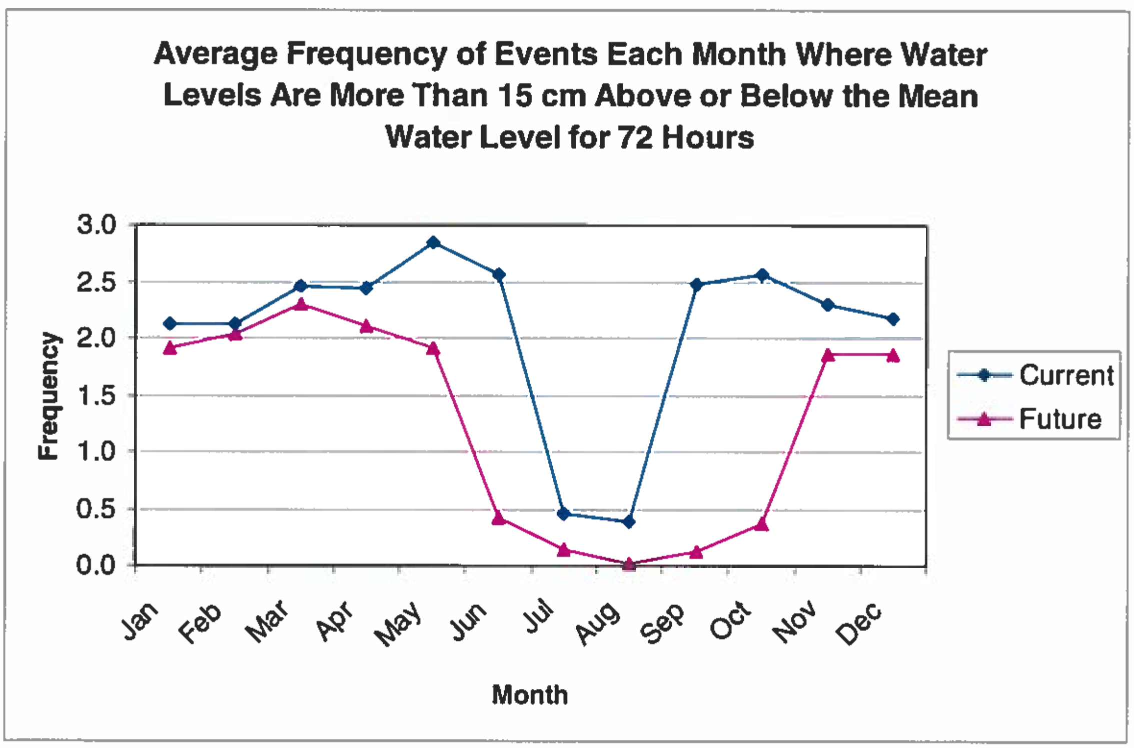
Table 1: Existing and Predicted Frequency of Excursions (±15cm)

	Existing Conditions			Future w. 2 cfs Outfall Pipe		
Hours	**72**	**168**	**336**	**72**	**168**	**336**
Jan	2.1	1.4	0.6	1.9	1.3	0.7
Feb	2.1	1.3	0.4	2.0	1.3	0.4
Mar	2.5	1.4	0.5	2.3	1.3	0.6
Apr	2.4	1.4	0.4	2.1	1.4	0.6
May	2.8	1.3	0.3	1.9	0.8	0.2
Jun	2.6	1.3	0.4	0.4	0.2	0.1
Jul	0.5	0.1	0.0	0.1	0.0	0.0
Aug	0.4	0.1	0.0	0.0	0.0	0.0
Sep	2.5	1.4	0.4	0.1	0.0	0.0
Oct	2.6	1.1	0.4	0.4	0.2	0.0
Nov	2.3	1.3	0.5	1.9	1.3	0.5
Dec	2.2	1.4	0.6	1.9	1.2	0.7

4.3.1 Excursions Greater than 72 Hours (three days)

There was no appreciable difference in the frequency of excursions lasting longer than 72 hours (3 days) between the two conditions until June (**Figure 10**). At this point, the frequency of excursions was lower for the future conditions compared to existing conditions.

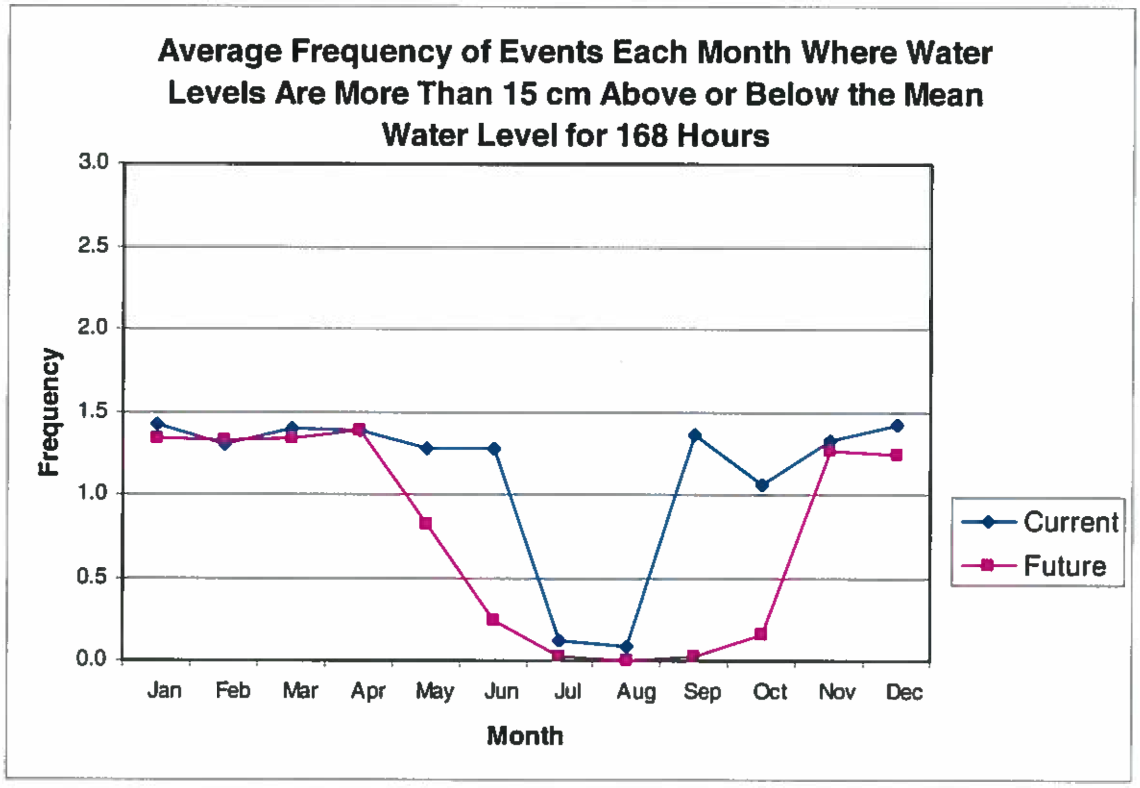
Figure 10: Average Frequency of 72-Hour (three-day) Excursions (Landau Associates, Inc.)



4.3.2 Excursions Greater than 168 Hours (seven days)

There was no appreciable difference in the frequency of excursions lasting longer than 168 hours (7 days) until April (**Figure 11**). At this point, the frequency of excursions was higher for existing conditions than for future conditions. This pattern lasted until November when the models predicted that the frequency of events under future conditions would equal the frequency of existing conditions.

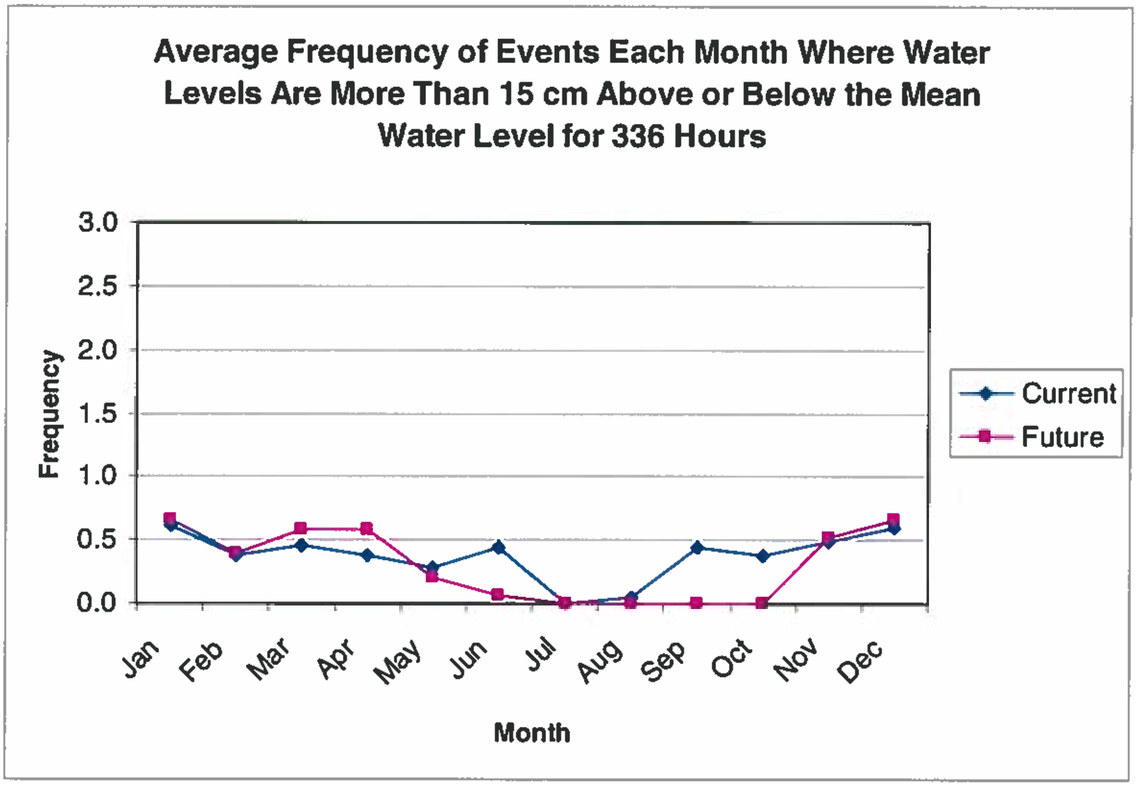
Figure 11: Average Frequency of 168-Hour (seven-day) Excursions (Landau Associates, Inc.)



4.3.3 Excursions greater than 336 hours (14 days)

The frequency of excursions lasting longer than 336 hours (14 days) was higher for the future condition compared to existing conditions until June (**Figure 12**). From June to October, existing conditions were higher than the predicted future conditions. Frequency of excursions for the future conditions was predicted to be greater than existing conditions from November on. It should be noted that 336-hour events occur infrequently in both the existing and future conditions. Because of this, and because the lake levels during the summer months will be relatively low, corresponding with plant communities that are relatively wet-adapted, the differences between the two conditions are likely insignificant.

Figure 12: Average Frequency of 336-Hour (14-day) Excursions (Landau Associates, Inc.)



It is important to note that these models show the frequency of excursions, but not the frequency of peak stages. The numbers presented in the wetland flux report show that the future conditions will have a lower amount of variability in water level changes compared to existing conditions, possibly indicating a more stable supply of hydrology.,

Hydrology graphs from Landau Associates, Inc. do show that there is likely to be a reduction in the amount of water coming into the lake system after the complete closure of the landfill. The landfill is treated as an impervious surface. Runoff from the landfill is currently planned to be shunted both to the lake and to a different drainage. Groundwater will be supplied mainly by that portion of the buffer north of Queen City Lake between the north edge of the lake and the landfill. The predicted result is a net reduction in hydrologic input.

4.4 Queen City Lake Plant Community Distribution – Current Conditions

The patterns of plant communities in and around the lake reflect the existing hydrologic conditions. We mapped the locations of the plant communities based on the results of our sampling transects. These results are shown on **Figure 4**.

The different plant communities generally appear to form concentric rings around the lake based on the duration, depth, seasonality, and frequency of inundation, soil saturation, and elevation.

Three wetland vegetation classes (based on Cowardin, *et al.*, 1979) at Queen City Lake are Palustrine emergent, Palustrine scrub-shrub, and Palustrine Forested. We also identified five types of plant communities: an emergent community, dominated almost entirely by *Polygonum persicaria*, two scrub-shrub communities (Pacific willow/bittersweet nightshade, and Pacific willow/black twinberry), and two forested communities (Black cottonwood/quaking aspen/red alder, and Douglas fir/big-leaf maple). The distribution of these communities is discussed below.

4.4.1 Distribution of Plant Communities

The lowest portion of the lake (i.e., elevations of 446.6 feet (0 ft stage) to approximately 448.6 feet (2 ft stage)) is inundated approximately 10 months of the year. Much of this area is underlain by organic soils. An emergent plant community dominated by smartweed (*Polygonum persicaria*) is present in the lowest area of the lake. Pacific and heartleaf willow (*Salix lasiandra* and *S. rigida*) and also black nightshade (*Solanum dulcamara*) are present in scattered thickets around the lowest portions of the lake.

Between elevations of 448.6 feet and 451.6 feet (2 ft to 5 ft stage), the emergent community is replaced with a scrub-shrub plant community dominated by Pacific willow and black twinberry (*Lonicera involucrata*).

On the north and west sides of QCL between 451.6 feet and 453.6 feet (5 ft to 7 ft stage), the scrub-shrub community is replaced with a deciduous forest community dominated by black cottonwood (*Populus trichocarpa*), red alder (*Alnus rubra*), and quaking aspen (*Populus tremuloides*). Approximately 20% of the zone above 451.6 feet remains a scrub-shrub plant community, now dominated by salmonberry (*Rubus spectabilis*), Scouler's willow (*Salix scouleriana*) and hardhack (*Spiraea douglasii*).

On the north and west sides of QCL above approximately 543.6 feet (7 ft stage), an upland coniferous forest dominated by Douglas fir (*Pseudotsuga menziesii*) and snowberry (*Symphoricarpos albus*) replaces the deciduous forest. Forest trees are second or third growth and average approximately 20 inches dbh. Himalayan blackberry (*Rubus discolor*) and Scot's broom (*Cytisus scoparius*) dominate some areas of upland, especially along the berm and the road.

The vegetation around the east and south sides of QCL are more difficult to differentiate. The plant communities here are generally affected, first, by the steep slopes present in these areas and, second, by the land use practices in these areas (the Boeing site to the east and the mine to the south.

4.4.2 Species Hydrologic Tolerance Ranges – Existing Conditions

We tabulated the plant species by elevation and seasonal inundation. The results of this analysis are contained on **Figure 13** below. **Table 2** contains the translations for the plant name codes used in **Figure 13**. The figure shows that knotweed can tolerate inundation for 10 months of the year. Other species, such as black twinberry (*Lonicera involucrata*), Pacific willow, black nightshade, and Scouler's willow can tolerate a wide range of inundation. These species can survive in conditions where they are not subjected to inundation, to where they are inundated for up to eight months of the year. Nine species, including Himalayan, cut-leaf and trailing blackberry (*Rubus discolor, R. laciniatus,* and *R. ursinus*), as well as red-osier dogwood (*Cornus stolonifera*), quaking aspen, salmonberry, sword fern (*Polystichum munitum*), black cottonwood, and red alder, appear to be able to withstand occasional inundation. These species are established above the elevation of the outlet pipe (451.6 ft elev.). Inundation at higher elevations occurs mostly during the winter, with occasional inundation events occurring into May. Species established above an elevation of 543.6 feet, such as Douglas fir and salal (*Gaultheria shallon*), are only rarely or never inundated.

4.5 Queen City Lake Plant Community Distribution – Predicted Changes

The closure of the landfill and the proposed modifications to the outlet structure of Queen City Lake will modify the hydrologic conditions in the lake. The intent is to use the storage and infiltration capacity available in the lake to compensate for the loss of storage capacity in the main gravel pit lake during the mine reclamation work. However, even allowing extra water to collect in the lake during peak storm events will not entirely compensate for a decrease in the surface water input to the lake from the landfill. Overall, the result will be a decrease in the mean water levels in the lake. Changes to mean water levels will have a greater impact on the plant community than infrequent peak storm events.
Plant community structure will be affected by both changes in the mean water levels and the frequency of peak storm events.

The overall size of the wetland is not expected to change because the lake basin will still be allowed to fill up during storm events in the spring. However, the depth and duration of inundation in the lower elevation portions of the wetland are expected to change. The potential response of the plant community as a result of these changes is discussed below.

Figure 13: Queen City Lake Plant Species Distributions by Elevation and Seasonal Inundation (See **Table 2** for species name code translation).

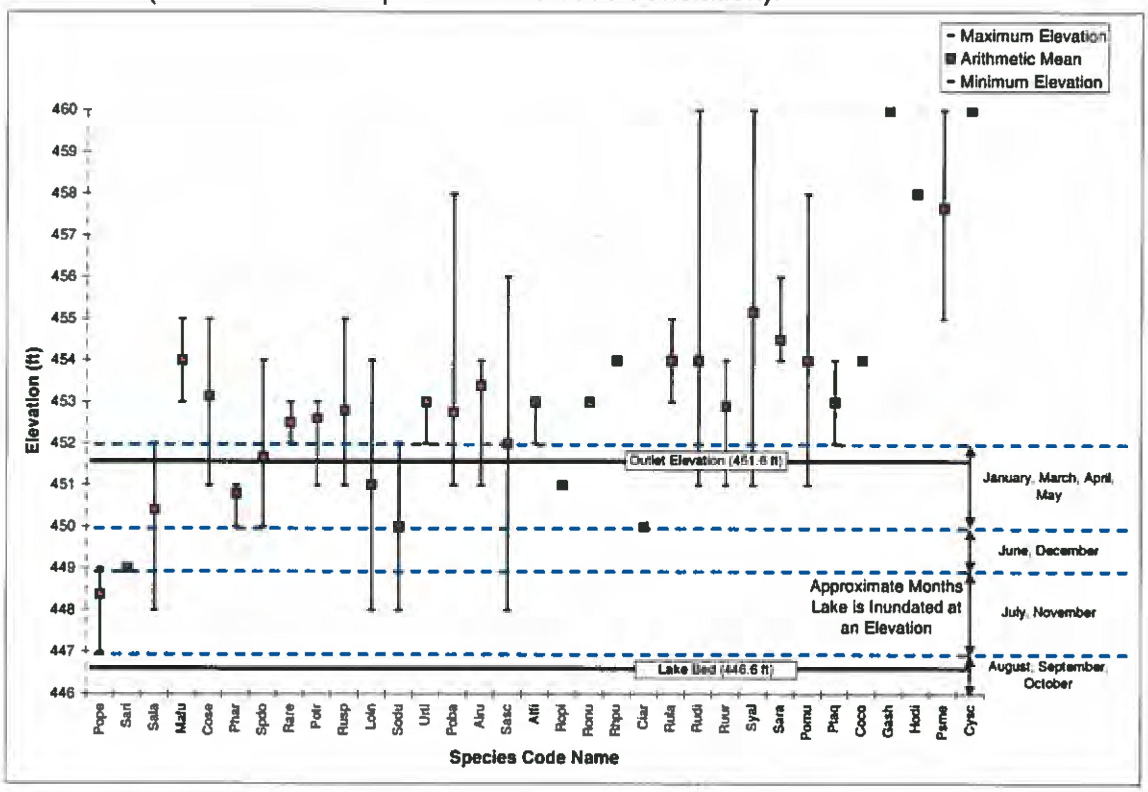


Table 2: Name Codes for Species Found at Queen City Lake used in **Figure 13.**

Scientific Name	Shorthand Code
Alnus rubra	Alru
Athyrium filix-femina	Atfi
Cirsium arvense	Ciar
Cornus sericea	Cose
Corylus cornuta	Coco
Cytisus scoparius	Cysc
Gaultheria shallon	Gash
Holodiscus discolor	Hodi
Lonicera involucrata	Loin
Malus fusca	Mafu
Phalaris arundinacea	Phar
Polygonum persicaria (sp?).	Pope
Polystichum munitum	Pomu
Populus balsamifera var. trichocarpa	Poba
Populus tremuloides	Potr
Pseudotsuga menziesii	Psme
Pteridium aquilinum	Ptaq
Ranunculus repens	Rare
Rhamnus purshiana	Rhpu
Rosa nutkana	Ronu
Rosa pisocarpa	Ropi
Rubus discolor	Rudi
Rubus lasciniatus	Rula
Rubus spectabilis	Rusp
Rubus ursinus	Ruur
Salix lasiandra	Sala
Salix rigida	Sari
Salix scouleriana	Sasc
Sambucus racemosa	Sara
Solanum dulcamara	Sodu
Spiraea douglasii	Spdo
Symphoricarpos albus	Syal
Urtica dioica	Urdi

4.5.1 Plant Response

The potential affects of a decreased duration of inundation are discussed below and illustrated on **Figure 14**. In general, the area of the lake that is likely to be inundated at any time during the year will decrease.

The overall area of emergent vegetation will likely be significantly reduced. This calculation and the associated map (**Figure 14**) were based on comparison of current and future mean water levels and LIDAR topographic contours of the site. Existing emergents at higher elevations will eventually be replaced by shrubs and small trees which are better adapted to a hydrologic regime with less inundation. The current emergent community of *Polygonum persicaria* will likely remain in the topographical low points on the lake bed. Plant communities and species that are more adapted to drier conditions (such as those currently found at higher elevations around the lake) will become established at lower elevations. These include an expanded amount of Pacific willow and black twinberry throughout the lakebed, and an increase in the amount of Scouler's willow throughout the scrub-shrub zone. The cottonwood-aspen community may extend lower towards the lakebed elevation over time.

Invasive weedy species, such as reed canarygrass, are often better adapted to extreme fluctuations in water levels than many native plants. Reducing the water level fluctuations may decrease the competitive advantage of some invasive species.

The reduction in depth and duration of ponding and soil saturation, and the associated reduction in area of the emergent plant community, may reduce plant community structure. These changes may result in a reduction in available habitat for aquatic invertebrates, amphibians, and birds. These impacts may be lessened by the expected reduction in the extremes of water level fluctuations resulting from moderated stormwater runoff from the Cedar Hills Landfill.

4.6 Wetland Functional Analysis

A wetland functional analysis for the lake was performed for both the existing hydrologic conditions and the predicted hydrologic conditions. We used Ecology's *Methods for Assessing Wetland Functions* (1999), commonly referred to as WAFAM, to perform these analyses. WAFAM uses a series of inputs into a model to calculate a numerical result for each function on a scale of 1 to 10. A result of 10 indicates that the wetland is performing a particular function at a high level while a result of 1 indicates that the wetland does not perform the function well. Results are summarized in below **(Table 3)**.

Table 3: Existing and Predicted Queen City Lake Functions and Values

Function **Water Quality**	**Existing**	**Future, 2 cfs Pipe***
Potential for Removing Sediment	10	10
Potential for Removing Nutrients	10	10
Potential for Removing Heavy Metals and Toxic Organics	6	4
Hydrology		
Potential for Reducing Peak Flows	10	10
Potential for Reducing Decreasing Downstream Erosion	10	10
Potential for Groundwater Recharge	3	2
Habitat		
General Habitat Suitability	8	8
Habitat Suitability for Invertebrates	7	7
Habitat Suitability for Amphibians	5	6
Habitat Suitability for Anadromous Fish	N/A	N/A
Habitat Suitability for Resident Fish	N/A	N/A
Habitat Suitability for Wetland Associated Birds	6	5
Habitat Suitability for Wetland Associated Mammals	6	6
Native Plant Richness	6	6
Primary Production and Export	N/A	N/A

*The 2 cfs outfall pipe is a baseline value for the purposes of calculating hydrologic conditions. The final outfall design may be more or less than 2 cfs depending on the final hydrologic inputs from landfill.

4.6.1 Functions and Values Based on Current Hydrologic Conditions

The results of the WAFAM analysis for Queen City Lake under current conditions are discussed below.

4.6.1.1 Water Quality

Queen City Lake currently has a high potential for removing sediments and nutrients from water. It rates somewhat moderate for the ability to remove metals and toxic organics. Sediment removal is a function of the residency of water within a wetland, while nutrient removal is a function of water residency and plant uptake.

4.6.1.2 Hydrology

According to the WAFAM model, the lake currently has a high potential for reducing both peak flows and downstream erosion and currently rates relatively low for its ability to recharge groundwater. These results represent the broad characterization of the WAFAM model inputs and do not account for the naturally occurring conditions in this instance. The Queen City Lake basin exhibits unusual characteristics in that the lake bottom has silty, organic soils and low infiltration rates, while the edges of the lake have very permeable soil that allow for rapid infiltration. As a result of rapid infiltration rates at higher water levels Queen City Lake is the main source of groundwater recharge to Aquifer 1. We conclude that the WAFAM model results do not accurately represent the groundwater recharge potential for the lake.

4.6.1.3 Habitat

The general habitat suitability of the lake was rated high. However, the habitat suitability for birds, mammals, invertebrates, and amphibians was mostly moderate. Because there is no permanent water or outflow to a fish-bearing stream, the potential for any fish habitat in the lake was not applicable. In addition, primary production and export was also not applicable since the lake is essentially a closed depressional system.

4.6.2 Functions and Values Based on Proposed Changes to Hydrologic Conditions

The results of the WAFAM analysis for Queen City Lake under the two predicted future conditions are discussed below. The differences between the two predicted future conditions were minor for most all categories.

4.6.2.1 Water Quality

There was generally no change in the potential for removal of sediments and nutrients for the two proposed conditions compared to current conditions. However, the ability to remove metals and toxic organics was reduced from moderate to moderately low in the proposed condition. This is likely the result of a smaller area of inundation based on the reduced input of stormwater as predicted by the closure of the landfill. The ability of a wetland to sequester metals and toxic organics is strongly influenced by emergent vegetation. The reduction in the duration, frequency, and depth of inundation is predicted to prefer scrub-shrub vegetation over emergent vegetation on the lakebed. However, improvements to the stormwater handling system at the landfill will likely reduce the need for removal of toxins from the surface water.

4.6.2.2 Hydrology

There was generally no change in the potential to reduce peak flows and downstream erosion for the proposed condition compared to current conditions. The proposed alteration of the outflow pipe is meant to allow more water storage and slower release through the pipe. There was a slight reduction in the ability of the lake to recharge groundwater. This is likely due to a decrease in lake water levels and the fact that higher infiltration rates can be achieved at higher water levels. Additional infiltration and detention facilities will be constructed adjacent to the current gravel pit in order to maintain high levels of groundwater recharge.

4.6.2.3 Habitat

General habitat suitability did not change from current conditions to the proposed conditions. The future condition with a smaller outlet pipe slightly improves the habitat suitability for amphibians.

5.0 SUMMARY AND CONCLUSIONS

Queen City Lake is an approximately 11.7-acre natural kettle that formed during the retreat of the last period of glaciation. It is identified by King County as a Category 1 wetland containing elements of palustrine emergent, palustrine scrub-shrub, and palustrine forested wetland. Hydrology for the lake is provided, for the most part, by storm and surface water discharge from the Cedar Hills Regional Landfill, located north of the Queen City Farm property.

Two actions are proposed that will affect the lake. The first action is the construction of Cedar Hills Regional Landfill stormwater improvements and the closure of the landfill by

2015, producing changes in stormwater discharge. The second is the reclamation of the gravel mine area of Queen City Farm. Reclamation of the gravel mine includes filling the main gravel pit lake, modifying the outflow structure of Queen City Lake, and creating additional stormwater management facilities adjacent to the gravel mine area.

Closure of the landfill will alter the current hydrology of the lake by creating a large area of vegetated capped landfill surface. Storm water and surface runoff will be managed to provide continued flow to the lake, but will provide less water to the lake than under current conditions. Carefully controlled discharge of storm and surface water runoff will lower the frequency of excursions of water level within the lake.

Modifying the outlet structure on Queen City Lake from its present 36" pipe to a smaller variable flow structure (2 cfs) will partially compensate for the loss of gravel pit lake's storage capacity. Modification of the pipe will take advantage of the natural storage capacity and infiltration capability of Queen City Lake to partially compensate for the retention capacity lost when the main gravel pit lake is filled. New stormwater management facilities will fulfill the remaining storage and infiltration requirements for the gravel mine reclamation project.

Alterations to the landfill will affect the hydrology of the lake by decreasing the total surface water input to the lake. As a result duration, depth, and frequency of inundation will be lower. Hydrologic models also indicate that extreme fluctuations in hydrology will be less frequent as a result of stormwater handling improvements at the landfill, and the mean water levels will be lower compared to current conditions. Reduction in the size of the outflow pipe from Queen City Lake, as a result of the gravel mine reclamation project, will help moderate the decrease in surface water inputs from the landfill by allowing slower outflow rates and thus more storage and infiltration within the lake.

The anticipated hydrologic changes are not expected change the overall size of the wetland. However, the decrease in mean water levels is expected to have an impact on the plant community distribution within the wetland. The effect on plant communities will likely be a reduction in the area of emergent vegetation in favor of scrub-shrub and the establishment of a forested community at lower elevations around the lake. Changes to the distribution of plant communities will alter the habitat functions of wetland. While habitat for some species may become more scarce, habitat for another species may become more plentiful. Assessment of one habitat type over another is beyond the scope of this study. The wetland functional assessment indicates that, overall, wetland functions will remain largely the same between the current and future conditions.

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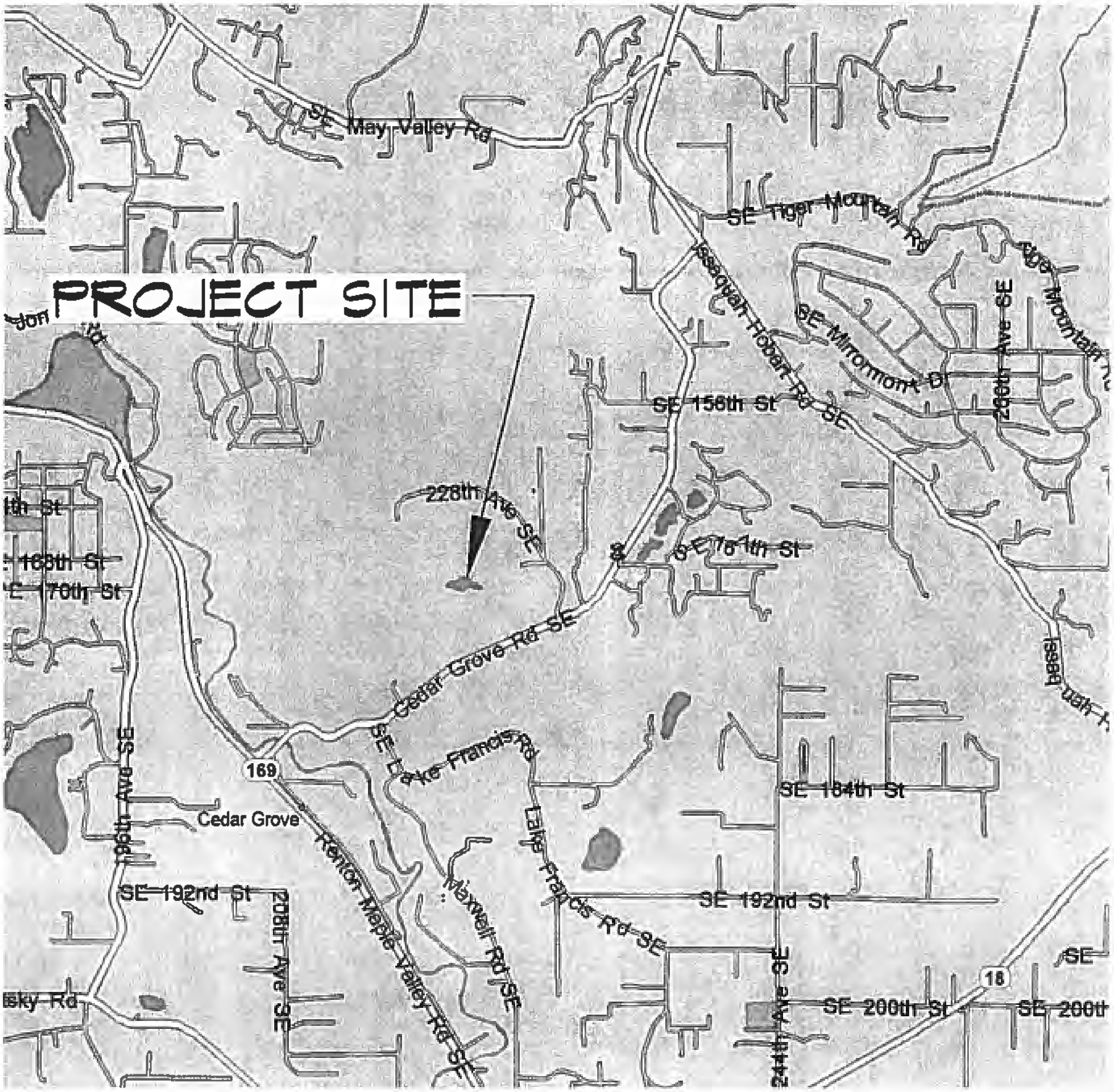

SOURCE: MICROSOFT STREETS AND TRIPS, 2002.

NORTH

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FIGURE #1

VICINITY MAP
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
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SCALE		
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DATE		
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REVISED		

1

N

LEGEND

QCF Total Site

References: Aerial photograph provided by the Washington State Geospatial Data Archive (Photo date ca. 2002). Roads and streams are from King County GIS data.

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FIGURE #2

QUEEN CITY FARMS SITE MAP
QUEEN CITY LAKE
KING COUNTY, WASHINGTON

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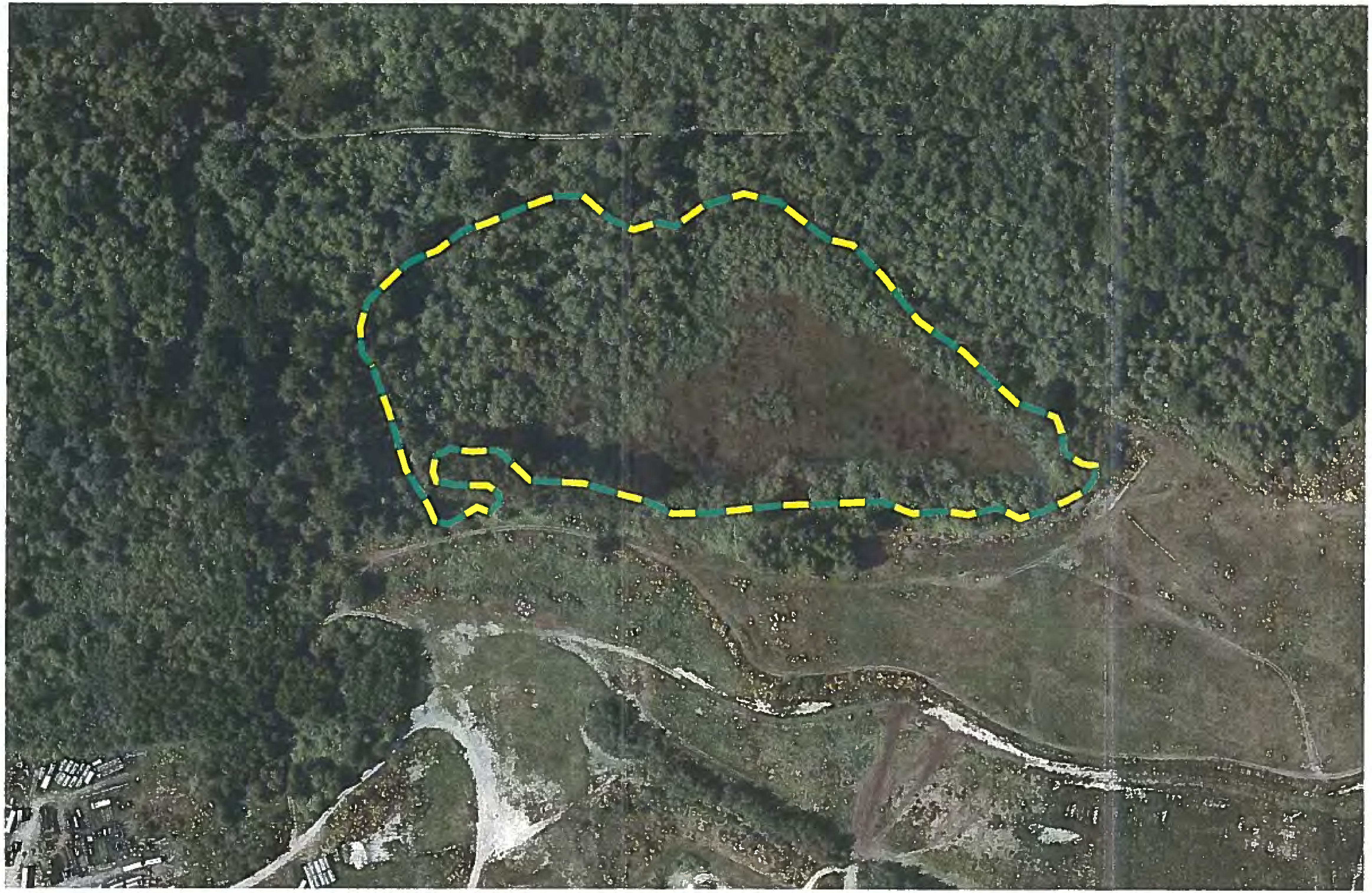
SCALE 1 in : 800.00 ft		2
DATE 01-31-07		
REVISED		



References: Aerial photograph provided by the Washington State Geospatial Data Archive (Photo date ca. 2002). Wetland boundary is from survey data supplied by Landau and Associates.

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FIGURE #3

QUEEN CITY LAKE SITE MAP
QUEEN CITY LAKE
KING COUNTY, WASHINGTON

DESIGN	DRAWN DRT	PROJECT 377
SCALE 1 in : 200.00 ft		3
DATE 01-31-07		
REVISED		

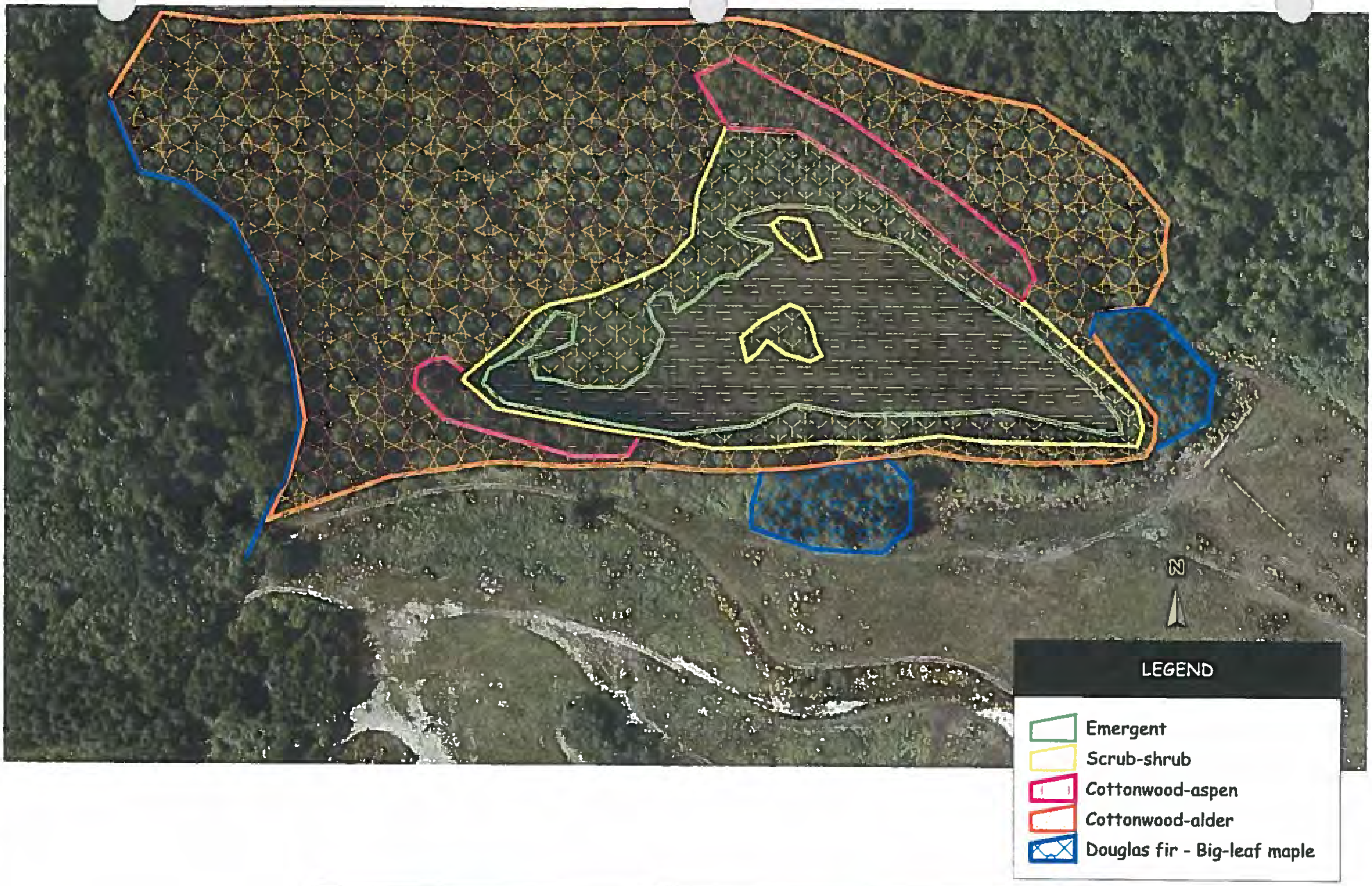

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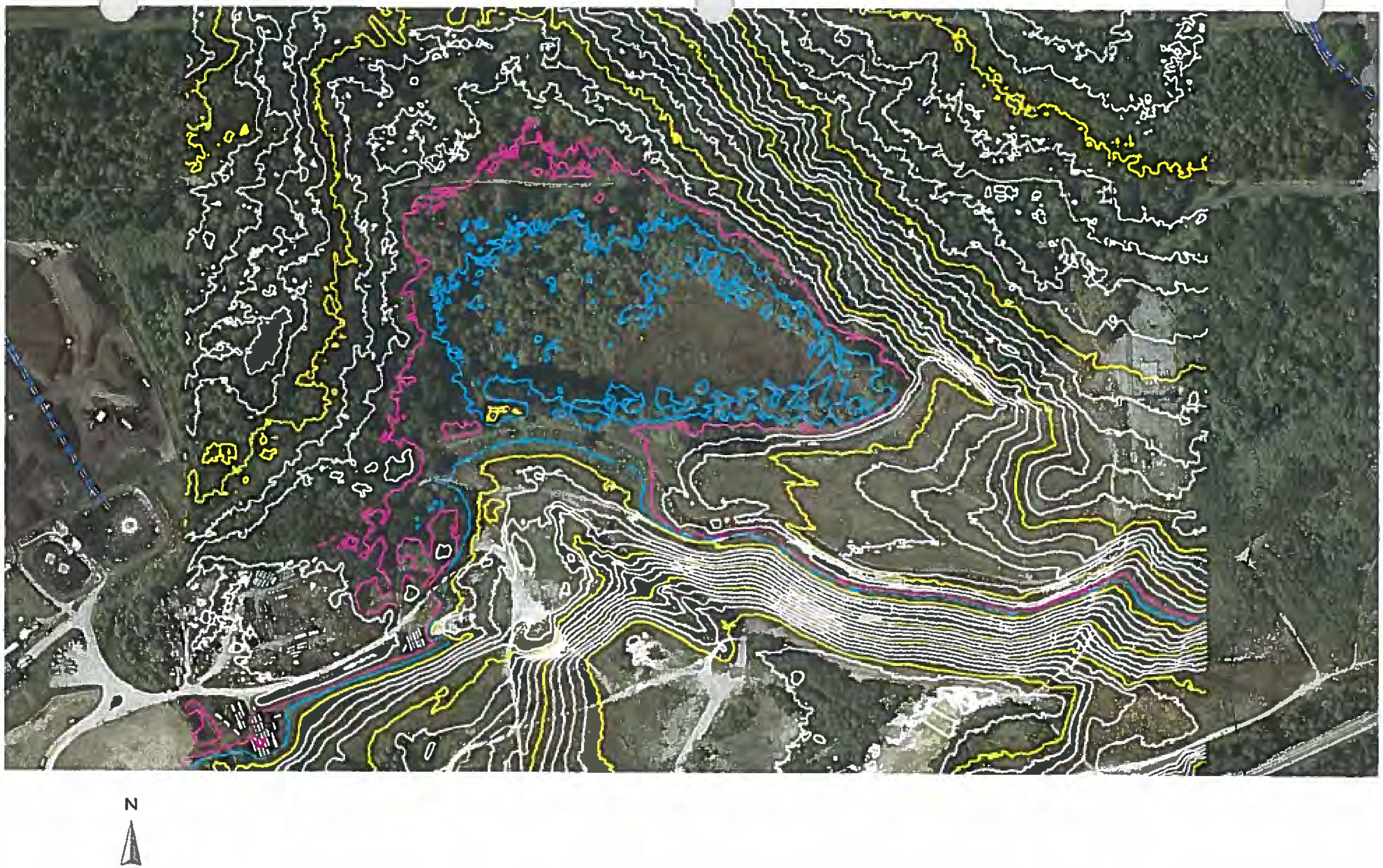
FIGURE 4

PLANT COMMUNITY DISTRIBUTIONS
QUEEN CITY FARMS
KING COUNTY, WASHINGTON

DESIGN	DRAWN DRT	PROJECT 377
SCALE 1 in : 200.00 ft		
DATE 01-31-07		4
REVISED		



LEGEND

- 444 Lake Bed Elevation
- 460 Top of Bank Elevation

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FIGURE 5

LIDAR BASED 5-FOOT CONTOURS
QUEEN CITY FARMS
KING COUNTY, WASHINGTON

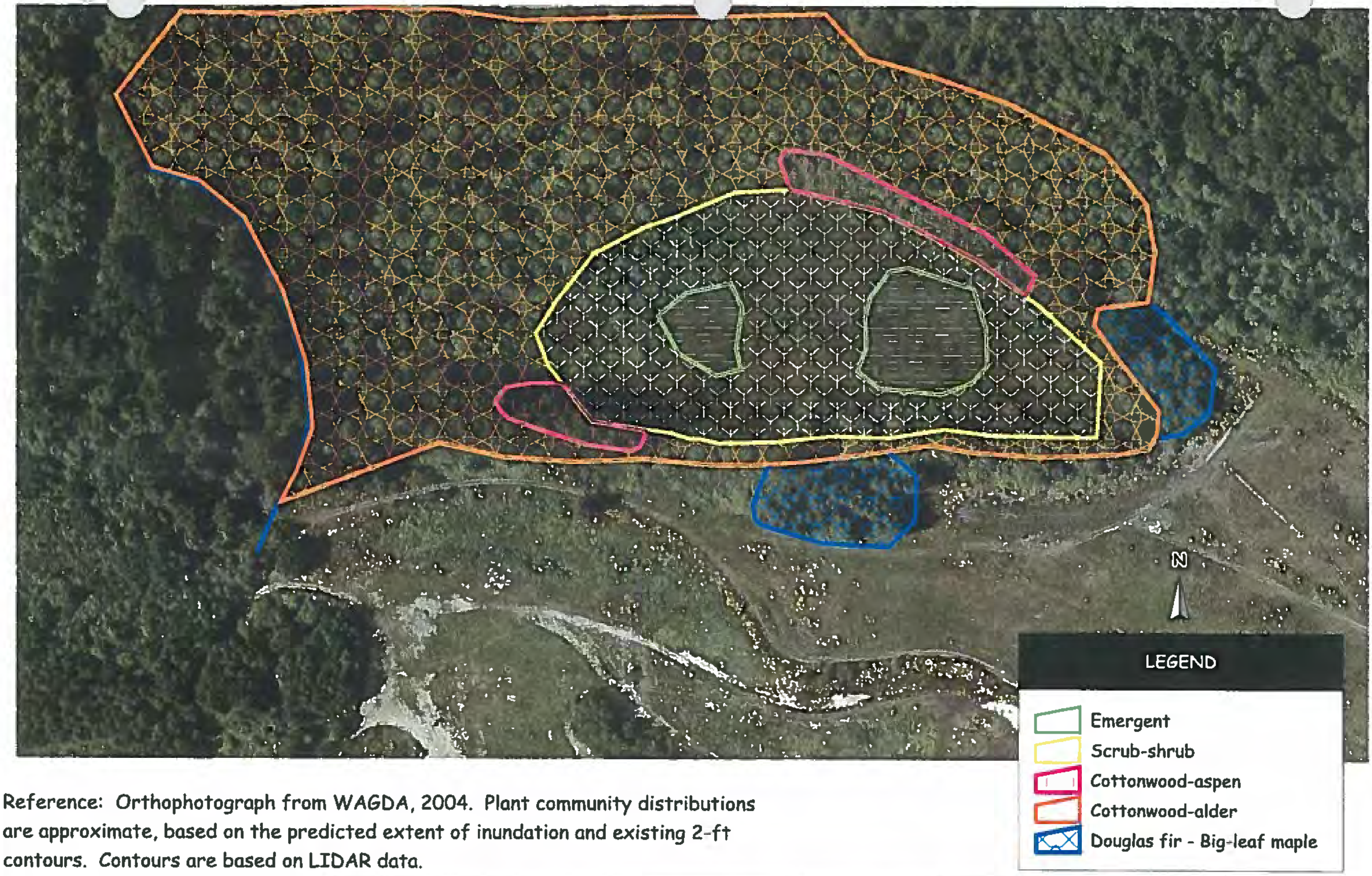
DESIGN	DRAWN	PROJECT
	DRT	377

SCALE 1 in : 400.00 ft

DATE 01-31-07

REVISED

5



Reference: Orthophotograph from WAGDA, 2004. Plant community distributions are approximate, based on the predicted extent of inundation and existing 2-ft contours. Contours are based on LIDAR data.

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FIGURE 14
PLANT COMMUNITY DISTRIBUTIONS
DECREASED HYDROLOGY SCENARIO
QUEEN CITY FARMS
KING COUNTY, WASHINGTON

DESIGN	DRAWN	PROJECT
	DRT	377
SCALE 1 in : 200.00 ft		
DATE 01-31-07		14
REVISED		

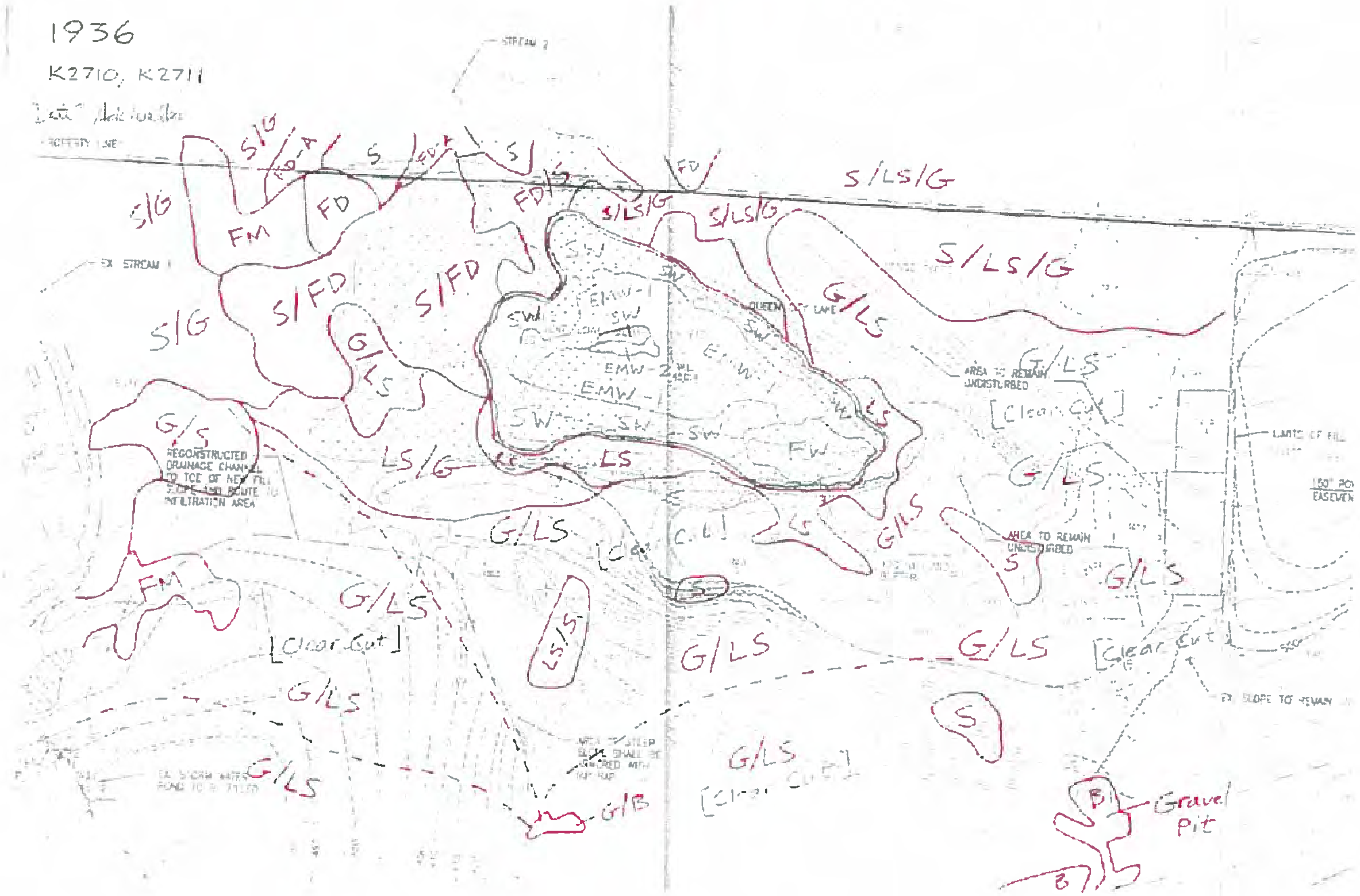

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FIGURE #A1

1936 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377

SCALE
N.T.S.

DATE
01-31-05

REVISED

A1

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FIGURE #A2

1960 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377

SCALE
N.T.S.
DATE
01-31-05
REVISED

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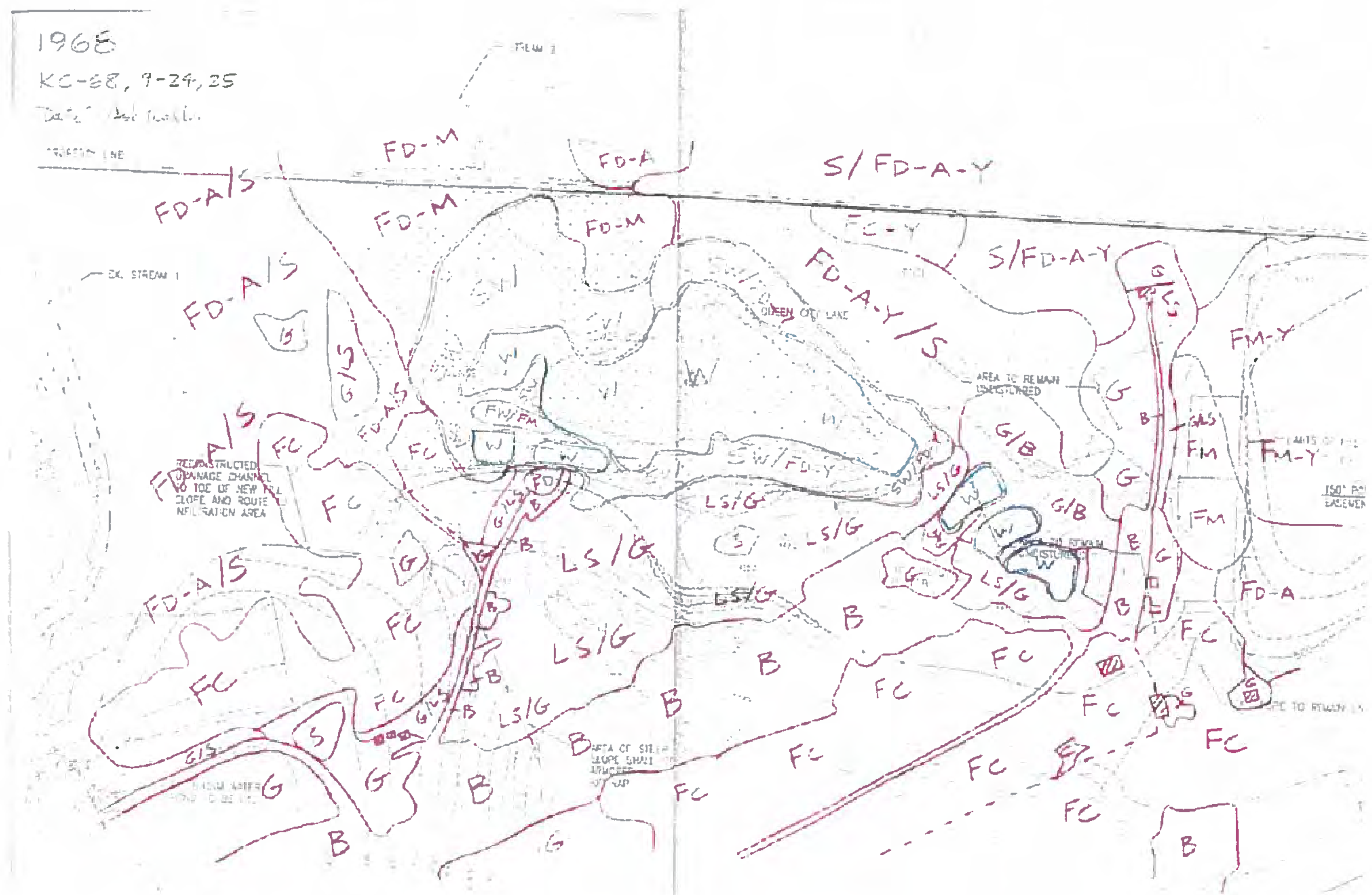

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FIGURE #A3

1968 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377
SCALE N.T.S.		
DATE 01-31-05		A3
REVISED		

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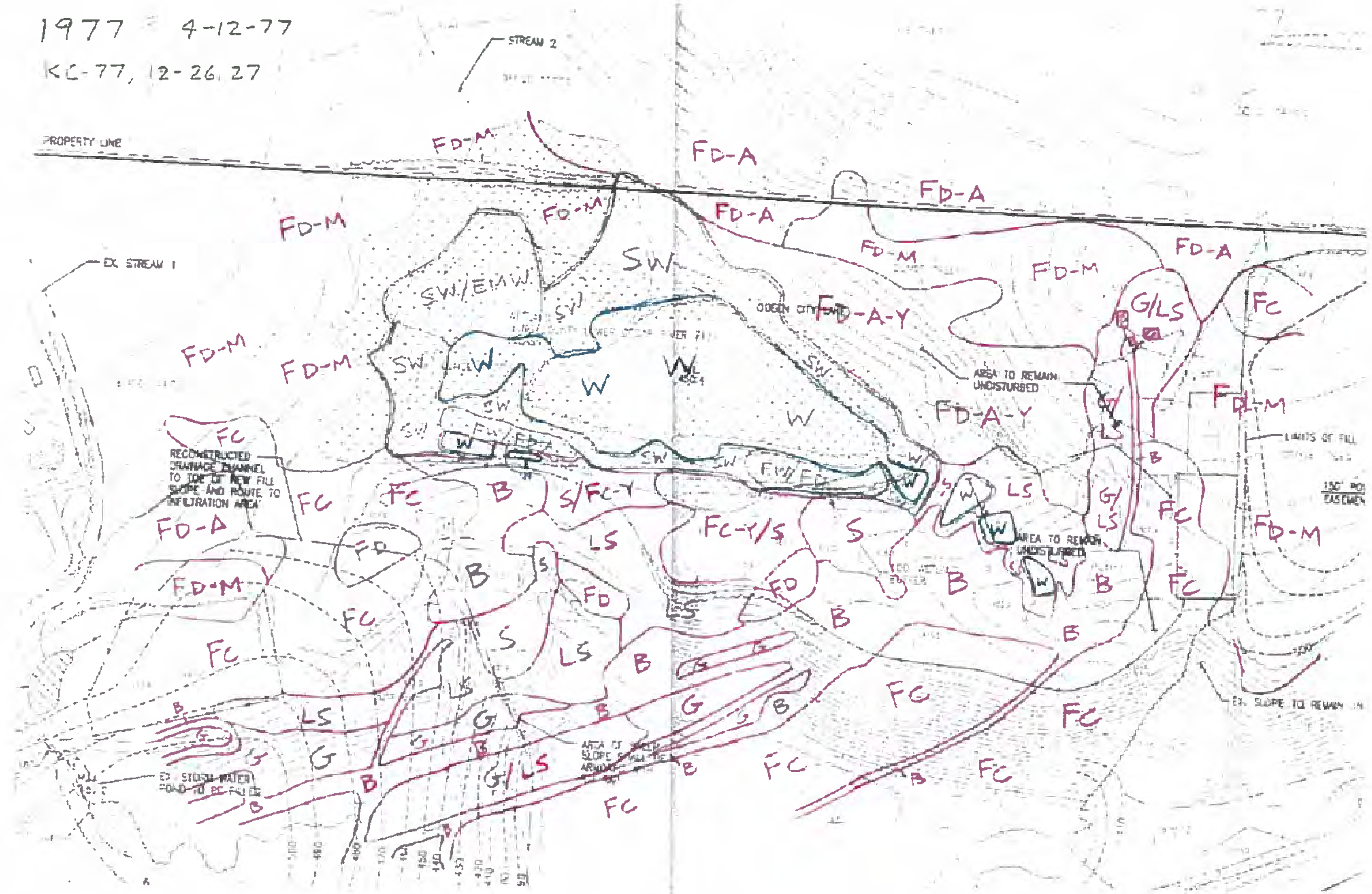

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Resource & Environmental Planning
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Woodinville, Washington 98077
Bus (425)861-7550 - Fax (425)861-7549

FIGURE #A4

1977 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377
SCALE N.T.S.		
DATE 01-31-05		A4
REVISED		

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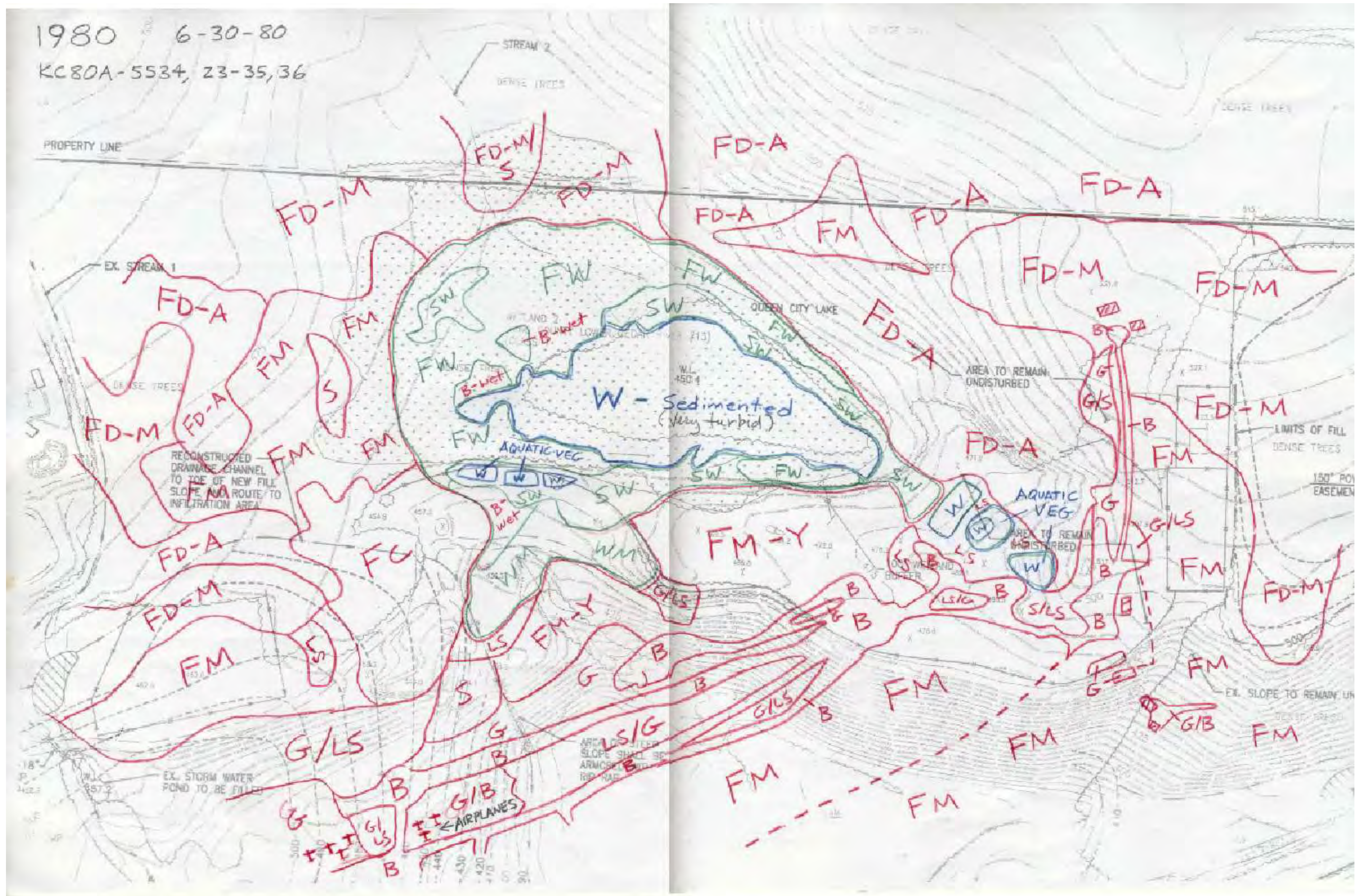

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FIGURE #A5

1980 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377

SCALE N.T.S.

DATE 01-31-05

REVISED

A5

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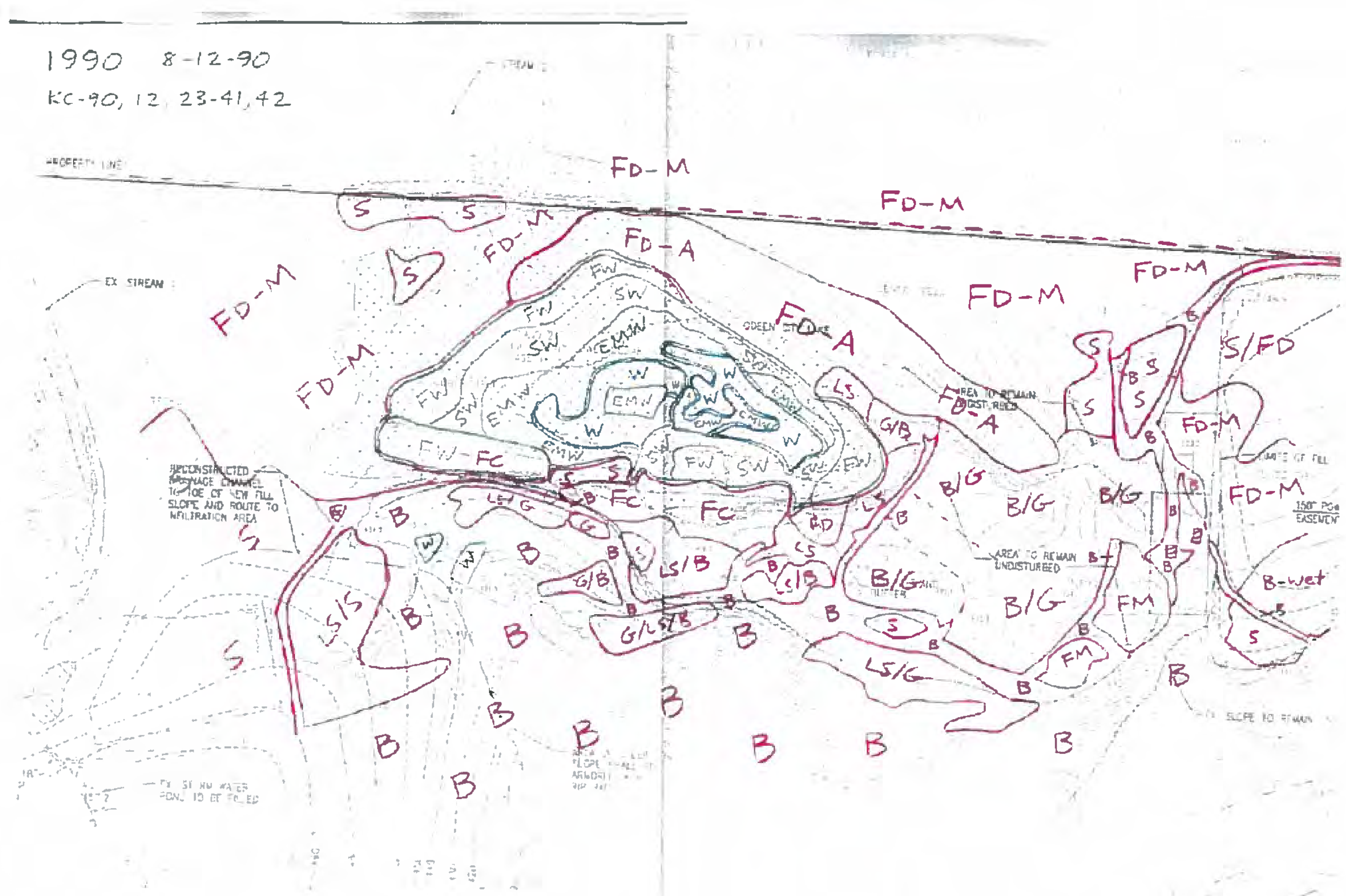

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FIGURE #A6

1990 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377
SCALE N.T.S.		
DATE 01-31-05		A6
REVISED		

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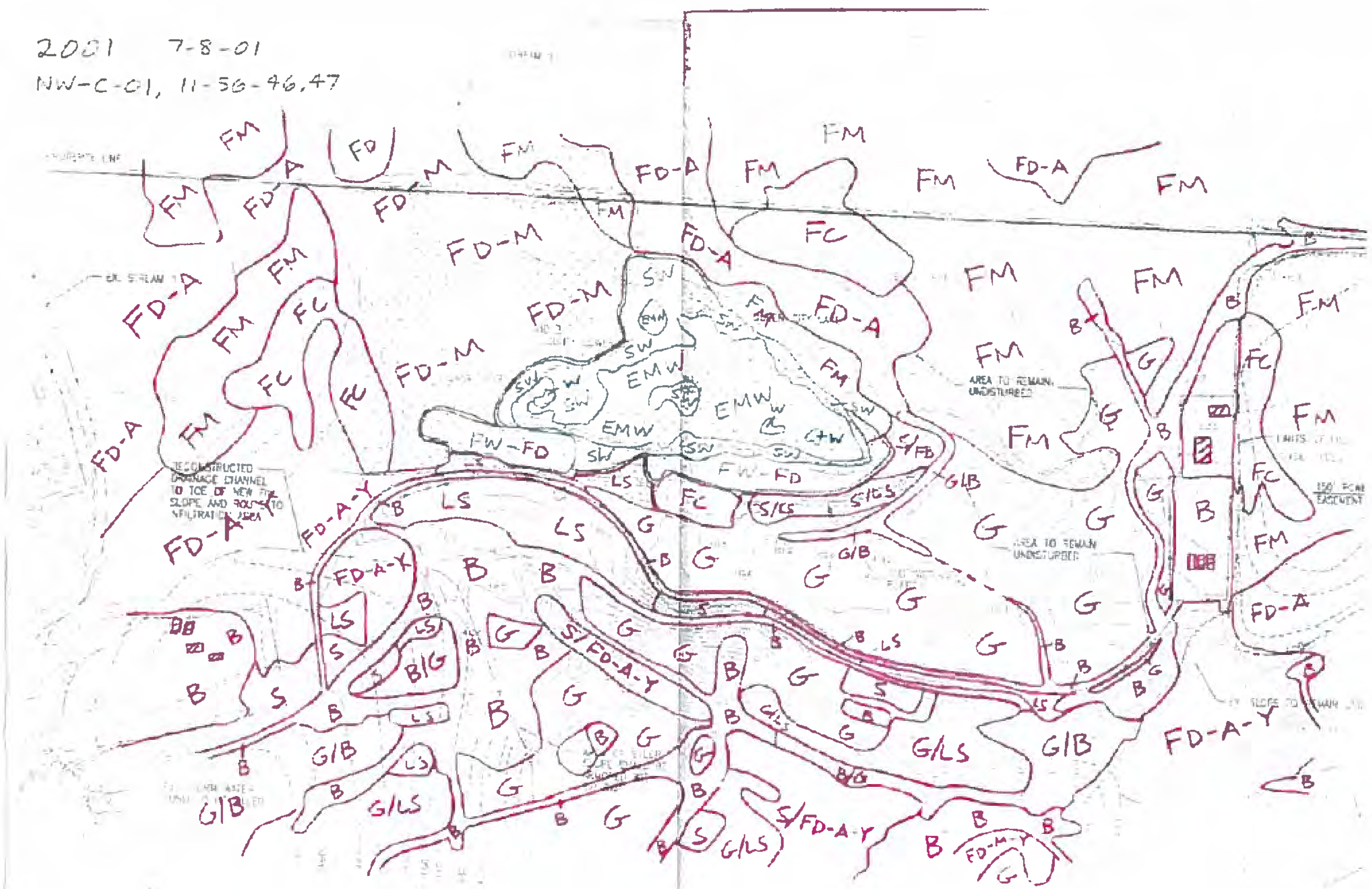

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FIGURE #A7

2001 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377

SCALE N.T.S.
DATE 01-31-05
REVISED

A7

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FIGURE #A8

2004 PHOTO-INTERPRETATION
QUEEN CITY LAKE
KING COUNTY, WA

DESIGN	DRAWN	PROJECT
	DRT	377
SCALE N.T.S.		
DATE 01-31-05		A8
REVISED		

Z:\DRAWING\300-399\tal377\Outgoing\01-25-07\TAL-377 scans (01-25-07).dwg

CLEAR CREEK SOLUTIONS, INC.

6200 Capitol Blvd. Suite F
Tumwater, WA 98501
360-943-0304
www.clearcreeksolutions.com

DATE: 22 November 2006

TO: Jennifer Olson, Project Scientist, Landau Associates

CC:

FROM: Joe Brascher, President, C.E.O.

SUBJECT: Queen City Wetland Flux Analysis

As part of the Queen City Lake hydroperiod analysis Landau Associates contracted with Clear Creek Solutions to assist in the continuous modeling of the lake. This analysis is based on previous modeling work performed by W&H Pacific using the hydrologic modeling program KCRTS. Landuse data and project location were provided by W&H Pacific and incorporated into a WWHM3 model by Clear Creek Solutions staff.

The 8-year special Landsburg precipitation gage with a scaling factor of 1.0 was used in KCRTS modeling work. The 8-year special record in the KCRTS modeling incorporates eight precipitation events that have been modified to attempt to reproduce specific return period precipitation events. This record is not used by the WWHM3 model. Only the 50-year historic Landsburg record is used in the WWHM3. The result is that both models will produce slightly different results. For the purposes of hydroperiod analysis it is best to use the long-term historic record because it is important to model the lake under as many different seasonal conditions as possible.

The landuse conditions for both models are identical. Four scenarios were modeled and are listed in Table 1 below.

Table 1.

WWHM3 Modeling Scenarios – Queen City Lake

Scenario	Till Forest Area (ac)	Till Grass Area (ac)	Impervious Area (ac)	Total Area (ac)
1	132.41	132.77	74.82	340
2	331.50		8.50	340
3	331.50		8.50	340
4	331.50		8.50	340

Scenario 1: Existing Conditions (2000)

The existing conditions scenario represents the condition of the watershed as of the year 2000. Queen City Lake has a 36-inch diameter discharge culvert at elevation 451.62.

Scenario 2: Future Conditions (2015) with existing outlet.

The future conditions (scenarios 2, 3, and 4) represent the future state of the basin after development. This includes infiltration facilities installed to mimic undeveloped conditions that existed in 1979. It is for this reason that the site has been modeled as mostly forested till with

8.5 acres of impervious area set aside to represent the area of the lake. In scenario 2 the lake maintains the same outlet configuration as the existing conditions model.

Scenario 3: Future Case (2015) with no discharge.
The future watershed conditions represented in scenario 3 are identical to those represented in scenario 2. In this scenario the 36-inch diameter culvert has been removed and no outlet structure is modeled. All discharge from the lake is represented by infiltration.

Scenario 4: Future Case (2015) with 2 cfs discharge.
The future watershed conditions represented in scenario 4 are also identical to those represented in scenario 2. In this scenario the 36-inch diameter culvert has been replaced with a discharge configuration at the same elevation that allows for a maximum discharge of 2 cfs.

Results
For each scenario hydroperiod analysis has been performed.

The results have been summarized into two hydroperiod tables for each scenario. Both tables display similar information in a different format. The first table displays the hydroperiod results for total hours exceeded. The second table displays the results in terms of average annual hours of exceedence.

The column definitions for the first table are as follows:

1) Month: total of the results for the month over the 50 years of analysis.
2) Mean Stage (cm): the mean stage of water in the lake over the 50 years of analysis.
3) Excursions (total hours): the total number of hours that the stage in the lake was 15 cm above or below the mean stage of the lake for that month.
4) Long Excursions (total hours): total number of hours that the stage in the lake remained 15 cm above or below the mean stage of the lake for a period of longer than 72 hours.
5) Excursions 168 (total hours): total number of hours that the stage in the lake remained 15 cm above or below the mean stage of the lake for a period of longer than 168 hours.
6) Excursions 336 (total hours): total number of hours that the stage in the lake remained 15 cm above or below the mean stage of the lake for a period of longer than 336 hours.

The column definitions for the second table are as follows:

1) Month: averaging the results for this month over the 50 years of analysis.
2) Mean Stage (cm): the mean stage of water in the lake over the 50 years of analysis.
3) Excursions (average hours): the average annual hours that the stage in the lake was 15 cm above or below the mean stage of the lake for that month.
4) Long Excursions (average hours): the average annual hours that the stage in the lake remained 15 cm above or below the mean stage of the lake for a period of longer than 72 hours.
5) 168 Hour Excursions (average hours): the average annual hours that the stage in the lake remained 15 cm above or below the mean stage of the lake for a period of longer than 168 hours.
6) 336 Hour Excursions (average hours): the average annual hours that the stage in the lake remained 15 cm above or below the mean stage of the lake for a period of longer than 336 hours.

Scenario 1

Table 1.1 – Total Excursions for Existing Conditions

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(total hours)	(total hours)	(total hours)	(total hours)
Jan	92.7	30930	106	71	31
Feb	83.6	27477	106	65	19
Mar	72.7	29608	123	70	23
Apr	56.2	29603	122	69	19
May	30.5	32091	142	64	14
Jun	25.3	32530	128	64	22
Jul	10.9	5830	23	6	0
Aug	9.6	5392	19	4	2
Sep	19.6	32972	124	68	22
Oct	40.6	31157	128	53	19
Nov	86.2	29708	115	66	24
Dec	96.0	30564	109	71	30

Table 1.2 – Average Annual Excursions for Existing Conditions

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(average hours)	(average hours)	(average hours)	(average hours)
Jan	92.7	619	2.1	1.4	0.6
Feb	83.6	550	2.1	1.3	0.4
Mar	72.7	592	2.5	1.4	0.5
Apr	56.2	592	2.4	1.4	0.4
May	30.5	642	2.8	1.3	0.3
Jun	25.3	651	2.6	1.3	0.4
Jul	10.9	117	0.5	0.1	0.0
Aug	9.6	108	0.4	0.1	0.0
Sep	19.6	659	2.5	1.4	0.4
Oct	40.6	623	2.6	1.1	0.4
Nov	86.2	594	2.3	1.3	0.5
Dec	96.0	611	2.2	1.4	0.6

Scenario 2

Table 2.1 – Total Excursions for Future Conditions with 36-Inch Diameter Culvert

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(total hours)	(total hours)	(total hours)	(total hours)
Jan	76.7	31014	102	71	31
Feb	70.6	27808	104	65	19
Mar	57.3	29371	124	70	23
Apr	41.8	30514	108	69	19
May	17.0	25768	94	64	14
Jun	12.8	5157	21	64	22
Jul	3.1	1946	7	6	0
Aug	1.2	1168	1	4	2
Sep	3.1	2270	6	68	22
Oct	8.8	4921	18	53	19
Nov	48.6	31146	100	66	24
Dec	73.2	30653	101	71	30

Table 2.2 – Average Annual Excursions for Future Conditions with 36-Inch Diameter Culvert

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(average hours)	(average hours)	(average hours)	(average hours)
Jan	76.7	620	2.0	1.4	0.6
Feb	70.6	556	2.1	1.3	0.4
Mar	57.3	587	2.5	1.4	0.5
Apr	41.8	610	2.2	1.4	0.4
May	17.0	515	1.9	1.3	0.3
Jun	12.8	103	0.4	1.3	0.4
Jul	3.1	39	0.1	0.1	0.0
Aug	1.2	23	0.0	0.1	0.0
Sep	3.1	45	0.1	1.4	0.4
Oct	8.8	98	0.4	1.1	0.4
Nov	48.6	623	2.0	1.3	0.5
Dec	73.2	613	2.0	1.4	0.6

Scenario 3

Table 3.1 – Total Excursions for Future Conditions with No Discharge

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(total hours)	(total hours)	(total hours)	(total hours)
Jan	97.2	32150	80	59	36
Feb	89.3	29167	83	62	26
Mar	69.8	30487	94	61	30
Apr	47.4	30705	99	67	31
May	18.3	27860	107	47	11
Jun	14.7	5222	22	12	3
Jul	3.2	1959	7	1	0
Aug	1.2	1168	1	0	0
Sep	3.1	2270	6	1	0
Oct	8.8	4921	18	8	0
Nov	54.0	31314	83	58	28
Dec	91.3	32341	82	62	33

Table 3.2 – Average Annual Excursions for Future Conditions with No Discharge

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(average hours)	(average hours)	(average hours)	(average hours)
Jan	97.2	643	1.6	1.2	0.7
Feb	89.3	583	1.7	1.2	0.5
Mar	69.8	610	1.9	1.2	0.6
Apr	47.4	614	2.0	1.3	0.6
May	18.3	557	2.1	0.9	0.2
Jun	14.7	104	0.4	0.2	0.1
Jul	3.2	39	0.1	0.0	0.0
Aug	1.2	23	0.0	0.0	0.0
Sep	3.1	45	0.1	0.0	0.0
Oct	8.8	98	0.4	0.2	0.0
Nov	54.0	626	1.7	1.2	0.6
Dec	91.3	647	1.6	1.2	0.7

Scenario 4

Table 4.1 – Total Excursions for Future Conditions with 2 cfs Discharge

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(total hours)	(total hours)	(total hours)	(total hours)
Jan	84.4	31457	96	67	33
Feb	76.5	28506	102	66	20
Mar	61.2	29622	115	67	29
Apr	43.7	30614	105	69	29
May	17.3	26295	96	41	10
Jun	13.4	5158	21	12	3
Jul	3.1	1948	7	1	0
Aug	1.2	1168	1	0	0
Sep	3.1	2270	6	1	0
Oct	8.8	4921	18	8	0
Nov	51.2	31156	93	63	26
Dec	79.2	31052	93	62	33

Table 4.2 – Average Annual Excursions for Future Conditions with 2 cfs Discharge

Month	Mean Stage	Excursions	Long Excursions	168 Hour Excursions	336 Hour Excursions
	(cm)	(average hours)	(average hours)	(average hours)	(average hours)
Jan	84.4	629	1.9	1.3	0.7
Feb	76.5	570	2.0	1.3	0.4
Mar	61.2	592	2.3	1.3	0.6
Apr	43.7	612	2.1	1.4	0.6
May	17.3	526	1.9	0.8	0.2
Jun	13.4	103	0.4	0.2	0.1
Jul	3.1	39	0.1	0.0	0.0
Aug	1.2	23	0.0	0.0	0.0
Sep	3.1	45	0.1	0.0	0.0
Oct	8.8	98	0.4	0.2	0.0
Nov	51.2	623	1.9	1.3	0.5
Dec	79.2	621	1.9	1.2	0.7

Technical Memorandum

TO: Queen City Farms, Inc.

FROM: Katherine Saltanovitz, PE and Meghan Veilleux, EIT

DATE: April 4, 2018

RE: **Queen City Lake Basin – King County Runoff Time Series Modeling**
Queen City Farms Phase III Refill
Maple Valley, Washington
Project No. 0992002.050.051

Introduction

Stormwater management at the Queen City Farms (QCF) site comprises several components including infiltration areas, constructed detention ponds, and enhanced storage in Queen City Lake. The currently permitted refill plan for QCF includes adding an outlet structure to the lake to increase the stormwater storage available within the lake, while allowing for emergency overflow of large storm events. W&H Pacific completed a study in December 2006 that calculated the peak stormwater runoff release rates and peak stage/elevation within the lake to model changes to lake wetland hydrology and provide recommendations for the proposed lake outlet structure (W&H Pacific 2006).

The next phase of the QCF refill (referred to as the Phase III Refill) is proposed to include rerouting the existing Tributary 316A from its current infiltration location into Queen City Lake. The purpose of this current technical memorandum is to update the December 2006 study to model the effect on water levels in Queen City Lake if Tributary 316A is redirected into the lake and to determine if any changes are needed to the proposed outlet structure.

Queen City Lake Hydrology

Queen City Lake has no natural surface water outlet. Water infiltrates into the underlying sediments and eventually to an aquifer, that underlies the lake bottom (LAI 2007). Water infiltration rate within the lake is dependent on the water elevation. Landau Associates, Inc. (LAI) estimated the infiltration rates by using the relationship of the lake level with lake volume, as presented in a technical memorandum dated January 29, 2007 (LAI 2007).

The infiltration rates for the varying lake depths are presented in Table 1, which is reprinted from Table 2 of the January 29, 2007 LAI memorandum (LAI 2007).

Table 1 – Queen City Lake Estimated Infiltration Rates (reprinted from 2007 TIR Figure 2)

Lake Depth (ft)	Estimated Infiltration Rate (cfs)	Estimated Lake Surface Area (sf)	Estimated Infiltration Rate per Unit Area (in/hr)
0 to 1	0.3	100,000	0.13
1 to 2	0.8	240,000	0.14
2 to 3	1.2	320,000	0.16
3 to 4	1.6	420,000	0.16
4 to 5	1.8	470,000	0.17
5 to 6	2.0	520,000	0.17
6 to 7	3.0	640,000	0.20
7 to 8	7.0	675,000	0.45
8 to 9	15.7	750,000	0.90

Notes:

1. Bottom of the lake elevation assumed to be 444 ft MSL.
2. Queen City Lake surface area estimated from Figure 3-10 of the Queen City Farms RI Report (Landau Associates 1990).

Abbreviations:

ft = feet/foot
cfs = cubic feet per second
sf = square feet
in/hr = inch per hour

Queen City Lake does have an existing 36-inch outlet pipe that was installed in 1991 to control erosion of the gravel pit face. This pipe drains to another on-site infiltration area. This pipe will be removed as part of the currently permitted refill and replaced with a new engineered outlet structure.

Surface water enters Queen City Lake as runoff from the Cedar Hills Sub-basin, which covers approximately 340 acres and includes a portion of the Cedar Hills Regional Landfill (CHRL; LAI 2007). In the December 2006 study by W&H Pacific, this basin was modeled as till forest conditions. The surface area of the lake is considered an impervious surface for all analyses. W&H Pacific delineated the Cedar Hills Sub-basin from Attachment 1 provided in a stormwater report written by King County Solid Waste Division and LiDAR topographic mapping data from 2000.

Drainage Basins

Tributary 316A, which drains the Maple Hills Sub-basin, currently flows through the Queen City Farms site and infiltrates in the Main Infiltration Area. As part of the Phase III Refill, it is proposed to reroute Tributary 316A into Queen City Lake. The boundary of the Maple Hills sub-basin is shown on Figure 1. The entire Maple Hills sub-basin was modeled to represent the runoff that will be directly flowing into the lake from Tributary 316A.

The lake will continue to receive runoff from the Cedar Hills Sub-basin as well. The Cedar Hills sub-basin includes CHRL property, some buffer area east and west of the CHRL boundary limits, as well as Queen City Farm's property. The boundary of this sub-basin was updated based on site development that has occurred since 2007, and on recent aerial photographs; it is shown in Figure 1.

The lake will also receive approximately 21 acres of runoff from the gravel refill mound, which currently flows to Main Gravel Pit Lake. The drainage area that will be redirected to Queen City Lake from Main Gravel Pit Lake is designated as the Phase III Fill drainage basin, as shown on Figure 1.

Outlet Structure Design

Consistent with the 2006 W&H Pacific study, there are two design parameters for the outlet structure:

1. It is assumed that wetland species have been established from the bottom portion of the lake to an elevation, which is approximately equal to the invert of the existing 36-inch corrugated metal outlet pipe. The new outlet structure will maintain this minimum water level.
2. The maximum lake water level will be maintained at a depth less than or equal to 9 ft to protect the adjacent superfund site barrier wall. Major (greater than 100-year return frequency) storm events will be routed through the lake with a high flow overflow structure.

King County Runoff Time Series Modeling

Because the drainage entering Queen City Lake has changed significantly, including the addition of a new basin, the King County Runoff Time Series (KCRTS) Modeling previously performed by W&H Pacific has been updated to reflect these changes.

Drainage Basins and Land Cover

The previous W&H Pacific study included only the Cedar Hills Sub-basin as a surface water source for Queen City Lake, with the following land cover (Attachment 2):

Till Forest:	331.5 acres
Impervious:	8.5 acres
Total:	340.0 acres

Since 2006, the Cedar Hills Sub-basin has changed slightly. The boundary was modified and a compost pad was added that allows infiltration, decreasing some forested area. The land cover in this basin has not changed significantly since the previous study; therefore, in KCRTS, it is modeled as the same land cover (till forest).

With the plan to reroute Tributary 316A to Queen City Lake, the Maple Hills Sub-basin is added to the modeling as a contributing upstream basin. Based on aerial base maps (Terrain Navigator Pro 2000), this sub-basin appears to be mostly forested, with some developed and residential areas in the upper northwest corner of the basin. Additionally, a portion of the Cedar Grove Composting property overlaps with the basin. These areas have been conservatively modeled as impervious, with the rest of the drainage basin modeled as "Outwash Forest". A portion of the new compost pad is contained in the Maple Hills Sub-basin, but is not included in the drainage area since it is routed to its own infiltration basin rather than entering the lake. Additionally, due to the Phase III Refill project, the drainage patterns have changed, resulting in a portion of the Maple Hills Sub-basin that will no longer drain to Tributary 316A, and subsequently Queen City Lake.

The Phase III Refill project also results in a portion of the Main Gravel Pit Sub-Basin to be routed to Queen City Lake. Based on modeling used for sizing other stormwater facilities for QCF, the refill area is modeled as "Till Pasture". A summary of the sub-basins that were modeled in KCRTS are shown in Table 2.

Table 2. Queen City Lake Sub-Basin Summary

Sub-Basin			
Cedar Hills		**Maple Hills**	
Till Forest: Till Pasture: Impervious:	324 ac 21 ac 8.5 ac	Outwash Forest: Impervious:	145 ac 57 ac
TOTAL:	**353.5 ac**	**TOTAL:**	**202 ac**

Therefore, the total contributing drainage area to Queen City Lake in the Phase III Refill is 556 ac.

Lake Volume

Lake stage-storage volume was calculated from topographic survey presented in the W&H Pacific 2006 study. Attachment 3 shows the resulting contours generated from the surveyed data, the location of the existing 36-inch corrugated metal outlet pipe, and the location of the existing overflow point. Together with the cross-sections and LiDAR, a volume of a reservoir calculation was performed. The stage-storage volume calculation and chart for the existing Queen City Lake were shown in Figures A2 and A3 of the 2006 study. Figure A2 was used to develop Table 3, which shows stage and storage volume of Queen City Lake up to a maximum depth of 10.9 ft. Figure A3 is reproduced as Table 3 and Figure 2 of this memorandum. As reported in the 2006 study, the total storage volume of Queen City Lake below a depth of 9.0 ft is approximately 62.0 acre-feet (ac-ft), based on the stage-storage curve shown in Figure 2.

Table 3 Queen City Lake Volume of a Reservoir Calculation

Elevation (ft)	Stage (ft)	Elevation Difference (ft)	Area (sf)	Incremental Volume (cf)	Total Volume (cf)	Total Volume (ac-ft)
446.6	0		0		0	0
		0.4		37		
447	0.4		276		37	0.0008
		1.0		28,829		
448	1.4		81,470		28,866	0.6627
		1.0		126,733		
449	2.4		178,228		155,599	3.5721
		1.0		209,894		
450	3.4		243,242		365,493	8.3906
		1.0		275,747		
451	4.4		309,583		641,240	14.7208
		1.0		340,166		
452	5.4		371,695		981,406	22.5300
		1.0		401,890		
453	6.4		432,861		1,383,296	31.7561
		1.0		471,378		
454	7.4		510,974		1,854,674	42.5774
		1.0		540,975		
455	8.4		571,541		2,395,648	54.9965
		1.0		602,137		
456	9.4		633,261		2,997,786	68.8197
		1.5		971,082		
457.5	10.9		661,618		3,968,867	91.1127

Abbreviation/Acronym:

cf = cubic foot/feet

Outlet Structure

The originally designed outlet structure for Queen City Lake has a two-stage discharge. The Stage 1 discharge is a 12-inch orifice set at the elevation of the existing 36-inch outlet pipe (451.6 ft), which ensures that design parameter 1 will be satisfied (no impacts to wetland vegetation below this elevation). The Stage 2 discharge is an overflow weir set just below the maximum water elevation of 9.0 ft above the lake bottom (455.6 ft), which ensures that design parameter 2 will be satisfied. The sizing of the weir was based on the 100-year peak inflow to Queen City Lake, which was originally modeled by W&H Pacific as 40.8 cfs. This modeling was updated to account for the additional basin area, and is attached in Appendix A. The new peak inflow to Queen City Lake, used for overflow weir sizing, is 77.05 cfs.

Model Scenarios

The following two model scenarios were run for the updated Phase III refill.

Scenario A – New Basins, Original Outlet

The purpose of this scenario is to model the new Phase III Refill sub-basin conditions with the 2006 W&H Pacific parameters for the two-outlet reservoir routing (infiltration and orifice discharge rates). The infiltration rates were as presented in Table 1. The orifice discharge rates were set to match the rates presented in the 2006 W&H Pacific study. (However, W&H Pacific did not provide orifice discharge rate calculations.) The Stage 1 and Stage 2 discharge elevations and diameters remained unchanged.

The KCRTS input and modeling results can be found in Appendix A, and are summarized in Table 3.

Scenario B – 12-inch Orifice with Updated Discharge Rates

For this scenario, the two-outlet reservoir routing file was updated to include the Stage 1 and Stage 2 discharge flow rates calculated based on orifice and weir equations. The Stage 1 discharge elevation and diameter remained unchanged. The Stage 2 discharge elevation (overflow weir) was lowered 1 ft to an elevation of 454.56 ft.

The discharge rate was determined using the orifice equation for the Stage 1 flow (discharge out of the low flow orifice) and the weir equation for the Stage 2 flow (discharge out of the overflow weir).

The KCRTS input, including orifice flow calculations, and modeling results can be found in Appendix B, and are summarized in Table 3.

Table 4 – Summary of 2- and 100-year Peaks for All Scenarios

Scenario	Total drainage area (ac)	Orifice Diameter	2-year infiltration rate (cfs)	2-year Peak Release Rate (cfs)	2-year Stage Elevation (ft)	100-year infiltration rate (cfs)	100-year Peak Release Rate (cfs)	100-year Stage Elevation (ft)
2006 E	340	12”	2.93	3.38	6.33	15.70	7.02	8.96
A	556	12”	5.67	4.67	7.07	15.70	8.81	10.92
B	556	12”	5.16	5.31	6.94	15.70	39.75	8.82

Conclusions

Rerouting Tributary 316A to Queen City Lake will affect the peak inflow to the lake as well as the discharge rates from the proposed outlet structure. Compared to the Phase II Technical Information Report design, Queen City Lake inflows will increase from 40.76 cfs to 77.05 cfs. Peak 2-year outflows will increase from 3.38 cfs to 5.31 cfs, and 100-year peak outflows will increase from 7.02 cfs to 39.75 cfs. To account for the higher outflow from the lake, the Stage 2 outlet elevation will be decreased from 455.6 ft to 454.6 ft to prevent the lake level from exceeding 9 ft. The Stage 2 outlet dimensions and both the Stage 1 outlet elevation and diameter remain unchanged.

LANDAU ASSOCIATES, INC.

Meghan Veilleux, EIT
Staff EIT

Katherine Saltanovitz, PE
Associate

MDV/KMS/EFW/jrc
[\\EDMDATA01\PROJECTS\992\002\WIP\T\2017 TIR UPDATE\KCRTS TM (SEC 6.10)\QUEEN CITY LAKE BASIN-KCRTS MODELING.DOCX]

Attachments: Figure 1: Drainage Basins Map
Figure 2: Queen City Lake – Stage Storage, Existing Condition Chart

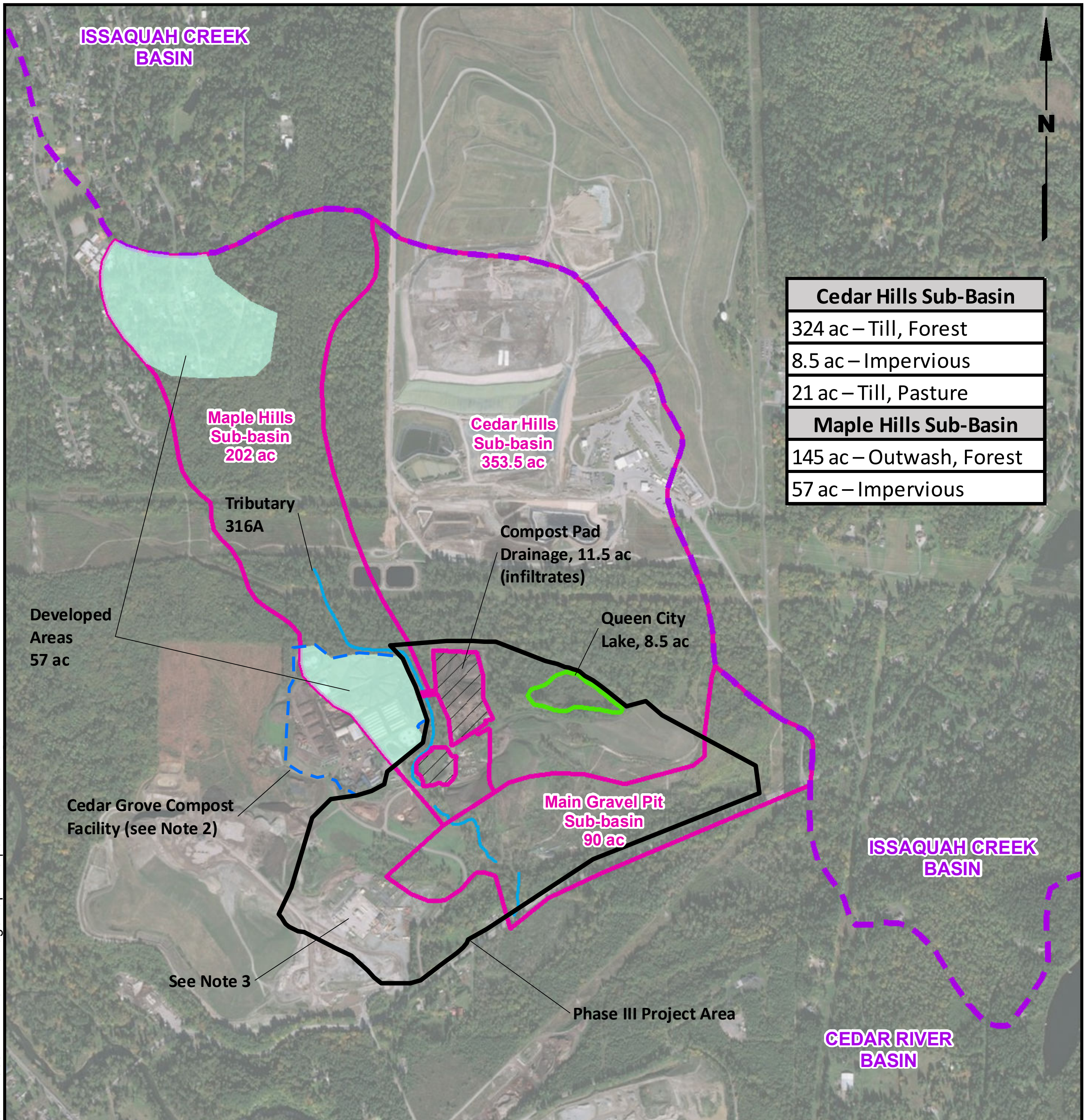
Appendix A: Scenario A, KCRTS Modeling Results
Appendix B: Scenario B, KCRTS Modeling Results

Attachment 1: Cedar Hills Regional Landfill – KCRTS Analysis Final Development Scenario Sub-Basin Boundaries Based on 2000 Topography
Attachment 2: Cedar Hills Regional Landfill 2000 Basin Topo – Queen City Lake
Attachment 3: Queen City Lake – Storage Routing and Water Level, Existing Conditions

References

LAI. 2007. Technical Information Report, Queen City Farms Refill Project, Cedar Grove Road SE, King County, Washington. Landau Associates, Inc. August 13, 2007.

W&H Pacific. 2006. Letter: Queen City Farms - Storage Routing and Water Level Analysis, Phase 3 Outlet Structure Sizing. From Michael Gomez, W&H Pacific, to Brian Butler, Senior Associate Geologist, Landau Associates, Inc. December 13.



Cedar Hills Sub-Basin
324 ac – Till, Forest
8.5 ac – Impervious
21 ac – Till, Pasture
Maple Hills Sub-Basin
145 ac – Outwash, Forest
57 ac – Impervious

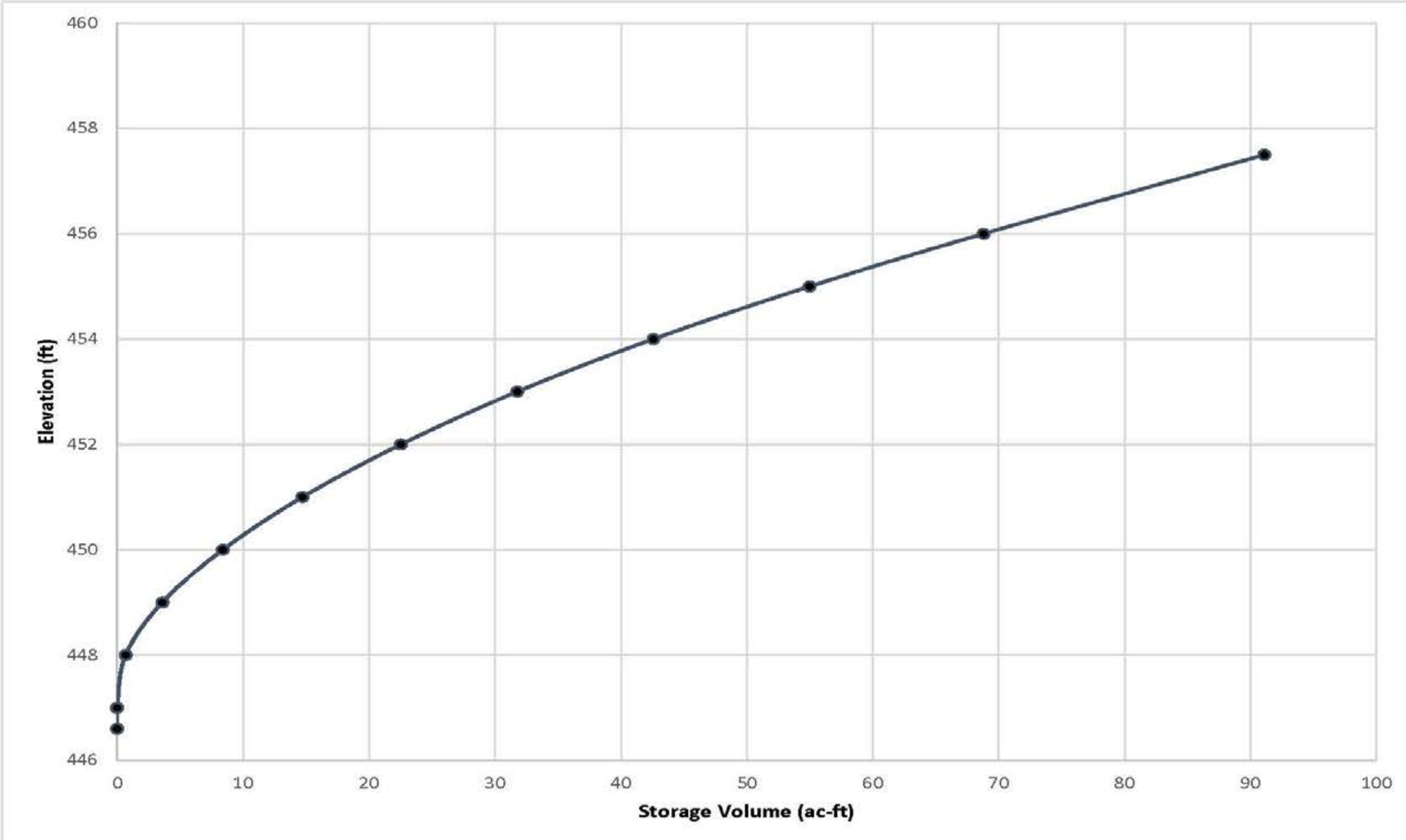
Legend
- Basin Divide
- Sub-basin
- Infiltration

0 1,500 3,000
Scale in Feet

Base Map Source: Terrain Navigator 2002
Data Sources: Smayda 2005; King County 2006

Notes
1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.
2. The entire Cedar Grove Compost Facility drains to a sanitary sewer. The portion of the Facility within the Maple Hills sub-basin has been included in the hydraulic modeling as a conservative measure.
3. Stormwater that falls in the portion of the Phase III Fill outside the Maple Hills and Main Gravel Pit sub-basins flows to the Cedar Shores stormwater ponds.

Landau Associates	Queen City Farms Phase III Refill Maple Valley, Washington	**Drainage Basins Map**	Figure 1



Note: Figure is reprinted from the W&H Pacific technical memorandum "Queen City Farms - Storage Routing and Water Level Analysis Phase 3 Outlet Structure Sizing" dated December 13, 2006.

LANDAU ASSOCIATES

Queen City Farms
Phase III Refill
Maple Valley, Washington

Queen City Lake Stage Storage, Existing Condition Chart

Figure 2

APPENDIX A

Scenario A, KCRTS Modeling Results

APPENDIX A-1

qcl-WHP.RS2

Two Outlet Reservoir Routing File

Stage (Ft)	Discharge (CFS) A	Discharge (CFS) B	Storage (Cu-Ft)	Perm-Area (Sq-Ft)
0.00	0.000	0.000	0.	0.
0.40	0.300	0.000	37.	0.
1.40	0.800	0.000	28866.	0.
2.40	1.200	0.000	155599.	0.
3.40	1.600	0.000	365493.	0.
4.40	1.800	0.000	641240.	0.
5.40	2.000	0.430	981406.	0.
6.40	3.000	3.590	1383296.	0.
7.40	7.000	5.210	1854674.	0.
8.40	15.700	6.440	2395648.	0.
9.40	15.700	7.470	2997786.	0.
10.90	15.700	8.790	3968867.	0.

446.60 Ft : Base Reservoir Elevation
0.0 Minutes/Inch: Average Perm-Rate

APPENDIX A-2

a-infil.pks

Flow Frequency Analysis
Time Series File:a-infil.tsf
Project Location:Landsburg

---Annual Peak Flow Rates--- -----Flow Frequency Analysis-------
Flow Rate Rank
Time of Peak - - Peaks - - Rank Return Prob
(CFS) (CFS) (ft) Period

15.70	2	2/10/01	5:00	15.70	10.92	1	100.00	0.990
2.20	7	1/07/02	2:00	15.70	9.74	2	25.00	0.960
12.77	4	3/01/03	7:00	15.70	8.75	3	10.00	0.900
1.80	8	3/04/04	6:00	12.77	8.06	4	5.00	0.800
9.93	5	1/05/05	20:00	9.93	7.74	5	3.00	0.667
5.67	6	1/19/06	12:00	5.67	7.07	6	2.00	0.500
15.70	3	11/24/06	11:00	2.20	5.60	7	1.30	0.231
15.70	1	1/10/08	6:00	1.80	4.42	8	1.10	0.091
Computed Peaks				15.70	10.53		50.00	0.980

APPENDIX A-3

a-orif.pks

Flow Frequency Analysis
Time Series File:a-orif.tsf
Project Location:Landsburg

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------				
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	(ft)	Rank	Return Period	Prob
7.77	2	2/10/01 5:00	8.81	10.92	1	100.00	0.990
1.06	7	1/07/02 2:00	7.77	9.74	2	25.00	0.960
6.03	4	3/01/03 7:00	6.80	8.75	3	10.00	0.900
0.008	8	3/04/04 6:00	6.03	8.06	4	5.00	0.800
5.62	5	1/05/05 20:00	5.62	7.74	5	3.00	0.667
4.67	6	1/19/06 12:00	4.67	7.07	6	2.00	0.500
6.80	3	11/24/06 11:00	1.06	5.60	7	1.30	0.231
8.81	1	1/10/08 6:00	0.008	4.42	8	1.10	0.091
Computed Peaks			8.46	10.53		50.00	0.980

APPENDIX A-4
Queen City Lake
Scenario A - Stage-Storage-Infiltration-Discharge Results

Bottom of lake 446.6 ft

Elevation (ft)	Area (SF)	Stage (ft)	Total Volume (CF)	Infiltration Rate (CFS)	Discharge (CFS)
446.6	0	0	0	0	0
447	276	0.4	37	0.3	0
448	81,470	1.4	28,866	0.8	0
449	178,228	2.4	155,599	1.2	0
450	243,242	3.4	365,493	1.6	0
451	309,583	4.4	641,240	1.8	0
452	371,695	5.4	981,406	2.0	0.43
453	432,861	6.4	1,383,296	3.0	3.59
454	510,974	7.4	1,854,674	7.0	5.21
455	571,541	8.4	2,395,648	15.7	6.44
456	633,261	9.4	2,997,786	15.7	7.47
457.5	661,618	10.9	3,968,867	15.7	8.79

2-year infiltration rate: 5.67 cfs
2-year peak flow release rate: 4.67 cfs
Stage 1 Elevation (2-year storm): 453.67
7.07 ft

100-year infiltration rate: 15.70 cfs
100-year peak flow release rate: 8.81 cfs
Stage 2 Elevation (100-year storm): 457.52
10.92 ft

100-year peak inflow rate (weir design): 77.05 cfs

APPENDIX A-5

combinedbasin.pks

Flow Frequency Analysis
Time Series File:combinedbasin.tsf
Project Location:Landsburg

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------			
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	Rank	Return Period	Prob
47.84	2	2/09/01 14:00	77.05	1	100.00	0.990
16.75	8	1/05/02 16:00	47.84	2	25.00	0.960
34.49	4	2/28/03 16:00	44.97	3	10.00	0.900
28.84	6	8/26/04 1:00	34.49	4	5.00	0.800
31.87	5	1/05/05 10:00	31.87	5	3.00	0.667
26.91	7	1/18/06 16:00	28.84	6	2.00	0.500
44.97	3	11/21/06 9:00	26.91	7	1.30	0.231
77.05	1	1/09/08 7:00	16.75	8	1.10	0.091
Computed Peaks			67.31		50.00	0.980

APPENDIX B

Scenario B, KCRTS Modeling Results

APPENDIX B-0
Queen City Lake
Scenario B - Orifice Flow Calculation

Bottom of lake 446.6 ft

Stage 1

Orifice dia	12	in
Orifice IE	451.6	ft
Orifice height	5.0	ft

$$Q_{orifice} = C \times A \times \sqrt{2gh}$$

C	0.61	(sharp edge orifice)
A	0.785	SF (area of orifice)
g	32.2	fps

Stage 2

Weir length	12	ft (perimeter of grate)
Riser IE	454.56	ft
Riser (weir) height	7.96	ft

$$Q_{weir} = 3.0 \times L \times H^{1.5}$$

Stage (ft)	Water Height above Stage 1 Control, h (ft)	Orifice Flow, Stage 1 (cfs)	Water height above Stage 2 Control, H (ft)	Weir Flow, Stage 2 (cfs)	Total discharge flow (cfs)
0	0	0	0	0	0
0.4	0	0	0	0	0
1.4	0	0	0	0	0
2.4	0	0	0	0	0
3.4	0	0	0	0	0
4.4	0	0	0	0	0
5.4	0.4	2.43	0	0	2.43
6.4	1.4	4.55	0	0	4.55
7.4	2.4	5.96	0	0	5.96
7.97	3.0	6.63	0.01	0.06	6.69
8.4	3.4	7.09	0.44	10.67	17.76
9.4	4.4	8.06	1.44	62.51	70.57
10.9	5.9	9.34	2.94	181.9	191.24

APPENDIX B-1

qcl-2018-B.RS2

Two Outlet Reservoir Routing File

Stage (Ft)	Discharge (CFS) A	Discharge (CFS) B	Storage (Cu-Ft)	Perm-Area (Sq-Ft)
0.00	0.000	0.000	0.	0.
0.40	0.300	0.000	37.	0.
1.40	0.800	0.000	28866.	0.
2.40	1.200	0.000	155599.	0.
3.40	1.600	0.000	365493.	0.
4.40	1.800	0.000	641240.	0.
5.40	2.000	2.430	981406.	0.
6.40	3.000	4.550	1383296.	0.
7.40	7.000	5.960	1854674.	0.
7.97	7.000	6.690	2163030.	0.
8.40	15.700	17.760	2395648.	0.
9.40	15.700	70.570	2997785.	0.
10.90	15.700	191.240	3968867.	0.

446.60 Ft : Base Reservoir Elevation
0.0 Minutes/Inch: Average Perm-Rate

APPENDIX B-2

b-infil.pks

Flow Frequency Analysis
Time Series File:b-infil.tsf
Project Location:Landsburg

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------				
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	(ft)	Rank	Return Period	Prob
15.70	2	2/09/01 19:00	15.70	8.82	1	100.00	0.990
1.97	7	1/06/02 22:00	15.70	8.56	2	25.00	0.960
9.67	4	3/01/03 7:00	15.52	8.39	3	10.00	0.900
1.80	8	3/04/04 5:00	9.67	8.10	4	5.00	0.800
7.00	5	1/06/05 23:00	7.00	7.85	5	3.00	0.667
5.16	6	1/19/06 11:00	5.16	6.94	6	2.00	0.500
15.52	3	11/24/06 7:00	1.97	5.23	7	1.30	0.231
15.70	1	1/09/08 11:00	1.80	4.42	8	1.10	0.091
Computed Peaks			15.70	8.73		50.00	0.980

APPENDIX B-3

b-orif.pks

Flow Frequency Analysis
Time Series File:b-orif.tsf
Project Location:Landsburg

---Annual Peak Flow Rates---			-----Flow Frequency Analysis-------				
Flow Rate (CFS)	Rank	Time of Peak	- - Peaks - - (CFS)	(ft)	Rank	Return Period	Prob
26.08	2	2/09/01 19:00	39.75	8.82	1	100.00	0.990
2.02	7	1/06/02 22:00	26.08	8.56	2	25.00	0.960
10.09	4	3/01/03 7:00	17.53	8.39	3	10.00	0.900
0.040	8	3/04/04 5:00	10.09	8.10	4	5.00	0.800
6.54	5	1/06/05 23:00	6.54	7.85	5	3.00	0.667
5.31	6	1/19/06 11:00	5.31	6.94	6	2.00	0.500
17.53	3	11/24/06 7:00	2.02	5.23	7	1.30	0.231
39.75	1	1/09/08 11:00	0.040	4.42	8	1.10	0.091
Computed Peaks			35.19	8.73		50.00	0.980

APPENDIX B-4
Queen City Lake
Scenario B - Stage-Storage-Infiltration-Discharge Results

Bottom of lake 446.6 ft

Elevation (ft)	Area (SF)	Stage (ft)	Total Volume (CF)	Infiltration Rate (CFS)	Discharge (CFS)
446.6	0	0	0	0	0
447	276	0.4	37	0.3	0
448	81,470	1.4	28,866	0.8	0
449	178,228	2.4	155,599	1.2	0
450	243,242	3.4	365,493	1.6	0
451	309,583	4.4	641,240	1.8	0
452	371,695	5.4	981,406	2.0	2.43
453	432,861	6.4	1,383,296	3.0	4.55
454	510,974	7.4	1,854,674	7.0	5.96
454.57	545,497	7.97	2,163,030	7.0	6.69
455	571,541	8.4	2,395,648	15.7	17.76
456	633,261	9.4	2,997,786	15.7	70.57
457.5	661,618	10.9	3,968,867	15.7	191.24

2-year infiltration rate: 5.16 cfs
2-year peak flow release rate: 5.31 cfs
Stage 1 Elevation (2-year storm): 453.54
6.94 ft

100-year infiltration rate: 15.70 cfs
100-year peak flow release rate: 39.75 cfs
Stage 2 Elevation (100-year storm): 455.42
8.82 ft

100-year peak inflow rate (weir design): 77.05 cfs

ATTACHMENT 1

Cedar Hills Regional Landfill – KCRTS Analysis Final Development Scenario Sub-Basin Boundaries Based on 2000 Topography

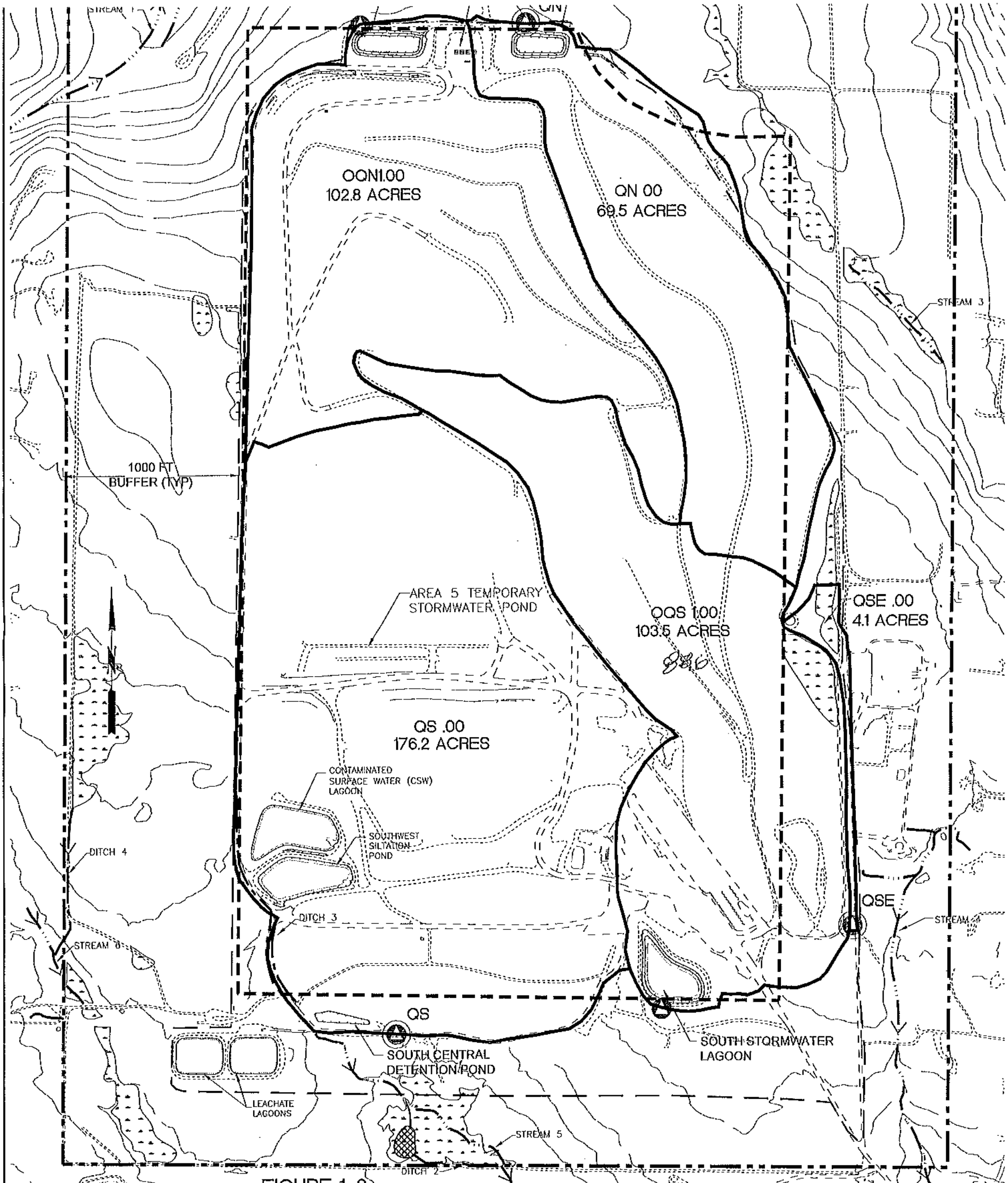


FIGURE 1-2

CEDAR HILLS REGIONAL LANDFILL

KCRTS ANALYSIS FINAL DEVELOPMENT SCENARIO

SUB-BASIN BOUNDARIES BASED ON 2000 TOPOGRAPHY

BUFFER EDGE

PROPERTY LINE

STUDY AREA

Approximate location for discharge point
See Appendix "B" for exact locations

Designated discharge point

NOTE:
AS THE LANDFILL IS CLOSED OUT (FINAL COVER ANTICIPATED 2013), SUB-BASIN BOUNDARIES MAY CHANGE SLIGHTLY. THE EFFECT WILL BE TO RE-ALLOCATE FLOWS BETWEEN SUB-BASINS. TOTAL DISCHARGE WILL NOT BE AFFECTED.

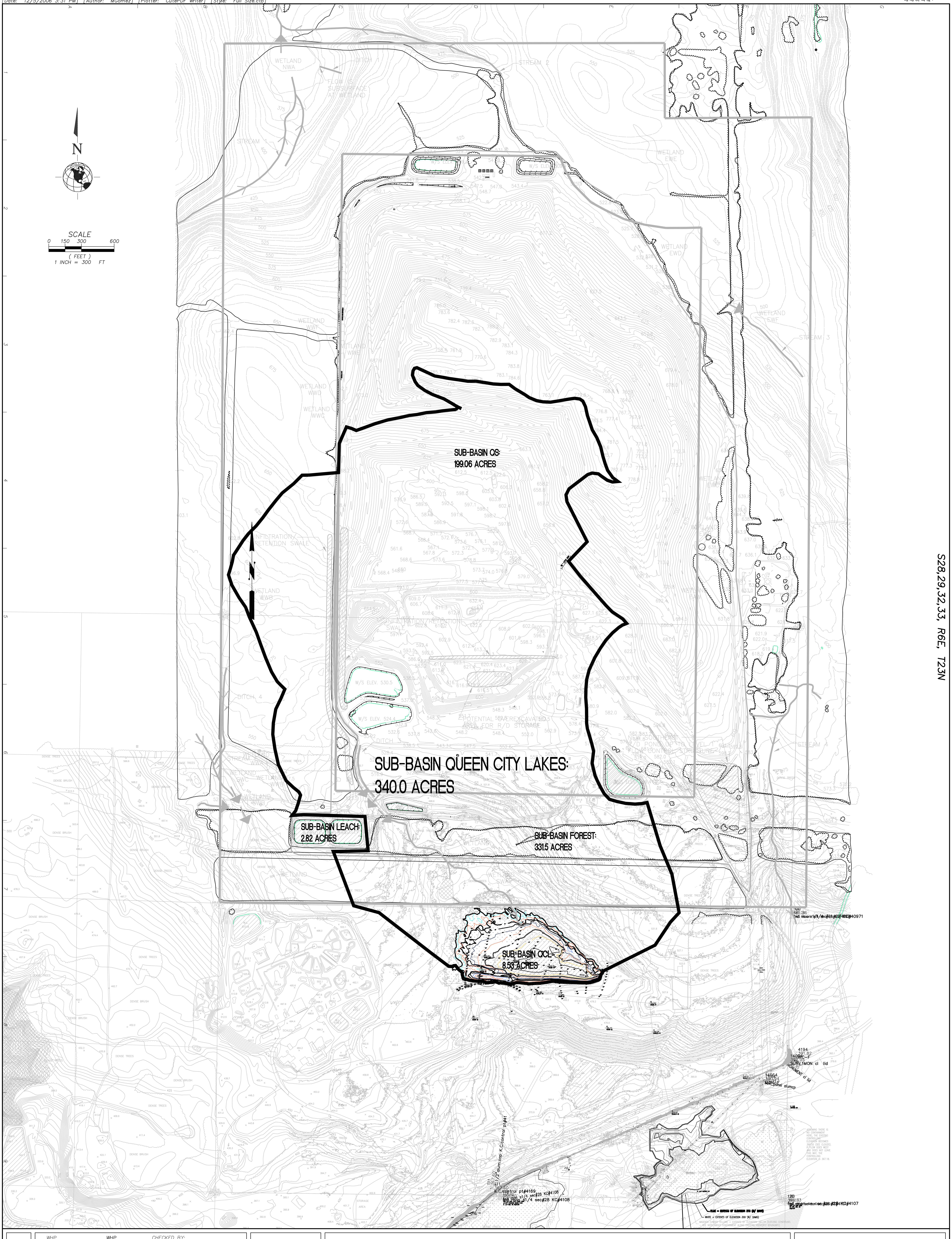
Department of Natural Resources and Parks

300 0 300 600 900
SCALE IN FEET

ATTACHMENT 2

Cedar Hills Regional Landfill 2000 Basin Topo – Queen City Lake

[Path: P:\Landau Associates\035144\Design\Drawings\Civil\035144-QCL-FIG-I1.dwg] [CHRL-2K-USE]
[Date: 12/5/2006 3:31 PM] [Author: MGomez] [Plotter: CutePDF Writer] [Style: Full Size.ctb]



WHP	WHP	CHECKED BY:	
DRAWN BY:	MAG	APPROVED BY:	
LAST EDIT:	12/5/2006	PLOT DATE:	12/05/06

DATE	BY	REV#	REVISION	CK'D	APPR.

SHEET FIG I1

QUEEN CITY FARMS
CHRL 2000 BASIN TOPO
QUEEN CITY LAKE

KING COUNTY WA

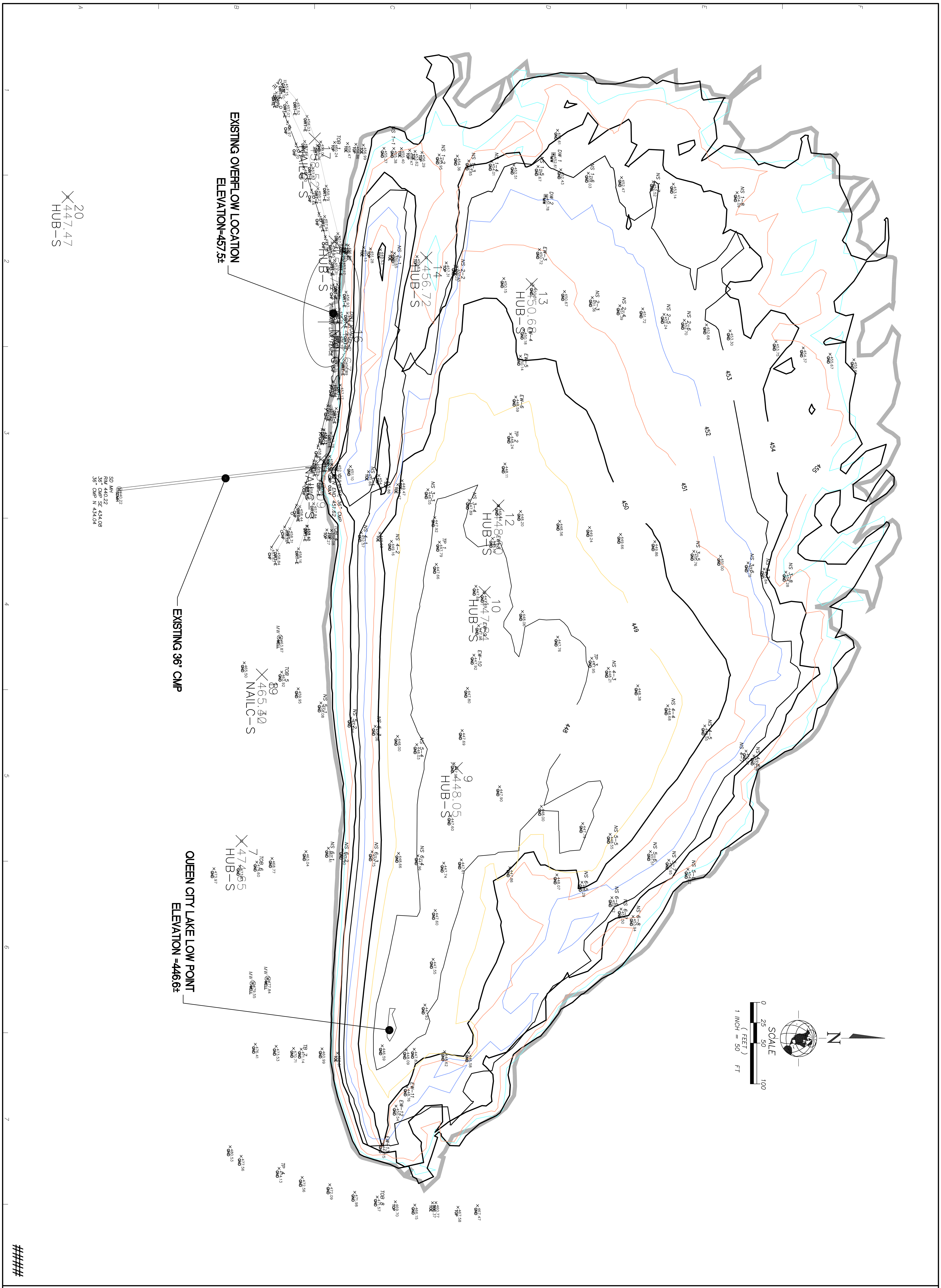
SCALE: 1"=300'
PROJECT NO. 035144
DRAWING FILE NAME: 035144-QCL-FIG-I1

W&H PACIFIC
3350 Monte Villa Parkway
Bothell, Washington 98021-8972
(425)951-4800
(425)951-4808
whpacific.com
Planners • Engineers • Surveyors • Landscape Architects

ATTACHMENT 3

Queen City Lake – Storage Routing and Water Level, Existing Conditions

P:\Landau Associates\035144\Design\Drawings\Civil\035144-QCL-FIG-A1.dwg, FIG A1, 12/5/2006 3:21:27 PM, MGomez



WHP	MG/SS	CHECKED BY:	MG/SS
DRAWN BY:	MAG	APPROVED BY:	MAG
LAST EDIT:	12/4/2006	PLOT DATE:	12/04/06

DATE	BY	REV#	REVISION	CK'D	APPR

PRELIMINARY

EXPIRES 02/20/07

QUEEN CITY LAKE

STORAGE ROUTING AND WATER LEVEL EXISTING CONDITIONS

KING COUNTY WA

SCALE:

PROJECT NO. 035144

DRAWING FILE NAME: 035144-QCL-FIG-A1

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Planners • Engineers • Surveyors • Landscape Architects

SHEET 1/1

FIG A1

TECHNICAL MEMORANDUM

TO: Mr. Alan Wallace, Williams Kastner and Gibbs

FROM: Eric Weber, L.G., Project Manager

DATE: January 29, 2007

**RE: GEOLOGY, SURFACE WATER HYDROLOGY AND GROUNDWATER HYDROLOGY
QUEEN CITY FARMS REFILL PROJECT
MAPLE VALLEY, WASHINGTON**

This technical memorandum presents a summary of geologic, surface water, and groundwater conditions in the vicinity of the Queen City Farms (QCF) Refill Project located in Maple Valley, Washington (Figure 1). A thorough understanding of the geologic, surface water, and groundwater conditions is necessary to support stormwater management design, identification and mitigation of issues, and geotechnical engineering recommendations associated with the QCF Refill Project. This summary of site geology and hydrology is based on the results of previous hydrologic and geologic studies of the QCF property and adjacent properties. This summary is included in the Technical Information Report (TIR) and includes information applicable to elements of Sections 3 and 4 of the TIR.

SITE CONDITIONS AND PROJECT DESCRIPTION

The QCF property currently consists of a large pit (QCF gravel pit), which was formerly a sand and gravel mine. The QCF Refill Project consists of reclaiming the mine by refilling the gravel pit with clean fill material. The refill plan indicates that up to 115 ft of fill will be placed in the gravel pit to achieve final grades. Currently, the QCF gravel pit is occupied by a seasonal lake, referred to as the Main Gravel Pit Lake. Several other surface water features also exist on the property. These include Queen City Lake located north of the Main Gravel Pit Lake, East Airstrip Spring, and Queen City Farms Spring. Surface water features are shown on Figure 2 and discussed in more detail in Section 4.0 of this TIR.

A portion of the QCF property is also a hazardous waste cleanup site. Cleanup of this site is managed by The Boeing Company (Boeing) with oversight by the Environmental Protection Agency (EPA). As part of this cleanup, numerous wells have been installed on the site and extensive hydrogeological evaluation has been performed (Landau Associates 1990, 1991, 2011; EcoChem and Boeing 2006). Two primary elements of the remedy are a slurry wall, which isolates contaminated portions of Aquifer 1, and monitored natural attenuation of a groundwater contamination plume in Aquifer 2 (Aquifers 1 and 2 are discussed later in more detail later in this memorandum). The Cedar Hills Landfill (CHRL) is located north of the property, as shown on Figure 2.

The key concerns for reclamation of the QCF gravel pit are stormwater management, potential effects on wetland conditions near Queen City Lake, maintaining the integrity of the hazardous waste cleanup site remedies, and fill stability. An understanding of the site geology and hydrology is important to designing a reclamation plan that will minimize these potential impacts.

SITE INVESTIGATIONS

Previous hydrologic and geologic studies of QCF property were performed by Landau Associates and others. A summary of these previous studies is provided below. Additional information is presented in the *Queen City Farms Remedial Investigation Report* (Landau Associates 1990), the *Queen City Farms Supplemental Remedial Investigation Report* (Landau Associates 1991), *Queen City Farms 2005 Annual Monitoring Data Report* (EcoChem and Boeing 2006) and the. These reports are included as Attachment 1 to this memorandum (pdf format) on compact disk.

Between approximately 1986 and 2000, numerous wells and test pits were installed at the QCF site and adjoining CHRL to investigate geologic and groundwater conditions. Well locations are shown on Figure 3. Boring logs and water level readings associated with these wells are presented in the reports in Attachment 1. Numerous test pits were completed to evaluate the hydrology of the Main Gravel Pit Lake (Landau Associates 1993). These test pits are presented in the *Main Gravel Pit Lake Surface Soil Mapping Report* included in Attachment 1.

GEOLOGY

The QCF property lies within the general physical provenance known as the Coalfield Drift Plain, which is an upland area that extends from the Cedar River Valley to the Cascade Range. The QCF property and CHRL to the north represent an area of the drift plain that is further segmented on all sides by valleys and troughs. The general physical geography in the vicinity of QCF property is shown on Figure 4. The general soil types described in the King County Soil Survey for the QCF property is Alderwood and Everett gravelly sandy loam.

The uplands drift plain is mantled by glacial till that has relatively low permeability. In the valley and trough areas, the till is generally missing. Beneath the glacial till is a series of glacial and interglacial layers. Twelve separate stratigraphic units have been identified on the QCF property. These layers are summarized on Figure 5. The general occurrence of the till is shown on the surficial geologic map on Figure 6.

The geology at the site consists of glacial till (Units D and Dr) at the surface that pinches out towards Cedar Grove channel. In the vicinity of the channel, the till is overlain by coarse grain gravel recessional glacial outwash deposits (Unit C) and underlain by coarse grain advance glacial outwash gravel deposits (Unit E). Where the till is missing, Units C and E form a continuous sequence of coarse grain gravel deposits that were extensively mined at the site. Queen City Lake is a kettle lake, a common glacial lake feature in the Puget Sound region. Kettle lakes form when a remnant ice block left behind by the retreating glacier melts and leaves a depression of moderate- to low-permeability ice-contact deposits (Unit B). Units F through J represent interglacial deposits consisting of varying layers of fine to medium sand and silt. Unit U is a thick clay sequence that underlies the southeastern portion of the QCF site and extends into the Issaquah Creek basin.

Stratigraphic information at QCF has been summarized into five cross sections. Cross section locations are shown on Figure 7. Cross sections are shown on Figures 8 through 12.

SURFACE WATER HYDROLOGY

The main surface water feature on the site is Queen City Lake, located on the north portion of the property. Review of aerial photographs since 1936 and United States Geological Survey (USGS) maps shows no historic surface water outlet from Queen City Lake (i.e., all the water in the lake infiltrated into the geologic materials).

A seasonal lake also exists in the QCF gravel pit. In the late 1980s, mining formed a depression in the gravel pit; the lake that formed in the depression became known as the Main Gravel Pit Lake. Prior to mining, there was no seasonal lake. This man-made lake represents the surface expression of the regional aquifer (i.e., Aquifer 2); in the summer the lake goes dry. The planned reclamation will eliminate the seasonal Main Gravel Pit Lake.

The mining also exposed a set of springs in the north face of the gravel pit, known as the East Airstrip Springs or springs SP-4(a, b, c, d). These springs represent discharge from the perched aquifer (i.e., Aquifer 1). The discharge from these springs flowed down the face of the gravel pit into the Main Gravel Pit Lake at rates as high as 1,000 gpm (2.2 cfs).

Also as part of mining activities, Cedar River Tributary 316A was defined on the site. This tributary flows along the east side of Cedar Grove Composting, down the hillside, and discharges to the Main Infiltration Area (Figure 2). Prior to gravel mining, this tributary infiltrated into permeable soil prior to reaching the Main Infiltration Area. The Main Infiltration Area is a natural recharge area that is capable of infiltrating very high volumetric rates. In addition to Tributary 316A, high stormwater flow rates from Stoneway's Cedar Shores Pit have historically been directed to the Main Infiltration Area.

There are three drainage sub-basins that together cover most of the proposed QCF Refill Project site: the Queen City Lake, Main Gravel Pit Lake, and Maple Hills Sub-basins. These sub-basins are most important for the purposes of design and implementation of the refill plan. The Queen City Lake drainage sub-basin consists primarily of the southern portion of the CHRL and Queen City Lake. The Main Gravel

Pit Lake drainage sub-basin consists of areas that discharge directly to the lake. The Maple Hills drainage sub-basin consists of areas that flow into Tributary 316A and eventually discharge to the Main Infiltration Area. All three of these drainage sub-basins are considered tributary to the Cedar River. Groundwater that infiltrates onsite discharges directly to the Cedar River via Tributary 316A or discharges to Issaquah Creek via Mason Creek. In particular, surface water that infiltrates at the Main Infiltration Area appears to discharge predominantly at the Queen City Farms Spring. The spring discharge flows into a culvert that directs the discharge into the wetland south of Cedar Grove Road SE. The approximate locations of these three drainage sub-basins are shown on Figure 13.

Queen City Lake Sub-Basin

The Queen City Lake sub-basin is approximately 370 acres; approximately 280 of these acres are CHRL active or closed landfill areas. The remaining acreage is landfill buffer areas and wooded and wetland portions of the QCF property. The actual extent and attributes of this sub-basin have continually changed since the 1960s due to landfilling on the south end of the landfill. Consequently, stormwater runoff patterns have changed. King County Solid Waste Division (KCSWD) has estimated stormwater runoff from the south end of the landfill at various time periods. They have subdivided the sub-basin into a primary basin (QS) of about 176 acres and a smaller basin (QOS) of about 103 acres (King County 2005). Figure 14 shows attributes of the landfill fill areas and ponds. Figure 15 shows basin boundaries as of 2005.

Prior to landfill development, the modeled 100-yr stormwater runoff event produced a combined 25.4 cfs of runoff from areas QOS and QS (KCSWD 2005). Based on 1979 land use conditions, modeled runoff had increased to 75.4 cfs (65.2 cfs from QS and 10.2 cfs from QOS); based on year 2000 conditions, modeled 100-yr runoff was estimated at a combined 113.8 cfs (KCSWD 2005). Capital projects underway on the south end of the landfill are predicted to eliminate 100-yr peak discharge from the QOS basin to QCFs by about 2008. This will be accomplished by increasing detention storage and routing approximately 1.8 cfs to an infiltration channel along Cedar Grove Road SE (King County Department of Transportation 2006). It is also the stated intent of KCSWD to return stormwater runoff conditions to 1979 conditions after closure of the landfilling in the QS basin by about 2015. Modeling completed by Landau Associates (Landau Associates 1990) estimated monthly stormwater runoff in the basin from January 1987 to August 1989. The maximum monthly runoff volume was modeled at 7,303,000 ft^3 for March 1987. This volume corresponds to an average flow of 2.3 cfs for that month. The average monthly winter time runoff (November through April) was less than 2 cfs.

Queen City Lake is dry from early August to November. The lake fills due to winter precipitation and stormwater run-on. Historically, the lake has fluctuated over a 9-ft depth range as storm water flow fills the lake and water infiltrates out of the lake into the underlying Aquifer 1. In February 1991, a 36-inch pipe was installed from Queen City Lake to a ravine that discharged to the Main Gravel Pit Lake. The pipe was installed as an emergency erosion control device. The pipe effectively limited the fluctuations in the lake to approximately 5.5 ft. Prior to installation of the outflow pipe, the average winter time water level in the lake was about Elevation 438.5 ft NGVD 29; after outflow pipe installation the average winter-time water level dropped to about Elevation 435 ft. Annual hydrographs of water levels in Queen City Lake are presented for years 1987 through 1993 in Attachment 2. Aquifer 1 water level data from 1987 to 1993 are presented in Attachment 3.

Main Gravel Pit Lake Sub-Basin

The Main Gravel Pit Lake sub-basin is about 80 acres and contains the Main Gravel Pit Lake; the lake is not natural, but formed due to gravel mining activities. Discharge to the lake is from surface water runoff, discharge from the outflow pipe from Queen City Lake, and discharge from the East Airstrip Spring area. There is no outlet to the lake; all water in the lake infiltrates directly to Aquifer 2. The lake goes dry in about June and refills again in the fall with the onset of winter rain. The bottom of the lake is about Elevation 367 ft, NAVD88 (363.5 NGVD 29). Based on water level elevation readings in Aquifer 2 wells, the Main Gravel Pit Lake is the surface expression of Aquifer 2. A staff gauge installed in the lake was read 5 days a week during the winter and spring of 1991. Water levels in the lake rose quickly in response to storm events and quickly dissipated. The maximum depth of the lake was about 8.5 ft.[1] A hydrograph of water levels in the Main Gravel Pit Lake is shown on Figure 16. A cross section comparing Aquifer 2 water levels with the Main Gravel Pit Lake water levels during 1989 is shown on Figure 17. On about January 1, 1997, there was an extreme rain-on-snow event in the QCF area. During this period of time, the Main Gravel Pit Lake reportably filled to a depth of about 18 ft (about Elevation 391 ft, NAVD 88).

The East Airstrip Spring represented a significant surface water feature in the Main Gravel Pit Lake sub-basin. The springs represent discharge from perched Aquifer 1 and were formed when gravel mining exposed the Aquifer 1 clayey-silt layer aquitard. The springs were first observed on June 12, 1988. By November 1988, as mining and erosion continued in this area, three localized springs were observed; the maximum winter-time flow from these springs was initially estimated at about 160 gpm. On April 5, 1989, 1.25 inches of precipitation were recorded at CHRL. This storm event followed approximately 2 weeks of relatively steady rain and produced extreme runoff conditions in Queen City Lake sub-basin. Note that during this approximate time period, the final cover was being installed on the

[1] Later surveying of the high water mark on the lake indicated the maximum depth was about 10 ft.

South Solid Waste Area (Figure 14) and limited detention was in place to capture peak flows.[2] The April 5, 1989 storm event led to an increase in flow at the springs to about 1,000 gpm coincident with headward erosion toward the South Shore Gravel Pit (the surface expression of Aquifer 1) until only a small strip of land remained between the gravel pit and the springs (Landau Associates 1990). The configuration of the springs after this erosion event is shown on Figures 18 and 19. Initial erosion control measures were implemented in 1989 and were subsequently compromised by storm events. A final erosion control measure was implemented in February 1991 to stabilize the East Airstrip Spring and South Shore Gravel Pit areas to reduce the impact of future storm events (Landau Associates 1991). The erosion control project involved:

- Installing a 36-inch diameter culvert to divert Queen City Lake water directly to the Main Gravel Pit Lake (inlet invert elevation at 451.5 NAVD 88; outlet invert elevation about 410 ft NAVD 88). A manhole was installed with the pipe. Installation of the manhole breached the Aquifer 1 aquitard.

- Filling the South Shore Gravel Pit by grading surrounding sand and gravel into the area.

- Installing two permanent approximately 8-inch diameter drain pipes to control seepage from Aquifer 1 along the trench that carries the 36-inch culvert. Outflow from these pipes now represents the primary discharge from the spring. These outflow pipes are labeled EC-1 and EC-2.

- Filling and grading the East Airstrip Spring area.

A summary of the erosion control project is presented on Figure 20.

The East Airstrip Spring flow rate was periodically monitored during the period between its formation in June 1988 until construction of the Queen City Lake outflow in February 1991. The typical flow from the springs was observed to be at a combined rate of about 300 gpm (Landau Associates 1990). This changed in April 1989 with severe erosion in this area represented by conditions shown on Figures 18 and 19. During this period, maximum flow rates were estimated at 1,000 gpm. Subsequent grading that eliminated the South Shore Gravel Pit and the installation of the Queen City Lake outflow greatly reduced spring flow in this area. Flow rates have not been formally recorded since then; however, they are likely less than 200 gpm. During Phase 3 of the refill plan, the Queen City Lake outflow will be modified. After this modification, maximum flow rates at East Airstrip Spring will likely increase. The estimated future maximum flow rate is estimated to be 500 gpm, significantly less than historical maximum flow rates but somewhat more than the flow that was observed prior to April 1989. This future maximum rate is based on the following factors:

- The presence of the South Shore Gravel Pit adjacent to the eroded head of the spring (Figure 19) resulted in very high gradients at the spring. Backfilling at the pit and the head of

[2] The Southwest Siltation Pond was not constructed until 1990. The Leachate Lagoons were expanded in size in 1989 along with the capacity of the forcemain to the wastewater treatment plant.

the springs accomplished in 1991 resulted in lower gradients at the spring and, consequently, reduced spring flows.

- Stormwater management practices at CHRL have improved since 1989 resulting in lower runoff rates in Queen City Lake sub-basin compared to 1989.

- A modified outflow structure will discharge up to 2 cfs from Queen City Lake directly to an infiltration pond that recharges to Aquifer 2. This will reduce recharge to Aquifer 1 (compared to 1989) and subsequently reduce water levels in Aquifer 1. Lower water levels in Aquifer 1 will result in lower flow rates at the springs.

Maple Hills Sub-Basin

The Maple Hills sub-basin is about 175 acres. The sub-basin discharges to Tributary 316A, which flows south across the QCF property. Tributary 316A discharges to the Main Infiltration Area. Historically, the Main Infiltration Area also has received appreciable discharge from the Stoneway Cedar Shores gravel operation via a culvert under the Cedar Grove Composting access road and a ditch that ran from the culvert to the Main Infiltration Area. The source of this water was at one time the large Cedar Shores settling pond located in the southwest corner of the QCF property, as shown on Figure 2. This pond collected runoff from an approximately 70 acre portion of the Cedar Shores Pit. The current reclamation plan for the Cedar Shores Pit indicates that runoff from a relatively small 16.5-acre basin will be directed to the Main Infiltration Area via a stormwater detention pond. Discharge to the Main Infiltration Area has not been measured or calculated. However, there have been no reports of Main Infiltration Area overflowing. Due to reclamation at Cedar Shores, it is estimated that discharge to the Main Infiltration Area will be reduced from historical levels.

GROUNDWATER HYDROLOGY

A detailed conceptual hydrogeologic model has been developed for the QCF property. The main elements of this model are an upper perched aquifer (Aquifer 1) separated from a deeper aquifer sequence (Aquifers 2 and 3 and a deep water bearing zone) by a thick unsaturated zone. This conceptual model is shown schematically on Figure 21 and explicitly on Figure 22.

Aquifer 1

Aquifer 1 is a small, highly permeable aquifer that includes openwork gravels deposits. The estimated extent of Aquifer 1 is shown on Figure 23. Recharge to Aquifer 1 is primarily from leakage from Queen City Lake and direct recharge of surface water runoff. Maximum monthly recharge to Aquifer 1 was estimated by Landau Associates at 2.3 cfs for the period 1987 and 1988 (Figure 24). Discharge from Aquifer 1 is through spring flow (primarily the East Airstrip Spring) and leakage through the Aquifer 1 aquitard. The Aquifer 1 aquitard is considered to be very leaky (Landau Associates 1990). The conceptual model of recharge and discharge to Aquifer 1 is shown graphically on Figure 25.

During storm events and high rates of surface water runoff in Queen City Lake sub-basin, Aquifer 1 experiences high rates of recharge due to its shallow water table and permeable soils. The high recharge rates cause groundwater levels to rise sharply. However, because of the leaky nature of the Aquifer 1 aquitard, groundwater levels also fall quickly. Historically, water levels have fluctuated over 20 ft due to fluctuations in recharge and discharge. Installation of the Queen City Lake outflow in February 1991 reduced recharge to Aquifer 1 and subsequently reduced groundwater level fluctuations. Water levels in Aquifer 1 and water depth in Queen City Lake are shown on Figure 26 for 1988 (prior to installation of the Queen City Lake outflow). Aquifer 1 hydrographs from 1987 to 1993 are presented in Attachment 3.

A slurry wall was installed around contaminated soil in and above Aquifer 1 to isolate the contamination. A low permeability cap that extended out to the slurry wall was placed over the soil and keyed into the slurry wall. The slurry wall was constructed in 1996 with the purpose of isolating contaminated soil from coming into contact with groundwater. Installation of the slurry wall caused groundwater levels within the slurry wall to decline to the bottom of the Aquifer 1 aquitard. Groundwater levels in Aquifer 1 outside the slurry wall did not appear to be affected by slurry wall installation. The approximate extent of the slurry wall is shown on Figure 3 (refer to the technical memorandum in Section 6.7 of this TIR).

Aquifer 2

Aquifer 2 is part of a regional aquifer that includes Aquifer 3 and the Deep Water Bearing Zone (see Figures 21 and 22). This aquifer sequence consists of very fine to medium sand with stratified silt or silty zones. Aquifer 2, Aquifer 3 and the Deep Water Bearing Zone are separated from each other by relatively continuous silt layers identified as aquitards (Landau Associates 1990). While the lower portion of Aquifer 2 consists of silty sand of geologic Unit F (Figure 5), the upper portion of the aquifer consists of up to 15 ft of the basal section of Unit E. This portion of Unit E generally consists of sandy gravel and silty gravel that has higher permeability than Unit F sand.

Leakage from Aquifer 1 recharges Aquifer 2 through the lower unsaturated zone, which has an average thickness of 55 ft (Landau Associates 1990). Aquifer 2 is also recharged south of Queen City Lake by discharge from the Main Gravel Pit Lake, infiltration of precipitation and vertical flow from the wetlands in Cedar Grove Channel south of Cedar Grove Road SE. Historically, recharge to Aquifer 2 caused a groundwater mound directly beneath Aquifer 1 where recharge was highest. Creation of the Main Gravel Pit Lake appeared to cause the mound to shift slightly southward and westward. The highest groundwater levels in Aquifer 2 typically occur at Well F(2) directly north of the Main Gravel Pit Lake and Well S(2) directly west of the lake. Note that wells in Aquifer 2 have generally been installed both at the bottom of Aquifer 2 in Unit F [i.e., Well F(2)] and near the top of Aquifer 2 in Unit E or the top of Unit F [i.e., Well F(2a)]. Water level contours in Aquifer 2 and the top of Aquifer 2 are shown on Figures 27 and 28, respectively, for the time period April 1997 (when water levels are typically high in Aquifer 2).

In both Aquifer 2 and upper Aquifer 2, the location of the groundwater mound is offset north and/or west of the Main Gravel Pit Lake. The offset of the mound is likely caused due to the following:

- Even with the presence of the groundwater mound, there are high recharge rates present north of the lake due to leakage from Aquifer 1 and Queen City Lake.

- The hydraulic continuity of the Main Gravel Pit Lake and Aquifer 2 is restricted directly beneath the lake due to the application of fine grain gravel spoils to the lake bottom. Also, the permeable portion of Aquifer 2 (Unit E) has been mined away beneath the lake (Figure 11).

- A significant portion of the discharge from the Main Gravel Pit Lake appears to occur laterally through coarse gravel deposits along the gravel pit face.

- Recharge to Aquifer 2 in the channel south of Cedar Grove Road SE is limited by lower permeability ice contact deposits (geologic Unit B) and shallower silt layers (Figure 10).

Aquifer 2 groundwater flows laterally from the location of the mound to the north, south, and west. Northward groundwater flow is beneath the CHRL to Mason Creek.[3] Southward groundwater flow is beneath Cedar Grove Channel toward the Cedar River. Westward groundwater flow is toward Tributary 316A and the Cedar River. There appears to be a stronger westward component of flow in upper Aquifer 2 (Figure 28) compared to Aquifer 2 (Figure 27). Note that there is no significant groundwater flow toward the east and Issaquah Creek. This is because Aquifer 2 pinches out to the east due to the presence of thick silt and clay deposits that occupy much of the Issaquah Creek valley (see Figure 6). These silt and clay deposits were encountered in a number of borings drilled on the eastern portion of the facility [i.e., Wells K(2), G(2) and O(2); Figures 8 and 10]. Note that surface water infiltration that takes place in the area between the Main Gravel Pit Lake and well K(2) would be within the Issaquah Creek Basin based on surface topography (see Figure 13). However, groundwater in this area clearly does not flow eastward toward Issaquah Creek due to the absence of Aquifer 2 in that area.

Groundwater levels in Aquifer 2 fluctuate up to 10 ft seasonally in the vicinity of the mound. Also, the vertical hydraulic gradient of Aquifer 2 is relatively small. This is clear from the relationship between water levels from wells screened at the top of Aquifer 2 [i.e., Well E(2a)] and the bottom of Aquifer 2 [i.e., Well E(2)] as shown on the hydrographs on Figure 29. Groundwater levels in wells

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[3] Evaluations by King County (Aspect Consulting 2005) show a consistent northerly groundwater gradient beneath CHRL in the uppermost regional aquifer.

located in Cedar Grove Channel only fluctuate about 4 ft at the bottom of Aquifer 2 [i.e., Well O(2)] and about 8 ft near the top of the aquifer [i.e., Well O(2a)]. Additionally, there is a much stronger vertical gradient in Aquifer 2 in the vicinity of the channel as shown on the hydrographs on Figure 30. The stronger Aquifer 2 vertical gradient in the channel is likely due to the presence of layers of lower permeability soil in the aquifer in this location. These lower permeability soils include ice contact deposits observed in boreholes drilled in the channel.

Historical groundwater level monitoring indicates that Aquifer 2 has a high capacity to assimilate infiltration. This is because of the relatively high permeable soils that make up the upper portions of Aquifer 2 in the area beneath upland drift plain. This area, north of the current north slope of the gravel pit, contains highly permeable sand and gravel associated with geologic units C and E. For example, during periods of high recharge to the Main Gravel Pit Lake, no flooding or adverse impacts were recorded in Cedar Grove Channel or in the ditches along Cedar Grove Road SE.

In February 1991, the outflow pipe was constructed from Queen City Lake to the Main Gravel Pit Lake. On February 19, 1991, the cofferdam in Queen City Lake was released (Landau Associates 1991) and water flowed through the 36-inch outflow pipe (the pipe was full at the inlet). Water levels were subsequently measured in the Main Gravel Pit Lake. Based on the rate of increase in the lake water level, the increase in volume in the Main Gravel Pit Lake over the first 8 hours after the cofferdam was removed was equivalent to a recharge rate of 49 cfs. This lake recharge calculation did not account for discharge flowing out of the lake and infiltrating into the surrounding soil. The lake level rose for 32 hours after opening of the cofferdam, reaching a maximum depth of about 10 ft (Elevation 372 ft, NGVD 29) (Figure 16)[4]. During this time period, the average recharge rate was estimated at 19.1 cfs. Once again, this is a minimum recharge rate because it did not account for water flowing out of the lake into the surrounding soil. During this time period, continuous water level monitoring was conducted at Wells E(2a) located just north of the gravel pit and I(2a) located just south of the Main Gravel Pit Lake (Figure 3; Landau Associates 1991). During this time period, the maximum water level at Well E(2a) was about Elevation 365 ft (NGVD 29); the maximum water level at Well I(2a) rose to about Elevation 360 ft. In other words, water levels were still appreciably higher north of the gravel pit than south of the gravel pit. This relationship is consistent with water level data presented for the period 1997 to 2005 (EcoChem and Boeing 2006). During February 1991, there was no documentation of flooding in Cedar Grove Channel or along Cedar Grove Road SE south of the Queen City Farms property.

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[4] Note that the water level data presented on Figure 16 is based on multiple hand-held water levels. The maximum water level based on water level monitoring was about 371.25 ft NGVD 29. The maximum water level based on the high water mark was about 372 ft NGVD 29.

In early January 1997, an equivalent to a 100-yr stormwater runoff event occurred associated with a rain-on-snow event. During this time period, the Main Gravel Pit Lake apparently reached a depth of almost 18 ft (equivalent to approximately Elevation 380 ft NGVD 29). Unfortunately, no groundwater monitoring was conducted at this time; however, there was no reported flooding in Cedar Grove Channel. This anecdotal information is consistent with the ability of Aquifer 2 to assimilate high levels of recharge.

CONCLUSION

The geologic and hydrologic conditions of the site are major factors in the final design of the refill plan. Existing surface water bodies (i.e., Queen City Lake and the Main Gravel Pit Lake) are major elements in current stormwater management at the property. For example, the Main Gravel Pit Lake stormwater storage and infiltration function will no longer exist following completion of the planned reclamation. Infiltration of stormwater to the underlying groundwater aquifers is also a concern to maintain the cleanup remedies at the hazardous waste cleanup site located on the property. This understanding of geologic and hydrologic conditions provides the basis for the evaluations and recommendations described in other technical memorandums included in the TIR.

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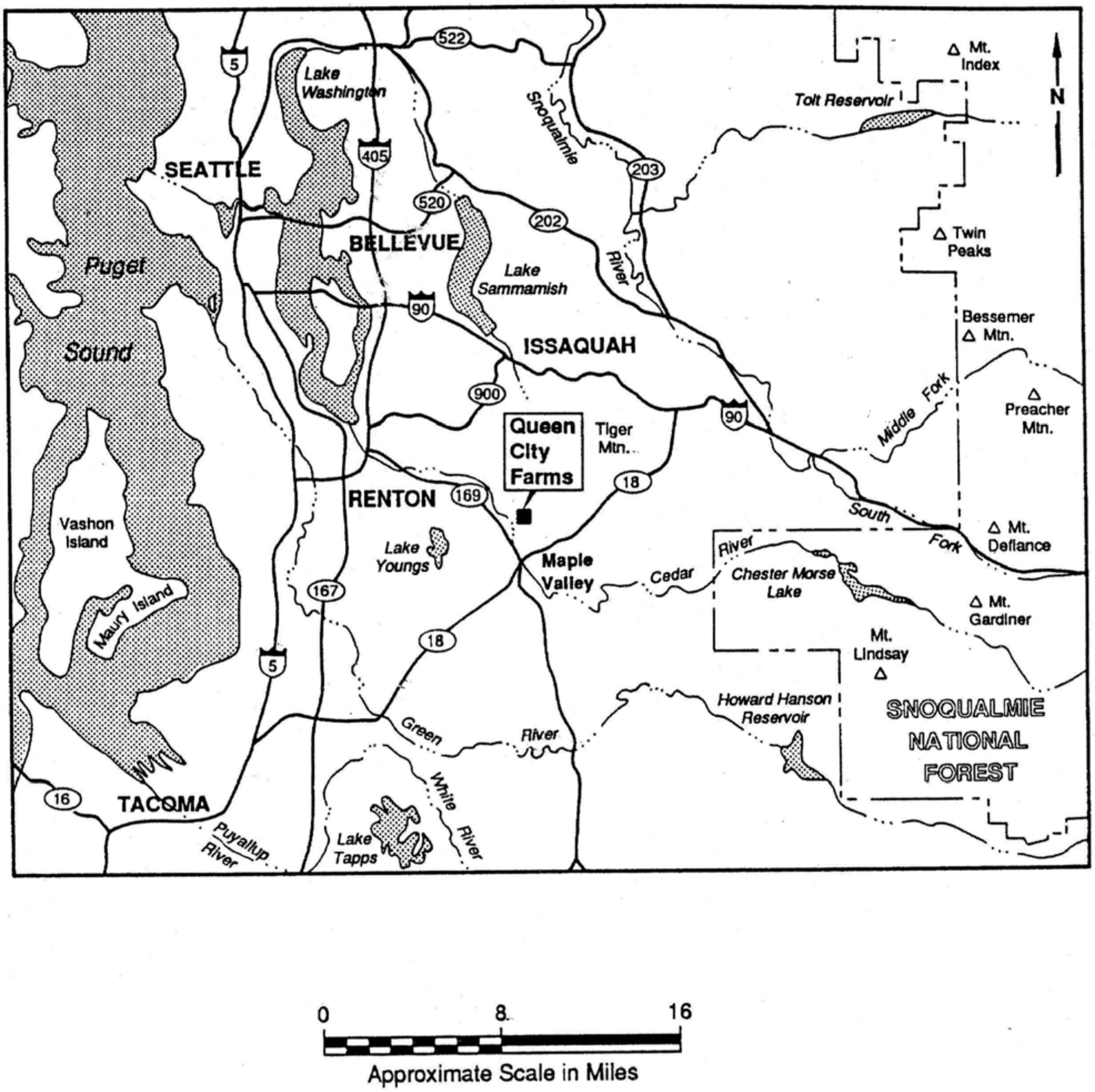
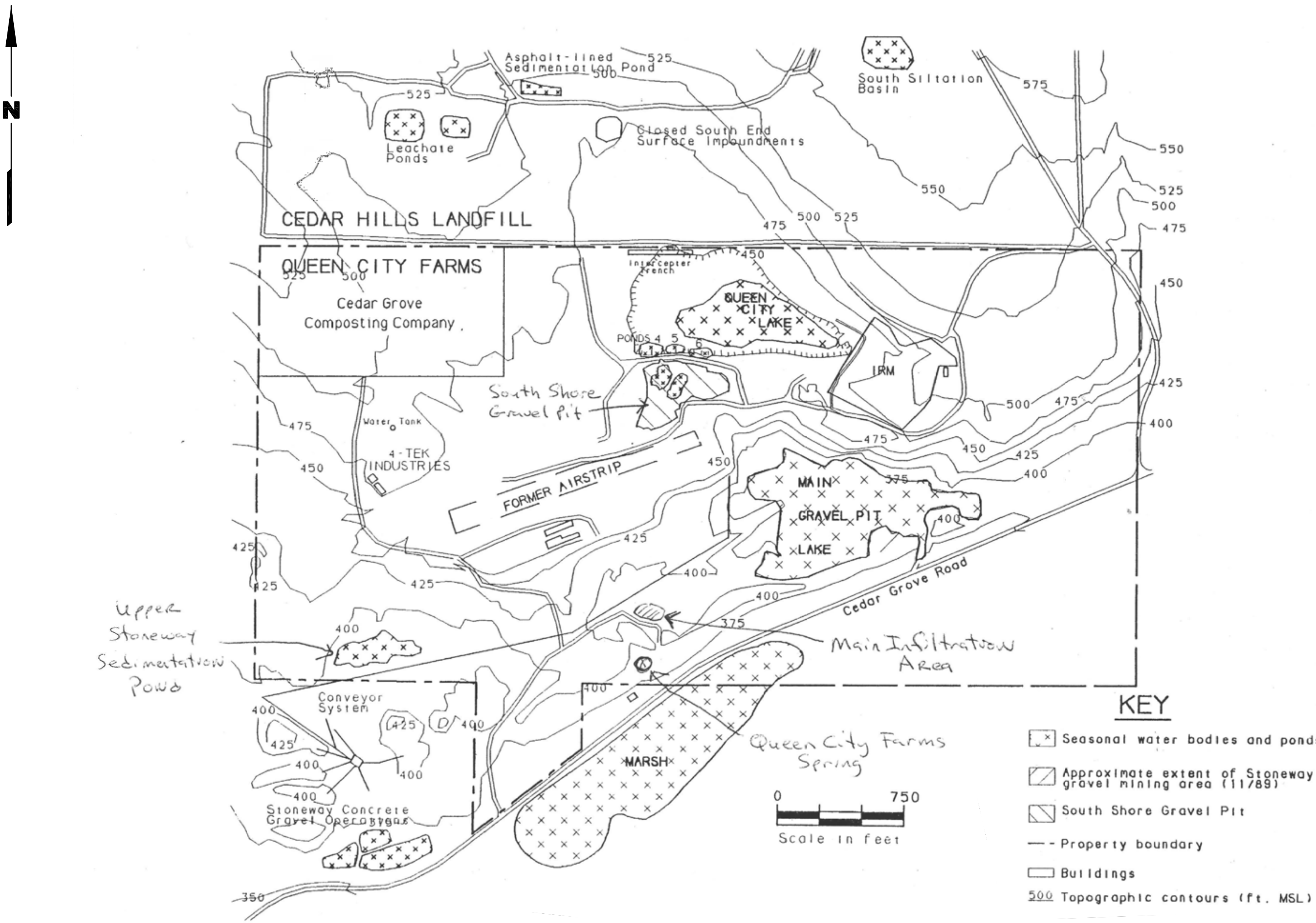


Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES	Queen City Farms Refill Project Maple Valley, Washington	**Vicinity Map**	Figure 1

Queen City Farms | I:\DATA\PROJECT\992 QCFT\Figures\Geology Report\CAD "Figure 1" 11/20/2006

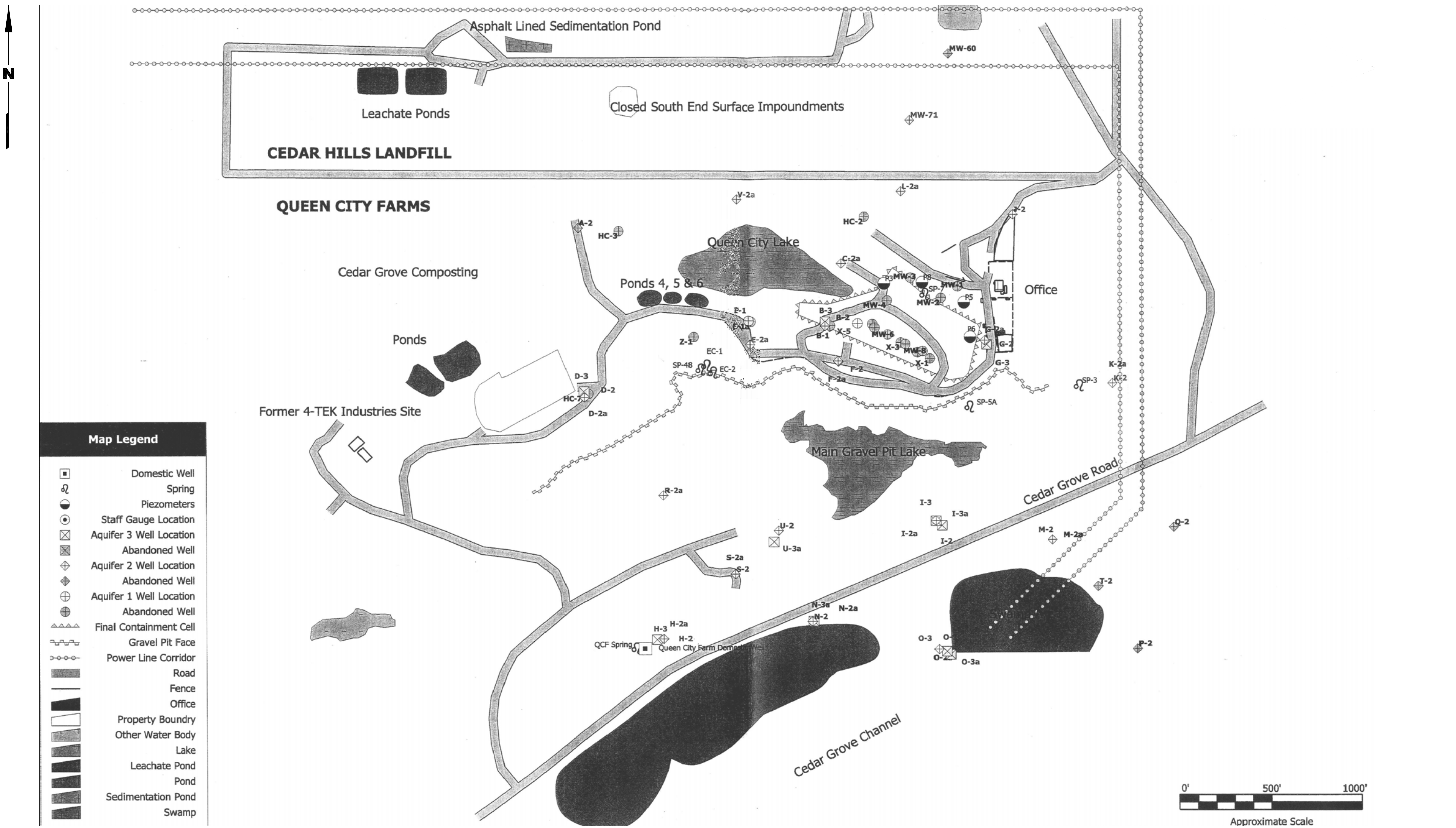


Source: Queen City Farms RI Report, 1990

Queen City Farms
Refill Project
Maple Valley, Washington

Site Topography

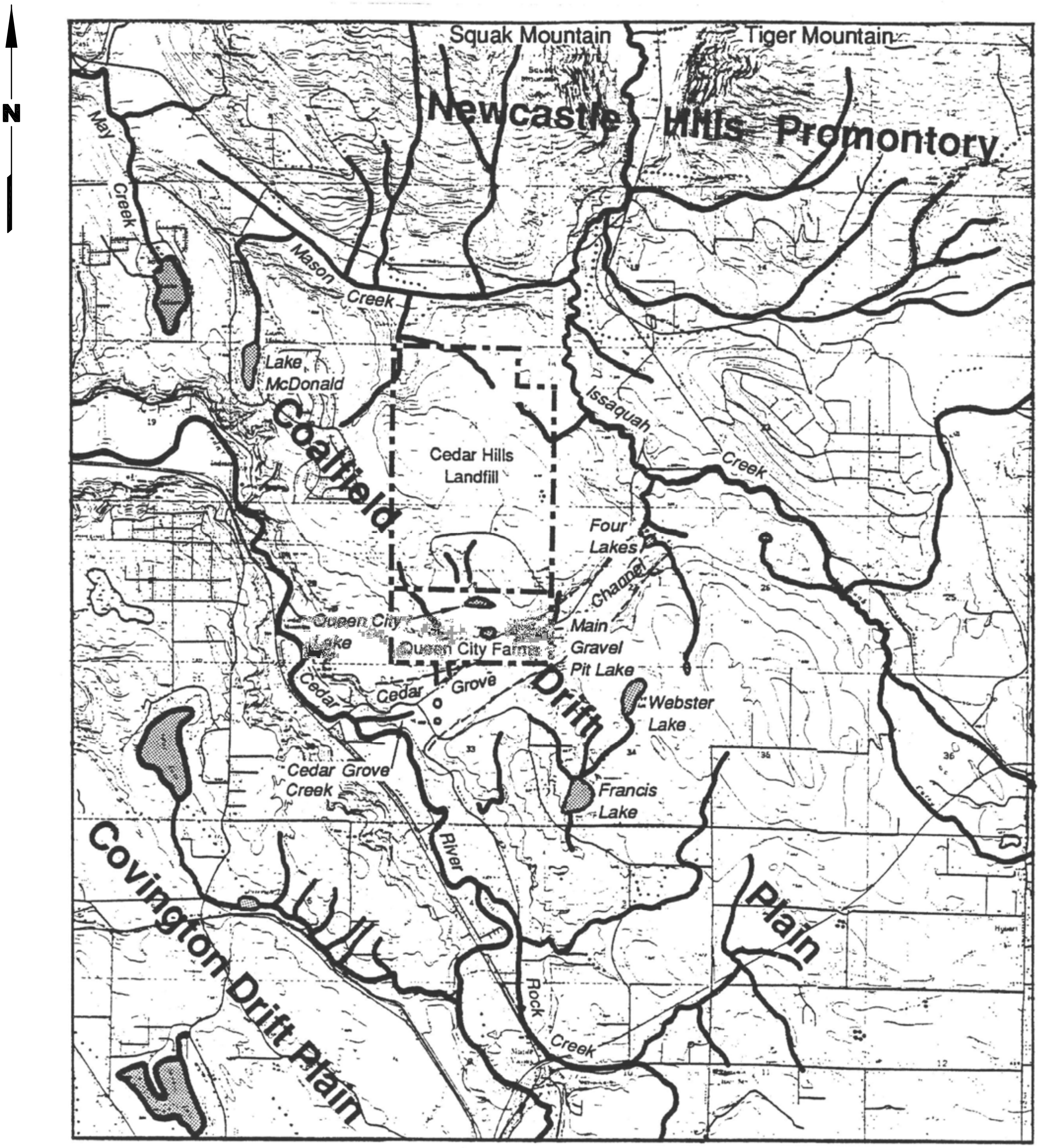
Figure 2



Source: Queen City Farms RI Report, 2005

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Monitoring Wells, Piezometers & Springs**	Figure 3



Source: Queen City Farms RI Report, 1990

Queen City Farms Refill Project Maple Valley, Washington	**Surface Features in the Vicinity of Queen City Farms**	Figure 4

Landau Associates

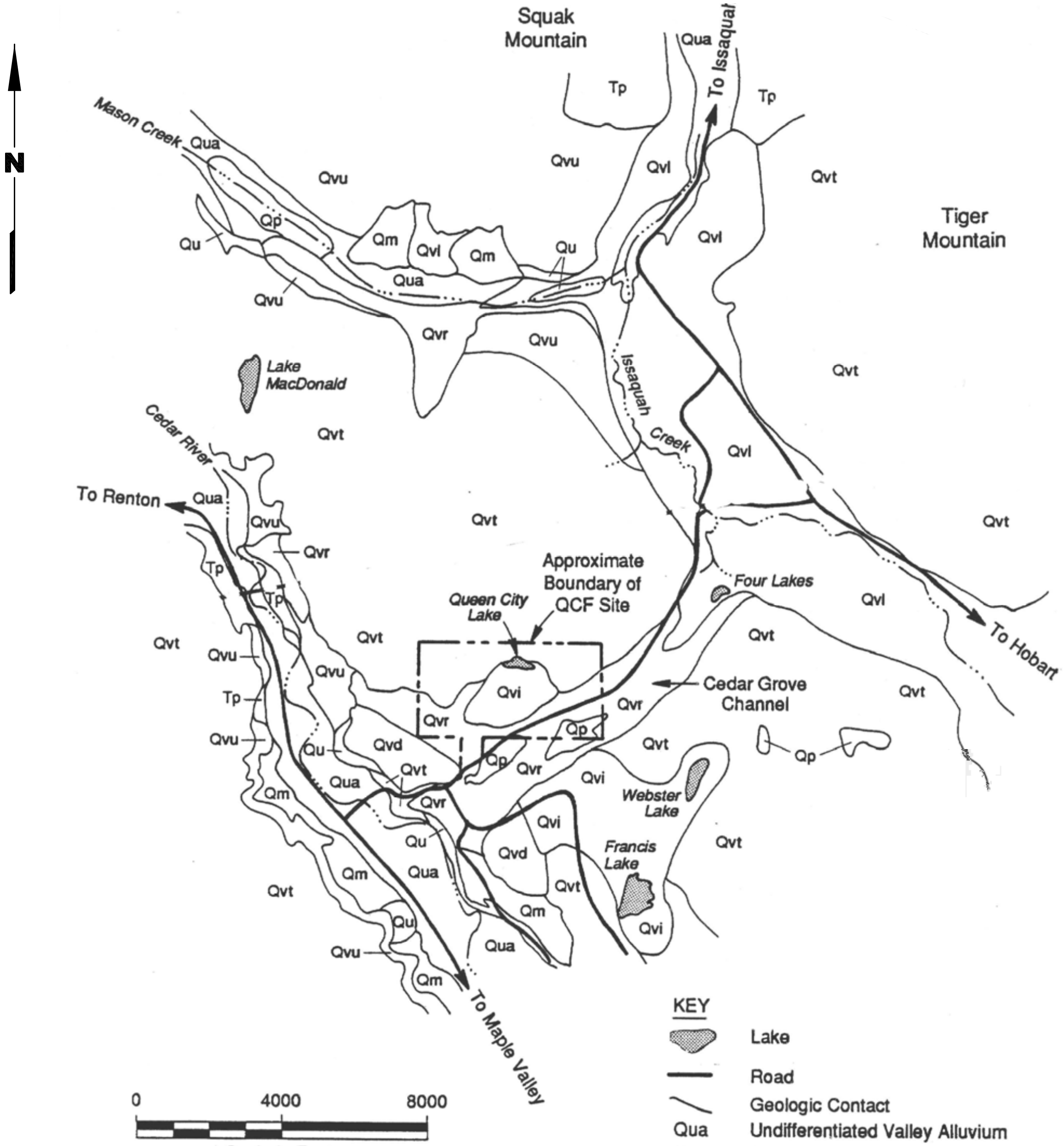
AGE	STRATIGRAPHIC UNIT	
RECENT	A	Clayey SILT, SILT, silty fine SAND
VASHON STADE	B	Poorly to well-sorted CLAY, SILT, SAND, and GRAVEL (ice contact deposit; locally water bearing within First Aquifer)
	C	Stratified, poorly to well-sorted SAND and GRAVEL with occasional interbedded clayey SILT and "Flow Till" (recessional outwash and deltaic deposits; locally water bearing within First Aquifer)
	Dr	Unsorted CLAY, SILT, SAND and GRAVEL (till deposited during recessional phase) (Till Aquitard)
	D	Unsorted CLAY, SILT, SAND and GRAVEL (till, Till Aquitard)
	E	Stratified poorly to well-sorted SAND and GRAVEL with occasional silty GRAVEL interbeds (advance outwash; locally water bearing within Second Aquifer)
PRE-VASHON UNDIFFERENTIATED DRIFT	F	Well-sorted silty fine SAND to medium SAND with occasional clayey SAND to silty CLAY interbeds (upper section is dark red-brown to orange-brown color, water bearing within Second Aquifer)
	G	SILT, clayey SILT, fine sandy SILT, and silty fine SAND (Second Aquitard)
	H	Well-sorted very fine to medium SAND with sandy GRAVEL and gravelly SAND interbeds (occasional wood fragments, water bearing within Third Aquifer)
	I	SILT and fine sandy SILT (containing abundant wood fragments)) to gravelly SILT with occasional SAND and sandy GRAVEL interbed (Woody Aquitard)
	J	Interbedded sandy GRAVEL, gravelly SAND, silty sandy GRAVEL, sandy SILT, and SAND (Fourth Aquifer)
	U	CLAY, with closely spaced slickensides with interbedded SILT and very fine SAND (Undifferentiated Pre-Vashon Quaternary lacustrine sediments)

Note: Stratigraphic Units I and J are based in part on interpretation and generalization of boring log data for monitoring wells MW-24, MW-53, and MW-54 at Cedar Hills Landfill.

Source: Queen City Farms RI Report, 1990

Stratigraphic Units Beneath Queen City Farms

Figure 5



KEY

	Lake
	Road
	Geologic Contact
Qua	Undifferentiated Valley Alluvium
Qp	Peat and Swamp Deposit
Qm	Mass-Wasting Debris
Qvu	Undifferentiated Vashon Drift
Qvt	Vashon Till
Qvr	Vashon Recessional Outwash
Qvl	Vashon Lacustrine Deposit
Qvi	Vashon Ice-Contact Deposit
Qvd	Vashon Deltaic Deposit
Qu	Pre-Vashon Undifferentiated Drift
Tp	Tertiary Bedrock; Sandstone, Siltstone with Tuff Breccia and Lava Flow

Notes:
1. See text, Section 3.3.1 for descriptions of geologic units.
2. General geological relationships after Rosengreen (1965) and Luzier (1969).
3. Geology on QCF represents observations made prior to gravel mining on the site.

Source: Queen City Farms RI Report, 1990

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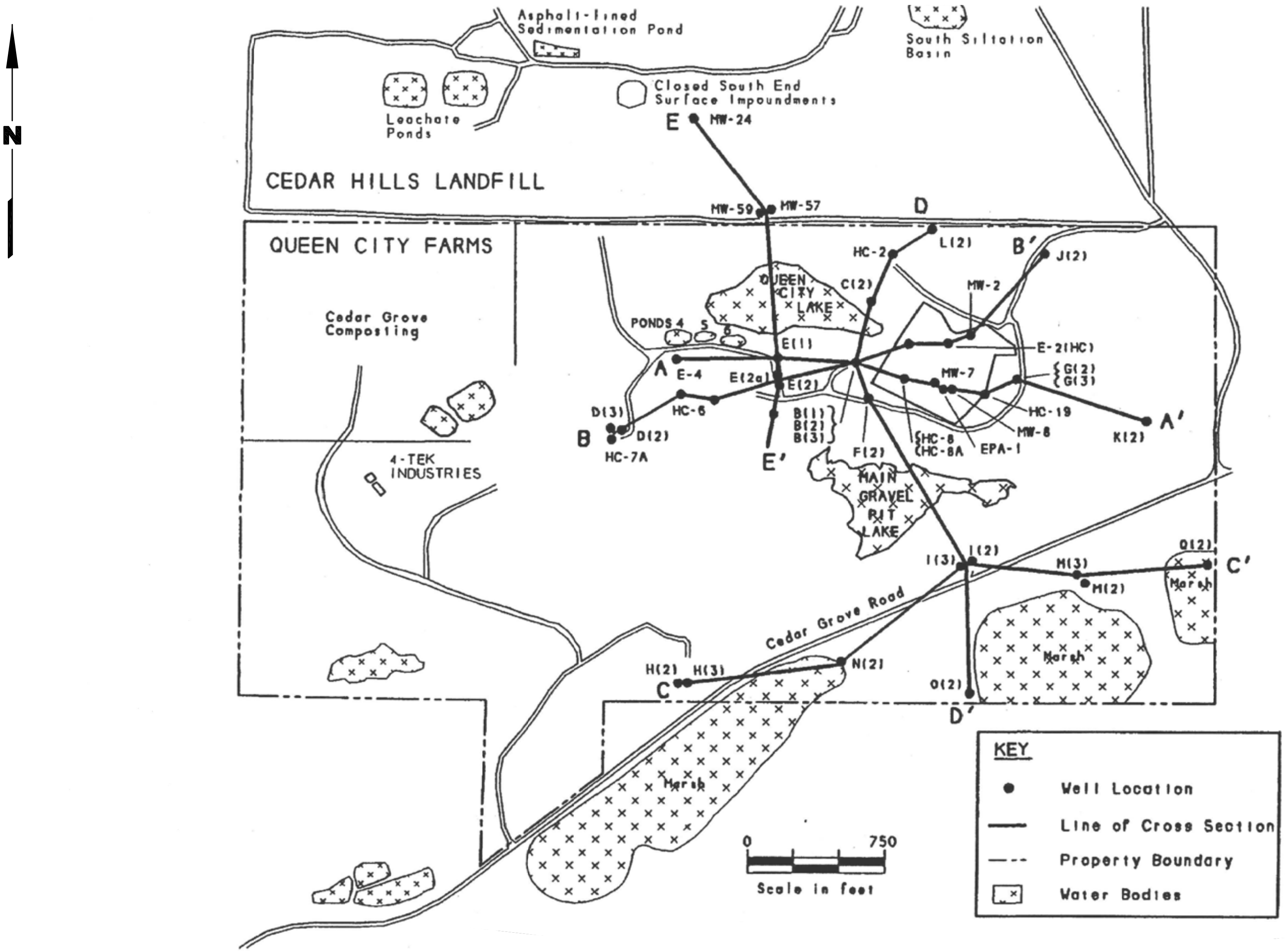
Queen City Farms
Refill Project
Maple Valley, Washington

Regional Surficial Geologic Map

Figure
6



Source: Queen City Farms Supplemental RI Report, 1991

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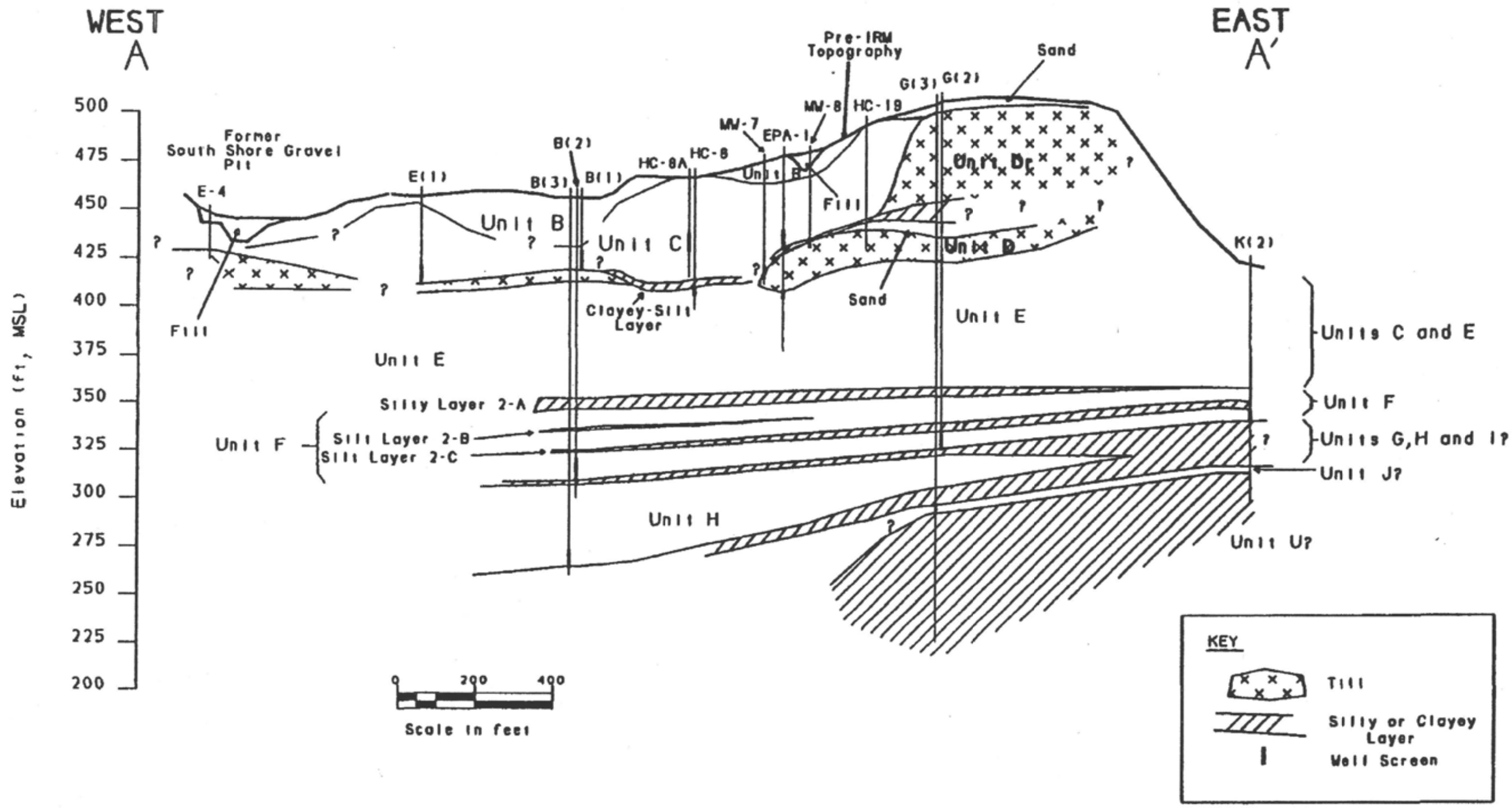
Queen City Farms
Refill Project
Maple Valley, Washington

Locations of Geologic Cross Sections

Figure 7



Source: Queen City Farms Supplemental RI Report, 1991

Queen City Farms
Refill Project
Maple Valley, Washington

Cross Section A-A'

Figure
8

LANDAU
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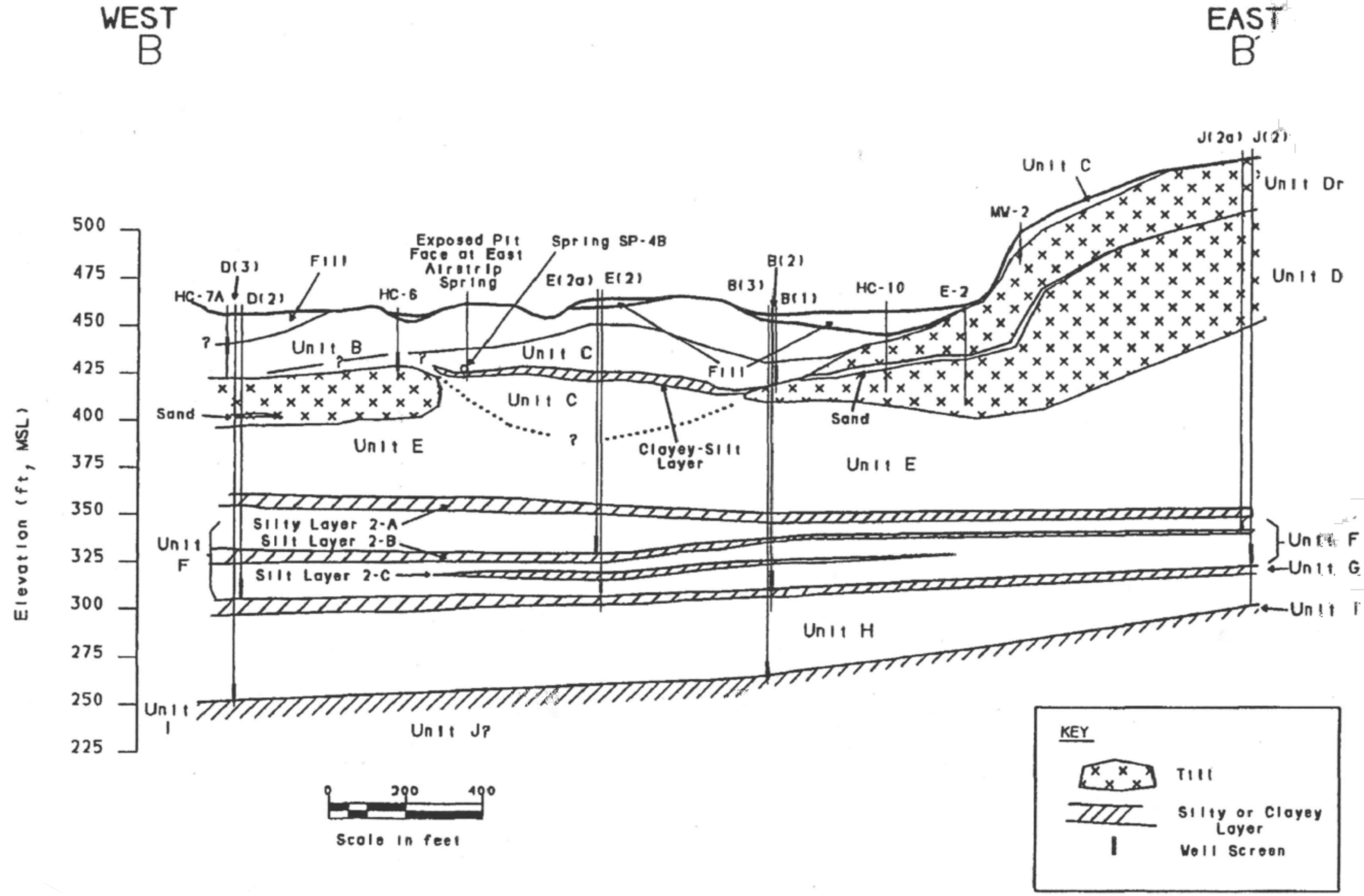


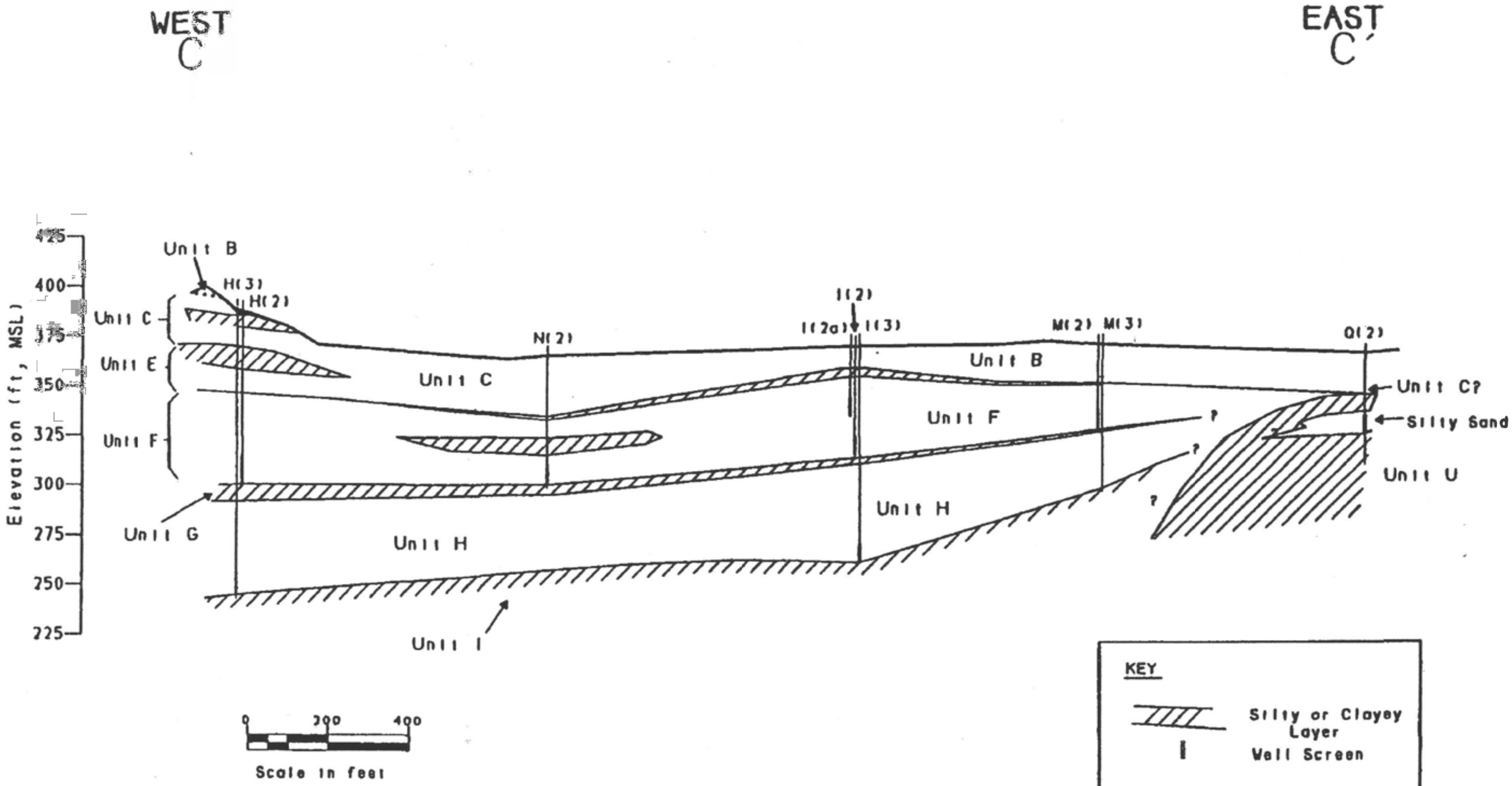
Source: Queen City Farms Supplemental RI Report, 1991

Landau Associates

Queen City Farms
Refill Project
Maple Valley, Washington

Cross Section B-B'

Figure 9



Source: Queen City Farms Supplemental RI Report, 1991

Queen City Farms
Refill Project
Maple Valley, Washington

Cross Section C-C'

Figure
10

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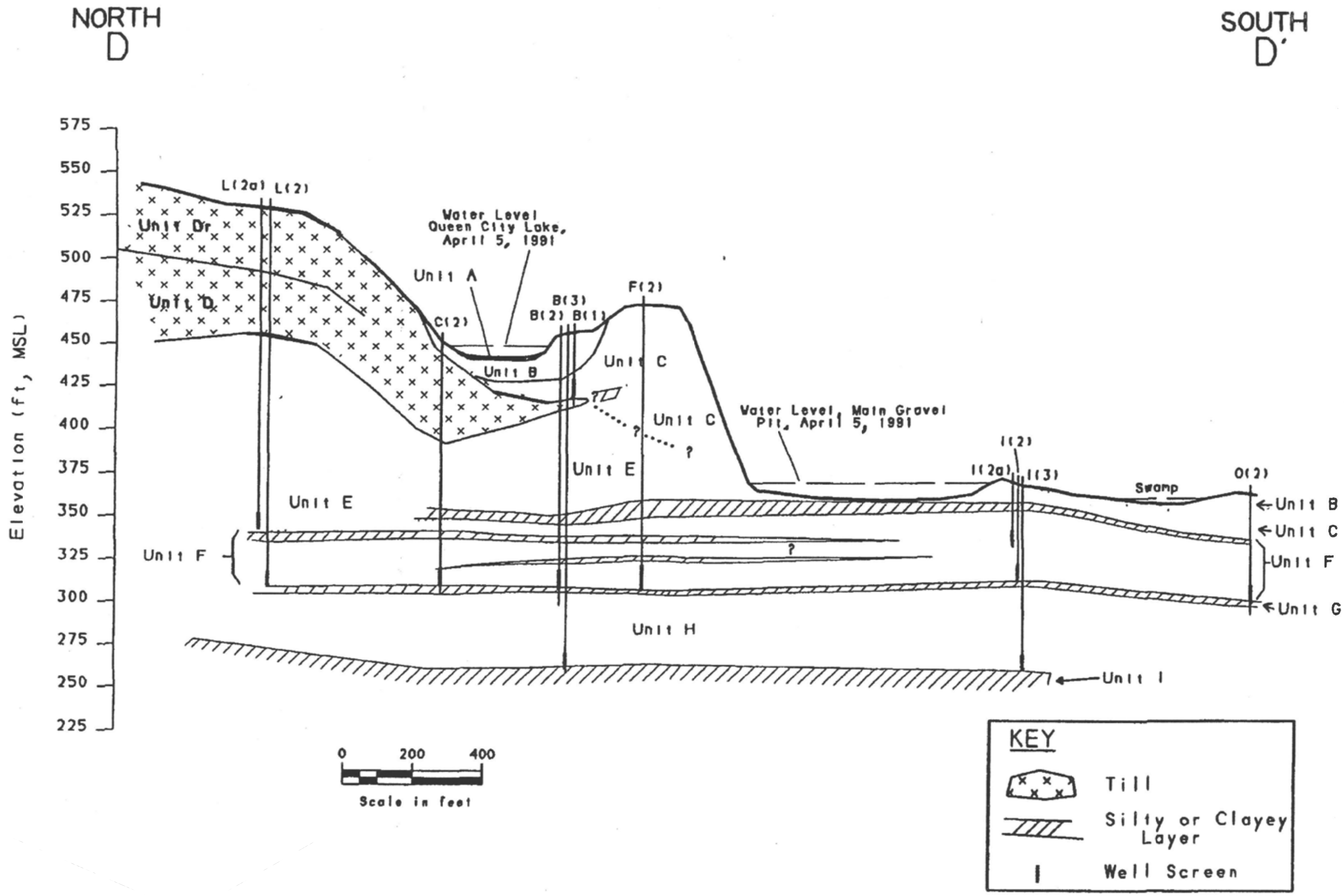


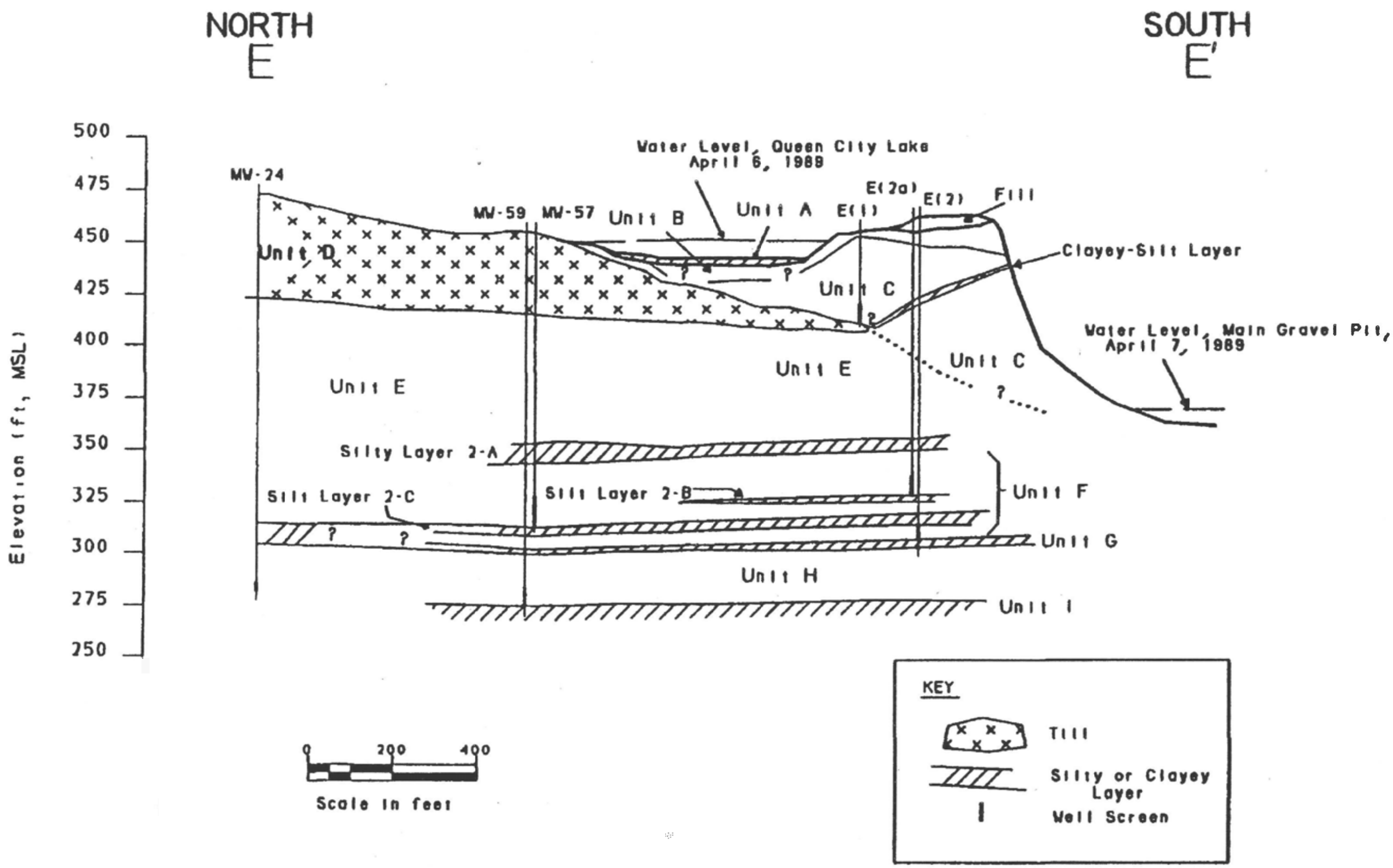
Source: *Queen City Farms Supplemental RI Report, 1991*

LANDAU ASSOCIATES

Queen City Farms
Refill Project
Maple Valley, Washington

Cross Section D-D'

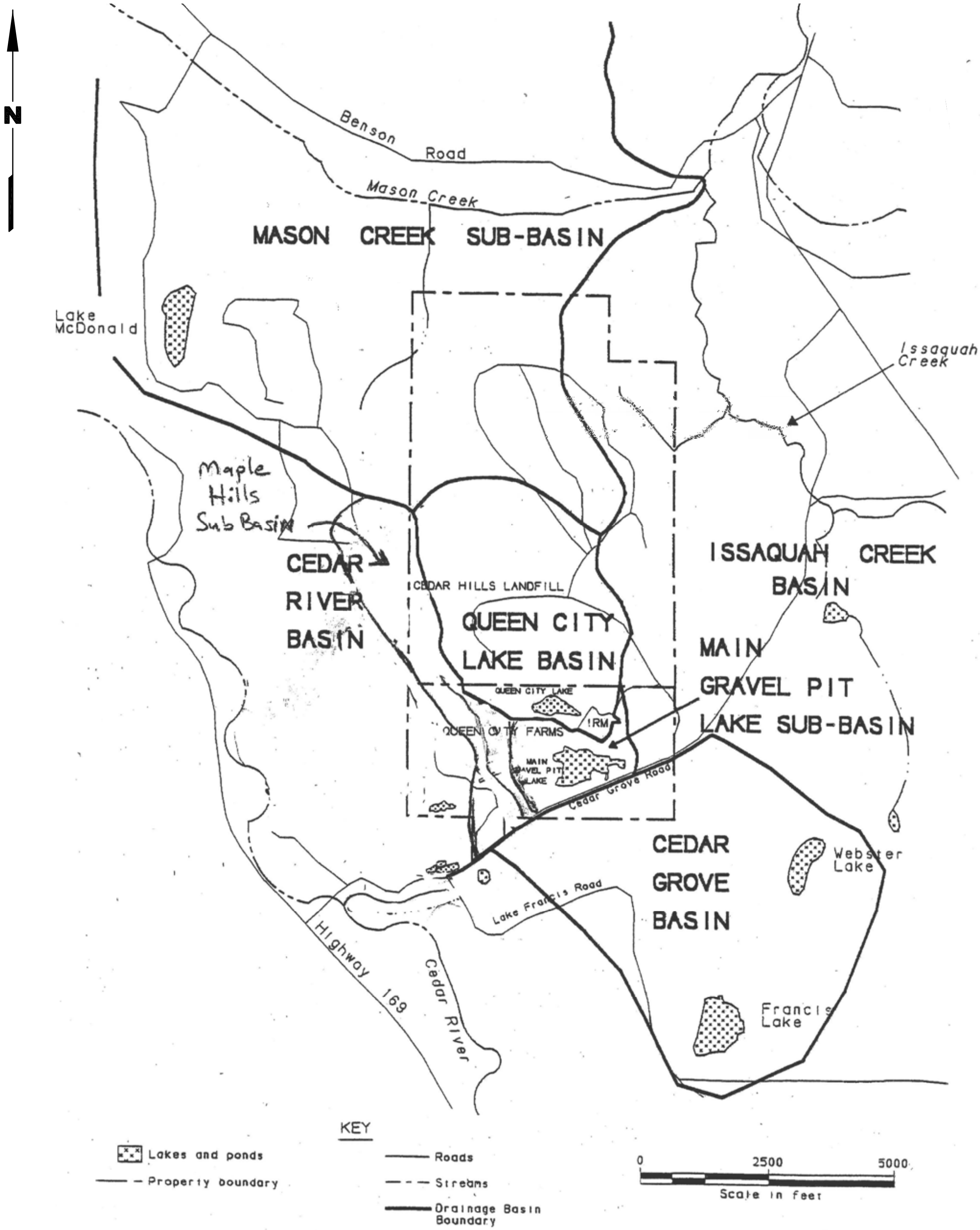
Figure 11



Source: Queen City Farms Supplemental RI Report, 1991

LANDAU ASSOCIATES

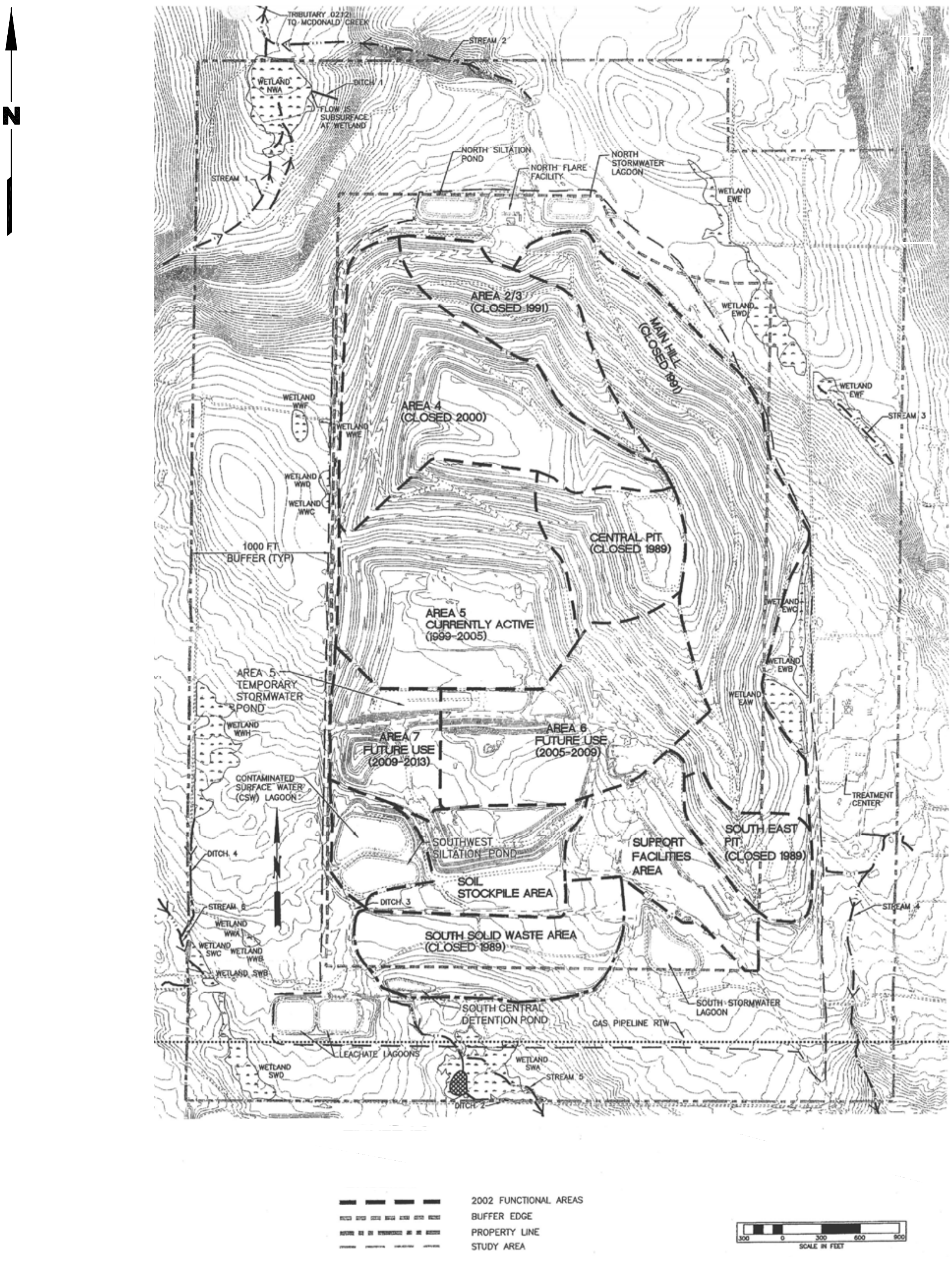
Queen City Farms Refill Project Maple Valley, Washington	**Cross Section E-E'**	Figure 12



Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Drainage Basins in the Vicinity of Queen City Farms**	Figure 13

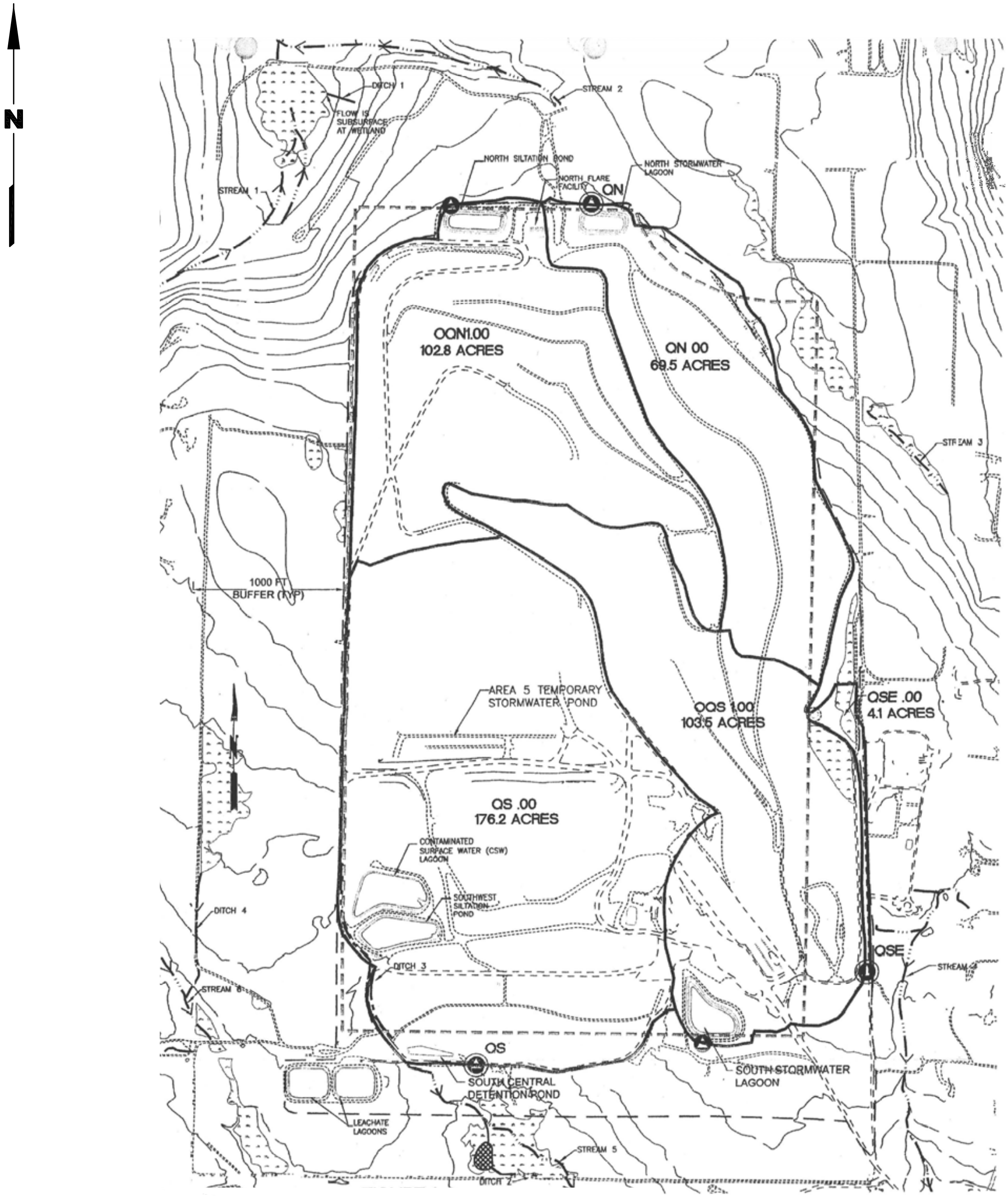


Source: King County Department of Natural Resources, 2005

Queen City Farms Refill Project Maple Valley, Washington	**Cedar Hills Regional Landfill Basemap**	Figure 14

LANDAU ASSOCIATES

Queen City Farms | I:\DATA\PROJECT\992 QCF\T\Figures\Geology Report\CAD "Figure 14" 11/21/2006



BUFFER EDGE
PROPERTY LINE
STUDY AREA
Approximate location for discharge point See Appendix "B" for exact locations
Designated discharge point

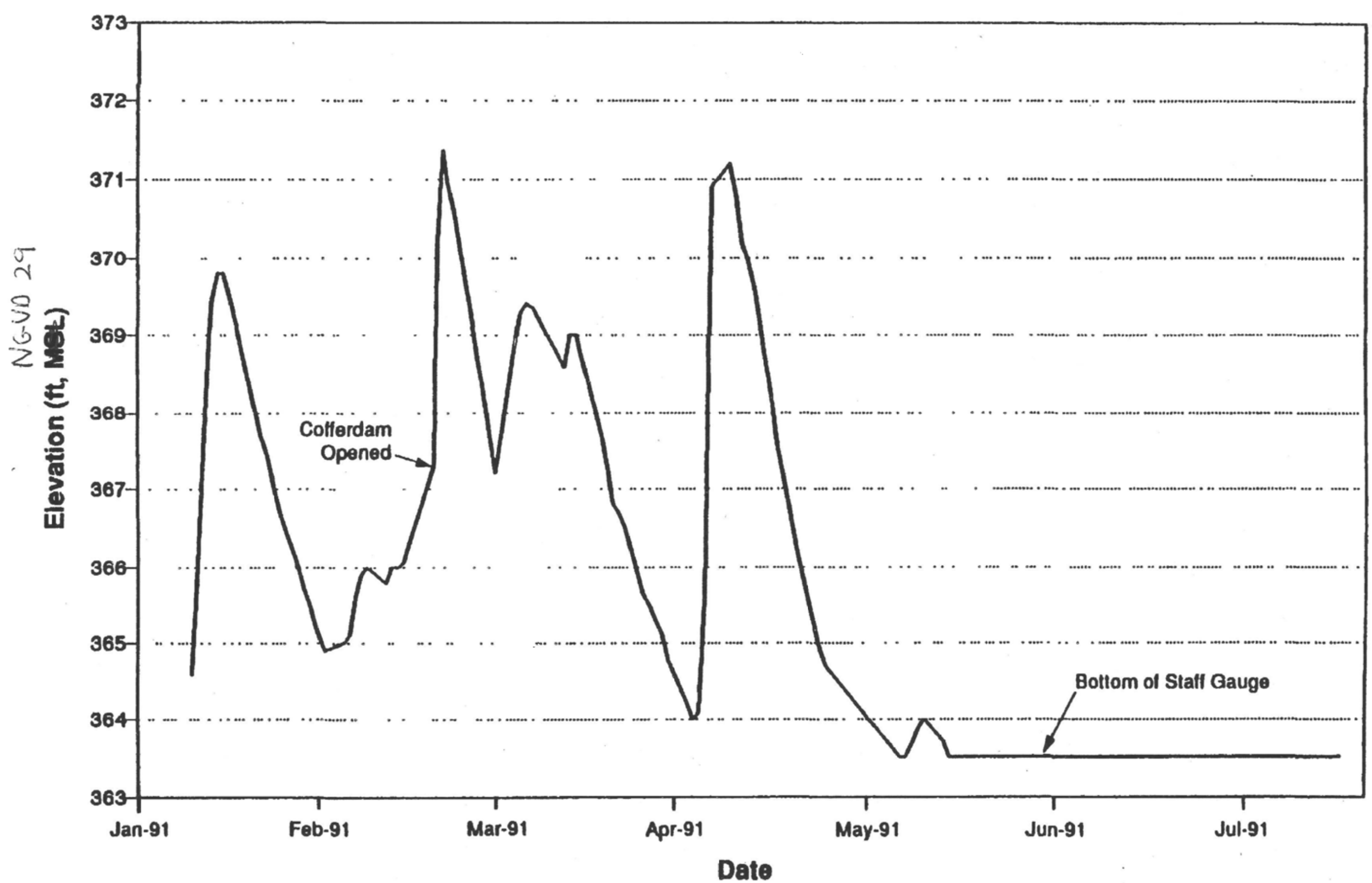
NOTE:
AS THE LANDFILL IS CLOSED OUT (FINAL COVER ANTICIPATED 2013), SUB-BASIN BOUNDARIES MAY CHANGE SLIGHTLY. THE EFFECT WILL BE TO RE-ALLOCATE FLOWS BETWEEN SUB-BASINS. TOTAL DISCHARGE WILL NOT BE AFFECTED.

300 0 300 600 900
SCALE IN FEET

Source: King County Department of Natural Resources, 2005

Queen City Farms | I:\DATA\PROJECT\992 QCF\T\Figures\Geology Report\CAD "Figure 15" 11/21/2006

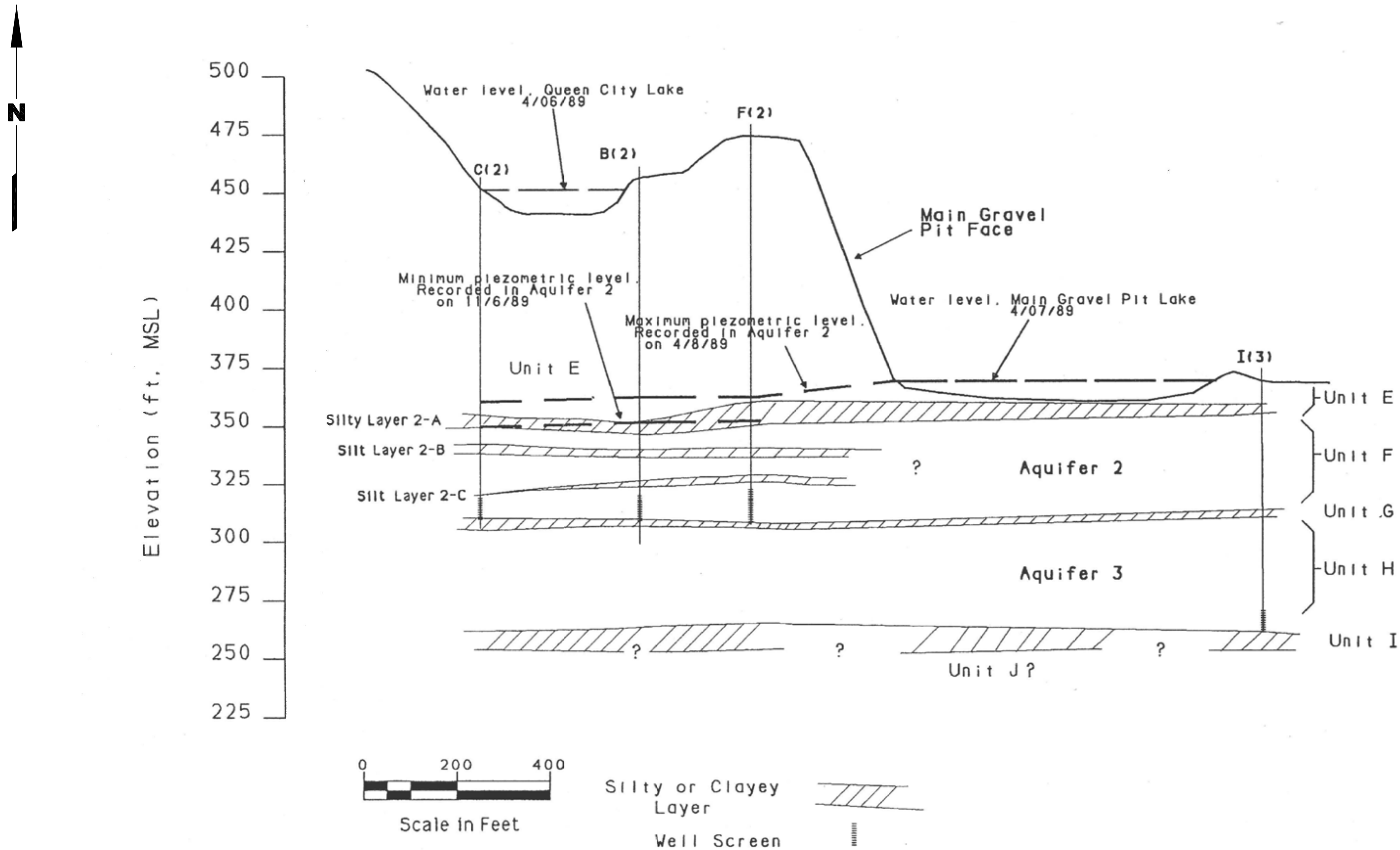
LANDAU ASSOCIATES	Queen City Farms Refill Project Maple Valley, Washington	**Cedar Hills Regional Landfill KCRTS Analysis Final Development Scenario Sub-basin Boundaries, 2000 Topo**	Figure 15



Source: Queen City Farms Supplemental RI Report, 1991

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Main Gravel Pit Water Levels**	Figure 16



Note: Geologic details based on Cross Section D-D', Appendix C, Figure C-5.

Source: *Queen City Farms RI Report, 1990*

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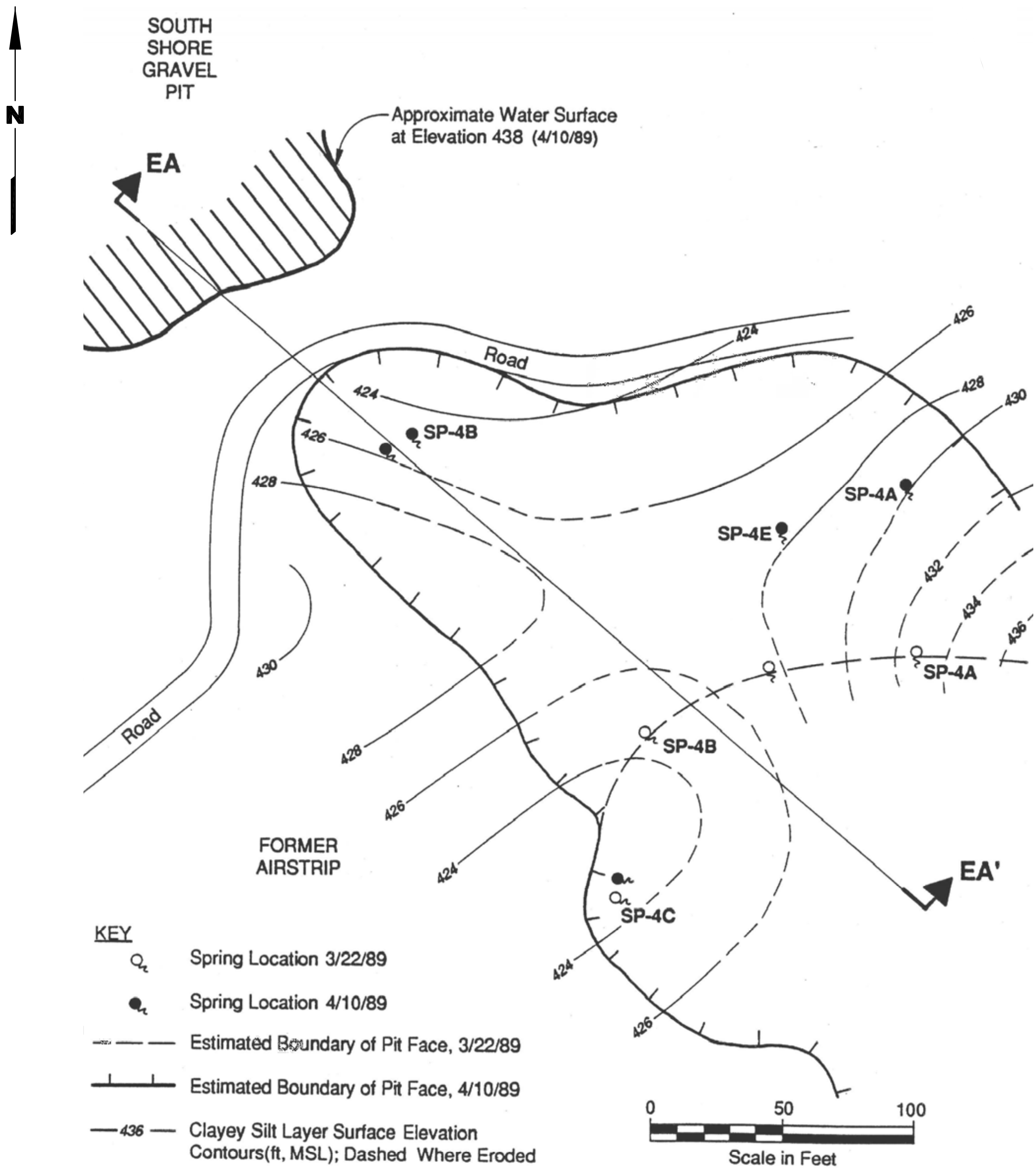
Queen City Farms
Refill Project
Maple Valley, Washington

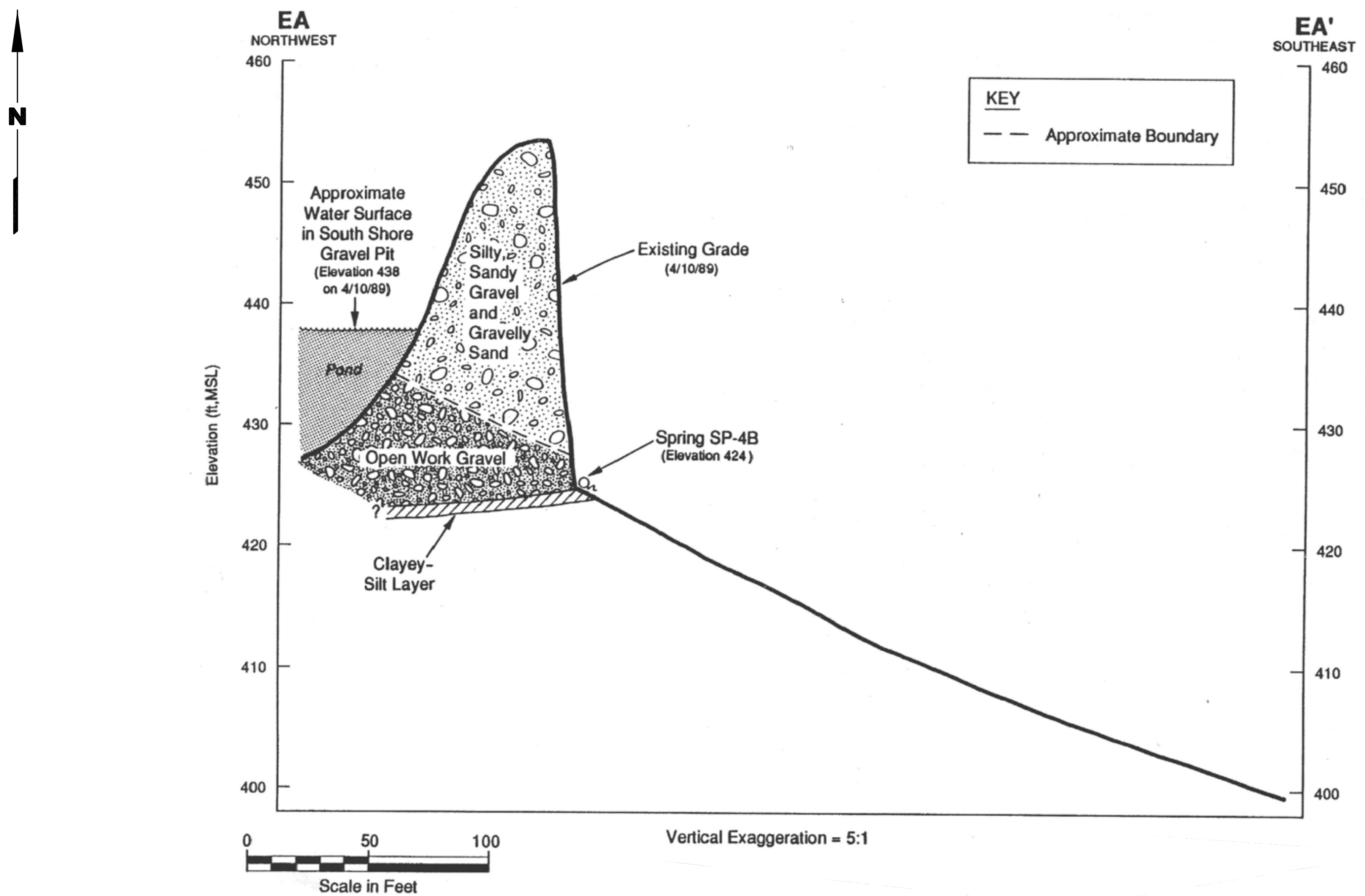
Cross Section Showing Main Gravel Pit Lake and Aquifer 2

Figure 17



Source: Queen City Farms RI Report, 1990

Landau Associates	Queen City Farms Refill Project Maple Valley, Washington	**Plan View of East Airstrip Spring Area**	Figure 18



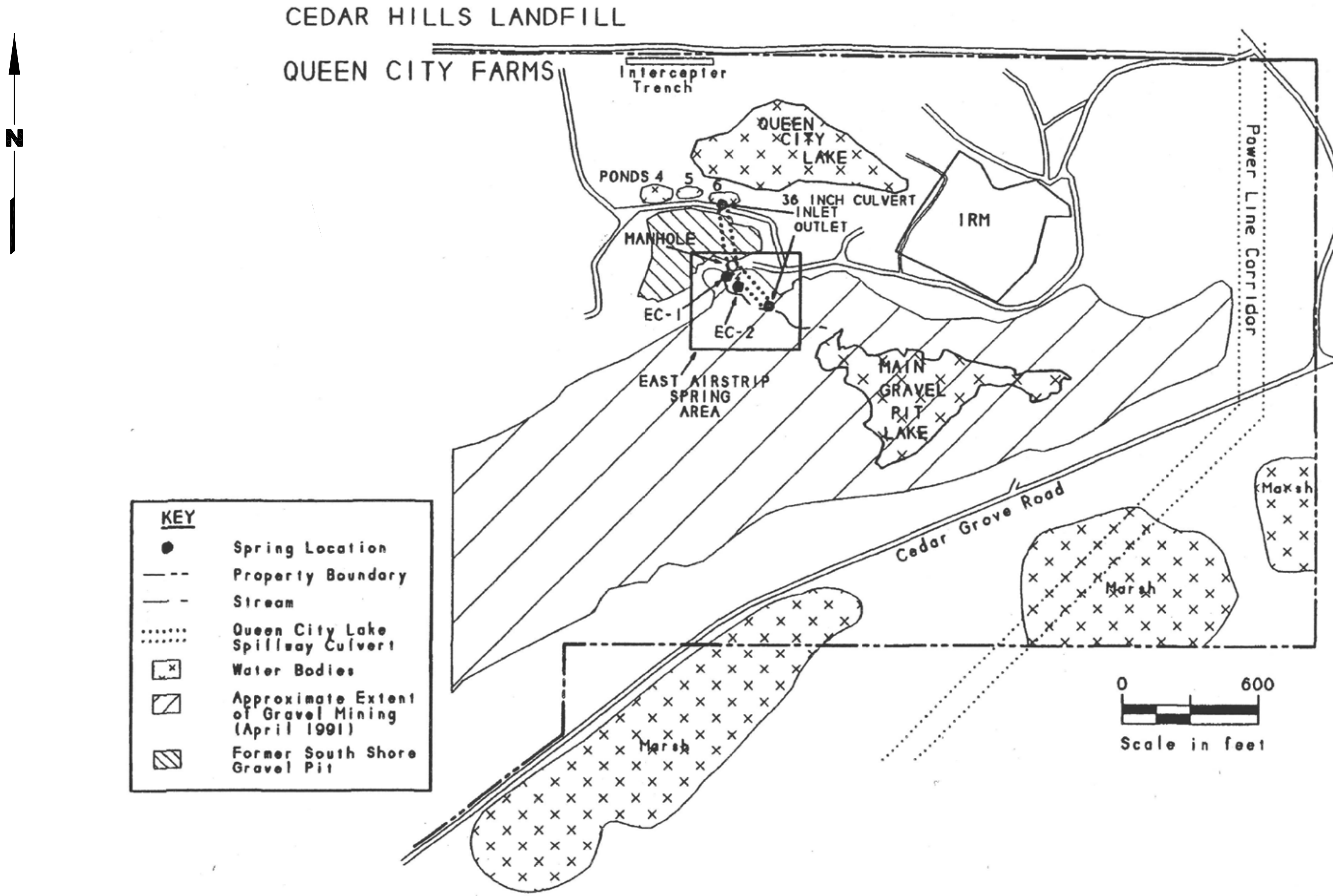
Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

Queen City Farms
Refill Project
Maple Valley, Washington

Cross Section EA-EA'
Profile of East Airstrip
Spring Area on 4/10/89

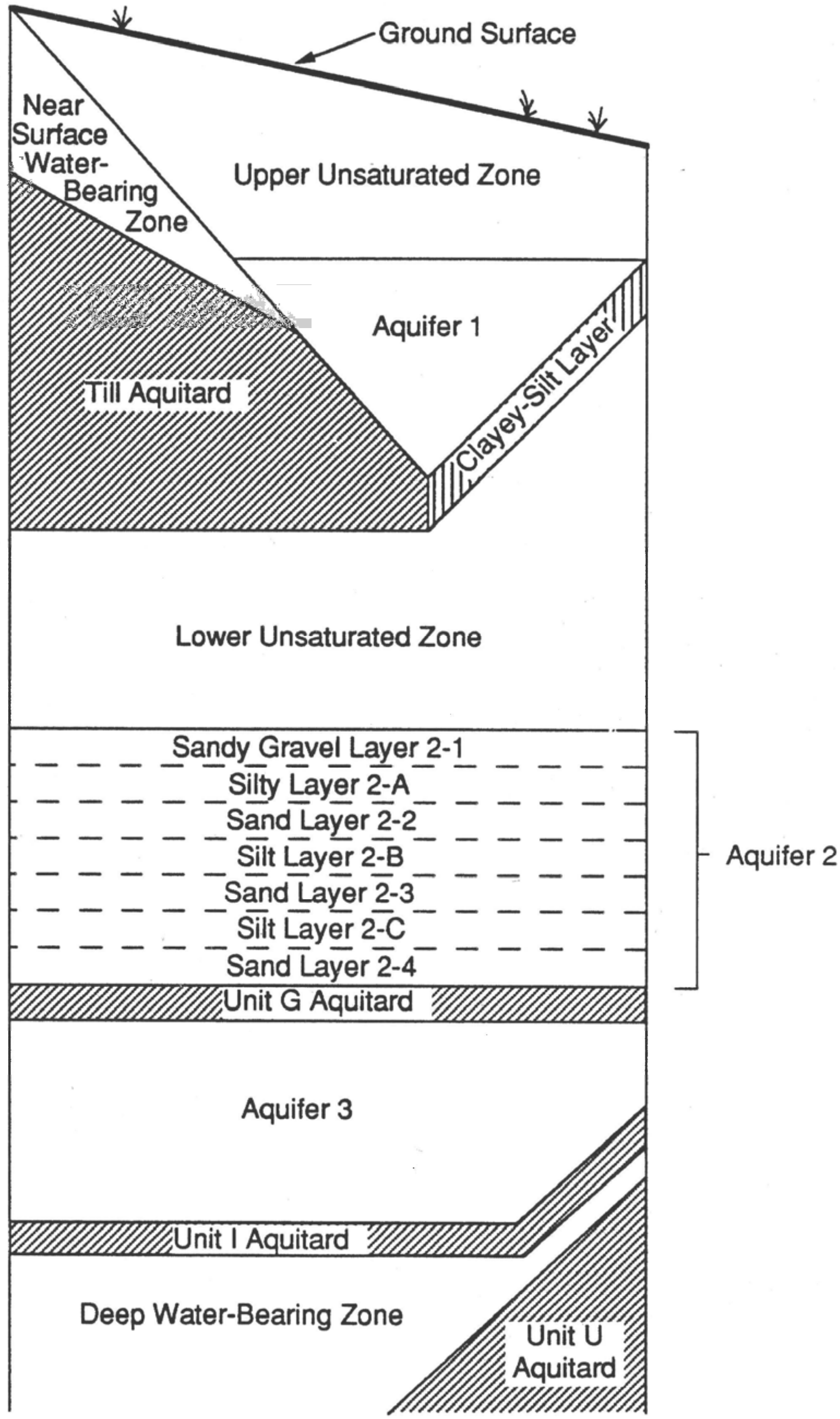
Figure
19



Source: Queen City Farms Supplemental RI Report, 1991

Queen City Farms Refill Project Maple Valley, Washington	**Erosion Control Project**	Figure 20

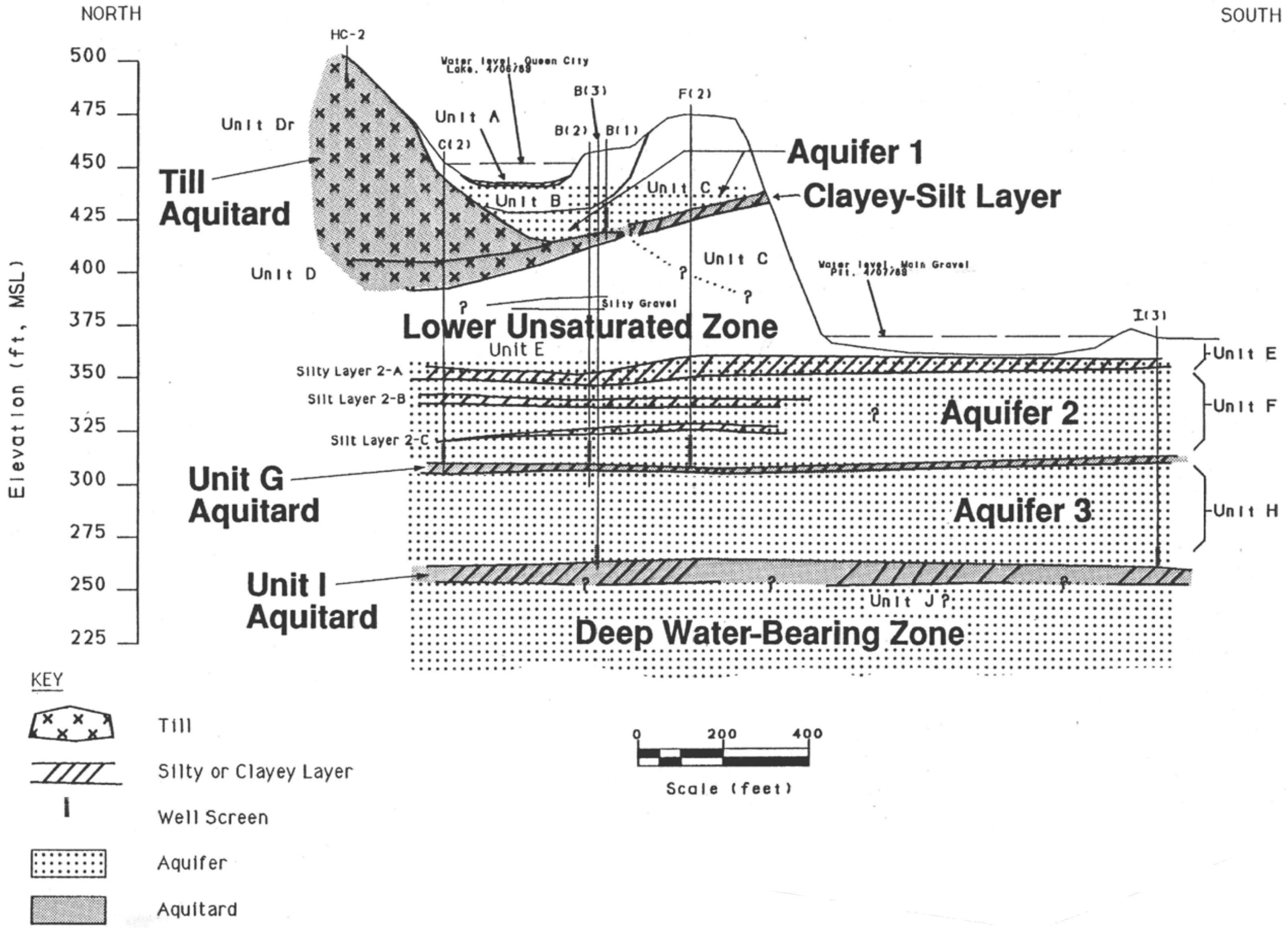
LANDAU ASSOCIATES



Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

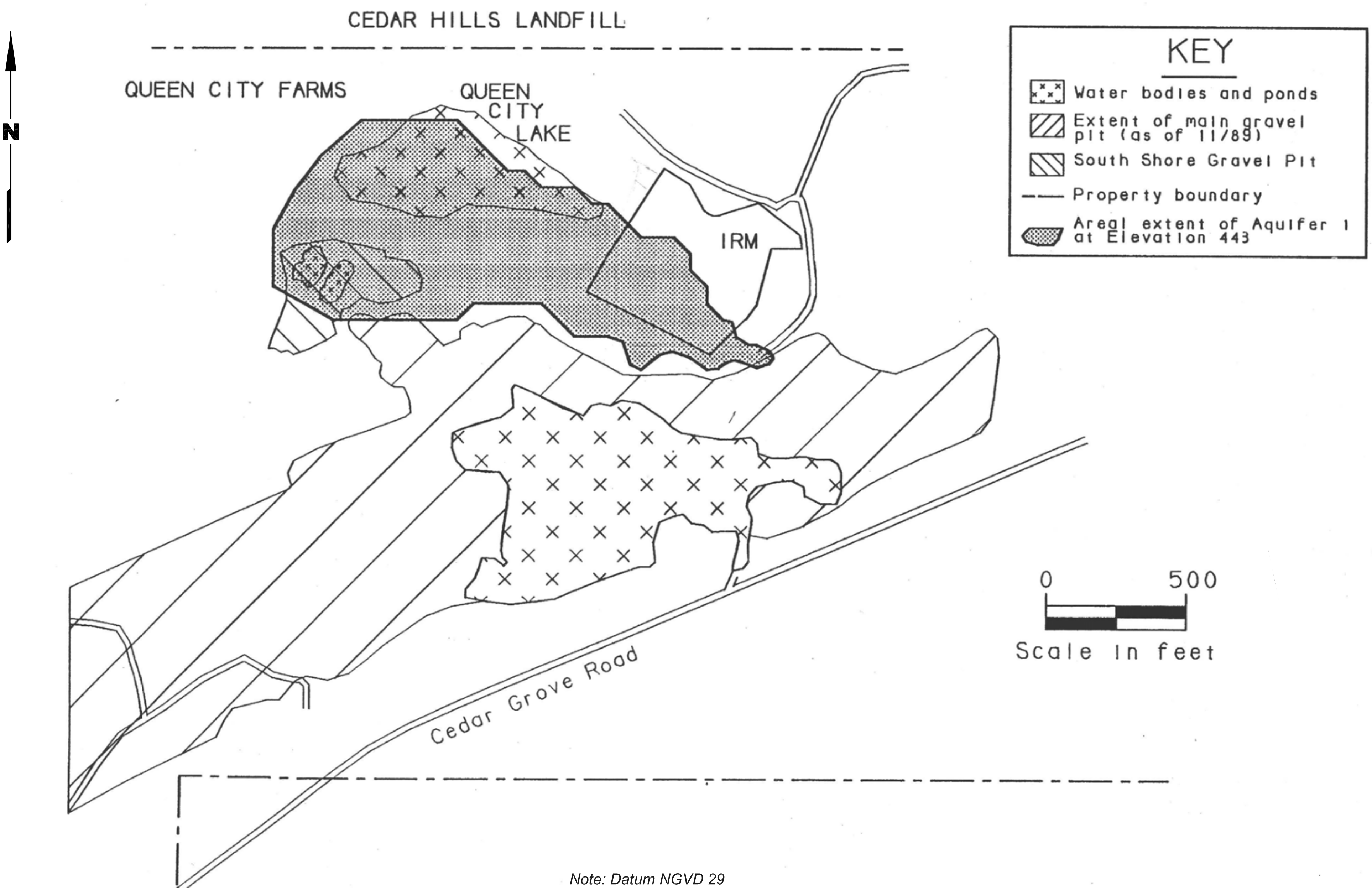
Queen City Farms Refill Project Maple Valley, Washington	**Schematic Representation of Hydrogeology at Queen City Farms**	Figure 21



Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

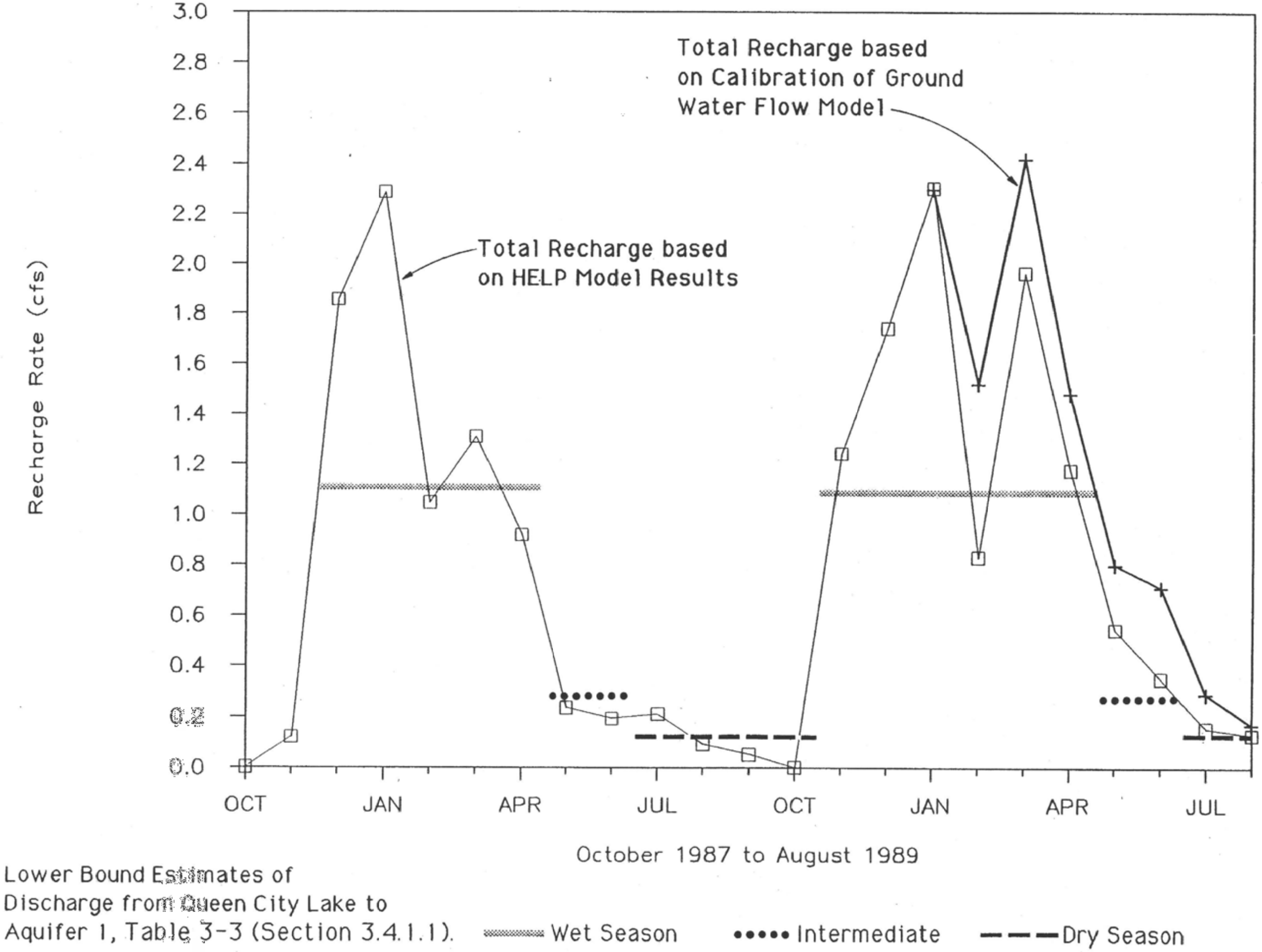
Queen City Farms Refill Project Maple Valley, Washington	**North - South Hydrogeologic Cross Section**	Figure 22



Note: Datum NGVD 29
Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Areal Extent of Aquifer 1 at Piezometric Head Elevation 443**	Figure 23



Source: Queen City Farms RI Report, 1990

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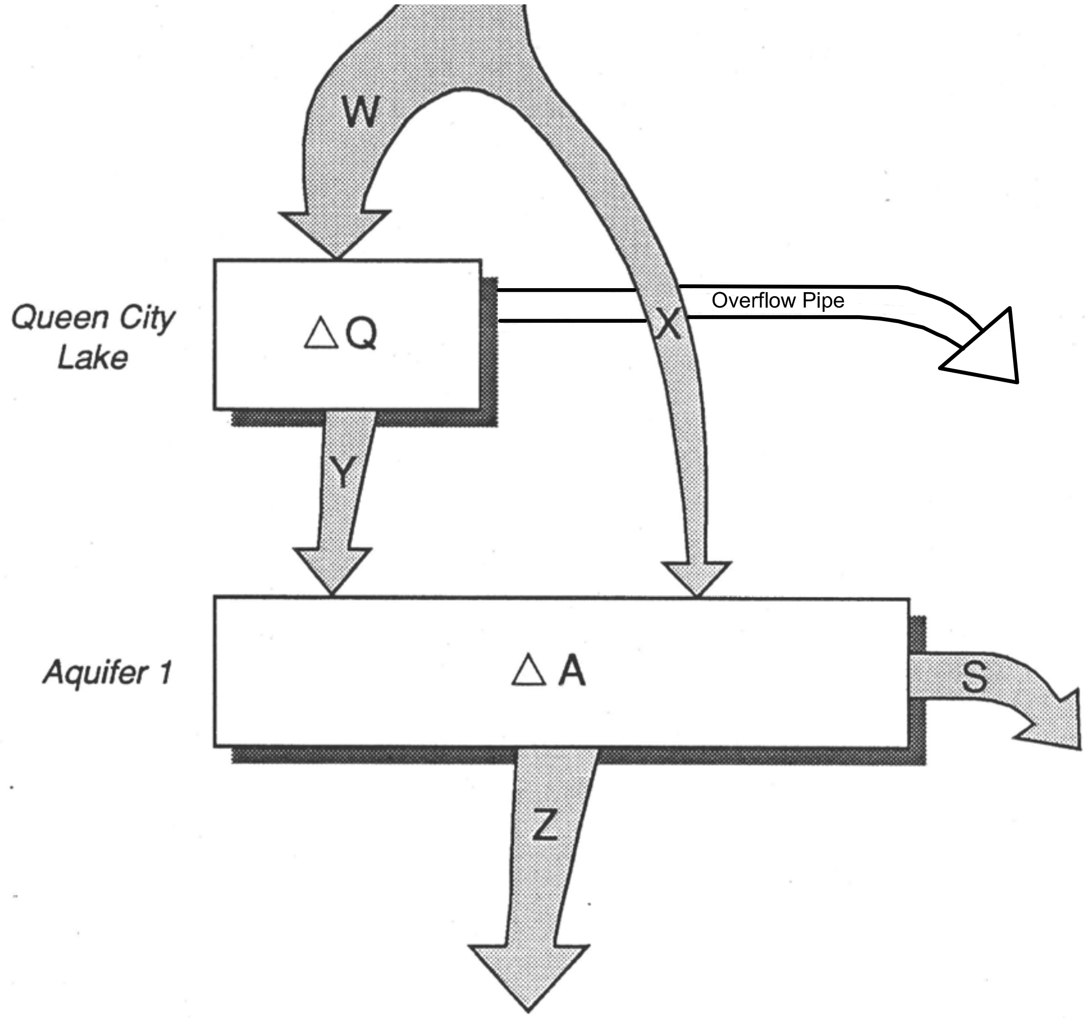
Queen City Farms
Refill Project
Maple Valley, Washington

Summary of Recharge Rates to Aquifer 1

Figure 24



V = Total Surface Water Runoff

W = Fraction of Surface Water Runoff Going Directly into Queen City Lake

X = Fraction of Surface Water Runoff Recharging Directly to Aquifer 1

Y = Recharge to Aquifer 1 through Queen City Lake Bottom

S = Discharge from Aquifer 1 through Springs or Subsurface Leakage through Soils along the Western Boundary of the Aquifer

Z = Discharge from Aquifer 1 through the Clayey-Silt Layer

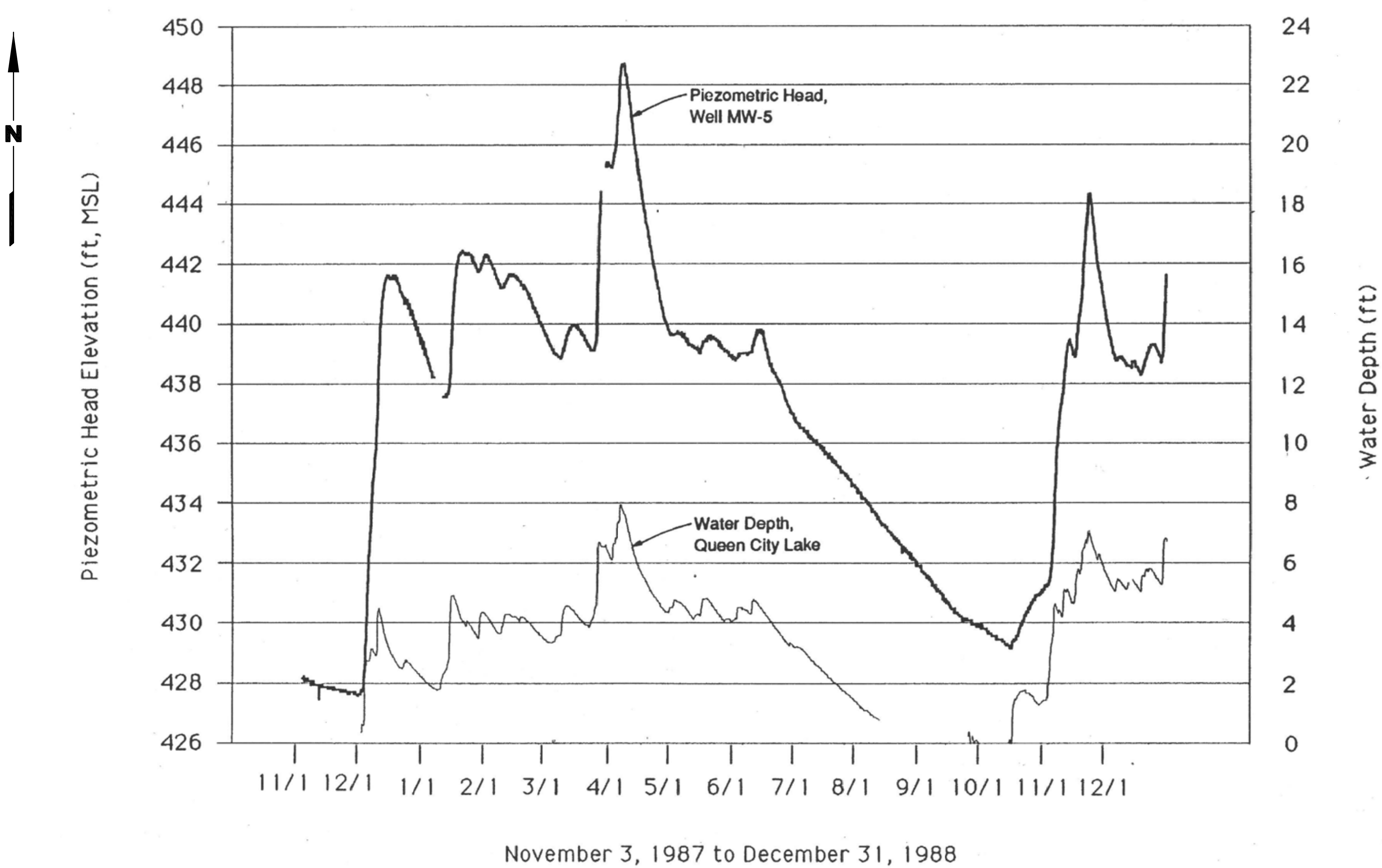
△Q = Change in Water Volume in Queen City Lake

△A = Change in Water Volume in Aquifer 1

Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

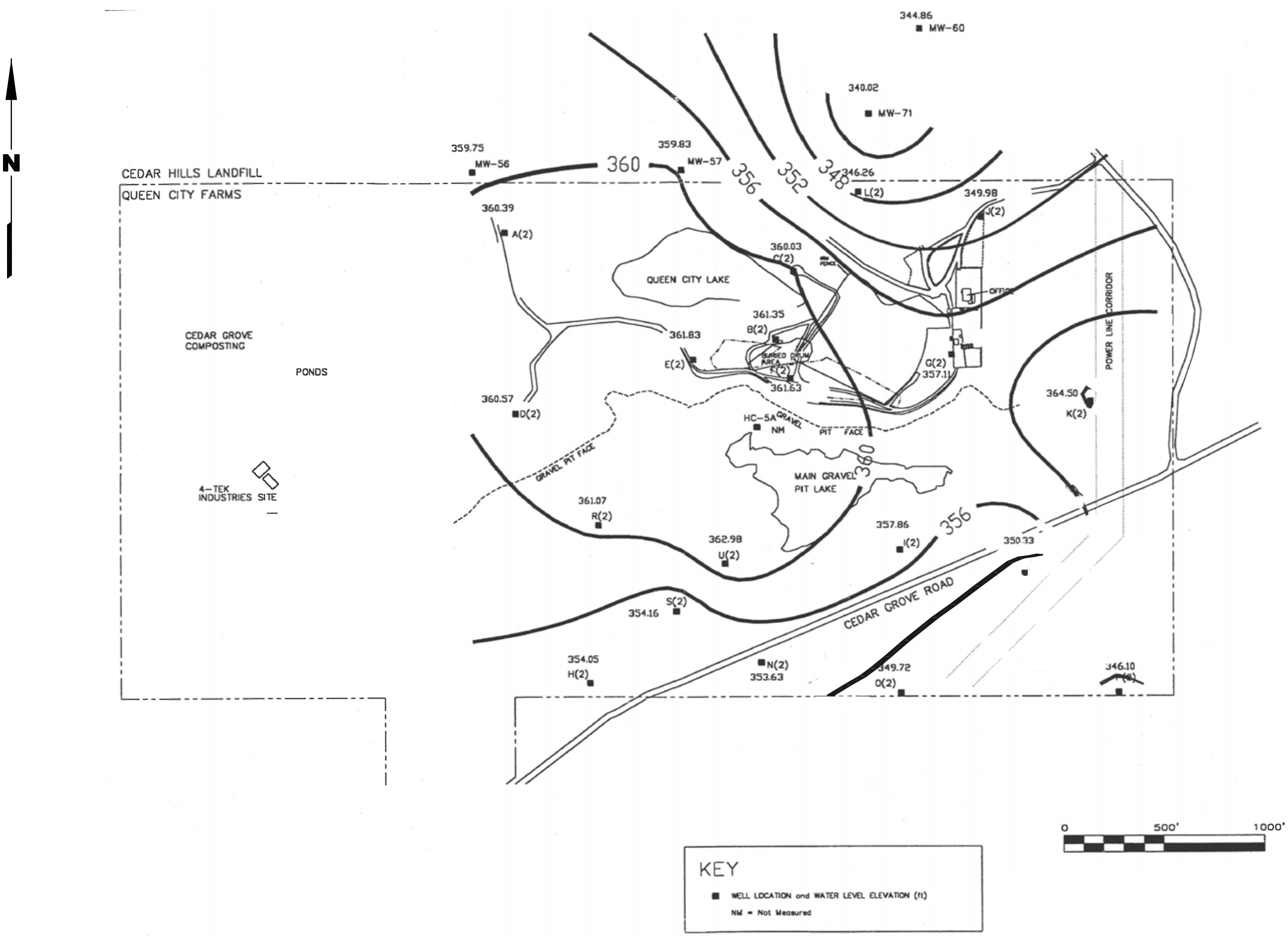
Queen City Farms Refill Project Maple Valley, Washington	**Conceptual Model of Recharge and Discharge for Aquifer 1**	Figure 25



Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES

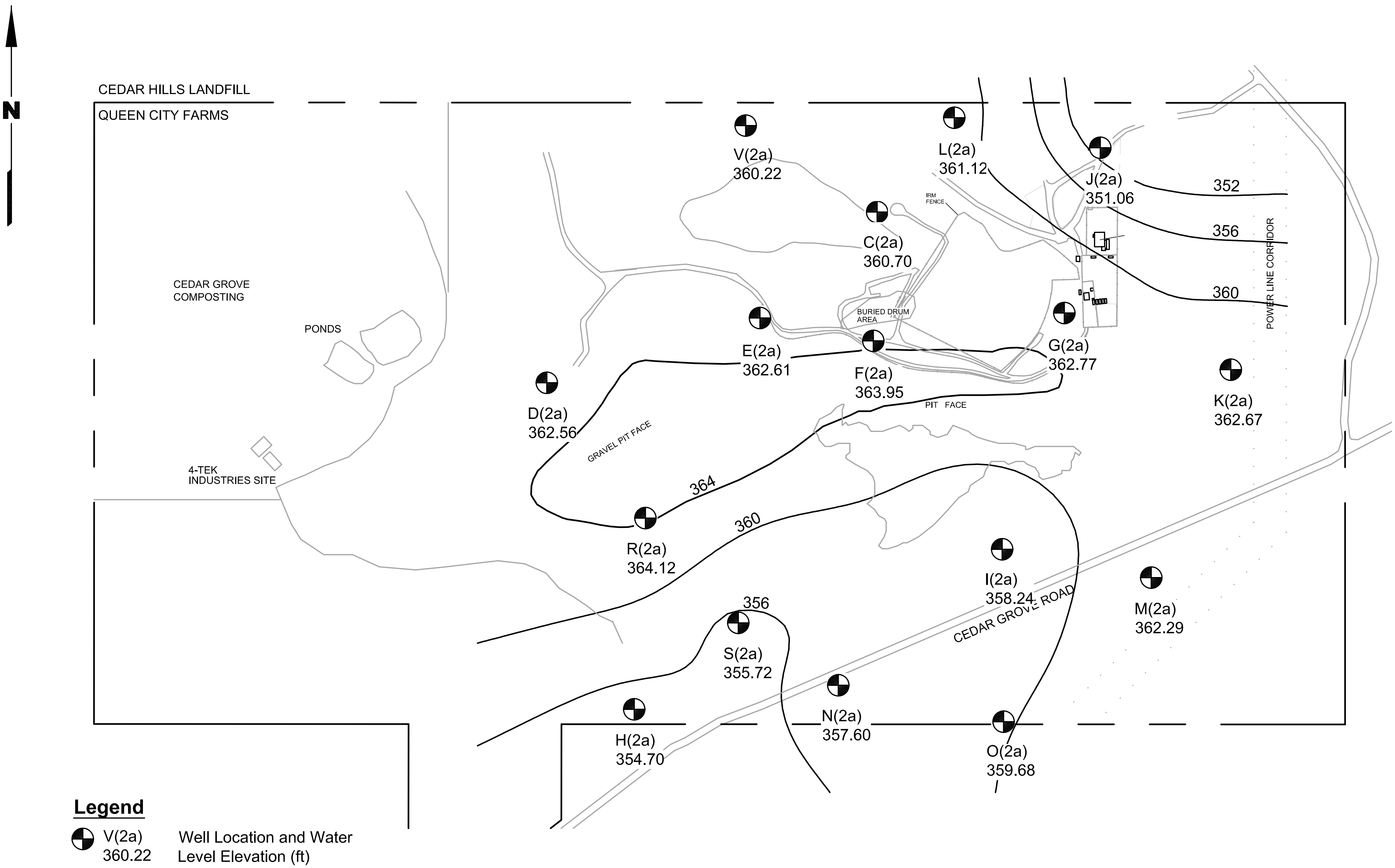
Queen City Farms Refill Project Maple Valley, Washington	**Aquifer 1 Response to Water Depth in Queen City Lake 11/3/1987 to 12/31/1988**	Figure 26



Source: Queen City Farms RI Report, 1998

Queen City Farms Refill Project Maple Valley, Washington	**Aquifer 2 Water Level Contours** **May 1997**	Figure 27

LANDAU ASSOCIATES



Base map source: Boeing, 1998

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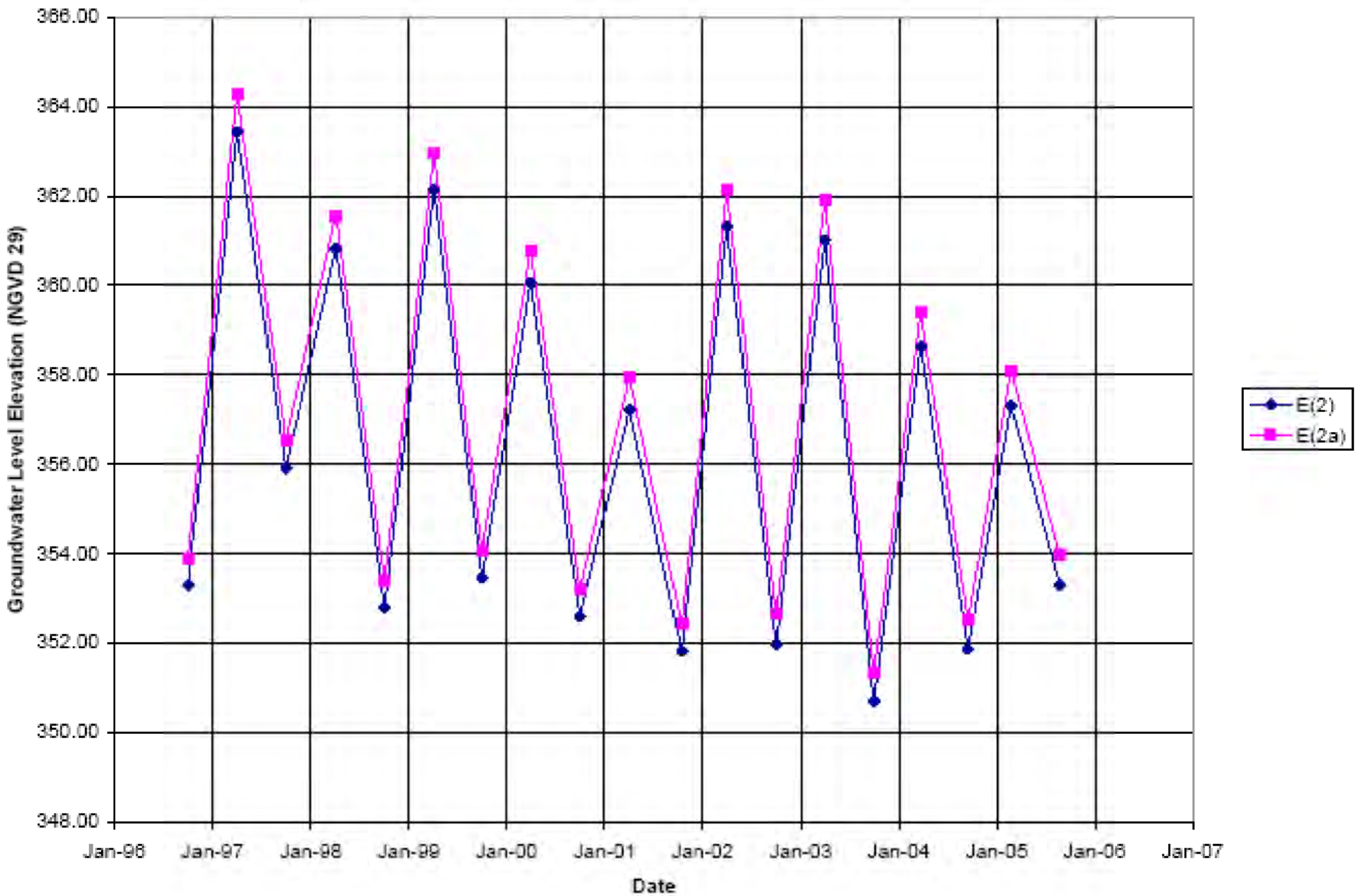
Queen City Farms
Refill Project
Maple Valley, Washington

Upper Aquifer 2
Water Level Contours
May 1997

Figure
28



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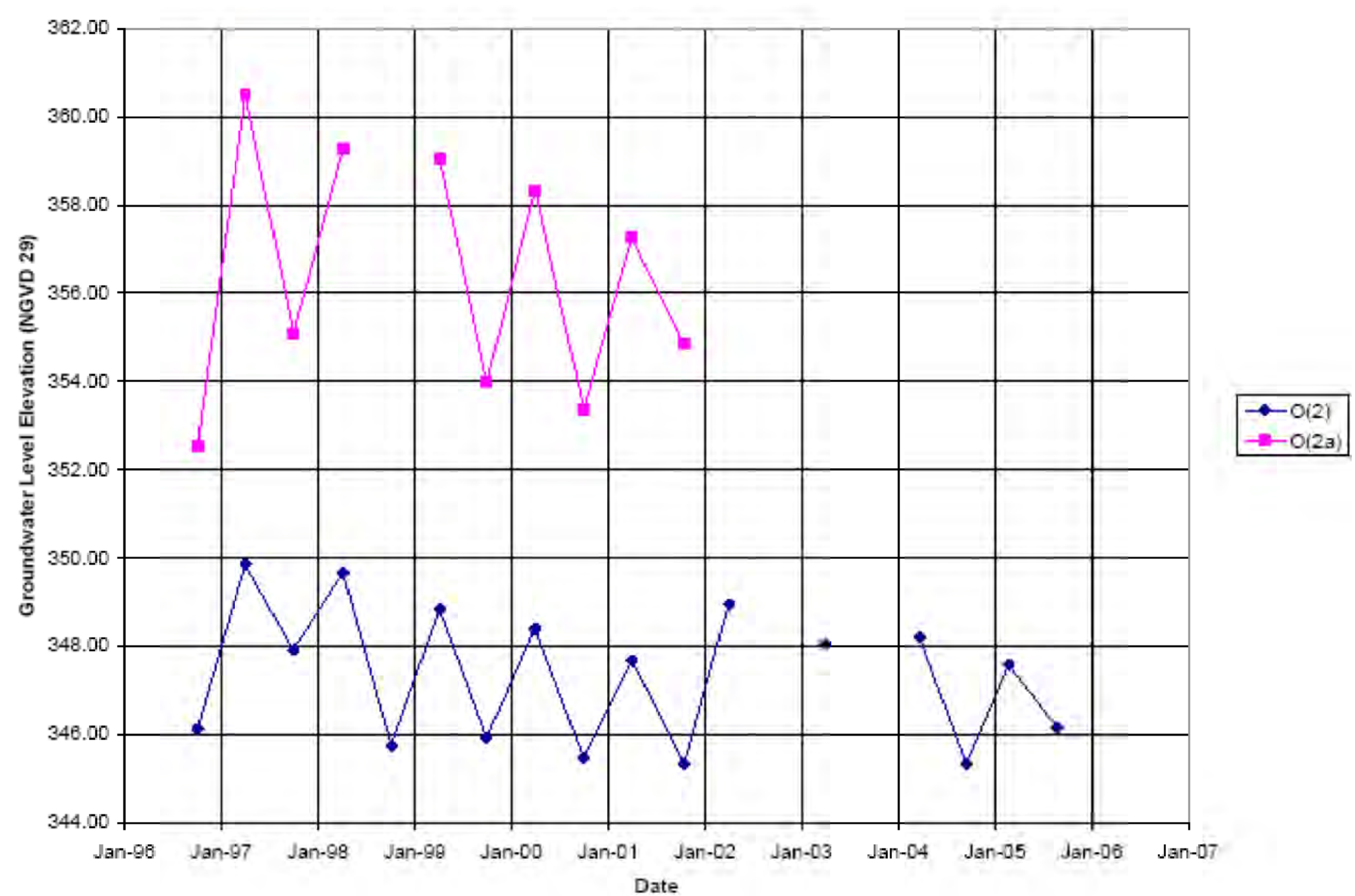
Queen City Farms Refill Project Maple Valley, Washington	**Aquifer 2 Hydrograph: Wells Located South of Queen City Lake**	Figure 29



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Queen City Farms
Refill Project
Maple Valley, Washington

Aquifer 2 Hydrograph: Wells Located in Cedar Grove Channel

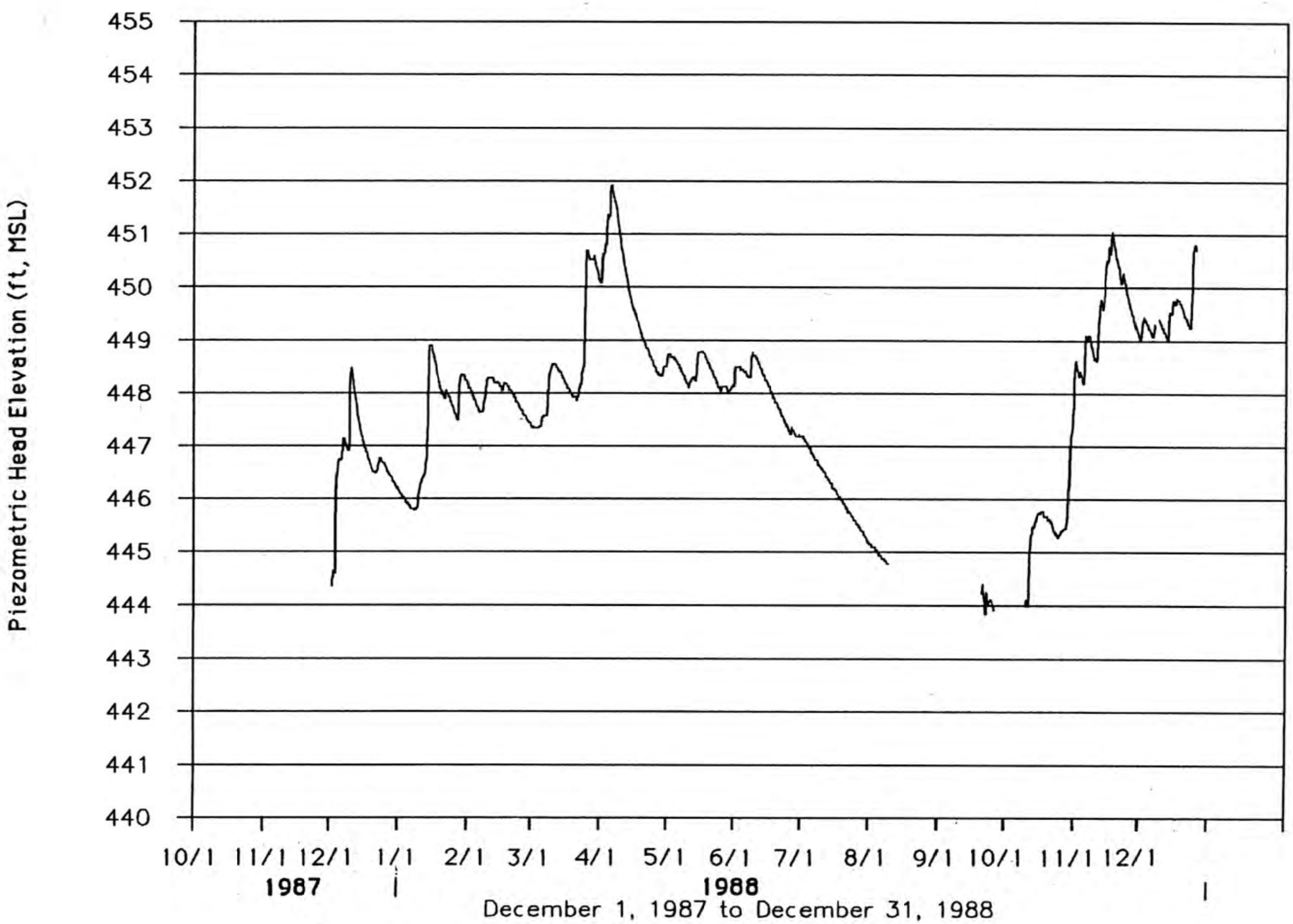
Figure 30

ATTACHMENT 1

Previous Studies

ATTACHMENT 2

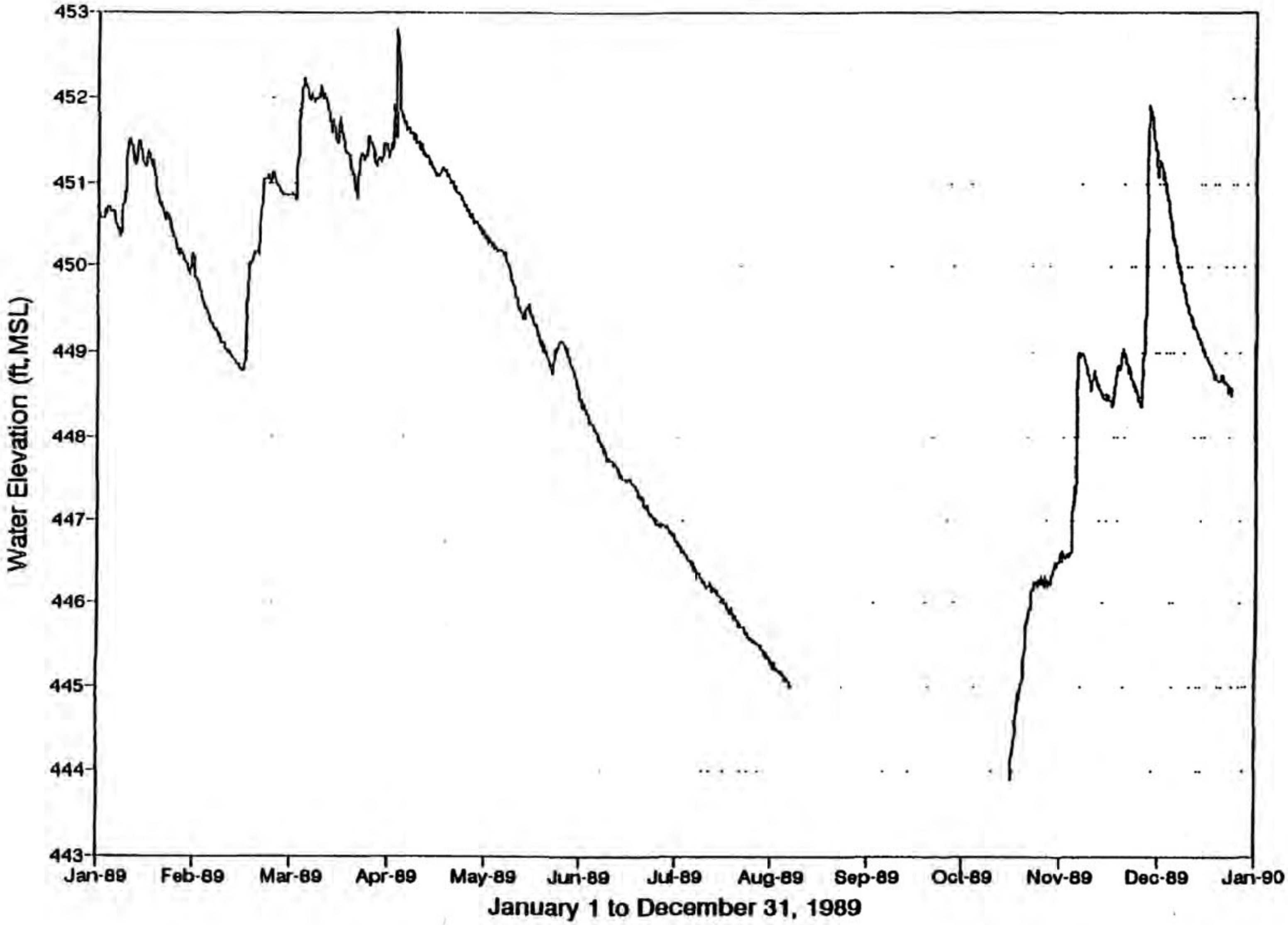
Queen City Lake, 1987 to 1993, Water Levels (NGVD 29)



LANDAU ASSOCIATES, INC.

Semi-Continuous Water Level Data: Queen City Lake
December 1, 1987 to December 31, 1988

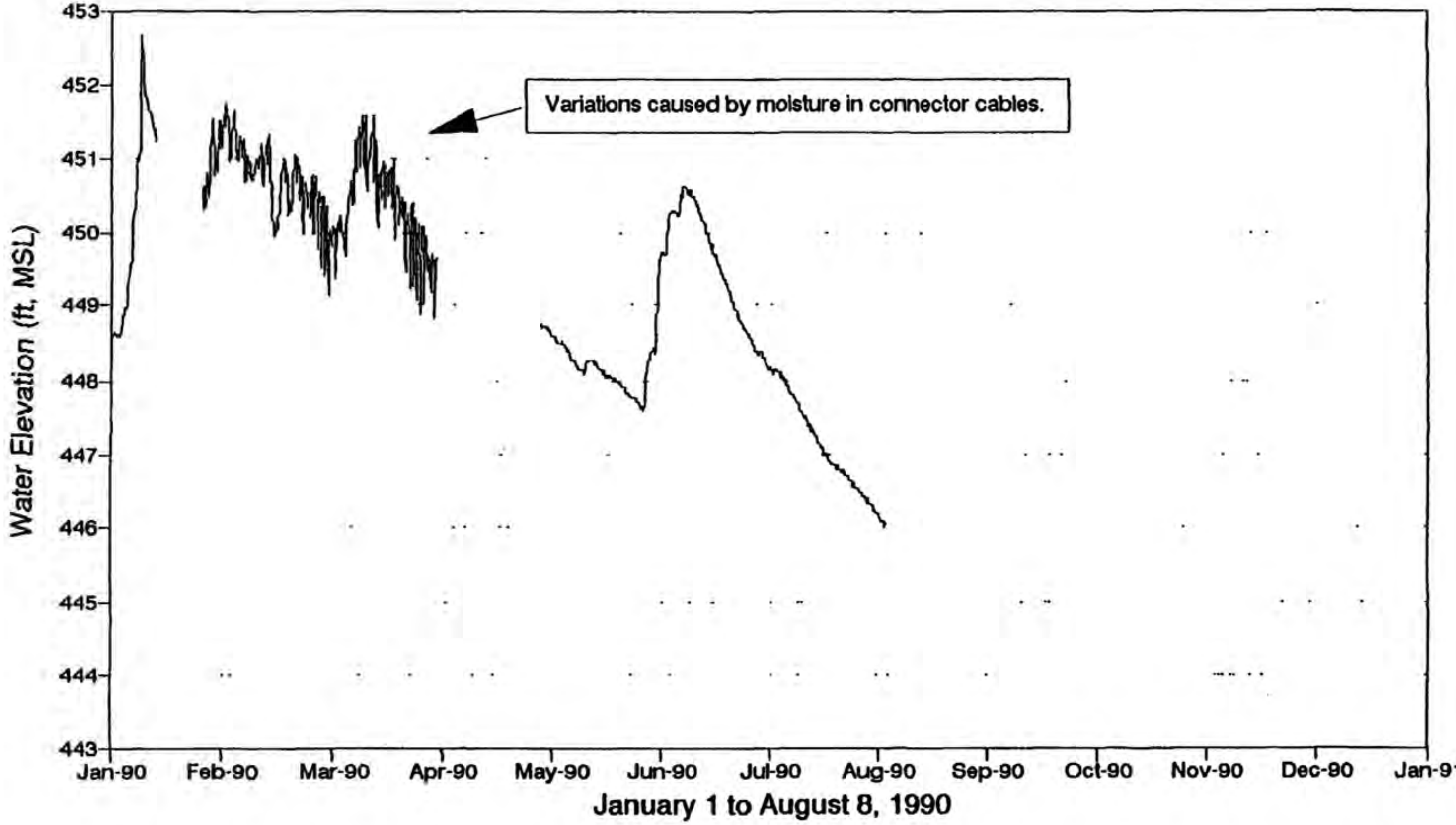
Figure G-3



Note Divisions on horizontal axis correspond to the approximate beginning of each respective month

Semicontinuous Water Level Data: Queen City Lake
January 1 to December 31, 1989

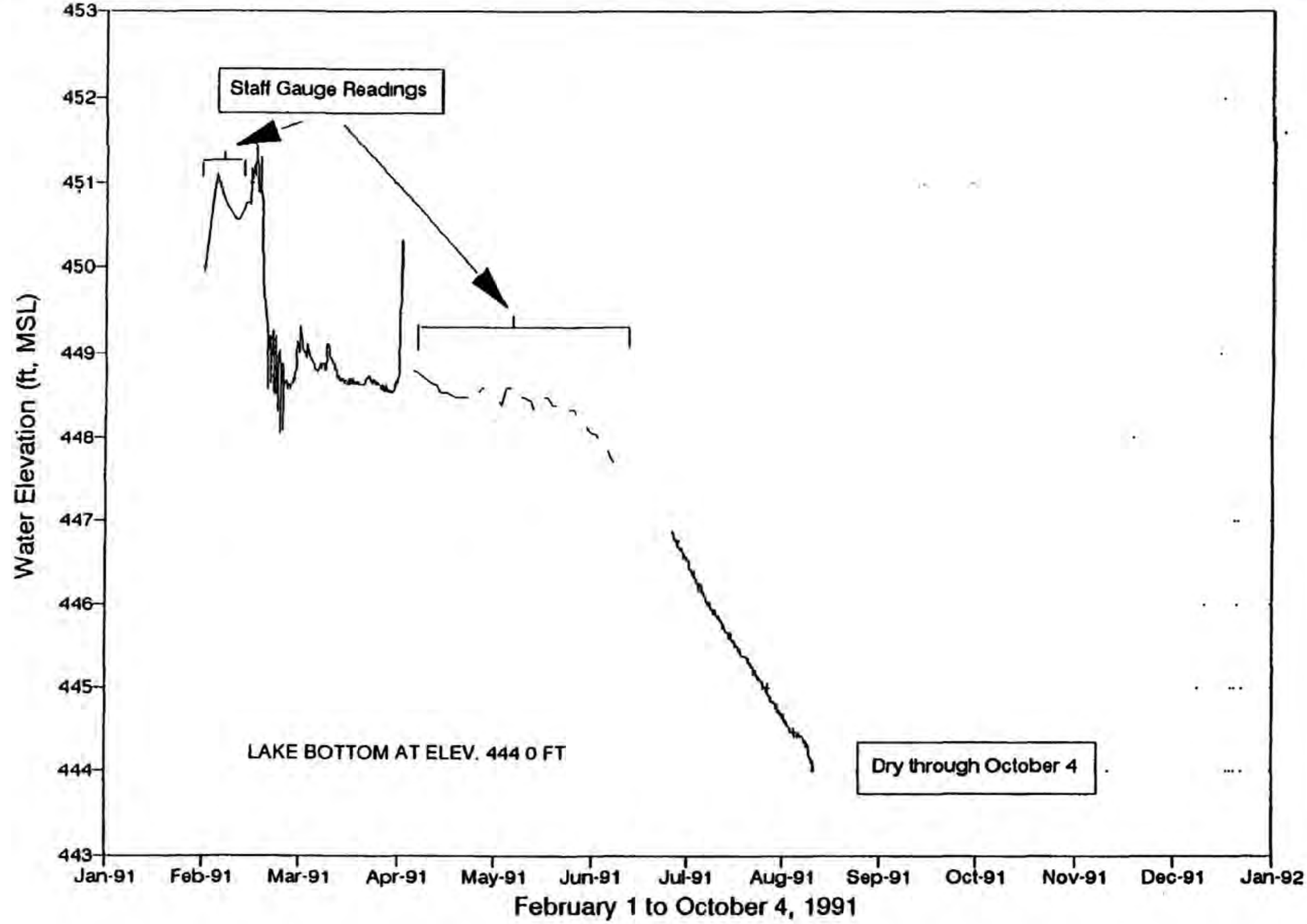
Figure G-6



Note Divisions on horizontal axis correspond to the approximate beginning of each respective month.

Semicontinuous Water Level Data: Queen City Lake
January 1 to August 8, 1990

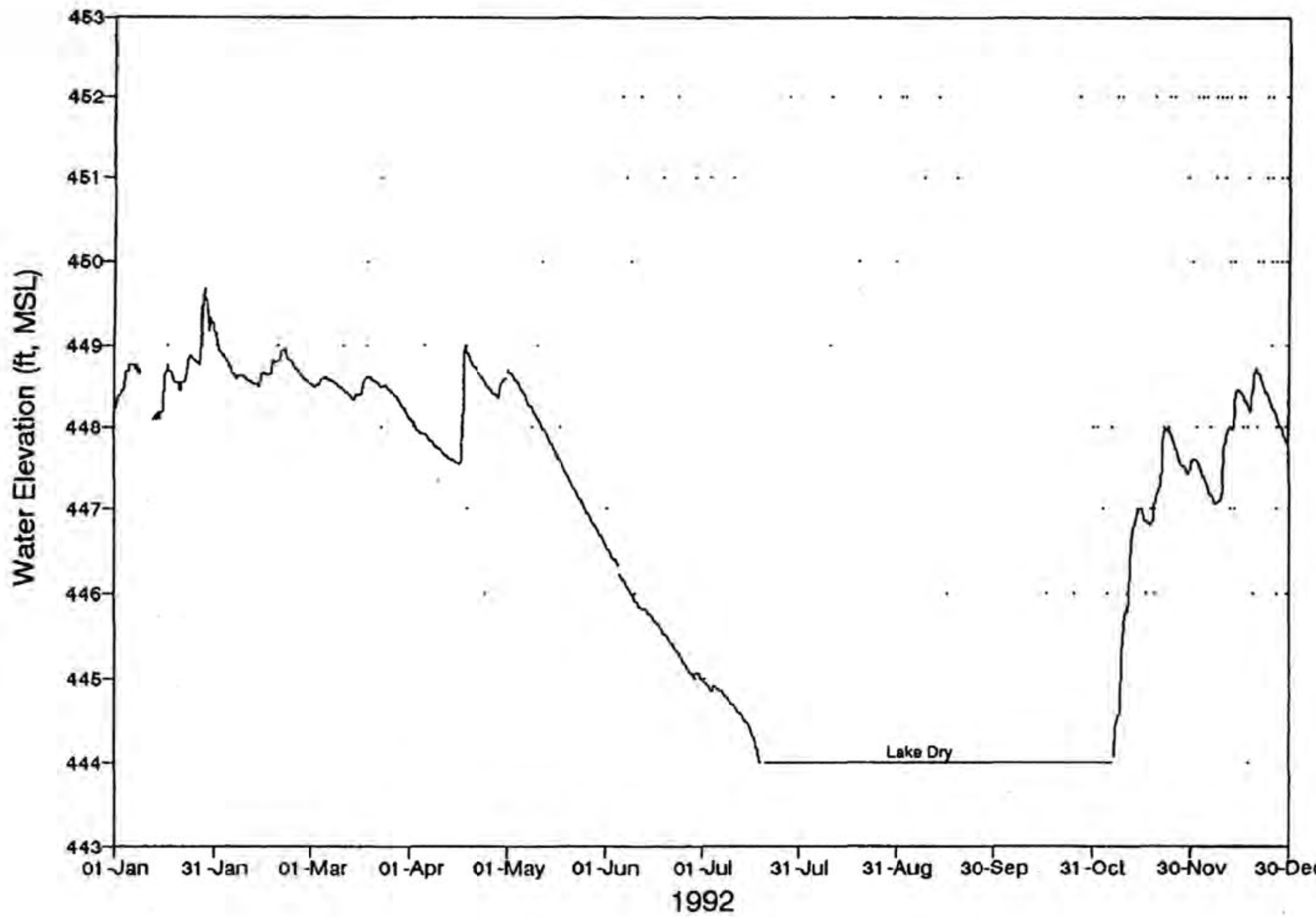
Figure G-7



Note Divisions on horizontal axis correspond to the approximate beginning of each respective month

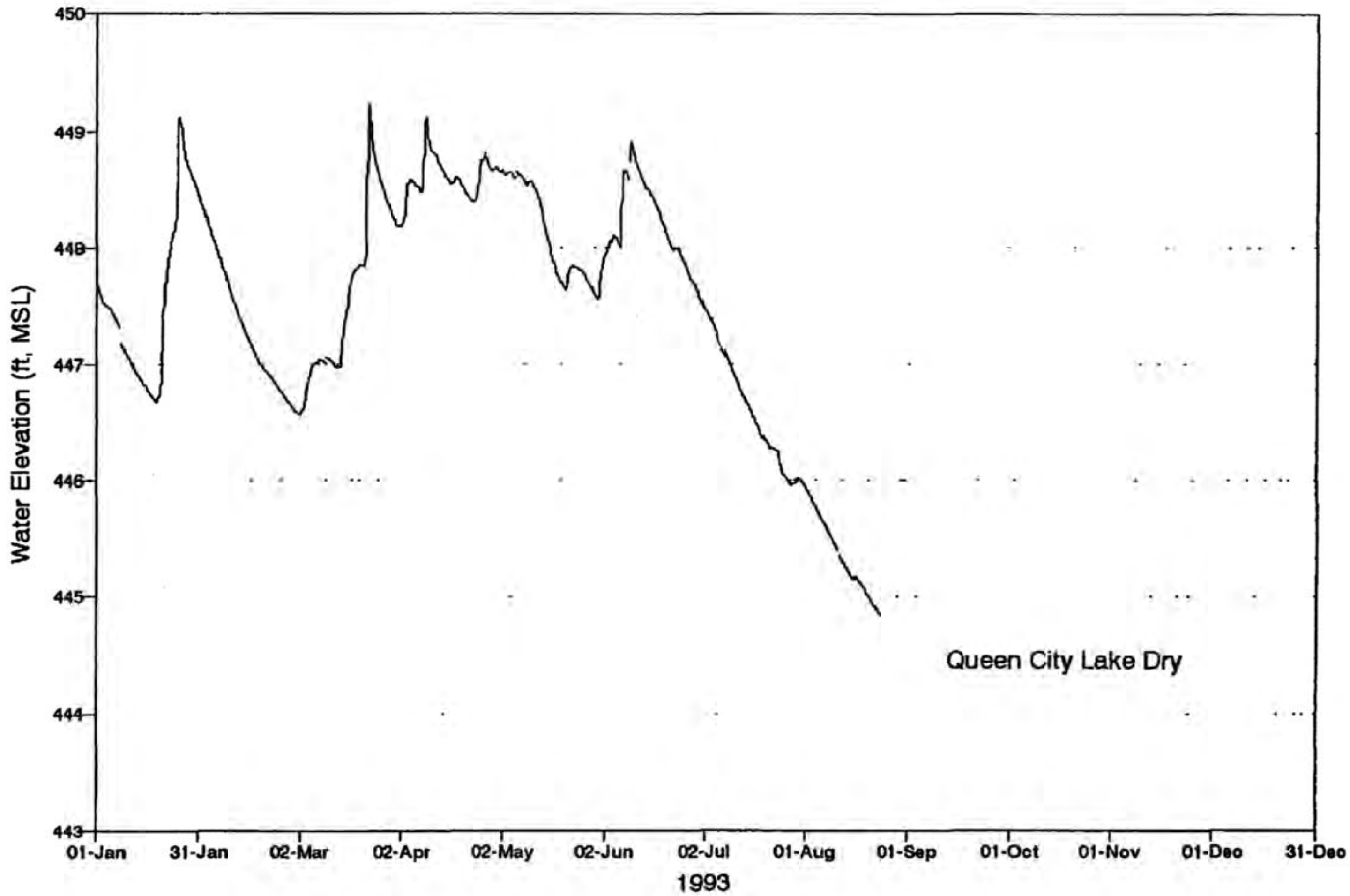
Semicontinuous Water Level Data: Queen City Lake
February 1 to October 4, 1991

Figure G-8



Semicontinuous Water Level Data: Queen City Lake

Figure B-8

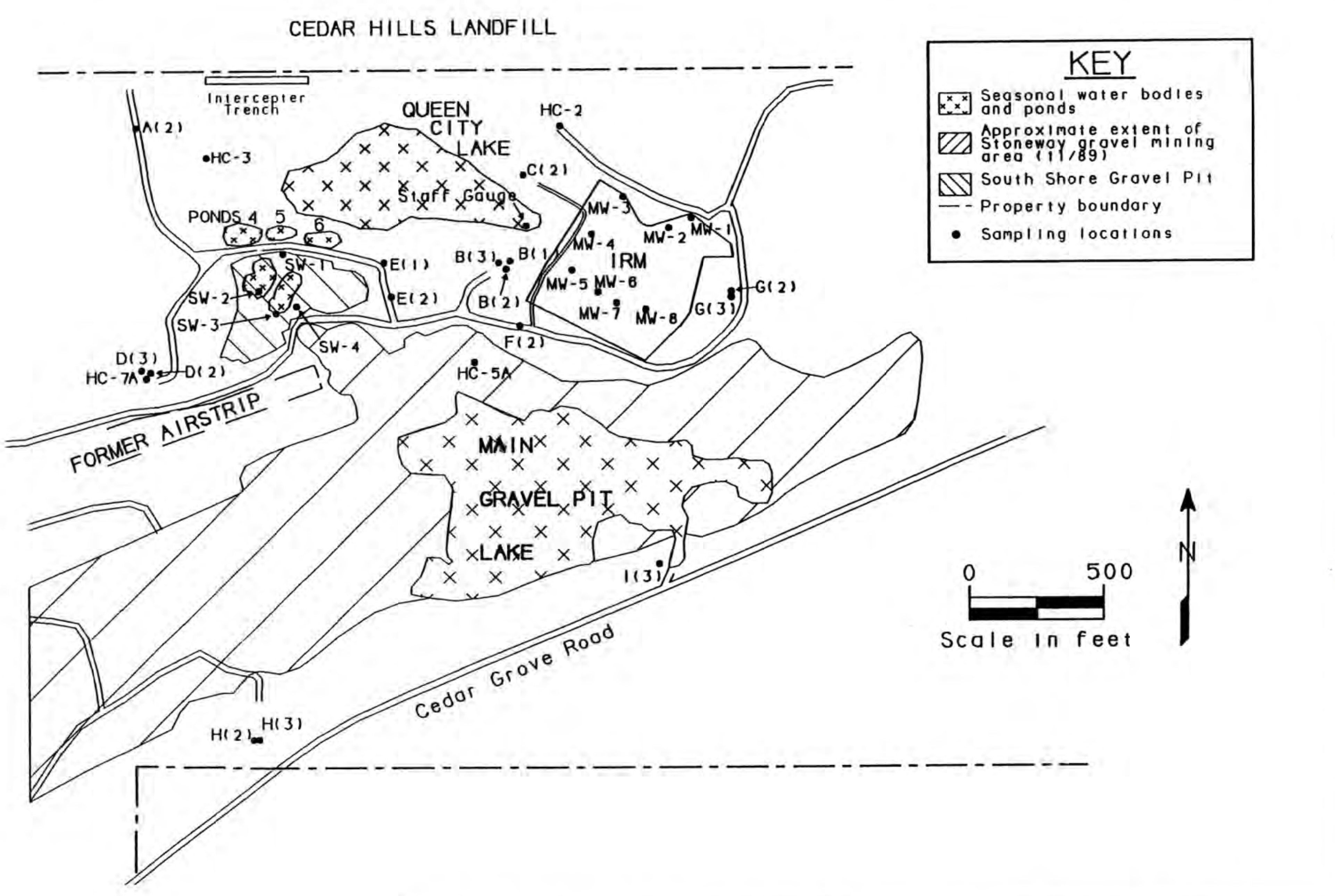


Semicontinuous Water Level Data: Queen City Lake

Figure A-10

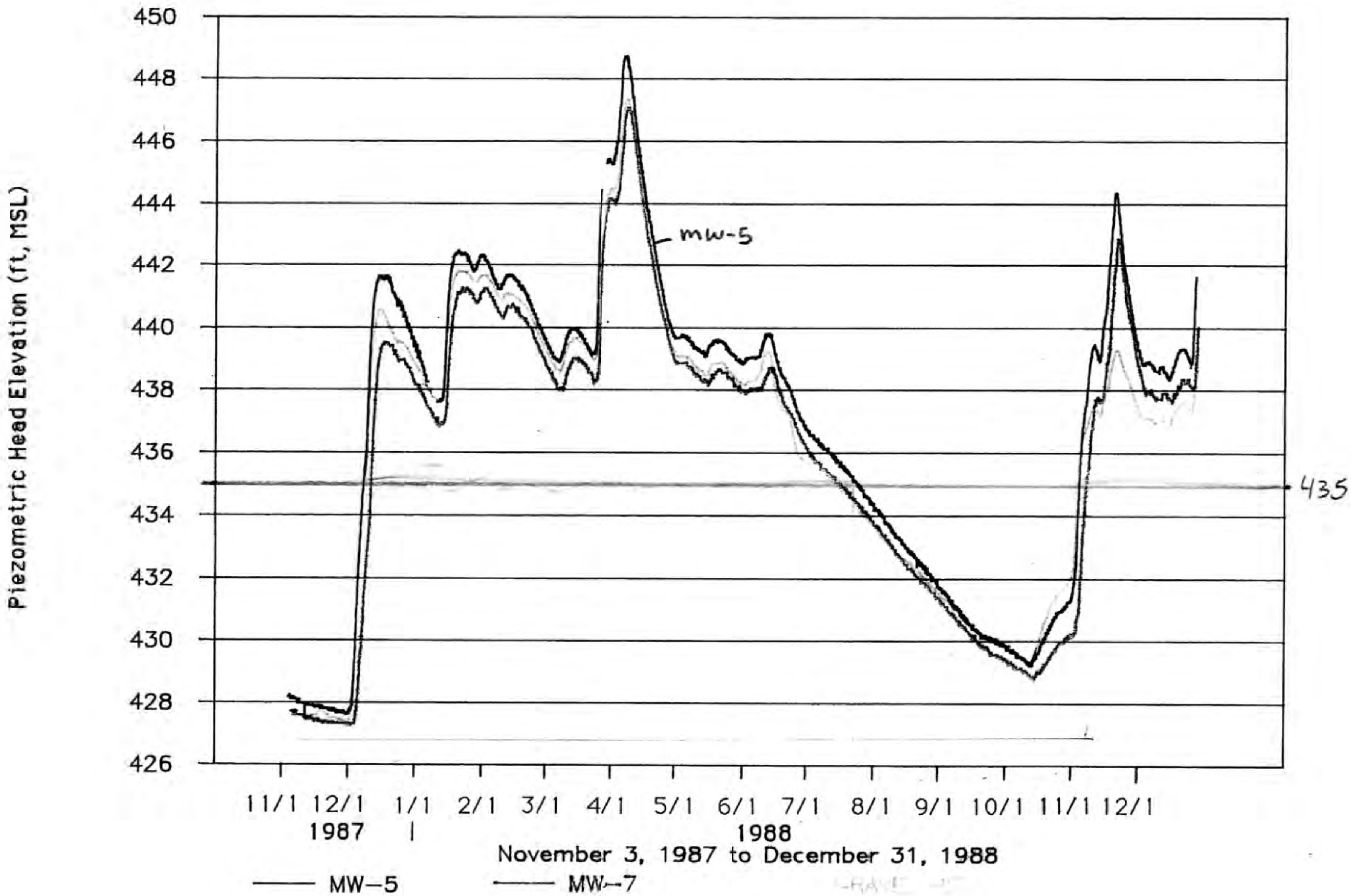
ATTACHMENT 3

Aquifer 1, 1987 to 1993, Water Levels (NGVD 29)



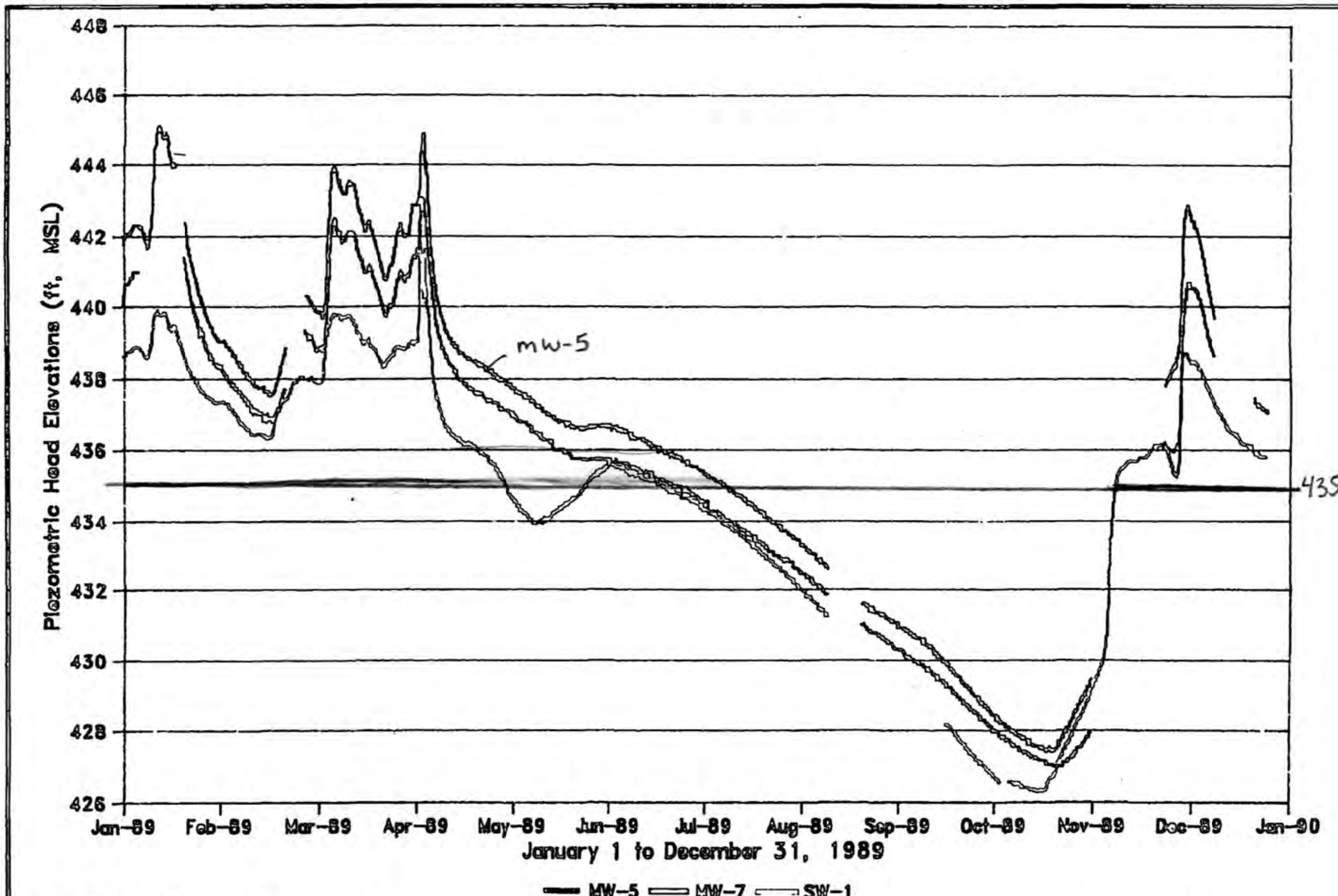
LANDAU ASSOCIATES, INC.	Piezometric Head and Water Level Monitoring Locations on QCF

Figure G-1



LANDAU ASSOCIATES, INC.	Semi-Continuous Piezometric Head Data: SW-1, Wells MW-5 and MW-7, November 3, 1987 to December 31, 1988

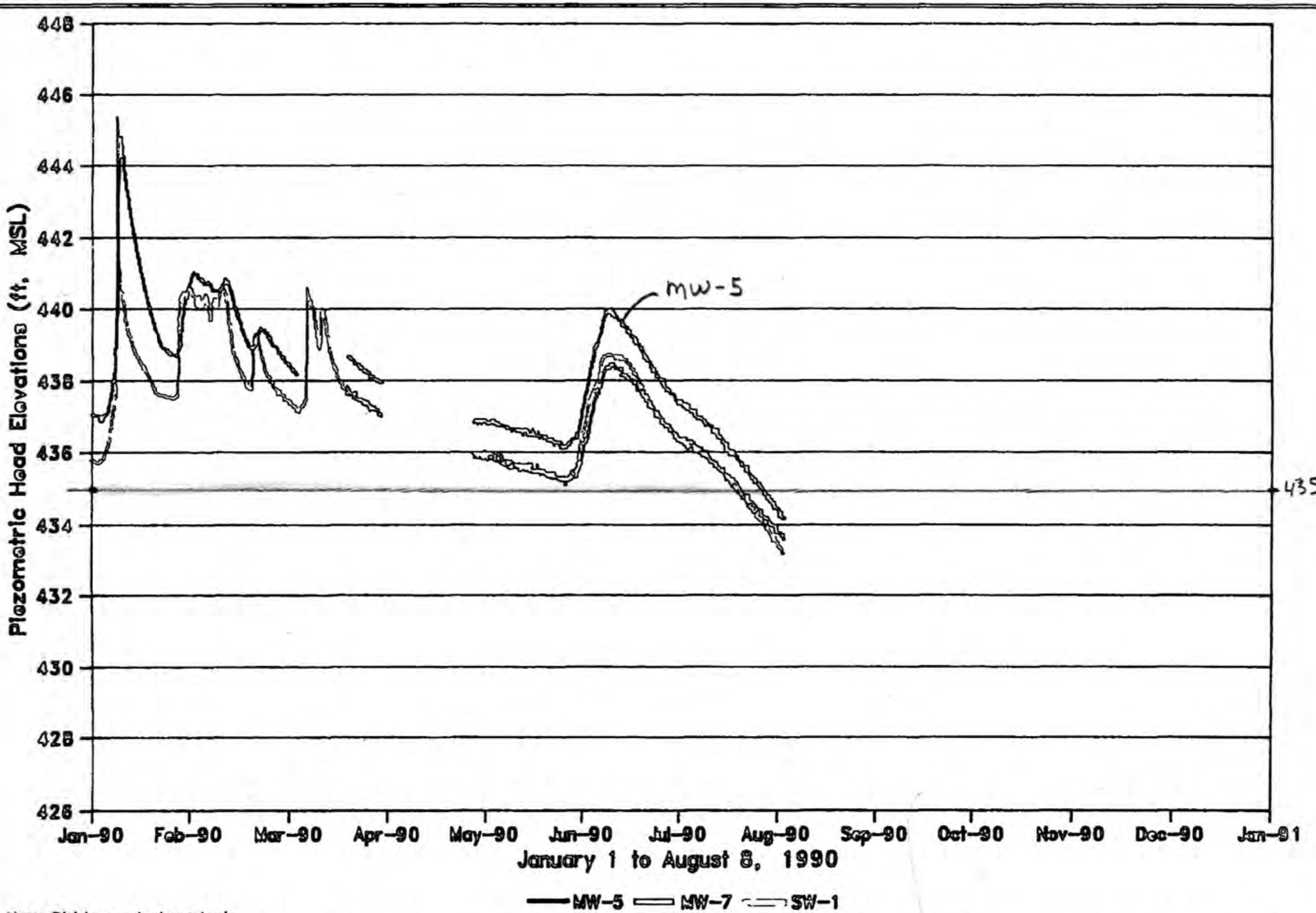
Figure G-7



Note: Divisions on horizontal axis correspond to the approximate beginning of each respective month

Semicontinuous Piezometric Head Data: SW-1 and Aquifer 1 Wells MW-5 and MW-7
January 1 to December 31, 1989

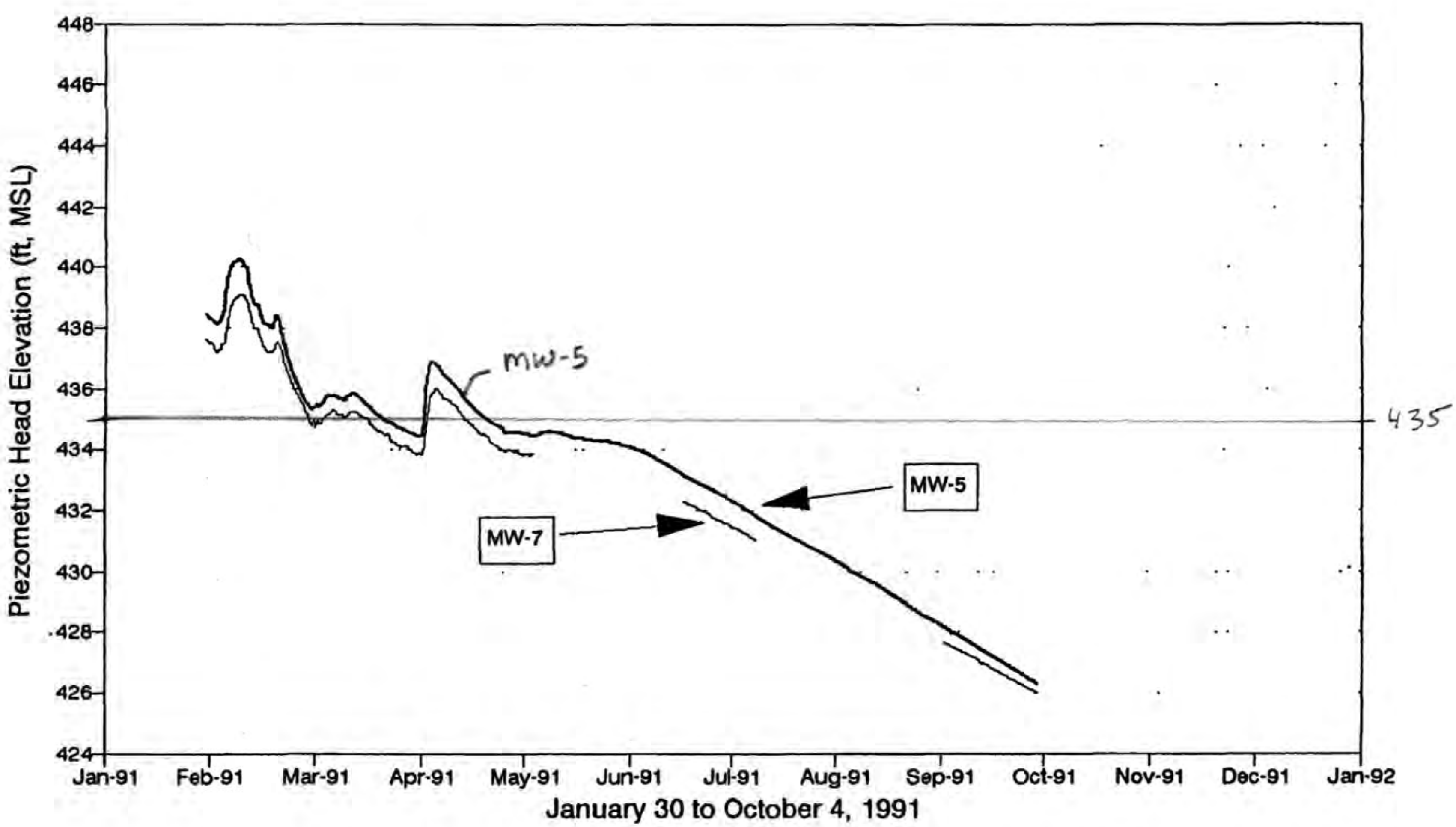
Figure G-9



Note: Divisions on horizontal axis correspond to the approximate beginning of each respective month.

Semicontinuous Piezometric Head Data. SW-1 and Aquifer 1 Wells MW-5 and MW-7
January 1 to August 8, 1990

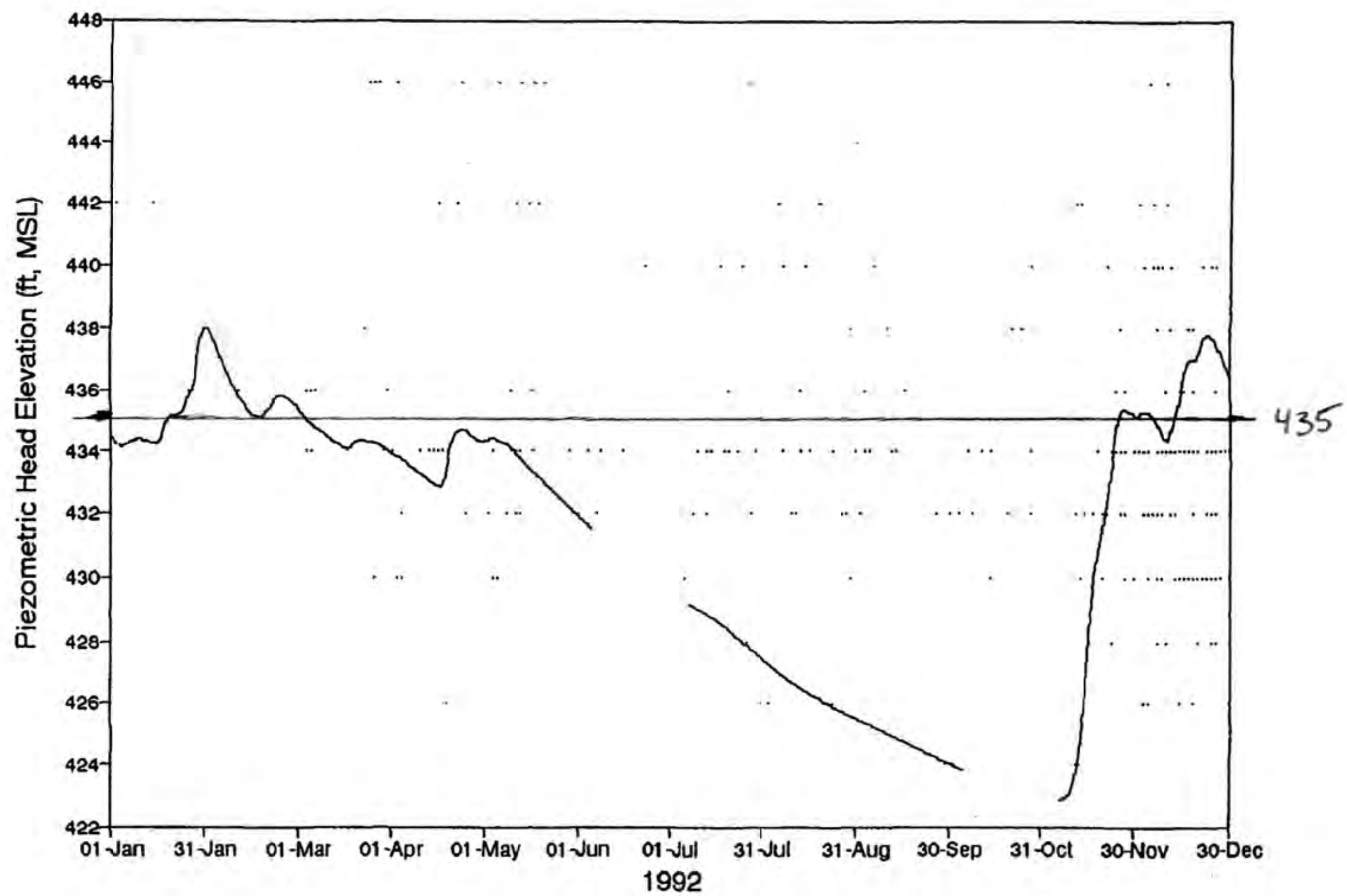
Figure G-10



Note: Divisions on horizontal axis correspond to the approximate beginning of each respective month.

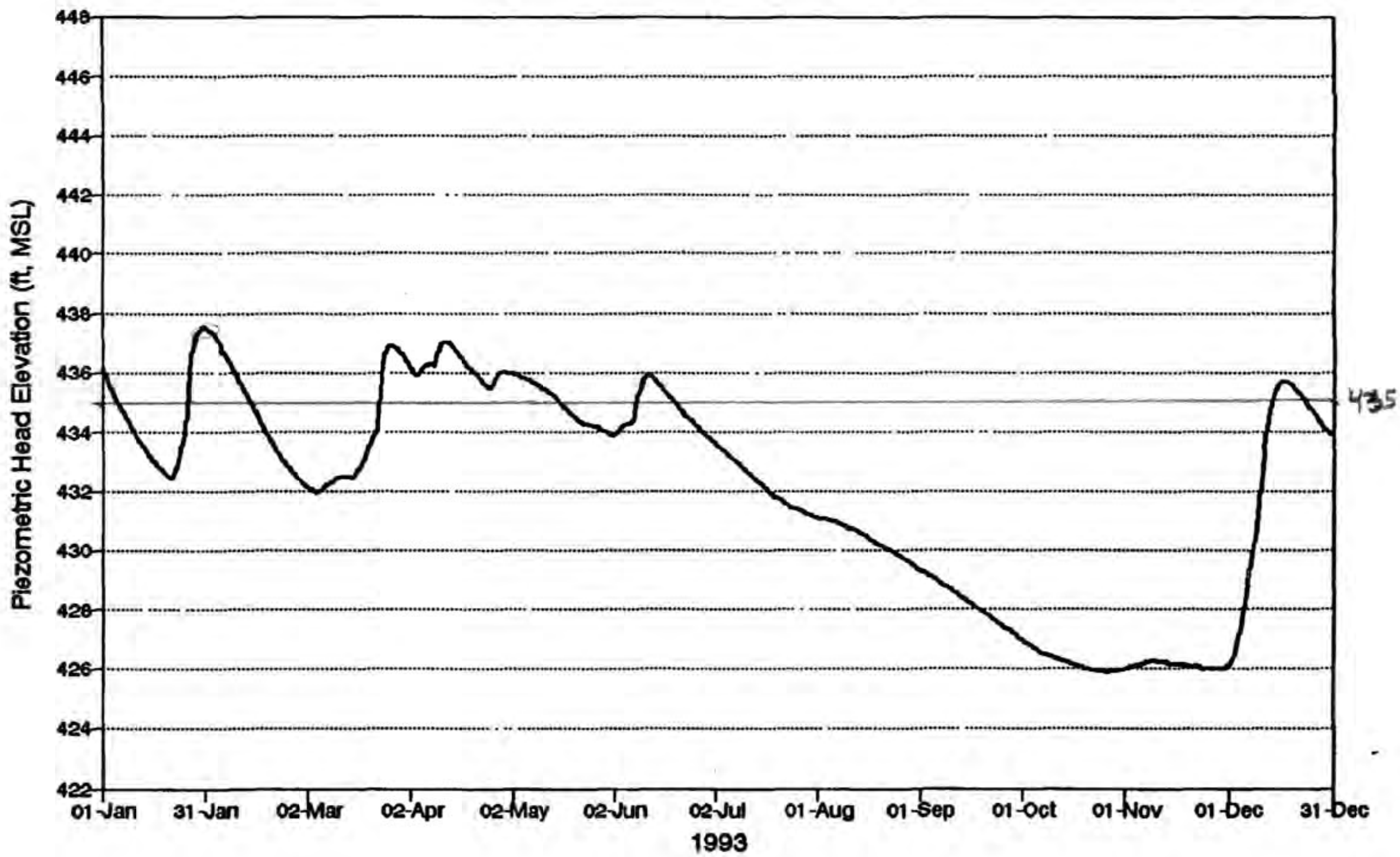
Semicontinuous Piezometric Head Data: Aquifer 1 Wells MW-5 and MW-7
January 30 to October 4, 1991

Figure G-11



Semicontinuous Piezometric Head Data: Aquifer 1, Well MW-5

Figure B-1



Semicontinuous Piezometric Head Data: Aquifer 1, Well MW-5

Figure A-1

Technical Memorandum

TO: Queen City Farms, Inc.

FROM: Eric Weber, LHG and Eve Henrichsen

DATE: November 5, 2018

RE: **Addendum Special Report 6.12**
Restoration of Tributary 316A and Impacts on Existing Wetlands
Cedar Grove Compost Facility
Maple Valley, Washington
Project No. 0992002.050.051

Introduction

This technical memorandum is a supplement to the Phase III grading permit application (PREA17-0003) technical information report (TIR). The permit application and TIR were submitted to King County (County) on April 10, 2018 (LAI 2018). In subsequent conversations, the County requested an evaluation of project impacts on wetlands located along the 316A stream segment that runs between the Cedar Grove Compost (CGC) facility and the finished product storage pad. This report provides an analysis of the expected influence of relocating the stream segment on wetland hydrology and function, based on the information available. The location of the stream segment, associated wetlands, and nearby features are shown on Figure 1.

Background

Queen City Farms, Inc. (QCF) submitted the Phase III permit application as part of gravel mine reclamation. The objective of the reclamation is to restore the topography and site hydrology to pre-mining conditions.

Current Site Hydrology

The hydrology of the subject property is presented in detail in the Technical Memorandum Geology, Surface Water Hydrology, and Groundwater Hydrology (Special Report 6.2[1]). Three drainage sub basins exist that, together, cover the CGC proposed finished project storage pad project area. The Queen City Lake drainage sub basin consists primarily of the southern portion of the Cedar Hills Regional Landfill (CHRL) and Queen City Lake. The Main Gravel Pit Lake (MGPL) drainage sub basin consists of areas that discharge directly to the lake; and the Maple Hills drainage sub basin consists of areas that flow into Tributary 316A and eventually infiltrate in the Main Infiltration Area (Special Report 6.2).

All three of these drainage sub basins are tributaries to the Cedar River. Surface water in all three drainage sub basins currently infiltrates to groundwater in either Wetland A (i.e., Queen City Lake),

[1] All special reports referenced in this technical memorandum refer to reports presented in the Technical Information Report for the Queen City Farms Phase III Refill Project (LAI 2018).

the Main Infiltration Area or the Main Gravel Pit Lake (Special Report 6.2). Tributary 316A historically, was a natural drainage channel that discharged to Wetland A or infiltrated into soil adjacent to Wetland A. The stream channel was re-routed to discharge to a forested area west of the current infiltration facility in the late 1980s. In the early 1990s, the lower portion of the stream, south of the compost facility, was rerouted again to its current location, where it discharges to a natural infiltration area (Main Infiltration Area) located approximately 200 feet (ft) north of Cedar Grove Road. The channel is designed to carry stormwater from the north of the property to the Main Infiltration Area. A historical summary of the attributes of 316A are provided in Special Report 6.6.

Stream Segment History

Historically, the intermittent drainage referred to as 316A flowed south off the upland drift plain and infiltrated near the west end of Queen City Lake. During mining operations, permeable gravel was removed, eventually creating surface water flow that extended across the pit floor. The flow was diverted multiple times to accommodate gravel mining and most recently, construction of the CGC manufacturing facility. The current channel is an engineered conveyance that includes rip-rap, culverts, and check dams. A detailed history of the 316A hydrology and channelization is presented in Section 6.6 of the TIR.

Stream Segment Hydrogeology

Tributary 316A drains an upland area that is known as the Coalfield drift plain. The drift plain is mantled by a relatively thick sequence of very dense and low permeability lodgment till. The presence of till results in excess precipitation flowing laterally as surface water or near-surface water toward low points in the topography such as the original 316A channel. The hydrology of this system is described in more detail in Special Report 6.2.

Surface flows of 316A infiltrated at the west end of Queen City Lake. This is because the depth to the low permeable till layer increased and the near surface soils were overlain by a thick sequence of permeable deltaic gravel deposits that are characteristic of the west end of Cedar Grove Channel (Special Report 6.2). Much of these gravel deposits have been mined away.

Current Conditions

Description of current conditions is based on information from the original wetland and stream delineation in 2007 and 2008, observations made in subsequent site visits completed in 2012 through 2018, and observations reported by site personnel.

Tributary 316A

Tributary 316A is a Type O water, meaning that it has no fish presence, and no surface connection to Type S, F, or N waters per King County Code (KCC; Chapter 21A.24.355).

Stream Segment Hydrology

The current 316A stream segment flows through a channel that was constructed in the early 1990s to route the previously channeled drainage around the newly built CGC facility. The till in the vicinity of the stream segment is about 50 ft thick (Special Report 6.2). The stream segment channel was constructed to be incised into the till.

Water gauge data is not available for Tributary 316A; however, there are a number of personal observations recorded during field visits. According to site personnel, the stream typically only flows during the winter and early spring. Field observations in 2007 and 2008 were documented in the Cedar Grove Compost Finished Compost Pad TIR (LAI 2015). There was no precipitation leading up to or during the February 2008 site visit and the stream was flowing freely. At the time of the June 2007 site visit, there was heavy rain and some ponding in depressions within the streambed was observed; however, the stream was not flowing. No flow was present during recent August or September 2018 site visits.

Instream Characteristics

The ordinary high water mark (OHWM) of the channel ranges from 10 to 12 ft wide (LAI 2015). The northern portion of the stream, between approximately Wetlands B and E, has a single channel, while the channel bed is less clearly defined in the southern portion of the study area near Wetland F. The substrate of the stream channel consists of medium to large angular to rounded cobbles (quarry spalls) with patches of vegetation interspersed, reflecting the artificial nature of the stream bed. Little to no fine-grained material was present in the stream channel. Vegetation found in the stream channel includes reed canarygrass (*Phalaris arundinacea*), American speedwell (*Veronica americana*), climbing nightshade (*Solanum dulcamara*), and saplings of red alder (*Alnus rubra*) and black cottonwood (*Populus balsamifera*).

Riparian Area

Riparian areas are vegetated and generally forested. Portions of the riparian area are wetland. Canopy vegetation in the riparian areas adjacent to the compost facility consists primarily of red alder and some black cottonwood. The understory consists of thick mats of Cleaver's bedstraw (*Galium aparine*) along with Robert geranium (*Geranium robertianum*) trailing blackberry (*Rubus ursinus*), Himalayan blackberry (*Rubus armeniacus*), stinging nettle (*Urtica dioica*), bracken fern *(Pteridium aquilinum*), sword fern (*Polystichum munitum*), and mixed grasses. Riparian areas in the portion of the stream south of the compost facility are dominated by willow species (shrubs to small trees; *Salix* spp.) (LAI 2015).

Within the compost facility, the stream bank on the west side of the stream is very steep and transitions to a hillside that slopes up to the facility. The bank on the east side of the stream is relatively steep, but short (1 to 3 ft high) in the northern portion of the study area. In the southern portion of the stream, the bank slopes down (up to 5 ft near Wetland D) or is flat (near Wetland F).

Soil in the riparian areas was moist during both site visits, but no groundwater or saturation was present at 12 inches below ground surface (bgs) in the northern portion of the study area. In the southern portion of the study area, soil was saturated outside of the stream channel and wetland corridors were present (Wetland F). The stream channel was sparsely vegetated with plant species typically adapted to wetland environments. A vegetation shift was present along the stream banks where upland herbaceous species became prevalent.

Wetlands

The following section describes wetlands located in the vicinity of Tributary 316A stream segment.

Wetland B

Wetland B is a 10,706 square foot (sf) riverine/depressional (hydrogeomorphic (HGM) classification seasonally saturated to temporarily inundated, palustrine emergent and forested wetland (Brinson 1993; Cowardin et al. 1979). Wetland B is a Category III wetland, based on Ecology's 2004 Rating System (Hruby 2006).

Wetland B currently receives hydrologic inputs from groundwater interflow and seasonal overbank flooding from Tributary 316A. Wetland B borders Tributary 316A for about 75 ft on its western edge. Hydroperiods present within the wetland include areas that are seasonally flooded, occasionally flooded, saturated only, and seasonally flowing stream adjacent to the wetland.

Wetland B has potential to provide water quality and habitat functions to a moderate extent and hydrologic functions to a low extent. The wetland also has the opportunity to provide these functions (including moderate opportunity to provide habitat functions).

The wetland has moderate potential for removing nutrients, metals, and toxins from overland flow associated with flooding of Tributary 316A due to the presence of temporarily saturated areas and organic soil. The wetland has a moderate potential for reducing peak flows and downstream erosion on the site due to the presence of depressions; however, other areas, such as the Main Infiltration Area and Wetland 31 (located downgradient of the wetland and stream, provide this function to a greater degree (surface water from Wetland B or Tributary 316A does not directly flow into any stream or river with flooding problems). The general habitat suitability of the wetland is moderate, as it is located adjacent to a stream, and has moderately extensive undisturbed buffers that connect to other wetlands and uplands. However, the wetland and stream have insufficient hydroperiod to provide habitat for aquatic associated or dependent species.

Wetland C

Wetland C is a 360 sf depressional, palustrine scrub-shrub wetland with hydrology that ranges from seasonally saturated to temporarily inundated. Wetland C is a Category IV wetland based on Ecology's 2004 rating system (Hruby 2006).

Hydrology in Wetland C is supported by surface runoff from the surrounding area and precipitation and groundwater interflow. Hydroperiods present within the wetland include areas that are seasonally inundated, occasionally inundated, and saturated only.

Wetland C has potential to provide water quality and hydrologic functions to a moderate or low extent; however, it lacks opportunity for both. These functions are generally limited because the wetland is very small and lacks connectivity to other surface waters. The wetland has low potential to provide habitat functions, but has moderate opportunity due to its location near other wetlands, streams, and upland buffers.

Wetland D

Wetland D is a 16,500 sf depressional system containing scrub-shrub and forested areas that range from seasonally saturated to permanently ponded. Wetland D is a Category III wetland based on Ecology's 2004 Rating System (Hruby 2006). A portion of this wetland has been rehabilitated as part of mitigation for critical area impacts associated with the construction of the new compost pad.

Hydrology in Wetland D is supported by groundwater interflow and surface runoff from the surrounding area. It may seasonally receive subsurface flow from Tributary 316A during the winter and spring. Hydroperiods present within the wetland include areas that are seasonally flooded or inundated, occasionally inundated, and saturated only.

Wetland D has moderate potential to provide water quality functions, habitat functions, and hydrologic functions. Wetland D has the potential to provide water quality functions, but lacks the opportunity to provide hydrologic functions.

The wetland has moderate potential for removing nutrients, metals, and toxins from overland flow due to the presence of seasonally saturated areas. The wetland has a moderate potential for water storage due to a few deep depressions; however, it does not have a surface connection to streams or other wetlands. The general habitat suitability of the wetland is moderate, as it is located adjacent to a stream, and has moderately extensive undisturbed buffers that connect to other wetlands and uplands. However, the wetland and stream have insufficient hydroperiod to provide habitat for aquatic birds, mammals, or fish. Primary production and export is not provided because the wetland has no outlet.

Wetland F

Wetland F is currently a 38,754 sf riverine system containing emergent, scrub-shrub, and forested areas that range from seasonally saturated to seasonally inundated. Wetland F is a Category III wetland based on Ecology's 2004 Rating System (Hruby 2006). Portions of Wetland F and its buffer were enhanced as part of compensatory mitigation efforts for impacts related to the compost pad construction. A portion of Wetland F, its buffer and the Tributary 316A channel were recently cleared and this action is under advisement under Code Enforcement #ENFR18-0684.

Wetland F is associated with Tributary 316A and is located south of Wetland D, between the existing compost pad and an equipment storage area (Figure 1).

Hydrology in Wetland F is supported by groundwater interflow and overbank flooding from Tributary 316A during winter and early spring. The primary source of hydrology is the stream, which flows through the wetland with little or no streambank to contain it. The portion of the stream flowing through Wetland F does not exhibit a distinct channel. While a bed has been constructed with angular quarry spalls, there is a lack of incised banks. Hydroperiods present within Wetland F include areas that are seasonally inundated, saturated only, and seasonally flowing stream in the wetland.

Wetland F provides water quality functions to a moderate extent, and has the opportunity to do so. It has potential to provide habitat functions to a moderate extent; however, the opportunity is low. While the wetland has potential to provide hydrologic functions to a high extent, it does not have opportunity to do so.

The wetland has moderate potential for removing nutrients, metals, and toxins from overland flow associated with flooding of Tributary 316A due to the presence of seasonally saturated areas and organic soil. The wetland has some potential for reducing peak flows and downstream erosion on the site due to presence of small depressions; however, other areas, such as the Main Infiltration Area and Wetland 31 (both located downgradient of the wetland and stream), provide this function to a greater degree (surface water from the Wetland F or Tributary 316A does not directly flow into any stream or river with flooding problems). The general habitat suitability of the wetland is moderate, as it is located adjacent to a stream, and is connected to other wetlands and uplands via the stream. The wetland and stream have insufficient hydroperiod to provide habitat for aquatic birds, mammals, or fish.

Future Conditions

Hydrology

After the completion of Phase III restoration activities, hydrology on the site will mimic pre-mining conditions. The restored Tributary 316A will flow east toward Queen City Lake instead of turning south at the compost pad. The restoration will channel flow eastward and end with a channel outlet spreader at the Queen City Lake wetland buffer. Surface flows will disperse through forested upland buffer and pond in Queen City Lake, where water will infiltrate (Plan Sheet 3).

The abandoned Tributary 316A stream channel segment will remain intact from the break-off point to the culvert at the southern access road. Although flows from the northern portion of the drainage basin will no longer be channeled to this reach, the watercourse drains a 13-acre area of land, and will likely exhibit flow after major precipitation events (Plan Sheet 5). A French drain will be installed on the south end of the basin (Tributary 316A Engineered Reach) at the culvert under the southern access road, which will act as a flow control device. The culvert and new drain will be at the same elevation as the old culvert, maintaining existing water levels on the north side of the access road.

Tributary 316A Restoration

The restoration of Tributary 316A will include the creation of 1,094 linear feet of stream channel which will be constructed between the existing Tributary 316A stream segment and Wetland A (Queen City Lake) (Plan Sheet 8). This restored stream will be constructed with a 6-ft channel and 14 ft between outer banks with a gentle meander. The substrate will consist of cobble material with boulders. The bank and riparian buffer area will be revegetated with native trees and shrubs and a wetland hydroseed mix containing native grasses and other herbaceous species. The proposed revegetation of native species will stabilize the bank and provide food and shelter for a wide range of insects and animals.

Because of the steepness of the slope in the eastern portion of the constructed stream channel, this segment will include multiple step pools with rock weirs, using boulders, anchored large woody debris, and instream plantings to slow flow and dissipate energy (Plan Sheet 10).

Wetlands

The following section describes anticipated conditions of the wetlands located along the existing Tributary 316A stream segment after the watercourse has been relocated.

Wetland B

It is expected that Wetland B will continue to receive adequate hydrologic inputs from groundwater interflow to support wetland conditions. After storm events, the old stream channel may occasionally provide surface water inputs to Wetland B.

Without the presence of an active stream, hydrogeomorphic (HGM) classification of Wetland B will convert from a riverine/depressional system to a depressional system (Brinson 1993).

The functions provided by Wetland B will continue to be served on the site. The opportunity to improve water quality will decrease because the stream draining developed areas will no longer flow through the wetland. However, the function of improving water quality of Tributary 316A flows will be served by Wetland A, another depressional wetland system, which has the potential for removing nutrients, metals, and toxins. Wetland B will continue to provide hydrologic and habitat functions at a similar level as prior to the Phase III stream restoration.

Wetland C

Wetland C will likely continue to receive hydrologic inputs from groundwater interflow sufficient to support wetland conditions. As a depressional system, which does not display a surface connection to Tributary 316A, Wetland C will not be likely to exhibit a significant change in hydrology.

Wetland C will continue to provide water quality, hydrologic and habitat functions at a similar level as prior to the Phase III stream restoration.

Wetland D

Hydrology in Wetland D is supported by groundwater interflow and surface runoff from the surrounding area. It is expected that Wetland D will continue to receive these hydrologic inputs at a level that will support wetland conditions. If Wetland D is seasonally receiving subsurface flow from Tributary 316A during the winter and spring, that input may continue in part with surface flow in the old channel following storm events.

Wetland D will likely continue to provide water quality, hydrologic and habitat functions at a similar level as prior to the Phase III stream restoration.

Wetland F

It is expected that Wetland F will continue to receive adequate hydrologic inputs from groundwater interflow to support wetland conditions. Following major precipitation events, the old stream channel may provide surface water inputs to Wetland F. With flow controlled at the outlet of Wetland F by the culvert and French drain system, future peak water levels will likely be similar to current peak water levels. Without the presence of an active stream, HGM classification of Wetland B will convert from a riverine system to a depressional system (Brinson 1993).

The overall functions provided by Wetland F will continue to be served on the site. The opportunity to improve water quality will decrease because the stream draining developed areas will no longer flow through the wetland. However, the function of improving water quality of Tributary 316A flows will be served by Wetland A, which will function to remove nutrients, metals, and toxins as surface water infiltrates. The wetland will continue provide hydrologic functions on a moderate level because water storage capacity will not change. Flows that were previously being stored in Wetland F will now be stored in Wetland A. Habitat functions are expected to remain similar before and after the stream restoration as the current wetland and stream system have insufficient hydroperiod to provide habitat for aquatic birds, mammals, or fish.

Conclusions

Based on available information, it is expected that existing wetlands along the Tributary 316A stream segment will continue to receive adequate hydrology to support wetland conditions after the stream restoration. Two of the wetlands will shift from riverine/depressional or riverine to depressional HGM class (Brinson 1993). Although some of the functions currently provided within these wetlands will shift to Wetland A, there is expected to be no loss of overall function within the site.

Limitations

The findings presented herein are based on our understanding and on our interpretation of hydrological conditions observed during the field investigations and reviews of background data. Within the limitations of scope, schedule, and budget, the findings presented in this report were prepared in accordance with generally accepted sensitive area investigation principles and practices in this locality at the time the report was prepared. We make no other warranty, either express or implied.

This report was prepared for the use of the Queen City Farms and applicable regulatory agencies. No other party is entitled to rely on the information, conclusions, and recommendations included in this document without the express written consent of LAI. Further, the reuse of information, conclusions, and recommendations provided herein for extensions of the project or for any other project, without review and authorization by LAI, shall be at the user's sole risk.

LANDAU ASSOCIATES, INC.

Eve Henrichsen
Project Scientist

Eric Weber
Principal Hydrogeologist

ERH/EFW/jrc
[Y:\992 QCF\002.050 PH III REFILL PERMIT\R\316A WETLAND SPECIAL REPORT\LAI_STREAM ROUTING IMPACT CEDAR GROVE COMPOST_TM 11052018.DOCX]

Attachment: Figure 1: Wetland and Stream Delineation Map

References

Brinson, M. 1993. Final Report: A Hydrogeomorphic Classification for Wetlands. East Carolina University. August.

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Wetland Delineations | V:\992\002\011\D\CGC Storage Pad MOD2 2009\Env Figs3-9.dwg (A) "Figure 3" 8/26/2009

LANDAU ASSOCIATES

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

Cedar Grove Composting
Critical Area Mitigation
King County, Washington

Wetland and Stream Delineation Map

Figure 1

Please note: COPIED FROM 2007 PHASE II TIR

TECHNICAL MEMORANDUM

TO: Mr. Alan Wallace, Williams Kastner and Gibbs

FROM: Eric Weber, L.HG

DATE: January 29, 2007

RE: QUEEN CITY LAKE ESTIMATED INFILTRATION RATES
QUEEN CITY FARMS REFILL PROJECT
MAPLE VALLEY, WASHINGTON

This technical memorandum presents estimated rates of surface water infiltration from Queen City Lake to the underlying soil and documentation for calculating these estimated rates. These data support the King County Runoff Time Series (KCRTS) modeling that is being used to determine the future outlet capacity required for Queen City Lake following completion of the Queen City Farms (QCF) Refill Project.

BACKGROUND

Queen City Lake is interpreted to be a kettle lake, formed in a depression left by a melting remnant ice block. Relatively low-permeability ice-contact deposits underlie the lake and more recent silt deposits cover the lake bed. Silt deposits have been observed to be up to 3 ft thick (Landau Associates 1990).

Queen City Lake has no natural surface water outlet. Surface water flows into the lake from runoff in the Queen City Lake basin [an approximately 370-acre basin that includes the southern portion of Cedar Hills Regional Landfill (CHRL)]. Water in the lake infiltrates into underlying sediments and eventually into the uppermost aquifer (Aquifer 1) (Landau Associates 1990). In late July or August of every year, the lake goes dry; in November or December, the lake fills due to a decline in evapotranspiration and an increase in rainfall. Water levels in the lake are shown on Figures 1 and 2.

Lake levels have historically fluctuated between Elevation 444 ft (NGVD 29 or MSL)[1] and Elevation 453 ft (NGVD 29), a range of about 9 ft. A discharge pipeline was installed in the southern portion of the lake in February 1991 in an effort to control erosion associated with a spring on the gravel pit face. The outflow pipe was 36 inches in diameter with the invert set at about Elevation 448 ft (NVGD 29). The installation of this pipe has limited lake levels to about Elevation 449 ft (NVGD 29) (Landau Associates 1993 and 1994).

INFILTRATION RATE ESTIMATIONS

Prior to installation of the outflow pipe from Queen City Lake, lake levels were measured on a semi-continuous basis. A relationship was developed that correlated lake level with lake volume (Landau Associates 1990). Subsequently, lake water levels were converted to estimates of continuous water volume in Queen City Lake. A graph of continuous water level volume in Queen City Lake is presented on Figure 3 for the winters of 1988 and 1989. These data were used to estimate the volumetric rate of decline in cubic feet per second (cfs), which can also be interpreted as the rate of infiltration of lake water into underlying sediments (and eventually into Aquifer 1) because no other outlet for the lake water existed during that time. The estimated volumetric rates of decline were reported in the Remedial Investigation (RI) report (Landau Associates 1990) and were considered minimum values because they did not consider concurrent recharge to the lake from continued surface water run on. The rates reported in the RI Report varied from 1.1 cfs at lake water depths of about 6 to 7 ft to 0.13 cfs at lake water depths of about 1.5 to 4 ft.

More recently, volumetric rates of decline for Queen City Lake were recalculated using data from four episodes of water level decline, each representing different ranges of lake water levels. These four episodes are identified on the graphs presented on Figures 1 through 3. The estimated rates of volumetric decline for these four episodes were calculated by dividing the change in lake water volume during an episode of water level decline divided by the time period of the decline. The calculated estimated rates of volumetric decline ranged from 0.8 cfs to 7.9 cfs. These estimated volumetric rates of decline are presented in Table 1.

The estimated volumetric rates of decline for episodes 1, 3, and 4 (1.1 cfs, 0.8 cfs, and 1.3 cfs, respectively) are similar to the estimated volumetric rates of decline presented in the RI report. The estimated rate for episode 2 is associated with very high lake water levels that correspond to lake water depths between 8 and 9 ft. At these lake depths, the volumetric rate of decline was estimated at 7.9 cfs. By contrast, the volumetric rate of decline associated with lake depths between 5 and 8 ft was only 1.3 cfs. The appreciable increase in rates of decline is attributed to higher lake infiltration rates along the perimeter of the lake where the silty lake bed sediments are thinner or absent and highly permeable gravel deposits are present. Lake water infiltrates through these more permeable deposits at the perimeter of the lake during high water levels when the lake expands beyond its typical limits.

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[1] All elevations in this memo are referenced to NGVD29 which is the same as mean sea level (MSL). Note that recent site surveys have generally been completed in NAVD 88, which is about 3.5 ft higher.

The recalculated volumetric rates of decline using data from four episodes of water level decline also represent lower bound estimates of volumetric rates of decline because, like those rates calculated in the RI report, these rates do not account for concurrent recharge from continued surface water run on. For the purposes of estimating infiltration rates more representative of actual conditions, the volumetric rates of decline were doubled to account for surface water run on. The resultant volumetric rates of decline (cfs) versus lake depth in feet were plotted on a graph and a line graph was created by connecting the plotted data. The line graph is shown on Figure 4. Infiltration rates for lake water depth intervals of one foot (e.g., 0 to 1 ft; 1 to 2 ft, etc) were then interpolated from this graph. The graph of interpolated infiltration rate estimates is also presented on Figure 4. Interpolated infiltration rates in cfs are tabulated in Table 2.

Estimated infiltration rates per unit area were calculated by dividing the estimated lake infiltration rates presented in Table 2 by the estimated lake surface area for a particular lake depth (e.g., lake depths of 1 ft, 2 ft, 3 ft, etc). Infiltration rates per unit area ranged from 0.13 inches/hour to 0.17 inches/hour when the lake depth was between 1 and 7 ft. These infiltration rates are consistent with loam to sandy loam soil based on the Washington State Department of Ecology's (Ecology) *Stormwater Management Manual for Western Washington* Table 3.7 (Ecology 2005). Infiltration rates in this range are consistent with infiltration through the silt bed of the lake. Infiltration rates per unit area for lake depths between 7 and 9 ft depth ranged from 0.45 to 0.9 inches/hour. These infiltration rates are consistent with loamy sand to sandy soil. Infiltration rates in this range reflect higher rates of infiltration associated with permeable sand and gravel along the perimeter of the lake.

CONCLUSION

Estimated infiltration rates from Queen City Lake to the underlying subsurface soil ranges from 0.13 inches/hour to 0.17 inches/hour at typical lake water depths of 1 to 7 ft. The infiltration rate significantly increases when lake water depths increase above 7 ft. At depths between 7 and 9 ft, the infiltration ranges from 0.45 to 0.9 inches/hour. The significant increase in infiltration rate at lake depths of 7 ft or greater is attributed to the increase in lake surface area and the lack of less permeable soil (i.e., silt) along the perimeter of the lake when the lake surface area expands beyond its typical limits.

REFERENCES

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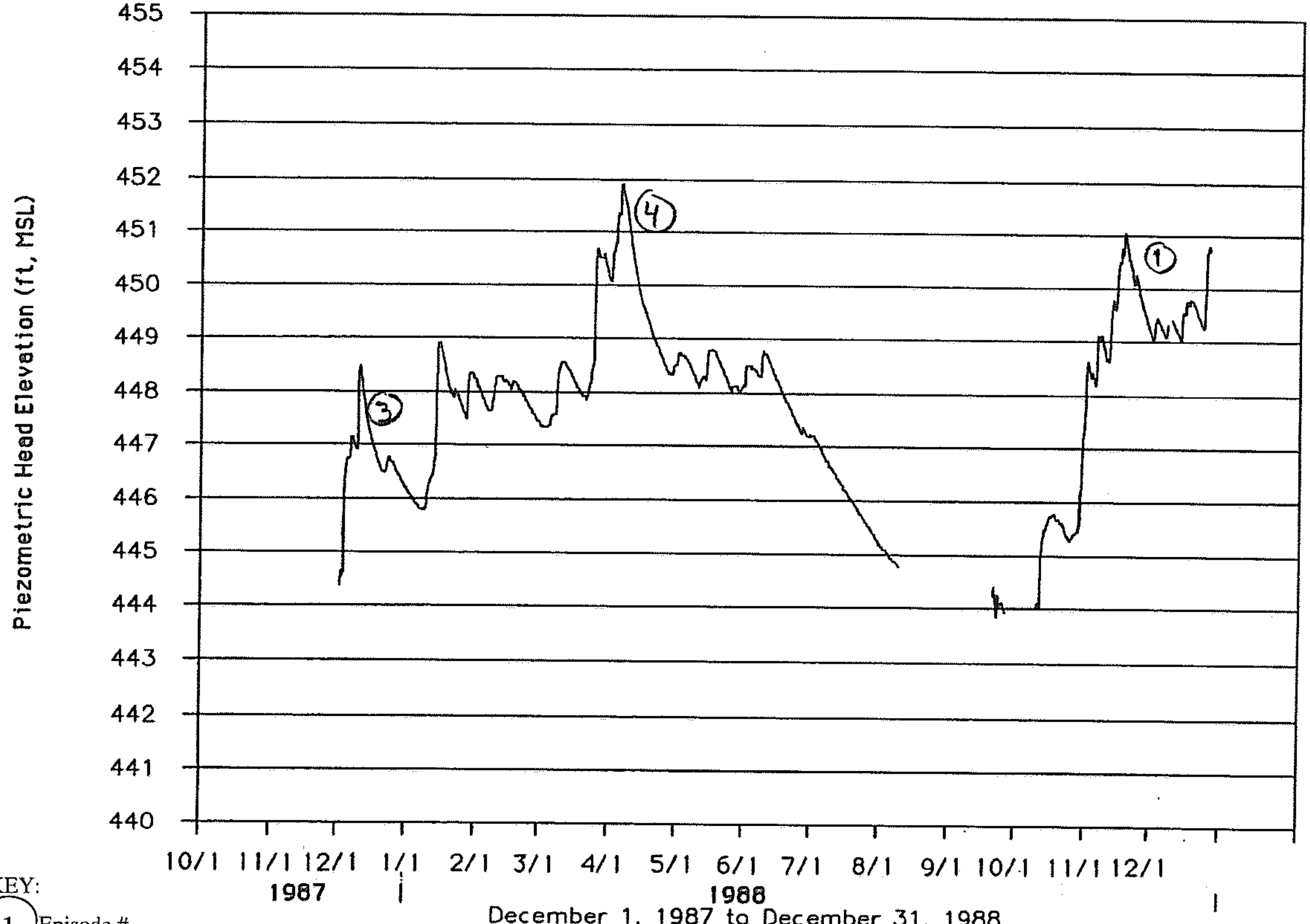
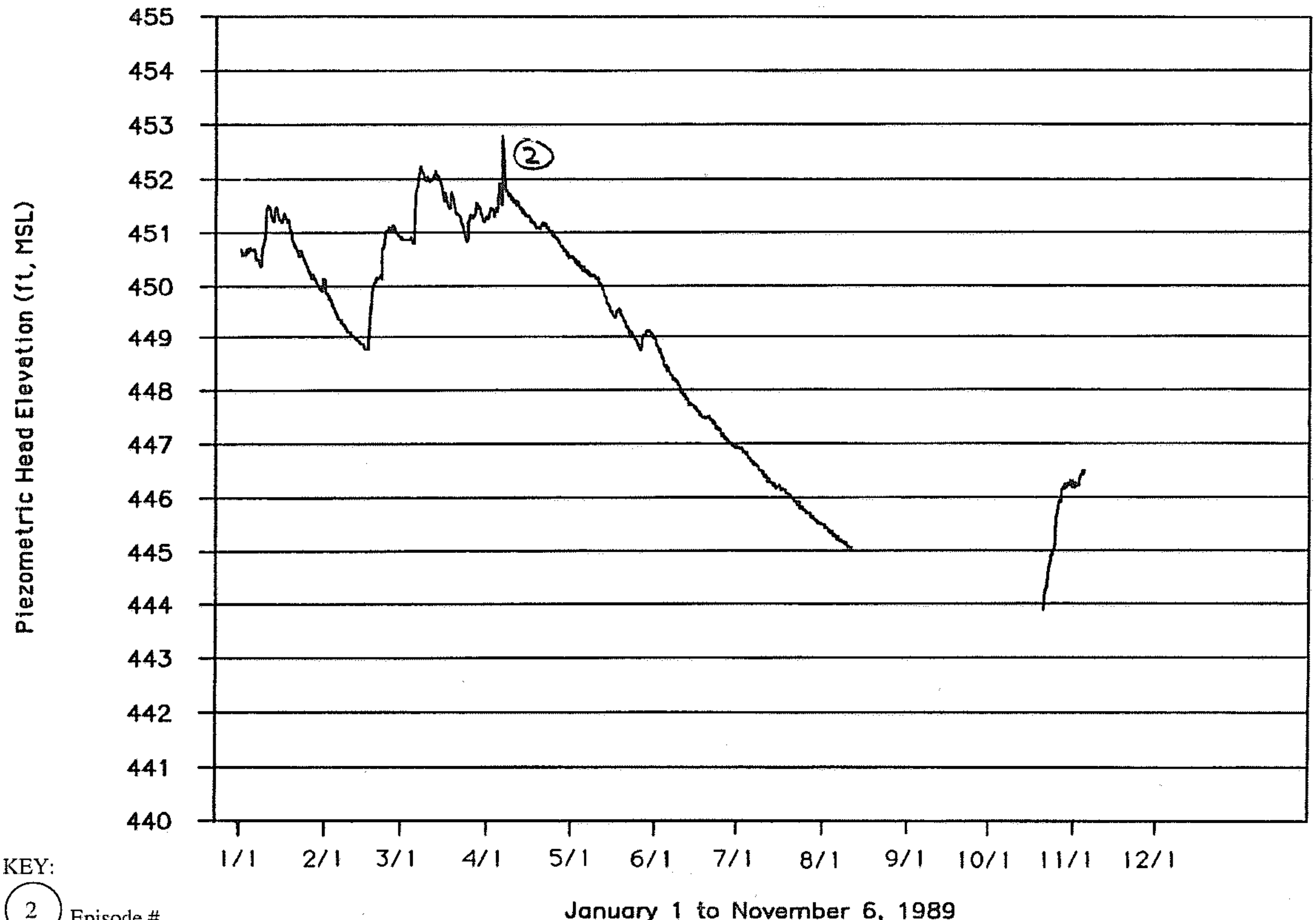
Ecology. 2005. *Stormwater Management Manual for Western Washington*. Volumes I-V. Publication Numbers 05-10-29 through 05-10-033. Washington State Department of Ecology. February.



KEY:
1 Episode #

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Semi-Continuous Water Level Data** **Queen City Lake** **December 1, 1987 to December 31, 1988**	Figure 1

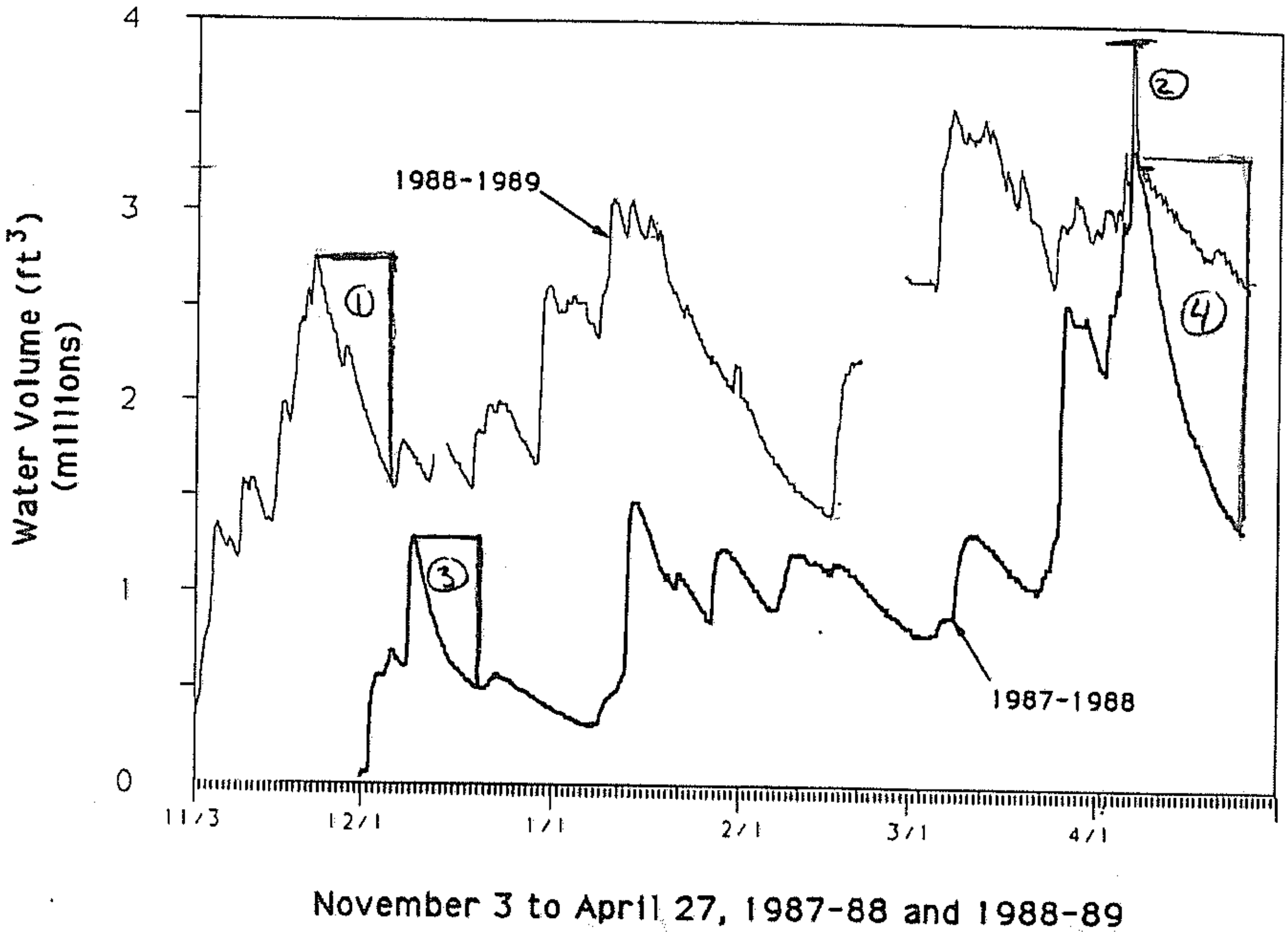


KEY:
2 Episode #

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Semi-Continuous Water Level Data Queen City Lake January 1 to November 6, 1989**	Figure 2

Source: 25-13-50 Boeing/QCF/RI Report 4/90



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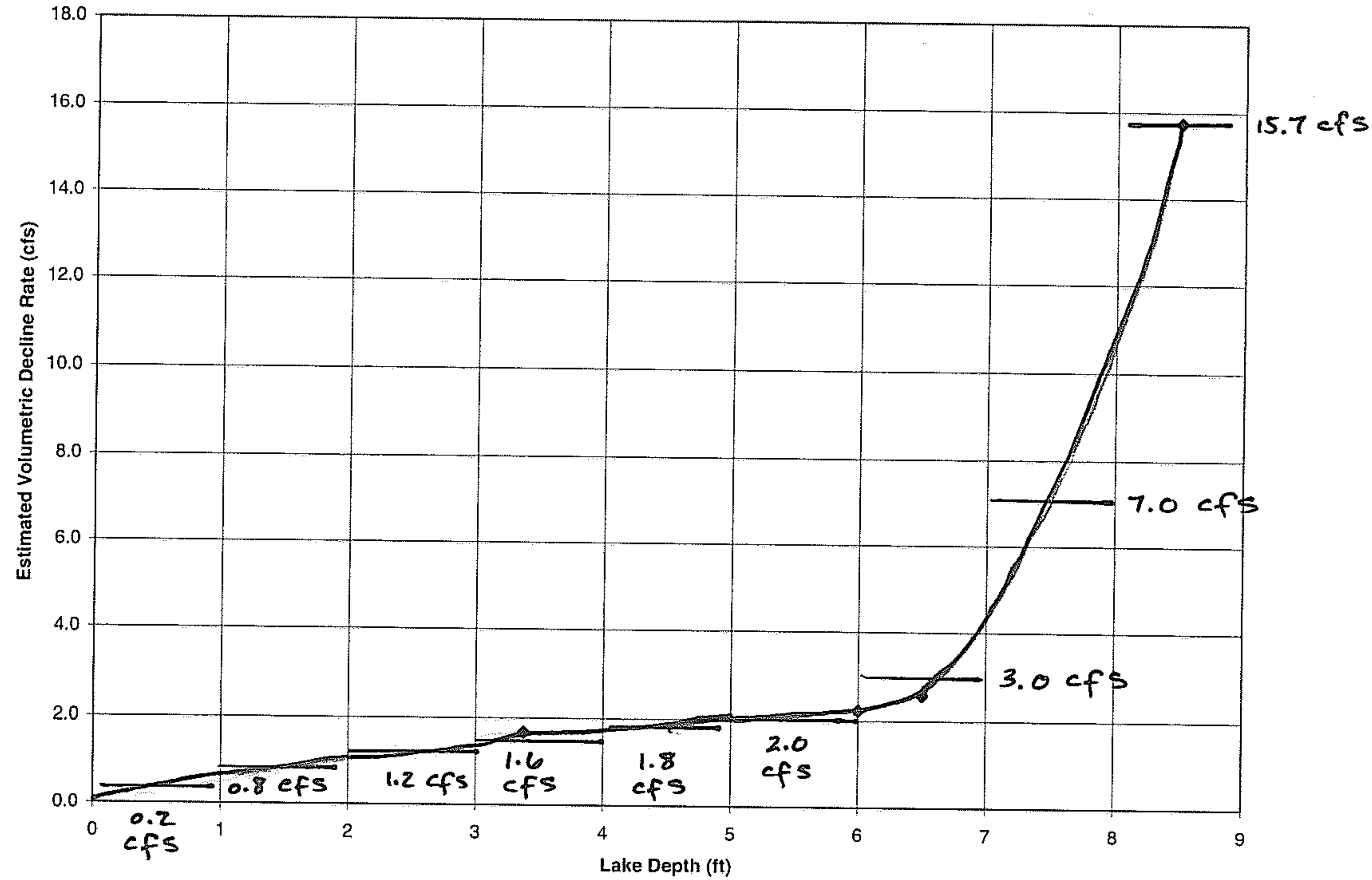
LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Calculated Water Volume in Queen City Lake for Consecutive Wet Seasons Monitored During the RI**	Figure 3



LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Infiltration Rate Estimates For Queen City Lake**	Figure 4

TABLE 1
VOLUMETRIC RATE OF DECLINE CALCULATION
FOR QUEEN CITY LAKE

Episode #	Range in Lake Level Depths (ft)	Days of Water Level Decline	Change in Water Volume (ft3)	Rate of Decline (ft3/sec)
1	5 to 7 ft	13	1250000	1.1
2	8 to 9 ft	1	680000	7.9
3	2.5 to 4.25 ft	11	800000	0.8
4	5 to 8 ft	18	2000000	1.3

Notes:
1. Episode #'s are listed on Figure 1.
2. Water level depths are based on Figures 2 and 3, assuming the bottom of the lake is at Elevation 444 ft.

TABLE 2
QUEEN CITY LAKE
ESTIMATED INFILTRATION RATES

Lake Depth (ft)	Estimated Infiltration Rate (cfs)	Estimated Lake Surface Area (ft^2)	Estimated Infiltration Rate per Unit Area (inches/hour)
0 to 1	0.3	100,000	0.13
1 to 2	0.8	240,000	0.14
2 to 3	1.2	320,000	0.16
3 to 4	1.6	420,000	0.16
4 to 5	1.8	470,000	0.17
5 to 6	2	520,000	0.17
6 to 7	3	640,000	0.20
7 to 8	7	675,000	0.45
8 to 9	15.7	750,000	0.90

Note:
1. Bottom of lake elevation assumed to be 444 ft, MSL.
2. Queen City Lake surface area estimated from Figure 3-10 of the Queen City Farms RI report (Landau Associates 1990).

Technical Memorandum

TO: Queen City Farms

FROM: Annabel Warnell and Daniel Simpson, PE

DATE: April 4, 2018

RE: **Slope Stability Analysis**
Queen City Farms Phase III Refill
Maple Valley, Washington
LAI Project No. 0992002.050.051

Introduction

This technical memorandum summarizes the results of slope stability analysis performed by Landau Associates, Inc. (LAI) in support of Queen City Farms' (QCF) Phase III Refill project, located northeast of the intersection of SE Lake Francis Road and Cedar Grove Road SE in Maple Valley, Washington (site). Our services have been provided as requested to support LAI's 2018 Technical Information Report, which is being prepared as a separate document related to the proposed refill project.

The general project location is shown on Figure 1. Figure 2 presents existing site topography, proposed Phase III refilling boundaries and site grades, and the locations of the geologic cross sections used in our slope stability analysis. Figure 3 shows approximate proposed fill limits and site grades for Phases I and II. Figures 4 through 9 summarize the findings of our slope stability analysis and show the estimated critical failure surfaces of the cross sections analyzed in this study.

This technical memorandum has been prepared based on our review of previous geotechnical evaluations of the site by others, LAI's original slope stability analysis conducted in 2007, the results of our slope stability analysis, and our experience with similar projects.

Project Understanding

We understand the scope of the refill project includes placing approximately 120 feet (ft) of imported fill in the QCF gravel pit to achieve design site grades. The refilling operation is divided into three phases, which will be conducted over the course of 10 to 20 years. Phase I is currently underway and involves placing fill to achieve preliminary site grades in the western–central portion of the site. Phase II filling in the eastern portion of the site has not yet begun, but is designed and permitted. LAI previously performed slope stability analysis for Phase II filling (LAI 2007). Phase III will consist of placing fill in the central portion of the site to establish design site grades. A maximum inclination of 4 horizontal to 1 vertical (4H:1V) has been established for slopes along the southern boundary of the overall refill zone, while other proposed design slopes are flatter.

We understand that the import fill that will be used throughout Phase I is highly variable and consists of a mixture of fine-grained soils, construction debris, and other material generally unsuitable for construction/commercial use. It is anticipated that similar fill material will be placed during Phases II and III. We also understand that the refilling operation generates revenue by accepting unwanted soil from other sites, and that there were no controls established regarding the engineering properties or compaction of the fill. At this time, there are no plans to develop the site after Phase III refilling.

Interpreted Subsurface Conditions

Subsurface conditions at the site were estimated using information obtained during previous site investigations (Earth Consultants, Inc. [ECI] 2005; LAI 2007). Due to the unknown composition and anticipated high variability of the existing and future import fill, the soil strength parameters used in our slope stability analysis were based on our experience with soils associated with similar refill projects.

The exact amount of import fill placed in the western—central portion of the site prior to Phase I is unclear, but does not affect the results of our slope stability analysis. We assume the grades prior to Phase I shown on Figure 3 are representative of estimated native recessional outwash elevations at the site.

During its 2005 explorations, ECI advanced three borings approximately ¼ mile west of the proposed Phase III refilling boundary. The boring logs document recessional outwash deposits from ground surface (elevation 340 ft) to about elevation 320 ft (North American Vertical Datum of 1988 [NAVD88]). Very dense glacial till/advance outwash deposits were reportedly observed below the recessional outwash to the maximum depths explored. We assume that subsurface conditions below the estimated bottom elevations of import fill consist of recessional outwash to elevation 320 ft underlain by very dense glacial till/advance outwash. The assumed subsurface conditions are generally consistent with geologic information for the project area obtained from *Surficial Geology of the Maple Valley and Hobart Quadrangles, Washington* (Rosengreen 1965). Table 1 presents the assumed soil properties used in our slope stability analysis.

Table 1. Assumed Soil Properties

Geologic Unit	Soil Moist Unit Weight (pcf)	Soil Internal Angle of Friction (ϕ, degrees)
Import Fill	115	28
Recessional Outwash	125	32
Advance Outwash/ Glacial Till	140	36

Note: All geologic units were assumed to be cohesionless.
pcf = pounds per cubic foot

We considered groundwater levels at the crest of each cross section between 1 and 72 ft below ground surface (bgs) and groundwater levels at the toe of each cross section as high as ground surface to 15 ft bgs. Where a cross section intersects an existing/proposed stormwater feature, we assumed groundwater levels reflected conditions in which the stormwater features were filled to their maximum design storage depths (i.e., 1 ft of free board). Table 2 summarizes the groundwater levels used in our slope stability analysis.

Table 2. Assumed Groundwater Levels

Geologic Cross Section	Groundwater Level at Crest of Slope		Groundwater Level at Toe of Slope	
	Depth bgs (ft)	Elevation (ft)	Depth bgs (ft)	Elevation (ft)
XX'	72	457	0	385
YY'	72	457	15	385
ZZ'	1	469	5	390

Note: Elevations reference NAVD88.
bgs = below ground surface
ft = feet

Seismic Conditions

The seismic conditions, including seismic design parameters and seismic site class, used in our slope stability analysis are summarized in Table 3 and were determined in accordance with the American Association of State and Highway Transportation Officials' (AASHTO's) *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* (2014). We used the 7 percent probability of exceedance in 75 years (nominal 1,000-year earthquake) design seismic event in our slope stability analysis. We assume no International Building Code-governed future site development, which would necessitate using a nominal 2,500-year earthquake in our slope stability analysis, will take place within the overall refill boundaries.

Table 3. Seismic Conditions

Site Class	Soil Profile Assumption	M	PGA (g)	S_1 (g)	S_S (g)	A_s (g)	F_a	F_v	F_{PGA}
D	Moderately Strong	7.1	0.407	0.299	0.907	0.445	1.137	1.802	1.093
E	Weak					0.367	1.011	2.804	0.900

A_s = site-adjusted peak ground acceleration
F_a, F_v = acceleration (0.2-second period) and velocity (1.0-second period) site coefficients, respectively
F_{PGA} = peak ground acceleration coefficient
g = force of gravity
M = design earthquake moment magnitude
PGA = peak ground acceleration
S_s, S_1 = 0.2-second and 1.0-second period spectral accelerations, respectively

Results of Static Slope Stability Analysis

When conducting our slope stability analysis, we considered the three geologic cross sections (XX', YY', and ZZ') shown on Figure 2. These cross sections were selected at the locations most likely to show signs of instability under static or seismic loading. Static stability typically is described with a factor of safety (FS). According to Section 1.2.3 of the National Cooperative Highway Research Program's (NCHRP) *Report 611*

(Anderson et al.; 2008), an FS of at least 1.3 is required for a slope to be considered statically stable. When analyzed under static conditions, all three cross sections yielded FS values in excess of 1.3.

Results of Seismic Slope Stability Analysis

Seismic slope stability analysis differs from static slope stability analysis in that a horizontal earthquake force (K_h), typically one-half of the peak ground acceleration (PGA), is applied to the slope model. A site PGA of 0.407 times the force of gravity yielded a K_h of approximately 0.2 for our seismic slope stability analysis. According to the Washington State Department of Transportation's *Geotechnical Design Manual* (WSDOT 2015), an FS of at least 1.05 is required for a slope to be considered seismically stable. FS values of 1.0 or less were calculated for all three cross sections in our seismic slope stability analysis. Therefore, the proposed finished slopes may exhibit down-slope movement during or shortly after the design seismic event. Down-slope movement resulting from the design seismic event is further discussed in the Conclusions and Recommendations section of this technical memorandum.

In general accordance with the method presented in the *Simplified Procedure for Estimating Earthquake-Induced Deviatoric Slope Displacements* (Bray et al. 2007), seismic slope displacement calculations were conducted to estimate slope displacement during the design seismic event. Upper-bound levels of design seismic event slope displacement were estimated by varying the shear wave velocity (V_S) of the import fill. Assuming a weak soil profile yields a V_S of 400 feet per second (ft/s), while a moderately strong soil profile yields a V_S of 900 ft/s. Table 4 summarizes the results of our slope stability analysis and design seismic event slope displacement estimates.

Table 4. Summary of Slope Stability Analysis and Design Seismic Event Slope Displacement Estimates

Geologic Cross Section	FS for Static Conditions	FS for Seismic Conditions	Slope Displacement for V_S = 400 ft/s (inches)	Slope Displacement for V_S = 900 ft/s (inches)
XX'	1.4	0.6	20.7	19.0
YY'	2.1	0.8	8.2	7.3
ZZ'	2.1	1.0	6.2	6.2

FS = factory of safety
ft/s = feet per second
V_s = soil shear wave velocity

Liquefaction Potential

It is our opinion that the native recessional outwash and advance outwash/glacial till soils at the site underlying the existing and future import fill materials are not prone to liquefaction during a seismic event. We assume the import fill used throughout previous and proposed refill efforts consists of well-graded soils that are non-uniform in particle size; boulders; construction debris, such as chunks of concrete and asphalt pavement; and other material unsuitable for construction/commercial purposes. Given the scale of the refill operation, we also assume that large, continuous zones comprised of similar material will not be present within the fill. Due to its assumed non-homogeneous composition, the import fill is unlikely to be susceptible to widespread liquefaction. However, local instances of seismically-induced liquefaction may occur in zones of saturated, poorly graded fill, which could result in localized slope displacement on the order of several tens of feet. Slope displacement resulting from seismically-induced liquefaction is further discussed in the Conclusions and Recommendations section of this technical memorandum.

Stormwater features at the crest and toe of cross section ZZ’ increase the risk of soil liquefaction due to potential groundwater level rise. Phase III import fill will be placed above the water table in cross sections XX’ and YY’ and is unlikely to increase the risk of liquefaction or soil flow failure.

Conclusions and Recommendations

- Slope stability analysis of the three cross sections, selected at the locations most likely to show signs of instability under static or seismic loading, indicates acceptable FS values under static conditions.
- During a design seismic event without liquefaction, an estimated soil displacement of up to 2 ft may occur along some portions of the Phase III refill boundaries. In our opinion, this level of performance is acceptable for undeveloped slopes.
- Widespread liquefaction of the import fill during the design seismic event appears unlikely. However, localized zones of liquefaction may occur, which could result in localized slope displacement on the order of several tens of feet.
- In our opinion, localized, liquefaction-induced slope displacement on the order of several tens of feet is an acceptable level of performance for undeveloped slopes, provided the owner is willing to accept the risk of slope displacement during the design seismic event.
- To minimize the possibility of liquefaction and liquefaction-induced flow failure (large slope deformation), or if better slope performance during the design seismic event is required, we recommend establishing quality control and compaction requirements for the Phase II and III import fill, particularly for fill placed in the vicinity of cross section ZZ’. LAI is available to provide import fill placement recommendations upon request.

Use of This Technical Memorandum

Landau Associates, Inc. prepared this technical memorandum for the exclusive use of Cedar Grove Composting in support of the Queen City Farms Phase III Refill project. Use of this technical memorandum by others or for another project is at the user’s sole risk. Within the limitations of scope, schedule, and budget, our services have been provided in accordance with generally accepted practices of the geotechnical engineering profession; no other warranty, express or implied, is made as to the professional advice included in this technical memorandum.

Closure

We trust this technical memorandum provides you with sufficient information to proceed, and we appreciate the opportunity to provide geotechnical services on this project. If you have questions or comments, or if we may be of further service, please contact the undersigned at (360) 791-3178.

LANDAU ASSOCIATES, INC.

Annabel Warnell
Senior Staff EIT

Daniel Simpson, PE
Senior Engineer

AMW/DCS/KMS/SRW/mcs
[P:\992\002.050\R\QUEEN CITY FARMS SLOPE STABILITY ANALYSIS TECHNICAL MEMORANDUM.DOCX]

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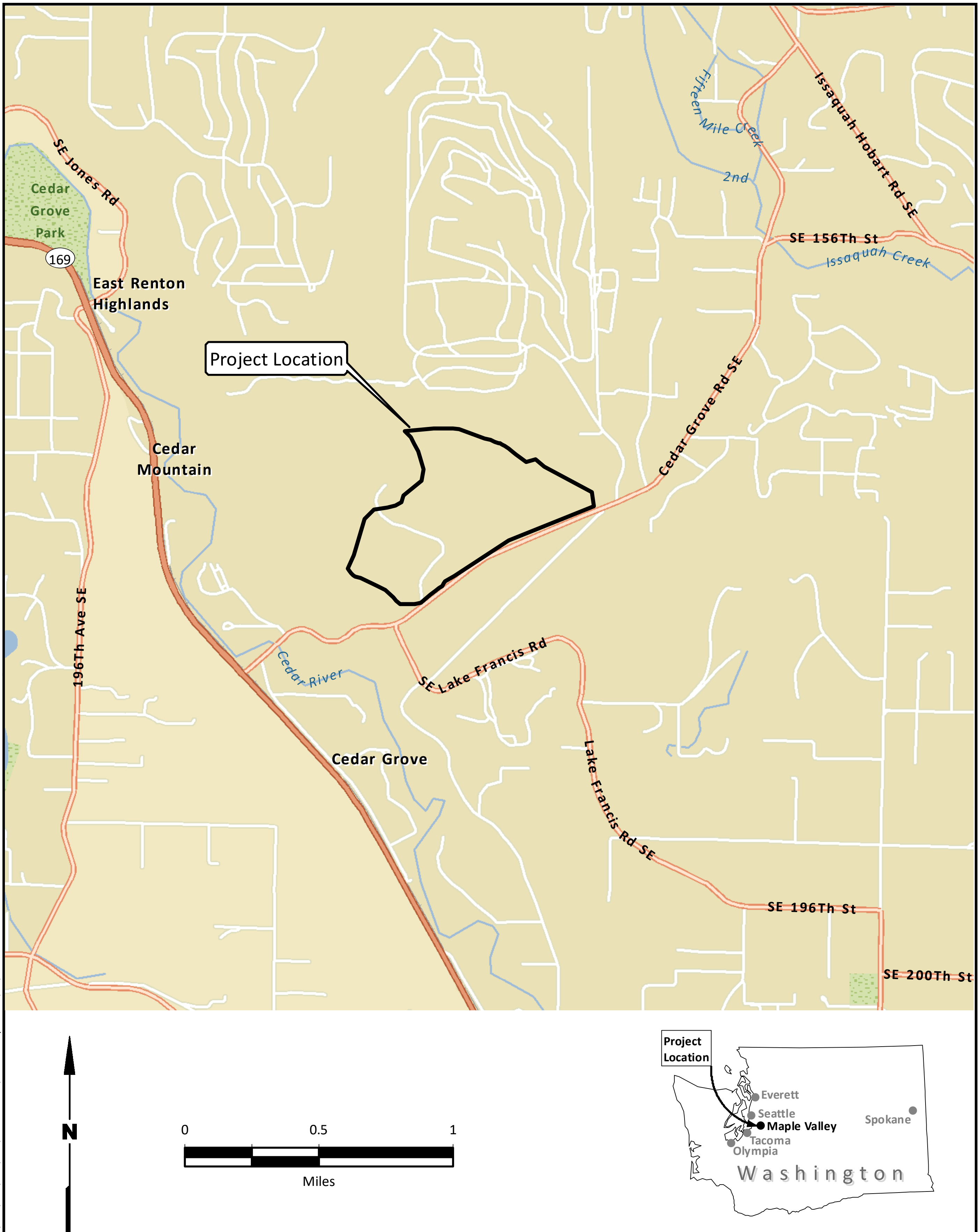
Attachments: Figure 1. Vicinity Map
Figure 2. Phase III Refill and Cross Sections
Figure 3. Phase I and II Refill
Figures 4–9. Cross Section Slope Stability Analysis–Static and Seismic Conditions



Data Source: Esri 2012

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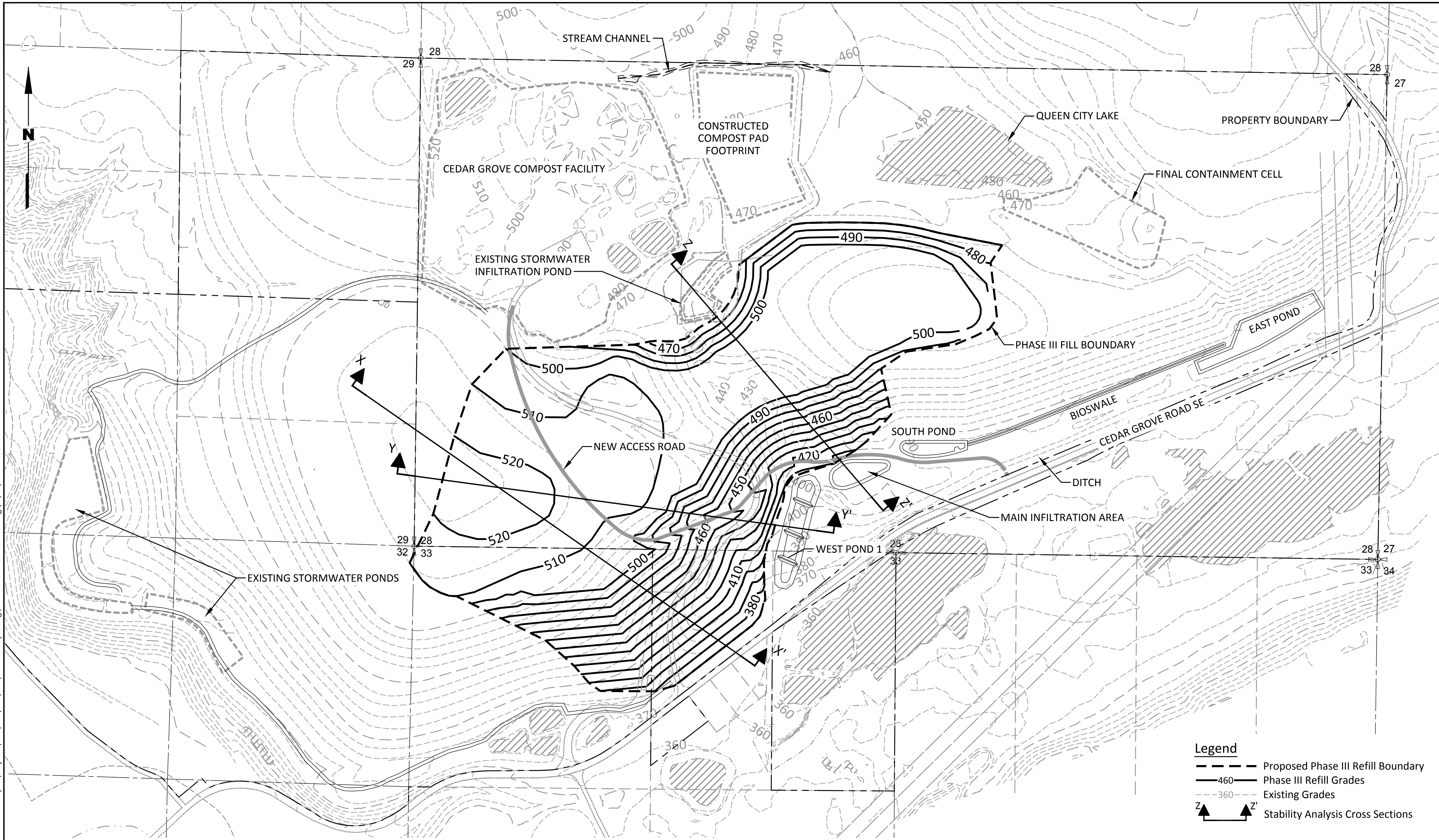
Queen City Farms Phase III Refill Maple Valley, Washington	**Vicinity Map**	Figure 1



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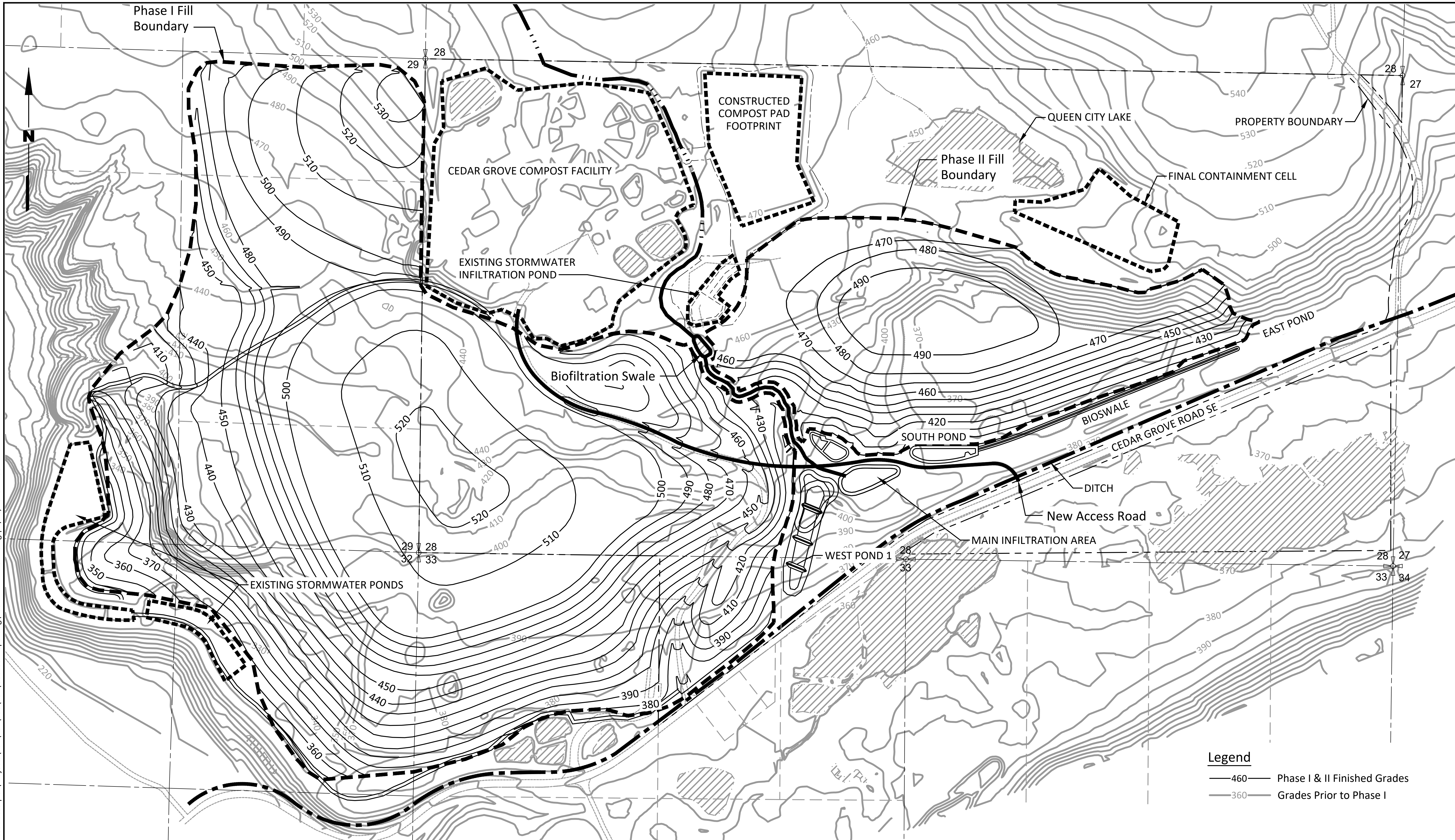
Queen City Farms Phase III Refill Maple Valley, Washington	**Phase III Refill and Cross Sections**	Figure 2



0 500 1000
Scale in Feet

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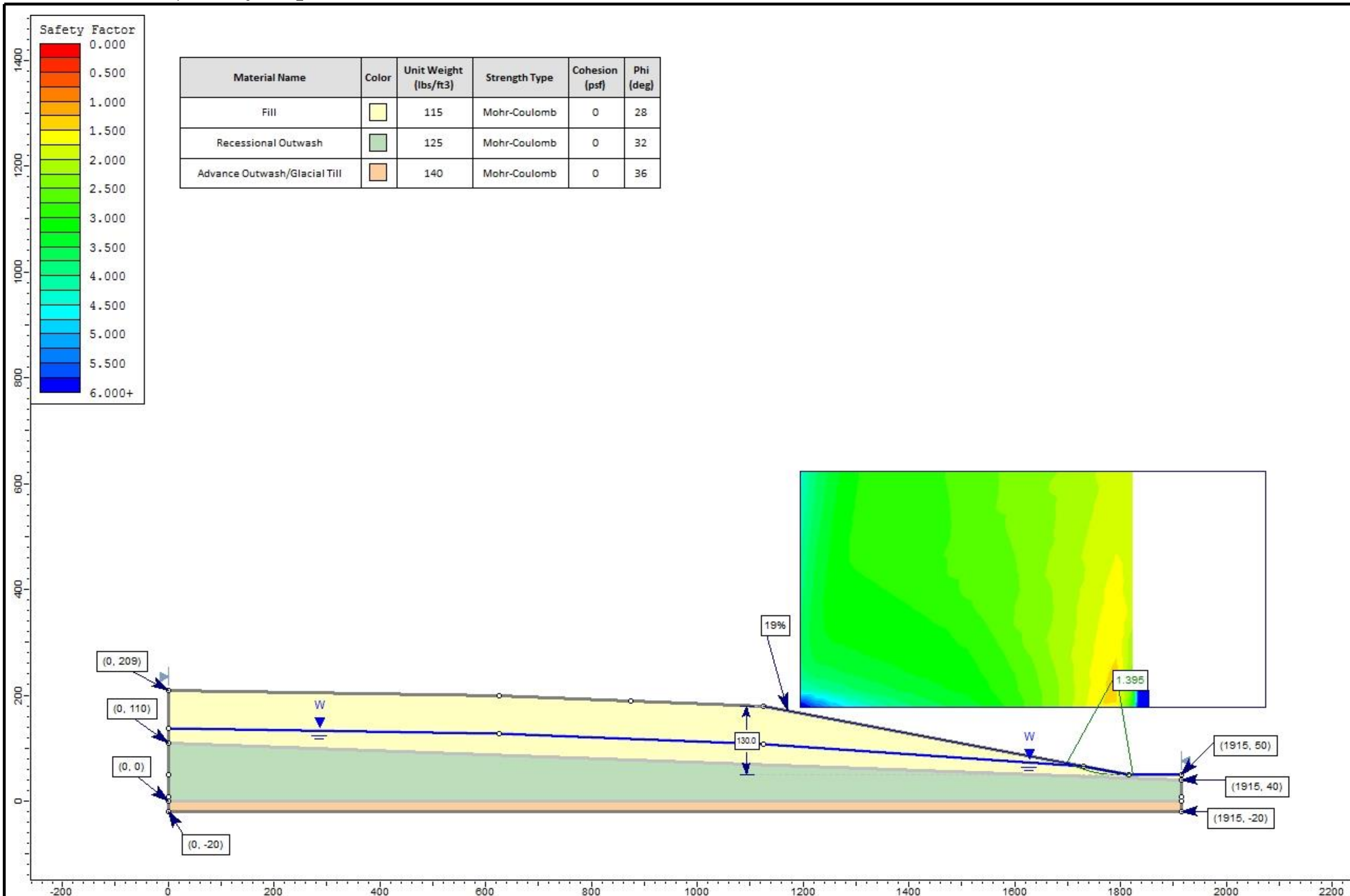
Queen City Farms Phase III Refill Maple Valley, Washington	**Phase I and II Refill**	Figure 3



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		115	Mohr-Coulomb	0	28
Recessional Outwash		125	Mohr-Coulomb	0	32
Advance Outwash/Glacial Till		140	Mohr-Coulomb	0	36

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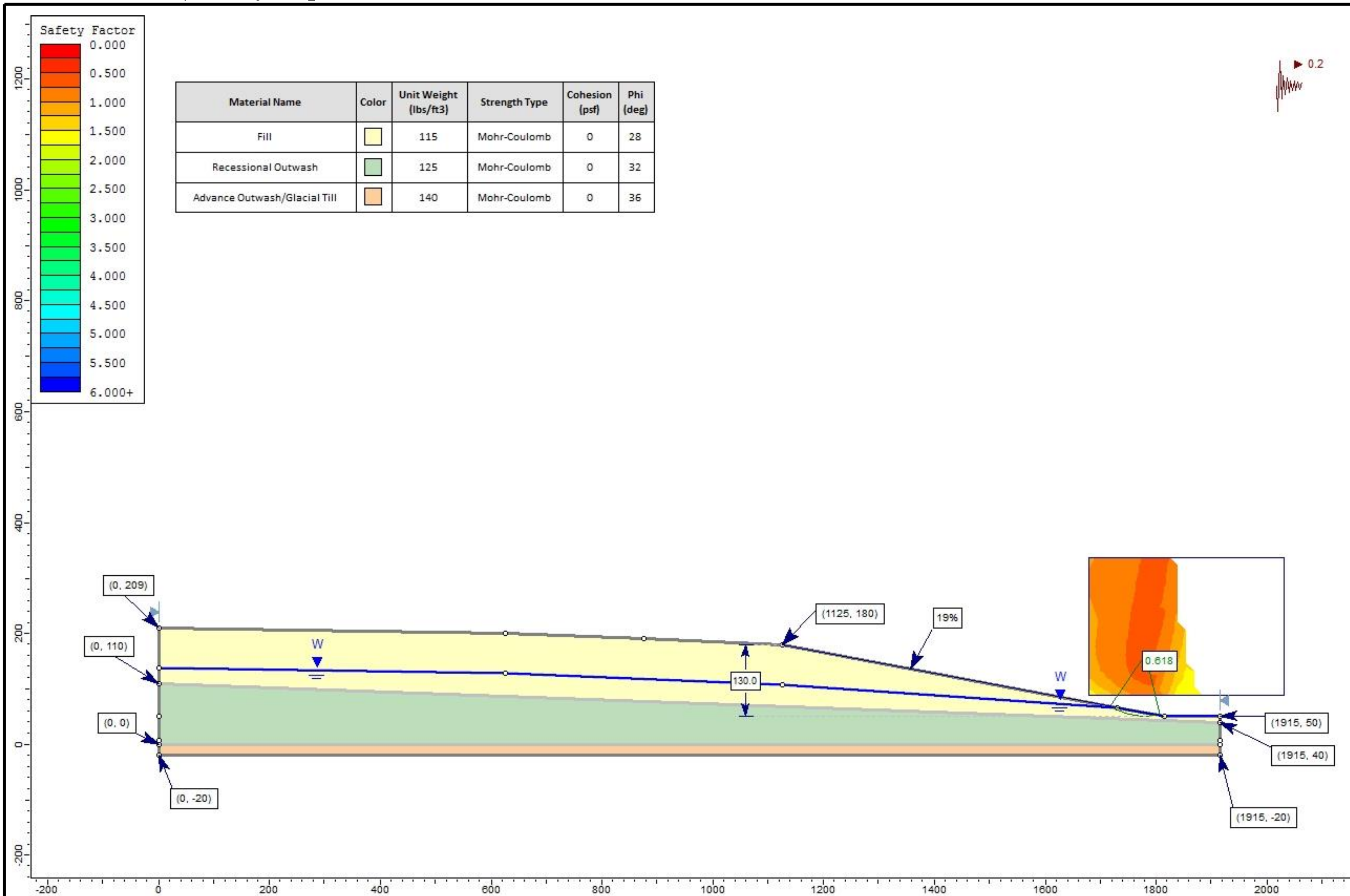
Queen City Farms Phase III Refill Maple Valley, Washington	Cross Section XX' Slope Stability Analysis Static Conditions	Figure 4

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		115	Mohr-Coulomb	0	28
Recessional Outwash		125	Mohr-Coulomb	0	32
Advance Outwash/Glacial Till		140	Mohr-Coulomb	0	36



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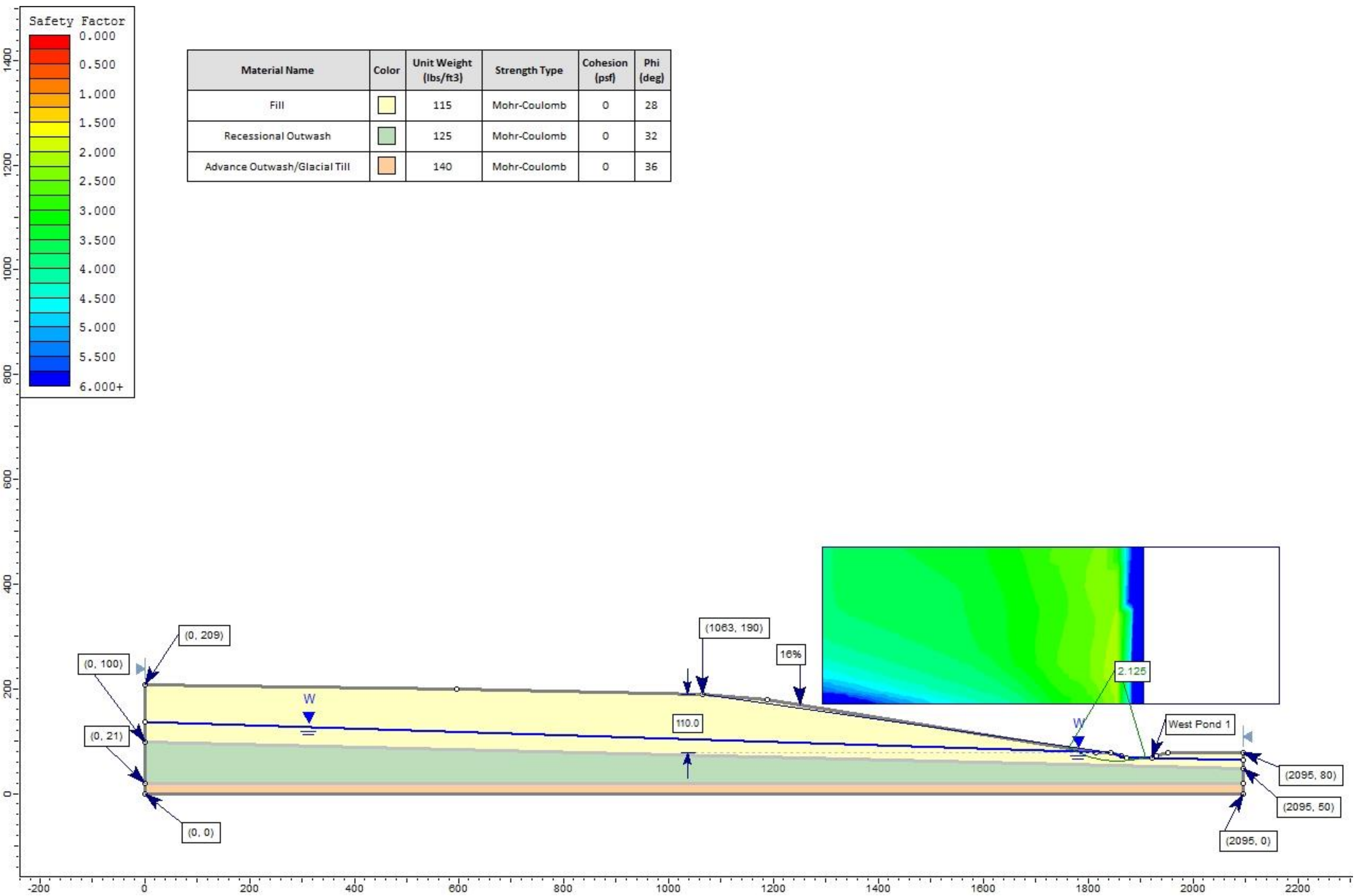
Queen City Farms
Phase III Refill
Maple Valley, Washington

Cross Section XX' Slope Stability Analysis
Seismic Conditions

Figure
5



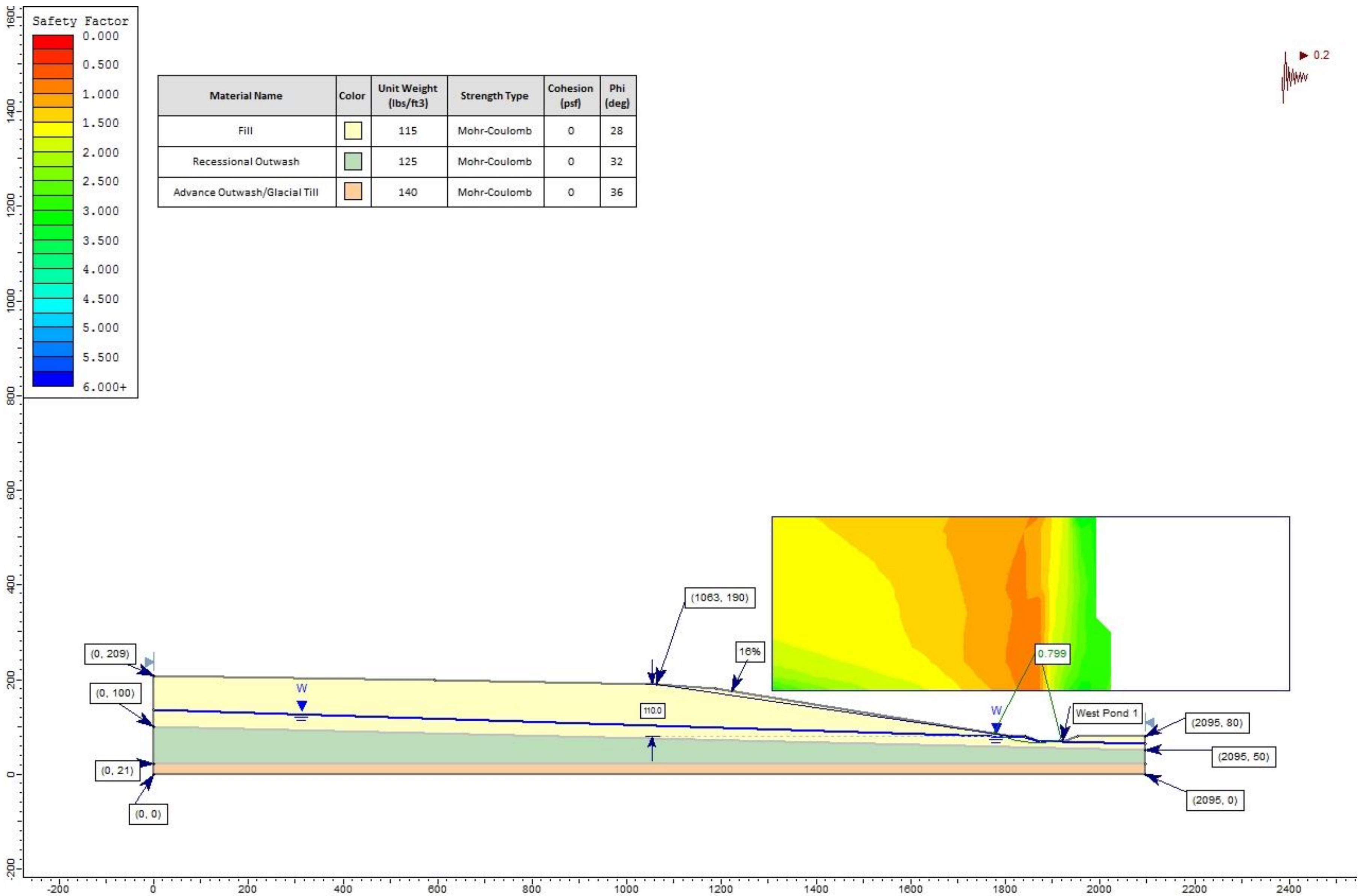
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		115	Mohr-Coulomb	0	28
Recessional Outwash		125	Mohr-Coulomb	0	32
Advance Outwash/Glacial Till		140	Mohr-Coulomb	0	36

LANDAU ASSOCIATES

Queen City Farms
Phase III Refill
Maple Valley, Washington

Cross Section YY' Slope Stability Analysis
Static Conditions

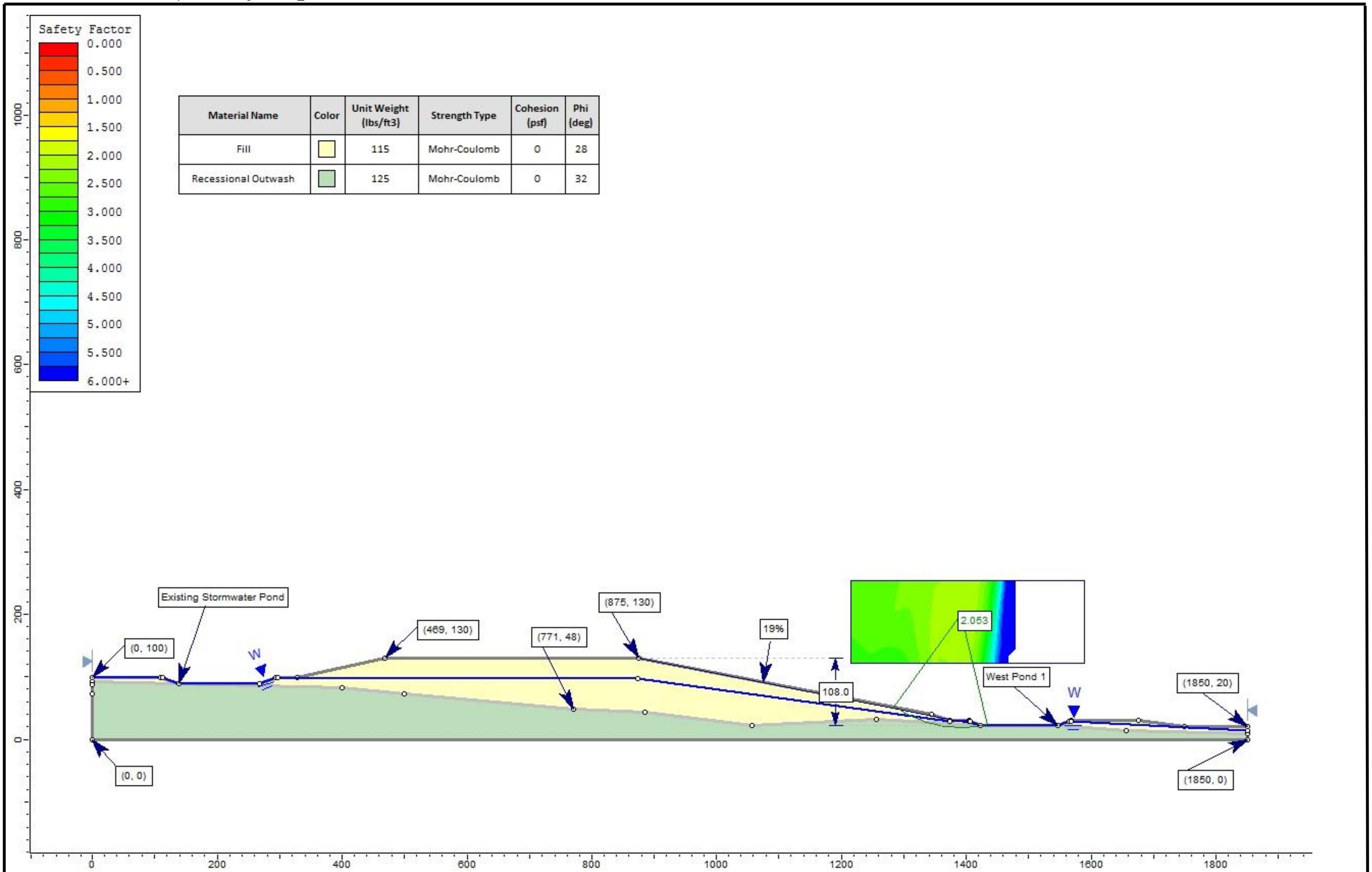
Figure
6



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		115	Mohr-Coulomb	0	28
Recessional Outwash		125	Mohr-Coulomb	0	32
Advance Outwash/Glacial Till		140	Mohr-Coulomb	0	36

Landau Associates

Queen City Farms Phase III Refill Maple Valley, Washington	Cross Section YY' Slope Stability Analysis Seismic Conditions	Figure 7



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		115	Mohr-Coulomb	0	28
Recessional Outwash		125	Mohr-Coulomb	0	32

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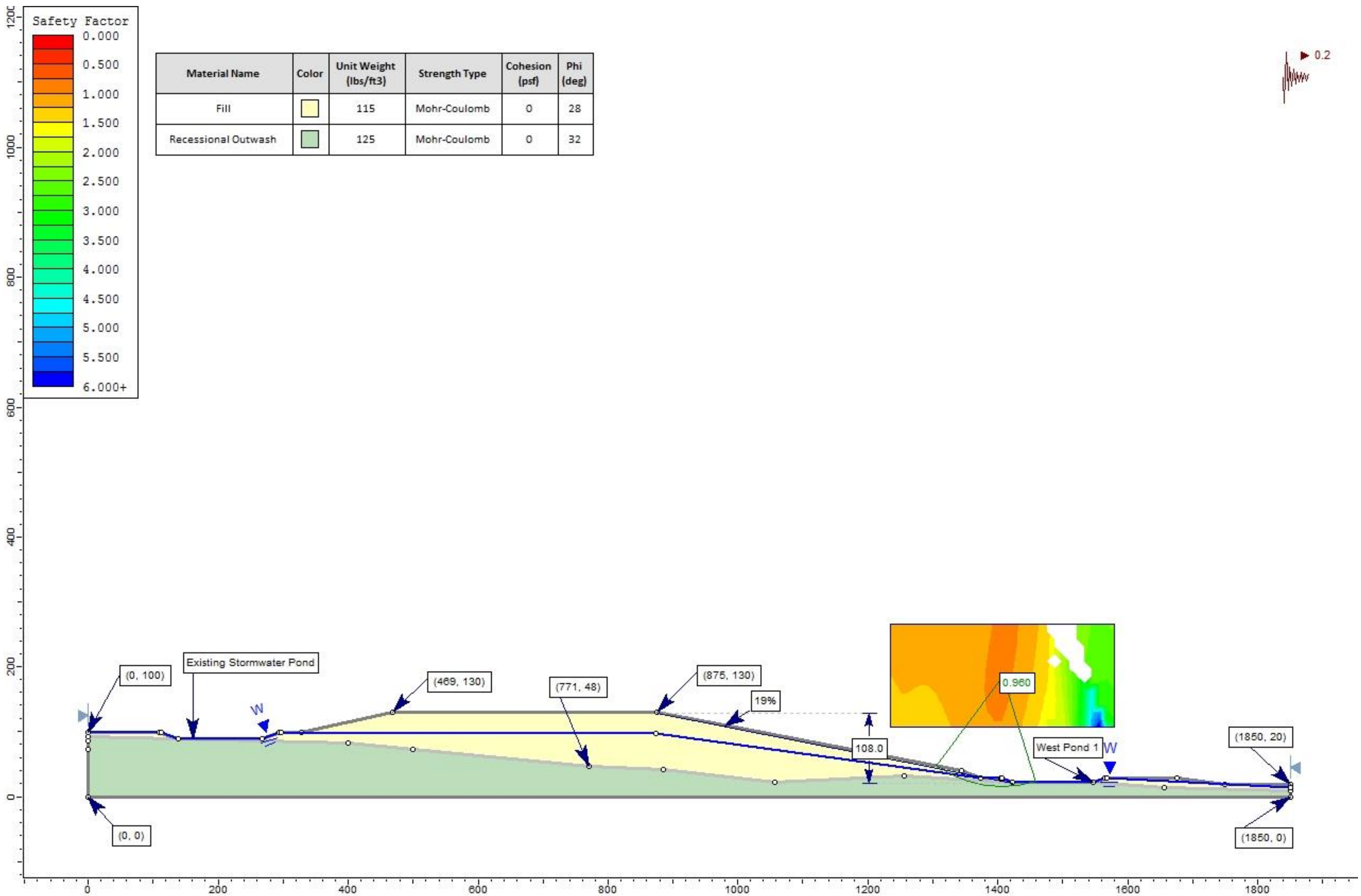
Queen City Farms Phase III Refill Maple Valley, Washington	Cross Section ZZ' Slope Stability Analysis Static Conditions	Figure 8

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill		115	Mohr-Coulomb	0	28
Recessional Outwash		125	Mohr-Coulomb	0	32



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Queen City Farms
Phase III Refill
Maple Valley, Washington

Cross Section ZZ' Slope Stability Analysis
Seismic Conditions

Figure
9

TECHNICAL MEMORANDUM

TO: Mr. Alan Wallace, Williams Kastner and Gibbs

FROM: Eric Weber, L.G.

DATE: January 29, 2007

RE: EVALUATION OF PROPOSED REFILL PLAN ON GROUNDWATER CONTAMINATION QUEEN CITY FARMS REFILL PROJECT MAPLE VALLEY, WASHINGTON

This technical memorandum has been prepared to summarize Landau Associates' evaluation of the potential effect of the proposed refill plan on groundwater contamination at the Queen City Farms (QCF) Superfund site. The planned refill consists of refilling the existing QCF gravel pit, which currently functions as a basin for stormwater storage and infiltration. These functions will be eliminated following completion of the refill plan and, therefore, recharge patterns to the underlying groundwater aquifers may be altered. The effect that these alterations may have on groundwater contamination at the QCF Superfund site is evaluated below.

BACKGROUND

The QCF property was listed on the U.S. Environmental Protection Agency's (EPA's) National Priorities List in 1984 due to its history as a disposal site for liquid industrial waste and the presence of hazardous constituents in the environment. The Boeing Company has conducted investigation and cleanup of the site under Consent Decrees between Boeing and EPA consistent with the Comprehensive Environmental Response, Cleanup, and Liability Act (CERCLA), also known as Superfund. Cleanup actions at the site have largely been implemented with the exception of ongoing annual or semiannual monitoring at groundwater monitoring wells. EPA issued a Record of Decision (ROD) in 1993 describing the final remedy for the site. The main elements of the remedy are:

- Containment of shallow contaminated soil by installing a slurry wall around the contaminated soil and groundwater area and extending an existing cap to the limits of the slurry wall to effectively isolate the contaminated soil. Consequently, volatile organic groundwater contamination in Aquifer 1 outside the slurry wall has declined to near non-detect levels.

- Monitoring of contaminated groundwater in Aquifer 2 and Aquifer 3 to evaluate the natural attenuation of volatile organic compounds (VOCs) trichloroethene (TCE) and 1,2-dichloroethene (1,2-DCE) and to ensure that contamination does not migrate off the site or affect water wells in the area (EPA 1998).

The remedy is working well. The slurry wall has successfully isolated shallow contamination and, consequently, TCE and 1,2-DCE concentrations have steadily declined within the Aquifer 2 plume (EcoChem and Boeing 2006). Groundwater quality has been monitored in lower Aquifer 2, upper Aquifer 2, and Aquifer 3. Contamination in lower Aquifer 2 is more widespread. Upper Aquifer 2 contamination is typically limited to the area beneath the slurry wall (the original shallow contamination source). The current distribution of TCE and 1,2-DCE in the lower portion of Aquifer 2 and upper portion of Aquifer 2 is presented as contour plots in Attachment 1. Groundwater contamination in Aquifer 3 is isolated primarily to the area around Wells I(3) and I(3a) [located next toW I(2)].

EVALUATION

The effect of the gravel pit refill on groundwater contamination is directly related to the remedies being implemented for cleanup of contaminated groundwater. The effect on the two primary remedies for groundwater contamination in Aquifers 1 and 2 from the planned refill is evaluated below.

Aquifer 1 Groundwater Contamination

Remediation of groundwater contamination in Aquifer 1 is dependent on containment of contaminated shallow soil and groundwater by the slurry wall. Therefore, the planned refill for the gravel pit is not expected to affect groundwater contamination in Aquifer 1, unless the refill has an effect on the integrity of the slurry wall (also known as the vertical barrier wall system). The refill plan is not anticipated to have on impact on slurry wall performance. The effect of potentially higher water levels on the vertical barrier wall system is evaluated in a separate technical memorandum (see Section 6.7 of the TIR).

Aquifer 2 Groundwater Contamination

Remediation of groundwater contamination in Aquifer 2 and Aquifer 3 is dependent on natural attenuation of VOC contaminants. A primary process in natural attenuation is dispersion of contaminants due to recharge of clean water. Natural attenuation has resulted in declining concentrations of VOCs throughout Aquifer 2 since installation of the slurry wall in 1996. A complete set of time series graphs of Aquifer 2 and Aquifer 3 wells are presented in the 2005 Annual Monitoring Data Report (Boeing and EcoChem 2006). These graphs are reproduced as Attachment 2 to this memorandum.

Implementation of the gravel pit refill plan will alter recharge patterns to Aquifer 2 and, therefore, could potentially impact natural attenuation of the Aquifer 2 plume. Any impact of the refill plan is not likely to be significant for the following reasons:

- Recharge patterns to Aquifer 2 will not appreciably change until Phase 3 of the refill plan is implemented. This will not likely occur for at least 5 years or more. Consequently, the current pattern of natural attenuation will essentially stay the same for 5 years or more. Groundwater concentrations will continue to decline over this time period.

- The refill plan does not call for any offsite discharge of storm water. All surface water on the site is managed as infiltration. Therefore, there will not be a decline in the magnitude of recharge to Aquifer 2 due to implementation of the refill plan.

- Currently, Aquifer 2 is recharged by leakage from Aquifer 1 and discharge from the Main Gravel Pit Lake. After the refill plan is implemented, Aquifer 2 recharge will be from increased leakage from Aquifer 1, recharge of the East Airstrip Spring discharge along the current north slope of the gravel pit, and infiltration at the East Retention Pond (East Stormwater Facility). Runoff from the refill area will also be recharged at the South Retention Pond (South Stormwater Facility), and the Main Infiltration Area. Therefore, the location of recharge to Aquifer 2 will not change significantly due to implementation of the refill plan.

- Recharge to Aquifer 2 is affected by the geology at the site. Discharge from the Main Gravel Pit Lake to Aquifer 2 currently occurs preferentially to the north of the lake. Aquifer 2 north of the lake is more permeable due to a relatively thick sequence of gravel (geologic unit E) that occupies the upper portion of the aquifer. Aquifer 2 south of the lake is less permeable due to thinner gravel deposits and the presence of silt and ice contact deposits. Also, the lake bottom is in direct contact with fine-grained silty sand deposits (geologic unit F) and is covered with fine-grained spoils; this limits vertical recharge from the lake to the aquifer. Direct recharge from the East Retention Pond will tend to flow preferentially to the northwest due to the higher permeability in upper Aquifer 2 in this area.

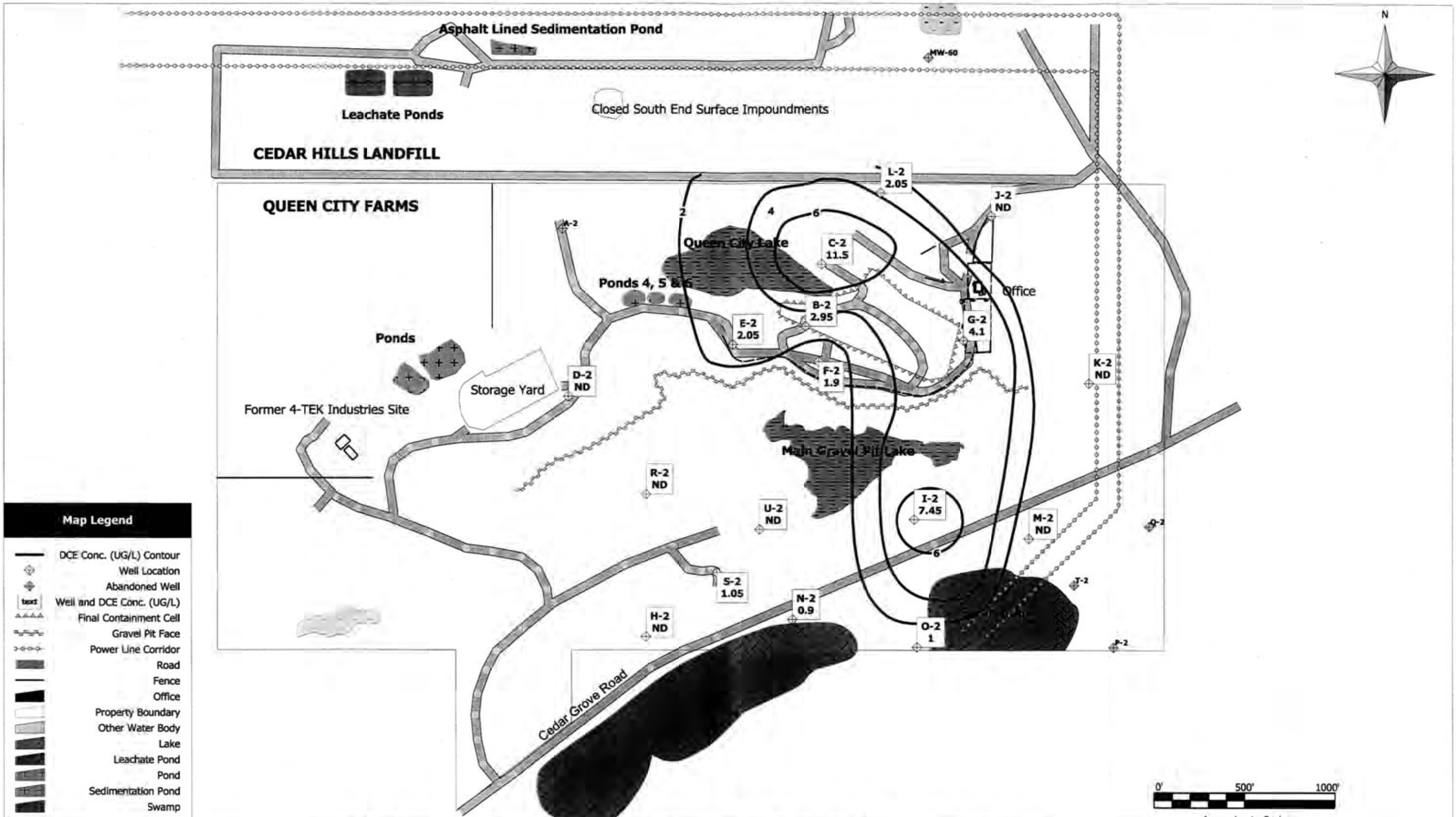
REFERENCES

EcoChem and The Boeing Company. 2006. *2005 Annual Monitoring Data Report, Queen City Farms, King County, Washington*. July.

EPA. 1998. *Superfund Fact Sheet, Queen City Farms, Maple Valley, Washington.* U.S. Environmental Protection Agency. June 16.

ATTACHMENT 1

Aquifer 2
VOC Contour Plots



BOEING - Queen City Farms Remediation Project

22715 S.E. 168th Way
Maple Valley, WA 98038

DCE Average Annual Concentrations - 2005
Method 524.2 - Lower Aquifer 2

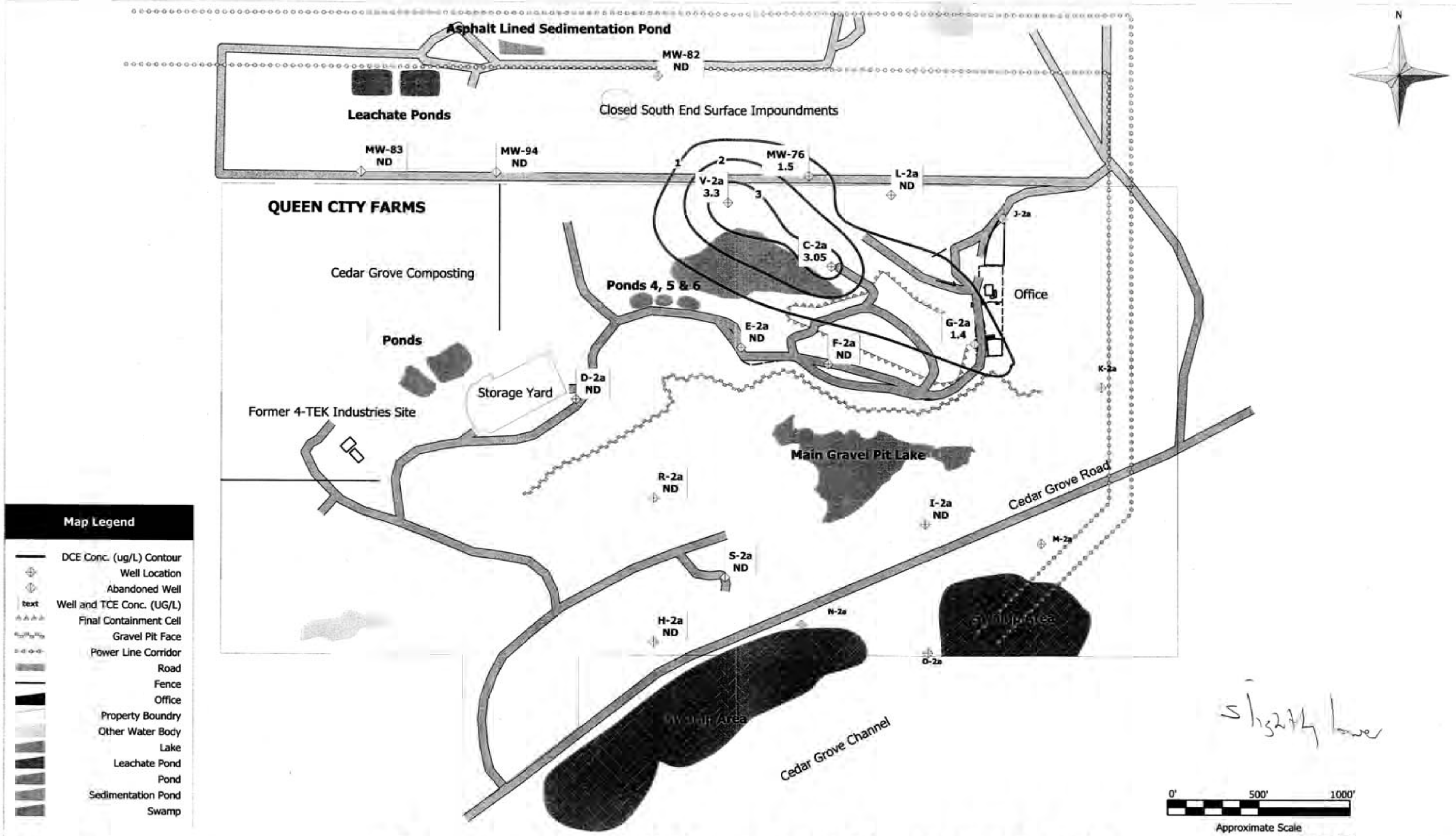
S:\098-BOE\QCF\2005\2005QCFBaseMap.map [DCE Contours LOWER]

Figure 2-8

Created: 4/5/06

Created By: MJL

Revision: 1.2



5/13/74 lower

BOEING - *Queen City Farms Remediation Project*

22715 S.E. 168th Way
Maple Valley, WA 98038

DCE Average Annual Concentrations - 2005
Method 524.2 - Upper Aquifer 2

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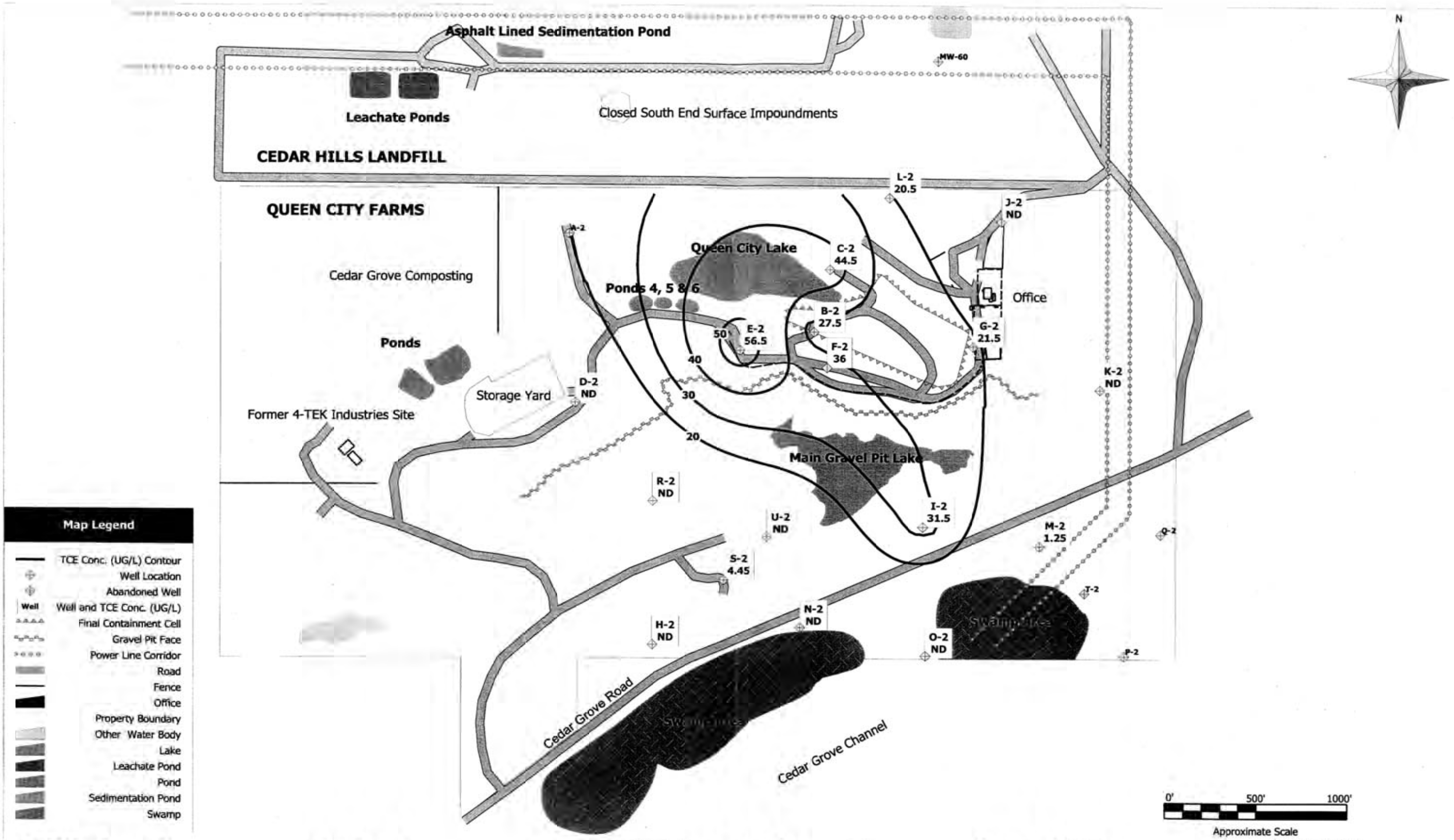
Revision: 2.0



BOEING - *Queen City Farms Remediation Project*

22715 S.E. 168th Way
Maple Valley, WA 98038

TCE Average Annual Concentrations - 2005
Method 524.2 - Lower Aquifer 2

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Figure 2-10
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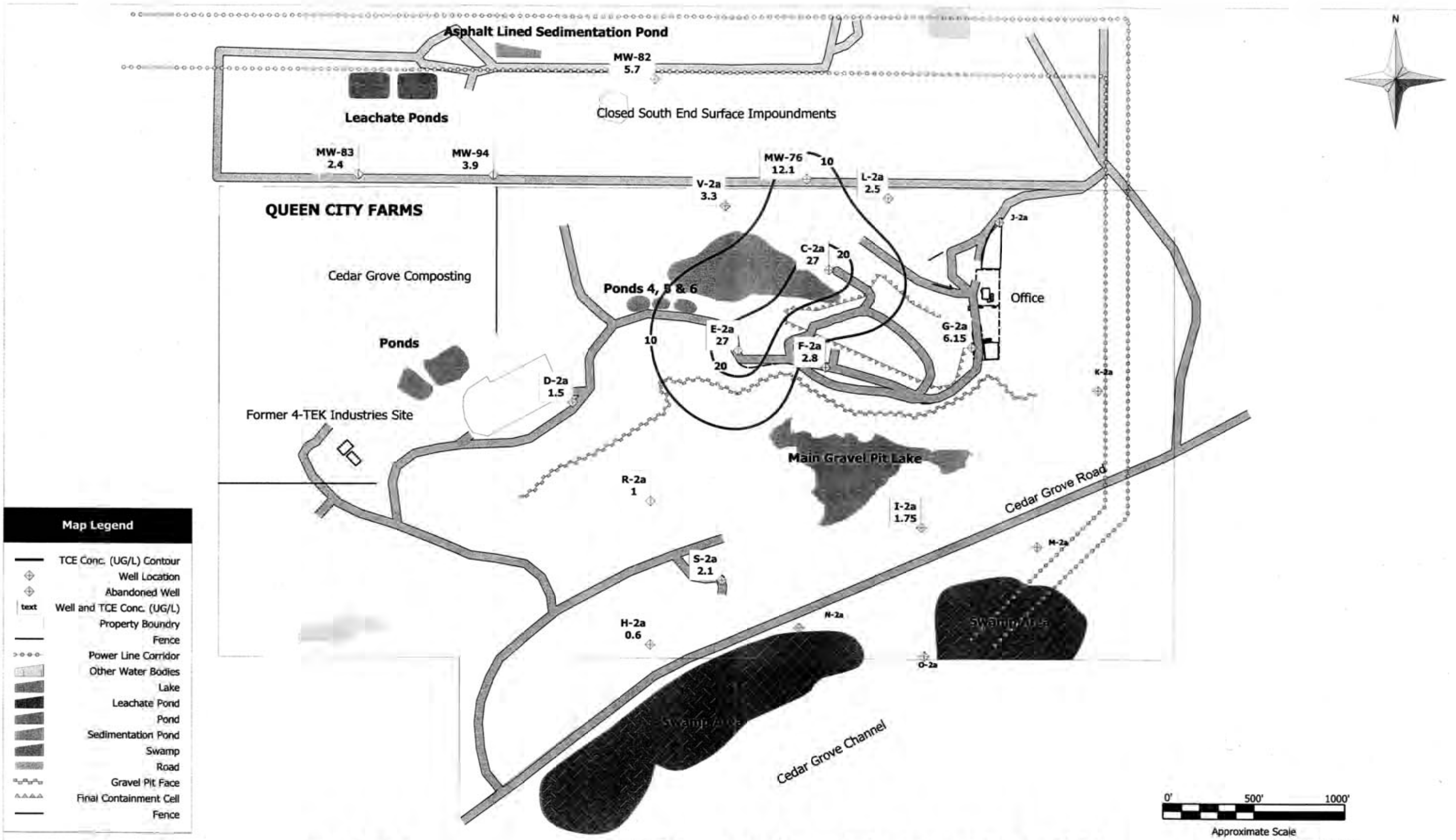
Revision: 1.0



BOEING - Queen City Farms Remediation Project

22715 S.E. 168th Way
Maple Valley, WA 98038

TCE Average Annual Concentrations - 2005
Method 524.2 - Upper Aquifer 2

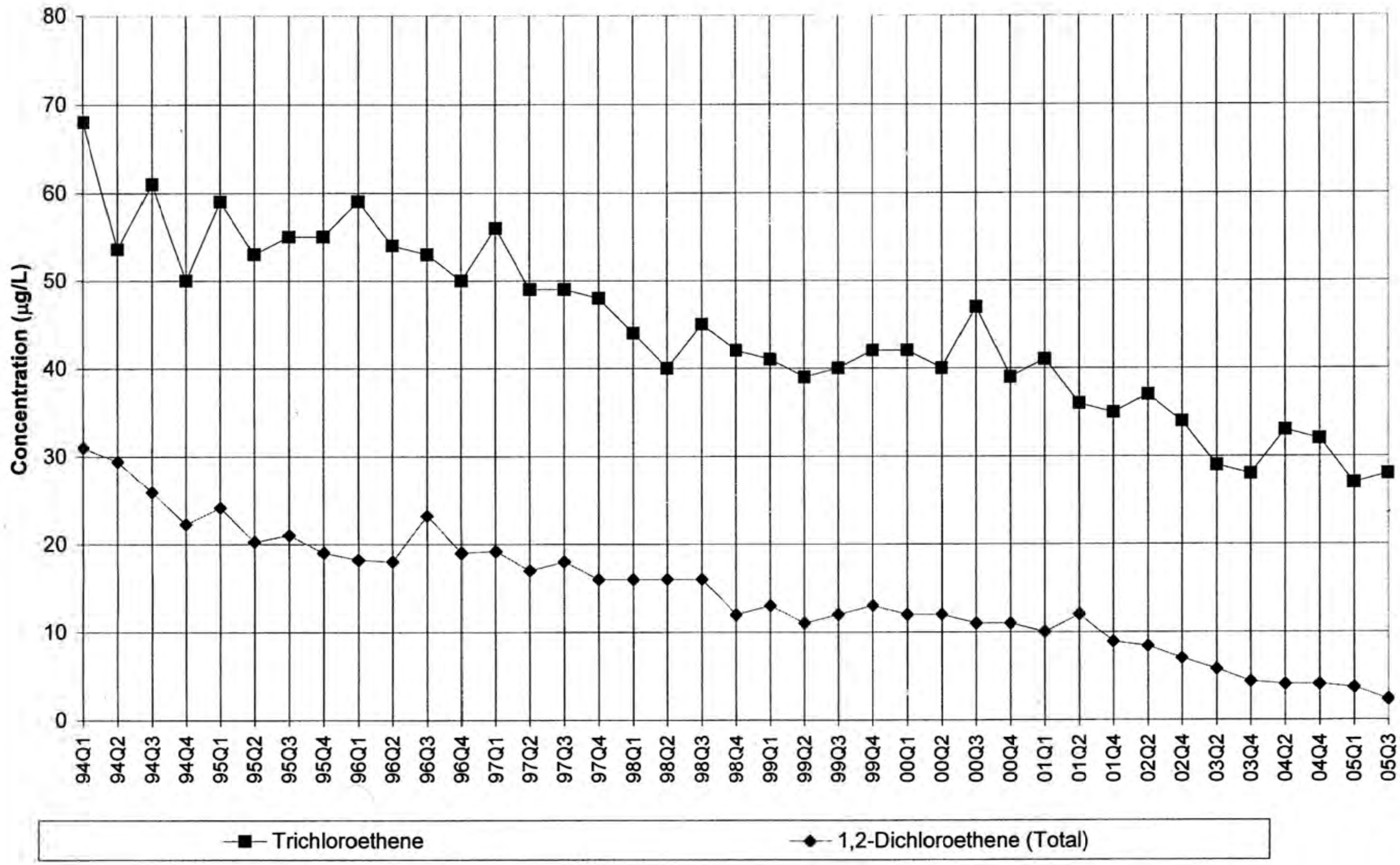
S:\098-BOE\QCF\2005\2005QCFBaseMap.map [TCE Contours UPPER]

Figure 2-11

Created: 4/5/06
Created By: MJL
Revision: 2.0

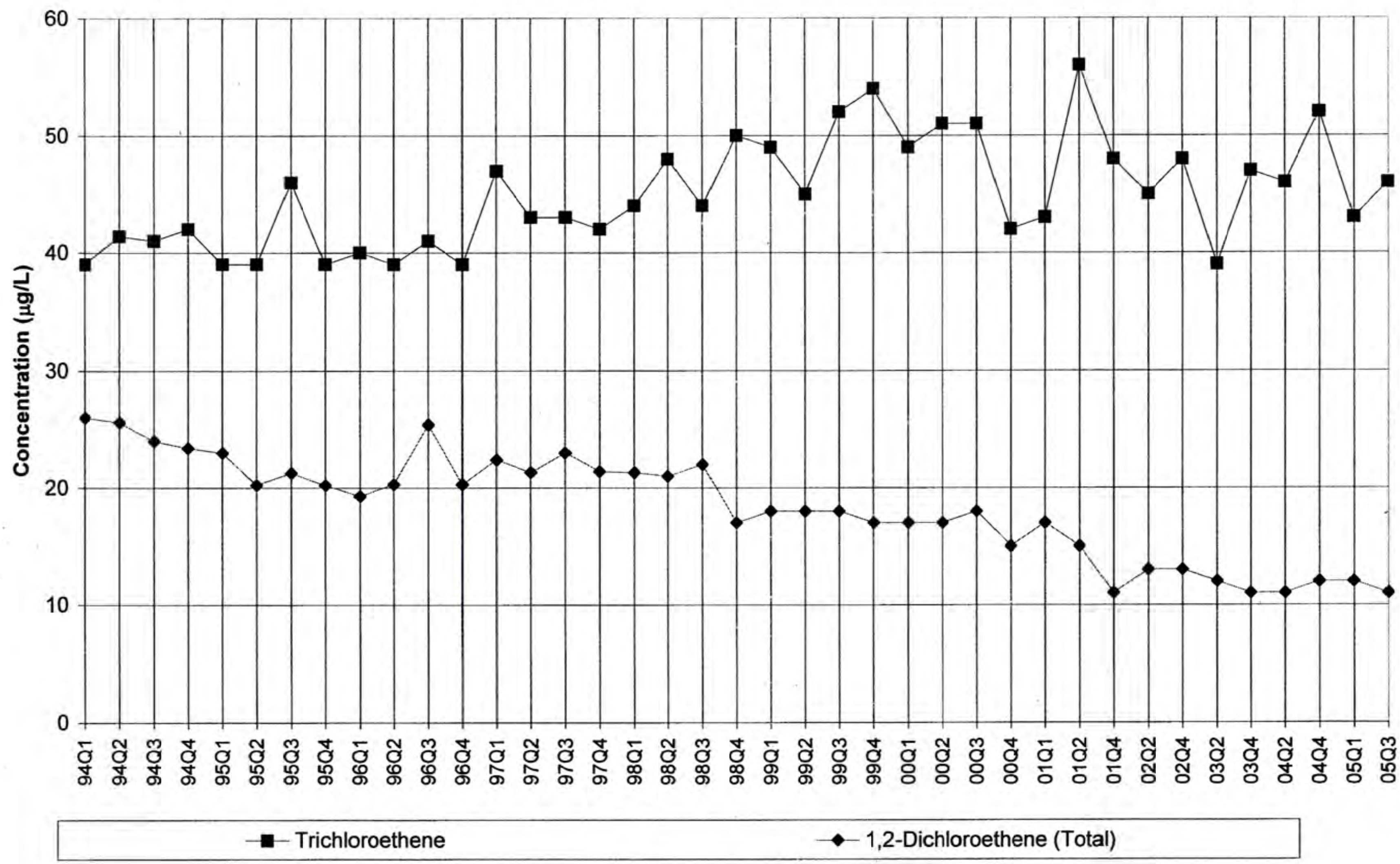
ATTACHMENT 2

Aquifers 2 and 3
VOC Time Series Plots



Not detected constituents are plotted as 0 µg/L

Figure C-6	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well B(2)**



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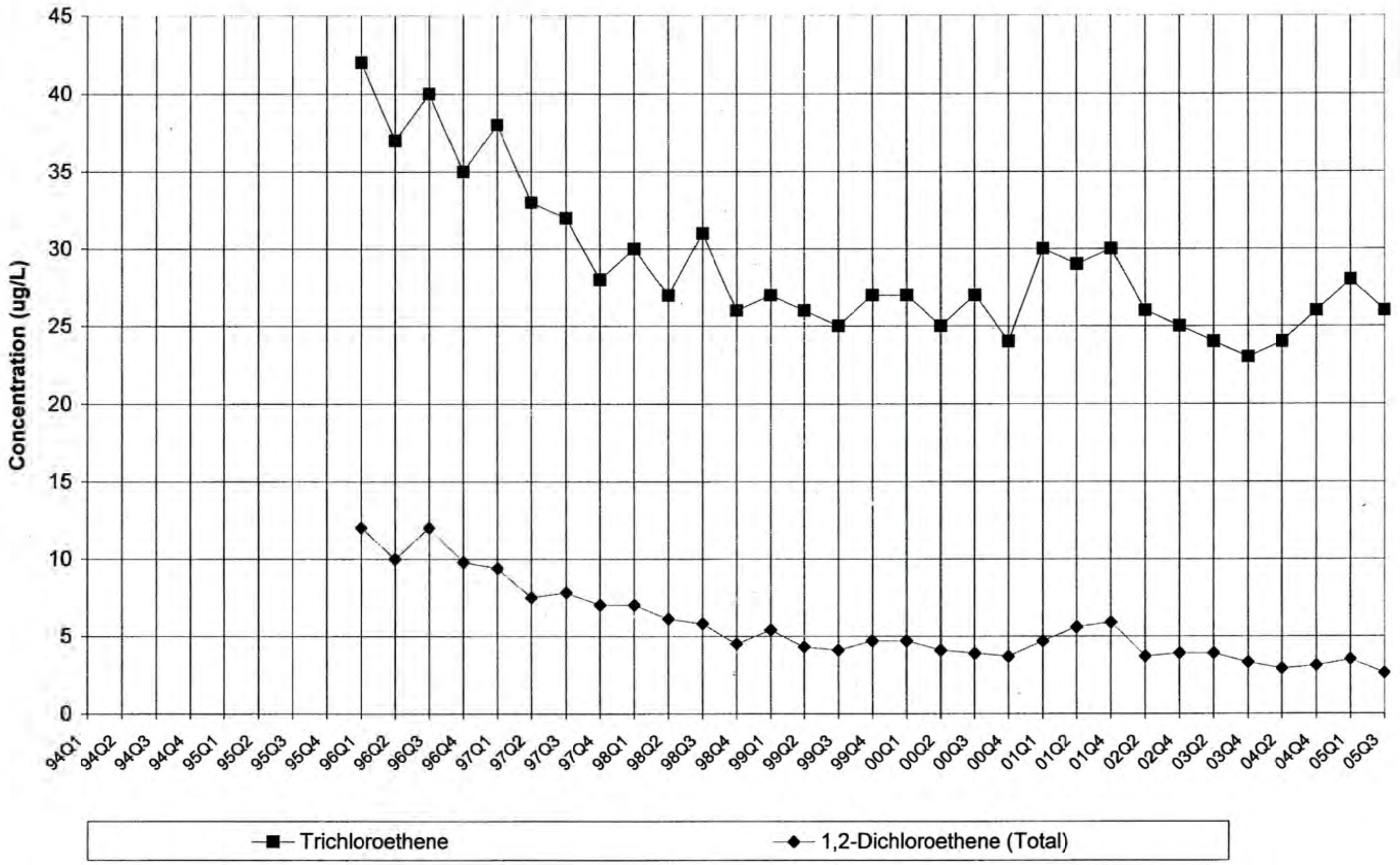
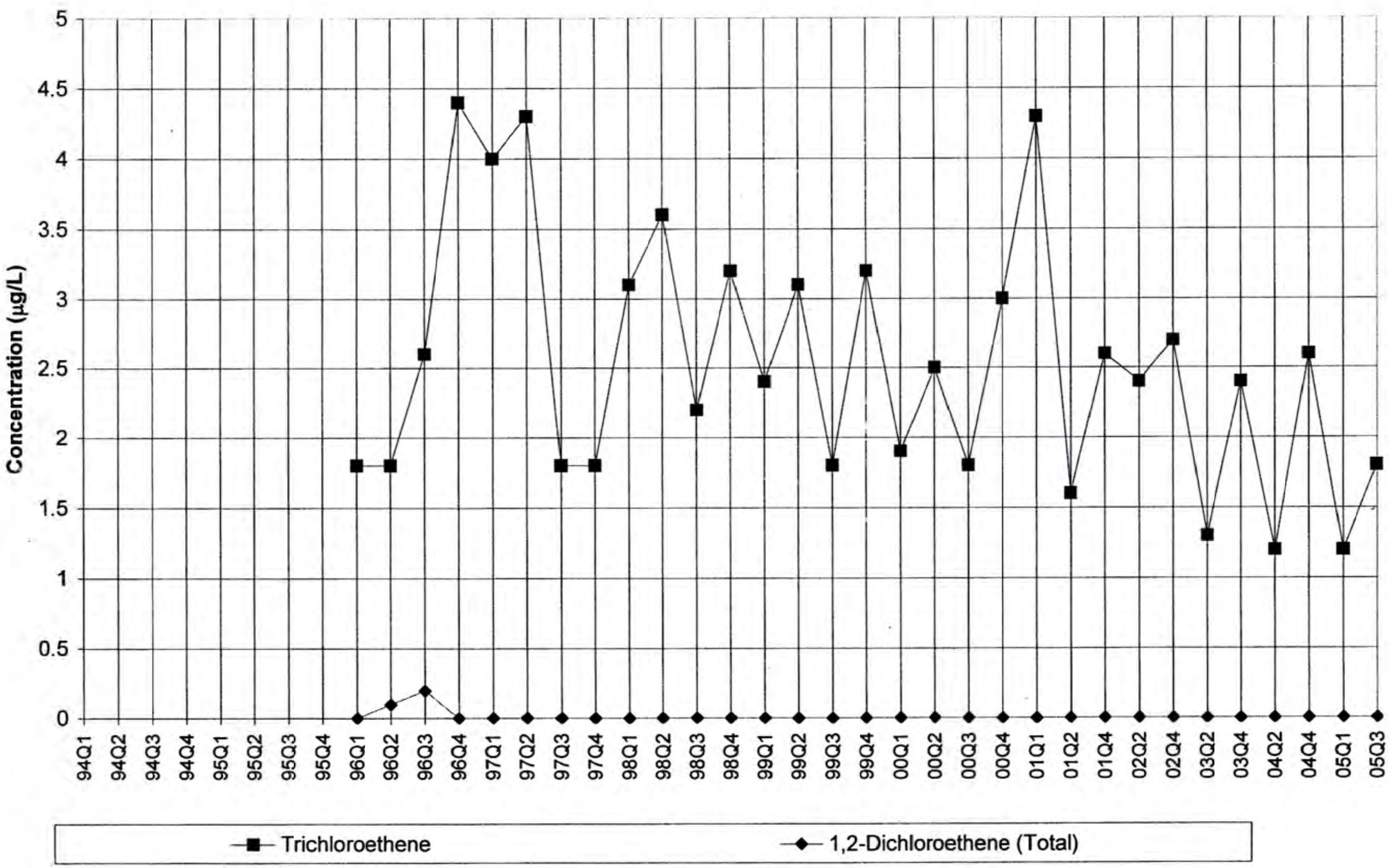


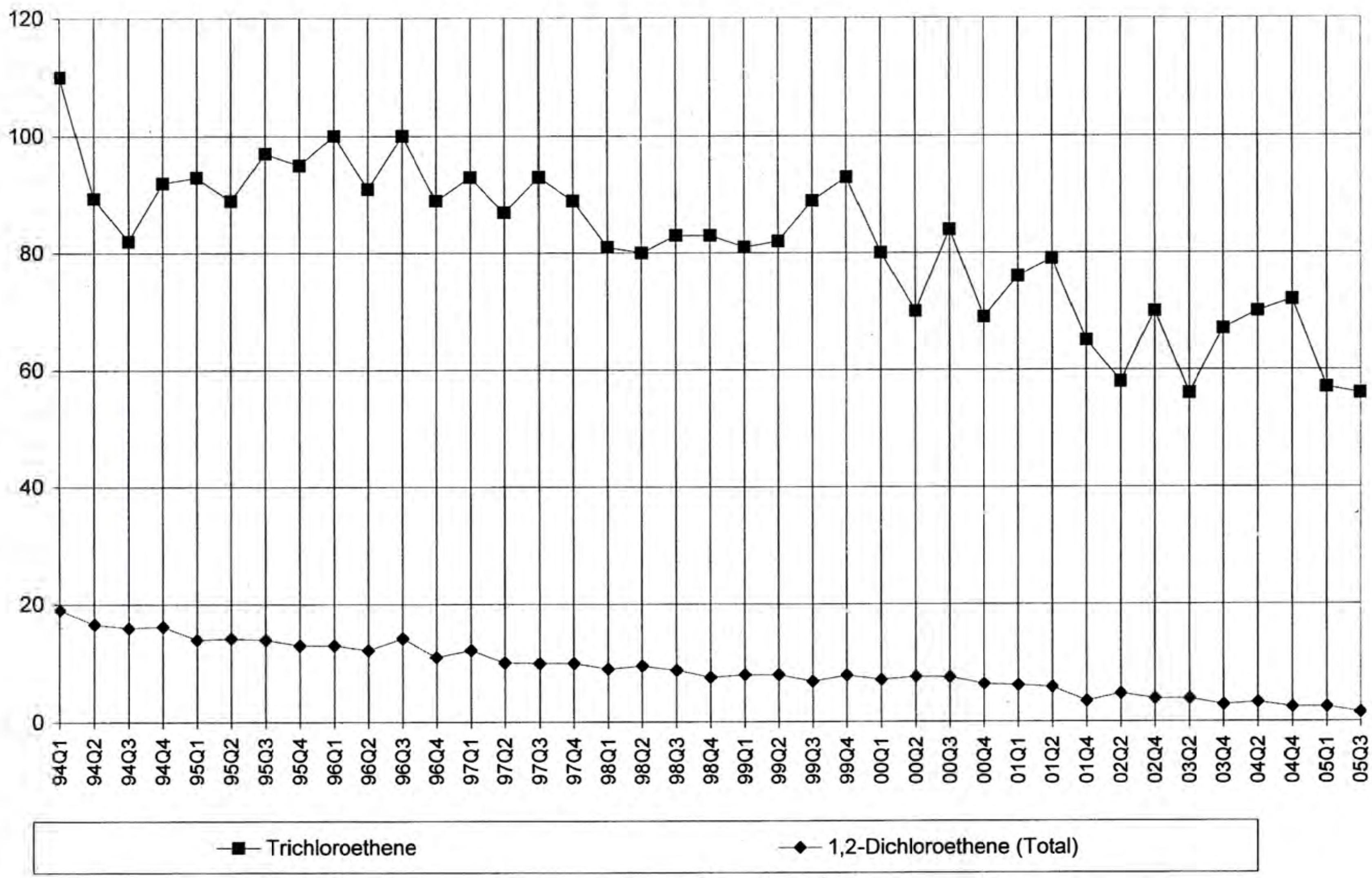
Not detected constituents are plotted as 0 µg/L

Figure C-8 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well C(2a)**



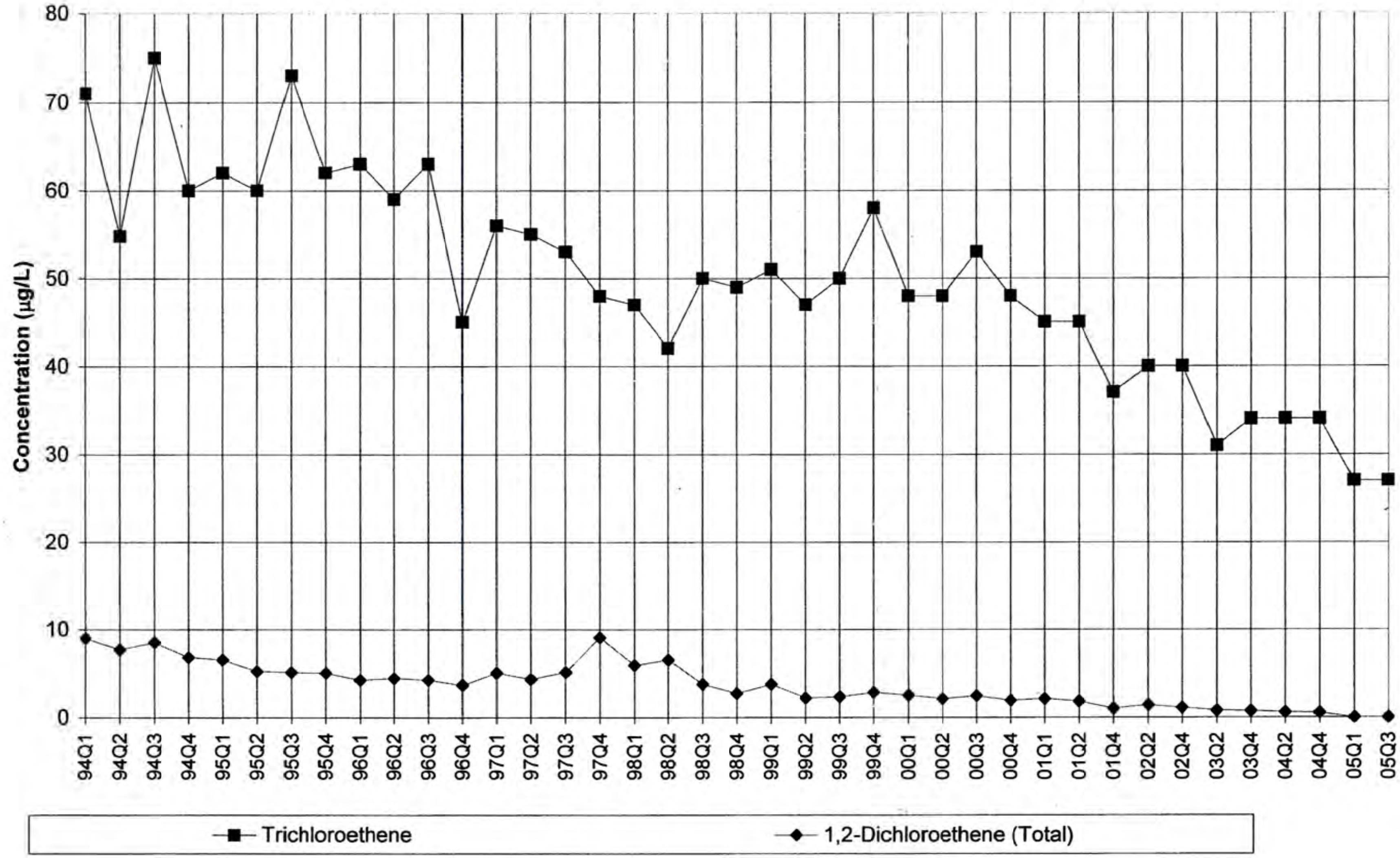
Not detected constituents are plotted as 0 µg/L

Figure C-9	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well D(2a)**



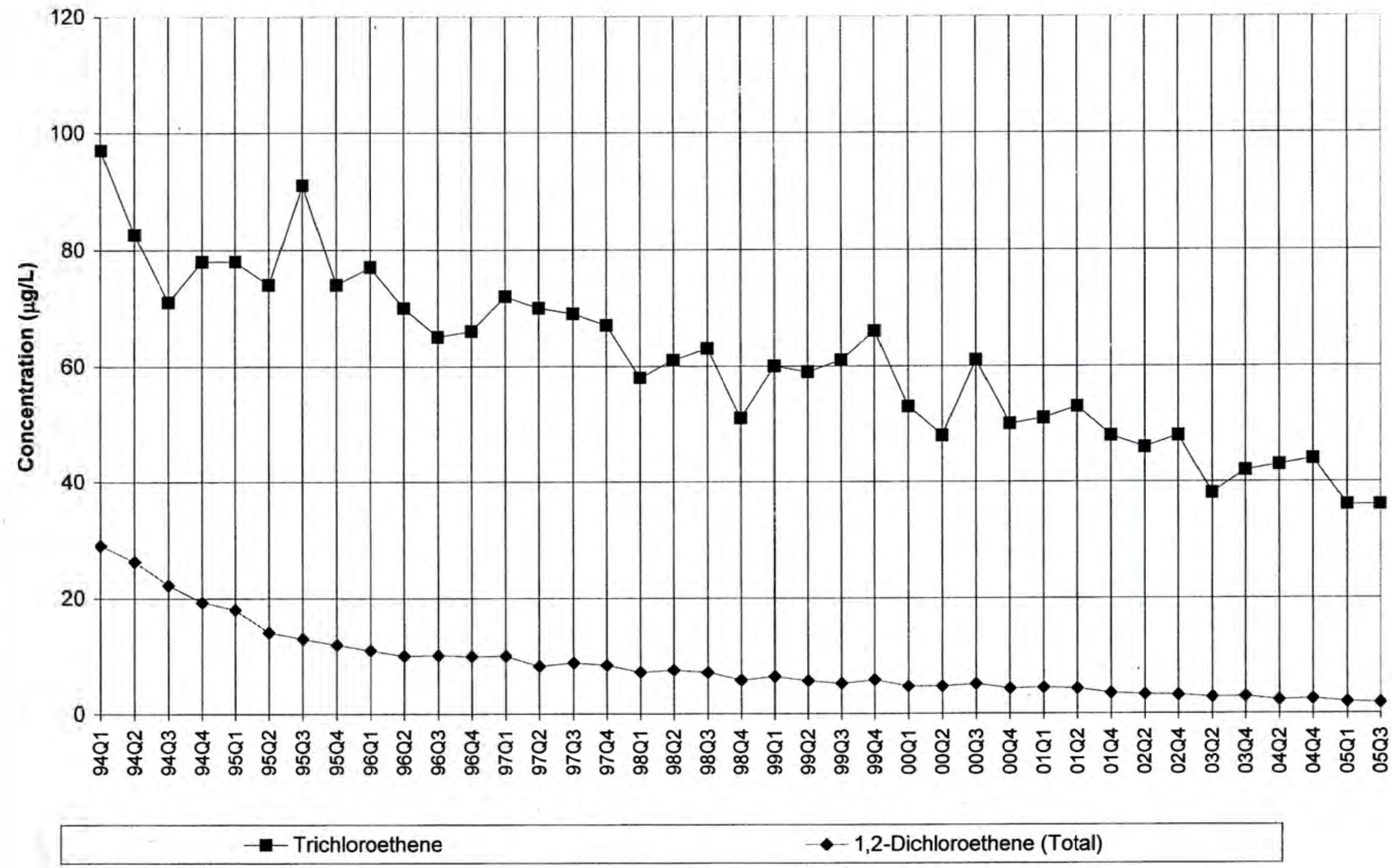
Not detected constituents are plotted as 0 µg/L

Figure C-10 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well E(2)**



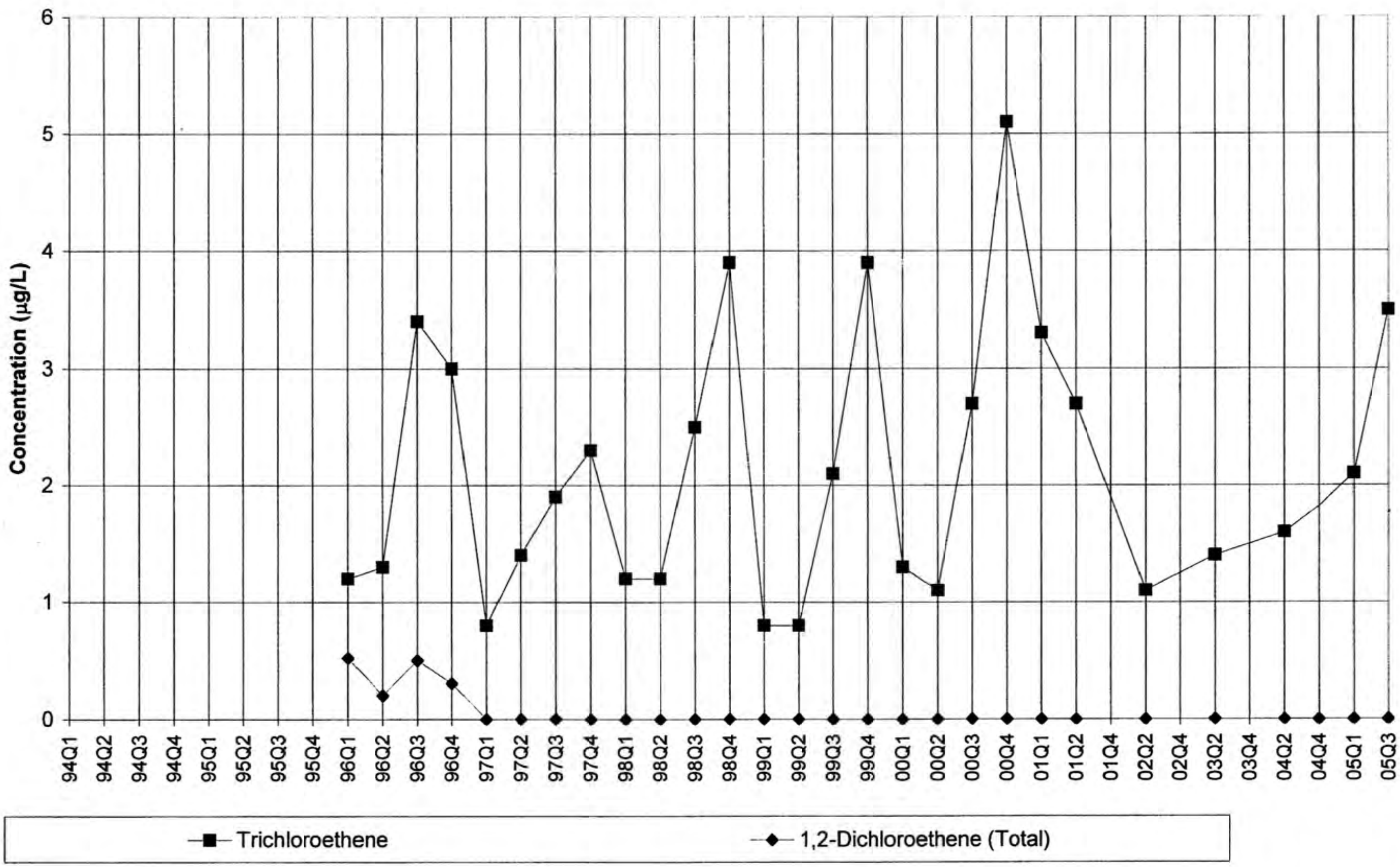
Not detected constituents are plotted as 0 µg/L

Figure C-11 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well E(2a)**



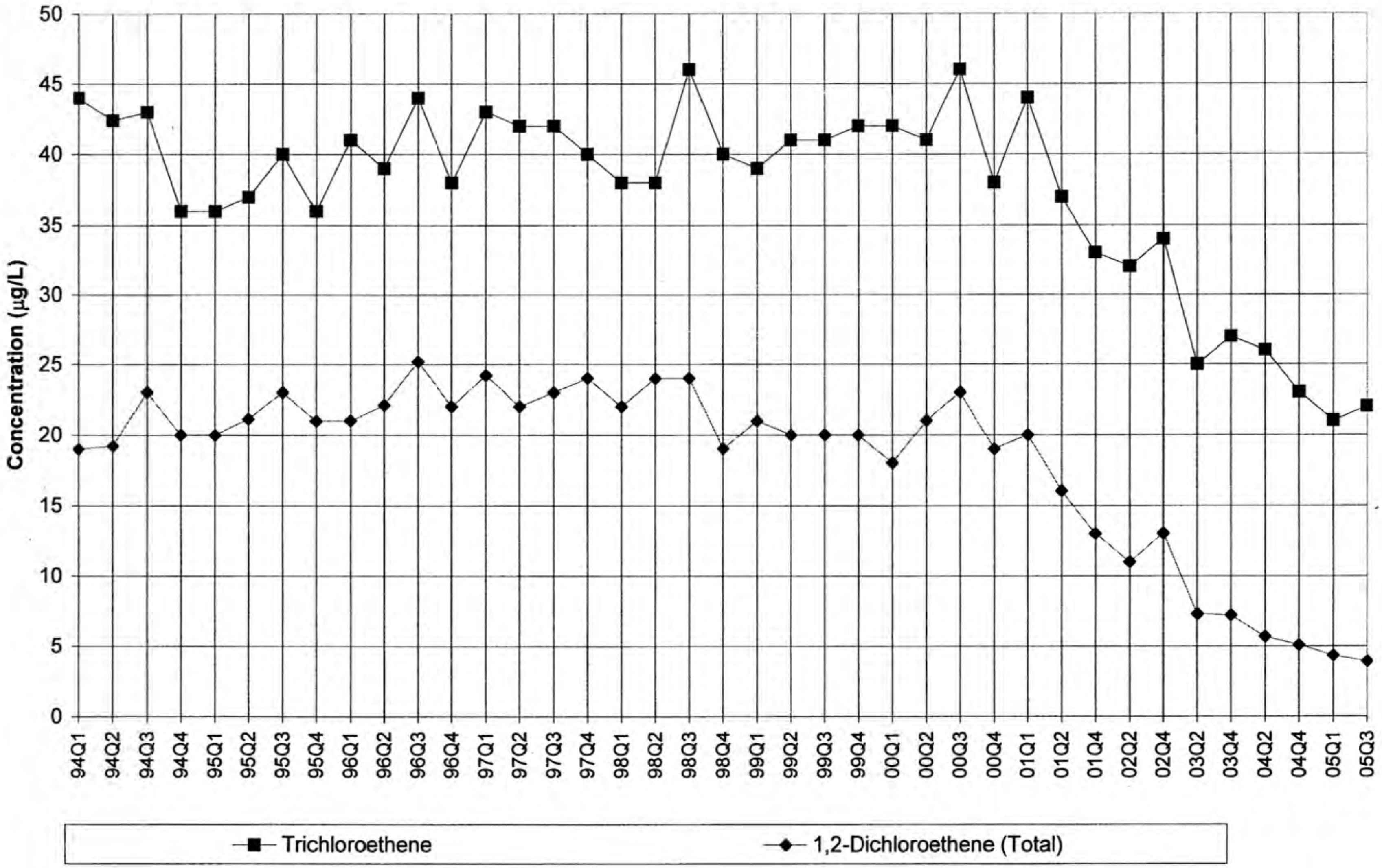
Not detected constituents are plotted as 0 µg/L

Figure C-12 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well F(2)**



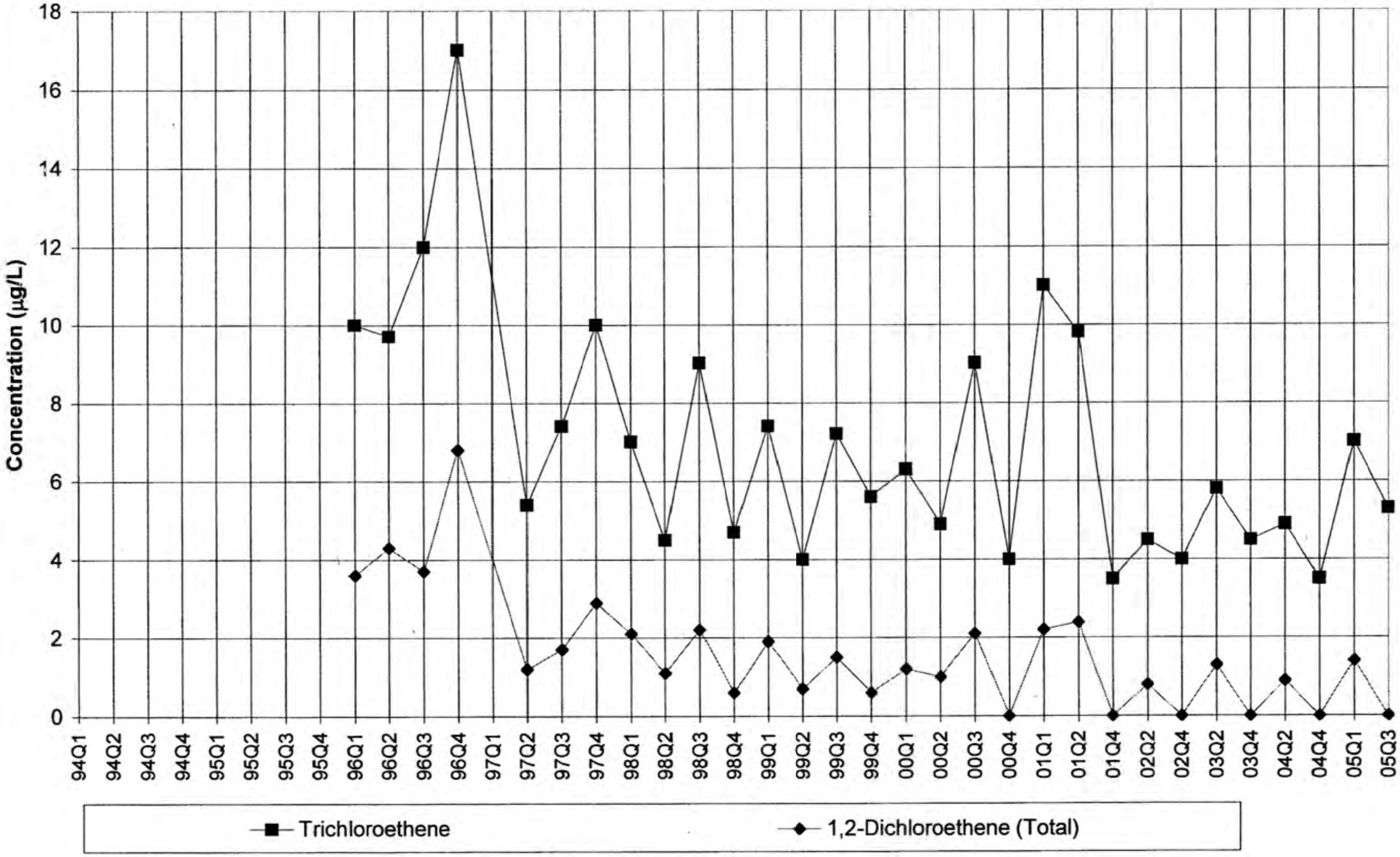
Not detected constituents are plotted as 0 µg/L

Figure C-13	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well F(2a)**

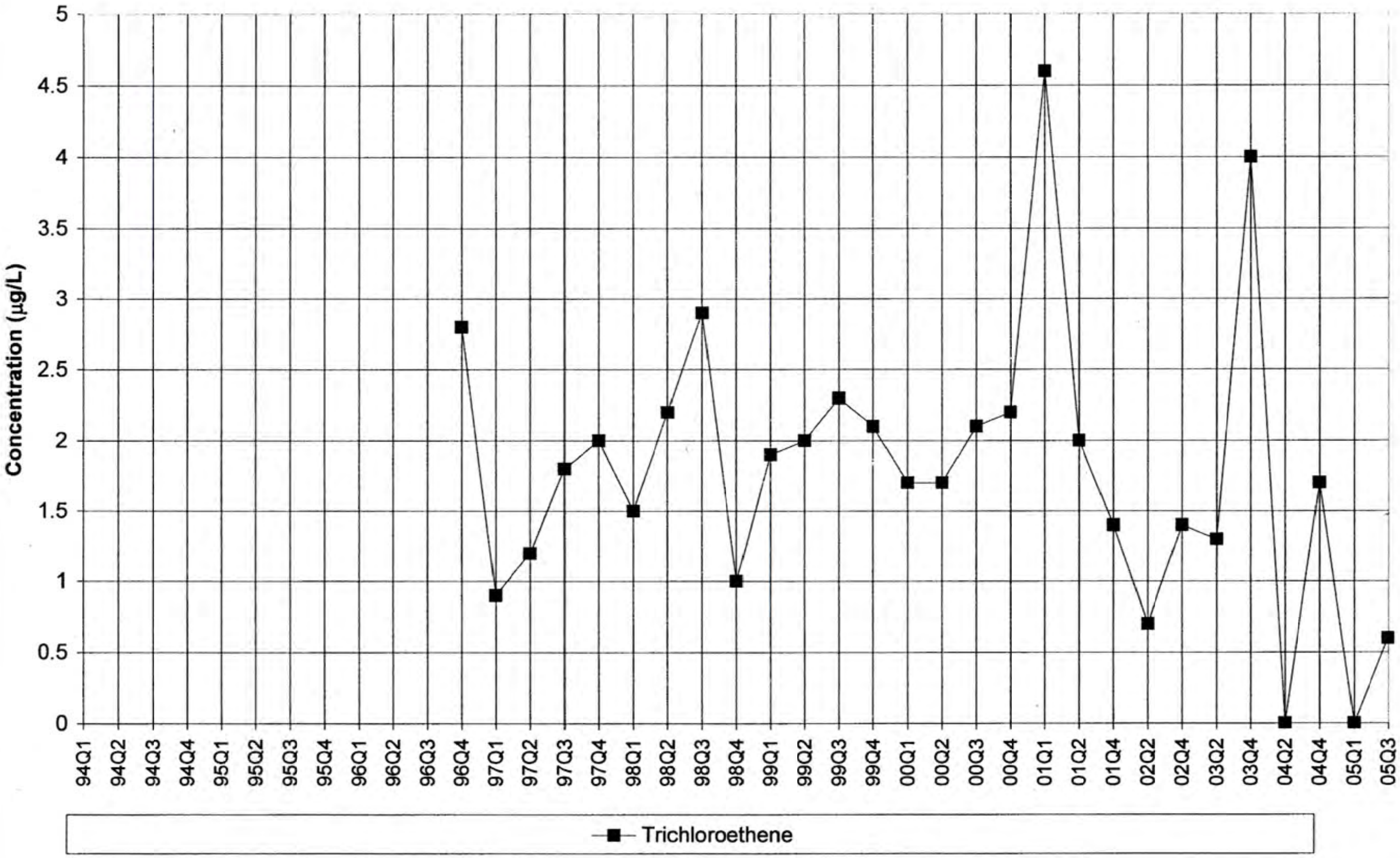


Not detected constituents are plotted as 0 µg/L

Figure C-14	Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well G(2)



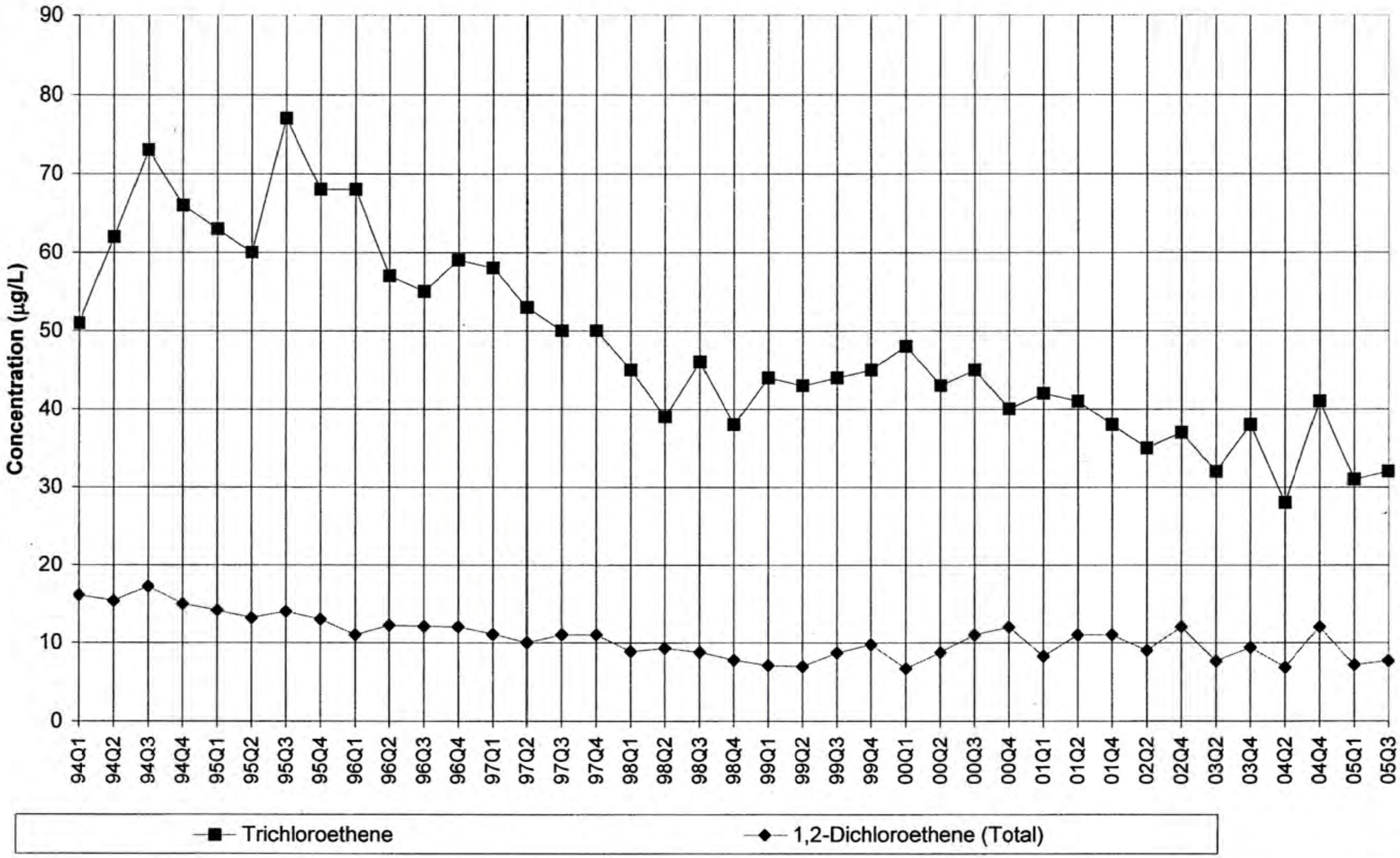
Not detected constituents are plotted as 0 µg/L	**Figure C-15**	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well G(2a)**



Not detected constituents are plotted as 0 µg/L

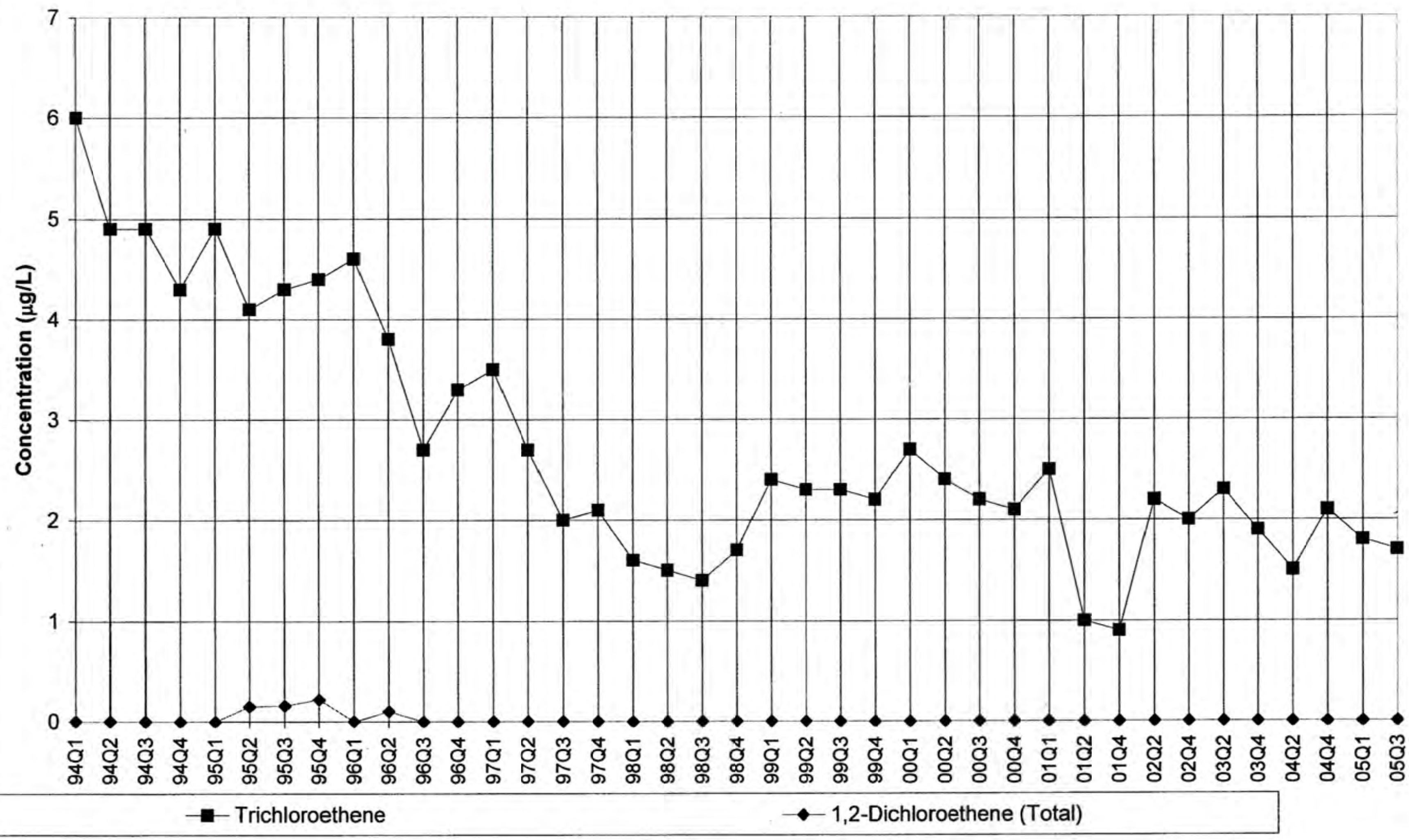
Figure C-16

Trichloroethene Concentrations for Well H(2a)



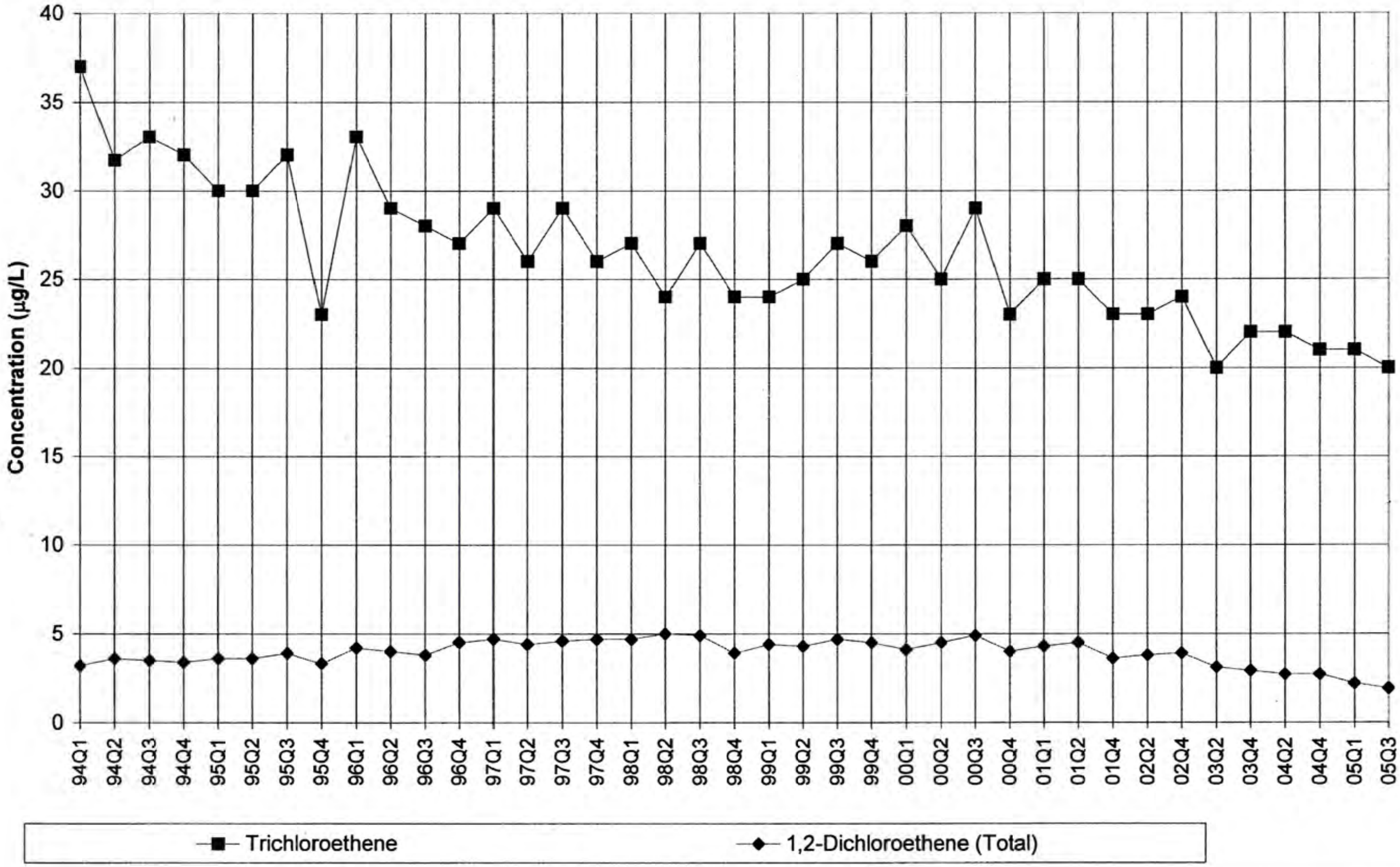
Not detected constituents are plotted as 0 µg/L

Figure C-17 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well I(2)**



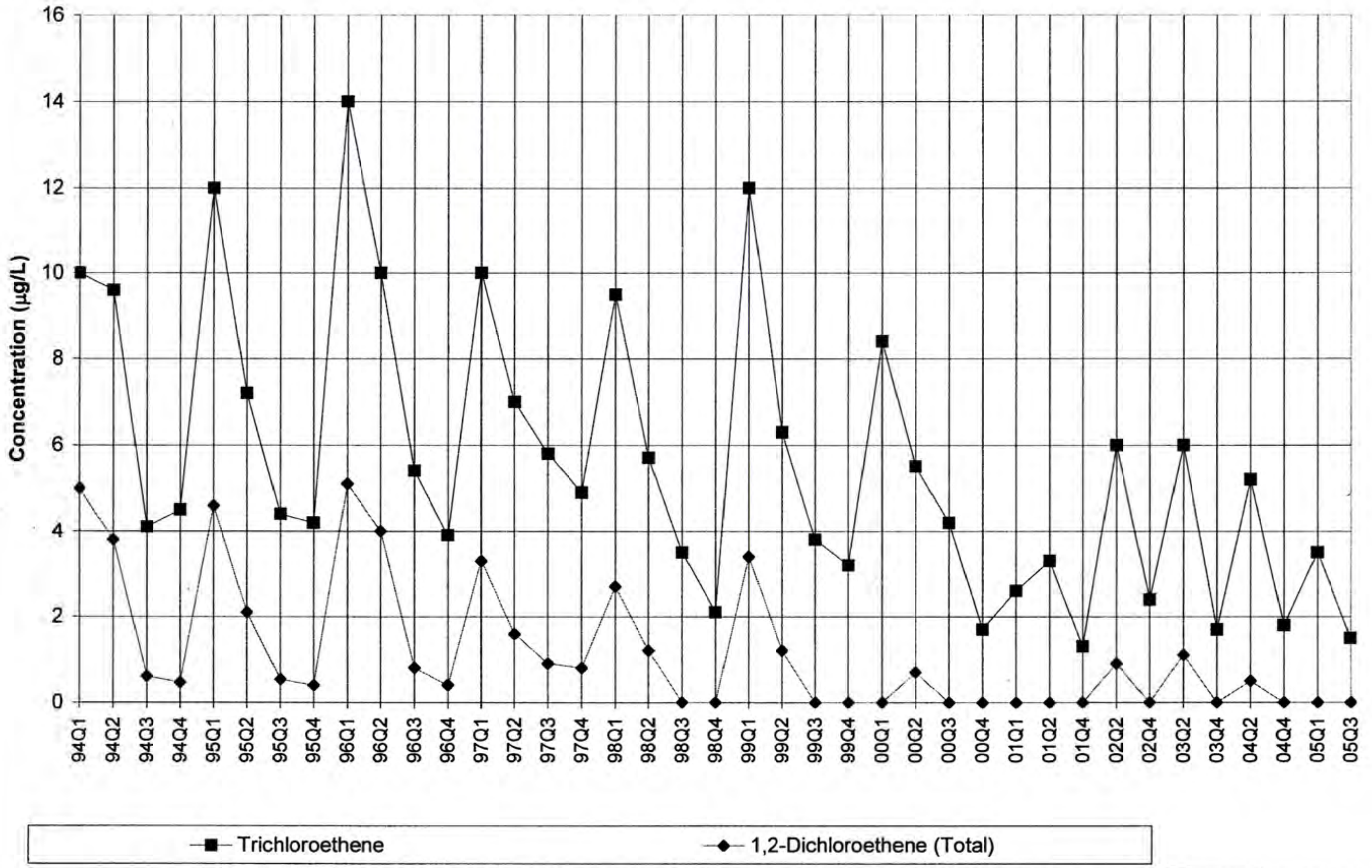
Not detected constituents are plotted as 0 µg/L

Figure C-18 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well I(2a)**



Not detected constituents are plotted as 0 µg/L

Figure C-19 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well L(2)**



Not detected constituents are plotted as 0 µg/L	**Figure C-20**	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well L(2a)**

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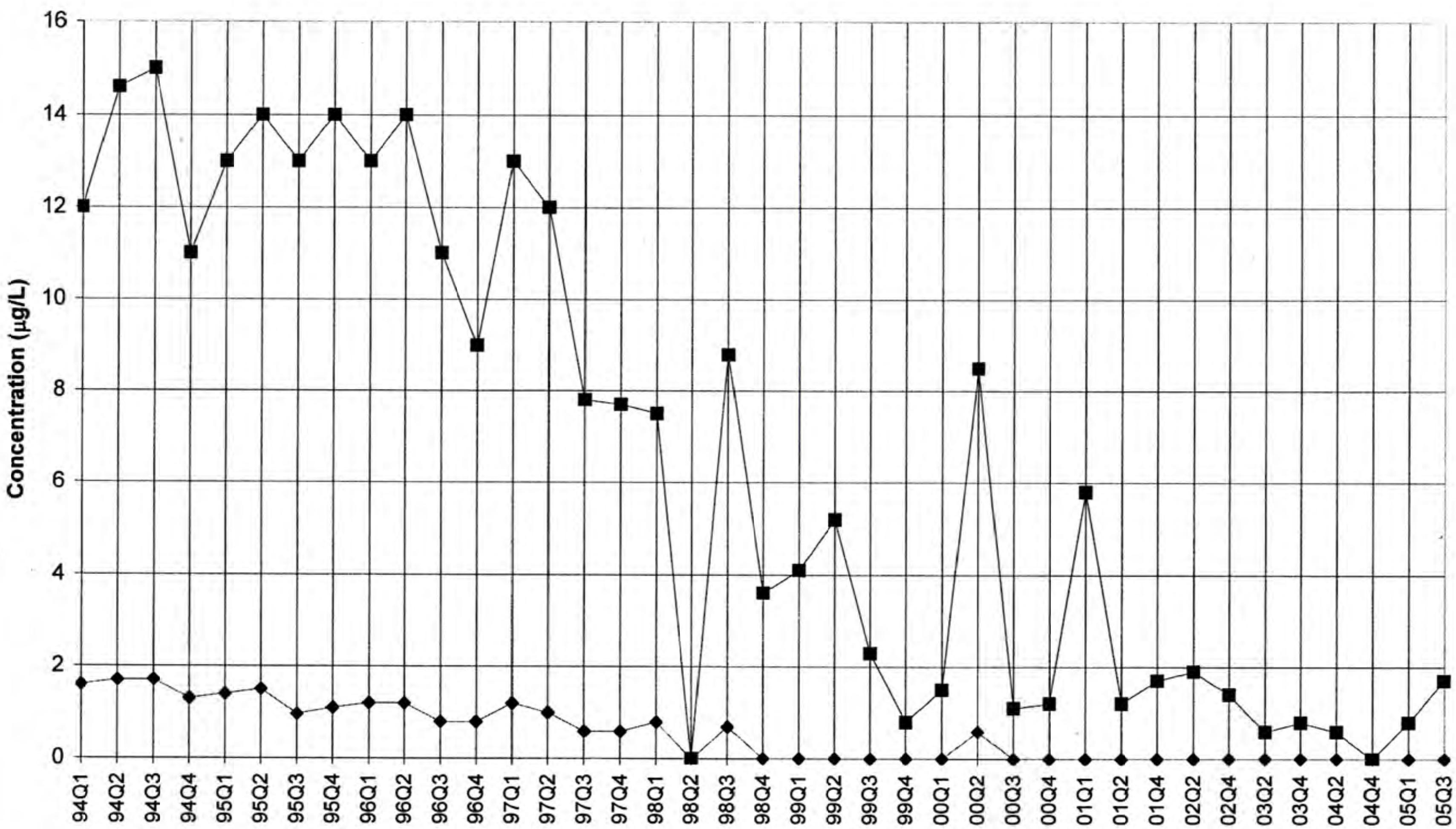
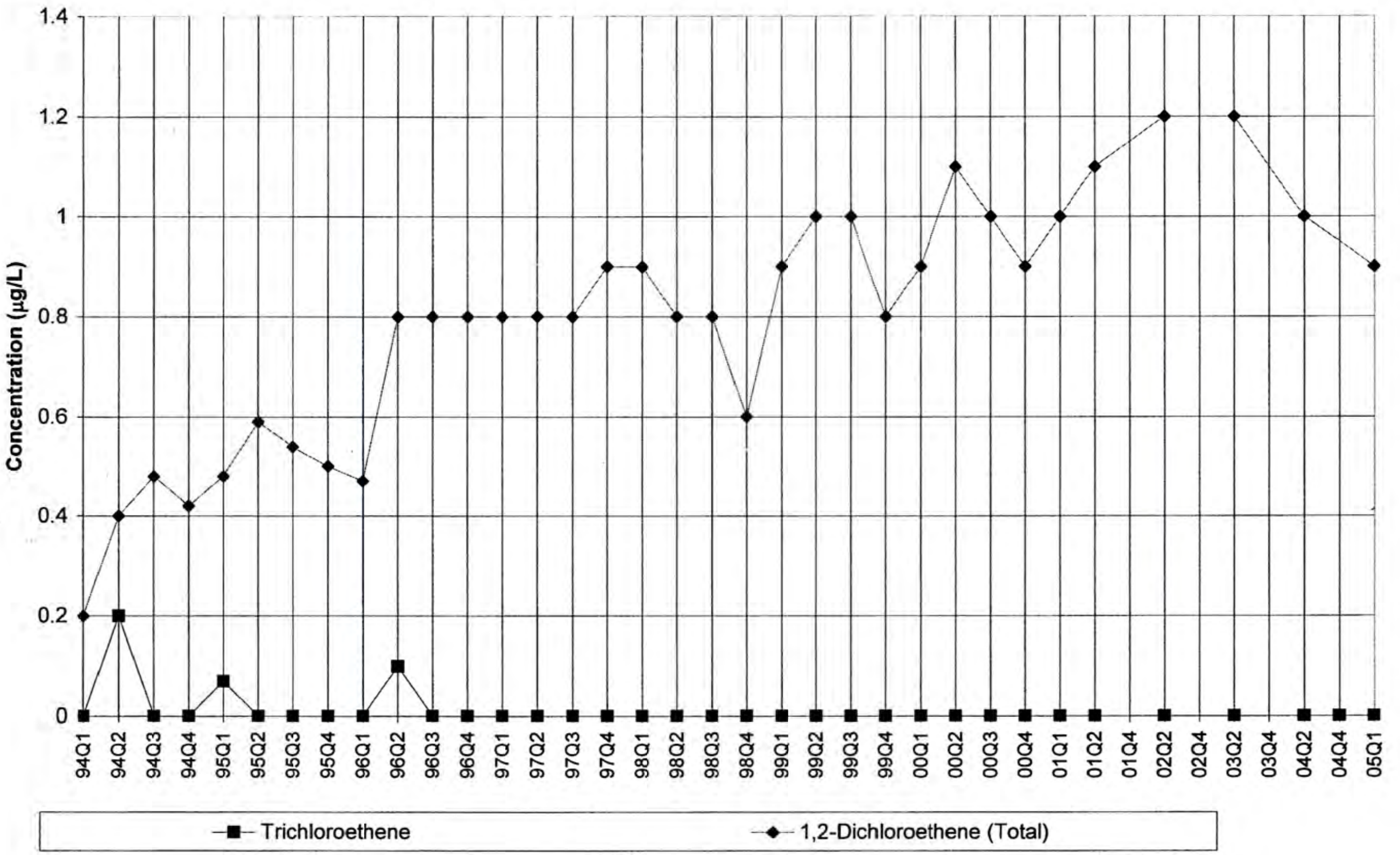
5/4/2006

EcoChem, Inc.

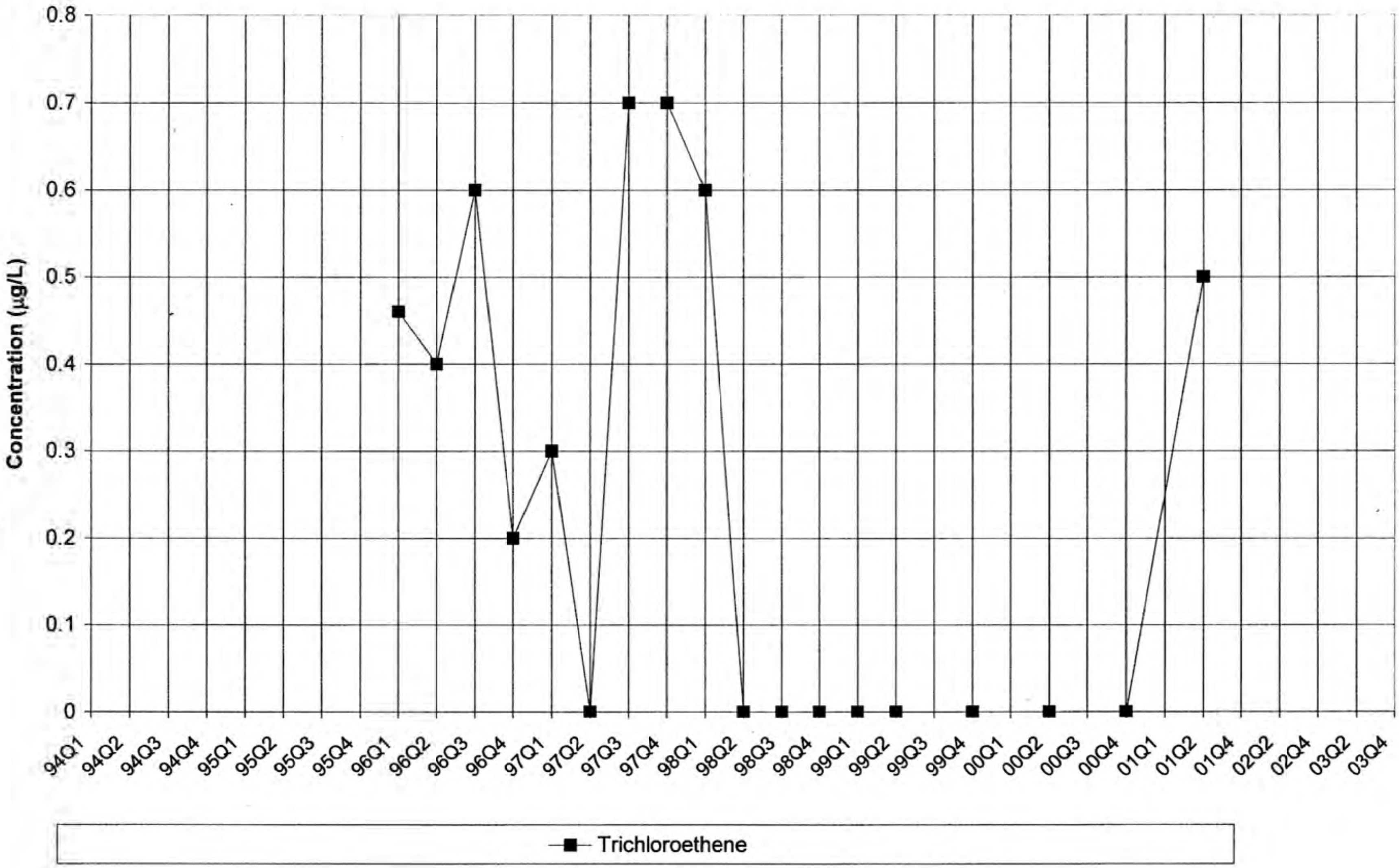


Not detected constituents are plotted as 0 µg/L	**Figure C-21**	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well M(2)**

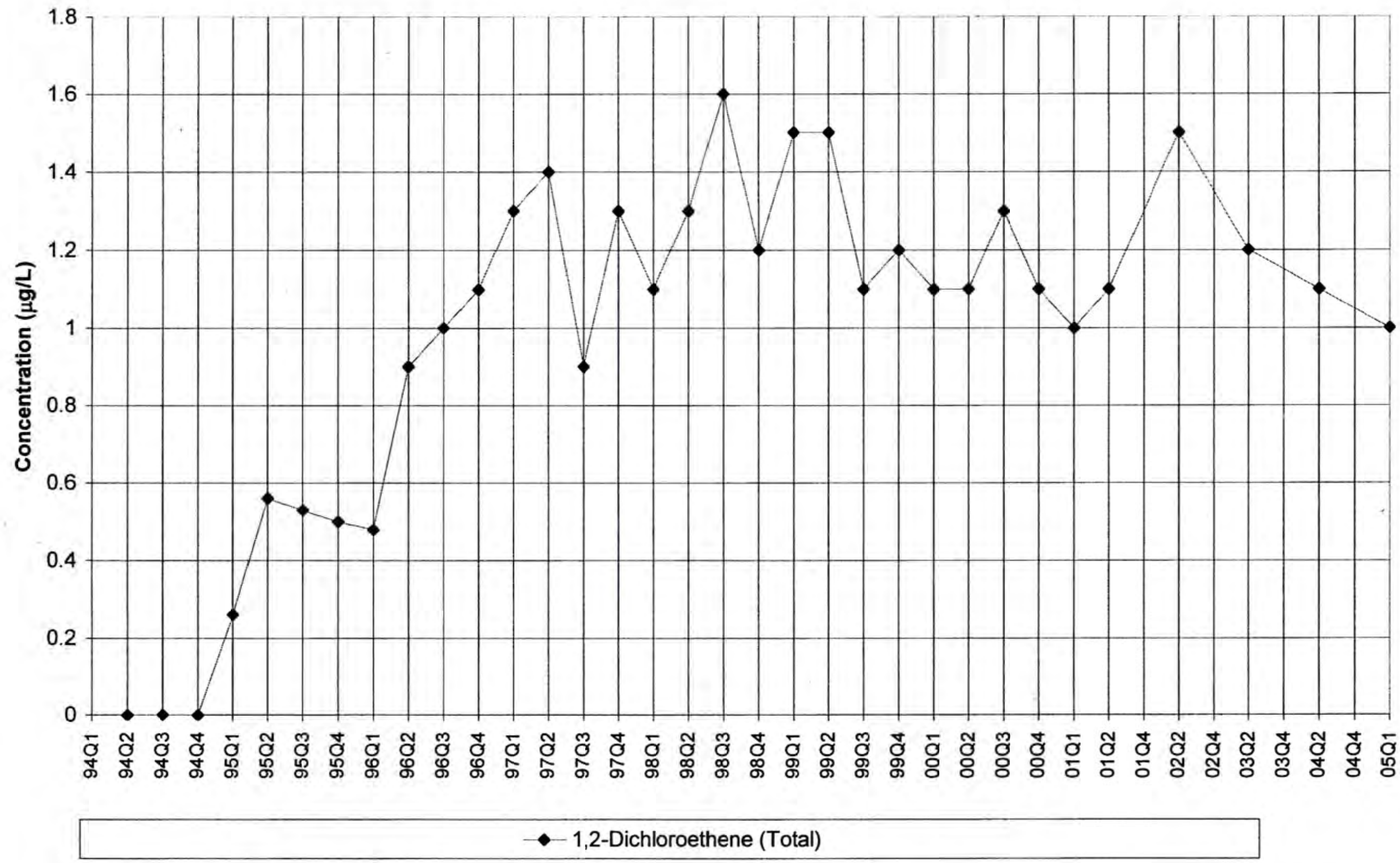


Not detected constituents are plotted as 0 µg/L

Figure C-23 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well N(2)**

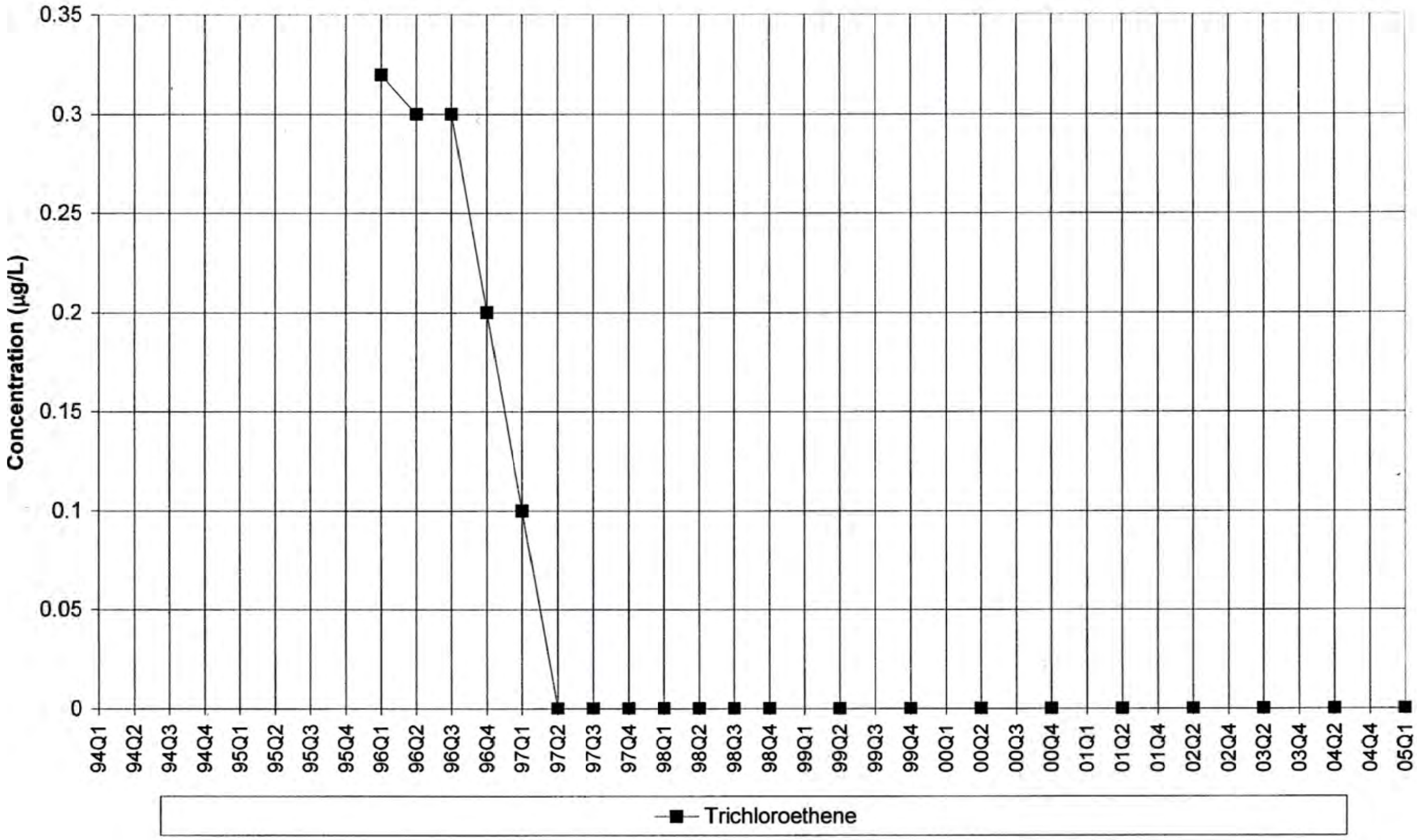


Not detected constituents are plotted as 0 µg/L	**Figure C-24**	**Trichloroethene Concentrations for Well N(2a)**



Not detected constituents are plotted as 0 µg/L

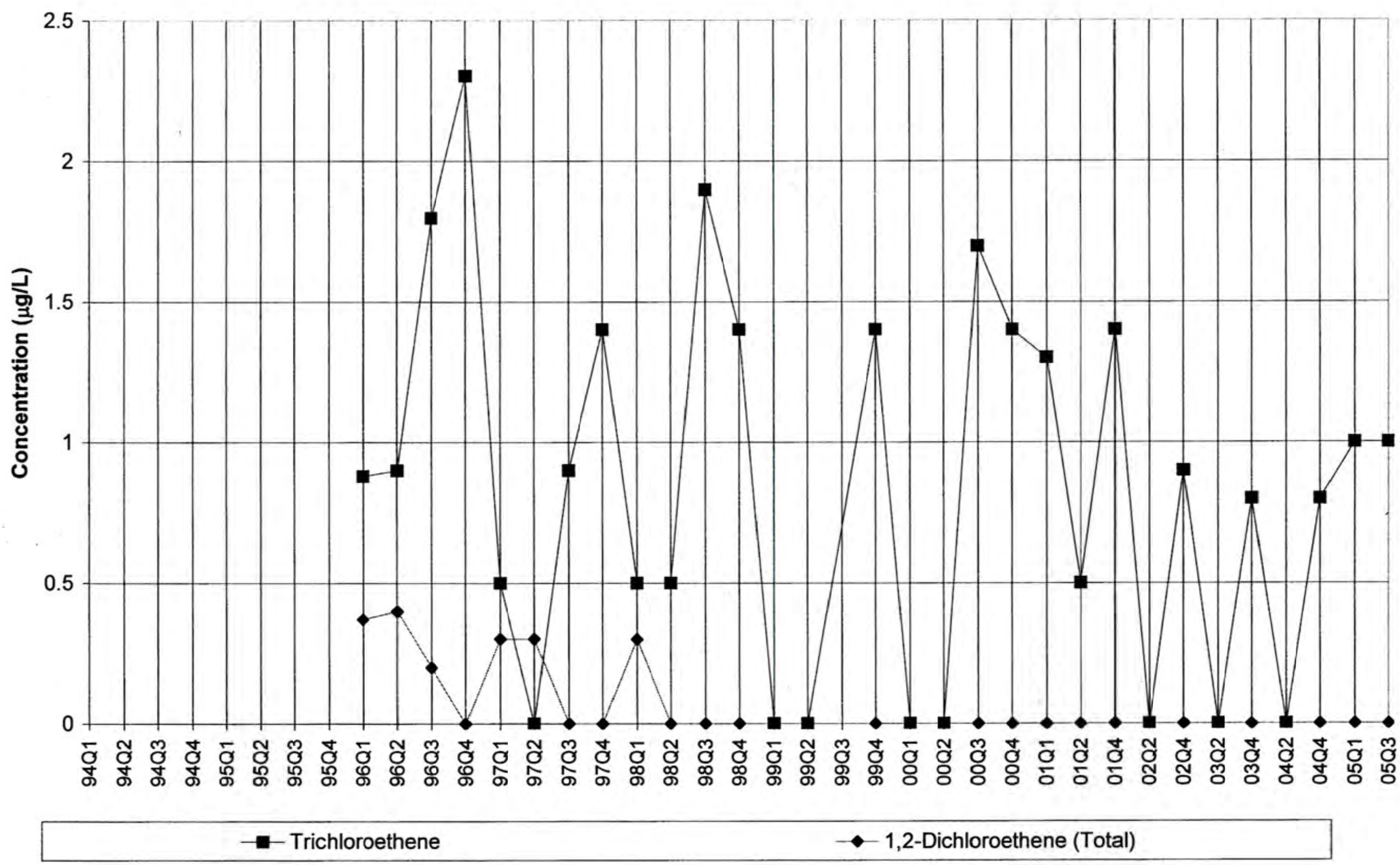
Figure C-25 **Total 1,2-Dichloroethene Concentrations for Well O(2)**



Not detected constituents are plotted as 0 µg/L

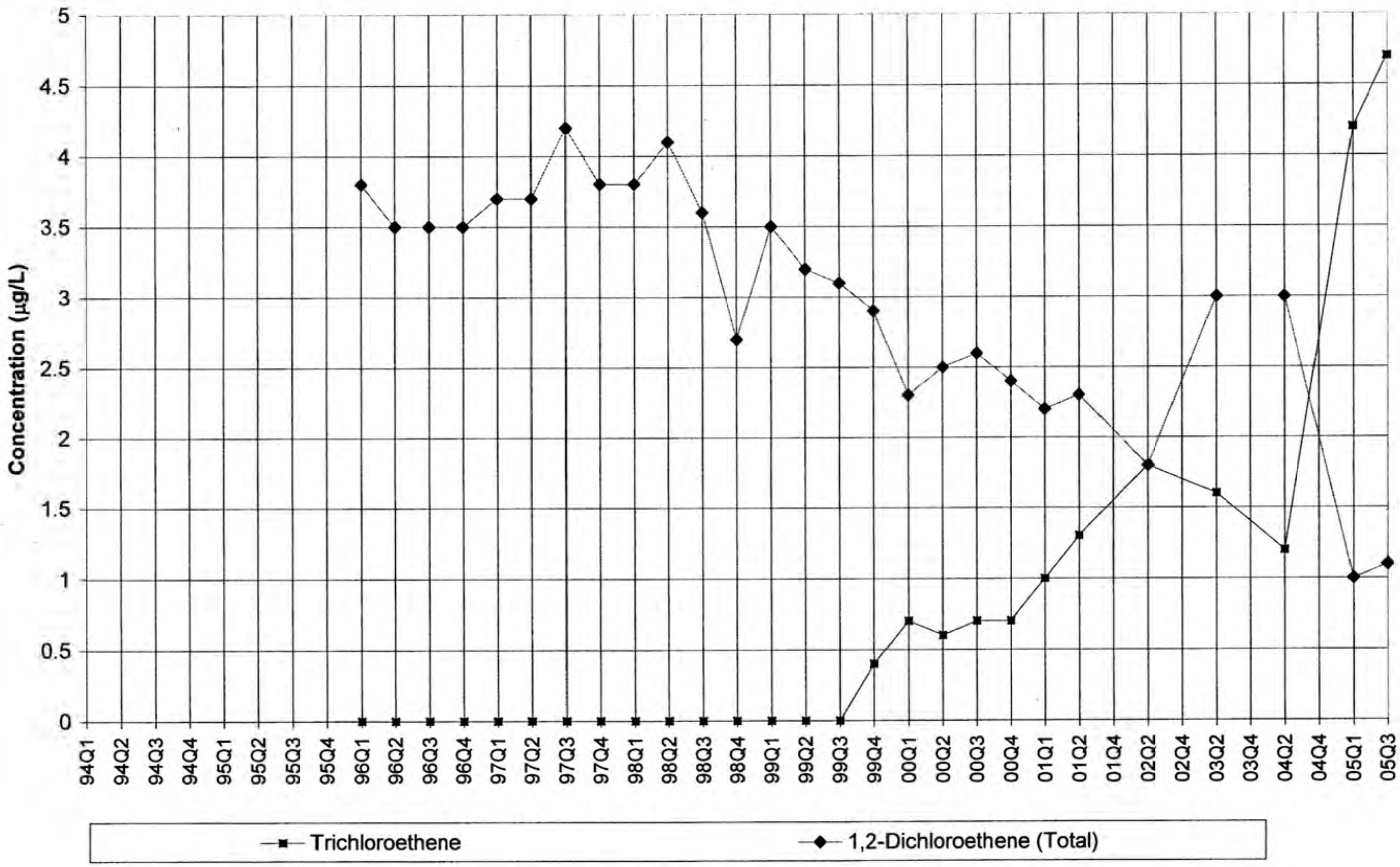
Figure C-26

Trichloroethene Concentrations for Well R(2)



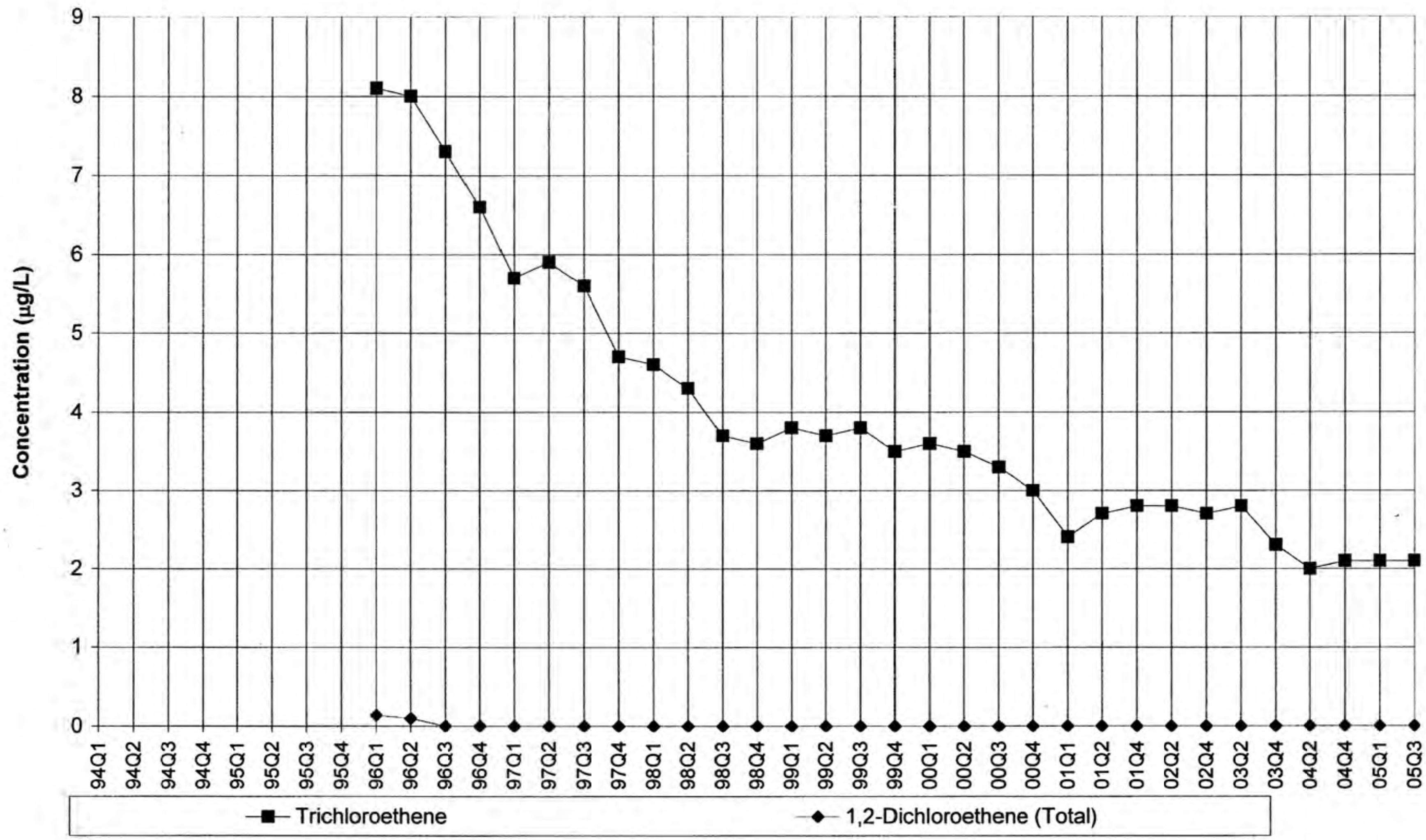
Not detected constituents are plotted as 0 µg/L

Figure C-27 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well R(2a)**



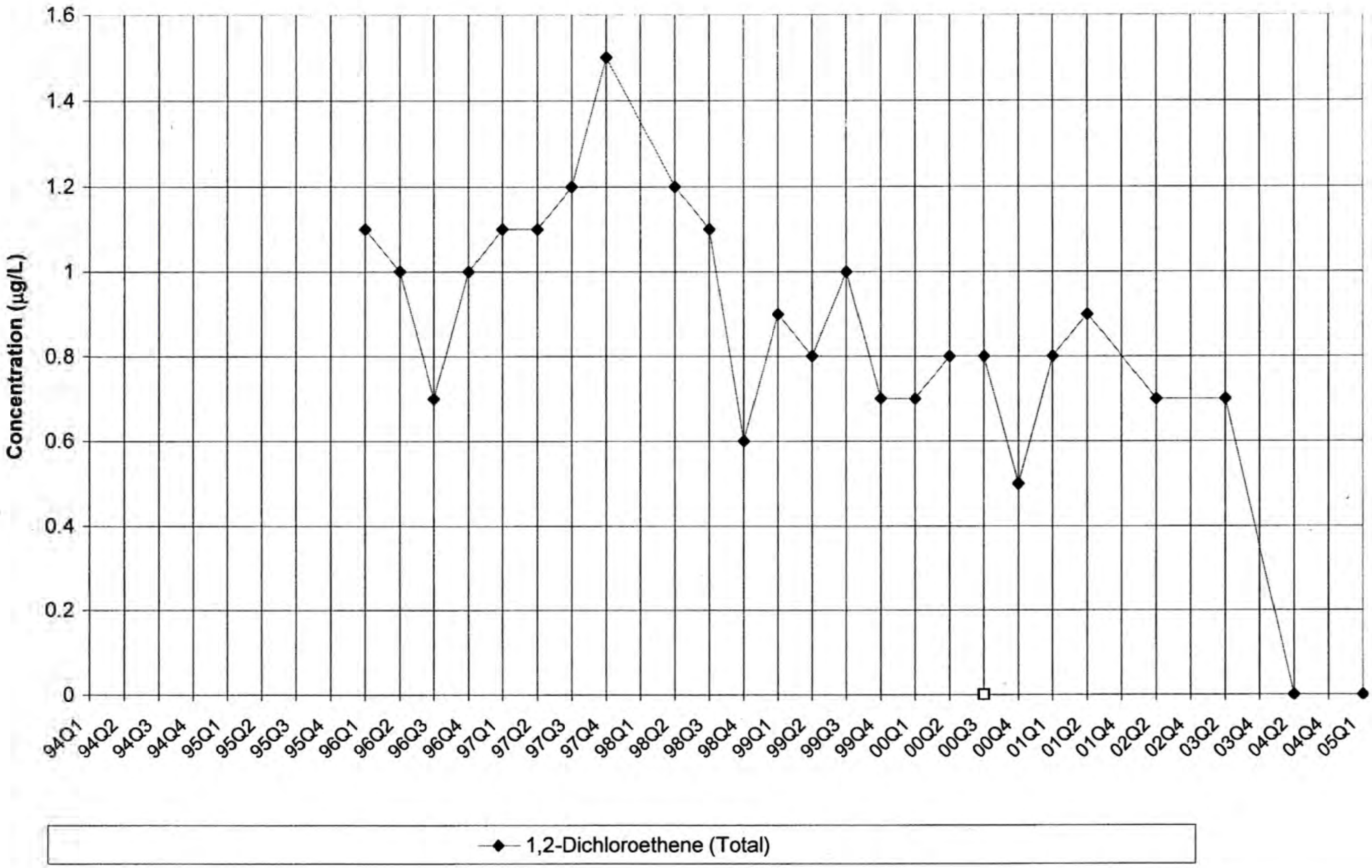
Not detected constituents are plotted as 0 µg/L

Figure C-28 **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well S(2)**



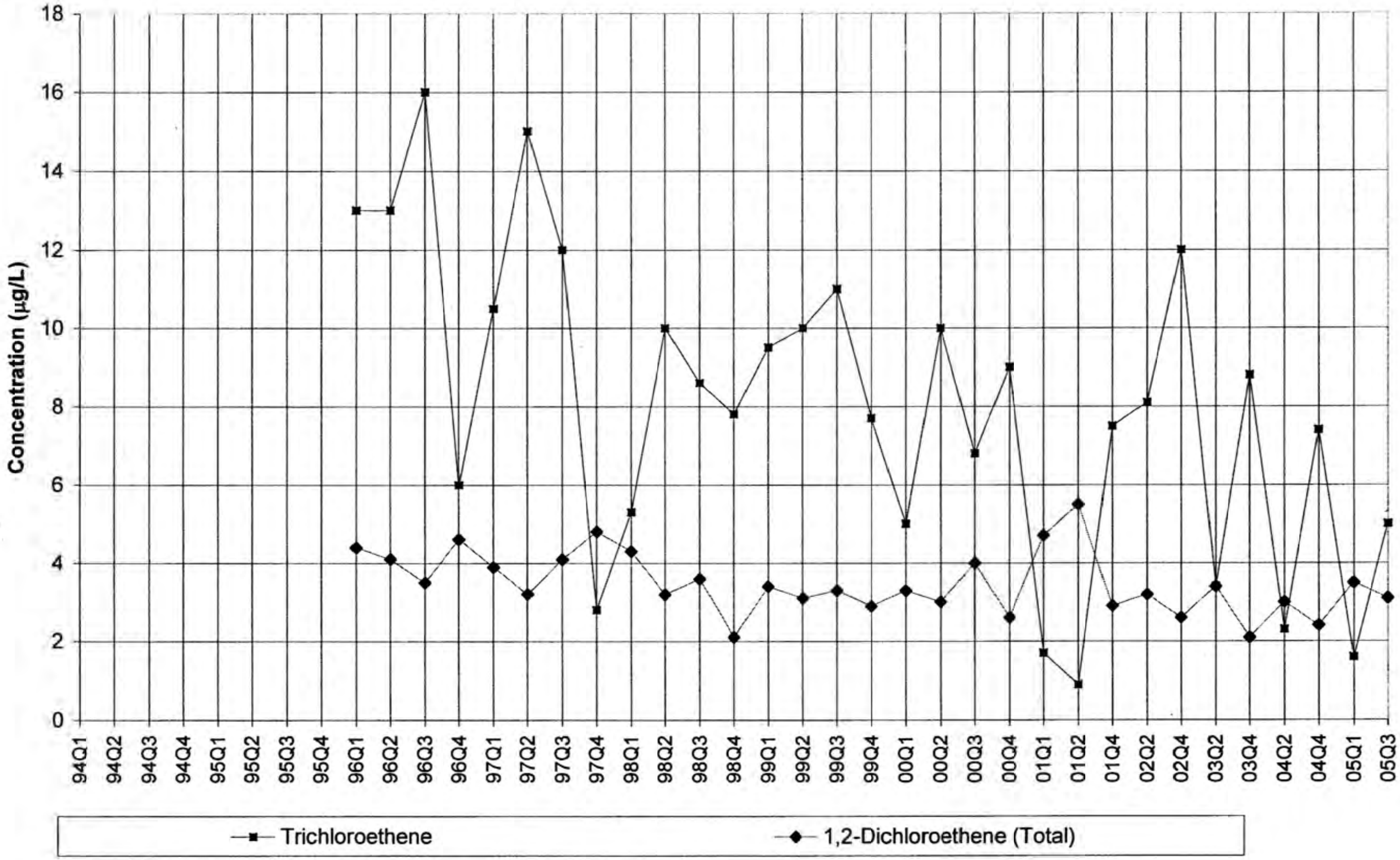
Not detected constituents are plotted as 0 µg/L

Figure C-29 | **Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well S(2a)**

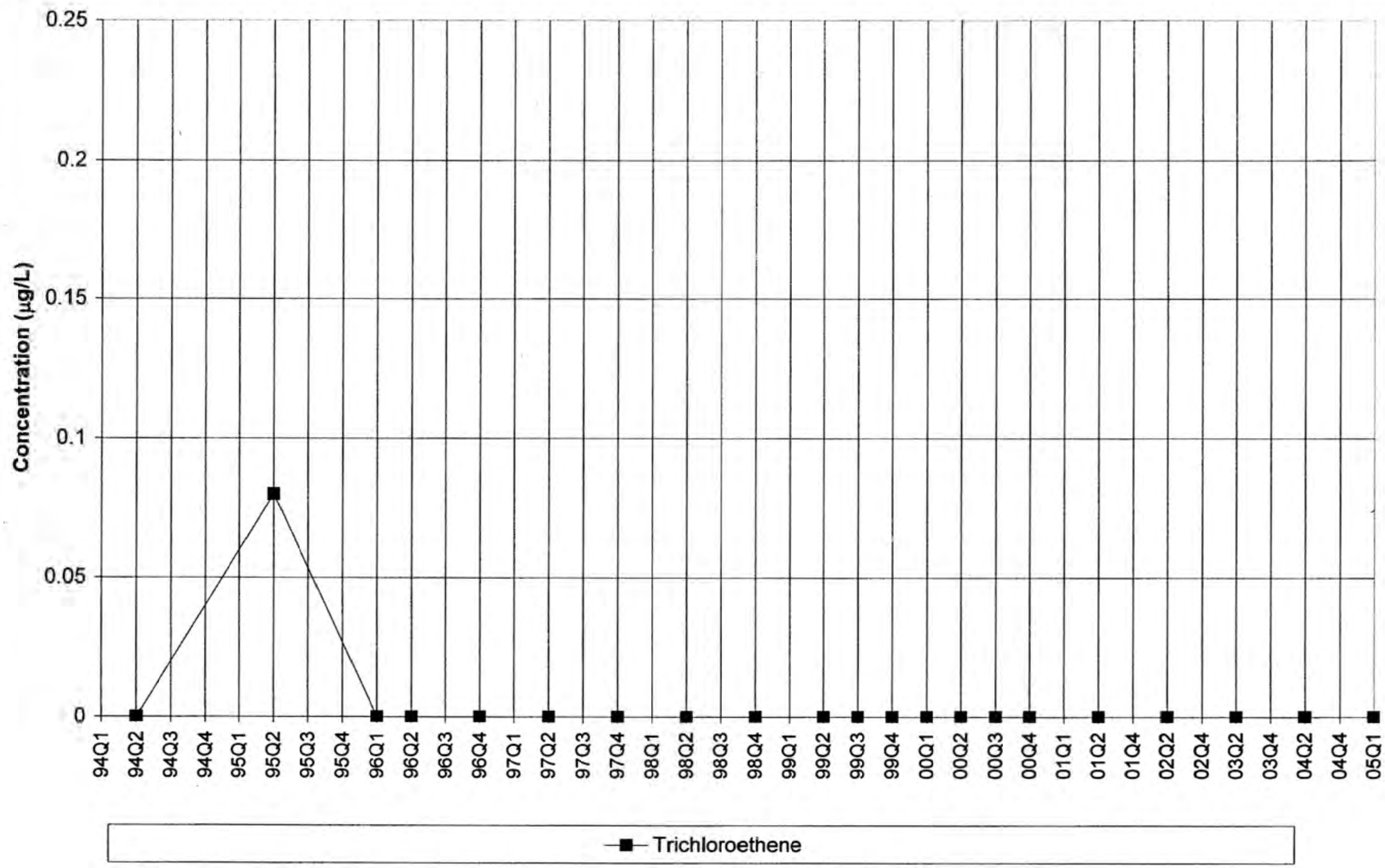


Not detected constituents are plotted as 0 µg/L

Figure C-30 **Total 1,2-Dichloroethene Concentrations for Well U(2)**



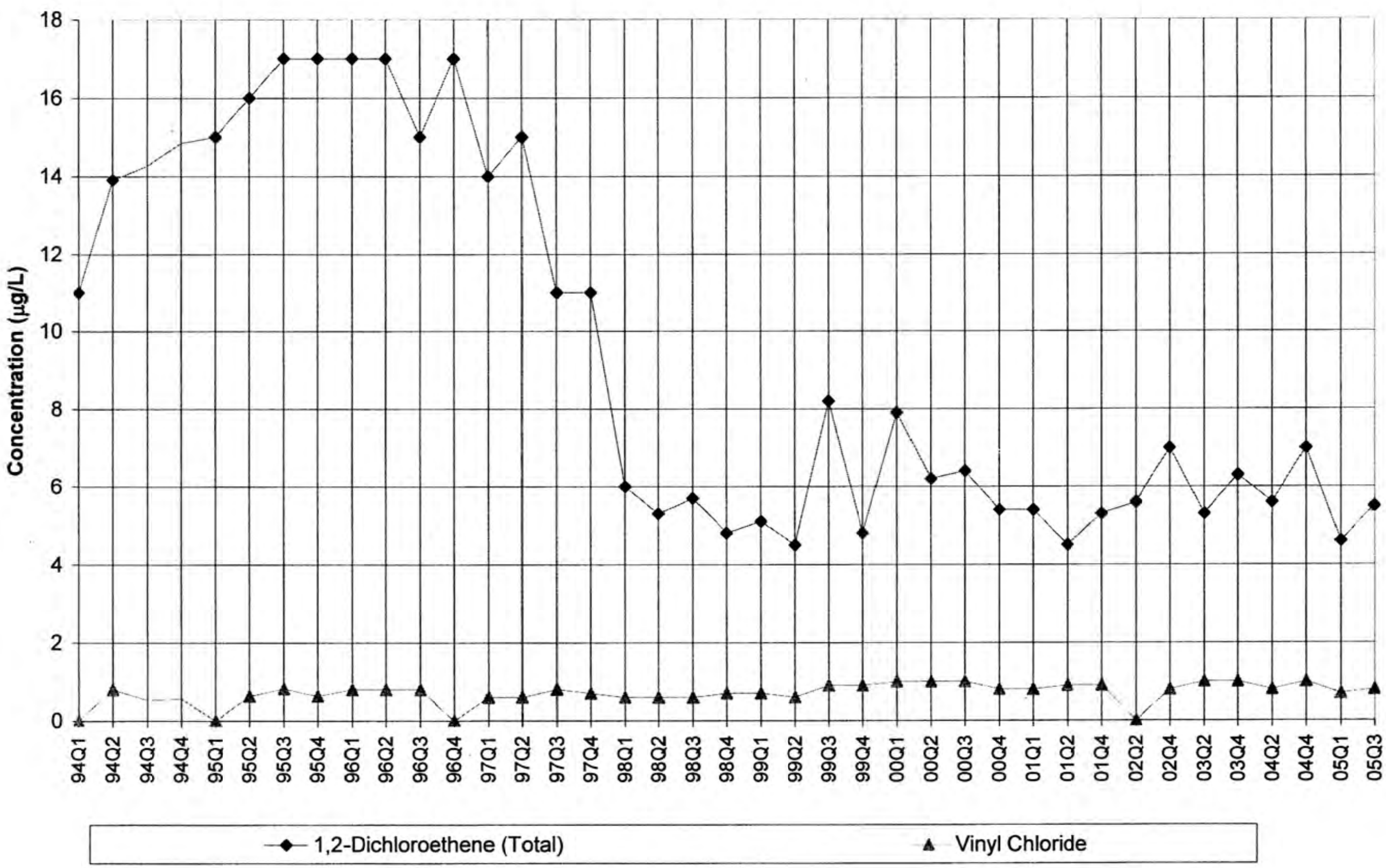
Not detected constituents are plotted as 0 µg/L	**Figure C-31**	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well V(2a)**



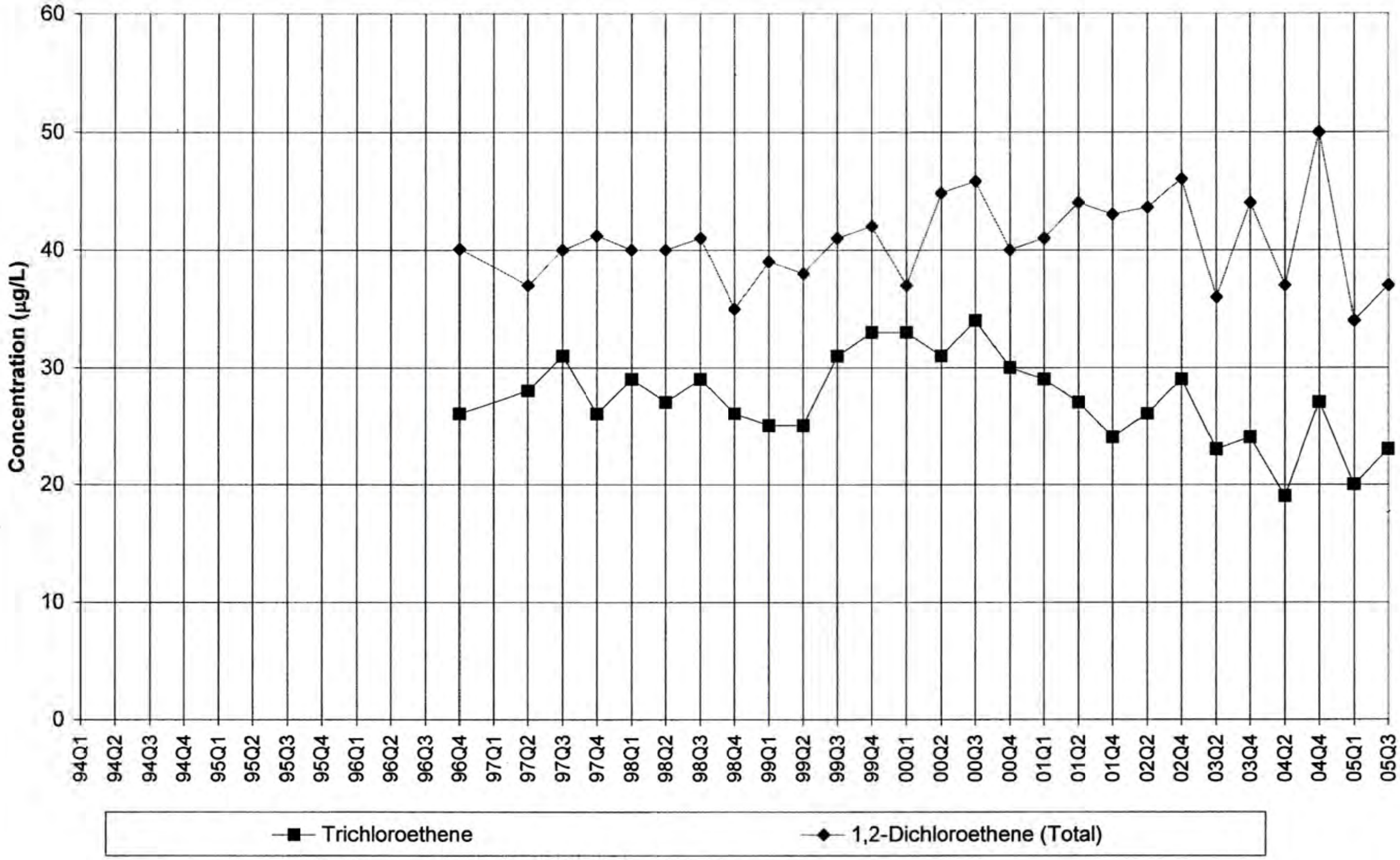
Not detected constituents are plotted as 0 µg/L

Figure C-32

Trichloroethene Concentrations for Well H(3)

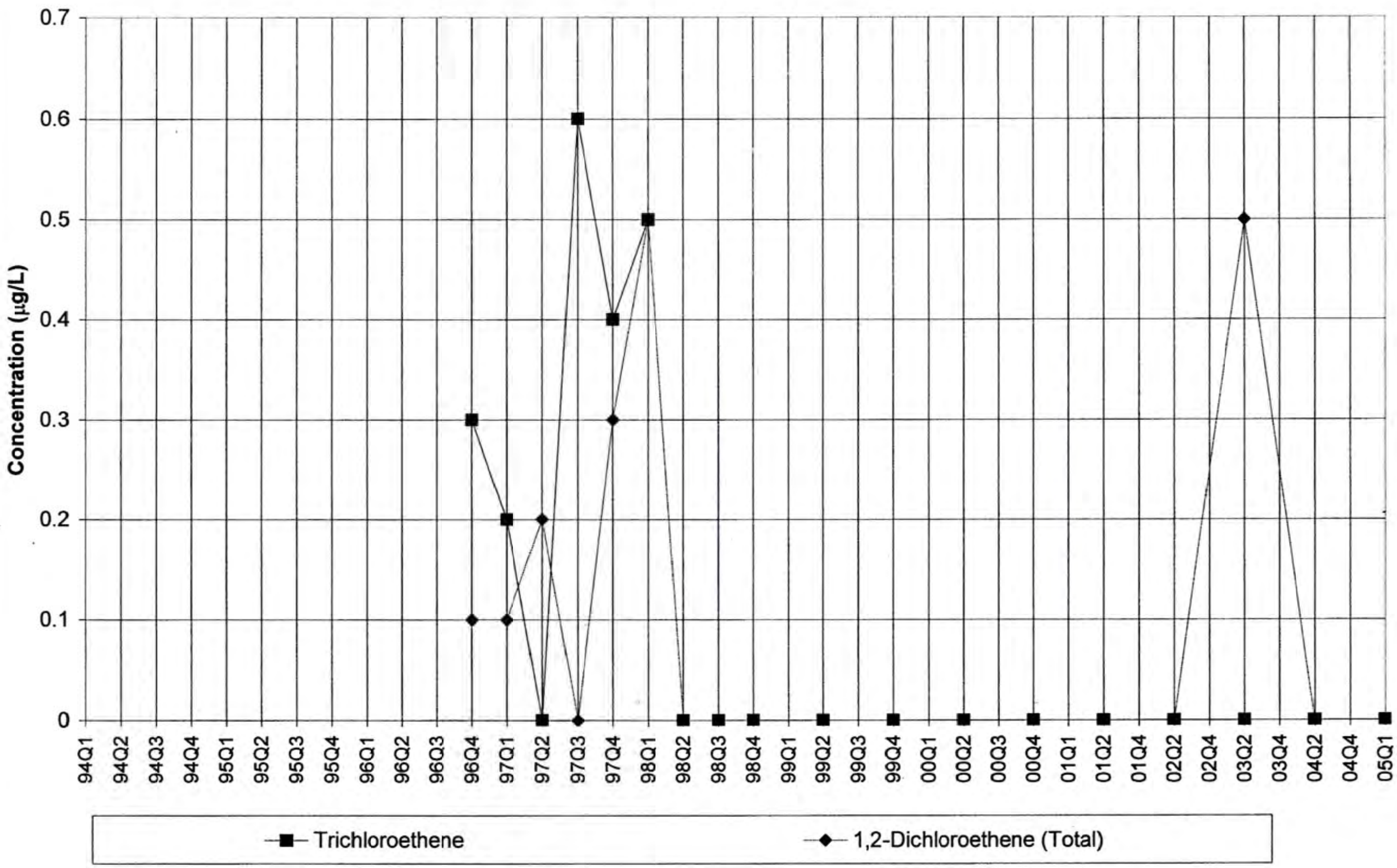


Not detected constituents are plotted as 0 µg/L	**Figure C-33**	**Total 1,2-Dichloroethene and Vinyl Chloride Concentrations for Well I(3)**



Not detected constituents are plotted as 0 µg/L

Figure C-34	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well I(3a)**



Not detected constituents are plotted as 0 µg/L	**Figure C-35**	**Trichloroethene and Total 1,2-Dichloroethene Concentrations for Well N(3a)**

Technical Memorandum

TO: Queen City Farms

FROM: Eric Weber, LHG and Bruce Stirling, PWS

DATE: April 4, 2018

RE: **Tributary 316A Historical Discharge Review**
Queen City Farms
Maple Valley, Washington
Project No. 0992002.050.051

Introduction

This technical memorandum presents information related to a small intermittent stream (now referred to as Tributary 316A) and associated engineered drainage channel on the Queen City Farms (QCF) property located in King County, near Maple Valley, Washington. Landau Associates, Inc. (LAI) has completed this technical memorandum as part of planning and permitting activities for a refill project (the Phase III Refill Project) at the former gravel mine on the property. The site location is shown on Figure 1.

Gravel mining was conducted from the 1970s through the 1990s on a roughly 393-acre area by Stoneway Concrete (Stoneway). Mining was conducted by Stoneway on property currently owned by both Cedar Shores, LLC and QCF. Refilling and reclamation of the gravel mine area is being conducted in three phases. Phase I is being refilled under grading permit GRDE15-0053 (formerly L03CG141) issued by King County (County) Department of Permitting and Environmental Review; Phase II will be refilled under grading permit GRDE15-0214 (formerly L04CG384) issued by the County; the Phase III permit application is in preparation. Refilling is being conducted in part to fulfill the requirements of the Washington State Department of Natural Resources (WDNR) reclamation permit number 70-010880. Refill areas and property ownership is shown on Figure 2.[1]

The engineered drainage channel was a stormwater feature that was developed to convey flow associated with the intermittent stream channel. The objective of this technical memorandum is to document the original location of discharge of the intermittent stream channel and how it has been modified over time. The scope of services for completion of this report included field evaluations; review of documents, drawings, and aerial photographs associated with the channel area; and discussions with regulatory agency personnel.

Purpose and Need

Refilling and reclamation of the Phase III Refill Area will require modifying the engineered drainage channel that is currently present in the refill area footprint. The feature in question is a man-made channel that was constructed during gravel mine reclamation activities in an area where no surface

[1] The Phase III refill overlaps the Phase I and the Phase II refill areas. The overlap is not shown on Figures 2 and 3.

drainage historically existed. The approximate location of the engineered drainage channel and other associated features in relationship to the Phase III Refill Area are shown on Figure 3.

History

In the early 1970s, a roughly 393-acre area on the QCF property and the adjacent Cedar Shores property was zoned Q-M (Quarry Mining). Sand and gravel was mined from this area through the 1990s. The area to the west was known as the Cedar Shores Pit. The area to the east was known as the QCF Pit; however, Stoneway mined the two pits continuously as part of a single project.

Prior to active mining in the 1970s, a pre-existing, unclassified stream (now referred to as Tributary 316A: Upper Reach) with intermittent flow entered the QCF property from the northwest. The stream terminated in the northwest corner of the property where it infiltrated into permeable sand and gravel soil and/or discharged to Queen City Lake. An aerial photograph from 1937 shows the stream terminating west of Queen City Lake. The stream is shown on US Geologic Survey (USGS) maps from 1968 and 1973 emanating from offsite and terminating at the toe of the slope that formed the bench or terrace north of Cedar Grove The stream is subsequently shown as an unclassified stream segment on a more recent map from King County iMAP under the layer that shows 1990 Sensitive Area Ordinance features. The 1937 aerial photograph, USGS maps, and the King County iMAP page are included in Attachment 1. Figure 2 shows the current location of the intermittent stream segment which is now diverted around the Cedar Grove Compost facility.

The unclassified stream segment is shown on a 1978 engineering drawing by B. Larson and Associates. In this drawing, the stream discharges at or near the west end of Queen City Lake in the area directly north of the former local airfield that was present on the property at that time. The drawing also shows the status of gravel mining operations in about 1978. This drawing is included as Attachment 2. Barghausen Consulting Engineers (Barghausen) also concluded based on interpretation of 1987 and 1985 aerial photographs that the tributary flowed toward and into Queen City Lake (Barghausen 2002). Barghausen (2002) references a 1985 map that also shows Tributary 316A flowing toward and into Queen City Lake.

Beginning in about 1989, the stream segment discharge was routed offsite to the west of QCF and the current location of the compost facility access road. In 1990, the County and Washington Department of Fish & Wildlife personnel visited the site to inspect the newly rerouted stream segment (Barghausen 2002). They concluded that the intermittent stream segment terminated in a dense stand of trees and thicket before percolating into the soil near a sedimentation pond located west of the current QCF property. Shortly thereafter, the Cedar Grove Compost Facility was constructed and the unclassified stream segment was routed around as shown on Figure 3.

In developing site refill and reclamation plans for the Cedar Shores Pit in 1989, Barghausen defined an engineered drainage channel that routed the unclassified stream segment down the pit face and across the pit floor where it discharged to an infiltration area within the Cedar Grove Channel. They renamed the pre-existing, intermittent stream segment and the planned engineered drainage channel as Tributary 316A. Prior to 1989, historical research did not show any county, state, or federal agency documentation that classified this stream segment and channel or that used the terminology Tributary 316A.

In 1991, Cedar Shores submitted a preliminary plan to the County to construct the QCF-engineered drainage channel. The final version of the plan, prepared by Parametrix, Inc. (Parametrix), was dated 1993; a copy of this plan is included in the Expanded Environmental Checklist for the Cedar Shores Gravel Mine Refill Project (Barghausen 2006). The current (2016) configuration of the engineered drainage channel (as shown on Figure 3) is consistent with the 1993 Parametrix design. Note that the channel includes three culvert segments and a bioinfiltration basin. The County-approved 1993 drainage channel alignment plans are included as Attachment 3. The channel is also represented in the 1991 gravel mine reclamation plan submitted to WDNR. This reclamation plan is included as Attachment 4.

In 2002, the County requested that the drainage identified by Barghausen as Tributary 316A be delineated, classified, and surveyed as part of the grading permit application for refilling the Cedar Shores Pit (King County 2002). The evaluation of Tributary 316A is presented in a sensitive area report prepared by Barghausen (2002). The evaluation was completed in accordance with the general site survey requirements of the County Stream Survey Report Criteria in effect at the time. The report is based on field visits, review of aerial photographs, and review of the Cedar Shores project files. The report concluded that the intermittent stream segment portion of Tributary 316A was a natural drainage channel that historically discharged near Queen City Lake (either infiltrating into gravel deposits or discharging into the Lake as described above). The description of Tributary 316A excerpted from the Barghausen Sensitive Area Report is included as Attachment 5.

The purpose of the engineered drainage channel was to convey runoff from the intermittent stream segment across the floor of the gravel pit to an infiltration area. The final conclusion of the sensitive area report (Barghausen 2002) was that the engineered drainage channel portion of Tributary 316A is "*an ephemeral, blind ditch that was designed and constructed to allow water flows to cross the (gravel mine)…. (t)his system should be considered as an engineered ditch that collects and routes surface water across the site to an infiltration system*".

Hydrology

The QCF property lies within the general physiographic province known as the Coalfield Drift Plan, which is an upland area that extends from the Cedar River Valley to the Cascade Range. QCF and Cedar Hills Regional Landfill to the north represents an area of the drift plain that is further segmented on all sides by valleys or troughs. The Cedar River bounds the area to the southwest, while Issaquah Creek bounds the area to the northeast. Mason Creek separates the drift plain from the

Newcastle Hills promontory to the north, while the Cedar Grove Channel bisects the drift plain directly south of QCF. Numerous small lakes on the drift plain are generally interpreted to be kettle lakes formed by melting ice block, which remained after the last glacial retreat. Queen City Lake is interpreted to be a kettle lake (LAI 1990).

A thick sequence of Vashon age[2] glacial till mantles the upland drift plain. In the vicinity of Cedar Grove Channel, the low permeability glacial till is absent and is replaced with sand and gravel that represents glacial outwash deposits. Underlying the Cedar Grove Channel are older pre-Vashon silt and sand deposits. Gravel mining operations removed much of the sand and gravel on the north side of the Cedar Grove Channel exposing till or older pre-Vashon deposits.

Historically, the unclassified intermittent stream segment flowed south across the QCF property on top of the glacial till until it hit the permeable sand and gravel deposits adjacent to Queen City Lake where it infiltrated or flowed into the lake. When mining removed the thick sequence of permeable sand and gravel deposits, the stream discharge was able to flow further south into the gravel pit on top of the low permeability till and pre-Vashon deposits. This flow was routed as an engineered drainage channel that directed the flow to an infiltration area (known as the Main Infiltration Area) located in unmined gravel deposits along the base of the channel (Figure 3).

Groundwater Recharge and Discharge

Recharge and discharge associated with Queen City Lake and the underlying aquifer sequence are well understood based on investigations performed at the nearby QCF Superfund site[3]. A schematic of the aquifer sequence is shown on Figure 4. A conceptual model of recharge and discharge is shown on Figure 5.

Historically, all surface water runoff upslope of the QCF site flowed into Queen City Lake (Route W on Figure 5) or the permeable soil adjacent to the lake (Route X on Figure 5). Queen City Lake is interpreted to be a kettle lake, formed in a depression left by a melting remnant ice block (LAI 1990). Relatively low permeability ice-contact sediments underlie the lake. Recent silt deposits that cover the lake bed have been observed to be up to 3 feet (ft) deep. Since Queen City Lake historically had no surface outlet the entire flow into the lake infiltrated through soil around the perimeter of the lake and as direct infiltration through bottom sediments (Route Y on Figure 5). The lake infiltration percolated directly into Aquifer 1, a small perched aquifer. Lake recharge to Aquifer 1 increases as the lake level rises and the surface area of the lake expands beyond the area of the lake bottom covered by relatively low permeability recent silt deposits. Thus at higher water levels, lake water comes into direct contact with more permeable ice-contact and outwash deposits which also underlie the lake bottom, providing for more rapid discharge to Aquifer 1 (LAI 1990).

[2] The Vashon stade of the Fraser glaciation is the most recent glacial episode in the Puget Sound area.
[3] The Queen City Farms Superfund site consists of the entire QCF property.

Historically, discharge from Aquifer 1 occurred either vertically through the Clayey-Silt Layer aquitard (Route Z on Figure 5) and eventually to Aquifer 2, or laterally through lower permeability soil along the western boundary of the aquifer (Route S on Figure 5) and eventually to the QCF Spring. The majority of this discharge eventually recharged the wetland and stream complex in Cedar Grove Channel via flow in Aquifer 2 or through QCF Spring discharge. The channel wetland and stream complex discharges to the Cedar River. Within this historical hydrologic cycle, groundwater storage played a significant role. For example, groundwater flow from Aquifer 1 through the Lower Unsaturated Zone (see Figure 4) to Aquifer 2 likely takes a month or more (LAI 1990). Groundwater flow rates within Aquifer 2 are estimated at about 250 ft/year (LAI 1990). Consequently, groundwater takes 4 or 5 years to flow the roughly 1,200 ft[4] within Aquifer 2 to the Cedar Grove Channel. The effect of the long groundwater residence time for infiltrating surface water is that recharge to the Cedar Grove Channel wetland and stream complex was relatively steady (i.e., constant over time) as Queen City Lake, Aquifer 1, the Lower Unsaturated Zone and Aquifer 2 served an important stormwater detention function.

Due to gravel mining, the hydrologic cycle on the QCF property changed. The East Airstrip Spring formed in 1988 due to mining in the area just south of the lake. Flow from the spring was estimated at as much as 2.2 cubic feet per second (cfs) directly into Main Gravel Pit Lake (MGPL; LAI 1990). The erosion control measure (ECM) was constructed in 1991 to stabilize the East Airstrip Spring Area (LAI 1992). The ECM involved installing a 36-inch diameter culvert to also divert Queen City Lake overflow directly to MGPL. Surface water features in the vicinity of the East Airstrip Spring are shown on Figure 6.

In the early 1990s, about the same time that the ECM was constructed, the Tributary 316A engineered drainage channel was also constructed. The channel facilitated the rapid routing of stormwater flow to the base of the Cedar Grove Channel to infiltrate near (about 200 ft) the QCF Spring where it discharges as surface water into a ditch along Cedar Grove Road. Historically this is recharge that would have flowed into Queen City Lake or infiltrated directly into Aquifer 1. The engineered drainage channel, the ECM and the formation of the East Airstrip Spring all contributed to disruption of the hydrologic cycle in the channel and the elimination of a substantial amount of groundwater storage. Eliminating groundwater storage resulted in the elimination of the associated stormwater detention function provided by the groundwater system.

Discussion

One of the primary objectives of the Phase II Refill (permit GRDE15-0214) is to restore the hydrology to a pre-mining configuration and therefore, recover much of the original function of the hydrologic cycle. The Phase II Refill results in elimination of MGPL, re-infiltration of the East Airstrip Springs, and increase of the stage level in Queen City Lake resulting in restoration of surface water storage and

[4] 1,200 ft is the horizontal distance in Aquifer 2 from the point of recharge beneath Aquifer 1 to the Cedar Grove Channel.

infiltration. The Phase III Refill has a similar objective by eliminating the engineered drainage channel and restoring the original Tributary 316A discharge into Queen City Lake. This restoration will result in infiltration of the stream discharge through the groundwater system and result in more steady discharge to the wetland and stream complex that flows into the Cedar River.

LANDAU ASSOCIATES, INC.

Bruce A. Stirling

Bruce Stirling, PWS
Senior Associate

Eric Weber

Eric Weber, LHG
Principal

JLS/BAS/EFW/jrc
[P:\992\002\WIP\R\PH3 TIR\SEC 6 TMS\6.6 HISTORICAL DISCHARGE REVIEW\TRIBUTARY 316A HISTORICAL DISCHARGE TM.DOCX]

Attachments: Figure 1: Vicinity Map
Figure 2: Refill and Ownership Map
Figure 3: 2016 Baseline Conditions
Figure 4: Schematic Representation of Queen City Farms Hydrology
Figure 5: Conceptual Model of Recharge and Discharge for Aquifer 1
Figure 6: Surface Water Features: 1992
Attachment 1: Miscellaneous Site Documentation
Attachment 2: 1978 Engineering Drawing
Attachment 3: 1993 Drainage Channel Realignment Plans
Attachment 4: 1991 Reclamation Plan
Attachment 5: 2002 Sensitive Area Report

References

Barghausen. 2002. Sensitive Areas Report, Queen City Farms/Cedar Shores Refilling Project, King County Tracking Number L02GI014, King County, Washington. Kent, WA: Barghausen Consulting Engineers.

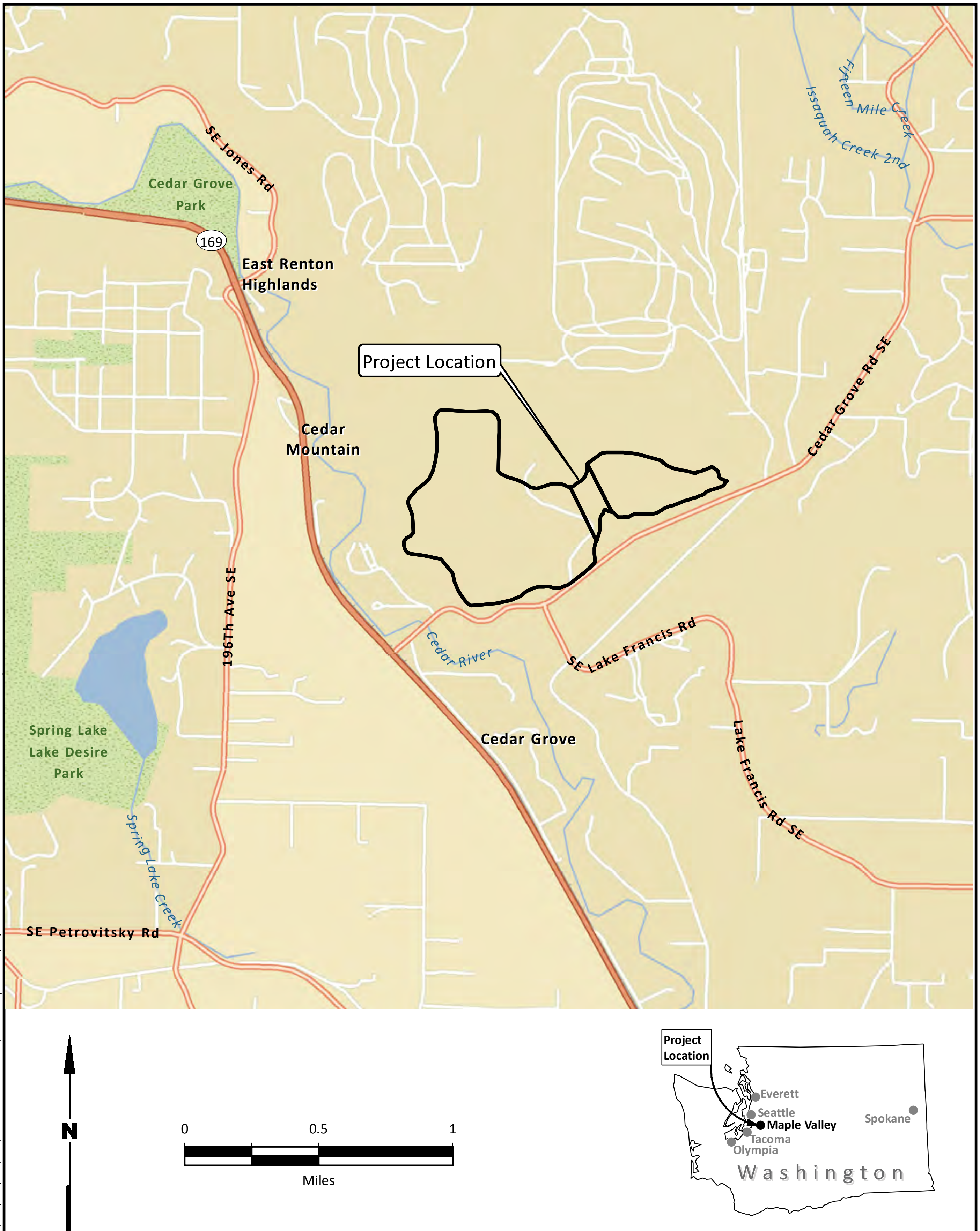
Barghausen. 2006. Expanded Environmental Checklist, Cedar Shores Gravel Mine Refill Project, King County File No. L03CG141, King County, Washington. Barghausen Consulting Engineers.

King County. 2002. L02GI014 Cedar Grove Fill Site.

LAI. 1990. Remedial Investigation Report, Queen City Farms, King County, Washington, Volume 1 of 2. Edmonds, WA: Landau Associates, Inc.

LAI. 1992. Supplemental Remedial Investigation Report, Queen City Farms, King County, Washington Volume I of II. Landau Associates, Inc.

LAI. 1992. Supplemental Remedial Investigation Report, Queen City Farms, King Count, Washington, Volume 1 of 2. July 31.

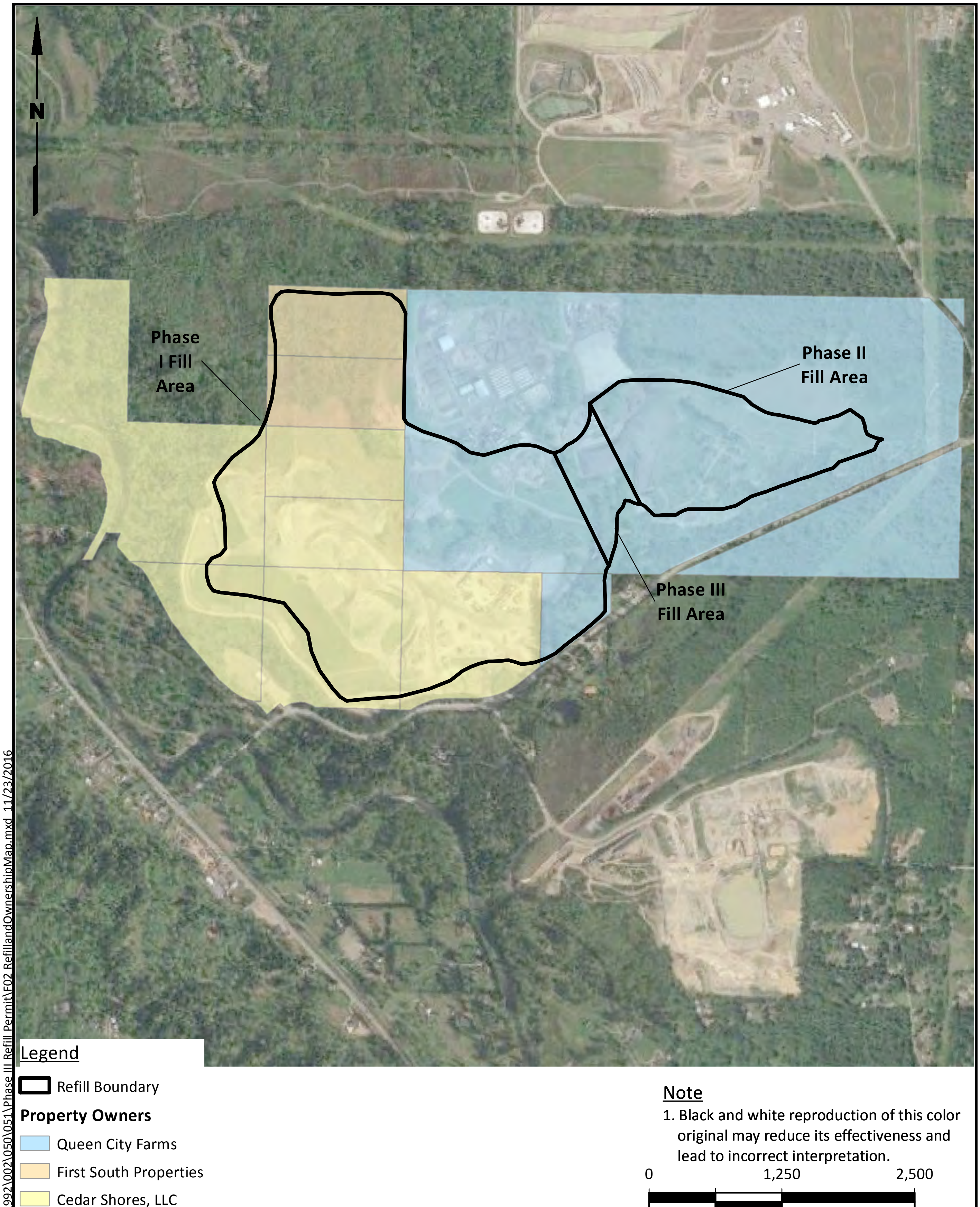


Data Source: Esri 2012

G:\Projects\992\002\050\051\Phase III Refill Permit\F01VicMap.mxd 11/23/2016

LANDAU ASSOCIATES

Queen City Farms Phase III Refill Maple Valley, Washington	**Vicinity Map**	Figure 1



Data Source: King County GIS; Google Earth Imagery, 6/27/2016.

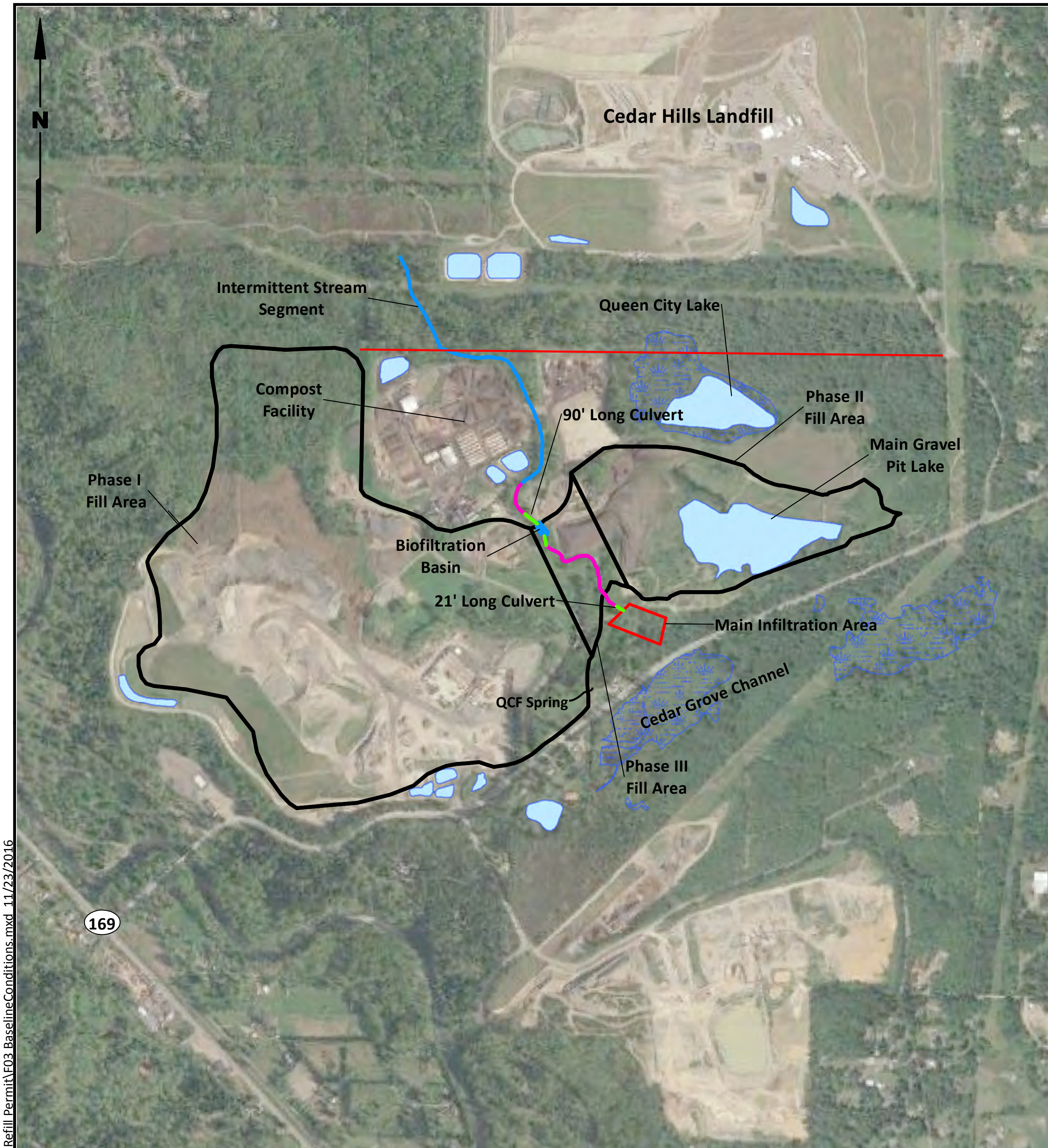
G:\Projects\992\002\050\051\Phase III Refill Permit\F02 RefillandOwnershipMap.mxd 11/23/2016

Landau Associates

Queen City Farms Phase III Refill
Maple Valley, Washington

Refill and Ownership Map

Figure 2



Legend

- Culvert
- Engineered Drainage Channel
- Intermittent Stream Segment
- Wetland
- Lake, Pond or Infiltration
- Refill Boundary

Note

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.

0 1,200 2,400

Scale in Feet

Data Source: Google Earth Imagery, 6/27/2016.

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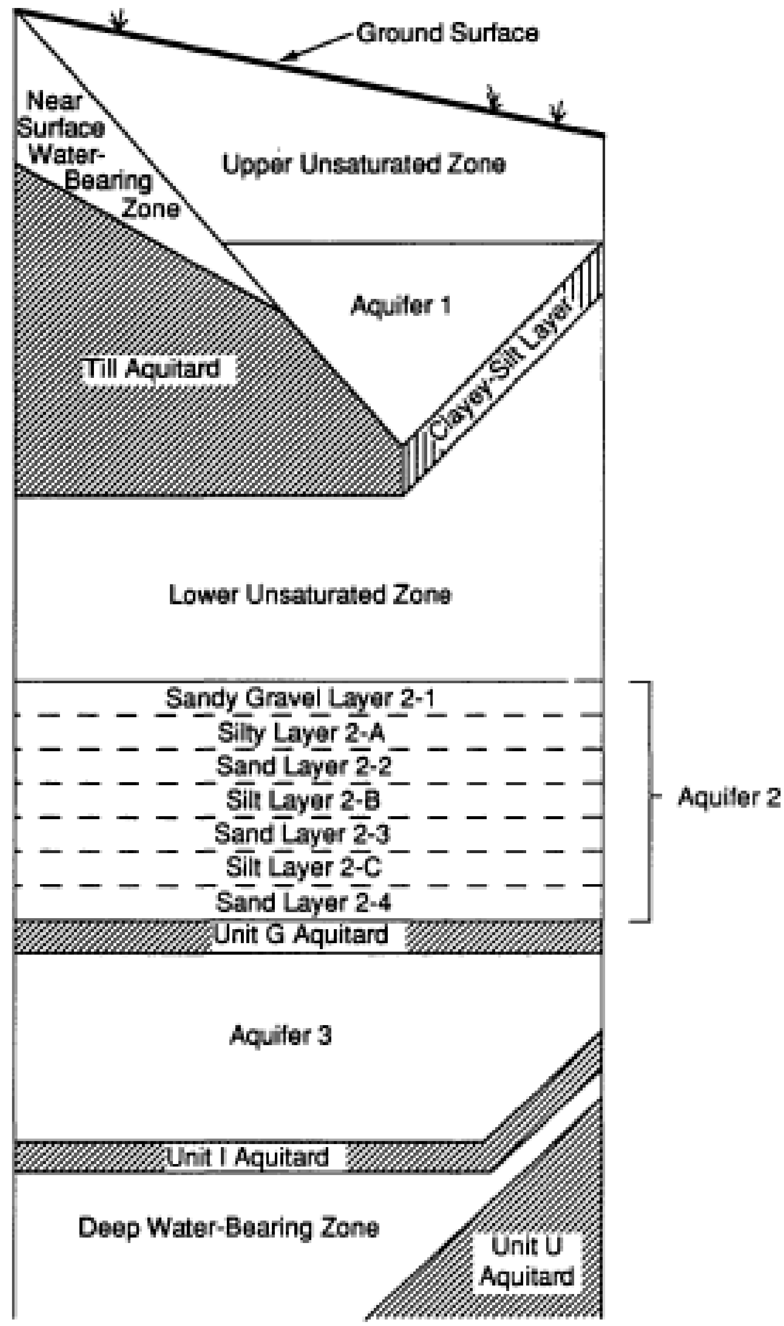
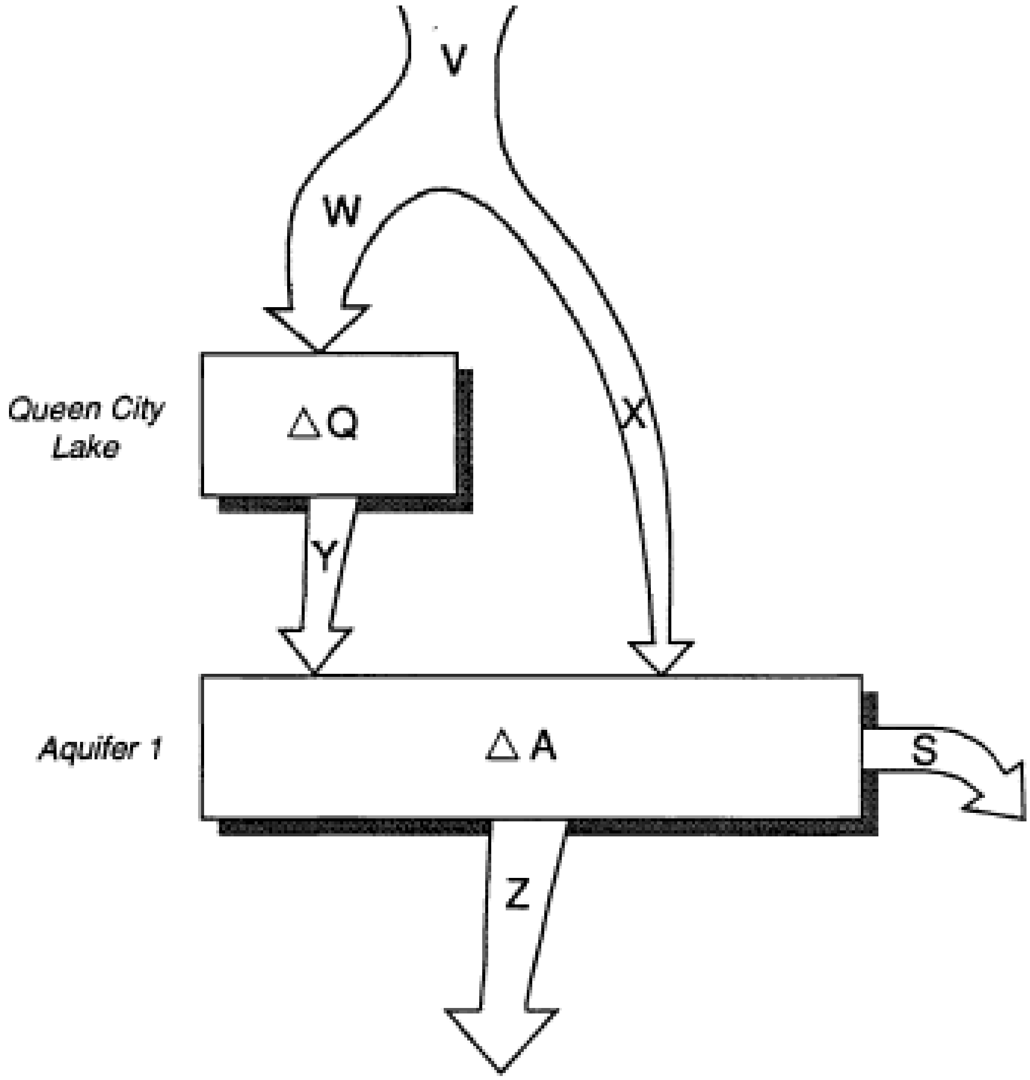
LANDAU ASSOCIATES

Queen City Farms Phase III Refill Maple Valley, Washington	**2016 Baseline Conditions**	Figure 3



Source: LAI 1990.

Schematic Representation of Queen City Farms Hydrology



V = Total Surface Water Runoff

W = Fraction of Surface Water Runoff Going Directly into Queen City Lake

X = Fraction of Surface Water Runoff Recharging Directly to Aquifer 1

Y = Recharge to Aquifer 1 through Queen City Lake Bottom

S = Discharge from Aquifer 1 through Springs or Subsurface Leakage through Soils along the Western Boundary of the Aquifer

Z = Discharge from Aquifer 1 through the Clayey-Silt Layer

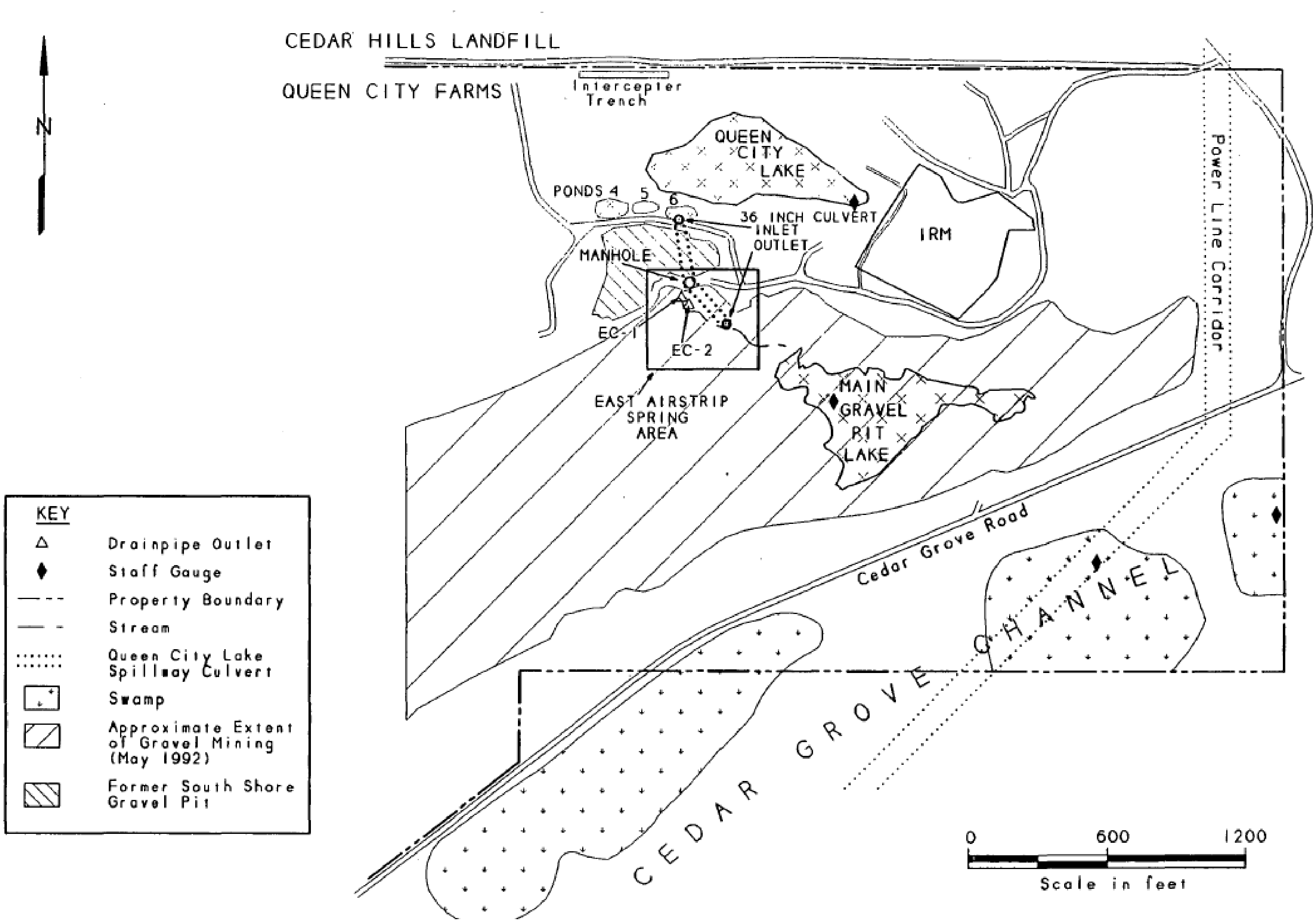
ΔQ = Change in Water Volume in Queen City Lake

ΔA = Change in Water Volume in Aquifer 1

Source: LAI 1990.

Conceptual Model of Recharge and Discharge for Aquifer 1

Figure 5



Source: LAI 1992.

LANDAU ASSOCIATES

Queen City Farms Phase III Refill Maple Valley, Washington	**Surface Water Features: 1992**	Figure 6

Attachment 1

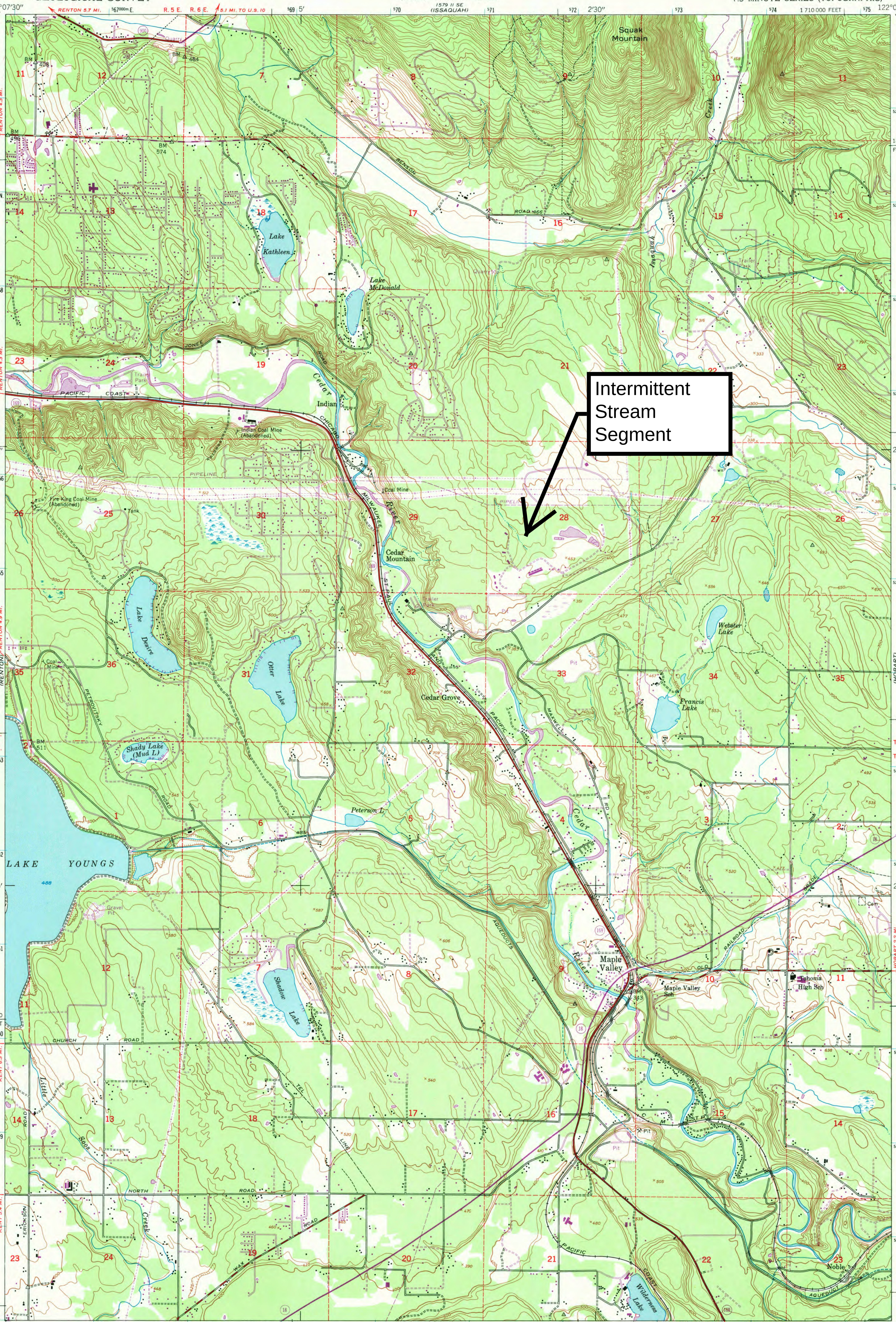
Miscellaneous Site Documentation


S.W.¼ TWP-23N-R6E.
INCLUDING SECTIONS 19,20,21,30,29,28,31,32 &33
1937 Aerial Photograph
Intermittent Stream Segment
Queen City Lake

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

UNITED STATES
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS

MAPLE VALLEY QUADRANGLE
WASHINGTON—KING CO.
7.5 MINUTE SERIES (TOPOGRAPHIC)



Mapped by the Army Map Service
Published for civil use by the Geological Survey

Control by USC&GS, USCE, and King County Engineer Office

Topography from aerial photographs by multiplex methods
Aerial photographs taken 1943. Field check 1949

Polyconic projection. 1927 North American datum
10,000-foot grid based on Washington coordinate system, north zone

1000-meter Universal Transverse Mercator grid ticks, zone 10, shown in blue

No distinction is made between dwellings, barns, commercial and industrial buildings

Revisions shown in purple compiled by the Geological Survey from aerial photographs taken 1968. This information not field checked

UTM GRID AND 1968 MAGNETIC NORTH
DECLINATION AT CENTER OF SHEET

SCALE 1:24000

CONTOUR INTERVAL 25 FEET
DATUM IS MEAN SEA LEVEL

THIS MAP COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS
FOR SALE BY U. S. GEOLOGICAL SURVEY, DENVER, COLORADO 80225, OR WASHINGTON, D. C. 20242
A FOLDER DESCRIBING TOPOGRAPHIC MAPS AND SYMBOLS IS AVAILABLE ON REQUEST

ROAD CLASSIFICATION
Heavy-duty — Light-duty
Medium-duty — Unimproved dirt
State Route

QUADRANGLE LOCATION

MAPLE VALLEY, WASH.
N4722.5—W12200/7.5

1949
PHOTOREVISED 1968
AMS 1578 I NE—SERIES V891

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

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MAPLE VALLEY QUADRANGLE
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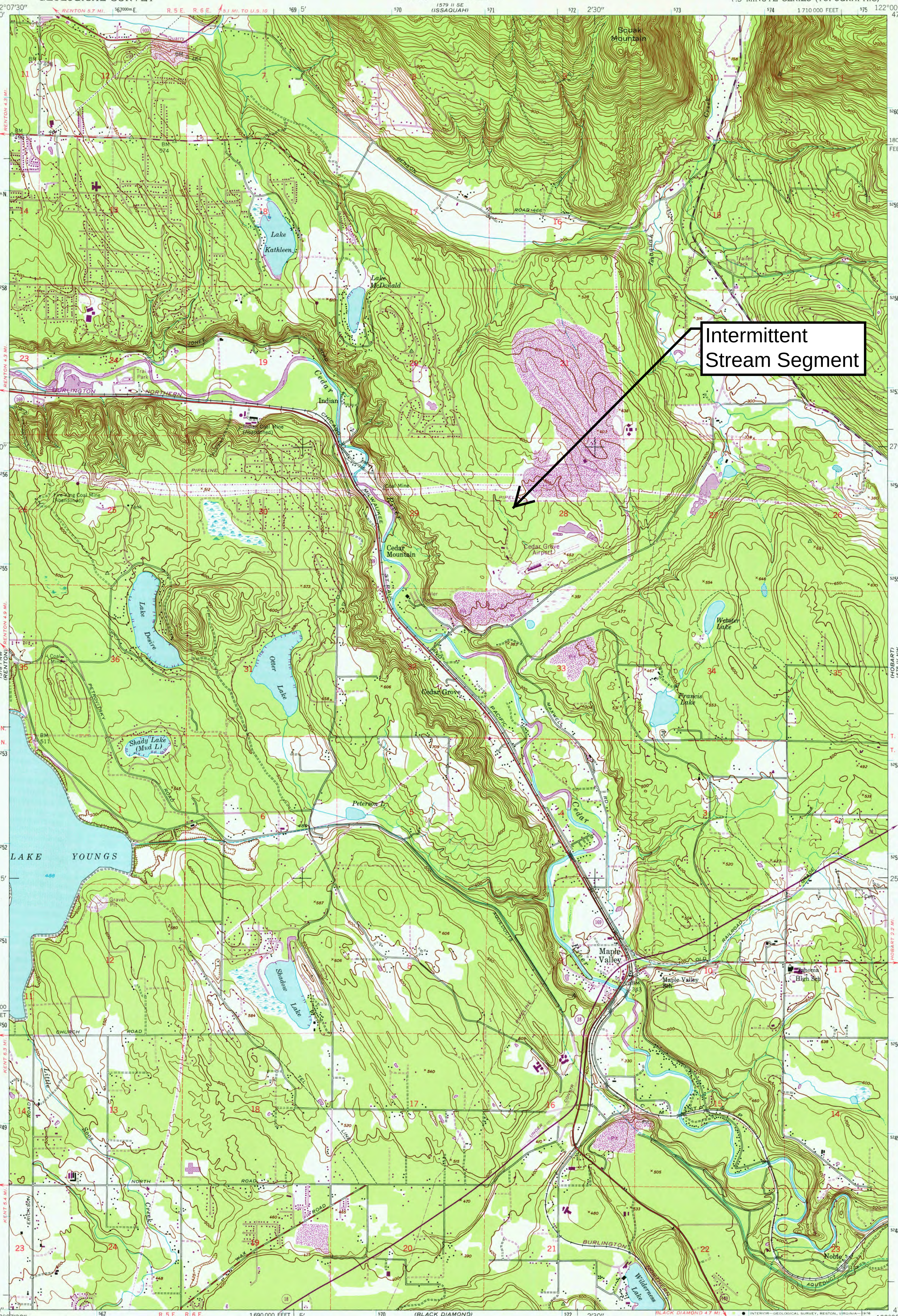

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Revisions shown in purple compiled by the Geological Survey from aerial photographs taken 1968 and 1973. This information not field checked

UTM GRID AND 1973 MAGNETIC NORTH DECLINATION AT CENTER OF SHEET

SCALE 1:24000

CONTOUR INTERVAL 25 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929

THIS MAP COMPLIES WITH NATIONAL MAP ACCURACY STANDARDS
FOR SALE BY U. S. GEOLOGICAL SURVEY, DENVER, COLORADO 80225, OR RESTON, VIRGINIA 22092
A FOLDER DESCRIBING TOPOGRAPHIC MAPS AND SYMBOLS IS AVAILABLE ON REQUEST

ROAD CLASSIFICATION
Heavy-duty
Light-duty
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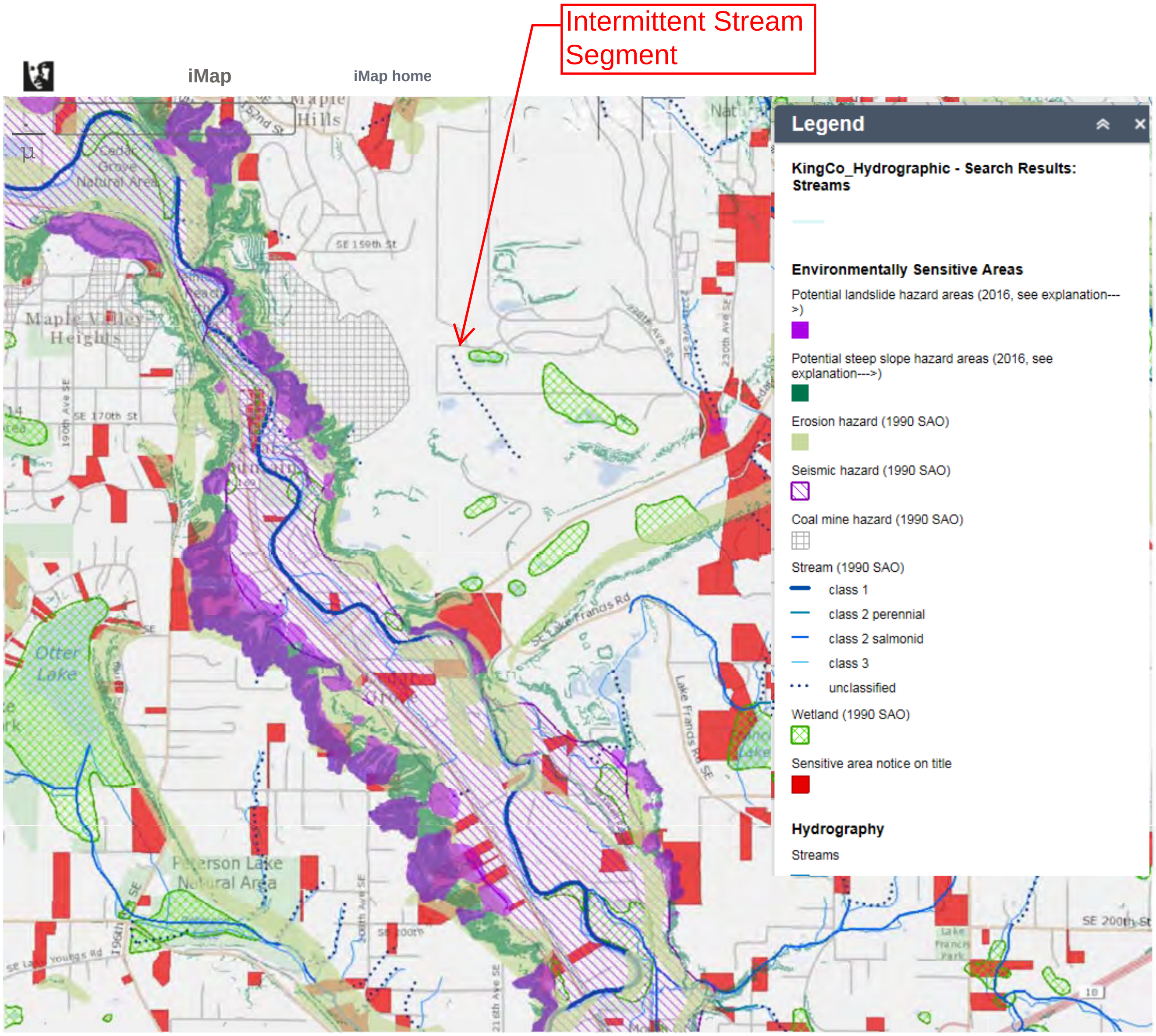
QUADRANGLE LOCATION

USGS
Historical File
Topographic Division

MAPLE VALLEY, WASH
N4722.5—W12200/7.5
1949
PHOTOREVISED 1968 AND 1973
AMS 1578 I NE—SERIES V891

MAR 1 1 1976



0.6km
0.4mi

-13,590,408.80934 6,018,710.64483 Meters

Attachment 2

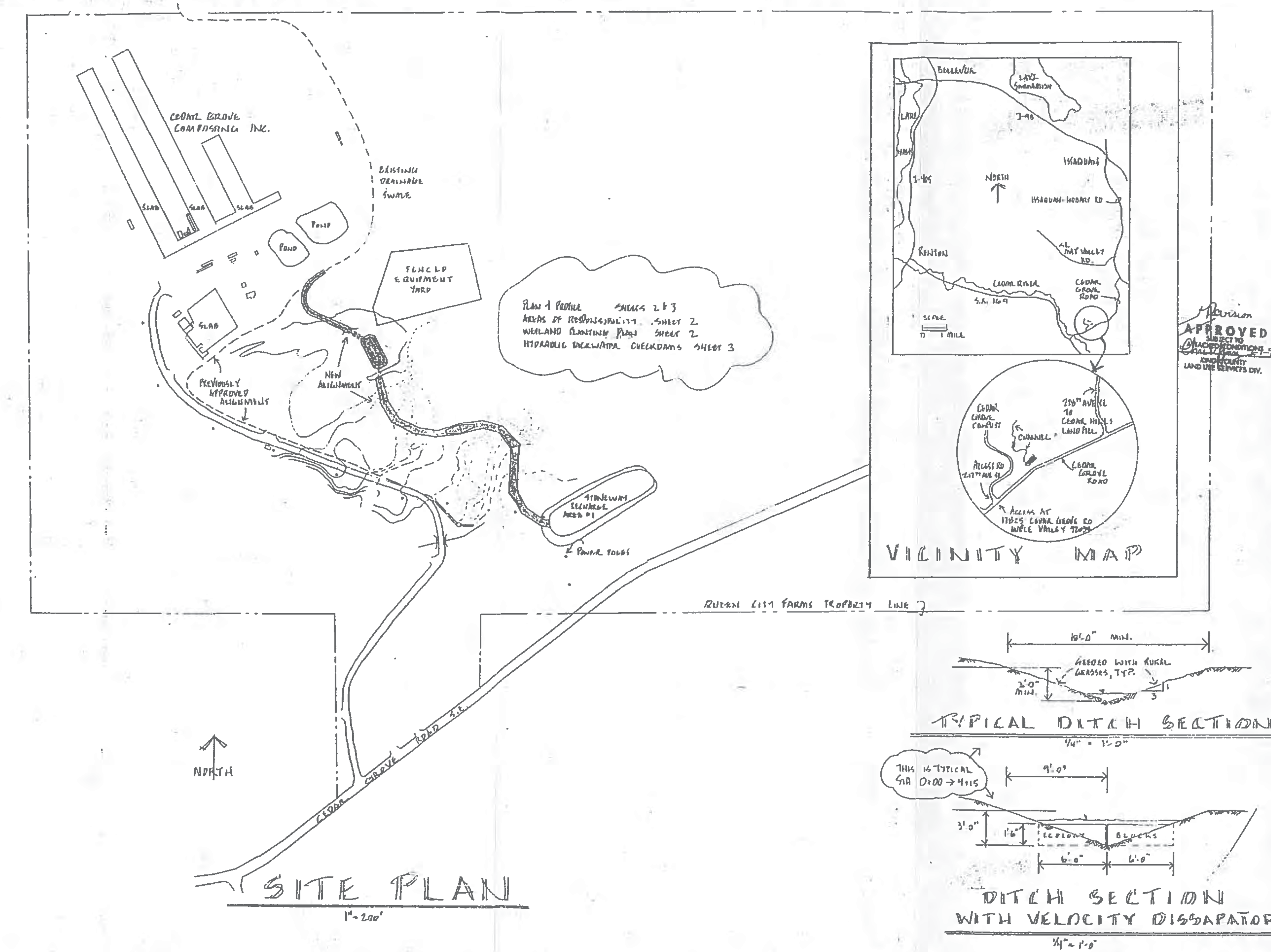
1978 Engineering Drawing


Intermittent Stream
Queen City Lake
EXCAVATION LIMITS TO 3' ABOVE W.T.
APPROX. GRAVEL LIMITS
B-N LEASE
EXCAVATED AREA
SEE 1968 REPORT FOR DATA THIS AREA
PLANT AND STOCKPILE SITE
SPRINGS
Cedar Grove Rd
CEDAR RIVER
SYMBOLS
F-1 - E-17 1978 DRILLING PROGRAM.
DU-1 - DU-6, BA-3, BA-4 1967 DRILLING PROGRAM.
CS & N.P. UNKNOWN
W.T. WATER TABLE
APPROXIMATE LATERAL LIMITS OF DEPOSIT. MAY EXCEED ECONOMIC EXCAVATION LIMITS.
MAP BASED ON 1967 AERIAL PHOTOGRAPHY BY KING COUNTY & U.S.E.D.
STONEWAY CONCRETE
RENTON, WA.
CEDAR GROVE GRAVEL PIT
MAPLE VALLEY
BY B. I. LARSEN & ASSOCIATES SEATTLE, WA.
DATE: 11-30-78
SCALE 1" : 200'

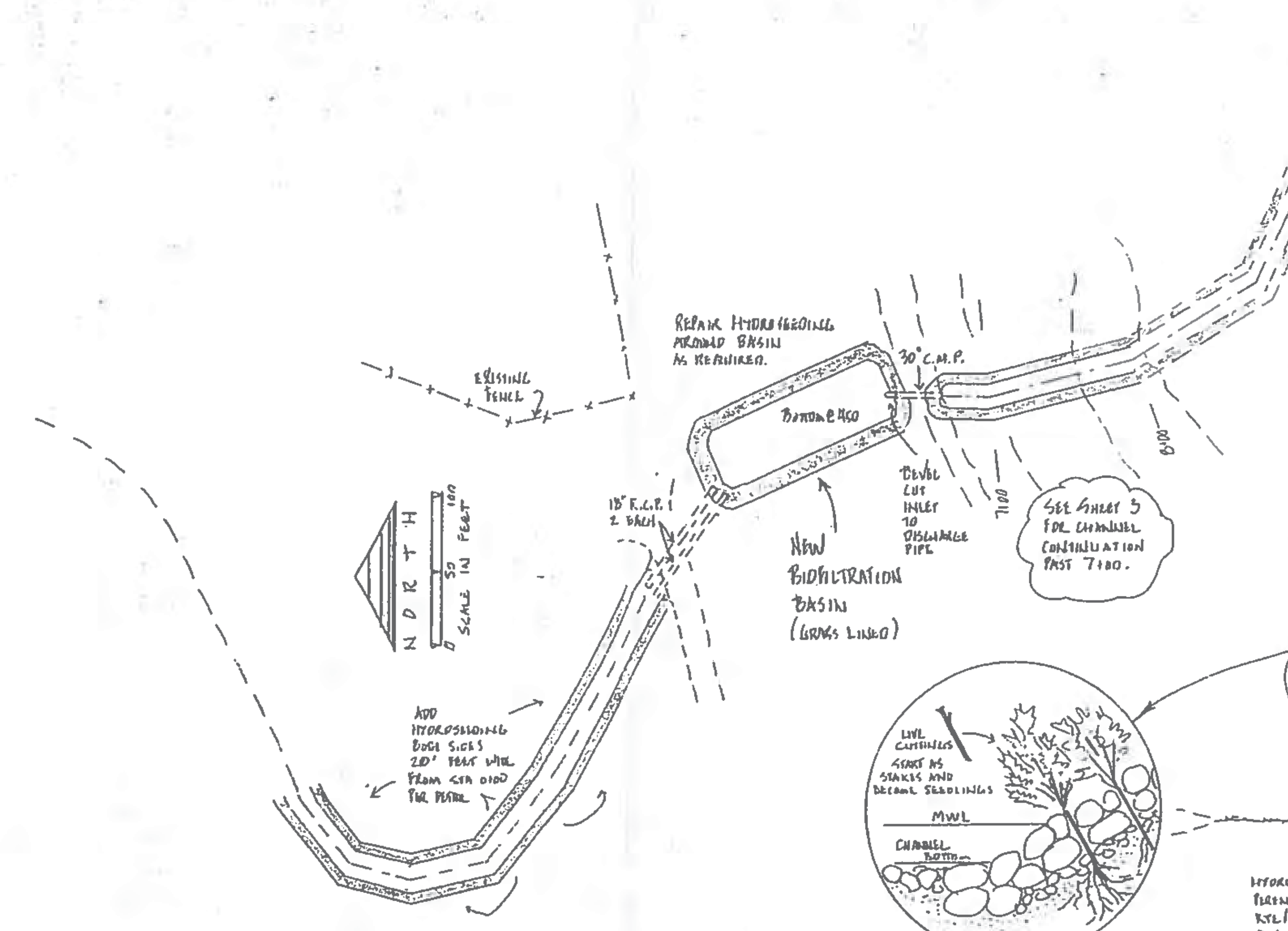
Attachment 3

1993 Drainage Channel Realignment Plans

APPENDIX F
1993 DRAINAGE CHANNEL REALIGNMENT PLANS



SCALE: NOTED	DATE: 7-15-92.	CHANGE ORDER NO. 1	SITE PLAN	QUEEN CITY FARMS	1/

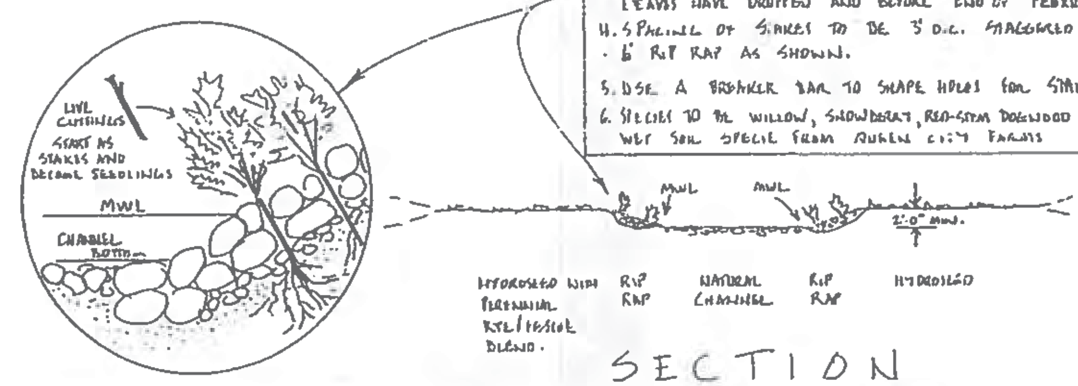


AREAS OF RESPONSIBILITY

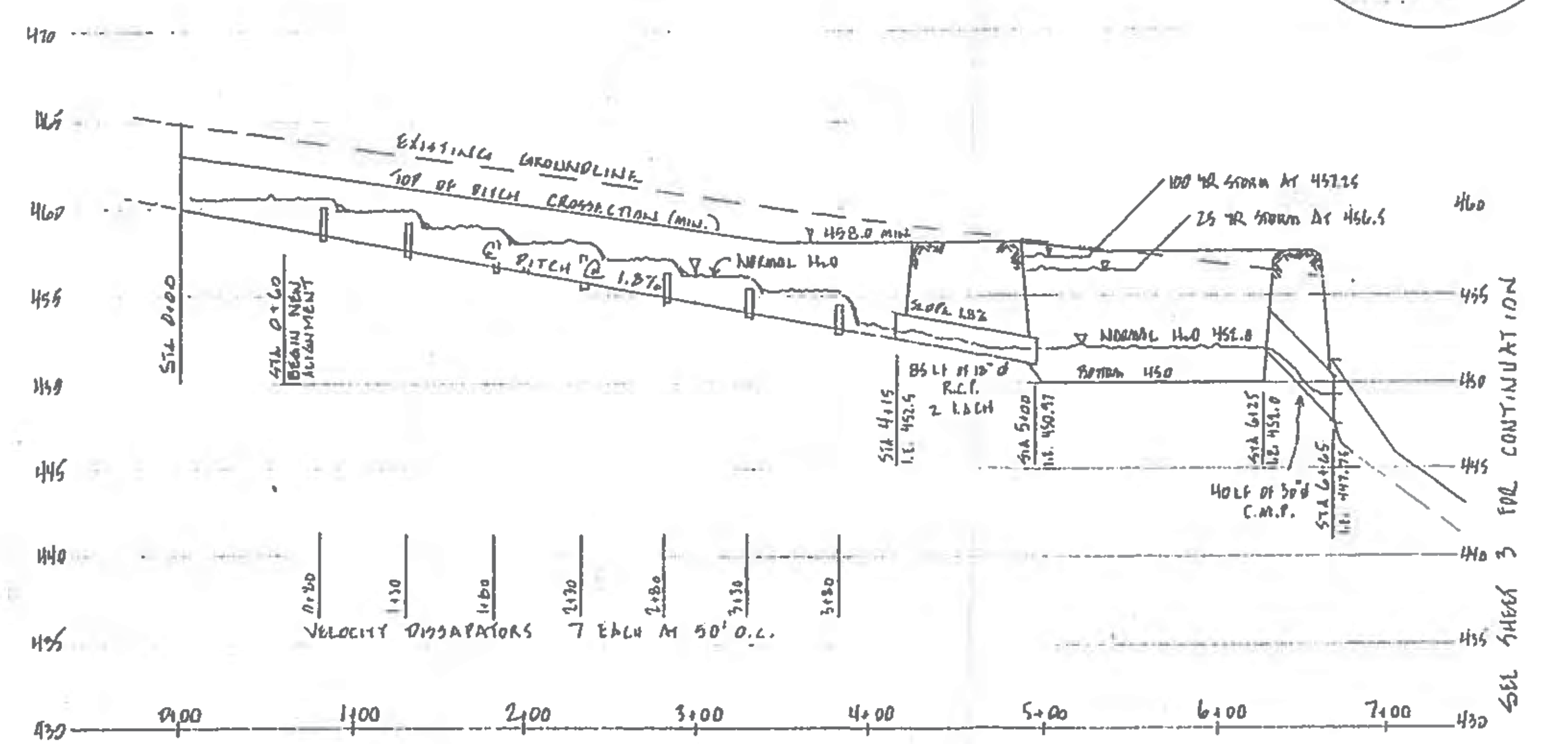
DITCH CONSTRUCTION:	
STA 0+00 → 6+65	QUEEN CITY FARMS
6+65 → 9+50	STONEWAY
BASIN @ 5+00	QUEEN CITY FARMS
RIP RAP PLACEMENT	STONEWAY
WETLAND PLANTING	QUEEN CITY FARMS
HYDROSEEDING	STONEWAY

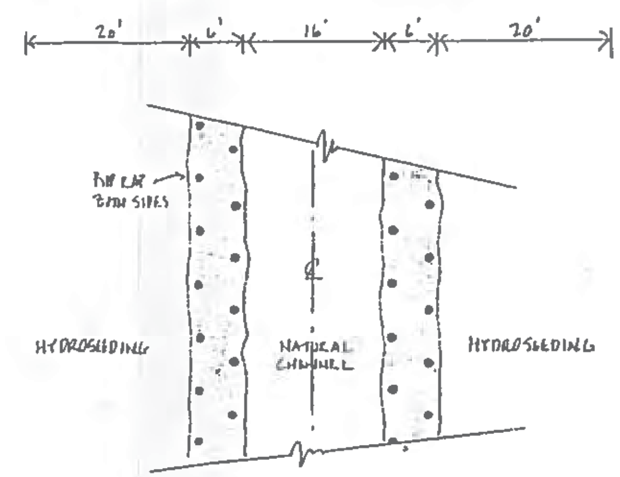
WETLAND PLANTING IN PERIMETER RIP RAPS

1. MAKE STAKES FROM NATIVE PLANT STEMS. CUT BOTTOM DIAGONALLY. ONE LATERAL BRANCH IS O.K. USE ½"–1" DIA.
2. AMEND EACH STAKE LOCATION WITH COMPOST IF POSSIBLE.
3. TIMING OF STAKING SHOULD BE AFTER 1ST FROST AND LEAVES HAVE DROPPED AND BEFORE END OF FEBRUARY.
4. SPACING OF STAKES TO BE 3' O.C. STAGGERED IN 6' RIP RAP AS SHOWN.
5. USE A BREAKER BAR TO SHAPE HOLES FOR STAKES
6. SPECIES TO BE WILLOW, SNOWBERRY, RED-STEM DOGWOOD OR OTHER WET SOIL SPECIE FROM QUEEN CITY FARMS



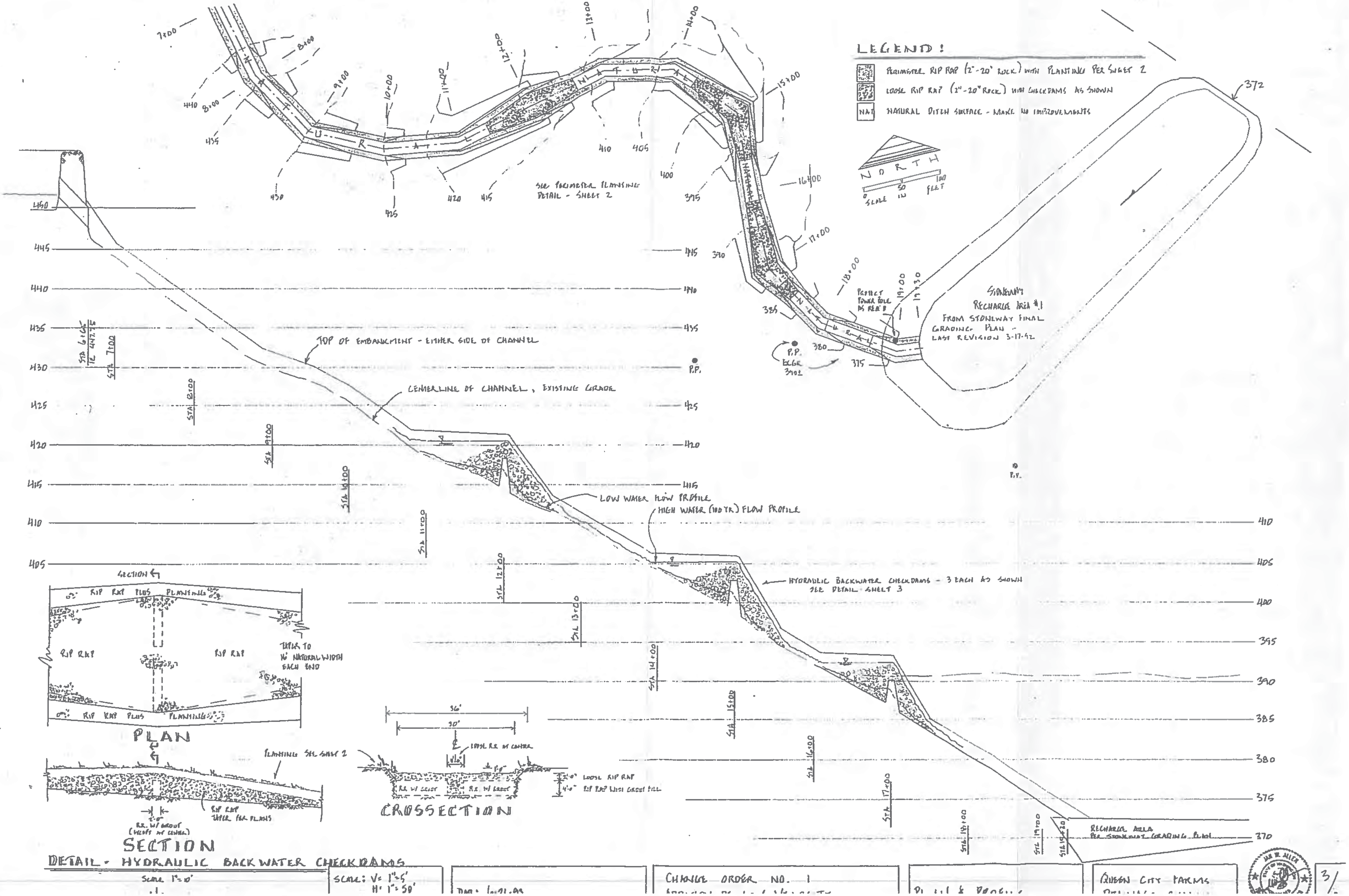
SECTION





WETLAND PLANTING PLAN
FOR PERIMETER RIP RAP AREAS
TYPICAL - BOTH SIDES OF DRAINAGE CHANNEL
SCALE 1"=10'

SCALE V= 1"=5' H= 1"=50'	DATE: 7-15-92	CHANGE ORDER NO. 1 ADDITION OF LOW VELOCITY	PLAN AND PROFILE	QUEEN CITY FARMS DRAINAGE CHANNEL	2/3

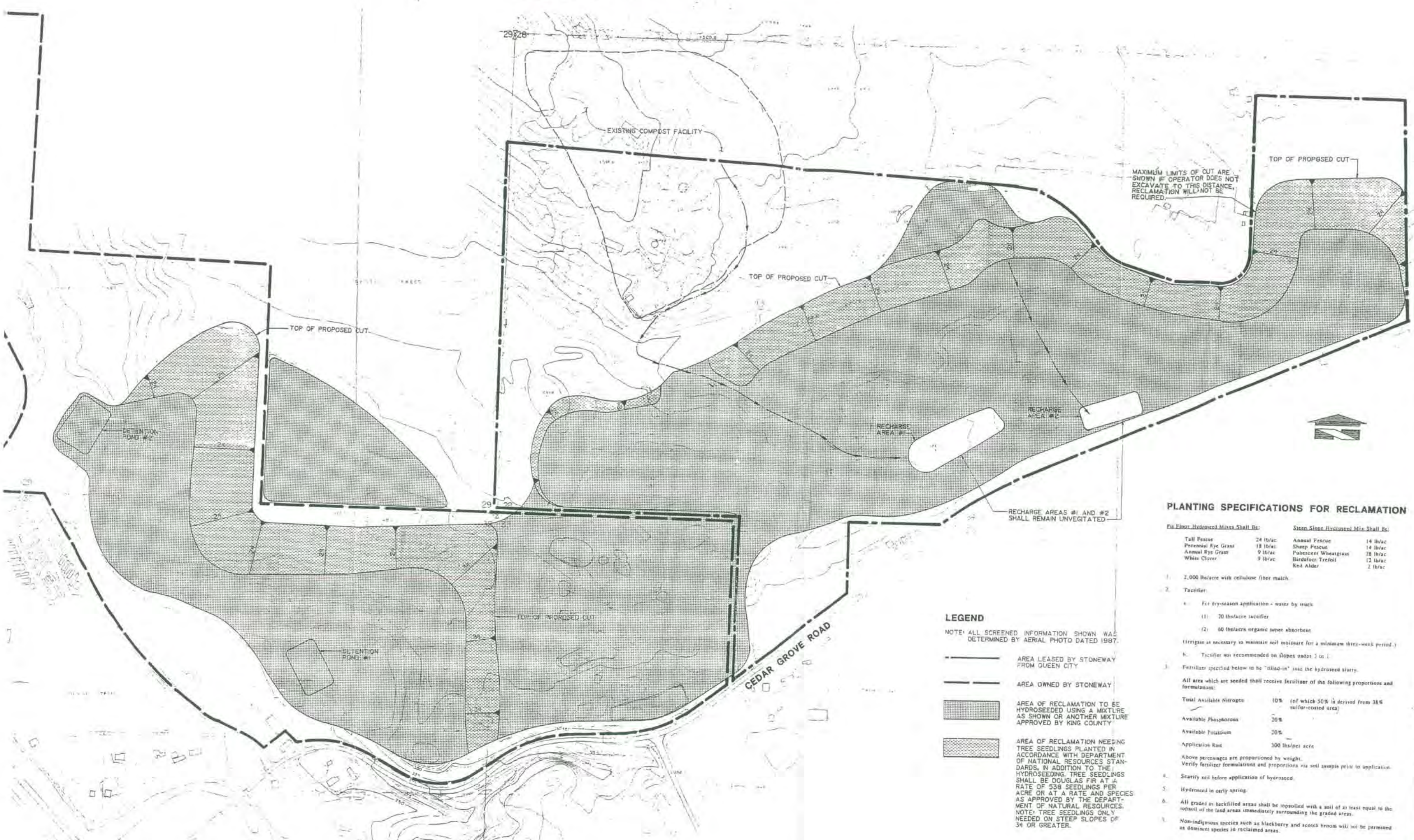

LEGEND:
PERIMETER RIP RAP (2"-20" ROCK) WITH PLANTING PER SHEET 2
LOOSE RIP RAP (2"-20" ROCK) WITH CHECKDAMS AS SHOWN
NAT NATURAL DITCH SURFACE - MAKE NO IMPROVEMENTS
NORTH
SCALE IN FEET
SEE PERIMETER PLANTING DETAIL - SHEET 2
PROTECT POWER POLE AS REQ'D
STONEWAY RECHARGE AREA #1 FROM STONEWAY FINAL GRADING PLAN - LAST REVISION 3-17-92
P.P. EDGE 390±
TOP OF EMBANKMENT - EITHER SIDE OF CHANNEL
CENTERLINE OF CHANNEL, EXISTING GRADE
LOW WATER FLOW PROFILE
HIGH WATER (100 YR) FLOW PROFILE
HYDRAULIC BACKWATER CHECKDAMS - 3 EACH AS SHOWN SEE DETAIL SHEET 3
RECHARGE AREA PER STONEWAY GRADING PLAN
RIP RAP PLUS PLANTING
RIP RAP
TAPER TO 16' NATURAL WIDTH EACH END
PLAN
PLANTING SEE SHEET 2
R.R. W/ GROUT
RIP RAP TAPER PER PLANS
SECTION
LOOSE RIP RAP
RIP RAP WITH GROUT FILL
CROSSECTION
DETAIL - HYDRAULIC BACKWATER CHECKDAMS
SCALE 1"=10'
SCALE: V: 1"=5' H: 1"=50'
CHANGE ORDER NO. 1
QUEEN CITY FARMS

Attachment 4

1991 Reclamation Plan

FINAL RECLAMATION PLAN

SCALE: 1" = 200'



LEGEND

NOTE: ALL SCREENED INFORMATION SHOWN WAS DETERMINED BY AERIAL PHOTO DATED 1987.

- AREA LEASED BY STONEWAY FROM QUEEN CITY
- AREA OWNED BY STONEWAY
- AREA OF RECLAMATION TO BE HYDROSEEDED USING A MIXTURE AS SHOWN OR ANOTHER MIXTURE APPROVED BY KING COUNTY
- AREA OF RECLAMATION NEEDING TREE SEEDLINGS PLANTED IN ACCORDANCE WITH DEPARTMENT OF NATIONAL RESOURCES STANDARDS, IN ADDITION TO THE HYDROSEEDING. TREE SEEDLINGS SHALL BE DOUGLAS FIR AT A RATE OF 538 SEEDLINGS PER ACRE OR AT A RATE AND SPECIES AS APPROVED BY THE DEPARTMENT OF NATIONAL RESOURCES. NOTE: TREE SEEDLINGS ONLY NEEDED ON STEEP SLOPES OF 3:1 OR GREATER.

PLANTING SPECIFICATIONS FOR RECLAMATION

Pit Floor Hydroseed Mixes Shall Be:		Steep Slope Hydroseed Mix Shall Be:	
Tall Fescue	24 lb/ac	Annual Fescue	14 lb/ac
Perennial Rye Grass	18 lb/ac	Sheep Fescue	14 lb/ac
Annual Rye Grass	9 lb/ac	Pubescent Wheatgrass	28 lb/ac
White Clover	9 lb/ac	Birdsfoot Trefoil	12 lb/ac
		Red Alder	2 lb/ac

1. 2,000 lbs/acre with cellulose fiber mulch.
2. Tacifier
   a. For dry-season application - water by truck
      (1) 20 lbs/acre tacifier
      (2) 60 lbs/acre organic super absorbent

   (Irrigate as necessary to maintain soil moisture for a minimum three-week period.)

   b. Tacifier not recommended on slopes under 3 to 1.
3. Fertilizer specified below to be "tilled-in" into the hydroseed slurry.

   All area which are seeded shall receive fertilizer of the following proportions and formulations:

Total Available Nitrogen	10%	(of which 50% is derived from 38% sulfur-coated urea)
Available Phosphorous	20%	
Available Potassium	20%	
Application Rate	300 lbs/per acre	

   Above percentages are proportioned by weight.
   Verify fertilizer formulations and proportions via soil sample prior to application.
4. Scarify soil before application of hydroseed.
5. Hydroseed in early spring.
6. All graded in backfilled areas shall be topsoiled with a soil of at least equal to the topsoil of the land areas immediately surrounding the graded areas.
7. Non-indigenous species such as blackberry and scotch broom will not be permitted as dominant species in reclaimed areas.

RECLAMATION NOTES

1. When a minimum 20-acre size portion of the site has been excavated to its desired elevation, the area shall be reclaimed. The mining area within Stoneway's lease from Queen City Farms, shall be reclaimed, starting at the eastern end, then in the direction to the west, in 20-acre increments. This entire area shall be reclaimed by mid-summer of 1992. The reclamation procedure is described on Sheet 3. Planting of the trees shall also commence during the hydroseeding period of time. The area outside of the Queen City lease shall also be reclaimed in 20-acre increments, as the final excavation occurs. This reclamation procedure shall commence from the western portion near Cedar River, to the easterly direction. This area will also use the notes and procedures for reclamation as described on Sheet 3. Any 20-acre increment that has been fully mined to its desired elevation, shall be reclaimed within one year of completed grading activities.
2. In addition to this plan, reclamation of the pit shall conform to all permit conditions.

For: **Stoneway Concrete**
1915 Maple Valley Highway
Renton, Washington 98055
(303) 226-1000

Scale: Horizontal 1" = 200' Vertical N/A

Designed HPG Drawn HPG Checked HPG Approved Date 10-01-9

Barghausen Consulting Engineers Inc.
Land Planning, Survey & Engineering Specialists
18215 72nd Ave South Kent, Wash. 98032 (206) 251-6222

HAL P. GRUBB — STAMP

Job Number 2789

Sheet 3 of 3

Attachment 5

2002 Sensitive Area Report

Sensitive Areas Report

Queen City Farms/Cedar Shores Refilling Project

King County Tracking Number: L02GI014
King County, Washington

Prepared for:
Emerald Services, Inc.
7343 East Marginal Way South
Seattle, WA 98108
And
Stoneway Rock and Recycling
1915 Maple Valley Highway
Renton, WA. 98055

November 18, 2002
Our Job No. 10252

Barghausen Consulting Engineers, Inc.

Civil Engineering, Land Planning, Surveying, Environmental Services
18215 72nd Avenue South, Kent, WA 98032 • (425) 251-6222 • (425) 251-8782 fax
www.barghausen.com

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The top of the bank above the ravine where Tributary 0315A is located has been mined and does not appear to have a water source draining from the top into the ravine. Approximately 100 feet beyond the top of the slope, to the west, is the head wall where Tributary 0315A begins. The channel contains cobbles, gravel and sand in its bed and has a shallow (less than one foot deep) sediment fan at its discharge point onto a terrace above the Cedar River floodplain. The terrace is approximately 500 feet wide near this tributary and slopes gently (nine percent) to the west towards the Cedar River. Neither wetlands nor drainage channels were observed on the terrace between the end of the this tributary and the top of the cliff above the Cedar River floodplain. At approximately the 250-foot elevation the terrace ends and the topography drops sharply down a fifty to sixty foot high cliff. Wetland 105 and the Cedar River floodplain are located at approximately the 200-foot elevation near the base of the cliff (Figure 6). The cliff face directly above the Cedar River floodplain that is located approximately 500 feet west and 150 feet south of this tributary location is eroding and has created a sediment fan at the base of the slope. This is typical of cliffs located along the Cedar River.

Vegetation along the riparian corridor includes big leaf maple, western red cedar, red alder, vine maple (*Acer circinatum*), sword fern, butterfly bush, thimbleberry (*Rubus parviflorus*), Himalayan blackberry, elderberry, and dewberry. Vegetation within the channel includes red alder, vine maple, and stinging nettle (*Urtica dioica*). Soils in the channel range from cobbles, gravel and sand in the upper channel to gravel and sand in the sediment fan at the lower end of the tributary. Above the south bank of the upper end of the channel is an overflow pipe that discharges water from the on-site infiltration system into a bounder and cobble lined area with a vertical concrete dissipater. There was no erosion observed in the channel adjacent to the dissipater system. At the head wall some gravel has been exposed by water that discharges into this tributary via a seasonal seep (Photograph 4). Water was not present in the channel or near the ground surface during the site visit. The channel is a blind system with water entering via the seep and ending on the terrace where it infiltrates into the ground. Surface water from this system does not connect to other surface waters downstream. Information collected at specific points along the channel are located on the Data Sheets in Appendix E. Photographs of the channel are located in Appendix D (Photographs 4 through 14).

This tributary is an ephemeral, blind stream. In accordance with the King County Zoning Code 21A.06.1240 this system would be a Class 3 stream which is not used by salmonids (Table 1).

7.3 Tributary 0316A

Tributary 0316A was a natural drainage that historically discharged to an area near Queen City Lake. This channel has been rerouted several times over the last thirty years. The current location of the channel was part of a preliminary reclamation plan to allow surface water flows to cross the site to a specific infiltration (recharge) area. The final location of the infiltration area was determined through extensive negotiation with the property owners, the mining company, local residence and regulatory agencies. No documentation was located that indicated that collecting and rerouting surface water flows through the site was mitigation for a stream channel. The channel was engineered as a V-shaped ditch with ecology block velocity dissipaters, rip rap check dams, a grass lined biofiltration basin near the upper third of the realigned channel, native vegetation staking in the rip rap to reduce erosion and provide cover for the channel, and an engineered recharge area to allow the surface water flows to infiltrate into the ground. The Drainage Channel Realignment Plans as signed and approved by King County on September 1, 1993 are located in Appendix F.

Aerial photograph review showed the following for Tributary 0316A. The 1936, 1960, 1968, 1974, and 1985 photos show Tributary 0316A flowing from its current location onto the site, and then flowing towards the west end of Queen City Lake. The 1989 photo shows several ditches on the north side of the airport (that was present on the site) rerouting water from this tributary toward the west. The 1990 photos show that the Cedar Grove Compost site was constructed. The tributary was rerouted around the eastern and southern boundaries of the Cedar Grove Compost site. Water then flowed into the Wetland 8 area. The 1991 photo shows the tributary flowing through the Wetland 8 area and then flowing toward the southwest toward the sedimentation ponds described in Section 6.0 of this report. The 1997 photo shows that the tributary was rerouted to the south along the east side of the Cedar Grove Compost access road. The tributary ends near an east-west trending dirt road near the south site boundary. No trees or shrub vegetation were observed along the tributary. The 2000 photo shows the current configuration of the tributary with tree and shrub cover.

Extensive review of the Cedar Grove Compose and Stoneway Rock and Recycling files indicated the following regarding the current location of Tributary 0316A. Prior to construction of an airport on the site the tributary flowed toward and into Queen City Lake (1978 and 1985 maps). By 1987 ditches constructed along the north site of the airport runway rerouted the flows in the tributary to the west. Mining operations were taking place to the south near the Wetland 8 area and a shallow pond (Wetland 8) was excavated to collect water from the tributary and reroute it to the sedimentation ponds (1987 and 1988 maps). In 1988 a SEPA checklist was submitted to King County to allow construction of the Cedar Grove Compost facility. On January 1989 a DNS was issued by King County DDES which included routing water in the tributary around the perimeter of the Cedar Grove Compost facility and toward the west. On February 13, 1990, Chris Tiffany, King County Grading Department, and Gayle Kreitman, Washington State Department of Fisheries, visited the site to inspect the drainage swale designated as Tributary 0316A. Their inspection determined that the seasonal drainage swale (Tributary 0316A) that originates uphill from the Cedar Grove Compose facility terminated in a dense stand of trees and thicket before it percolated into the soil near the east end of the sedimentation pond (the sedimentation ponds discussed in Section 6.0 of this report). Their site visit was intended to determine if the drainage swale discharged into wetland that had been inventoried by King County. The swale did not discharge to inventoried wetland and in March of 1990 the annual grading permit was renewed for work at the Cedar Grove Compost facility. On October 30, 1990, a Revised SEPA Checklist addendum was submitted to King County Department of Public Health. On August 30, 1991 a MDNS was issued for continuing activities on the Cedar Grove Compost facility, which stated that stormwater pathways must be planted to provide additional filtration of stormwater runoff. In January 1991, Paramentrix completed a Preliminary Plan to Realign the Queen City Farms Drainage Channel (Tributary 0316A). The channel was to be located directly adjacent to the Cedar Grove Compost facility access road. The final 370 feet of the channel that would flow through an area that was still being mined was to be piped and discharge into an infiltration pond located east of the current infiltration area. The final location of the infiltration area was determined through extensive negotiation with the property owners, the mining company, local residence and regulatory agencies. The final Queen City Farms Drainage Channel Realignment Plans and Addendum were signed and approved by Mr. Chris Tiffany, King County Grading Department in September 1993, and are located in Appendix F.

Information for this channel was gathered from the infiltration area at the lower end of the system to approximately 100 feet above the area where the system is proposed to be rerouted during refill of the site. Vegetation along the riparian corridor includes red alder, butterfly bush, Himalayan blackberry, willow, black cottonwood, Scott's broom, sword fern, salmonberry, dewberry, velvet grass, bentgrass, fescue, orchard grass, ryegrass (*Lolium perenne*), and white clover (*Trifolium repens*). Vegetation within the channel includes red alder, willow, butterfly bush, red osier dogwood (*Cornus stolonifera*), teasel (*Dipsacus sylvestris*), horsetail (*Equisetum arvense*), and reed canarygrass. Vegetation in the biofiltration basin includes spiked rush, knotweed, cattail, nightshade (*Solanum dulcamara*), bulrush, and velvet grass. Vegetation in the infiltration basin includes red alder, willow, cattail, bulrush, sedges (*Carex spp.*), and rushes. Soils in the channel range from cobbles and gravel to sand and silt. The infiltration area contains a one- to six-inch deep layer of silt underlain by gravel. The riparian areas are rip rap lined slopes. Beyond the slopes are grassy mowed fields, and gravel lots used as storage yards. There were no erosion faces observed in the channel. Water was not present in the channel or near the ground surface during the site visit. The channel is a blind system with water entering the system up slope and off of the site and ending on the terrace where it infiltrates into the ground. Surface water from this system does not connect to other surface water downstream. Only a small area (about 5 acres) of the site currently discharges to Tributary 0316A (Barghausen Consulting Engineers, Inc., 1998). Information collected at specific points along the channel are located on the Data Sheets in Appendix E. Photographs of the channel are located in Appendix D (Photographs 15 through 31).

This tributary is an ephemeral, blind ditch that was designed and constructed to allow surface water flows to cross the site. In accordance with the King County Zoning Code 21A.06.1240 this system would be a Class 3 stream which is not used by salmonids. This system should be considered as an engineered ditch that collects and routes surface water across the site to an infiltration system

8.0 WILDLIFE AND HABITAT EVALUATION

Wildlife directly observed on the site, during the three site visits in September and October of 2002, include crows, ravens, gulls, deer, and several species of songbirds. The deer were observed near the infiltration ponds located southeast of Tributary 0315A. A deer leg that had been dragged and fed on by coyotes was observed in the central portion of the Tributary 0316A corridor. Coyote prints and scat was located in the area of the deer leg. No plant or animal species listed federally or by the state as threatened or endangered were observed on the site.

Squirrels, flickers, pileated and downy woodpeckers, several species of songbirds, garter snakes, and Pacific chorus frogs were observed off of the site near the Cedar River. During review of documents for this site it was noted that elk frequent the local area near the Cedar River. Nest sites for herons, raptors and other birds were not observed on or near the site during the three site visits.

A review of the Washington State Department of Fish and Wildlife database indicated that the nearest habitats listed in the database are (a) the Queen City Lake wetland located northeast of the project (this wetland is an EPA hazardous waste superfund site), (b) the King County Lower Cedar River Wetland 31 and 32 (Figure 2), (c) the King County Lower Cedar River Wetland 105 (Figure 2), and (d) the Cedar River and riparian zones along the river. The Cedar River its wetlands and riparian zones are listed as habitats for a large number of avian and terrestrial species including large and small mammals. The Cedar River (Stream LLID 1222590476452) is used by Chinook Salmon, Coho Salmon, Winter Steelhead, Resident Cutthroat Rainbow Trout, Kokanee Salmon, Dolly Varden/Bull Trout, and Riffle

TECHNICAL MEMORANDUM

TO: Mr. Alan Wallace, Williams Kastner and Gibbs

FROM: Dave Pischer, P.E. and Eric Weber, L.G.

DATE: January 29, 2007

RE: EVALUATION OF HIGHER WATER LEVELS IN QUEEN CITY LAKE ON THE EXISTING QUEEN CITY FARMS VERTICAL BARRIER WALL SYSTEM
QUEEN CITY FARMS REFILL PROJECT
MAPLE VALLEY, WASHINGTON

This technical memorandum has been prepared to summarize Landau Associates' evaluation of the potential effect of higher water levels in Queen City Lake on the existing vertical barrier wall system at the Queen City Farms (QCF) Superfund site. The higher water levels will likely result from planned modification of the existing Queen City Lake outflow outlet structure. This planned modification is associated with the planned reclamation (refilling) of the QCF gravel pit. Plans for modification of Queen City Lake outflow are as follows:

- Prior to implementation of Phase 3[1] of the refill plan, the current 36-inch Queen City Lake outflow pipe with an invert elevation of 448.1 ft (NGVD 29) will be removed.

- A new Queen City Lake discharge pipe will be installed at the same invert elevation as the old discharge pipe. The new pipe will be designed to discharge 2 cfs[2] from Queen City Lake to an infiltration pond east of the Main Gravel Pit Lake. The effect of this smaller outflow pipe will be to maintain average water levels in Queen City Lake near their present levels while allowing the lake to fill up as a storage reservoir during extreme storm events

- A new emergency outflow will be installed at about Elevation 453 ft (NGVD 29) (equivalent to a depth of 9 ft in Queen City Lake) to address uncertainty in containing a 100-yr storm event within Queen City Lake.[3] The emergency outflow will discharge to the eastern infiltration pond.

[1] Filling of the gravel mine pit will take place in three phases. Phase 3, the last phase will result in elimination of the Main Gravel Pit Lake and filling around the East Airstrip Spring. A final schedule has not been determined for filling; however, it is unlikely that Phase 3 will commence before 2010.

[2] Maximum monthly recharge to Queen City Lake was estimated at about 2.5 cfs based on the 1987/88 and 1988/89 water years. Average winter time (November through April) monthly recharge to Queen City Lake was estimated at less than 2 cfs (Landau Associates 1990).

[3] Surface water modeling performed as part of the refill plan design indicates that a 100-yr storm event will cause a maximum water level depth of about 8.5 ft in Queen City Lake. Therefore, the emergency outflow structure is intended only as a backup to account for uncertainty in the storm flow analysis.

As discussed in the *QCF Vertical Barrier Wall System Task Remedial Design Report* (TRD Report; Kennedy/Jenks 1996) and the *Final Project Closure Report* (Kennedy/Jenks 1998), the vertical barrier wall system consists of the following three main elements:

- A 3- to 4-ft wide vertical soil-bentonite barrier wall (slurry wall) keyed into or sufficiently embedded through the hydrogeologic units serving as the aquitard system below Aquifer 1
- A multi-layered cover system over the area enclosed by the slurry wall, and
- A surface water drainage system.

The as-built alignment of the slurry wall relative to Queen City Lake and the planned refilling limits for the QCF gravel pit are shown on Figure 1, and the corner points and stationing of the slurry wall are shown on Figure 2. The location of the surface water drainage system that discharges to Queen City Lake is shown on Figure 3.

As discussed in this technical memorandum, it is concluded that the planned higher water levels in Queen City Lake will not adversely affect the performance of the QCF vertical barrier wall system. However, some relatively minor modifications to some of the outfall pipes associated with the final cover drainage system may be appropriate due to a rise in the level of Queen City Lake during a 100-yr storm event.

ELEVATION DATUM

It is important to recognize that the elevations shown on recent site surveys and the plans for refill of the QCF gravel pit are referenced to the North American Vertical Datum of 1988 (NAVD 88). The elevation datum used for most of the previous site investigations and design/construction of the QCF vertical barrier wall system is the National Geodetic Vertical Datum of 1929 (NGVD 29), which is based on mean sea level (MSL). In the vicinity of the project site, elevations referenced to NAVD 88 are about 3.5 ft higher than those referenced to NGVD 29. For example, the existing 36-inch-diameter Queen City Lake outflow pipe has an invert at Elevation 448.1 ft (NGVD 29 or MSL) or Elevation 451.6 ft (NAVD 88)

BACKGROUND WATER LEVEL INFORMATION

Queen City Lake represents the discharge location for an approximate 370-acre basin that contains the southern portion of King County's Cedar Hills Regional Landfill (CHRL). Historically (i.e., prior to February 1991), all surface water runoff from this basin flowed into Queen City Lake and infiltrated into the underlying soil. At that time, Queen City Lake had no surface water outlet and recharged directly to Aquifer 1, a shallow perched aquifer beneath the lake. Queen City Lake is typically dry from early August to November.

Aquifer 1 is a small, highly permeable aquifer that includes openwork gravel deposits. Recharge to Aquifer 1 is primarily through leakage from Queen City Lake and direct recharge of surface water runoff. Discharge from Aquifer 1 is through spring flow (primarily the East Airstrip Spring) and leakage through the Aquifer 1 aquitard. During storm events and high rates of surface water runoff into the Queen City Lake basin, Aquifer 1 experiences high rates of recharge due to its shallow water table and permeable soil. The high recharge rates cause Aquifer 1 water levels to rise sharply. However, because of the leaky nature of the Aquifer 1 aquitard, groundwater levels also fall quickly. Historically, Aquifer 1 water levels have fluctuated over 20 ft due to fluctuations in recharge and discharge.

In February 1991, an outflow structure was installed along the south side of Queen City Lake. The outflow consisted of a 36-inch-diameter pipe with an invert at Elevation 448.1 ft (NGVD 29). The outflow routed water from Queen City Lake to a ravine that discharged directly to the Main Gravel Pit Lake. Prior to installing the outflow structure, the lake fluctuated approximately 9 ft, from about Elevation 444 to 453 ft (NGVD 29). After the outflow structure was installed, the lake only fluctuated about 4.5 ft, from about Elevation 444 to 449.5 ft (NGVD 29; see Attachment 1). Installation of the outflow structure reduced recharge from the lake to the underlying Aquifer 1.

Historical data records indicate that water levels in Aquifer 1 declined in 1991 after installation of the Queen City Lake outflow pipe. Prior to installation of the outflow pipe, the average winter time water level in Aquifer 1 was about Elevation 438.5 (NGVD 29), and the maximum recorded water level elevation during the period from 1988 to 1993 was about Elevation 448.5 ft (NGVD 29) recorded in April 1988. After the installation of the outflow pipe in February 1991, the average winter time water level in Aquifer 1 was about Elevation 435 ft (NGVD 29), and the maximum water level during the period from 1988 to 1993 was recorded at about Elevation 438 ft (NGVD 29).[4] Installation of the outflow pipe caused a decline in the average winter time Aquifer 1 water level of about 3.5 ft. Pipe installation reduced the maximum Aquifer 1 water levels about 10 ft. Aquifer 1 water level data from 1987 to 1993 are presented in Attachment 2.

The QCF slurry wall, which was constructed in 1996, was keyed into or sufficiently embedded through the hydrogeologic units serving as the aquitard system below Aquifer 1. Following its installation, Aquifer 1 water levels within the slurry wall dropped as expected due to the leaky nature of the aquitard system below Aquifer 1, and Aquifer 1 water levels outside the slurry wall have fluctuated

[4] Aquifer 1 water levels from 1987 through 1993 were characterized based on continuous (e.g., hourly or daily readings) water level readings recorded by a datalogger.

seasonally between about Elevation 423 to 434 ft (NGVD 29).[5] Installation of the slurry wall does not seem to have affected Aquifer 1 water levels outside the slurry wall. Aquifer 1 water level data during the period from 1996 to 2005, based on the *2005 Annual Monitoring Data Report* (EcoChem and Boeing 2006), are presented in Attachment 3.

ANTICIPATED WATER LEVEL MODIFICATIONS

Modification of the outflow structure to Queen City Lake prior to implementing Phase 3 of the refill plan will impact Aquifer 1 water levels. These modifications call for replacing the current outflow structure with a new structure capable of discharging a maximum of about 2 cfs. Because 2 cfs is more than the estimated average winter-time (November through April) runoff in the Queen City Lake sub-basin, the modifications are predicted to have a minor (less than 2 ft rise) impact on the average winter-time water levels in Aquifer 1. However, the maximum water levels may increase. Based on the continuous water level monitoring level record prior to installation of the original outflow pipe (November 1987 to February 1991), the maximum water level in Aquifer 1 was about Elevation 448.5 ft (NGVD 29) in April 1988. This water level was maintained for a short period of time (about one day; Attachment 2). Elevation 448.5 ft (NGVD 29) is considered a future maximum Aquifer 1 water level after implementation of modifications to the Queen City Lake outflow structure for the following reasons:

- During 1988 and 1989, storm water runoff from the landfill was likely abnormally high. The CHRL South-Central Detention basin was not constructed until 1988. The CHRL Southwest Siltation Pond was not constructed until 1990 (King County Solid Waste Division 2005).

- Excluding April 1988, the highest water level in Aquifer 1 during the period November 1987 to February 1991 was Elevation 445 ft (NGVD 29; Attachment 2).

- CHRL has upgraded and expanded their storm water handling capability. By the end of 2007, CHRL plans to have completed a pipeline that discharges up to 1.8 cfs of surface water flow from the landfill into an infiltration trench along Cedar Grove Road SE (King County Department of Transportation 2006). Modeling (TIR Sections 6.1 and 6.10) indicate that the affect of proposed CHRL capital improvements will result in lower average water levels in Queen City Lake compared to current conditions.

- 100-year storm event modeling of the Queen City Lake sub-basin predicts that Queen City Lake will rise to a maximum water level depth of about 8.5 ft under modified outflow pipe conditions. This is slightly lower than the maximum water level depth recorded in Queen City Lake prior to implementation of the current outflow structure (Attachment 1)

- A new outflow structure (capable of discharging 2 cfs) will replace the old one with the goal of maintaining current average water levels in Queen City Lake and Aquifer 1. This new structure will mitigate the impact of any future extreme storm event.

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[5] Aquifer 1 water levels after 1997 were characterized based on hand-held reading typically collected twice a year.

Once Phase 3 of the refill plan is implemented, maximum Aquifer 1 water levels should be less than Elevation 448.5 ft (NGVD 29). Average Aquifer 1 water levels should be less than Elevation 437 ft (NGVD 29). The bottom of the Aquifer 1 aquitard along the slurry wall alignment is estimated to be Elevation 415 ft (NGVD 29) or greater (Figure 4; Landau Associates 1990). Therefore, the maximum head acting on the slurry wall (assuming that the area inside the slurry wall is dry) is about 33.5 ft. The slurry wall was designed for a maximum head differential of 47 ft (Kennedy/Jenks 1996). An estimate of the top elevation of the Aquifer 1 aquitard developed from boring information presented in the *LNAPL Immobilization Task Remedial Design Report* (Landau Associates 1994) is presented on Figure 4. A cross section through the slurry wall area was presented in the *LNAPL Immobilization Remedial Design Report* (Landau Associates 1996). The cross section location is shown on Figure 5; the cross section is shown on Figure 6.

BARRIER WALL HYDRAULIC STABILITY EVALUATION

As discussed in the Section 2.2.4 of the *QCF Vertical Barrier Wall System TRD Report* (Kennedy/Jenks 1996) and in Section 3.1.2 of the Final Report (Kennedy/Jenks 1998), the design of the slurry wall included consideration of the anticipated hydraulic pressures and gradients acting on the wall to check the stability against:

- Piping/blowout of the soil-bentonite backfill into adjacent formation materials
- Hydraulic fracture within the wall, and
- Erosion/piping of the wall backfill at the base of the wall into adjacent coarser formation materials due to under-seepage.

A brief summary of the slurry wall design for these stability assessments, and the effect of higher Aquifer 1 water levels on these design stability assessments, are presented below.

Hydraulic Head Design Assumptions

During design, numerical modeling of groundwater flow in the vicinity of the wall was conducted to attempt to quantify the worst-case hydraulic head conditions along the barrier wall system. The modeling suggested that approximately 12 ft of additional head might build up on the northern side of the upgradient portion of the wall (Kennedy/Jenks 1996). Assuming an initial 35 ft saturated thickness of Aquifer 1 plus 12 ft of head build-up after wall construction, and that the inside of the wall would be dewatered, the maximum hydraulic head acting on the wall was conservatively assumed to be 47 ft for the purpose of the design stability assessments (Kennedy/Jenks 1996). This evaluation was considered to be consistent with Landau Associates' conclusion that the maximum estimated head across the barrier wall would be 40 ft or less (Landau Associates 1992).

Because of these conservative assumptions used for the slurry wall design stability assessments, the potential increase in maximum Aquifer 1 water levels to Elevation 448.5 ft (NGVD 29) should not adversely affect the performance of the QCF barrier wall, as discussed below.

Piping/Blowout Potential Evaluation

Blowout of the soil-bentonite backfill material and piping of the backfill material into the adjacent formation soils were evaluated based on the assumed maximum hydraulic gradient across the wall (i.e., the assumed 47-ft maximum hydraulic head differential divided by the wall width). If blowout occurs, piping of the finer-grained wall backfill materials into the coarser formation soils would occur until the filter characteristics of the formation effectively plug the flow of the wall backfill materials (Kennedy/Jenks 1996).

As discussed in the Section 2.2.4.1 of the TRD Report (Kennedy/Jenks 1996), the results of the blowout analysis indicated that the greatest potential for blowout and piping failure within the completed wall in response to the assumed maximum hydraulic gradient would occur between Stations 8+60 and 13+79 (i.e., along the northwestern portion of the wall adjacent to Queen City Lake; Figure 2). Increasing the width of the slurry wall between Station 8+60 and 13+79 from a minimum of 3 ft to 4 ft increased the factor of safety for blowout to above 2.5, and increasing the width of the slurry wall between Station 13+79 and 18+97 (i.e., along the northeastern portion of the wall) from a minimum of 3 ft to 3.5 ft also reduced the blowout potential in that segment of the wall.

Given the conservative assumption of a 47-ft maximum hydraulic head across the wall in the vicinity of Queen City Lake, the increase in the minimum wall thickness noted above, and that considerable migration of the water-bentonite slurry and the soil-bentonite backfill into the coarser formation soils likely occurred during wall construction, it is concluded that the potential increase in maximum Aquifer 1 water levels to Elevation 448.5 ft (NGVD 29) should not adversely increase the potential for blowout and piping of the backfill material into the adjacent coarser formation soils.

Hydraulic Fracture Potential Evaluation

As discussed in the Section 2.2.4.2 of the TRD Report (Kennedy/Jenks 1996), the potential for hydraulic fracture of the wall is based on a condition when pore water pressure increases within the wall backfill material exceed the total vertical stress developed within the backfill material.

Arching of the backfill material between the trench walls can reduce the vertical stress of the backfill material, and arching is greatly influenced by wall width, as well as the gradation, shear strength, compressibility, and unit weight of the backfill materials.

The potential for hydraulic fracture exists where a rise in groundwater generates excess pore water pressures that exceed the total vertical stress developed at some depth within the backfill material. During the design evaluation for hydraulic fracture potential, it was conservatively assumed that there would be unsaturated conditions within Aquifer 1 during wall construction, and that there would be a subsequent 47-ft rise in the groundwater level within Aquifer 1 along the upgradient side of the wall (between Stations 8+60 and. 18+97; Figure 2).

The design evaluation indicated that the greatest potential for hydraulic fracture existed between Stations 8+60 and 13+97 (i.e., along the northwestern portion of the wall adjacent to Queen City Lake). As a conservative design measure to limit hydraulic fracture potential, the width of the wall between Station 8+60 and 13+79 was increased from a minimum of 3 ft to 4 ft, and the width of the wall between Station 13+79 and 18+97 was increased from a minimum of 3 ft to 3.5 ft. Increasing the wall widths along these segments of the wall provided the most straight-forward means of reducing hydraulic fracture potential by diminishing the potential effect of arching on the vertical stresses within the wall backfill material. (Kennedy/Jenks 1996). The hydraulic fracture potential was also limited by using a well-graded backfill mix, which reduced its compressibility and using proper backfill placement methods to reduce backfill aching.

Given the conservative assumption of a 47-ft rise in groundwater levels along the outside of the wall in the vicinity of Queen City Lake, the increase in the minimum wall thickness noted above, use of a well-graded backfill mix to reduced its compressibility, and use of proper backfill placement methods to reduce backfill arching, it is concluded that the potential increase in maximum Aquifer 1 water levels to Elevation 448.5 ft (NGVD 29) should not adversely increase the hydraulic fracture potential within the slurry wall backfill material.

Erosion Potential Evaluation

As discussed in the Section 2.2.4.3 of the TRD Report (Kennedy/Jenks 1996), the wall design evaluated the potential for erosion of the backfill material into the adjacent formation materials at the toe of the wall due to horizontal under-seepage, as well as the potential for heave of the embedment soils on the inside of the wall due to horizontal under-seepage.

The design analysis assumed a maximum wall depth of 75 ft, a maximum hydraulic head differential of 47-ft across the wall, a no-flow boundary at some arbitrary depth below the wall, and horizontal steady-state flow within Aquifer 1. It was anticipated that these conditions could potentially exist (assuming no silt aquitard penetration) between Stations 8+60 and 11+16 (i.e., along the open-works gravel zone along the northwestern portion of the wall adjacent to Queen City Lake; Figure 2).

The design evaluation indicated that the calculated pore water pressures developed at the toe of the wall due to under-seepage would not exceed the total stress in the backfill material for a 75-ft deep, 4-ft wide wall, and thus erosion of backfill material into the adjacent formation materials at the toe of the wall would not occur, even in the unlikely event that horizontal flow occurs under the wall. Additionally, the factor of safety against heave of the embedment soils on the inside of the wall due to horizontal under-seepage was greater than 5 because the calculated pore water pressures are greatly exceeded by the effective stresses in the embedment soils (Kennedy/Jenks 1996).

Given the conservative assumption of a 47-ft maximum hydraulic head across the wall in the vicinity of Queen City Lake, the minimum 3-ft embedment of the wall into the aquitard system materials below Aquifer 1 which would represent a low-flow or no-flow boundary, and the presence of a vertical groundwater gradient in Aquifer 1 as opposed to steady-state horizontal flow, it is concluded that the potential increase in maximum Aquifer 1 water levels to Elevation 448.5 ft (NGVD 29) would not adversely increase the potential for erosion of backfill material or heave of the native soils at the toe of the wall.

BARRIER WALL HYDRAULIC PERMEATION

The soil-bentonite barrier wall was designed and constructed to have a permeability no greater than $1\text{x}10^{-7}$ cm/sec and be stable and resistant to degradation from hydraulic permeation of the wall and from adjacent groundwater movement. The wall was designed to be stable under loading conditions that included dewatering of the interior formation and a 47-ft hydraulic head differential acting across the wall. For the portions of the wall most likely to be affected by water level fluctuations in Queen City Lake and Aquifer 1, the typical 3-ft wall thickness was increased to a minimum of 3.5- to 4-ft. Specifically, the width of the wall between Station 8+60 and 13+79 (i.e., along the northwestern portion of the wall adjacent to Queen City Lake) was increased from a minimum of 3 ft to 4 ft, and the width of the wall between Station 13+79 and 18+97 (i.e., along the northeastern portion of the wall) was increased from a minimum of 3 ft to 3.5 ft.

Because the wall was designed to function under a long-term, conservatively high hydraulic head differential, and the modification, the Queen City Lake outflow outlet structure will not significantly increase average water levels in Aquifer 1, it is probable that Aquifer 1 water that permeates the wall under the planned future groundwater conditions would be less than or equal to that assumed during design of the QCF vertical barrier wall system.

SURFACE WATER DRAINAGE SYSTEM EVALUATION

The location of the surface water drainage features and the upgradient diversion system associated with the QCF final cover system is indicated on Figure 3, which is based on Sheet C-9 of the project record drawings. As shown on Figure 3, the majority of the site drainage features discharge to Queen City Lake.

As previously discussed, the bottom of Queen City Lake has been surveyed at Elevation 446.6 ft (NAVD 88) or 443.1 ft (NGVD 29). The proposed outflow outlet structure modification will allow a maximum of 9 ft of water in Queen City Lake during a 100-year storm event; i.e., up to Elevation 455.6 ft (NAVD 88) or 452.1 ft (NGVD 29).

As shown on Figure 3, the majority of the site drainage features discharge to Queen City Lake with pipe inverts above Elevation 452.1 ft (NGVD 29), which would be the maximum 9-ft ponding level during a 100-yr storm event. However, two of the outfall pipes have existing inverts below Elevation 452.1 ft (NGVD 29). These include the outfall for the upgradient diversion system [invert at Elevation 449.89 ft (NGVD 29)], and the outfall for the central surface drainage system [invert at Elevation 449.30 ft (NGVD 29)]. Unless modified, the outfalls for these two drainage systems would be temporarily submerged during a 100-yr storm event, but high water in Queen City Lake would not back up within the outfall piping up to the pipe invert at the closest upgradient manhole or catch basin associated with these two outfalls (i.e., high water in Queen City Lake would not cause backflow into the drainage systems).

If temporary submersion of these two outfalls is not considered appropriate, the outfalls could be shortened and modified to raise the outfall pipe inverts to above Elevation 452.1 ft (NGVD 29), or the 12-inch PVC outfalls could be fitted with Tideflex™ backflow prevention valves. Some minor modification of the riprap erosion control blanket along the northwest perimeter of the cover system and the outfall energy dissipators may also be appropriate.

CONCLUSIONS

Because of the conservative assumptions used for the slurry wall design, it is concluded that potential changes in Aquifer 1 water levels due to the planned modifications to the Queen City Lake outflow outlet structure will not adversely affect the long-term performance of the QCF barrier wall system. However, as discussed above, some relatively minor modifications to some of the outfall pipes associated with the final cover drainage system may be appropriate due to a rise in the level of Queen City Lake during a 100-yr storm event.

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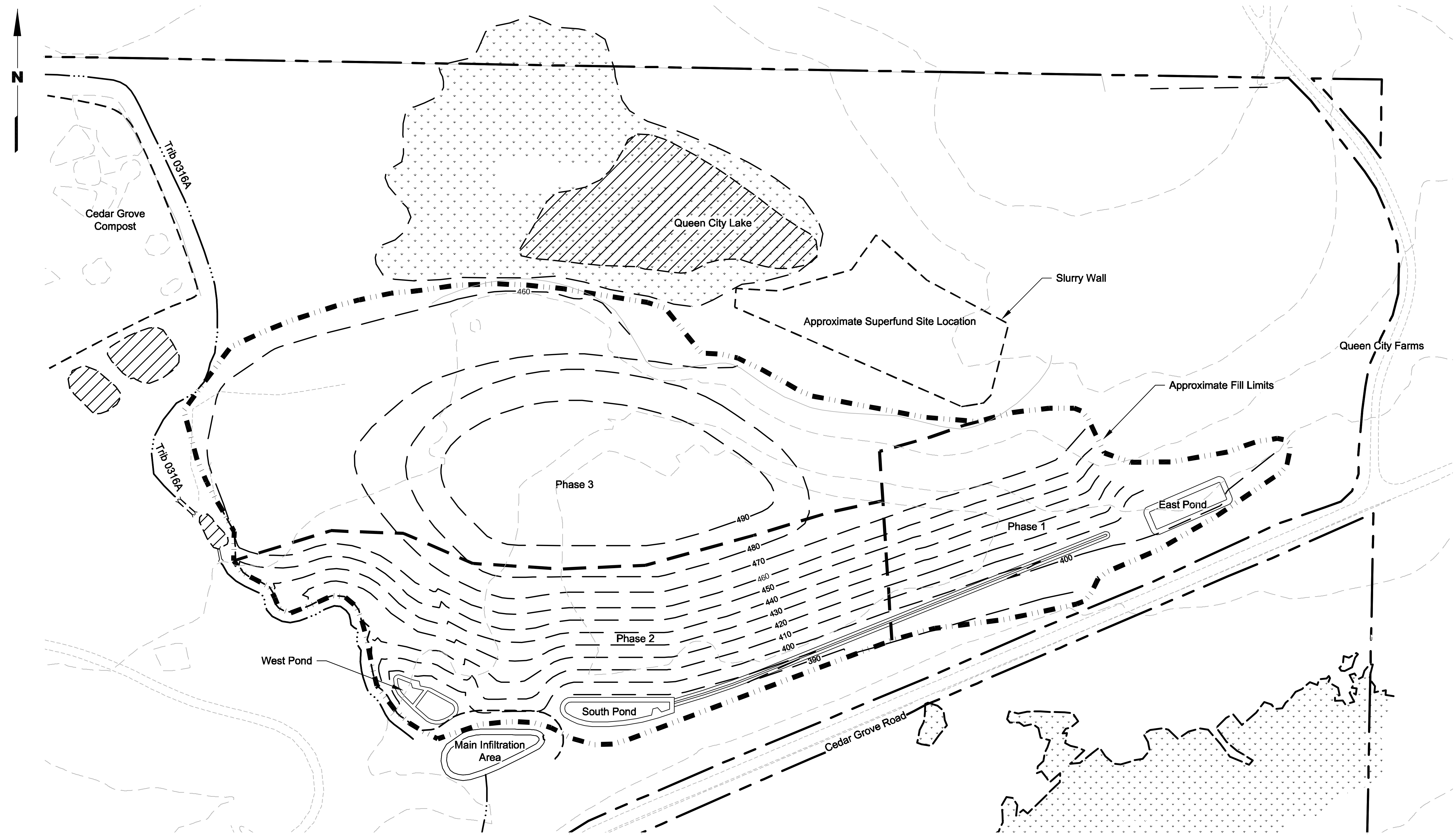


Queen City Farms
Refill Project
Maple Valley, Washington

Gravel Mine Reclamation Plan

Figure
1

LANDAU ASSOCIATES

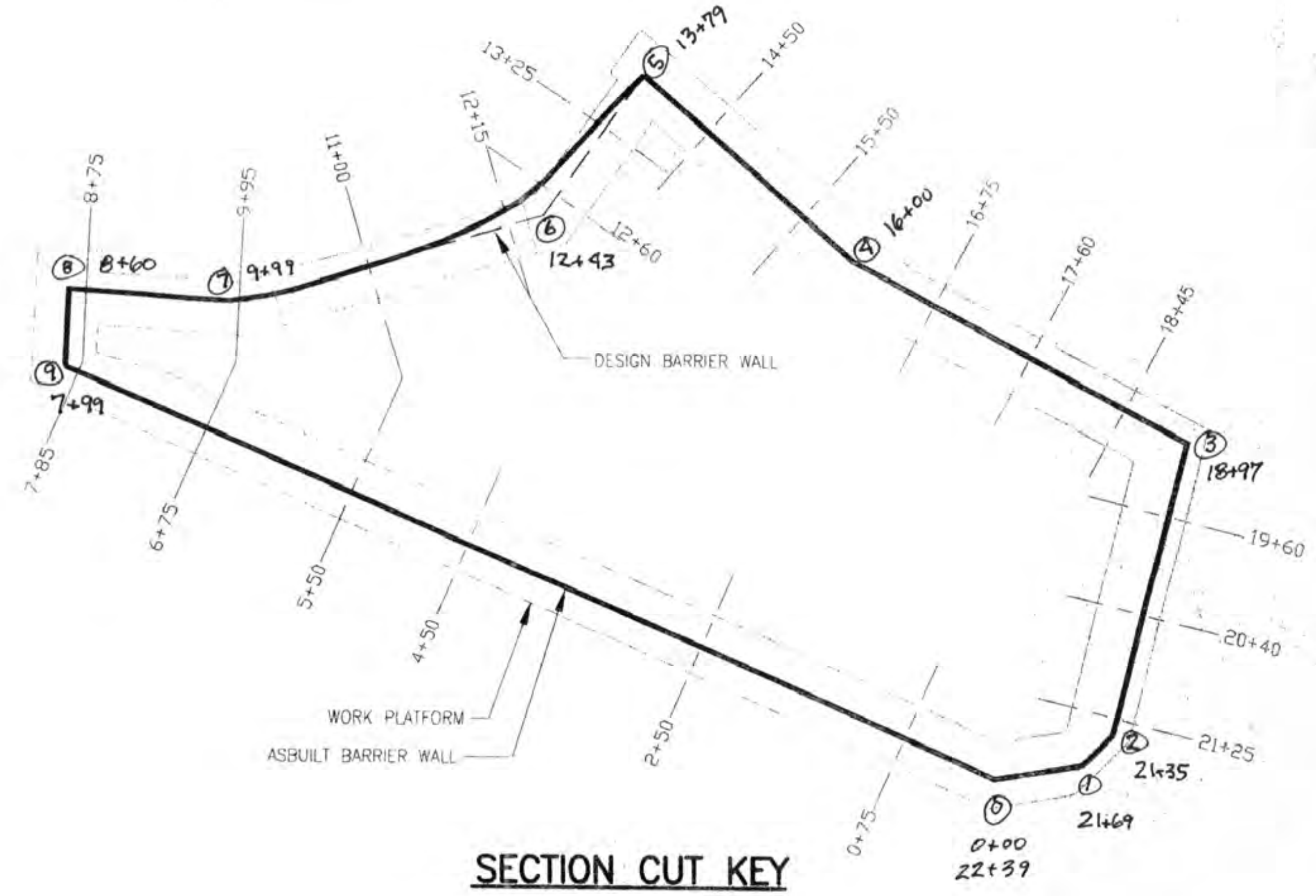
Queen City Farms | V:\992\001\011\D\TechMemos_FINAL\6.7 Slurry Wall\CAD\Figure 1.dwg (A) "Figure 1" 1/30/2007

N

NOTES:

1. LAYOUT THE ALIGNMENT OF THE CENTERLINE OF THE BARRIER WALL AND INSTALL OFFSET STAKES FOR FUTURE USE.

POINT	NORTH	EAST	ELEVATION
0	166050.32906	1701094.0067	479.32
1	166061.79308	1701163.6288	482.36
2	166085.38689	1701187.0427	484.29
3	166316.82173	1701244.8567	505.73
4	166457.12756	1700982.6386	482.44
5	166603.31380	1700817.4920	461.82
6	166491.80106	1700738.8584	459.97
7	166422.94294	1700505.0736	454.95
8	166431.15042	1700366.1484	459.04
9	166370.64694	1700362.3601	462.51



SECTION CUT KEY

RECORD DRAWING

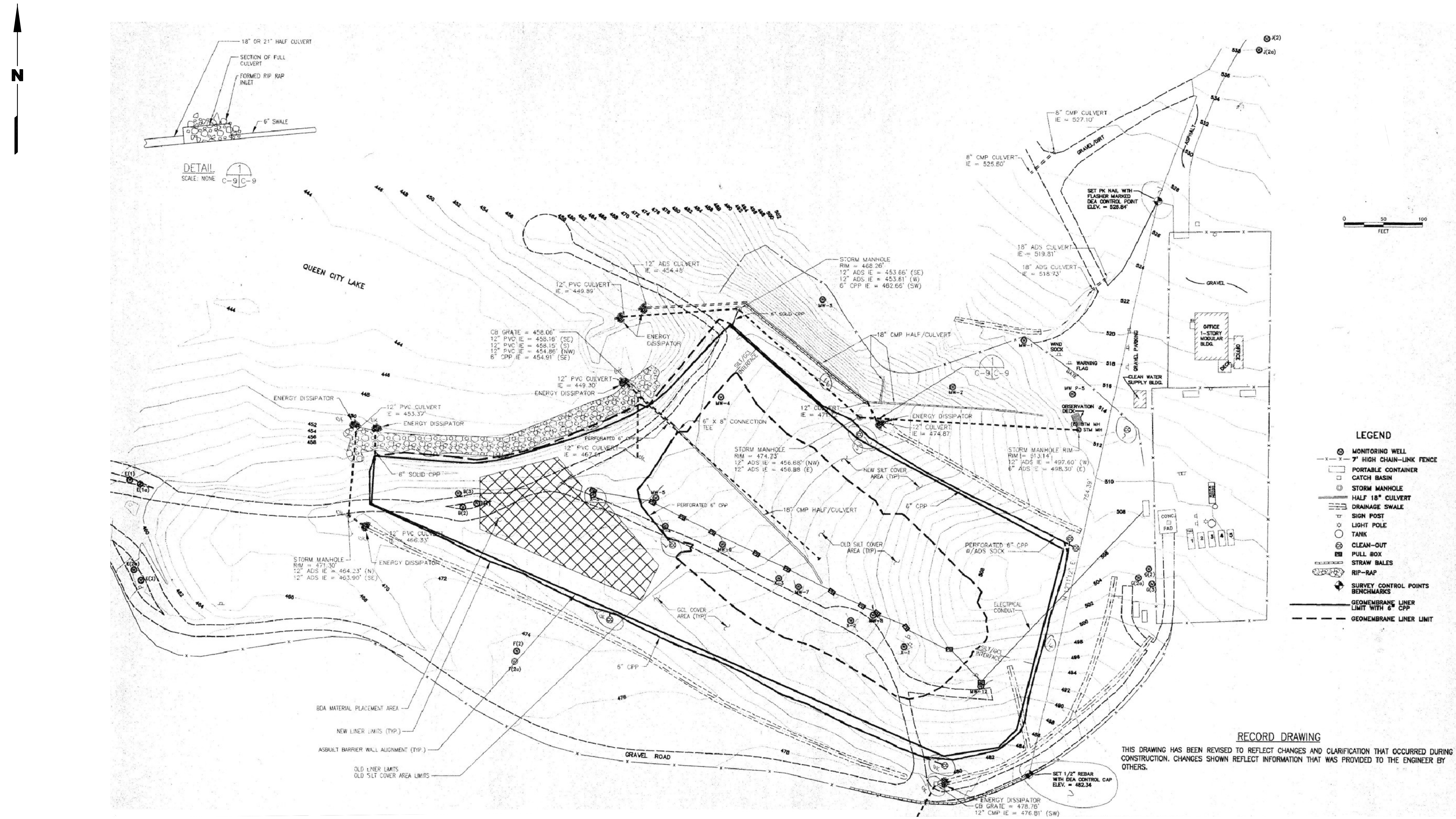
THIS DRAWING HAS BEEN REVISED TO REFLECT CHANGES AND CLARIFICATION THAT OCCURRED DURING CONSTRUCTION. CHANGES SHOWN REFLECT INFORMATION THAT WAS PROVIDED TO THE ENGINEER BY OTHERS.

Not to Scale

Source: Kennedy Jenks Consultants

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**Slurry Wall Alignment**	Figure 2

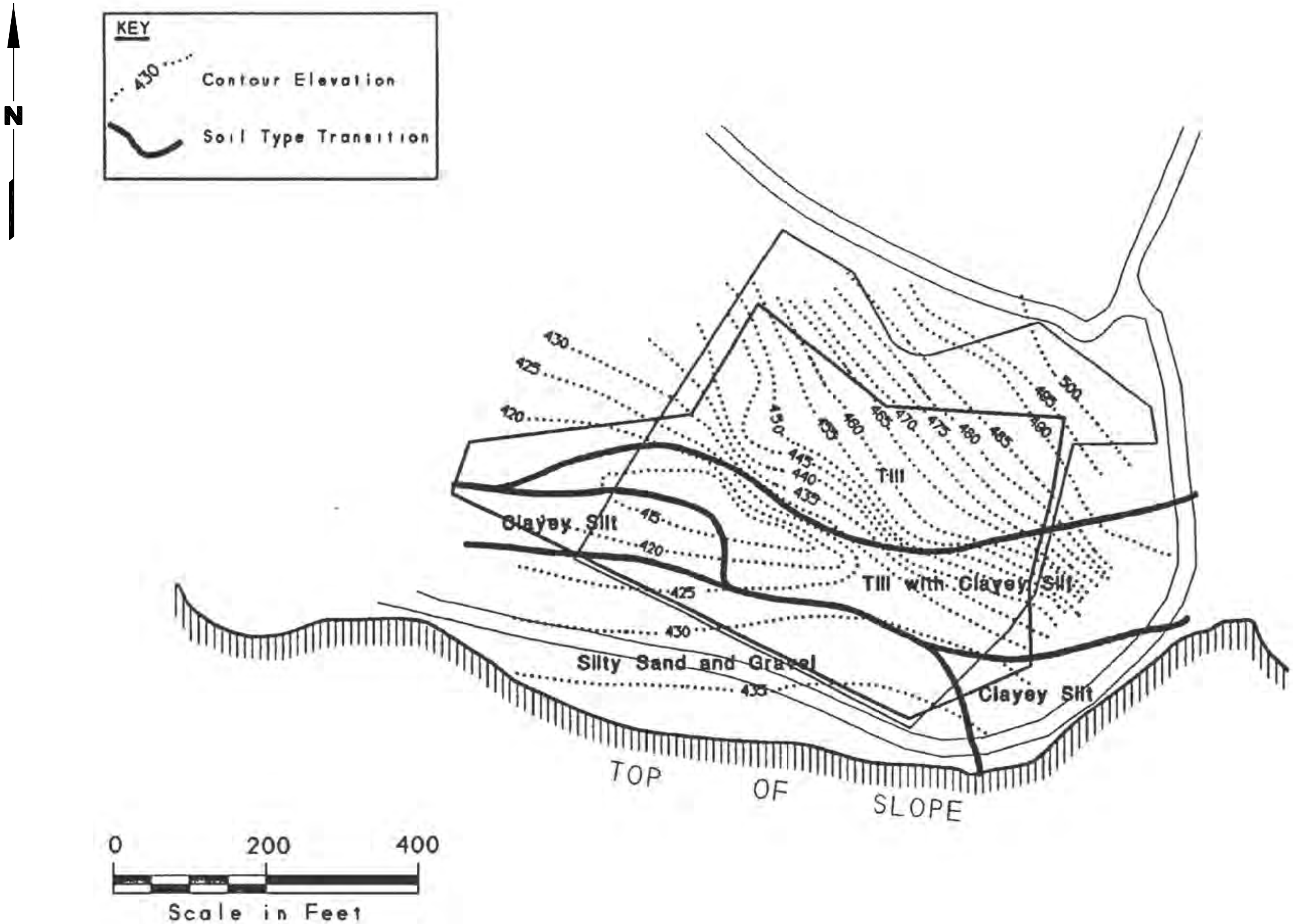


Source: Kennedy Jenks Consultants, 1997

Queen City Farms | I:\DATA\PROJECT\992 QCF\R\Tech Memos\Slurry Wall\CAD\Figure 3.dwg (A) "Figure 3" 12/05/2006

LANDAU ASSOCIATES

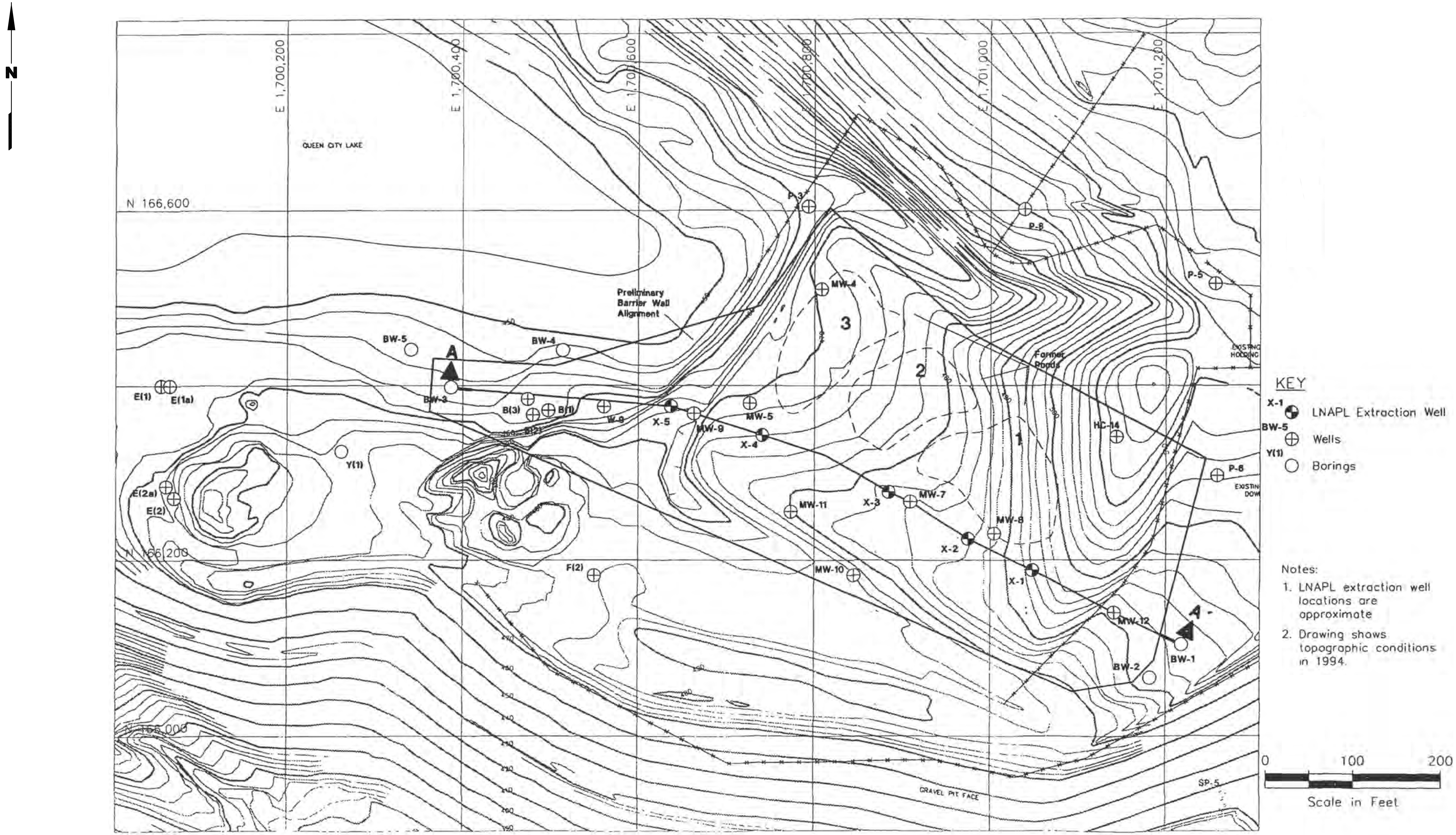
Queen City Farms Refill Project Maple Valley, Washington	**Record Final Grading and Drainage Plan**	Figure 3



Source: Queen City Farms Remediation Project, Landau Associates, 1994

Landau Associates

Queen City Farms Refill Project Maple Valley, Washington	**Upper Surface of the Aquifer 1 Aquitard System**	Figure 4

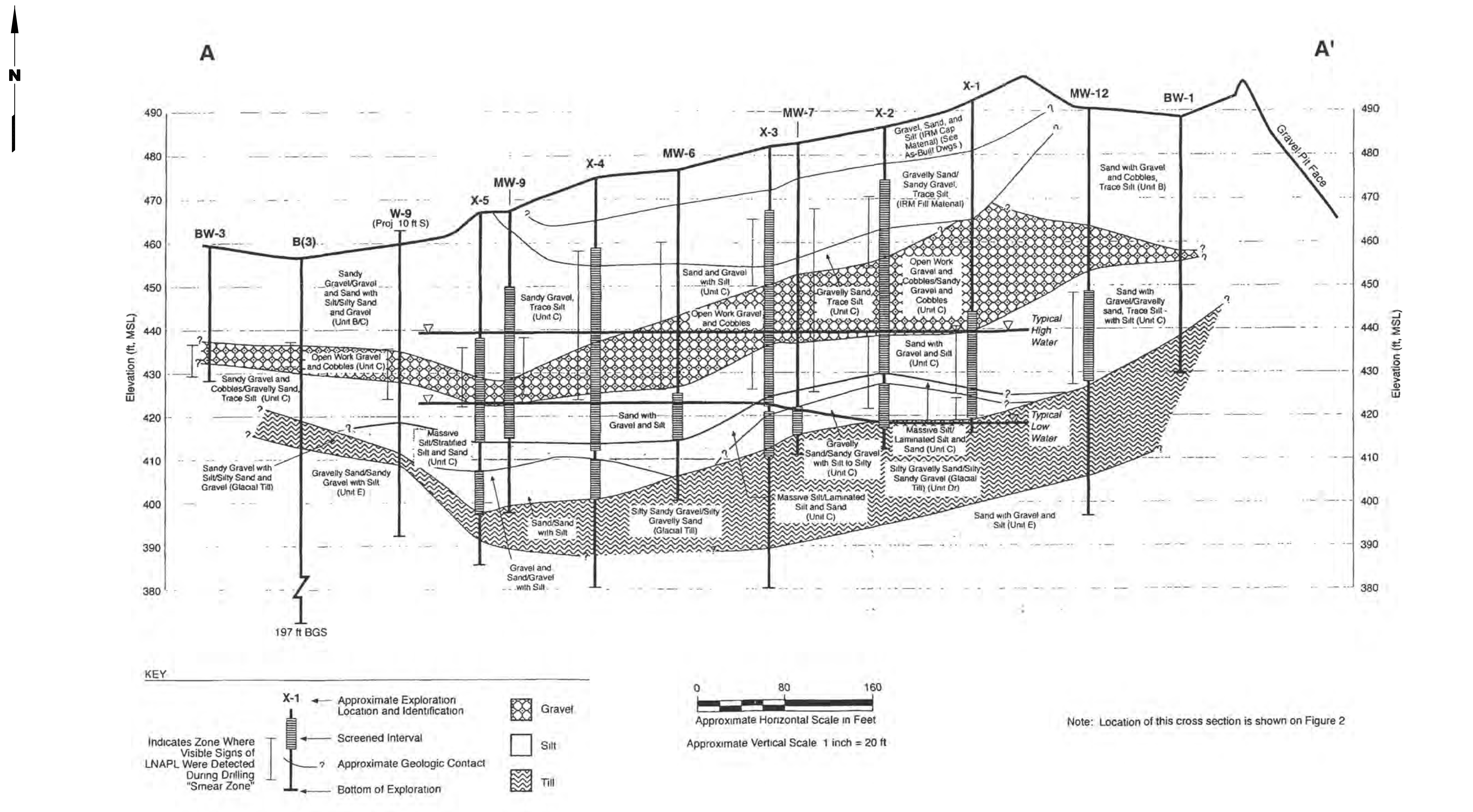


Source: Remedial Action Report, Landau Associates, 1996

LANDAU ASSOCIATES

Queen City Farms Refill Project Maple Valley, Washington	**LNAPL Immobilization Well Locations**	Figure 5

Queen City Farms | I:\DATA\PROJECT\992 QCF\RI\Tech Memos\Slurry Wall\CAD\Figure 5.dwg (A) "Figure 5" 12/05/2006



Note: Location of this cross section is shown on Figure 2

Source: Remedial Action Report, Landau Associates, 1996

Queen City Farms Refill Project Maple Valley, Washington	**Cross Section A-A'**	Figure 6

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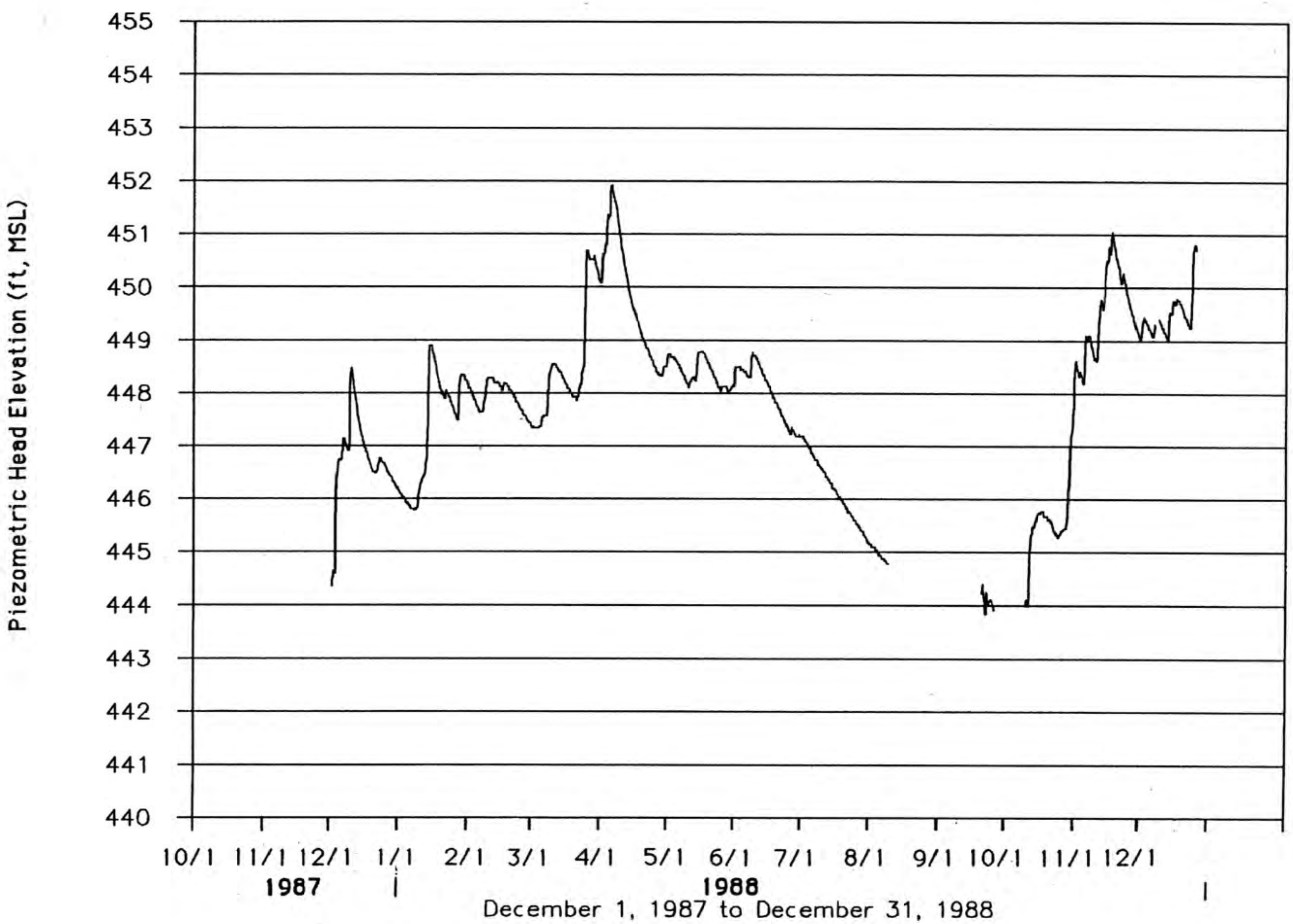
ATTACHMENT 1

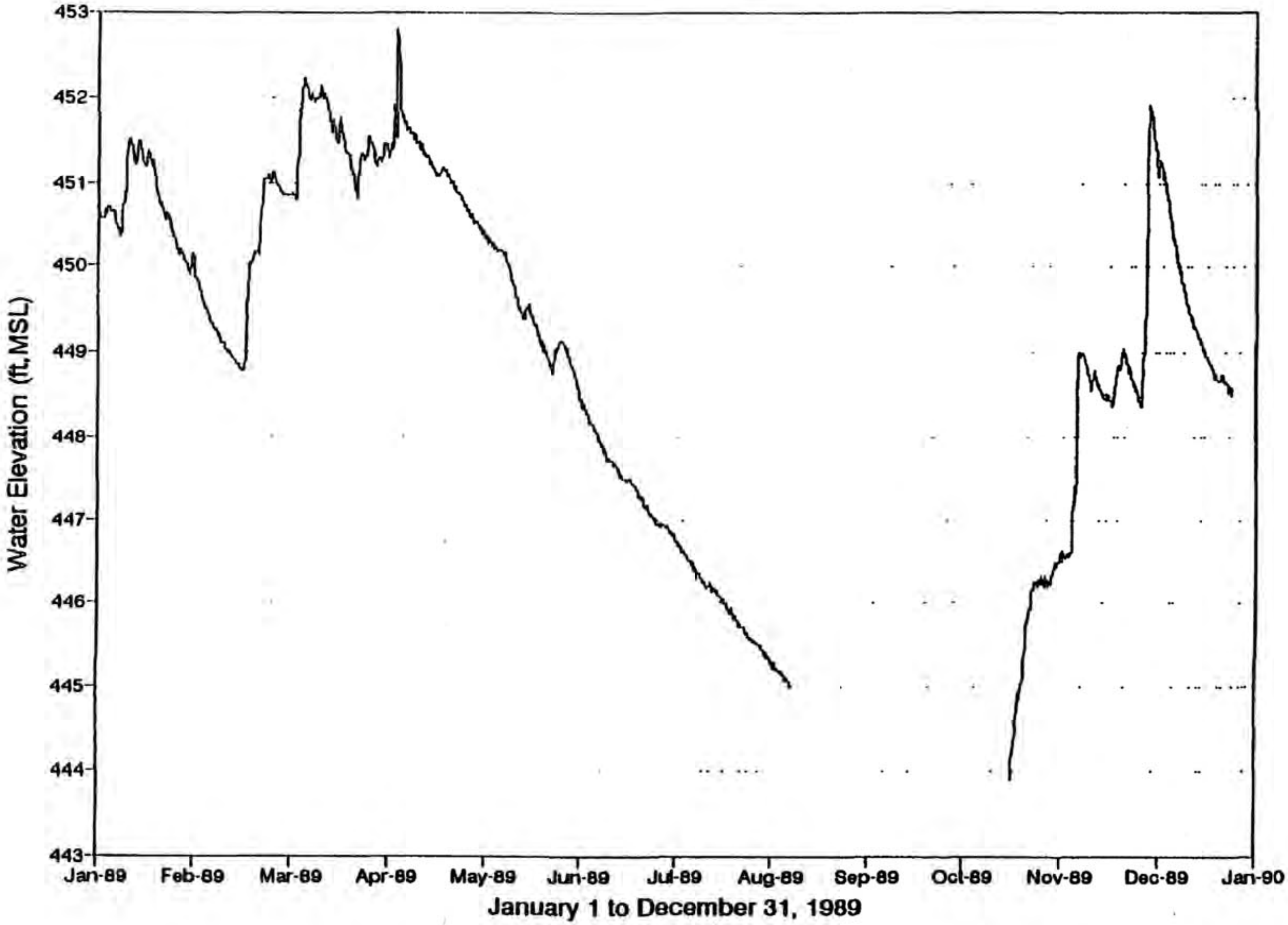
Queen City Lake, 1988 to 1993
Water Levels (NGVD 29)



LANDAU ASSOCIATES, INC.	Semi-Continuous Water Level Data: Queen City Lake December 1, 1987 to December 31, 1988

Figure G-3

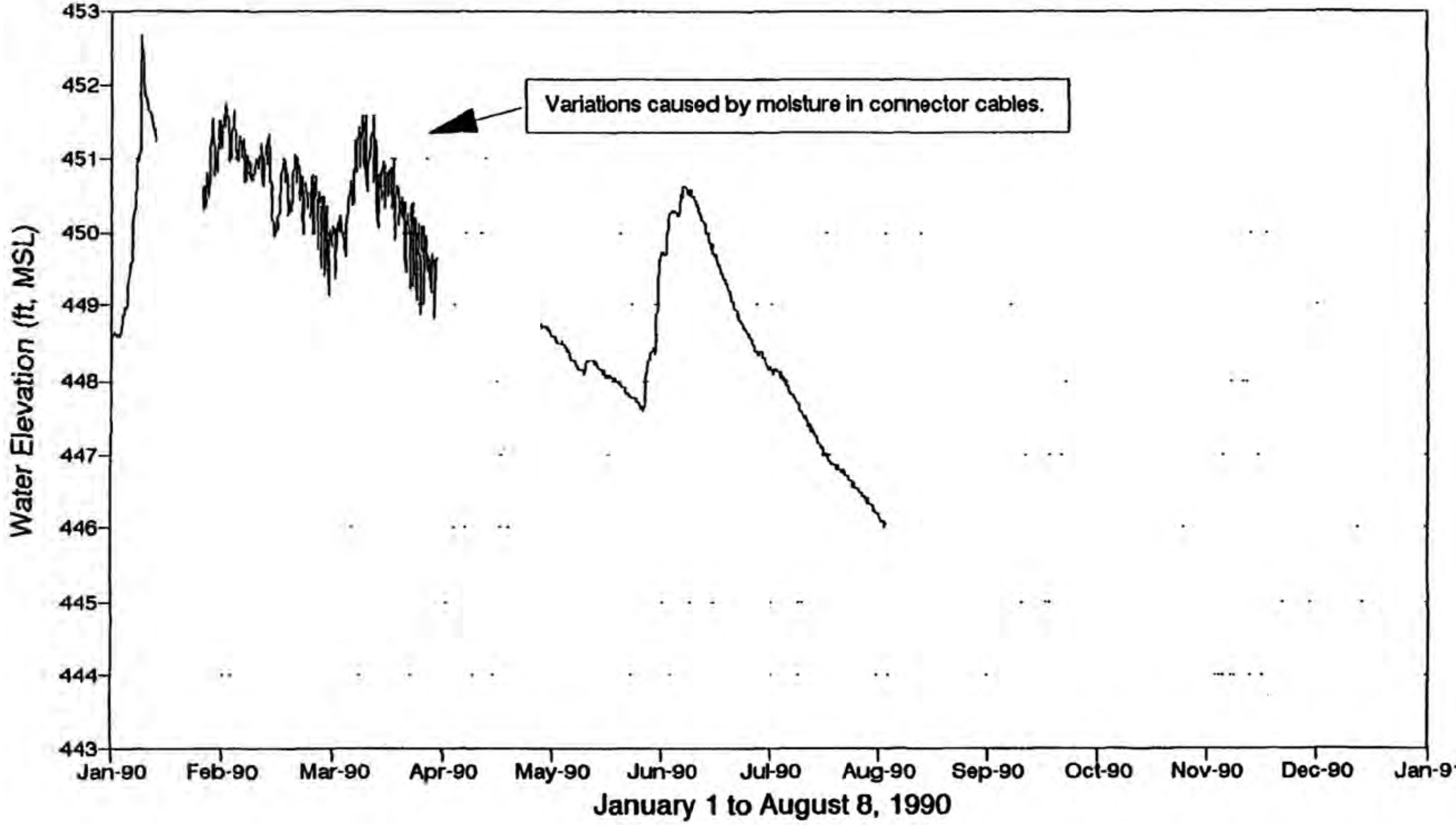
G-6



Note Divisions on horizontal axis correspond to the approximate beginning of each respective month

Semicontinuous Water Level Data: Queen City Lake
January 1 to December 31, 1989

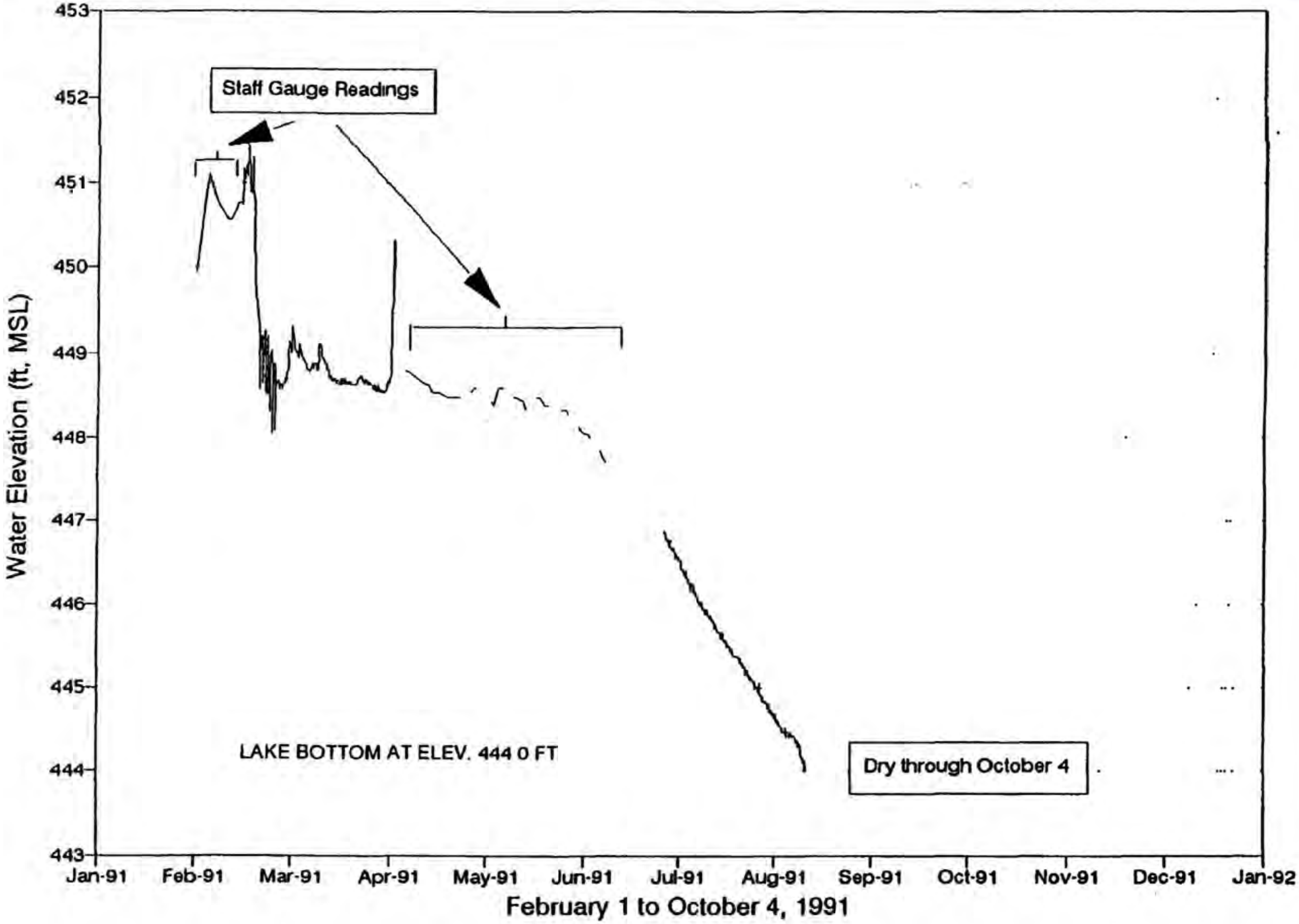
Figure G-6



Note Divisions on horizontal axis correspond to the approximate beginning of each respective month.

Semicontinuous Water Level Data: Queen City Lake
January 1 to August 8, 1990

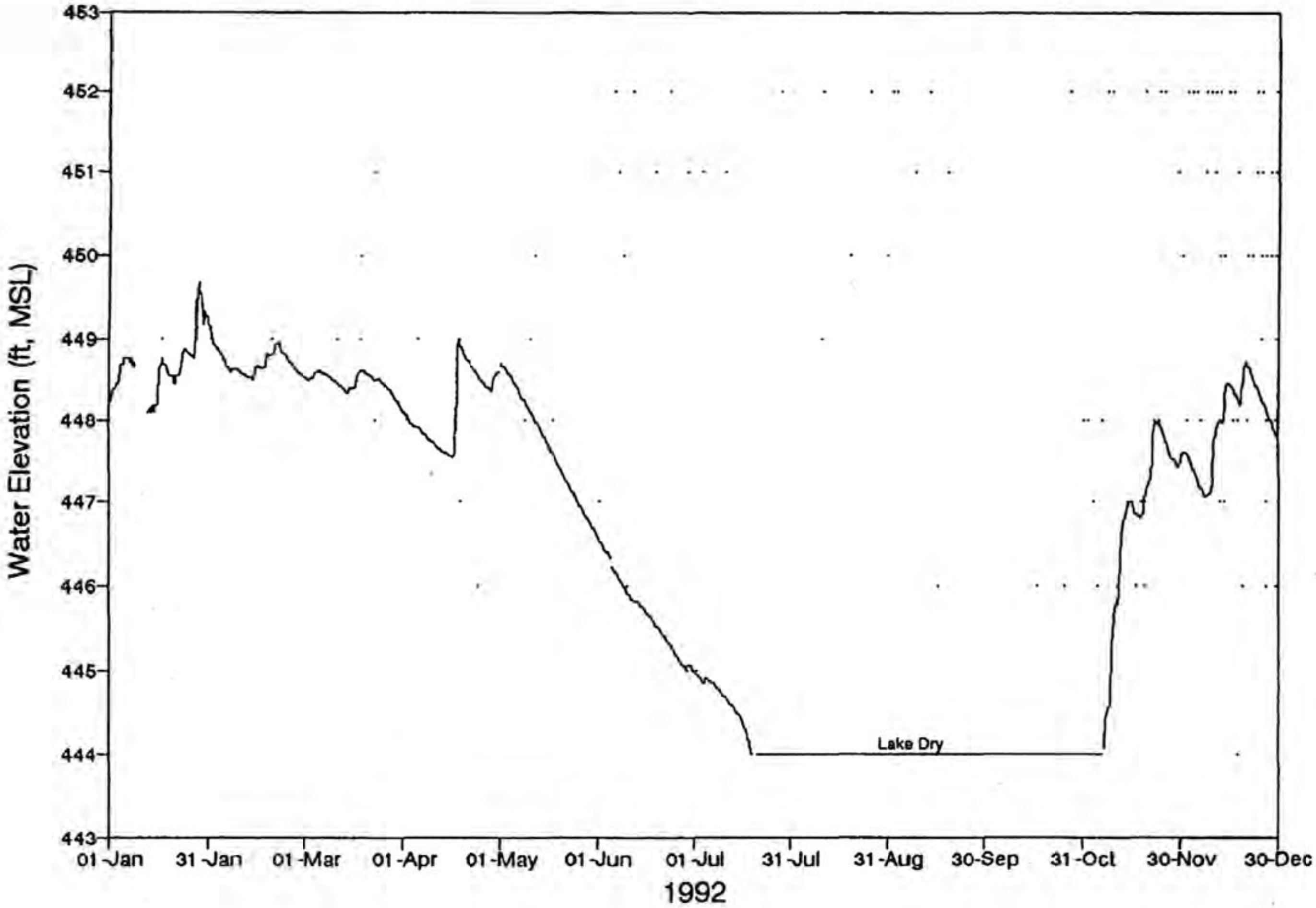
Figure G-7



Note Divisions on horizontal axis correspond to the approximate beginning of each respective month

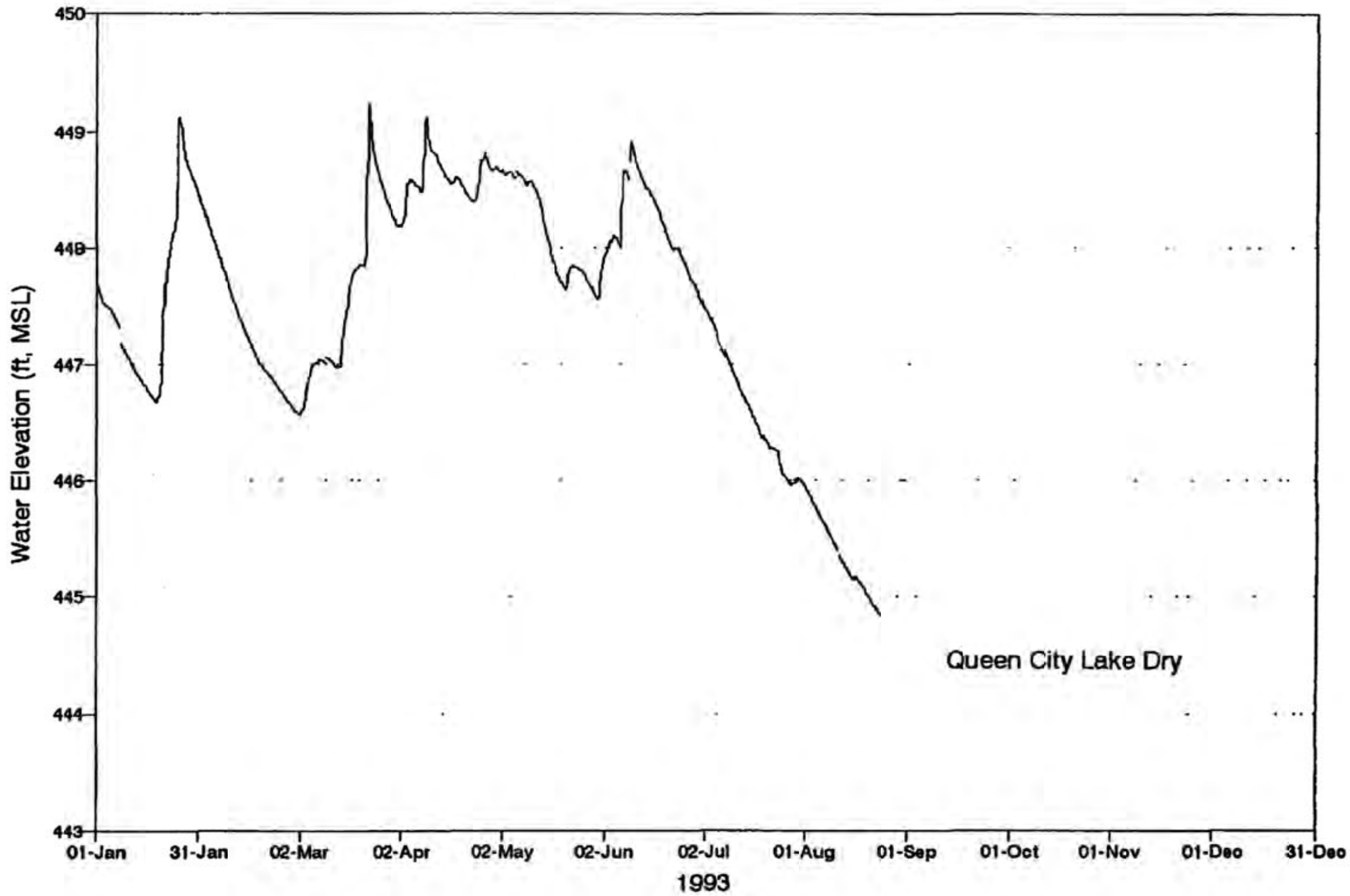
Semicontinuous Water Level Data: Queen City Lake
February 1 to October 4, 1991

Figure G-8



Semicontinuous Water Level Data: Queen City Lake

Figure B-8

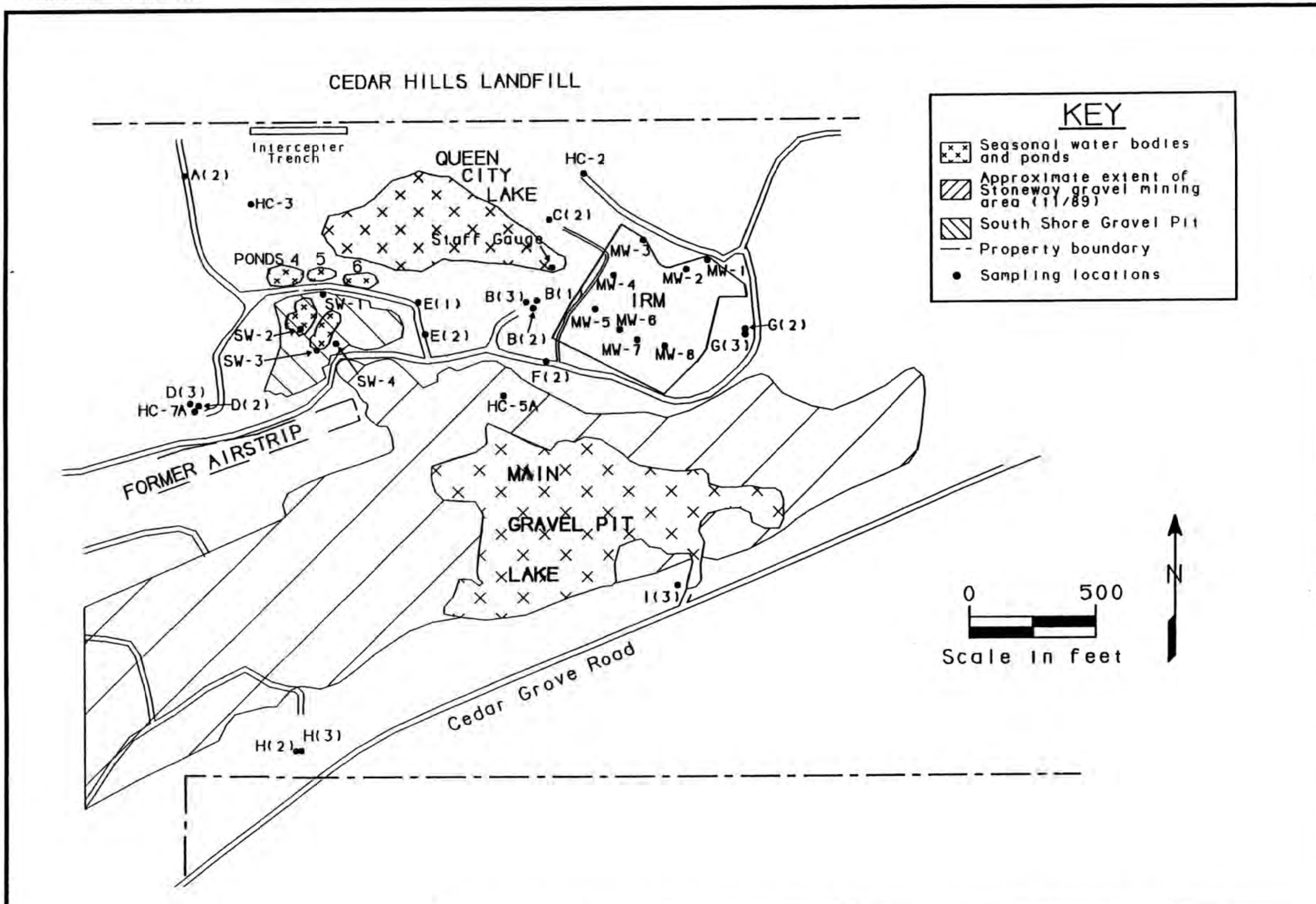


Semicontinuous Water Level Data: Queen City Lake

Figure A-10

ATTACHMENT 2

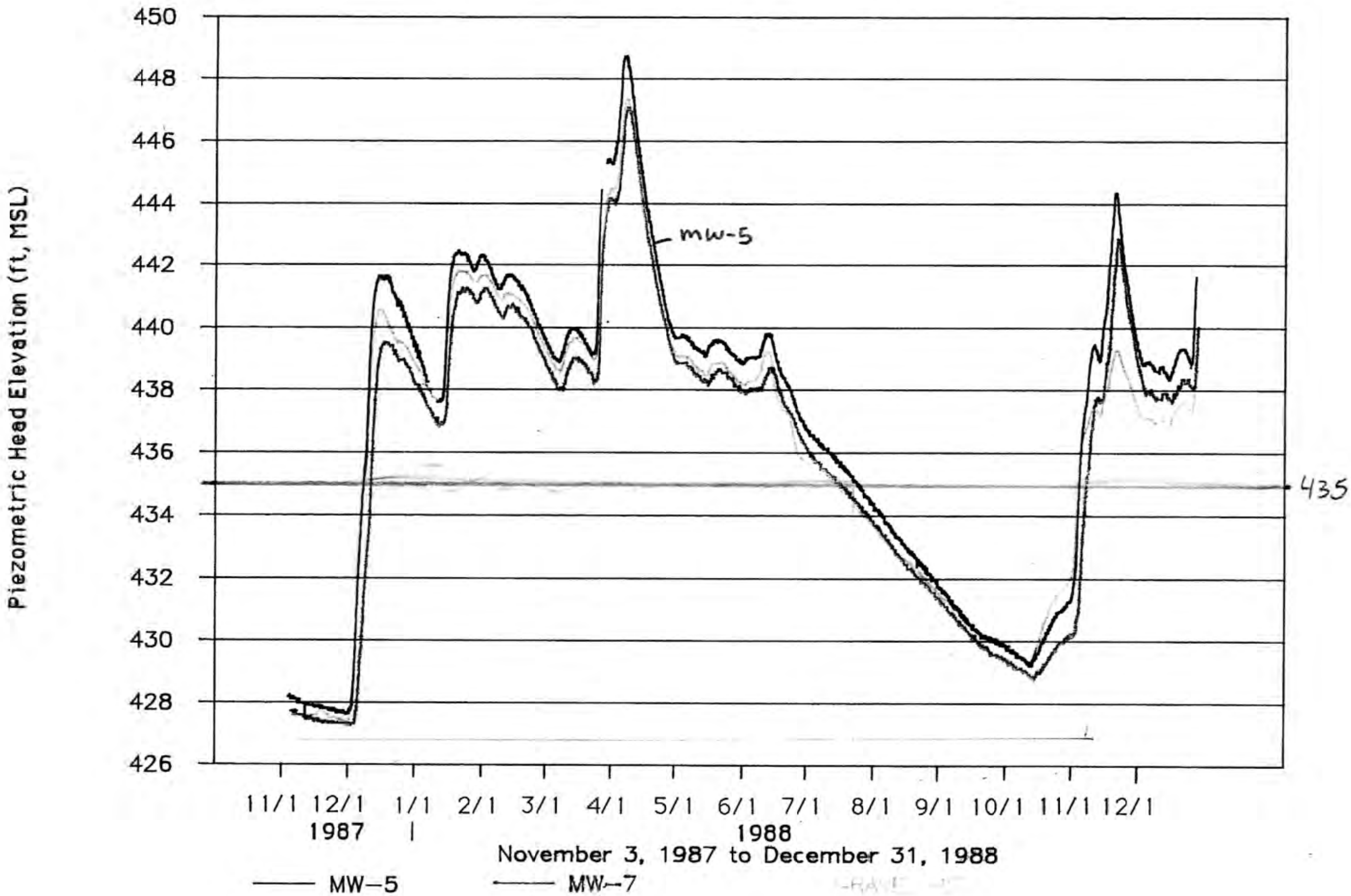
Aquifer 1, 1988 to 1993, Water Levels (NGVD 29)



LANDAU ASSOCIATES, INC.

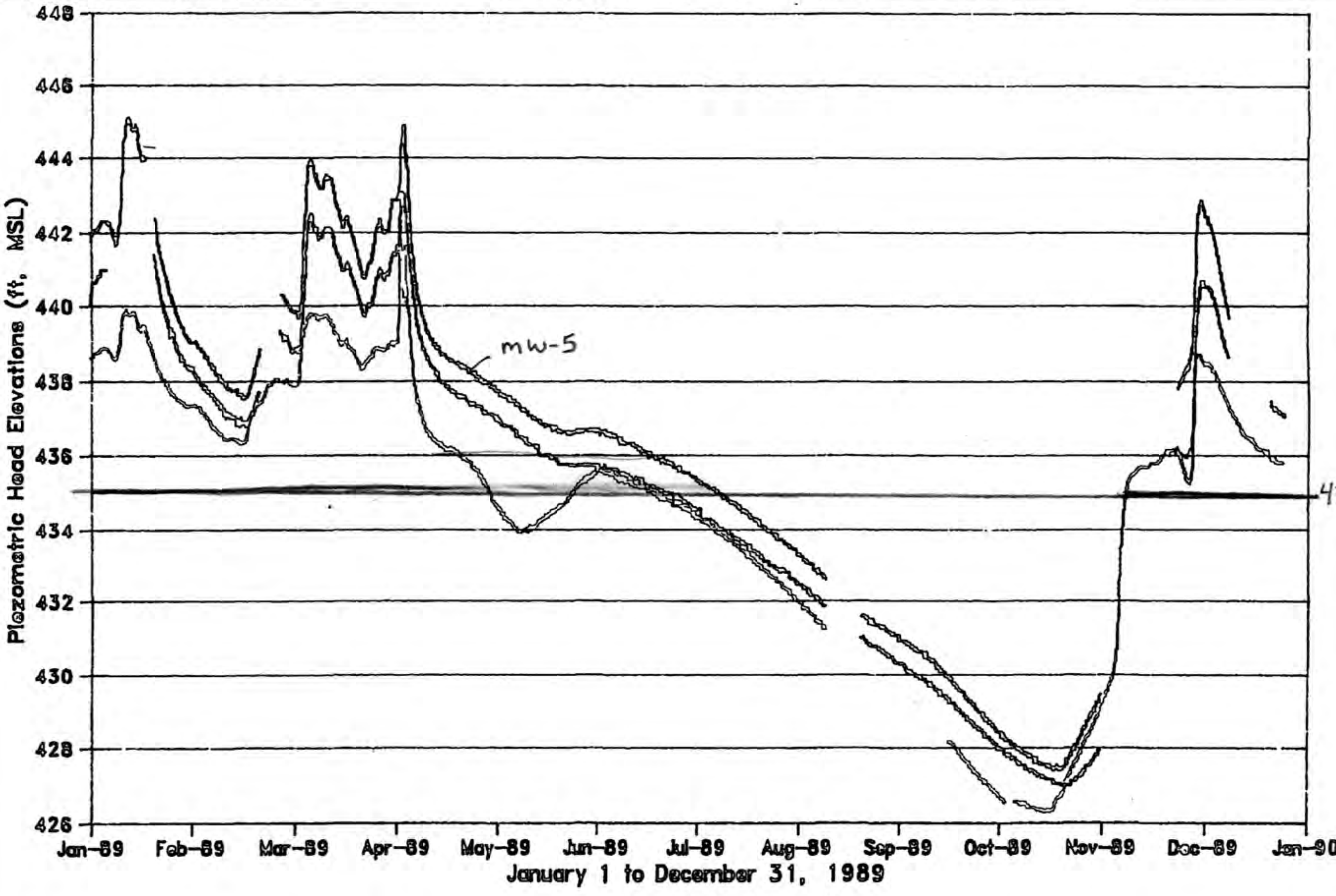
Piezometric Head and Water Level Monitoring Locations on QCF

Figure G-1



LANDAU ASSOCIATES, INC.

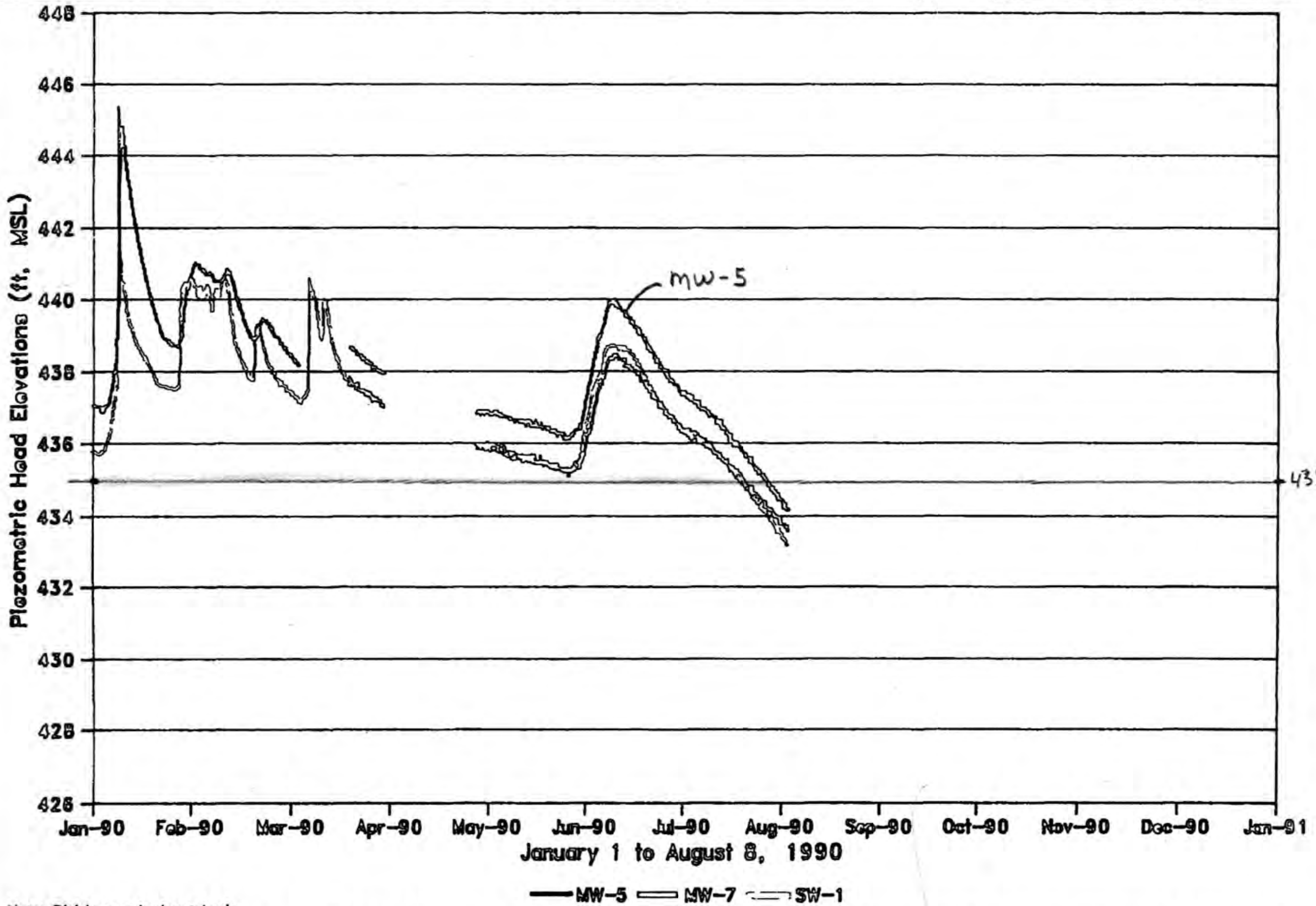
Semi-Continuous Piezometric Head Data: SW-1, Wells MW-5 and MW-7, November 3, 1987 to December 31, 1988



Note: Divisions on horizontal axis correspond to the approximate beginning of each respective month

Semicontinuous Piezometric Head Data: SW-1 and Aquifer 1 Wells MW-5 and MW-7
January 1 to December 31, 1989

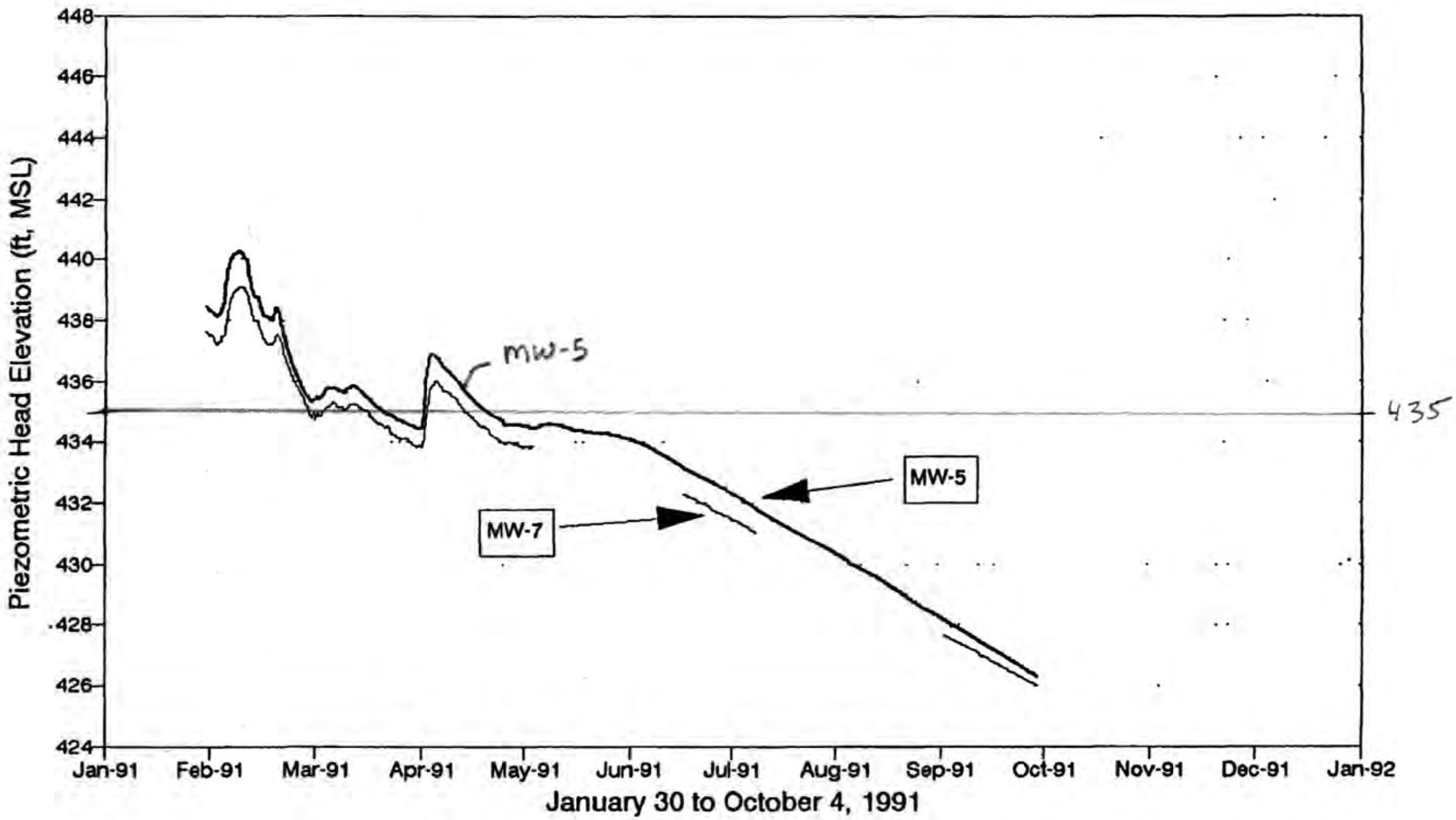
Figure G-9



Note: Divisions on horizontal axis correspond to the approximate beginning of each respective month.

Semicontinuous Piezometric Head Data. SW-1 and Aquifer 1 Wells MW-5 and MW-7
January 1 to August 8, 1990

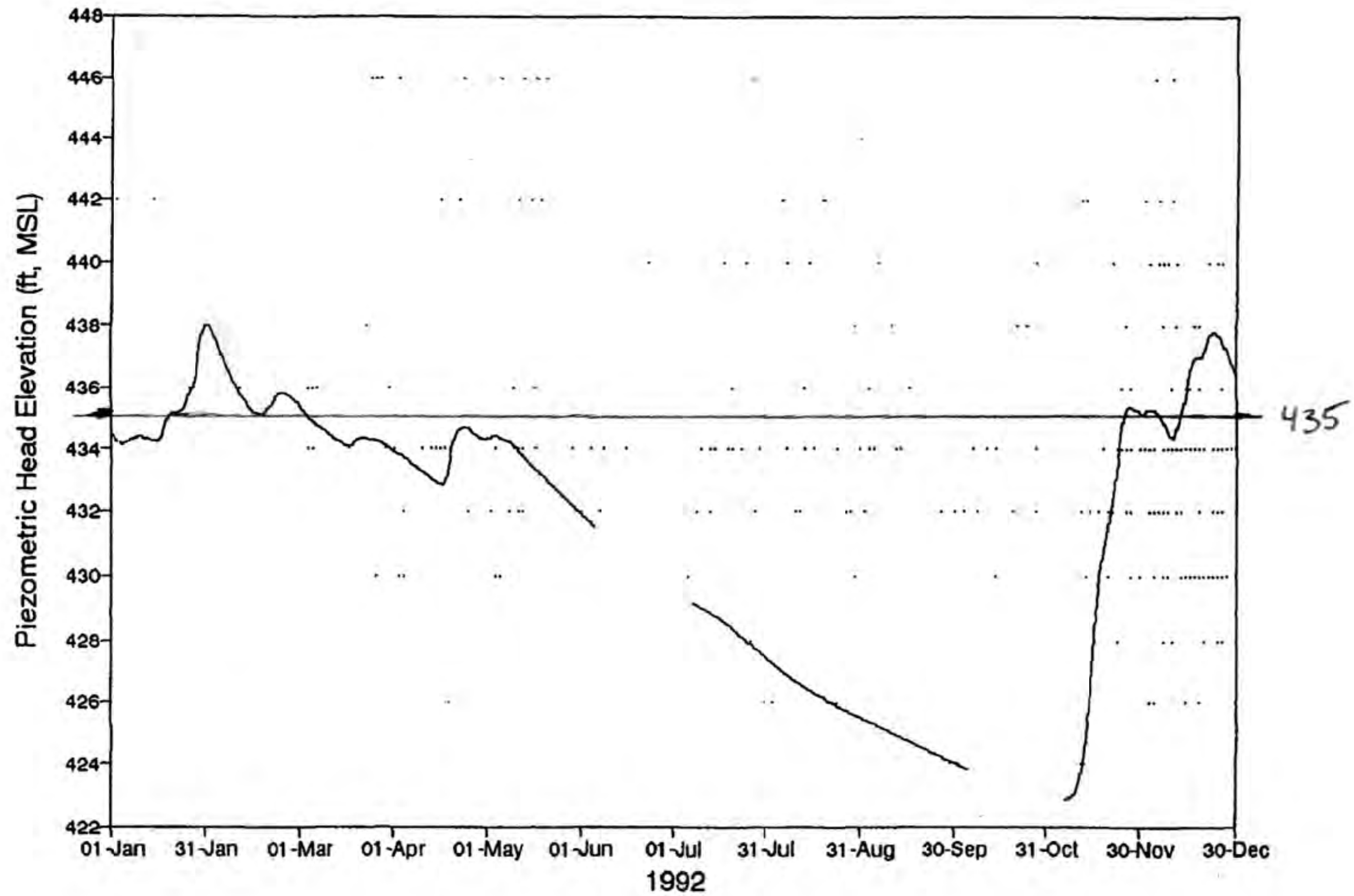
Figure G-10



Note: Divisions on horizontal axis correspond to the approximate beginning of each respective month.

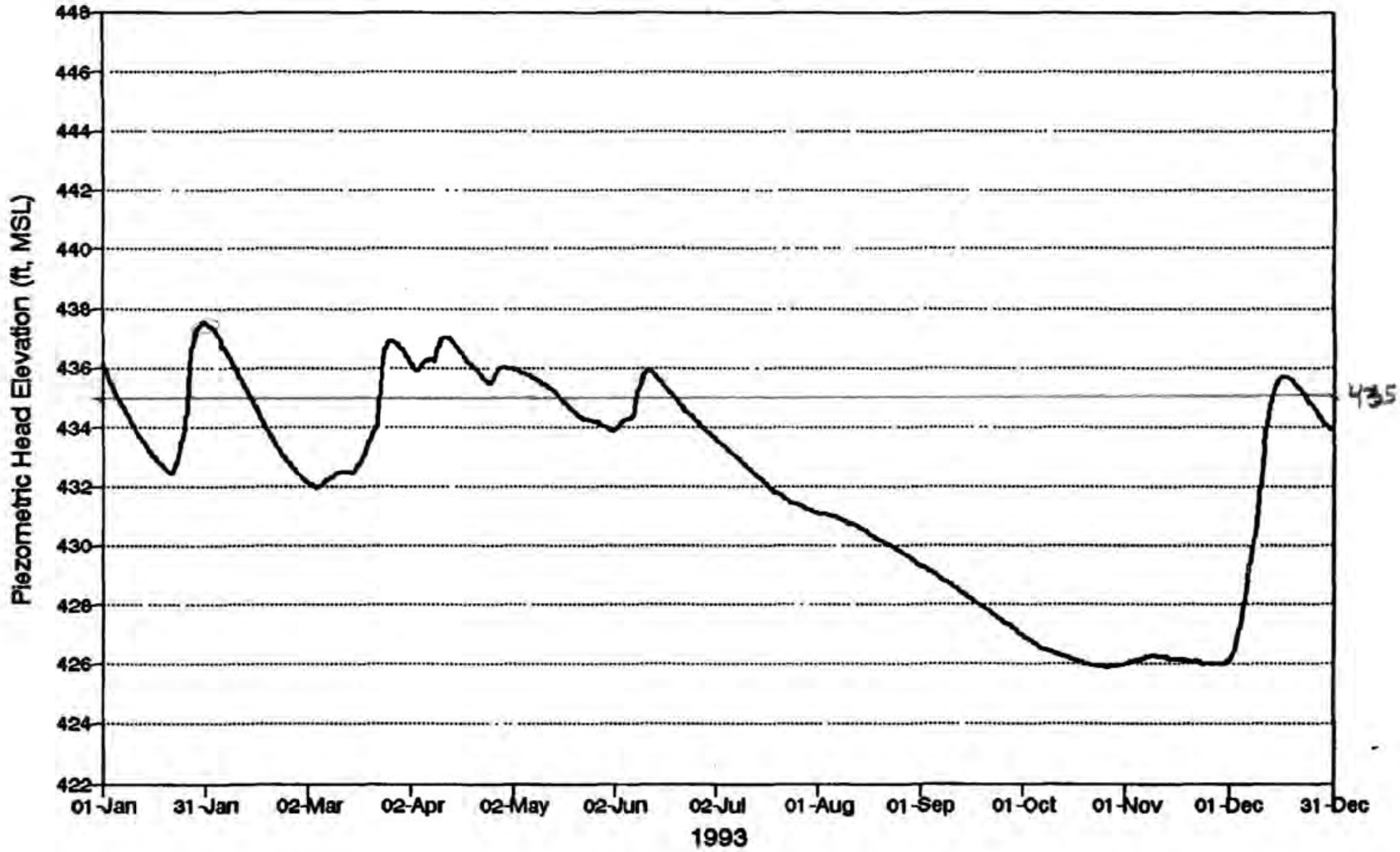
Semicontinuous Piezometric Head Data: Aquifer 1 Wells MW-5 and MW-7
January 30 to October 4, 1991

Figure G-11



Semicontinuous Piezometric Head Data: Aquifer 1, Well MW-5

Figure B-1



Semicontinuous Piezometric Head Data: Aquifer 1, Well MW-5

Figure A-1

ATTACHMENT 3

Aquifer 1, 1996-2005,Water Levels (NGVD 29)

Figure A-2

Aquifer 1 Outside the IRM

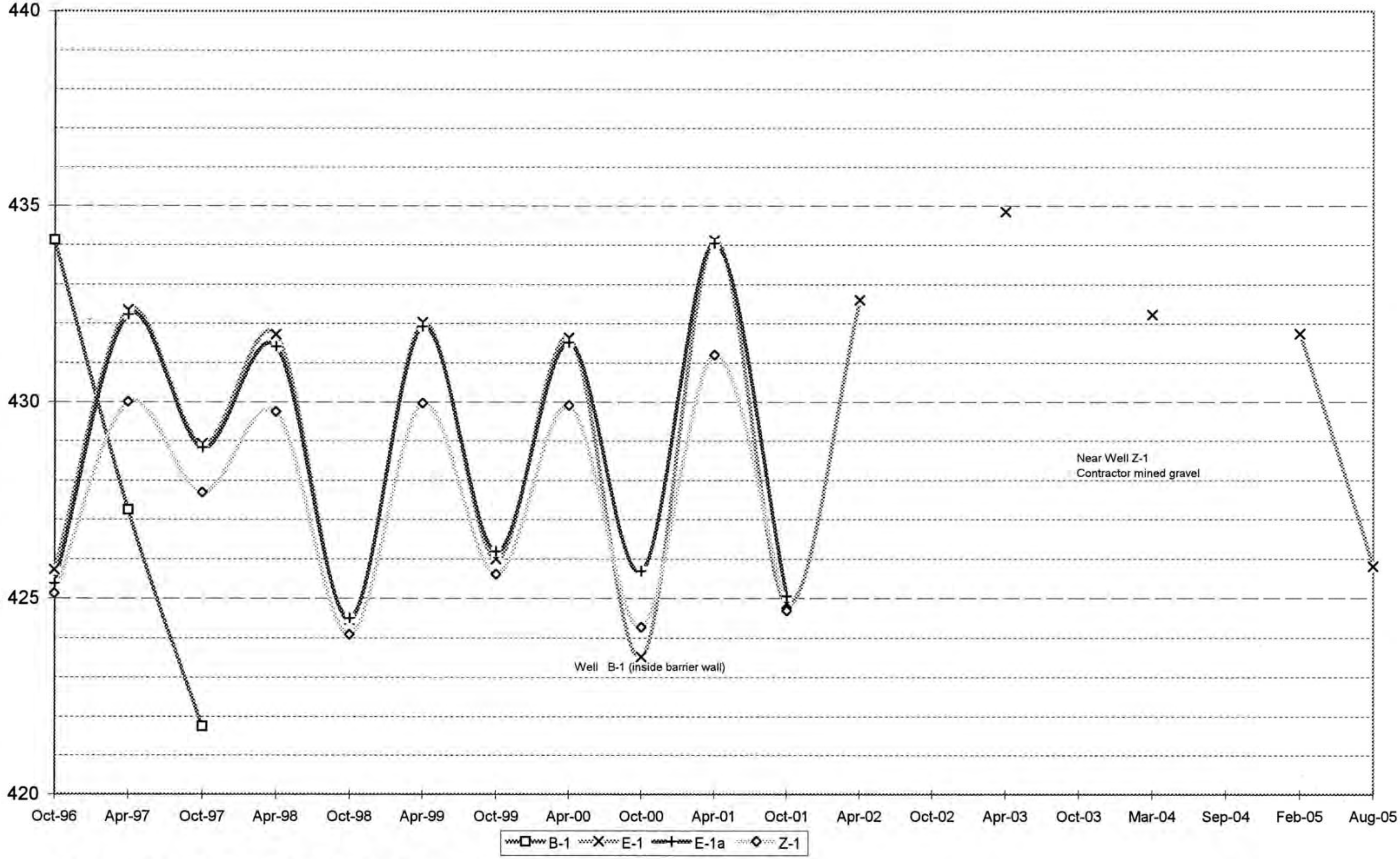
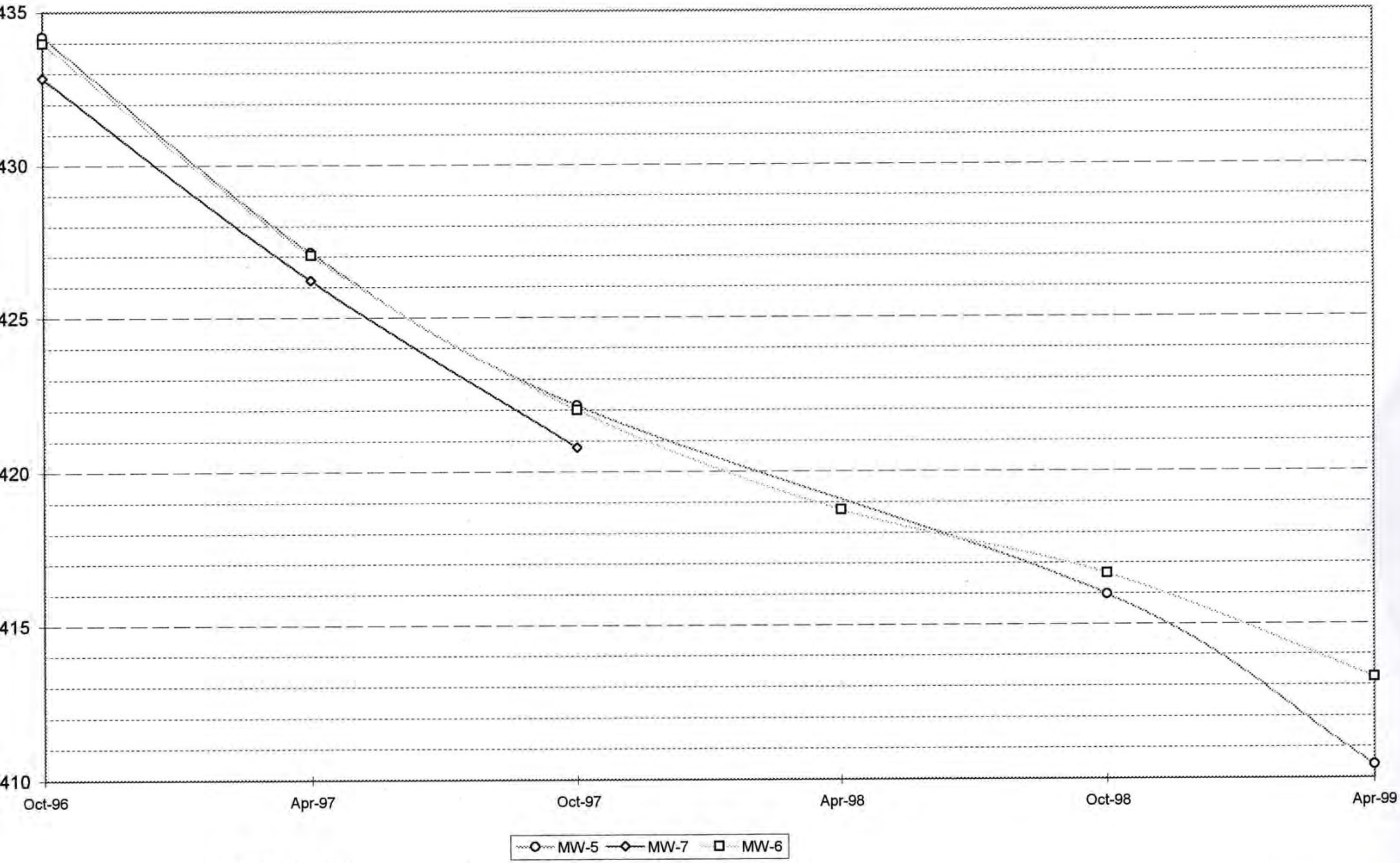


Figure A-3

Aquifer 1 Wells inside the IRM



TECHNICAL MEMORANDUM

LANDAU ASSOCIATES
ENVIRONMENTAL | GEOTECHNICAL | NATURAL RESOURCES

TO: Mr. Alan Wallace, Williams Kastner and Gibbs

FROM: Edward J. Heavey, P.E. and Eric Weber, L.G.

DATE: January 25, 2007

RE: SURFACE WATER INFILTRATION EVALUATION
QUEEN CITY FARMS REFILL PROJECT
MAPLE VALLEY, WASHINGTON

This technical memorandum presents geotechnical evaluations for infiltration of surface water at the Queen City Farms (QCF) Refill Project site. The purpose of this study was to review readily available geologic and geotechnical information in the project area, complete additional subsurface explorations to further characterize subsurface conditions, complete field tests to determine *in situ* infiltration rates, perform limited geotechnical laboratory testing to determine the gradation characteristics of the site soil, and develop geotechnical conclusions and recommendations for infiltration of surface water runoff at the site. The general project location is shown on the Vicinity Map, Figure 1. Figure 2 shows the approximate location of explorations completed for this study.

PROJECT BACKGROUND

The QCF gravel pit is a former sand and gravel mine that is currently occupied by a seasonal lake called the Main Gravel Pit Lake. Prior to mining, the lake did not exist. The refill plan for the gravel pit indicates that up to approximately 120 ft of fill will be placed to achieve final grades once the gravel pit refill is complete. Refilling will eliminate the seasonal Main Gravel Pit Lake. The other main surface water feature on the site is Queen City Lake, located on the north portion of the property. Review of aerial photographs back to 1936 and United States Geological Survey (USGS) maps indicate that Queen City Lake has never had a natural surface water outlet (i.e., all the water in the lake infiltrated into the underlying Aquifer 1). In February 1991, a 36-inch pipe was installed from Queen City Lake to a ravine that discharged to the Main Gravel Pit Lake. The pipe was installed as an emergency erosion control device. The pipe effectively limited the fluctuations in the lake to approximately 5.5 ft.

Other surface water features onsite include the QCF spring located southwest of the Main Gravel Pit Lake and a set of springs, known as the East Airstrip Springs, that were exposed in the north face of the gravel pit during mining. The discharge from the QCF spring flows into a road side ditch along Cedar Grove Road SE; part of this discharge flows through a culvert under the road and into the adjacent wetland. The discharge from the East Air Strip springs flowed down the face of the gravel pit into the

Main Gravel Pit Lake at rates as high as 1,000 gpm (2.2 cfs). The location of the seasonal Main Gravel Pit Lake, Queen City Lake, Queen City Farms Spring, and East Airstrip Springs are shown on Figure 2.

The refill plan calls for refilling the depression that contains the Main Gravel Pit Lake. Consequently, the storage and infiltration function associated with this lake will be removed. This function will be replaced by the following refill design features:

- The current outflow for Queen City Lake will be modified to allow the lake to fluctuate over a wider range. This will increase storage and infiltration associated with the lake. Lake levels will be lower than historical levels (pre 1991) but higher than current levels.

- East Airstrip Spring discharge is expected to increase somewhat as water levels in Queen City Lake increase. The spring discharge will be collected and discharged to an infiltration gallery along the north slope of the pit face. The estimated future maximum flow from the East Airstrip Springs is 500 gpm (1.1 cfs).

- The East Stormwater Facility will be constructed east of the Main Gravel Pit Lake. The modified Queen City Lake outflow will be discharged to the East Stormwater Facility infiltration pond (also known as the East Stormwater Retention Pond), shown on Figure 2. Some water collected in the infiltration pond will discharge to the infiltration/conveyance channel along Cedar Grove Road SE being constructed by King County Solid Waste Division (King County Department of Transportation 2006) and some water will discharge directly to the sediments and soil below the pond.

- The South Stormwater Facility will be constructed south of the gravel pit refill area. The South Stormwater Facility infiltration pond (referred to as the South Stormwater Retention Pond) will collect and infiltrate runoff from the reclaimed area. Some of the water in the pond will discharge to the Main Infiltration Area and some water will discharge directly to the sediments and soil below the pond. The planned South Stormwater Retention pond location is shown on Figure 2.

- The West Stormwater Facility will be constructed southwest of the gravel pit refill area. The West Stormwater Facility detention pond (referred to as the West Stormwater Detention Pond) will be constructed to collect runoff from the reclaimed area. Water collected in the detention pond will discharge to the Main Infiltration Area. The planned location for the West Stormwater Detention pond is shown on Figure 2.

EXISTING CONDITIONS

The following sections provide a summary of the existing surface and subsurface soil and groundwater conditions at the site. Our understanding of the surface and subsurface soil and groundwater conditions are based on review of explorations completed by others and test pits excavated by Landau Associates on November 2 and 9, 2006.

Surface Conditions

The gravel pit consists of a roughly triangular-shaped depression south of Queen City Lake. The depression is about 1,600 ft long (E-W) and varies in width from about 200 to 800 ft. A moderately steep slope is present along the northern edge and west edge of the depression. The slope has an average gradient of about 40 percent and is a remnant of the mining operation. The western portion of the slope is generally irregular and was created by refilling of the area to the west of the gravel pit. Overall slope gradients vary from about 35 percent at the north end to about 12 percent at the south end. A berm, about 15 ft in height, is present along the southern boundary of the gravel pit. The berm was likely created during mining operations.

The floor of the gravel pit is sparsely vegetated due to seasonal flooding. The berm to the south and the area to the east are generally vegetated with small alder and scattered scotch broom. The slopes along the north and west sides are heavily vegetated with scotch broom and grasses.

Soil Conditions

Soil conditions in the planned Main Infiltration Area, the planned South Stormwater Retention pond location, along the north slope of the gravel pit, along the planned bioswale location, and at the planned East Stormwater Retention pond were evaluated by excavating test pits in these locations. Soil conditions were also evaluated by collecting soil samples at these locations and performing gradation tests. The soil conditions for the test pits and soil conditions based on the gradation tests are described below.

Field explorations completed for this study consist of obtaining nine surface samples of soil on the north slope of the gravel pit, excavating 13 test pits in areas where infiltration facilities are planned, and completing two *in situ* infiltration tests. Surface samples from the north slope of the gravel pit (S-1 through S-9) were obtained on October 4, 2006 by excavating shallow, hand-dug test pits. The approximate locations of where the surface samples were obtained are shown on Figure 2. Test pits TP-1 through TP-13 were excavated in the vicinity of the Main Infiltration Area, the planned South Stormwater Retention pond location, along the planned bioswale location, and at the planned East Stormwater Retention pond location. *In situ* infiltration tests were completed at the planned South Stormwater Retention Pond location (Infiltration Test 1) and at the planned East Stormwater Retention pond location (Infiltration Test 2). The approximate locations of the explorations are shown on Figure 2.

The field exploration program was monitored by a geologist from Landau Associates who also obtained representative soil samples, maintained a detailed record of the observed subsurface soil and groundwater conditions, and described the soil encountered by visual and textural examination. Each representative soil type observed in the test pits was described using the soil classification system shown on Figure 3, in general accordance with ASTM D2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure)*. Logs of the exploratory test pits are presented on Figures 4 through 10. These logs represent our interpretation of subsurface conditions identified during the field exploration program.

Disturbed bulk samples of the soil encountered in the test pits were obtained at frequent intervals. Samples obtained from the test pits were taken to our laboratory for further examination and testing. The laboratory testing program, which was performed in general accordance with the ASTM standard test procedures, consisted of visual inspection of the soil samples to confirm our field soil descriptions, and natural moisture content and grain size determinations on selected samples.

The natural moisture content was determined in general accordance with ASTM D2216 test procedures. The results of the natural moisture content determinations are indicated adjacent to the corresponding samples on the summary logs of test pits[1] (Figures 4 through 10). The grain size distribution was determined in general accordance with ASTM D422 test procedures. The results are presented in the form of grain size distribution curves on Figures 11 through 15.

North Slope Area

Subsurface conditions observed along the north slope of the gravel pit generally consist of outwash deposits (i.e., Units C and E as defined in Landau Associates 1990). The outwash deposits observed at the explorations generally consisted of very sandy gravel with silt to trace silt. The outwash deposits described for borings completed previously (Landau Associates 1990, 1991) in the north slope area generally describe the outwash deposits as a stratified deposit of sandy to very sandy gravel with variable silt content and occasional interbeds of sand to silty sand. In the north slope area, the outwash deposits reportedly extend to about elevation 350 ft (NGVD 29) (Landau Associates 1990). Groundwater levels in the north slope area generally mirror water levels in the adjacent the Main Gravel Pit Lake, but are somewhat lower.

Main Infiltration Area

Soil conditions observed in the test pits excavated in the Main Infiltration Area (TP-8 and TP-9) generally consisted of about 1 ft of surficial fill over recessional outwash deposits. The fill material consisted of silty sand with variable gravel content. The recessional outwash was observed to consist of 2 to 3 ft of open work gravel overlying very sandy gravel, which extends to the depth explored [about 7 ft

[1] Logs were not prepared for surface soil samples S-1 through S-9).

below ground surface (BGS) in TP-8 and about 6.5 ft BGS in TP-9]. Groundwater was not encountered in these test pits.

South Stormwater Retention Pond Area

Soil conditions observed in the test pits excavated in the South Stormwater Retention Pond area (TP-6, TP-7, and TP-10) generally consisted of fill over recessional outwash deposits, except at test pit TP-6, which was excavated at the far eastern edge of the pond.

At test pit TP-6, fill was encountered to the depth explored, about 6 ft BGS. The fill consisted of about 4 ft of silty, very gravelly, fine to medium sand and about 2 ft of clayey silt with gravel. At test pit TP-7, about 2 ft of fill consisting of silty sandy gravel was encountered. At test pit TP-10, about 6 ft of fill consisting of gravelly, silty, fine to medium sand and silty, sandy gravel with cobbles was encountered.

Outwash deposits were encountered in test pit TP-7 between a depth of about 2 and 7 ft BGS and consisted of very gravelly, fine to coarse sand with cobbles and very sandy gravel with cobbles and boulders. At test pit TP-10, outwash deposits were encountered between a depth of about 6 and 8.5 ft BGS and consisted of very sandy gravel with trace silt.

Pre-Vashon deposits (i.e., Unit F as defined in Landau Associates 1990) were encountered in test pit TP-7 below a depth of about 7 ft BGS and extended to the depth explored, about 8.5 ft BGS. The Pre-Vashon deposits observed consisted of fine to medium sand with silt.

Moderate groundwater seepage was encountered in test pit TP-6 at a depth of about 5 ft BGS. Slight groundwater seepage was encountered in test pit TP-10 at depths of about 4 and 5 ft BGS. Groundwater was not encountered in the test pit TP-7.

East Stormwater Retention Pond Area

Soil conditions observed in the test pits excavated in the East Stormwater Retention Pond area (TP-11 through and TP-13) generally consisted of fill overlying outwash deposits. The fill generally consisted of silty, fine to medium sand with variable amounts of gravel. The fill was extended to a depth of about 3 ft BGS in test pits TP-11 and TP-13, and to a depth of about 2 ft BGS in test pit TP-12. The outwash deposits that were observed in these test pits generally consisted of gravel with variable amounts of sand. The outwash deposits extended to the depth of each test pit, about 6 ft BGS in TP-11 and TP-12, and about 7 ft BGS in TP-13. Groundwater was not encountered in any of these test pits.

Bioswale Area

Soil conditions observed in the test pits excavated along the length of the bioswale (TP-1 through and TP-5) generally consisted of fill with the exception of test pit TP-4, where recessional outwash deposits were encountered beneath the fill. The fill observed at each test pit was of variable composition. At test pit TP-1, the fill consisted of sandy, clayey silt and extended the full depth of the test pit, about 4 ft BGS. At test pit TP-2, the fill consisted of silty, sandy gravel and graded to very sandy gravel with depth. The fill at test pit TP-2 extended the full depth of the test pit, about 4 ft BGS. At test pit TP-3, the fill consisted of silty sandy gravel and graded to very sandy gravel with silt with depth. The fill at TP-3 extended the full depth of the test pit, about 5 ft BGS. At test pit TP-4, the fill consisted of silty, very gravelly sand and very sandy gravel with silt and cobbles and extended to about 8 ft BGS. At TP-5, the fill consisted of silty, fine to medium sand with variable amounts of gravel and extended the full depth of the test pit, about 6 ft BGS. Recessional outwash deposits consisting of very sandy gravel with silt were encountered beneath the fill in test pit TP-4. The recessional deposits extend from 8 ft BGS to the full depth of test pit TP-4, about 9 ft BGS. Groundwater was not encountered in any of these test pits.

Gradation Testing

The long-term infiltration rate can be correlated to D_{10} size (in mm) obtained from the ASTM D422 gradation test (Massmann and Butchart 2000). The D_{10} value is the effective diameter of the particle corresponding to 10 percent on the particle size slot. Hence, 10 percent of the particles are finer and 90 percent are coarser than the effective size. The following table lists the D_{10} values obtained from the gradation testing of the recessional outwash deposits.

SUMMARY OF D_{10} VALUES

Exploration/ Sample No.	Depth (ft)	Soil Classification	D_{10} (mm)	Remarks
S-1	Surface	GP	0.281	
S-2	Surface	GP-GM	0.190	
S-3	Surface	GP-GM	0.154	
S-4	Surface	GW	0.277	
S-5	Surface	GP	0.378	
S-6	Surface	GP	0.296	
S-7	Surface	GW	0.470	
S-8	Surface	GP	0.355	
S-9	Surface	GW	0.413	
TP-7	6.5	GP	0.378	Infiltration test completed at 4 ft
TP-8	6.0	GP	0.296	
TP-9	6.0	GP	0.470	
TP-10	6.0	GP	0.355	
TP-11	4.0	GP/SP	0.413	
TP-11	5.5	GP	1.54	
TP-12	4.0	GP	0.46	
TP-12	6.0	GP	0.317	
TP-13	4.5	GP-GM	1.089	Infiltration test completed at 4.5 ft
TP-13	6.5	GP	2.196	

ESTIMATED INFILTRATION RATES

Infiltration field testing was conducted in the South and East Stormwater Retention Pond area. Data collected from these tests were used to compute infiltration rates for each of these areas. The infiltration test methods and results are presented below.

Infiltration Field Testing Methods

An infiltration test was performed at test pit TP-7 (Infiltration Test 1) for design of the South Stormwater Retention Pond and at test pit TP-13 (Infiltration Test 2) for design of the East Stormwater Retention Pond. Infiltration Test 1 (IT-1) was completed on November 2, 2006, and Infiltration Test 2 (IT-2) was completed on November 9, 2006.

For IT-1, test pit TP-7 was excavated to the approximate dimensions of 4 ft wide by 6 ft long and about 4 ft deep. The test pit was filled with water from a water truck and maintained at a depth of about 1.9 ft until the flow rate stabilized at about 20 gpm. The water was then shut off and allowed to infiltrate. The water level in the test pit was recorded every 4 minutes until the rate of infiltration stabilized.

For IT-2, test pit TP-13 was excavated to the approximate dimensions of 3.5 ft wide by 5 ft long and about 4.5 ft deep. The test pit was filled with water from a water truck and maintained at a depth of about 3 ft until the flow rate stabilized at about 47.5 gpm. The water was then shut off and allowed to infiltrate. The water infiltrated within 3 minutes.

Infiltration Testing Results

The following table summarizes the result of the infiltration tests.

SUMMARY OF FIELD INFILTRATION TESTING

Test Location	Soil Classification	Infiltration Rate (inches/hour)
IT-1	GP	80
IT-2	GW	261

In addition to the field test completed for this study, King County completed a series of *in situ* infiltration tests along the north shoulder of Cedar Grove Road SE on June 15, 2006 (King County Department of Transportation 2006). The infiltration testing consisted of three falling head tests completed in accordance with the procedure in the *King County Surface Water Design Manual* (King County 2005), and two large-scale infiltration tests, ITH-1 and ITH-2, similar in scope to the two infiltration tests performed for this study. The approximate locations of the two large-scale King County infiltration tests are shown on Figure 2. Infiltration Test ITH-1 was located about 300 ft southwest of

IT-2, and ITH-2 was located about 950 ft east of IT-1. The soil types were similar to those tested for this study. The computed infiltration rates are summarized in the table below.

SUMMARY OF FIELD INFILTRATION TESTING COMPLETED BY KING COUNTY (2006)

Test Location	Soil Classification	Infiltration Rate (inches/hour)
ITH-1	GP	300
ITH-2	GP-GM	68

The field infiltration rates obtained by King County for similar soil types are consistent with the field infiltration rates determined by this study.

The *2005 King County Surface Water Design Manual* provides a simplified method for computing the preliminary design infiltration rate from the results of the *in situ* infiltration tests. Correction factors are applied to the field infiltration rate to account for uncertainties in testing, facility geometry, groundwater, and plugging. According to the *2005 King County Surface Water Design Manual*, the computed design infiltration rate cannot exceed 20 inches per hour.

The equation below is used to estimate the maximum design infiltration rate:

$$I_{design} = I_{measured} \times F_{testing} \times F_{geometry} \times F_{plugging}$$

The *2005 King County Surface Water Design Manual* recommends using a correction factor for testing of 0.5 for large scale testing.

The correction factor for geometry is given by the relationship below. According to the *2005 King County Surface Water Design Manual*, $F_{geometry}$ must be between 0.25 and 1.0.

$$F_{geometry} = 4D/W + 0.05$$

Where: D = the depth from the bottom of the pond to the maximum seasonal water table or nearest impervious layer, whichever is less
W = width of the facility.

Groundwater underlies the proposed infiltration ponds at a depth of 15 ft or greater (Landau Associates 1990, 1991). The East Stormwater Retention Pond has a proposed preliminary width of about 150 ft, and the proposed preliminary widths of the Main Infiltration Area and the South Stormwater Retention Pond are about 100 ft. Therefore, assuming a depth to groundwater of at least 15 ft, the computed correction factor for geometry for the Main Infiltration Area and South Stormwater Retention Pond is about 0.65, and about 0.45 for the East Stormwater Retention Pond. The *2005 King County Surface Water Design Manual* recommends using a correction factor of 1.0 for plugging of coarse-grained soil. Therefore, using the relationship above, the computed preliminary design infiltration rate at infiltration test location IT-1 (South Stormwater Retention Pond) is 26 inches per hour and 58 inches per hour at infiltration test location IT-2 (East Stormwater Retention Pond).

CONCLUSIONS AND RECOMMENDATIONS

Because the *2005 King County Surface Water Manual* limits the infiltration rate at 20 inches per hour, a design infiltration rate of 20 inches per hour is recommended for design of the East and South Stormwater Retention Ponds. Because the grain size characteristics of the soil in the north slope area of the main gravel pit and the soil in the Main Infiltration Area are similar to those encountered in the South Stormwater Retention Pond area, a preliminary design infiltration rate of 20 inches per hour for infiltration is appropriate for design of infiltration facilities in the north slope area and the Main Infiltration Area.

Observation of the Main Infiltration Area over time indicates that this area is capable of infiltrating very high rates of recharge that exceed 20 inches/hour. For the north slope area, we recommend implementation of large scale infiltration tests to verify the infiltration rate in this area. These tests should be conducted between Elevation 410 and 420 NAVD 88; this elevation is equivalent to the approximate elevation of the planned East Airstrip Springs infiltration gallery. Further infiltration testing of the north slope area could most easily be conducted after implementation of Phase 2 of the fill project. At this point in the project, backhoe and water truck access to the north slope at the required elevations would be relatively straightforward.

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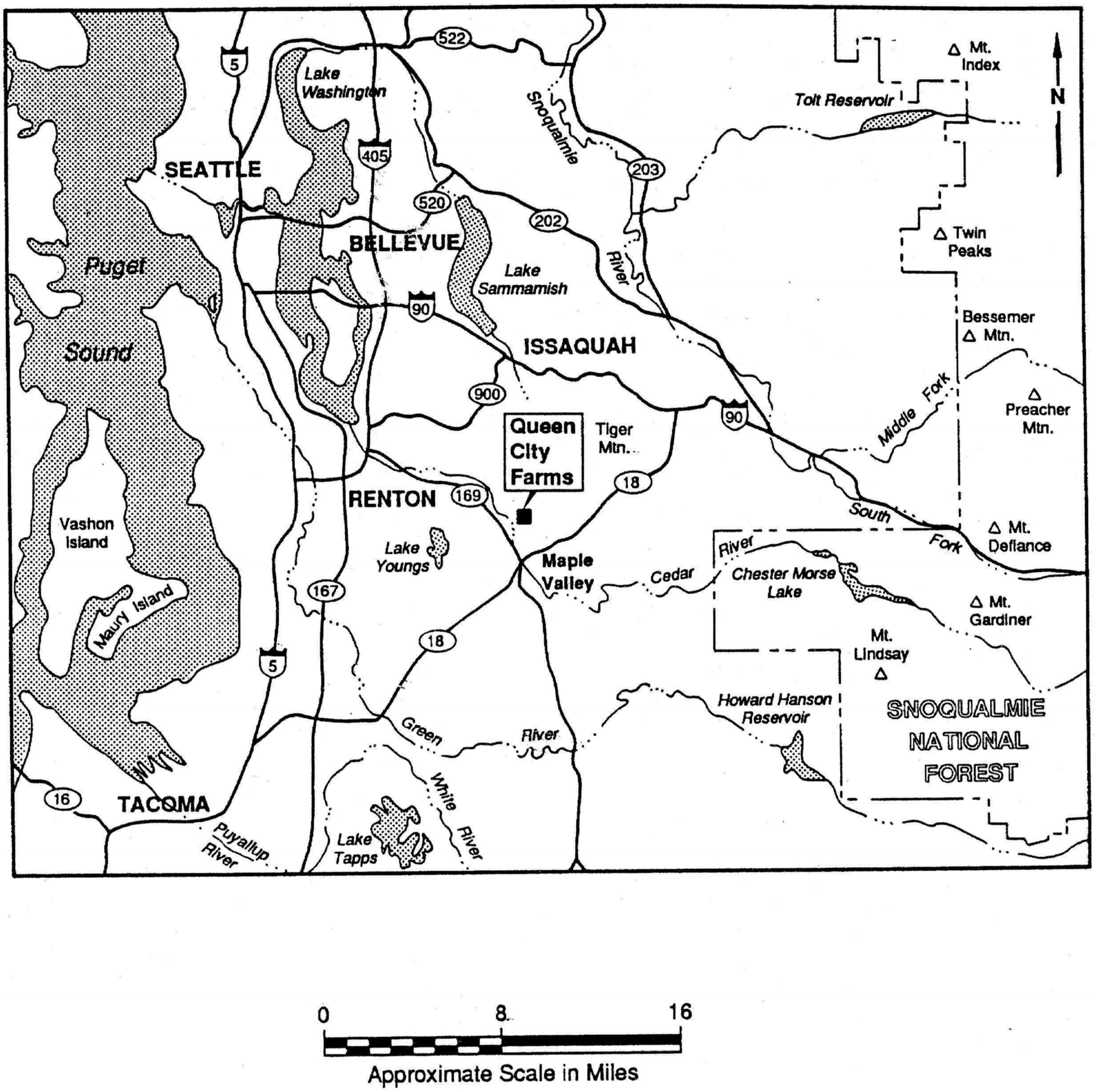
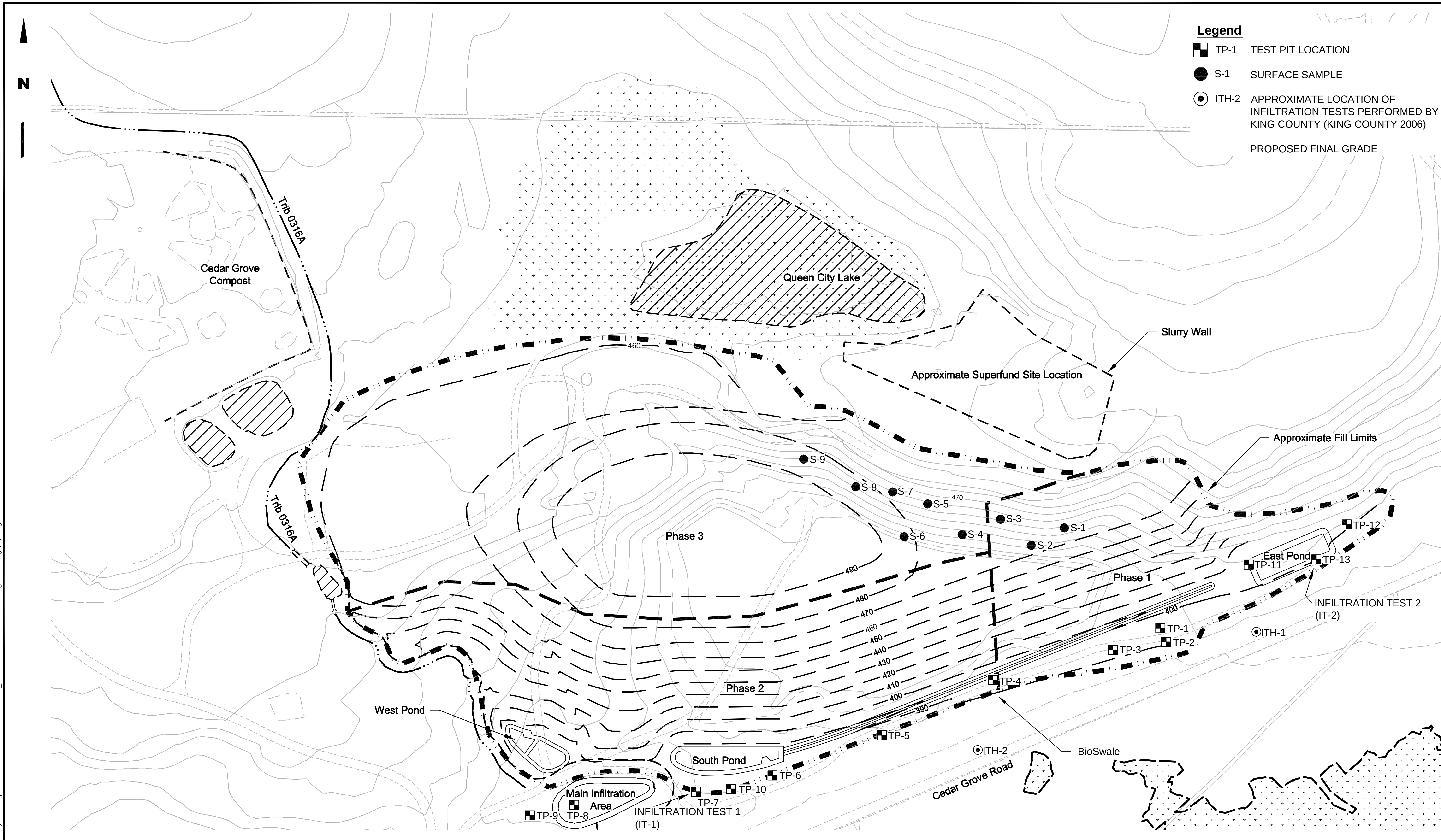


Source: Queen City Farms RI Report, 1990

LANDAU ASSOCIATES	Queen City Farms Refill Project Maple Valley, Washington	**Vicinity Map**	Figure 1



Queen City Farms | V:\992\001\011\D\TechMemos_FINAL\6.8 Infiltration Eval\Figure 2.dwg (A) "Figure 2" 1/30/2007

Landau Associates

Queen City Farms
Refill Project
Maple Valley, Washington

Exploration Plan

Figure
2

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL[1]	TYPICAL DESCRIPTIONS [2][3]
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)		GW	Well-graded gravel; gravel/sand mixture(s); little or no fines
				GP	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)		GM	Silty gravel; gravel/sand/silt mixture(s)
				GC	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)		SW	Well-graded sand; gravelly sand; little or no fines
				SP	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)		SM	Silty sand; sand/silt mixture(s)
				SC	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)			ML	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
				CL	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
				OL	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)			MH	Inorganic silt; micaceous or diatomaceous fine sand
				CH	Inorganic clay of high plasticity; fat clay
				OH	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL			PT	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT		AC or PC	Asphalt concrete pavement or Portland cement pavement
ROCK		RK	Rock (See Rock Classification)
WOOD		WD	Wood, lumber, wood chips
DEBRIS		DB	Construction debris, garbage

NOTES:

1. USCS letter symbols correspond to symbols used by the Unified Soil Classification System and ASTM classification methods. Dual letter symbols (e.g., SP-SM for sand or gravel) indicate soil with an estimated 5-15% fines. Multiple letter symbols (e.g., ML/CL) indicate borderline or multiple soil classifications.
2. Soil descriptions are based on the general approach presented in the *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*, outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the *Standard Test Method for Classification of Soils for Engineering Purposes*, as outlined in ASTM D 2487.
3. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

Primary Constituent:	> 50% - "GRAVEL," "SAND," "SILT," "CLAY," etc.
Secondary Constituents:	> 30% and ≤ 50% - "very gravelly," "very sandy," "very silty," etc.
	> 15% and ≤ 30% - "gravelly," "sandy," "silty," etc.
Additional Constituents:	> 5% and ≤ 15% - "with gravel," "with sand," "with silt," etc.
	≤ 5% - "trace gravel," "trace sand," "trace silt," etc., or not noted.

1/25/07 \\TAC01\DATA\DATA\PROJECT\992 QCF\T\DATA\992001.010.011.GPJ SCS WSDOT 1OF2

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Queen City Farms
Refill Project
Maple Valley, Washington

Soil Classification System and Key

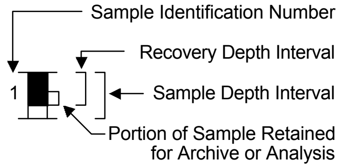
Figure 3 (1 of 2)

Drilling and Sampling Key

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Single-Tube Core Barrel
f	Double-Tube Core Barrel
g	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Rotosonic
5	Air Rotary (Rock)
6	Wash Rotary (Rock)
7	Other - See text if applicable

SAMPLE NUMBER & INTERVAL



Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
VST	Vane Shear Test
GT	Other Geotechnical Testing
CA	Chemical Analysis

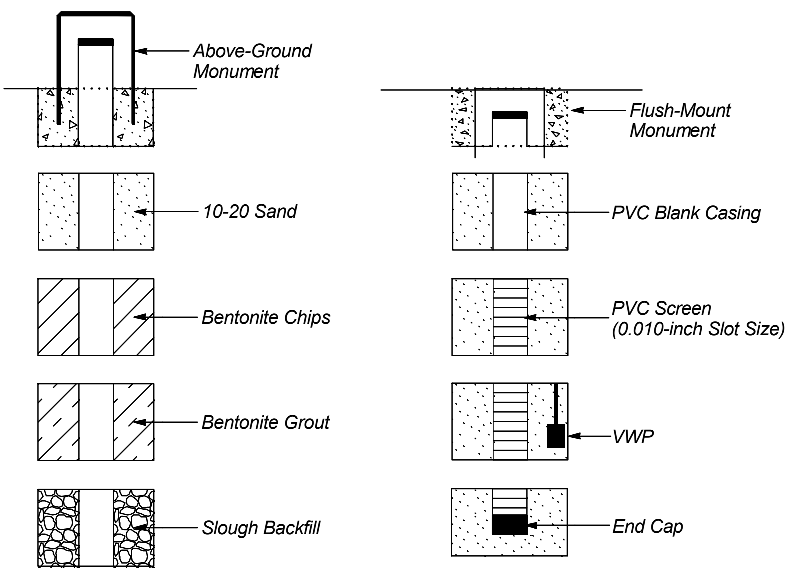
Groundwater

▼ Approximate water elevation at time of drilling (ATD).

▽ Approximate water elevation at other time(s). When multiple water levels are obtained other than ATD, only a representative range is shown. See text for additional information.

Note: Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

Well Log Graphics



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Queen City Farms Refill Project Maple Valley, Washington	Soil Classification System and Key	Figure 3 (2 of 2)

TP-1

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
0–4	S1	d			ML	Gray to light brown, fine to medium, sandy, clayey SILT with gravel (soft to medium soft, moist to wet) **FILL**	Groundwater not encountered.

Test Pit Completed 11/02/06
Total Depth of Test Pit = 4.0 ft.

Depth scale: 0, 2, 4, 6, 8, 10

TP-2

SAMPLE DATA				SOIL PROFILE			GROUNDWATER
Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	Graphic Symbol	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
0–1.5	S1	d			GM	Gray-brown, silty, sandy GRAVEL to sandy GRAVEL with silt (dense, moist) **FILL**	Groundwater not encountered.
1.5–4.3	S2	d	W = 12 GS		GP	Gray to brown, very sandy GRAVEL with trace silt (dense, moist)	

Test Pit Completed 11/02/06
Total Depth of Test Pit = 4.3 ft.

Depth scale: 0, 2, 4, 6, 8, 10

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Queen City Farms Refill Project Maple Valley, Washington	Log of Test Pits	Figure 4

TP-3

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
0				SM	Sod	Groundwater not encountered.
0–2.5	S1	d		GM	Light brown to gray, silty, sandy GRAVEL (dense, moist) **FILL**	
2.5–5	S2	d	W = 4 GS	GP-GM	Light brown to gray, very sandy GRAVEL with silt (dense, damp)	

Excavation Method: Rubber-tired Backhoe

Ground Elevation (ft): Not measured

Test Pit Completed 11/02/06
Total Depth of Test Pit = 5.0 ft.

TP-4

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
0				SM	Forest Duff	Groundwater not encountered.
	S1	d		SM	Light reddish brown, silty, very gravelly, fine to medium SAND with roots (medium dense, damp) **FILL**	
	S2	d		GP-GM	Light brown, very sandy GRAVEL with silt and cobbles to 9 inches (medium dense, damp)	
	S3	d				
	S4	d	W = 4 GS		Some fine roots	
8–9	S5	d	W = 5 GS	GP-GM	Light brown, very sandy GRAVEL with silt and roots (medium dense, damp) **RECESSIONAL OUTWASH**	

Excavation Method: Rubber-tired Backhoe

Ground Elevation (ft): Not measured

Test Pit Completed 11/02/06
Total Depth of Test Pit = 9.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Maple Valley, Washington

Log of Test Pits

Figure 5

TP-5

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
					Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
0				SM	Sod	
	S1	d		SM	Brown, silty, gravelly, fine to medium SAND with roots and cobbles (medium dense, moist) **FILL**	Groundwater not encountered.
4	S2	d	W = 16 GS	SM	Light yellow - brown, gravelly, silty fine to medium SAND and very sandy GRAVEL with silt (loose, moist)	
6						

Test Pit Completed 11/02/06
Total Depth of Test Pit = 6.0 ft.

TP-6

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
					Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
0				SM	Sod	
	S1	d		SM	Brown, silty, very gravelly, fine to medium SAND with roots and trace organics (loose, moist to wet) **FILL**	∇ Moderate
6	S2	d		ML	Mottled gray and light brown, clayey SILT with gravel (soft to medium stiff, wet)	
7						

Test Pit Completed 11/02/06
Total Depth of Test Pit = 7.0 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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Log of Test Pits

Figure 6

TP-7

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
					Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
0				SM	Sod	Groundwater not encountered.
	S1	d		GM	Brown, silty, sandy GRAVEL (medium dense, moist) **FILL**	
2	S2	d		SP	Gray - brown, very gravelly, fine to coarse SAND with cobble (medium dense, moist) **RECESSIONAL OUTWASH**	
4				GP	Grades to very sandy GRAVEL to sandy GRAVEL with cobbles to 12 inches and trace small boulders (14 inches max)	
6	S3	d	W = 4 GS			
				SP-SM	Reddish brown, fine to medium SAND with silt (dense to very dense, moist) **INTERGLACIAL DEPOSIT**	
8	S4	d	W = 14 GS			

Test Pit Completed 11/02/06
Total Depth of Test Pit = 8.5 ft.

TP-8

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
					Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
0				SM	Sod	Groundwater not encountered.
	S1	d		SM	Light brown to gray, gravelly, silty, fine to medium SAND with roots (loose, wet) **FILL**	
2	S2	d		GP	Light brown, GRAVEL (open worked) with trace silt (loose, moist) **RECESSIONAL OUTWASH**	
4	S3	d		GP-GM	Light brown, very sandy GRAVEL with silt, fine sand and cobbles (open worked) (loose, wet)	
6	S4	d	W = 6 GS		Grades trace silt	

Test Pit Completed 11/02/06
Total Depth of Test Pit = 7.0 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

992001.011 1/25/07 \\TAC01\DATA\DATA\PROJECT\992 QCF\T\DATA\992001.010.011.GPJ TEST PIT LOG

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Log of Test Pits

Figure 7

TP-9

Excavation Method: Rubber-tired Backhoe

Ground Elevation (ft): Not measured

SAMPLE DATA

Sample Number & Interval	Depth (ft)	Sampler Type	Test Data
S1	~1	d	
S2	~2	d	
S3	~4.5	d	
S4	~6.5	d	W = 6 GS

SOIL PROFILE

Depth (ft)	USCS Symbol	Description
0	SM	Forest Duff
0–1	SM	Light brown, silty, gravelly, fine to medium SAND (loose, moist) **FILL**
1–4	GP	Light brown GRAVEL (open work) (loose, damp) **RECESSIONAL OUTWASH**
4–6.5	GP-GM	Light brown, very sandy GRAVEL with silt, majority matrix supported (medium dense, moist)

GROUNDWATER

Groundwater not encountered.

Test Pit Completed 11/02/06
Total Depth of Test Pit = 6.5 ft.

TP-10

Excavation Method: Rubber-tired Backhoe

Ground Elevation (ft): Not measured

SAMPLE DATA

Sample Number & Interval	Depth (ft)	Sampler Type	Test Data
S1	~2	d	
S2	~3	d	
S3	~5–5.5	d	
S4	~5.5–7	d	W = 4 GS
S5	~8.5	d	

SOIL PROFILE

Depth (ft)	USCS Symbol	Description
0	SM	Sod
0–1	SM	Brown, gravelly, silty, fine to medium SAND (loose, moist) **FILL**
1–5	GM	Brown, silty, sandy GRAVEL with cobbles to 10 inches and brown, clayey silt clasts (medium dense, moist to wet)
5–6	SM	Brown, silty, gravelly, fine to coarse SAND (medium dense, wet)
6–8.5	GW/GP	Light brown, very sandy GRAVEL with trace silt (medium dense, damp) **RECESSIONAL OUTWASH**

GROUNDWATER

Depth (ft)	
4	Slight
5	Slight

Test Pit Completed 11/02/06
Total Depth of Test Pit = 8.5 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
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Maple Valley, Washington

Log of Test Pits

Figure 8

TP-11

SAMPLE DATA			SOIL PROFILE		GROUNDWATER
Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
			SM	Organic material	Groundwater not encountered.
			SM	Brown, silty, fine to coarse SAND with gravel (moist to wet) **FILL** Becomes light brown	
			GW	Gray - brown, very sandy GRAVEL (medium dense, moist to wet) **RECESSIONAL OUTWASH**	
S1 (4 ft)	d	W = 4 GS	GW	Gray - brown GRAVEL with sand and trace silt, scattered cobble (medium dense, moist to wet)	
S2 (6 ft)	d	W = 4 GS			

Depth (ft): 0–10

Test Pit Completed 11/09/06
Total Depth of Test Pit = 6.0 ft.

TP-12

SAMPLE DATA			SOIL PROFILE		GROUNDWATER
Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Excavation Method: Rubber-tired Backhoe Ground Elevation (ft): Not measured	
			SM	Organic material	Groundwater not encountered.
			SM	Brown, silty, fine to coarse SAND with gravel (medium dense, moist) **FILL** Becomes light brown and moist to wet	
			GW	Gray - brown, sandy GRAVEL with trace silt (medium dense, moist to wet) **RECESSIONAL OUTWASH**	
S1 (4 ft)	d	W = 5 GS	GW	Gray, sandy GRAVEL with cobble and trace silt (medium dense, moist to wet)	
S2 (6 ft)		W = 5 GS			

Depth (ft): 0–10

Test Pit Completed 11/09/06
Total Depth of Test Pit = 6.0 ft.

Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

992001.011 1/25/07 \\TAC01\DATA\DATA\PROJECT\992 QCF\T\DATA\992001.010.011.GPJ TEST PIT LOG

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Log of Test Pits

Figure 9

TP-13

SAMPLE DATA | **SOIL PROFILE** | **GROUNDWATER**

Excavation Method: Rubber-tired Backhoe

Ground Elevation (ft): Not measured

Depth (ft)	Sample Number & Interval	Sampler Type	Test Data	USCS Symbol	Soil Profile	Groundwater
0				SM	Organic material	Groundwater not encountered.
				SM	Brown, silty, fine to coarse SAND (medium dense, moist to wet)	
2					FILL	
				SM	Light brown, silty, fine to coarse SAND with gravel (medium dense, wet)	
				GW	Gray - brown GRAVEL with sand (medium dense, moist to wet)	
4					RECESSIONAL OUTWASH	
	S1	d	W = 6 GS	GW	Gray, sandy gravel GRAVEL with sand and scattered cobble (medium dense, wet)	
6						
	S2	d	W = 5 GS		Grades with sand	
8						
10						

Test Pit Completed 11/09/06
Total Depth of Test Pit = 7.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
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3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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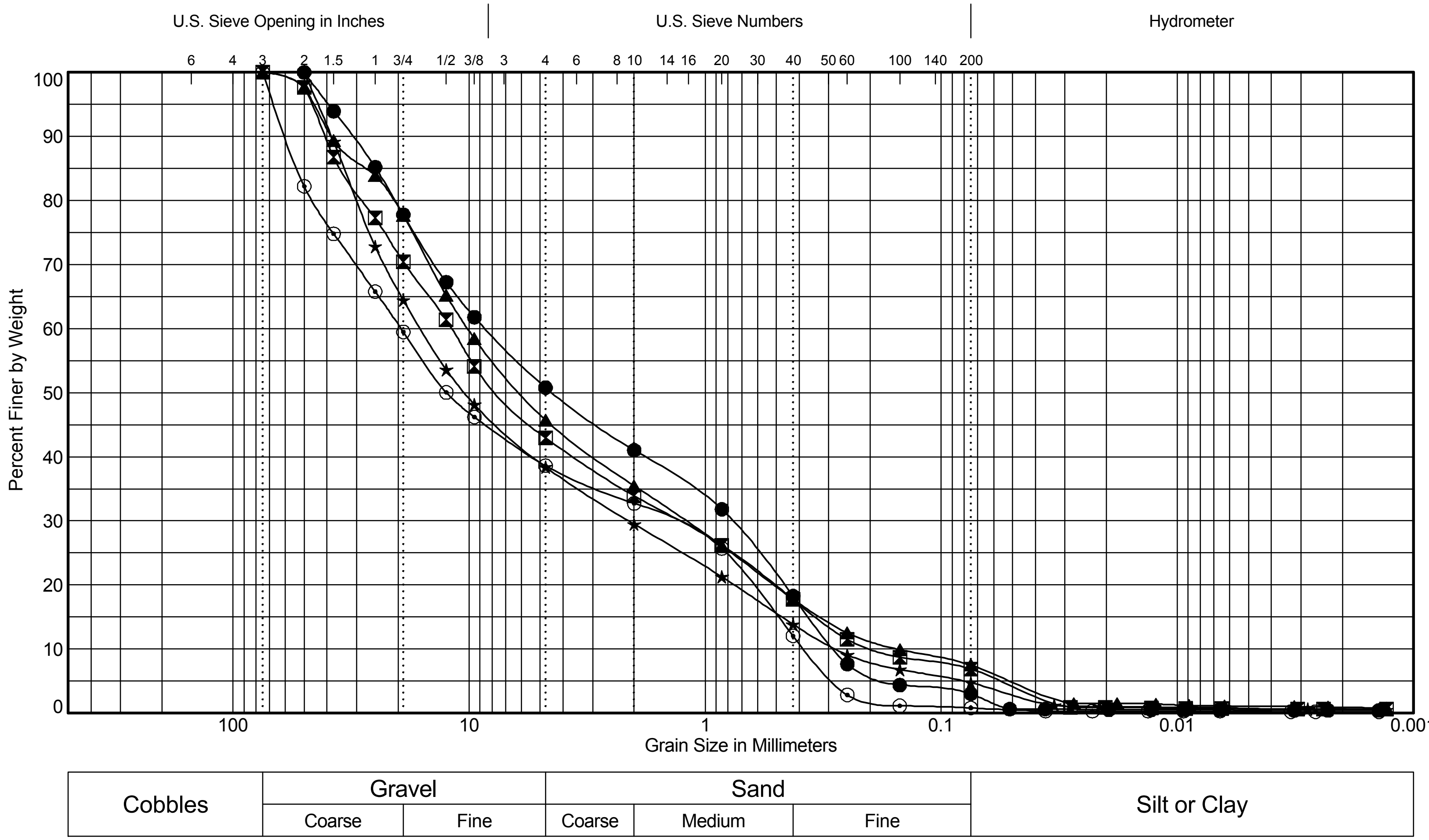
Log of Test Pits

Figure 10



Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	NA	S-1	NA		Very sandy GRAVEL with trace silt	GP
⧗	NA	S-2	NA		Very sandy GRAVEL with silt	GP-GM
▲	NA	S-3	NA		Very sandy GRAVEL with silt	GP-GM
★	NA	S-4	NA		Very sandy GRAVEL with trace silt	GW
⊙	NA	S-5	NA		Very sandy GRAVEL	GP

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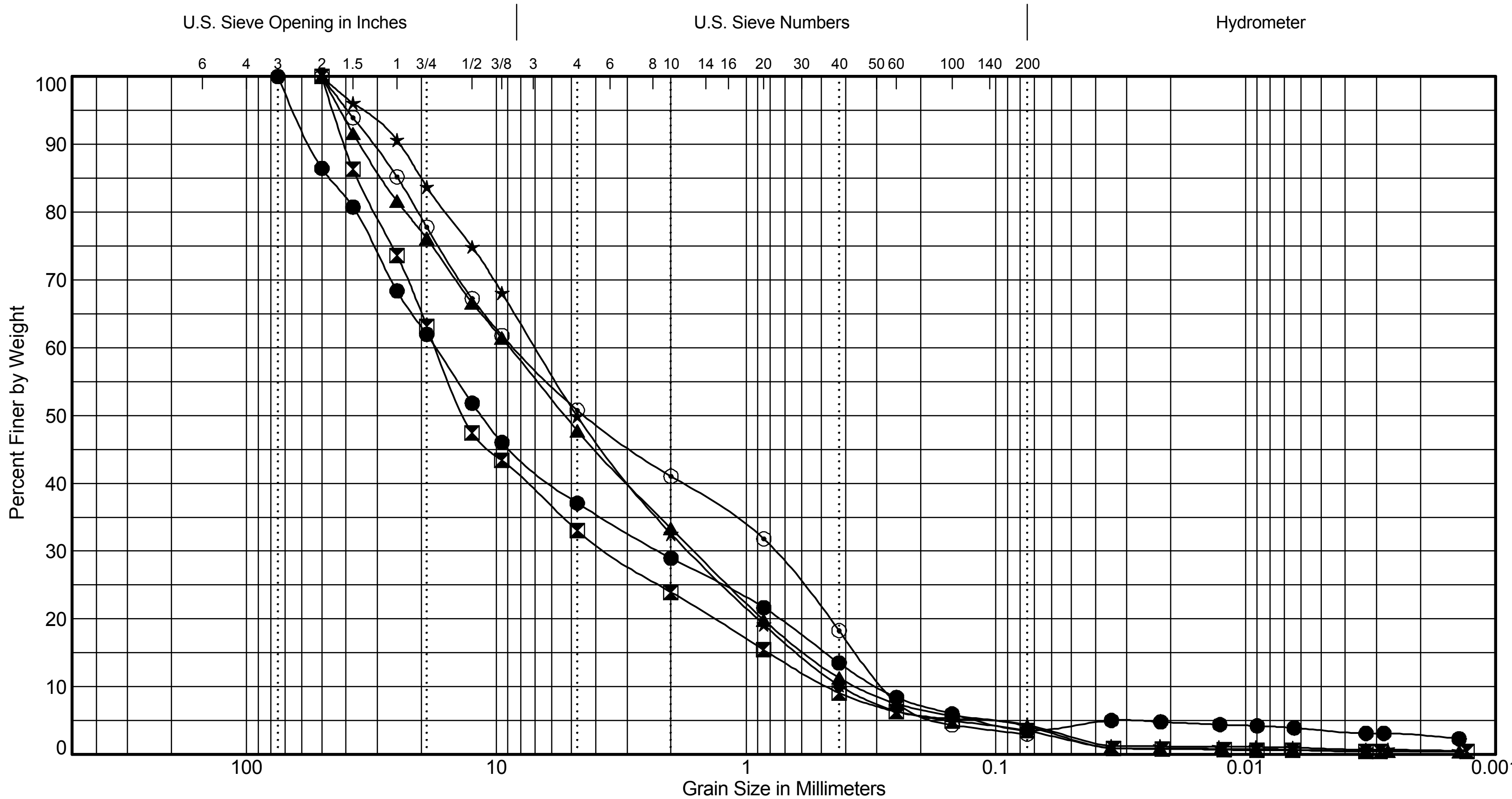
Maple Valley, Washington

Grain Size Distribution

Figure 11



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	NA	S-6	NA		Very sandy GRAVEL with trace silt	GP
⧗	NA	S-7	NA		Very sandy GRAVEL with trace silt	GW
▲	NA	S-8	NA		Very sandy GRAVEL with trace silt	GP
★	NA	S-9	NA		Very sandy GRAVEL with trace silt	GW
⊙	TP-02	S2	3.3	12	Very sandy GRAVEL/ very gravelly SAND with trace silt	GP/SP

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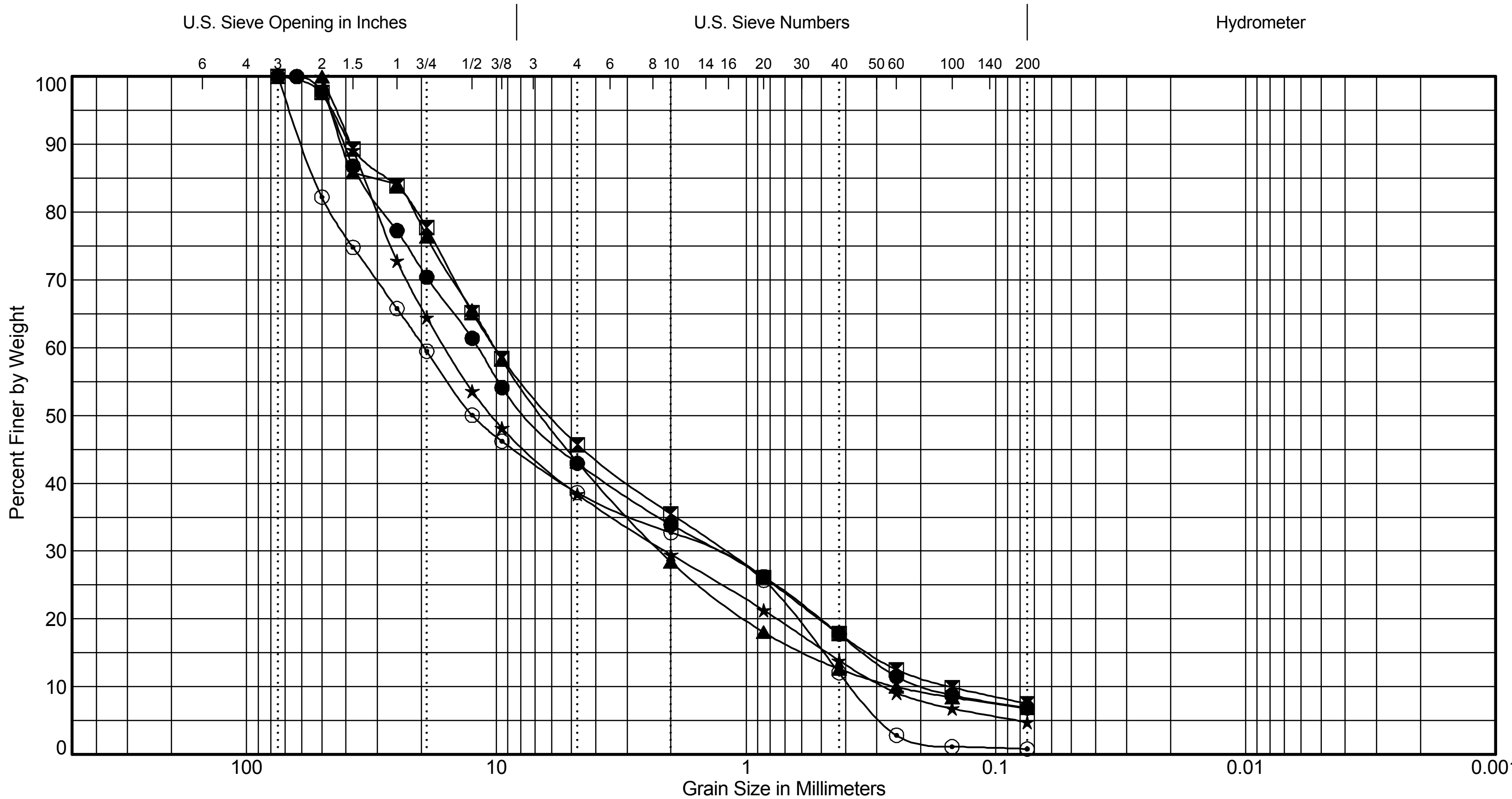
Grain Size Distribution

Figure
12



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	TP-03	S-2	3.5	4	Very sandy GRAVEL with silt	GP-GM
⊠	TP-04	S-4	6.5	4	Very sandy GRAVEL with silt	GP-GM
▲	TP-04	S-5	8.0	5	Very sandy GRAVEL with silt	GP-GM
★	TP-05	S-2	4.5	16	Very sandy GRAVEL with trace silt	GP
⊙	TP-07	S-3	6.5	4	Very sandy GRAVEL	GP

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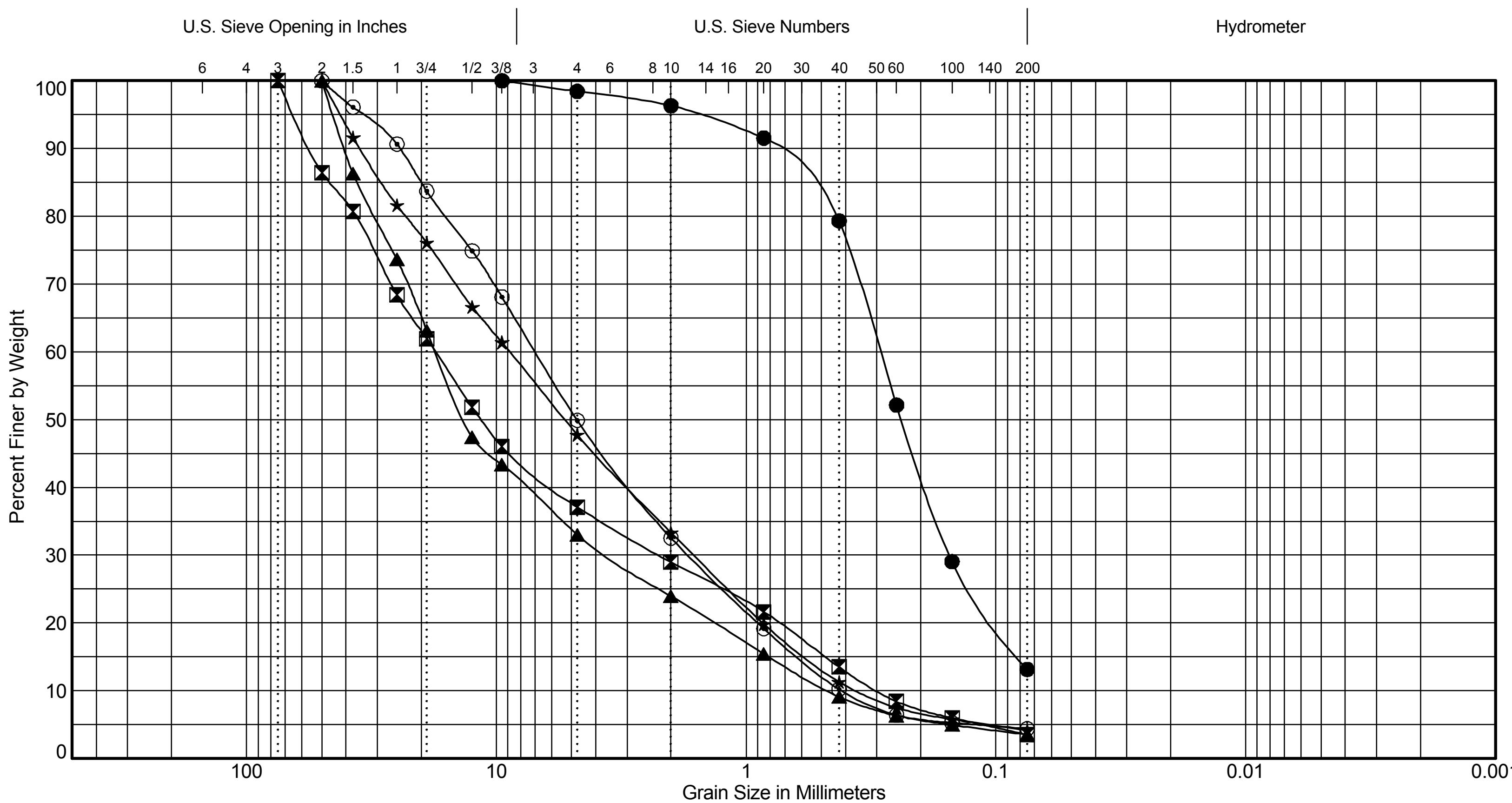
Refill Project
Maple Valley, Washington

Grain Size Distribution

Figure
13



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	TP-07	S-4	8.0	14	Fine to medium SAND with silt	SP-SM
⧗	TP-08	S-4	6.0	6	Very sandy GRAVEL trace silt	GP
▲	TP-09	S-4	6.0	6	Very sandy GRAVEL trace silt	GP
★	TP-10	S-4	6.0	4	Very sandy GRAVEL with trace silt	GP
⊙	TP-11	S-1	4.0	4	Very sandy GRAVEL with trace silt	GP/SP

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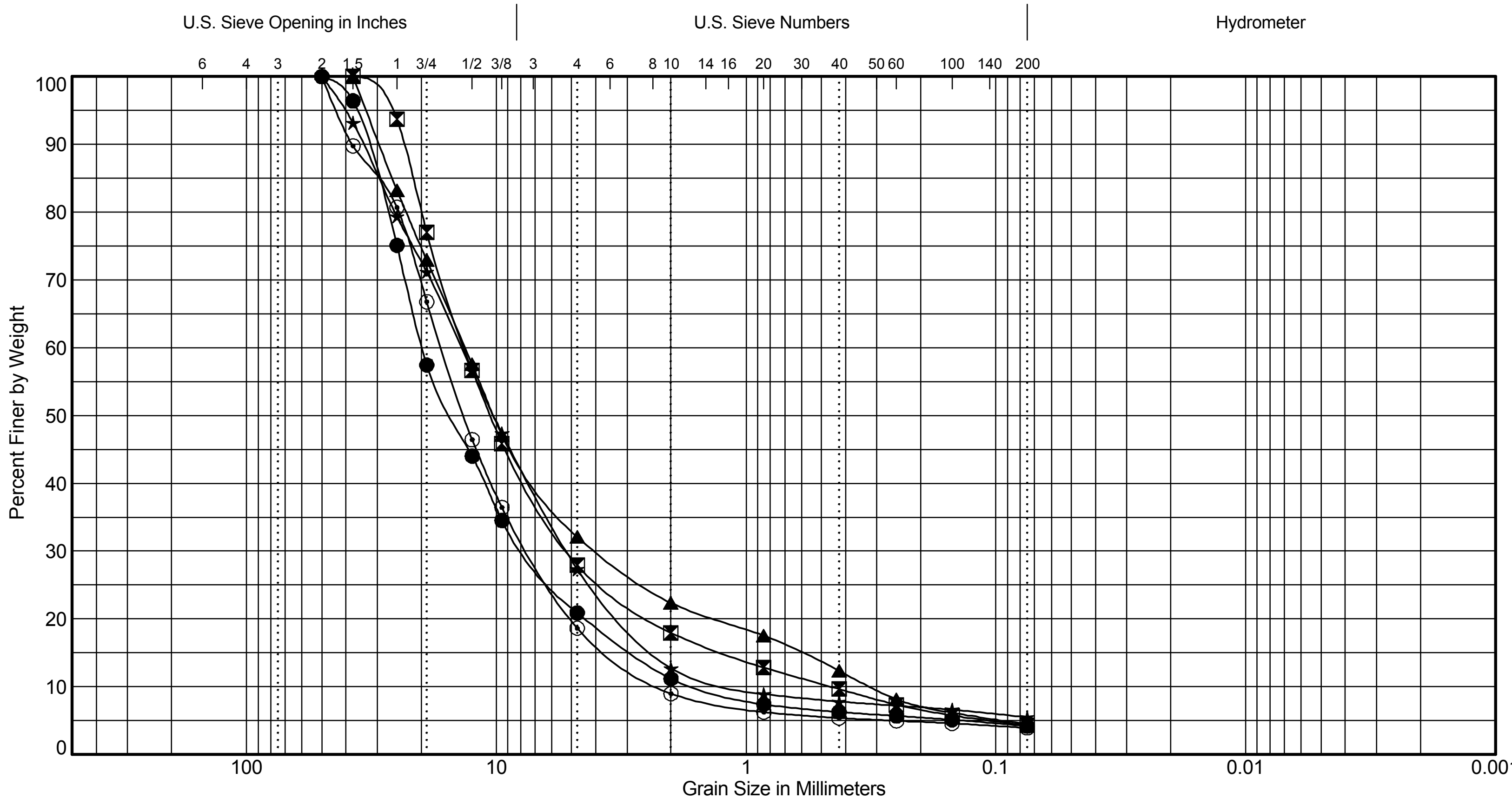
Maple Valley, Washington

Grain Size Distribution

Figure 14



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	TP-11	S-2	5.5	4	GRAVEL with sand and trace silt	GP
⧗	TP-12	S-1	4.0	5	Sandy GRAVEL with trace silt	GP
▲	TP-12	S-2	6.0	5	Sandy GRAVEL with trace silt	GP
★	TP-13	S-1	4.5	6	Sandy GRAVEL with silt	GP-GM
⊙	TP-13	S-2	6.5	5	GRAVEL with sand and trace silt	GP

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Queen City Farms Refill Project Maple Valley, Washington	Grain Size Distribution	Figure 15

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Best Management Practices for Reclaiming Surface Mines in Washington and Oregon

by David K. Norman,
Peter J. Wampler,
Allen H. Throop,
E. Frank Schnitzer,
and Jaretta M. Roloff

WASHINGTON
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Open File Report 96-2

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Front Cover: A reclaimed quarry in mountainous terrain. Naturally hazardous conditions (cliffs) are present in the immediate area. Chutes, spurs, scree slopes, and soil on the scree have created a natural appearance. Trees now grow on the slope where soil is located and complete the reclamation. The site will be used for forestry in the future. Note the person midslope for scale. Photo by M. A. Shawver.

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CHAPTER 6. LANDSLIDES AND SLOPE FAILURES

CHAPTER 7. REVEGETATION

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Preface

The term *best management practices* (BMPs) has generally been used to describe mechanical means of minimizing or eliminating water-quality problems. The BMPs presented here, however, apply as well to reclamation, planning, and specific methodologies to promote an integrated approach to mining. The techniques and guidance provided in this manual should not be construed as rules or laws, but merely the most effective and economical reclamation and mining practices known to Oregon Department of Geology and Mineral Industries (DOGAMI) and the Washington Department of Natural Resources (DNR) at the present time.

This manual provides information about planning the mine from start-up to final reclamation, incorporating water and erosion control during operation and reclamation, soil salvage and replacement, land shaping, and revegetation.

This manual was compiled and written by DOGAMI and DNR to provide technical information and guidance to landowners, land-use planners, and mine operators. We urge miners to use this manual as a resource in developing an environmentally and financially sound mine. However, while this manual is a broad overview of mine reclamation and development and other BMPs, it is not a comprehensive document, nor should it necessarily be considered the final word. Mining and reclamation will continue to evolve and improve. Locking in on technique or even just one BMP can be dangerous. Miners should consider the range of BMPs discussed here before selecting one to the exclusion of others.

Reclamation of mines, especially large mines, is a complex multidisciplinary undertaking and goes far beyond this document. Trained professionals such as agronomists, biologists, engineers, geologists, hydrogeologists, landscape architects, planners, and soil scientists can be helpful in planning and completing a mining project.

Implementation of BMPs is in everyone's best interest. For mine operators, using BMPs can result in more efficient and profitable mining. For society, BMPs can mean cleaner, more usable, and aesthetically pleasing lands. Effective reclamation as the final BMP at a site can reduce water pollution and loss of topsoil, provide fish and wildlife habitat, and allow timber production, agriculture, and other uses to be re-established.

Funding

This project was partially funded by U.S. Environmental Protection Agency grant X000798-01-0 as means of transferring technical information regarding mine regulation and environmental issues. The original grant was an agreement between Idaho, Oregon, and Washington in 1993 and has been referred to as the Tri-State agreement for mining. BMPs for mining already exist in Idaho and helped pro-

vide the impetus for Oregon and Washington to generate this BMP guidance.

Future Work

This second edition of the Best Management Practices manual incorporates the suggestions of many of our readers, including several new diagrams and topics. The manual continues to be a work in progress, improving through field experience and the feedback we receive from people using the manual. We would appreciate any comments, particularly on places where we have given too much or too little information. Comments should be directed to the authors.

Acknowledgments

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1 Maps as Management Tools

INTRODUCTION

Preparing accurate maps of the mining property and its surroundings is a key step in developing a surface mining operation. Maps allow geographic information to be summarized in a compact form. Their primary purpose is to describe geographic features and the spatial relations of these features. Maps benefit the operator by clearly defining the area in which mining is permitted, and they assist in long-range planning for both efficient use of the mine resource and timely reclamation.

TYPES OF MAPS

Surface mining regulations in both Washington and Oregon require that maps be submitted before mining permits are issued. To meet regulatory requirements, maps must provide sufficient detail to characterize the site. Types of maps that may be required for permit applications are:

- *A site access map* showing the regional setting of the site and how to get there from the nearest town.
- *A pre-mining topographic map* establishing the location and setting of the mine site as it exists before mining.
- *A geologic map* giving a detailed description of the geologic setting and the type of deposit to be mined (required only if specifically requested).
- *A reclamation sequence map* showing the borders and sequence of segments to be mined and reclaimed, including the directions in which soils will be moved during salvage and replacement, and the location of storage areas and other mine-related features.
- *A final reclamation map* and at least two intersecting cross sections showing the mine site as it will appear after reclamation and revegetation.
- *A revegetation map* showing the location and types of plants used for revegetation. (This may be combined with the final reclamation map if the information will not obscure contours.)

MAP SIZES

The map size preferred for review is 11 x 17 inches, which is easy to photocopy and store. If maps are small, they may be grouped on a single sheet of paper. If the maps submitted are larger than 11 x 17 inches or if they are in color, seven or more copies must be provided. The copies will be forwarded to other reviewing agencies.

Because 11 x 17 inches is generally not practical for internal working purposes, draft and working copies may be larger. For example, some larger mines may require a scale of 1″ = 200′ or 1″ = 400′ and thus large sheets. Draft and working copies may be reduced on a photocopier for submission. Make sure the map scale reflects any reduction.

BASIC ELEMENTS

Basic elements required on every map are the:

- map scale, both written out as a ratio and shown graphically as a bar or rake scale
- north arrow
- explanation block or legend
- title block

Map Scale

Every map, regardless of the size of the site, should include a scale that indicates the relationship between the size of features on the map and the size of the same features on the ground. Most scales are represented by stating that 1 inch on the map represents a certain number of inches, feet, or miles on the ground. For example, 1″ = 200′ means that 1 inch on the map represents 200 feet on the ground.

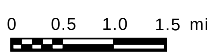
The scale that best represents a site will depend on the detail required and the size of the site, and the level of detail depends on the size and complexity of the mine. A map of a 50-acre rock quarry near a stream will normally require greater detail than a map of a 5-acre upland gravel extraction site. For some proposals, it may be acceptable to give only an approximate scale.

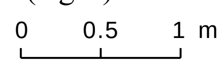
Site size	Suggested Map scale
3–6 acres	not less than 1″ = 50′
10–20 acres	not less than 1″ = 100′
20–80 acres	not less than 1″ = 200′
>80 acres	not less than 1″ = 400′

Note: If the map is reduced or enlarged, make sure the verbal scale is adjusted as well. Maps without a scale will not be accepted.

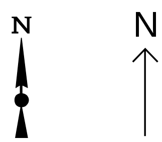
Graphic Scales

Map scales shown graphically should also be included. They will remain accurate when the map is reduced or enlarged. Examples of a bar scale (left) and a rake scale (right) are shown below:





North Arrow



All maps must show true north. This is typically done by drawing a line oriented N–S with an arrow pointing north. The north arrow in conjunction with the scale allows the map to be properly oriented during field inspections and to be related to other maps. Examples of north arrows are shown on the left.

Explanation Block

The explanation block or legend defines all symbols and patterns used and may contain the scale.

Title Block

The title block should contain the following information:

- title of map,
- application or permit number,
- name and address of applicant or permit holder(s),
- signature of applicant or permit holder(s),
- map or exhibit number, and
- date map was drawn or revised.

TOPOGRAPHIC CONTOURS

Topographic contours are lines on a map that connect points of equal elevation. For example, a 100-foot contour line links all points that have an elevation of 100 feet. Although not required on all maps, contours are useful in determining the steepness of slopes and the location of watercourses. Contours are deemed adequate for mine permitting if they accurately reflect the conditions of the site. Generally, contour intervals should be between 5 and 20 feet.

Typically, only large and/or complex sites require surveyed contour lines. Most applications for small sites can use a photocopied enlargement of a U.S. Geological Survey (USGS) topographic map. Enlarging a USGS 7.5-minute quadrangle (1″ = 2,000′) by 400 percent yields a map at a scale of 1″ = 500′. Care must be taken to ensure that the scale of the enlargement is accurate.

USGS maps are usually available at local hunting or sporting goods stores. They may also be ordered from the Washington Department of Natural Resources Photo and Map Sales (360-902-1234), the Nature of the Northwest Information Center (503-731-4444), or the U.S. Geological Survey (509-353-2524).

BOUNDARIES

Several types of boundaries may be required on maps: the permit area boundary, the mining area boundary (including present and future mining areas), and the property lines. The symbols for all should be included in the explanation block.

Permit Area Boundary

This is the boundary within which mining is permitted. Any mining, processing, or activity related to mining taking place outside this area constitutes mining without a permit and may invoke closure and/or civil penalties. In some places, the permit boundary may be coincident with the property boundary. However, the permit boundary may cross property lines and can include property held by different landowners. Once the boundary has been defined, changes to it typically require an amendment to the reclamation permit and may require land-use approval by the local jurisdiction.

The permit boundary is commonly indicated on maps as a dashed or solid line. This line type and width should be distinguishable from the property line boundary and should be clearly labeled as ‘permit boundary’.

Mining Boundaries

Mining boundaries show the areas to be mined or excavated. Several maps may be needed to show areas affected by short-term and long-term operations.

Boundaries of Cities and Counties

Boundaries of cities, counties, and other municipalities must be shown if they cross the map area.

Property Lines

Tax lot maps from the county assessor’s office are good sources of property line information. Property line locations are critical in determining setbacks to property lines and the likelihood of potential impacts to adjacent landowners.

The property line boundary is typically shown on maps as a solid line. The property line type and width should be distinguishable from the permit boundary line and should be clearly labeled. The letters 'PL' are commonly used to indicate a property line on maps, but this line and abbreviation must also be identified in the explanation block.

OTHER COMMON MAP ELEMENTS

The following map elements should be shown on one or more of the required maps.

Existing Watercourses, Ponds, and Wetlands

All streams, rivers, wetlands, and ponds on and adjacent to the site must be indicated on the map. Accurate location of these features allows reviewers to assess potential mining-related impacts and also aids the miner in the design of erosion and storm-water control systems to protect water quality.

Streams and rivers are represented by lines that are distinct from those used for haul roads, permit boundaries, and property lines. Ponds, wetlands, and lakes should be labeled and/or patterned to distinguish them from other mine features.

Processing Plant

Proper location of processing facilities makes good use of the topography for screening and noise control—for example, siting the facilities in a low area. (See Visual and Noise Screens, p. 3.6.) The location of the processing facilities can be labeled or a symbol may be used.

Haul Roads

Most roads can be placed to avoid potential problems. Proper location, construction, and drainage of roads can minimize turbid water and slope-stability problems. (See Passive Storm Water Control, p. 2.6, and Figs. 2.7 and 2.8.) Roads can be shown as lines whose width or line type (dashed, etc.) distinguish them from property lines and permit boundaries.

Soil and Overburden Stockpiles

Soil should be preserved for reclamation. The reclamation sequence map must show where topsoil, subsoil, and overburden will be stored until they are reapplied during reclamation. Soil stockpiles can be indicated by drawing a line around the proposed location, adding a distinctive pattern, and labeling the area 'topsoil', 'subsoil', or 'overburden'. (See Removing and Storing Topsoil and Subsoils, p. 3.13.)

Product Stockpiles and Waste-Rock Dumps

Stockpiles of usable rock and waste-rock dumps are generally indicated on maps by drawing a line around the proposed location, adding a distinctive pattern, and labeling the area 'stockpile' or 'waste dump'. Stability and potential erosion problems are criteria to be considered in selecting the location of a stockpile or dump. Site topography will influence these factors. (See Waste and Overburden Dumps and Stockpiles, p. 3.15.)

Interim Watercourses and Ponds

Temporary watercourses and ponds, including settling ponds and drainage ditches to control storm-water runoff, should be distin-

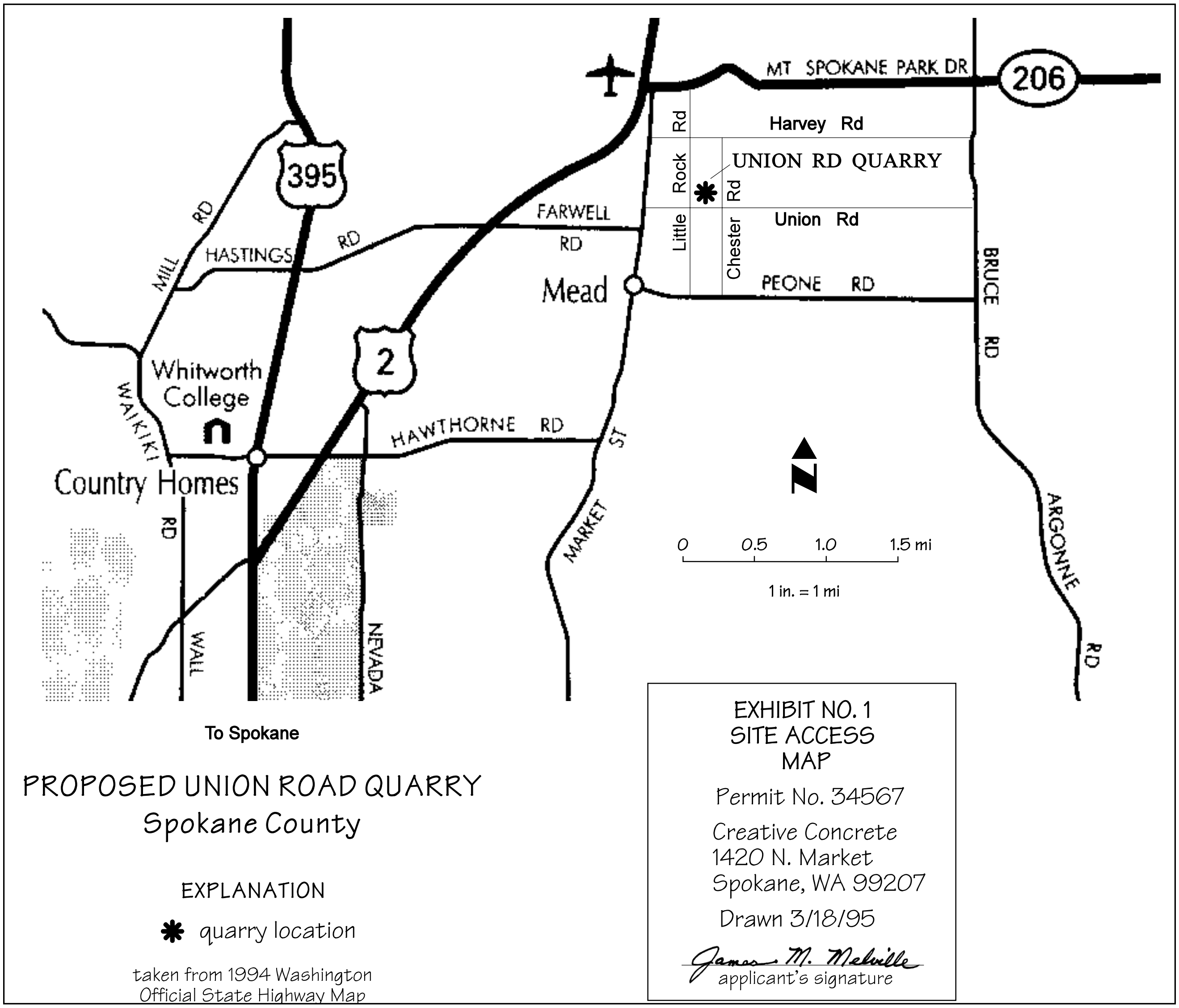


Figure 1.1. Site access map for the fictitious Union Road Quarry, taken from a highway map. Note verbal scale, bar scale, north arrow, and explanation and title blocks. (Not to scale; this map has been reduced to fit on the page.)

guished from permanent natural features. They may be represented by a unique line or pattern. (See Storm-Water and Erosion-Control Structures, p. 2.12.)

Typical Cross Sections

A cross section or profile shows what the mining site would look like if a vertical slice were taken through it. The purpose is to show the slope of the original land surface and reclaimed land surface, the water level of ponds and wetlands, and the types and placement of vegetation. Cross sections are usually taken through the areas that will show the most information. It is generally best if a cross section is drawn so that the vertical and horizontal scales are the same. In some cases, the vertical scale can be exaggerated to accentuate topographic features.

SITE ACCESS MAP

The site access map (Fig. 1.1) can be a copy or tracing of the pertinent part of a road map that clearly shows how to get to the site from

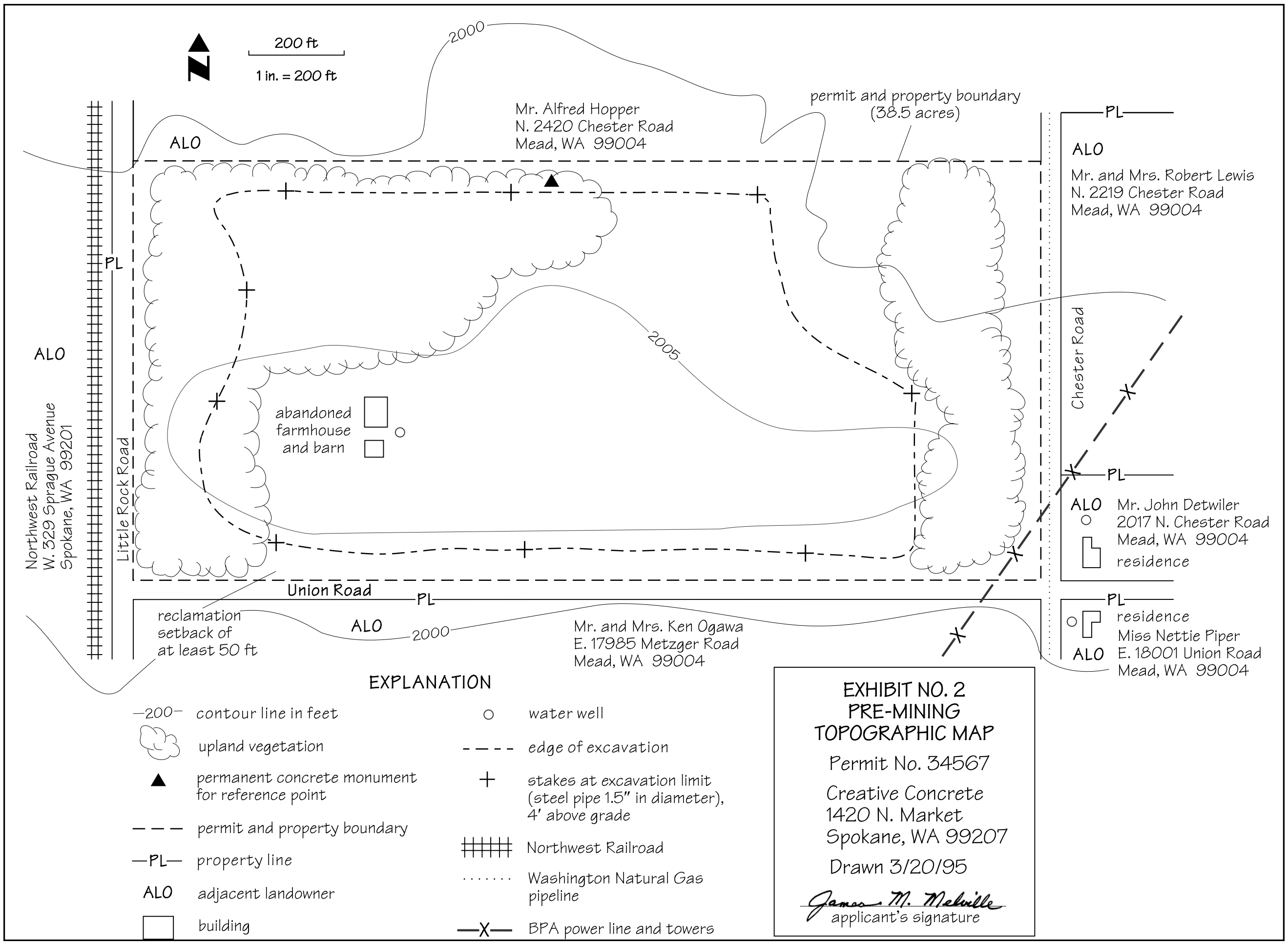


Figure 1.2. Pre-mining topographic map for the quarry in Figure 1.1. Note existing buildings and vegetation, pre-mining contours, verbal scale, bar scale, north arrow, and explanation and title blocks. (Modified from Norman and Lingley, 1992. Not to scale; this map has been reduced to fit on the page.)

the nearest town. The preferred size for this type of map is 8½ x 11 inches. A site access map shows the regional setting of the site and includes nearby geographical features and public road access to the site.

PRE-MINING TOPOGRAPHIC MAP

The pre-mining topographic map establishes the location and setting of the mine site (Fig. 1.2). It must show the following features:

- ❑ Permit area plus an appropriate border on all sides to show important adjacent features. The size of the border depends on site topography, drainage, neighbors, etc.
- ❑ Elevations and contours, natural ground slopes, drainage patterns, and other topographic features
- ❑ Boundaries and names of counties and municipalities (if they cross the map area)
- ❑ Boundaries of property ownership adjacent to the mine
- ❑ Names and addresses of adjacent property owners
- ❑ Locations and names of any other nearby mines

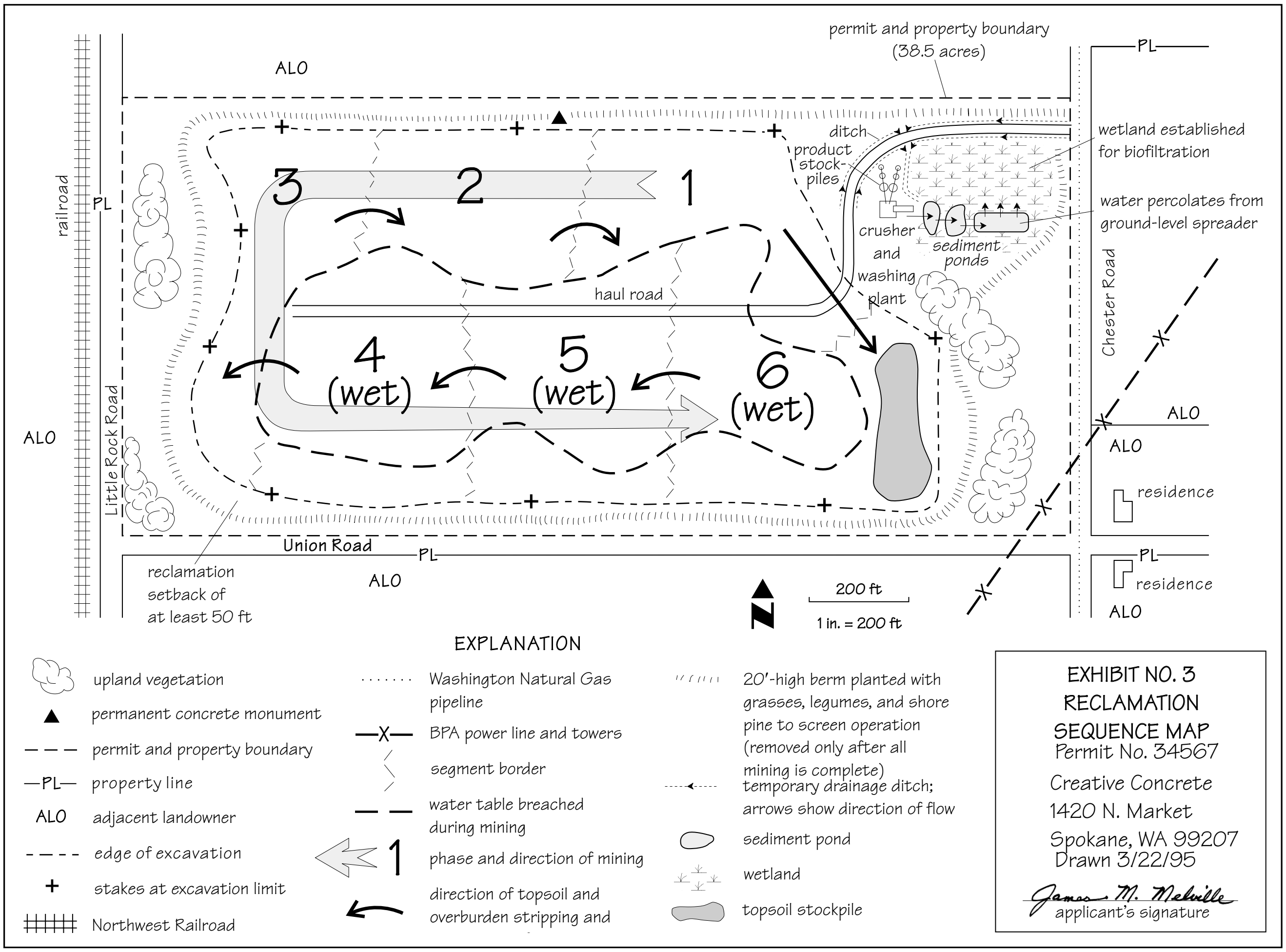


Figure 1.3. Reclamation sequence map for the site in Figure 1.2. This map shows the location and sequence of segments to be mined according to the operating and reclamation plan (counterclockwise from the northeast, in this instance), as well as details of soil placement, screening, and drainage. This site is mined first as a dry site, but as mining proceeds into the southern segments, the water table is penetrated. (Modified from Norman and Lingley, 1992. Not to scale; this map has been reduced to fit on the page.)

- ❑ Locations and names (if any) of all roads, railroads, utility lines, or any other rights of way
- ❑ Locations and names (if any) of all streams and natural and artificial drainways on or adjacent to the mine site
- ❑ Locations and names of significant buildings, parks, and other artificial features
- ❑ Locations and names (if any) of all wells, lakes, springs, and existing wetlands on or adjacent to the mine site
- ❑ Boundaries of the areas that will be disturbed by mining.

RECLAMATION SEQUENCE MAP

The reclamation sequence map shows the details of the plan for mining and segmental reclamation (Fig. 1.3). It should cover the same area as the pre-mining topographic map and display the following information:

- ❑ Permit area plus an appropriate border on all sides
- ❑ Boundaries of the areas that will be disturbed by mining

- ❑ Locations of all permanent boundary markers
- ❑ Locations of proposed access roads to be built in conjunction with the surface mining operation
- ❑ Locations and types of setbacks and berms
- ❑ Numbered segments and the direction and sequence of mining
- ❑ Soil storage areas and sequence of stripping, storing, and replacement on mined segments
- ❑ Overburden storage areas and sequence of stripping, storing, and replacement of overburden on mined segments
- ❑ Waste rock piles and how they will be reclaimed and stabilized
- ❑ Operation plant and processing areas
- ❑ Measures to be taken to protect adjacent surface resources, including prevention of slumping or landslides on adjacent lands
- ❑ Location and description of storm-water and erosion-control systems, including drainage facilities and settling ponds
- ❑ Other pertinent features.

FINAL RECLAMATION MAP

On most sites that require a state reclamation permit (reclamation plan), a description of the post-mining topography is usually sufficient, but for complex sites, post-mining topographic maps should be prepared (Fig. 1.4). This is a topographic map of the site as it will look after final reclamation, usually presented in the form of post-mining contour lines or post-mining pit outlines. It must show all applicable data required in the narrative portion of the reclamation plan and details of the mine reclamation. The map should cover the same area as the pre-mining topographic map, at the same scale, and should display the following information:

- ❑ Permit area plus an appropriate border on all sides
- ❑ Final elevations and contours, adjacent natural ground slopes, reclaimed drainage patterns, and other topographic features
- ❑ Locations and names (if any) of all roads, railroads, utility lines, or any other rights of way
- ❑ Locations and names (if any) of all streams and drainages
- ❑ Locations and names (if any) of significant buildings, parks, and other structures, facilities, or features
- ❑ Locations and names (if any) of all lakes, springs, and wetlands
- ❑ Location and depth of topsoil to be replaced
- ❑ Permanent drainage and water-control systems (with expanded view, if needed)
- ❑ Area to be revegetated and proposed species
- ❑ At least two cross sections (generally at right angles), with horizontal and vertical scales the same, that show the original and final topography and the water table (Fig. 1.5)

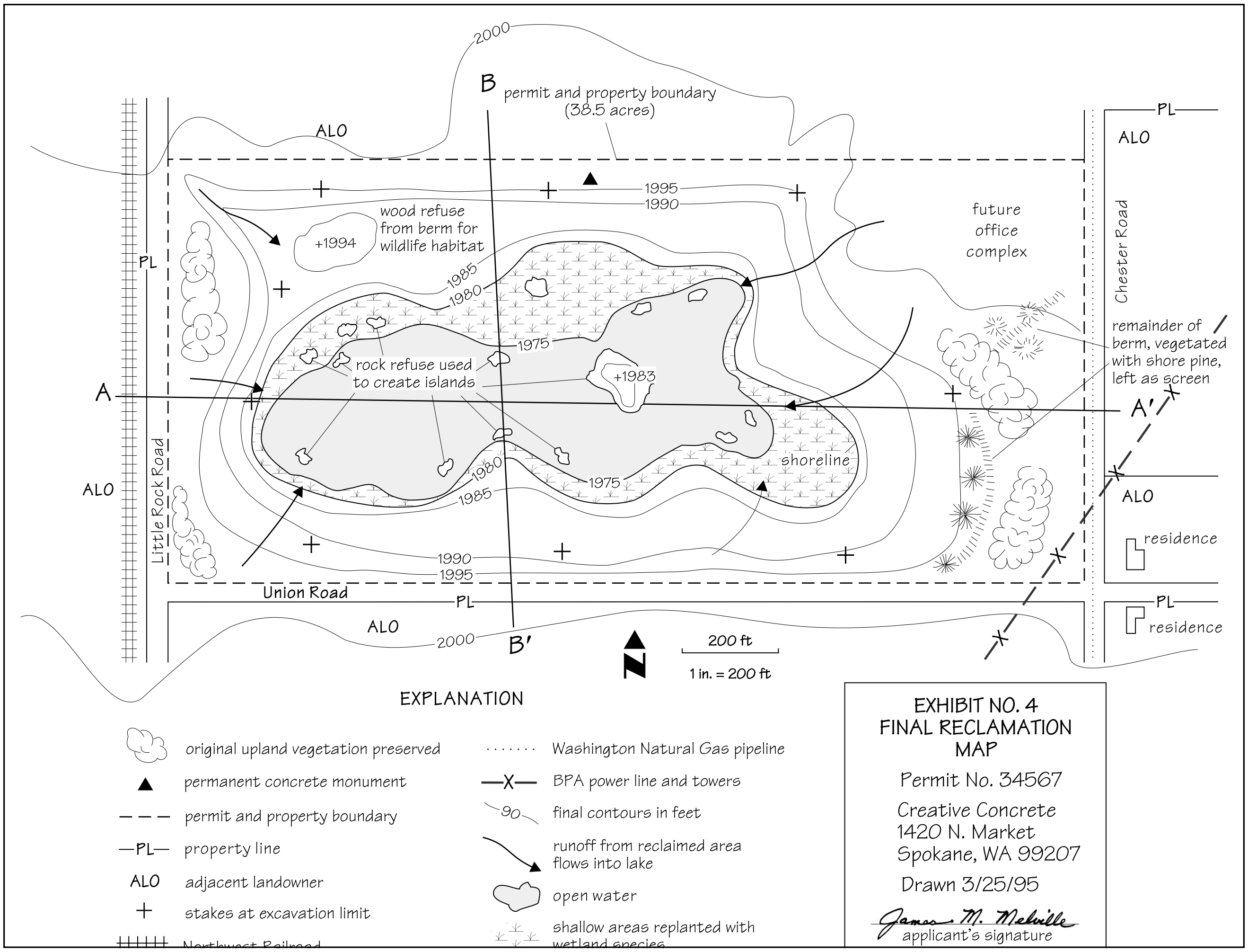


Figure 1.4. Final reclamation map of the site in Figure 1.2, showing how it will appear after reclamation. The site will accommodate a small office complex and wildlife habitat when it has been reclaimed. Cross sections A–A′ and B–B′ are shown in Figure 1.5. (Modified from Norman and Lingley, 1992. Not to scale; this map has been reduced to fit on the page.)

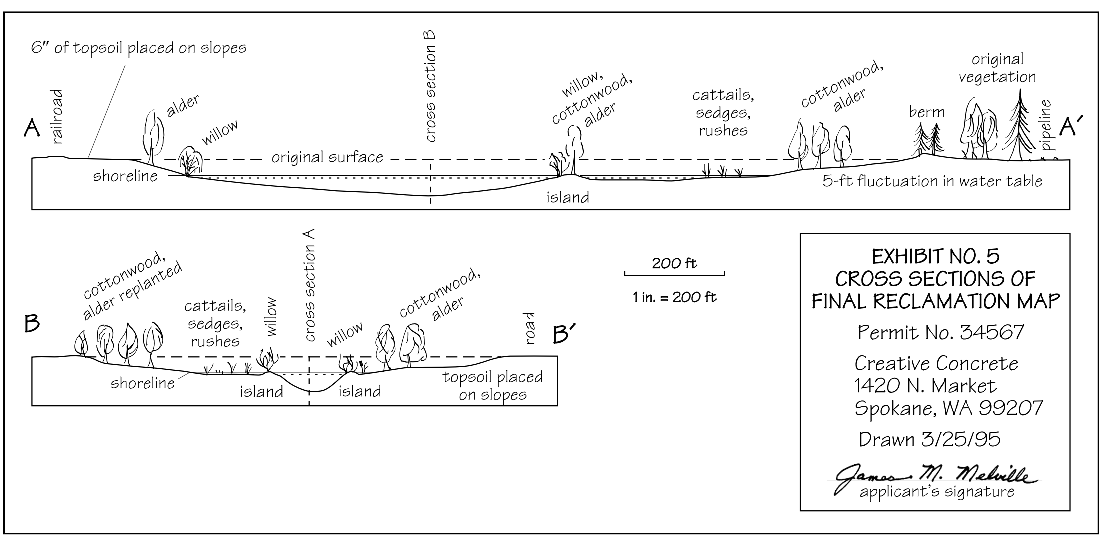


Figure 1.5. Cross sections for the final reclamation plan of the mine shown in Figure 1.4. The types and placement of vegetation and the slope of the pond banks are shown. (Modified from Norman and Lingley, 1992. Not to scale; this map has been reduced to fit on the page.)

- ❑ Other information pertaining to the permit and required by statute or special conditions of the permit.

GEOLOGIC MAP

In addition to the preceding four types of maps, a detailed description of the geologic setting and the type of deposit to be mined is sometimes required in geologically complex areas and for certain industrial mineral or metal mines.

MAP UPDATES

Current aerial photos or updated maps may be required as mining progresses.

REFERENCE

Norman, D. K.; Lingley, W. S., Jr., 1992, Reclamation of sand and gravel mines: Washington Geology, v. 20, no. 3, p. 20-31.

2 Storm-Water and Erosion Control

INTRODUCTION

Protecting water quality and preventing erosion are two important tasks mine operators must address. Federal legislation and increasing concern and scrutiny by state and local agencies and the public require that mine operators pay close attention to even small or temporary discharges of storm water. The quality of those discharges, particularly their turbidity, is a direct reflection of how sediment on the site is handled. Expensive solutions to water-quality problems can often be avoided by incorporating storm-water and erosion-control techniques into the mine development plan. For most mine sites, a good storm-water control system can minimize or even eliminate storm-water discharge during the operation phase. When mining ceases, erosion control is still necessary but should rely on techniques that can function without maintenance.

Controlling storm water and the erosion it causes requires integrated management starting at the top of the watershed above the mining area. No single action will produce permanently effective results. A good system has numerous individual components that must function separately but also respond as a unit during storms. The failure of one component can cause other components to fail and ultimately affect water quality. Furthermore, control practices are likely to change over the life of the operation. Good planning and constant maintenance are needed to keep the storm-water system working at peak efficiency.

This chapter describes basic techniques that can be combined to make a comprehensive storm-water and erosion-control system. Specific techniques appropriate to a given site depend on climate, topography, and the erodibility of the material present. The following general guidelines are applicable everywhere:

- ☛ Carefully plan the areas to be cleared in order to minimize disturbance.
- ☛ Retain sediment by using erosion-control BMPs.
- ☛ Interrupt the flow of surface water to reduce velocity.
- ☛ Use revegetation and mulching to stabilize cleared areas as soon as practical.
- ☛ Isolate fines produced during mining and processing.
- ☛ Develop a plan for maintaining storm-water and erosion-control structures. Follow the plan, and modify it as necessary to address changing conditions.

MAINTENANCE AND EMPLOYEE INVOLVEMENT

Although water quality is ultimately the operator's responsibility, maintenance of storm-water and erosion-control systems must be a priority for management and involve all mine employees. Managers should explain to staff why controlling storm water and erosion is so important. An effective program requires that everyone be on the lookout for seemingly insignificant situations that can snowball into major problems if not addressed in time.

We encourage operators and their employees to experiment with improving their storm-water systems. Operators should not feel limited to the information provided in this document. Common sense and innovation, with an emphasis on early recognition and response to erosion and sediment-transport problems, are the key to effective storm-water control.

EROSION

The rate of erosion is affected by four main factors (Fig. 2.1):

- *climate*, which determines how much rain and snow will fall on a site,
- *soil characteristics*, which determine erodibility and infiltration rates,
- *topography or slope*, which determines the velocity of runoff and the energy water will have to cause erosion, and
- *vegetation*, which slows runoff and prevents erosion by holding soils in place.

Each of these factors plays a role in determining which BMPs should be used to control erosion on a given site.

Erosion begins when raindrops displace soil particles. Raindrops may combine into sheets of water and flow over the surface (overland flow) to cause sheet erosion. Topography then concentrates water to produce rill and gully erosion. When water from rills and gullies combines, larger erosive streams and channels form (Fig. 2.2).

A single raindrop may move a splashed particle 2 feet vertically and 5 feet horizontally. The velocity of a raindrop is more than ten times higher than typical surface runoff velocities, which means that soil particles are more likely to be dislodged by raindrop impact than by surface runoff. Once the particles are mobilized, however, much less energy is required to keep them suspended or moving.

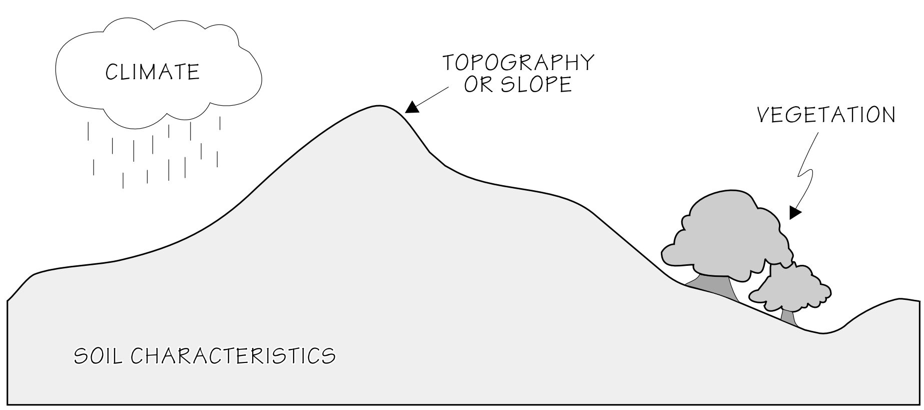


Figure 2.1. The rate of erosion depends on climate, soil characteristics, topography or slope, and vegetation.

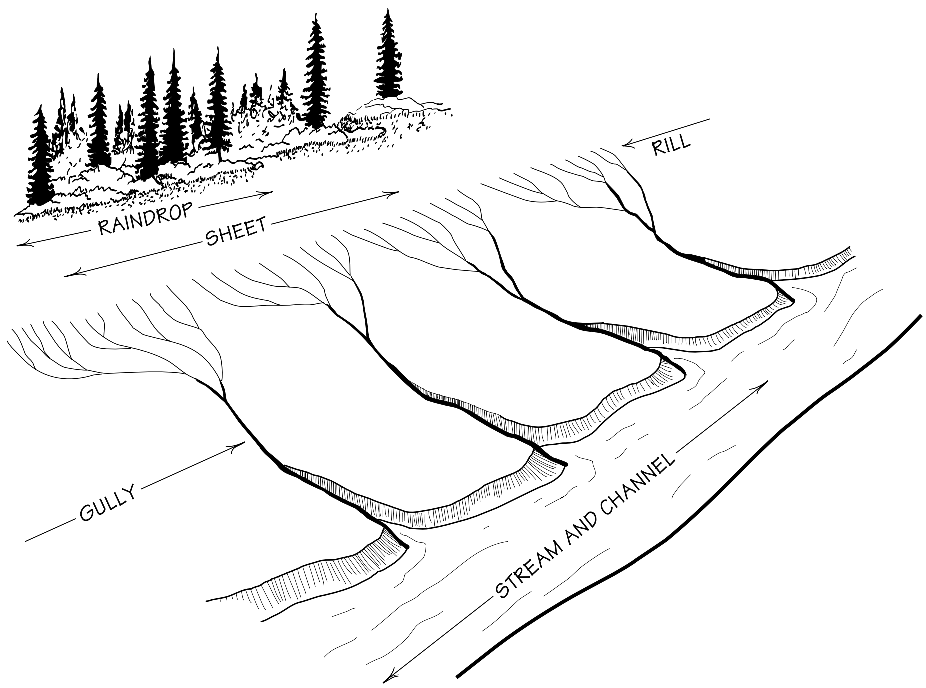


Figure 2.2. Topography created by different types of erosion. Raindrop erosion affects any bare surface. If the water does not infiltrate, raindrops combine into sheets of water (overland flow) to cause sheet erosion, and sheets further concentrate to produce rill and gully erosion. Water from rills and gullies then combines to form streams and channels. (Redrawn from Beckett, Jackson, Raedere, Inc., 1975.)

STORM-WATER REGULATION

The Washington Department of Ecology (DOE) and the Oregon Department of Environmental Quality (DEQ) regulate the discharge of storm water and waste water into public waters. The Stormwater Management Manual for the Puget Sound Basin (Washington State Department of Ecology, 1992) is a good source of 'best management practices' (BMPs) and is available from DOE.

For many mine sites, DOE requires a Stormwater Pollution Prevention Plan (SWPPP). As part of the SWPPP, an Erosion and Sediment Control Plan is required with the general discharge permit.

Mine sites in Oregon that discharge storm water off site need a Department of Environmental Quality (DEQ) storm-water permit, which can be obtained through DOGAMI-MLR. This typically requires the preparation of a storm-water plan to be submitted with the storm-water application. Sites that use water for processing and do not discharge water from the site must obtain a Water Pollution Control Facility Permit (WPCF permit) from DOGAMI-MLR. Sites that use water to process aggregate and discharge water from the site should contact DEQ to obtain an individual WPCF permit.

TURBIDITY AND SUSPENDED SEDIMENT

Erosion results in stream water that has high turbidity and a large sediment load. Turbid, sediment-laden water can adversely affect frogs and toads, clams, bottom-dwelling insects, and the appearance of stream systems. High levels of turbidity can also interfere with the feeding habits of fish, especially juveniles, and clog gills. Settleable solids can cover spawning gravels and suffocate eggs.

Turbidity

Turbidity is a measure of the amount of light that can pass through water in a straight line. Turbidity is reported as Nephelometric Turbidity Units (NTU). A high NTU value means that little light is transmitted through the water because it is absorbed or deflected by particles in the water.

Suspended Sediment

Suspended sediment is composed of settleable and nonsettleable solids. Settleable solids (sand- and silt-size particles) are heavier than water and will settle in calm water. Nonsettleable solids (clay-size particles) take a long time (or distance) to settle out of suspension—in some cases, years—and are the chief cause of turbidity.

In Washington, turbidity must not be more than 5 NTU greater than the background turbidity when the background turbidity is 50 NTU or less, or there must not be more than a 10 percent increase in turbidity when the background turbidity is more than 50 NTU. There is no standard for suspended solids or settleable solids in the water-quality regulations.

For example, in the sand and gravel general discharge permit, DOE is allowed by regulation to give a facility a 10:1 mixing zone to meet an effluent limit. DOE sets the end-of-pipe effluent limit at 50 NTU and assumes that the background level for turbidity in the receiving water is zero. With a 10:1 mixing zone, this should result in a 5 NTU final effluent quality at the end of the mixing zone.

In Oregon, all sites that have point-source discharges of storm water must have a storm-water discharge permit. As of January 1, 1998, storm-water discharge permits for mine sites will be administered by DOGAMI-MLR. The general storm-water permit contains performance benchmarks for storm-water plans. Benchmarks have been set for pH, total suspended solids, and oil and grease. If benchmarks are exceeded, the plan must be modified to address the deficiency. Turbidity must be less than 10 percent above the background of the receiving stream or river.

EROSION CONTROL

Assuming that the general guidelines given on p. 2.1 are being followed, the two most important things that can be done to minimize erosion, sedimentation, and turbidity are preventing raindrop erosion and slowing surface-water runoff velocities in the bare areas.

Practices that reduce erosion can be classified as either short- or long-term, although considerable overlap exists between the two. All require maintenance to be effective. They are described in detail later in this chapter.

Short-term erosion-control methods include:

- mulching,
- slash windrows,
- straw bales,
- filter fabric fences,
- jute netting and/or mulch fabrics,
- brush sediment barriers, and
- plastic coverings.

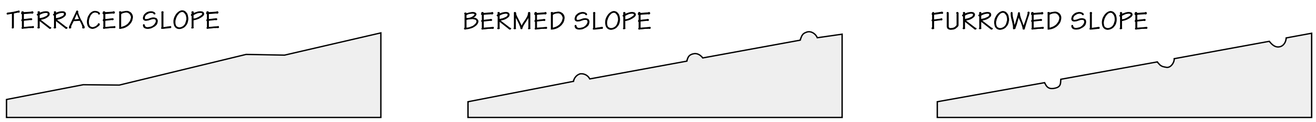


Figure 2.3. Small, discontinuous terraces, berms, and furrows can effectively slow runoff and decrease sediment transport. The relief is exaggerated for illustrative purposes. (From Banks, 1981.)

Long-term erosion-control methods include:

- vegetation,
- diversion ditches,
- rock check dams,
- rock-lined ditches, and
- contours, berms, swales, and ditches.

Controlling Raindrop Erosion

On flat ground, raindrop erosion is typically not a problem, but on slopes, more soil is splashed downhill than uphill. Covering steep slopes with plastic sheeting or mulch and/or revegetating bare areas reduces the erosion caused by raindrop impact. Gravel placed on berms or other bare areas at the plant site can also significantly reduce sediment movement during heavy rains.

Controlling Surface Runoff

Runoff velocities can be controlled by retarding flow and/ or breaking up or minimizing slope length. Retarding flow on a slope can be accomplished with organic debris or geotextiles. Small, discontinuous terraces, berms, and furrows on the overburden cut above the mine or on reclaimed slopes can effectively slow runoff and decrease sediment transport (Fig. 2.3). Benches cut in overburden or other unconsolidated material likely to erode should be sloped into the hillside and away from the center of the bench to allow drainage to either side (Fig. 2.4). For reclamation, benches and terraces should have shapes and dimensions that appear natural so they blend in with the landforms of the area.

Other methods for reducing runoff velocities involve long-term structures incorporated into the drainage-ditch system. (See Storm-Water and Erosion-Control Structures, p. 2.12.) These structures should be used in the interior of the mine in conjunction with settling

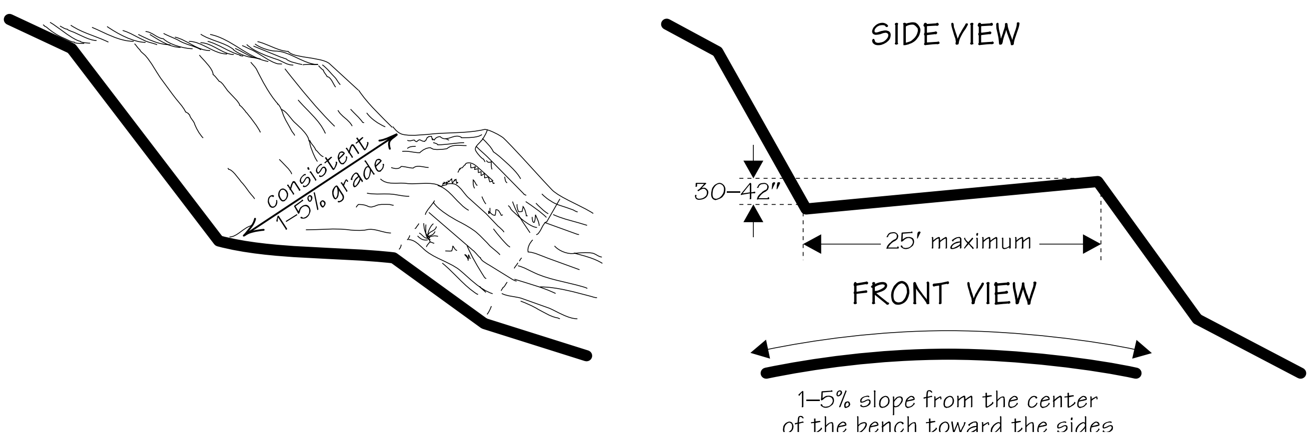


Figure 2.4. Benching and terracing of unconsolidated material to control runoff. Benches cut in overburden or other material likely to erode should be sloped into the hillside (side view) and away from the center of the bench (1–5% slope or grade) to allow drainage to either side (front view). (Modified from Law, 1984. Copyright © 1984 by Van Nostrand Reinhold Company Inc. Used by permission of the publisher.)

ponds. Using only one method is generally not successful. Attempting to trap or control sediment in settling ponds may not work unless some sediments have been dispersed and trapped upslope of the final pond or discharge point.

Long-term erosion-control methods are more cost-effective if properly planned and coordinated with mining activities. At many sites, short-term erosion control will be needed until long-term controls are established. Some methods, such as revegetation, can be effective in both the short and long terms.

STORM-WATER DIVERSION

Conventional storm-water control methods tend to concentrate flows using ditches, berms, and ponds. The best strategy for storm-water control, however, is to divert storm water and overland flow around the mining site and back into the original drainage (Fig. 2.5). Keeping 'clean' water separate from 'dirty' water is the easiest way to minimize the amount of water that has to be treated or contained. To do this, mine operators must know where and how much water enters the mine site during storms of various sizes. Depending on the size of the operation, the type and duration of precipitation, the type of material being mined, and the topography, passive control of storm water may be all that is needed.

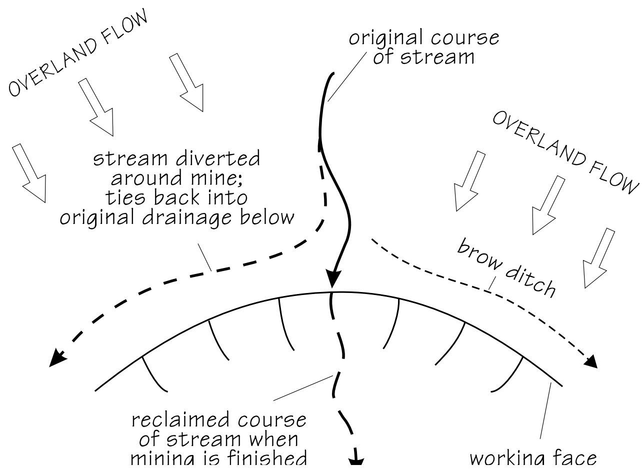


Figure 2.5. The best strategy for storm-water control is to divert streams and overland flow around the mining site. Not to scale.

If storm water cannot be diverted around the site, that water should be isolated from the storm water onsite to provide the best possible protection of surface waters.

PASSIVE STORM-WATER CONTROL

Passive storm-water control techniques rely on gravity to do their work. Their goal is to disperse storm water at numerous locations rather than to concentrate flows, which then have to be treated to remove sediment. Passive control structures are typically nonengineered and can easily be built at any mine site. They should be placed to prevent overland flow over any significant distance.

Small operations on permeable materials (such as sand and gravel, cinders, and pumice) and sites developed on flat or gently sloping terrain are good locations to use passive techniques. These techniques will also work on quarry sites where the rock is highly fractured and/or the size of the disturbance is fairly

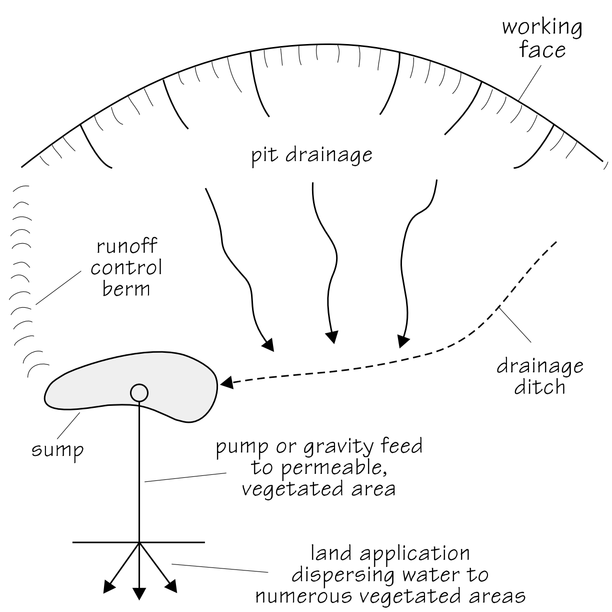


Figure 2.6. Berms and ditches divert runoff to a collection sump from which it can be dispersed into vegetated areas at numerous locations around the mine site. Not to scale.

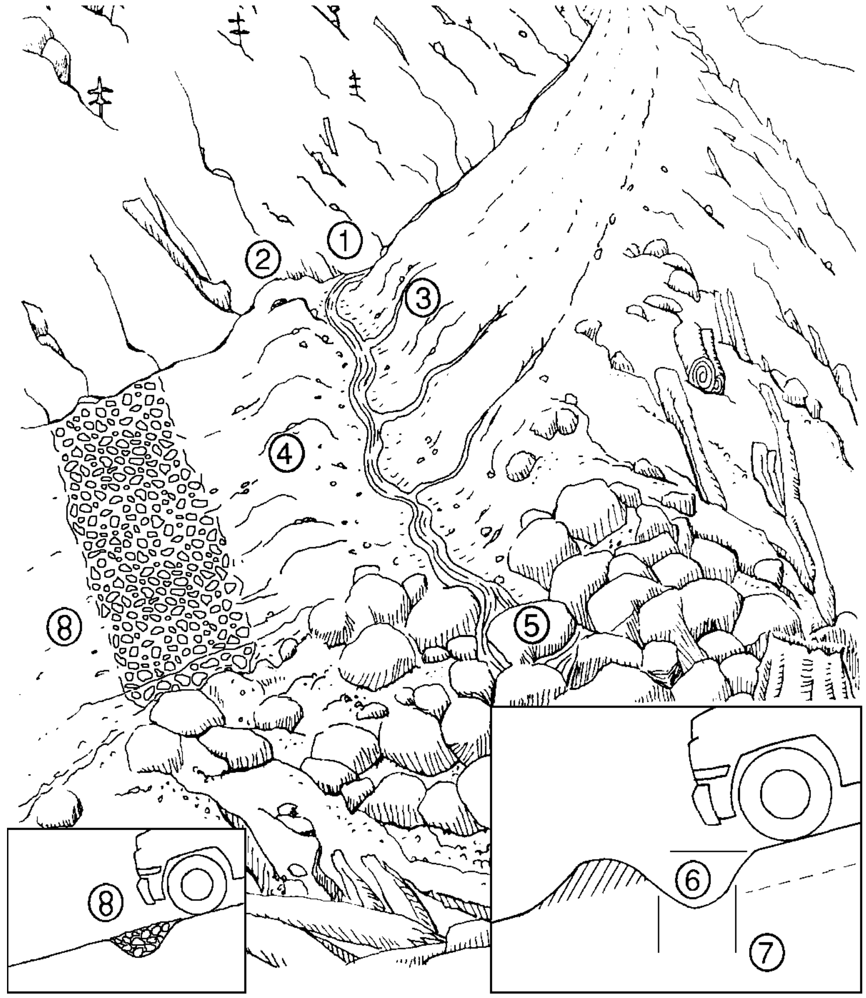


Figure 2.7. The water bar or cross-ditch intercepts, directs, and disperses surface-water flow off a road to stable sites on the downhill side of the road. 1, The cross-ditch is cut into the roadbed from the cutbank or ditchline completely across the road surface, extending beyond the shoulder of the road. 2, Physical blockage of the the ditchline is required to deflect water flow into the cross-ditch. 3, The cross-ditch should be placed at a minimum skew of 30^{o} to the ditchline—greater on steep road gradients. 4, The excavated material is spread on the downhill grade of the road, creating a berm. 5, Water should always be dispersed onto a stable slope with vegetation or riprap protection. 6, The cross-ditch berm should dip to allow vehicle crossover without destroying the ditch. 7, The cross-ditch must be cut to the depth of the ditchline to prevent water ponding and to ensure drainage from the ditchline. 8, An alternative to creating a water bar is to place a French drain in essentially the same configuration. The water bar and the French drain are shown together here for purposes of illustration. They would not normally be used so close together. (Modified from Chatwin and others, 1991.)

small. Passive techniques can and should be incorporated into designs for larger sites that require offsite discharge of storm water.

At most sites, roads and processing areas are the biggest sources of sediment because equipment is constantly being moved across them. Good road design and limiting traffic movement to specific areas can minimize disturbance and therefore sediment production.

The techniques suggested in the next few pages can reduce the amount of contaminated water that requires treatment prior to discharge offsite. Applying an appropriate combination of these techniques may eliminate offsite discharge of storm water altogether.

☛ Construct berms and ditches to divert runoff away from natural drainages and slopes and into vegetated areas around the mine site. If possible, select vegetated areas on gentle slopes. Doing this at numerous locations is the key to success (Fig. 2.6).

☛ Construct closely spaced water bars (Fig. 2.7) on roads susceptible to erosion, for example, ungraveled roads, roads with steep grades, and roads on highly erodible soils. Very little maintenance is required if water bars are properly constructed, placed in correct locations, and closely spaced.

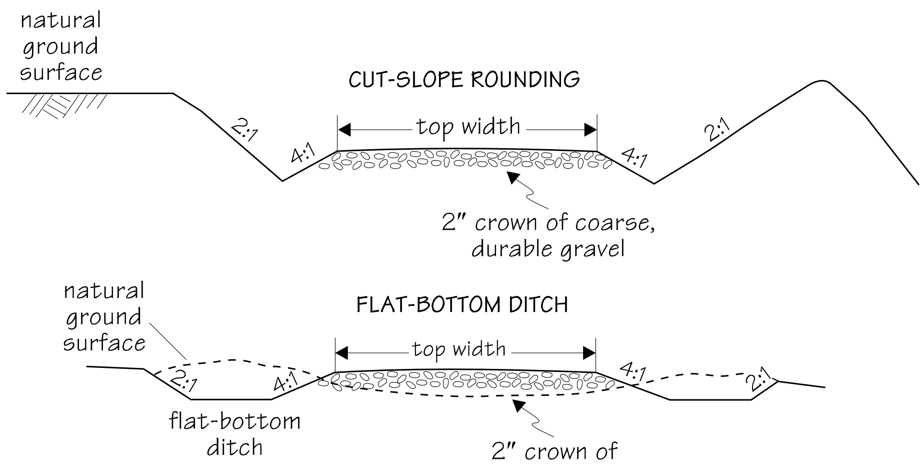


Figure 2.8. Profiles of elevated haul roads with drainage ditches on the sides to keep runoff from entering heavy traffic areas where it is more likely to pick up sediment. (Modified from U.S. Bureau of Land Management, 1992.)

Wide water bars, also called rolling ditches, can perform the same function as conventional water bars while providing smoother passage for vehicles.

- ☛ Use water bars on exploration roads above the mine cut or other roads that receive only occasional use.
- ☛ Elevate frequently used roads (Fig. 2.8), such as haul roads, and other heavy traffic areas to keep runoff away from these areas where it is more likely to pick up sediment.
- ☛ Make sure roads are well covered with durable, coarse rock of appropriate size.
- ☛ To retain storm water on wide working benches during the winter, use temporary berms.
- ☛ On the pit or quarry floor, establish and maintain a slope that allows turbid water to drain toward a low point where it can be collected in a pond or a sump to allow water to infiltrate (Fig. 2.9). This practice stops sediment-laden sheetwash from leaving the pit and may create beneficial wetlands after

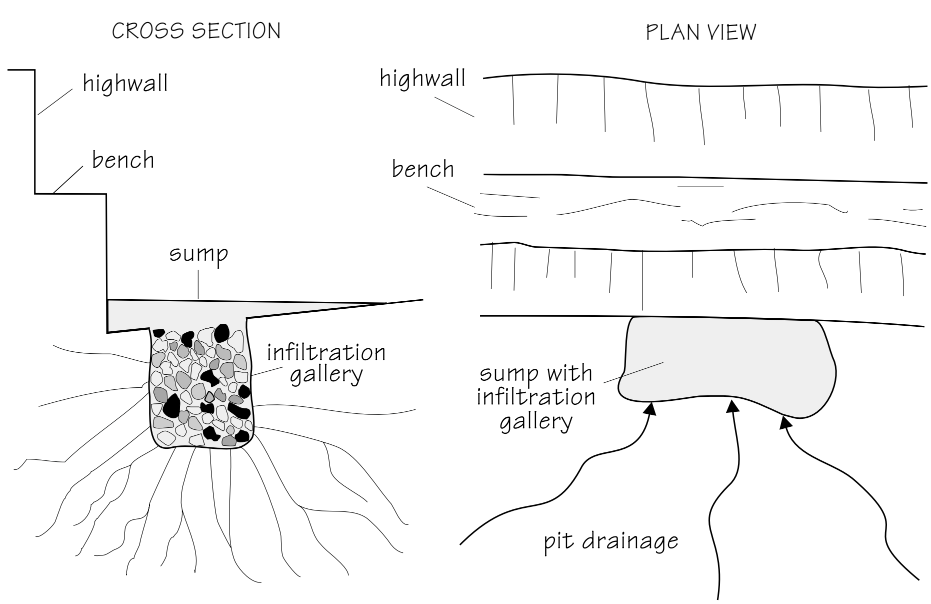


Figure 2.9. Establish and maintain a slope that allows water to drain toward the highwall to collect sediment and help form wetlands or to allow water to infiltrate (note infiltration gallery) if the area must be drained. This practice is not recommended if oil and grease are present as potential ground-water contaminants. Discharge to ground water may require a permit. (See also Fig. 2.26.)

reclamation. However, this method is not recommended if oil and grease are present to contaminate ground water.

- In both excavation and processing areas, develop and maintain places that will readily accept runoff and precipitation. For hard-rock sites, fracture the quarry floors and/or leave shot rock in place. For gravel and soft-rock quarries, rip and/or minimize areas compacted by heavy equipment.
- When processing rock on the excavation floor, make sure adequate drainage is provided. Fines produced during processing will potentially decrease permeability and increase runoff. This will likely result in an increase in the amount of turbid water to be treated.
- Use filter berms built of porous materials, such as sand and gravel or processed quarry rock that contains no 200-mesh or smaller material, to remove sediments. (See p. 2.19.)
- Use dry wells or infiltration galleries and horizontal subdrains to allow storm water to infiltrate into the ground rather than run off the site. (See p. 2.20 and 2.20.)
- Regrade, reshape, revegetate, and otherwise protect areas that have the potential to produce runoff or sediment.
- Minimize the disturbed area by maximizing the area reclaimed each fall.
- Establish and maintain vegetated buffer strips between disturbed areas and any natural drainage. Silt fines may be incorporated into the soil in these areas.
- Minimize the amount of water requiring treatment by isolating ground water from storm water. Sumps and trenches or shallow wells at the lowest point of the excavation can dewater the mine area prior to mining.

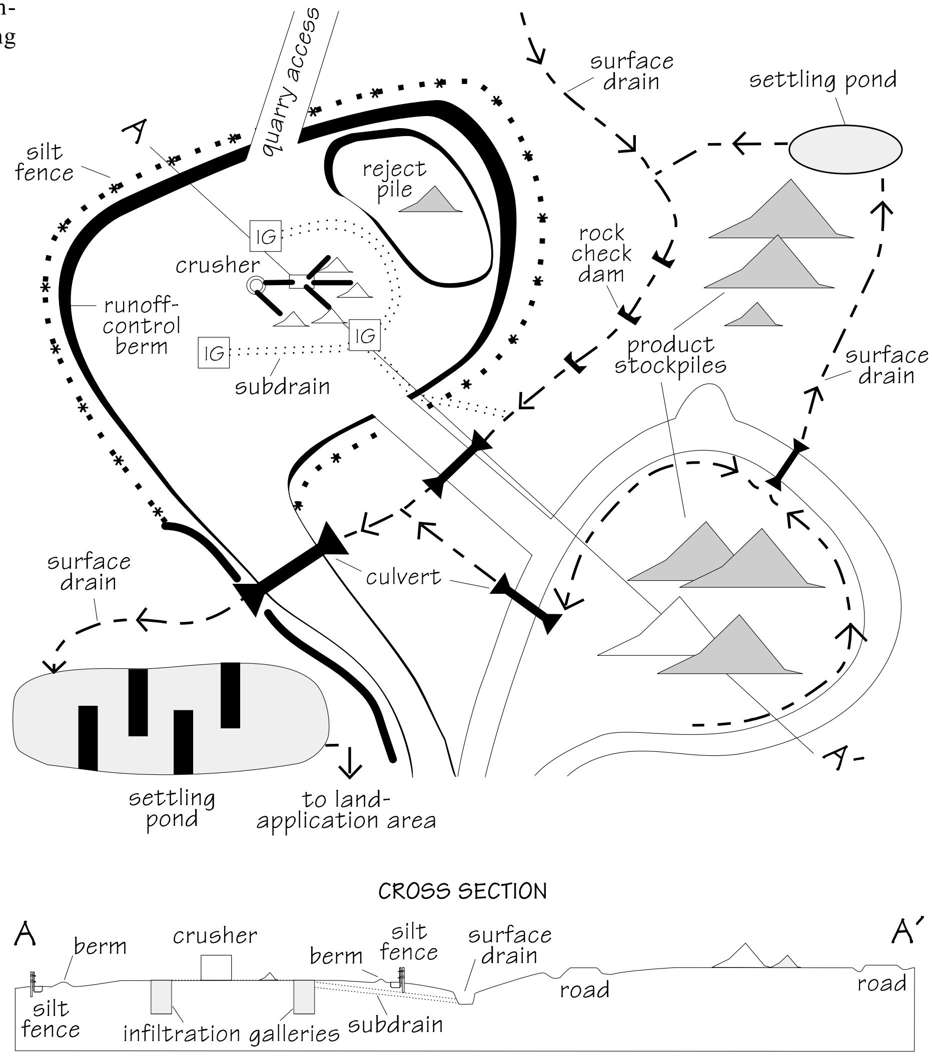
In Washington, any process water to be discharged to ground is regulated by the Department of Ecology. This includes process water discharged to dry wells and drain fields.

SEDIMENT CONTROL ON THE MINE SITE

If sediment gets into the water onsite, it can become an environmental contaminant requiring treatment. Removing soil fines from water can be a difficult and costly process. The best approach is to isolate the source of the sediment. Passive storm-water controls can reduce or eliminate suspended fines before they reach the settling pond system. Undersize or reject fines may be a saleable aggregate product and, in some mines, may be an appropriate or necessary soil amendment for reclamation. (See Replacing Topsoil and Subsoil, p. 4.5.)

Soils with sand as the dominant particle size are coarse-textured, light, and easily erodible. Water soaks into these soils rapidly. Silts and clays make fine-textured, heavy soils that are slow to erode and slow to drain. Clay-rich soils commonly cause the greatest im-

Figure 2.10. Hypothetical storm-water control at an upland processing area. IG, infiltration gallery.



pacts on water quality because they contain fine particles that settle slowly, travel far, and remain in suspension for a long time in settling ponds. Soils dominated by the clay fraction may require several large settling ponds in series. Flocculants can help settle clay particles. (See Flocculants, p. 2.26.)

One of the best methods for removing sediment from water is onsite land application. Turbid water is sent through dispersal systems that allow it to slowly soak into vegetated areas. The potential downslope/downstream impacts of land application should be assessed before constructing this type of control. (See Land Application, p. 2.25.)

For effective sediment control, operators need to determine both the dominant particle size of the source materials and the amount of precipitation and/or storm flow that can be anticipated. Particle-size analysis of soil, overburden, and reject fines produced from processing may be necessary at some sites to determine if they are likely to erode into the storm-water system. Ideally, representative storm-water runoff from the site or from a similar site (if mining

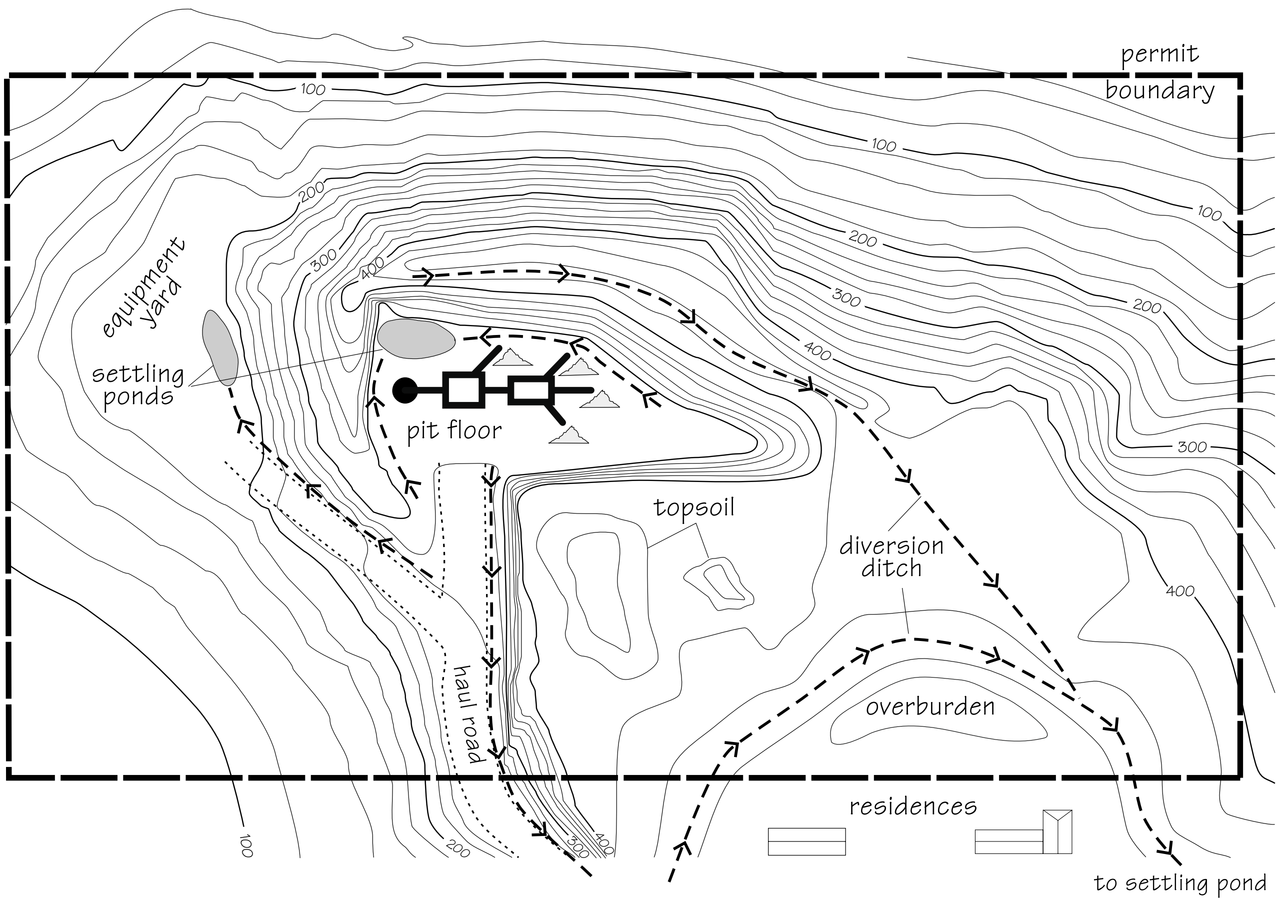


Figure 2.11. An example of a storm-water control system at a quarry site. Figure 3.5 shows visual and noise screening techniques at the same site.

has not yet started) should be sampled to predict the size range of the suspended particles that may require treatment.

The two basic methods of removing sediments are by filtering and by gravity separation. Filtering may be accomplished by using:

- designed sand, gravel, or rock graded filters with appropriate size gradations and layers,
- undisturbed soils or embankments,
- filter fabrics,
- infiltration galleries,
- French or trench drains, and
- dispersal (sheet flow) through vegetated areas.

Gravity separation requires that water velocity be reduced to facilitate settling. Settling ponds or dispersal on flat terrain (as in land application) use gravity separation. In still water, a sand particle (0.05–2 mm) will settle at rates of 1 foot/second to 1 foot/several minutes. A silt particle (0.05–0.002 mm) may take several minutes to 6 hours to settle 1 foot. Clay particles (<0.002 mm) can take from 1 day to several months to settle. Pond surface area, retention time, and the particles' settling velocity determine the effectiveness of a settling pond system.

STORM-WATER AND EROSION-CONTROL STRUCTURES

The techniques discussed above and the structures described below can be organized in many different ways. The erosion/sedimentation controls at a site will likely change over time as the configuration of the site changes. Examples of storm-water control systems for an upland processing area and a quarry floor are shown in Figures 2.10 and 2.11, respectively. The profile shown in Figure 2.10 illustrates possible proper drainage techniques in a processing area. The location and choice of the various structures and techniques are site- specific.

Conveyance Channels and Ditches

Channels and ditches are permanent, designed waterways shaped and lined with appropriate vegetation or structural material to safely convey runoff to a sediment pond, vegetated area, or drainage. The advantages of open channels are that they are generally inexpensive to construct, can be lined with vegetation, and make it easy to trace the water. One disadvantage of grass-lined channels is that they may, if improperly designed, erode during high flows and become a source of sediment themselves.

The design of a channel or ditch cross section and lining is based primarily on the volume and velocity of flow expected in the channel. If flow is low and slow, grass channels are preferred to riprap or concrete lining. Although concrete channels are efficient and easy to maintain, they allow runoff to move so quickly that channel erosion and flooding can result downstream. Grass-lined or riprap channels (Fig. 2.12) more closely duplicate a natural system. Riprap and grass-lined channels, if designed properly, also remove pollutants via biofiltration (removal of pollution by plants). Engineered channels are recommended when the discharge will be greater than 50 cubic feet per second.

In addition to the primary design considerations of capacity and velocity, other important factors to consider when selecting a cross section and lining are land availability, compatibility with surrounding environment, safety, maintenance requirements, and outlet conditions.

Slash Windrows and Brush Sediment Barriers

Most mine sites have to be cleared of woody vegetation prior to mining. Slash windrows and brush barriers can be easily and inexpensively constructed with the vegetative debris. These are effective for filtering coarse sediment and reducing water velocity.

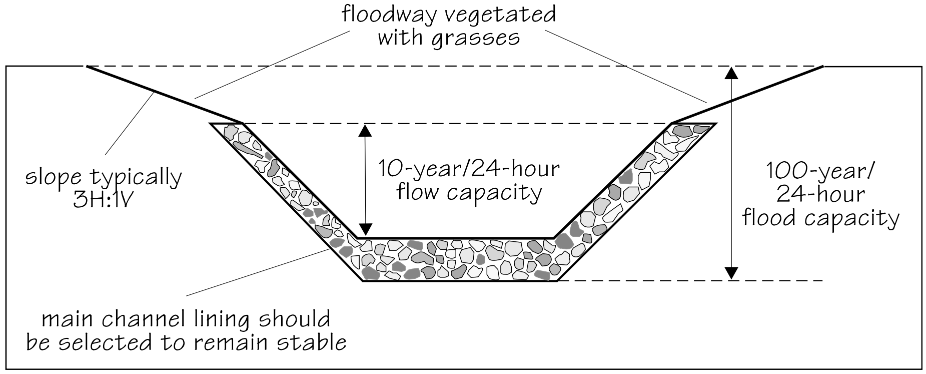


Figure 2.12. Details of construction for a rock-lined diversion ditch.

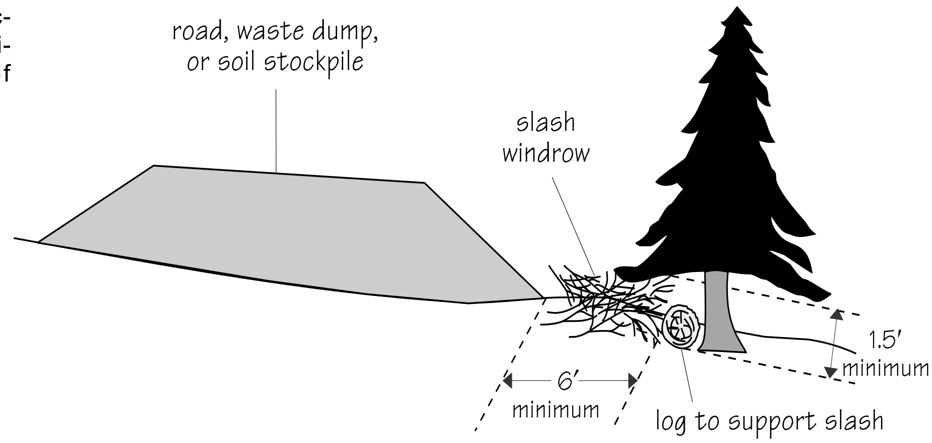


Figure 2.13. Details of construction of a slash windrow filter. (Modified from Idaho Department of Lands, 1992.)

Slash windrows are constructed by piling brush, sticks, and branches into long rows below the area of concern. The windrow may be supported at the base by large logs or rocks (Fig. 2.13).

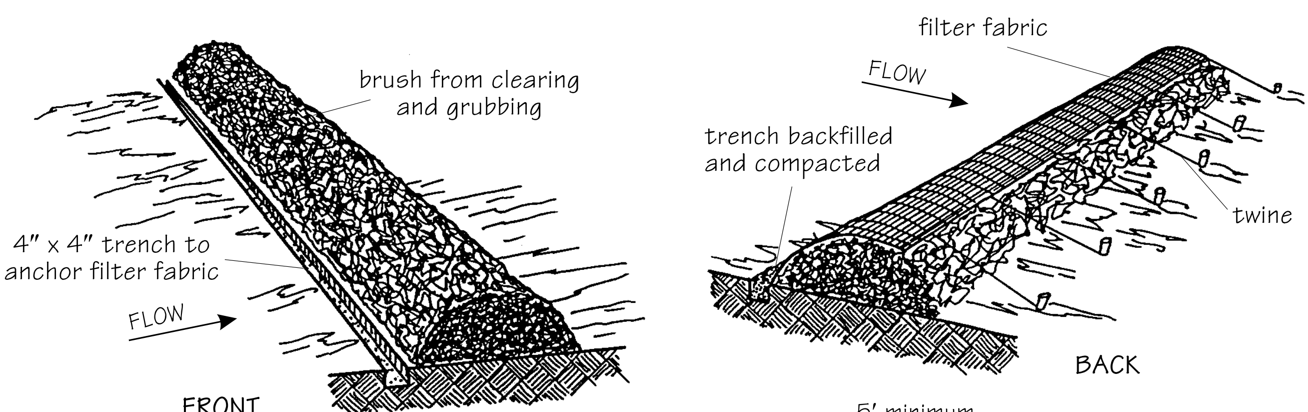
Brush sediment barriers require somewhat more effort, planning, and expense, but they are generally more effective than slash windrows. Brush sediment barriers are linear piles of slash, typically wrapped in filter fabric or wire mesh. Construction details are provided in Figure 2.14.

☛ Slash windrows should be used below roads, overburden and soil stockpiles, and any other bare areas that have short, moderate to steep slopes.

☛ Brush sediment barriers are most effective on open slopes where flow is not concentrated; they can help prevent sheet flow and rill and gully erosion during heavy rains.

Straw Bales

Straw bales are a well-known temporary erosion-control method (Fig. 2.15). They are fairly cheap and readily available. However, they are frequently installed incorrectly, making them ineffective.



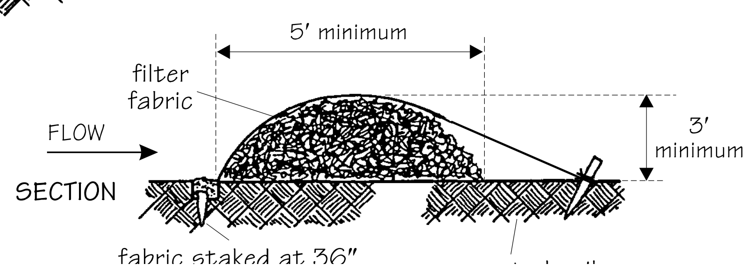


Figure 2.14. Details of brush sediment-barrier construction. The effectiveness of brush barriers is greatly increased when filter fabric is used. (Adapted from Idaho Department of Lands, 1992.)

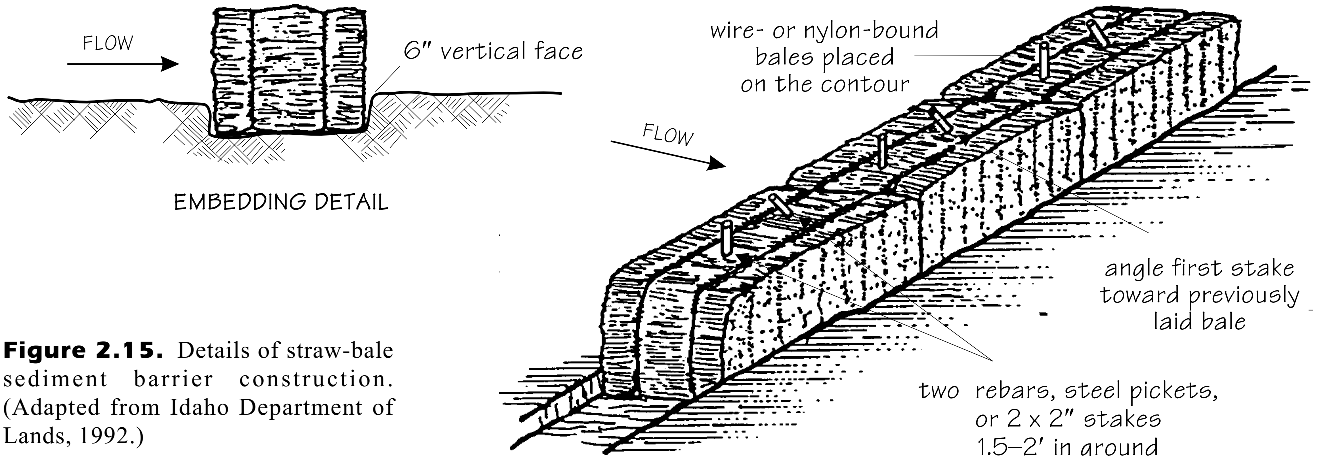


Figure 2.15. Details of straw-bale sediment barrier construction. (Adapted from Idaho Department of Lands, 1992.)

Simply placing straw bales on the ground surface without proper anchoring and trenching will provide only minimal erosion control. Proper ground preparation, placement, and staking are necessary to provide a stable sediment barrier. Straw bales also require frequent repair and replacement as they become clogged with sediment. Only certified weed-free straw should be used.

Straw bales used in conjunction with a check dam or filter berm constructed of sand and gravel, as shown in Figure 2.16, provide a more effective erosion-control system that requires less maintenance and can handle larger volume flows.

☛ Straw bales are most practical below disturbed areas where rill erosion occurs from sheet runoff.

☛ Straw bales may be used in minor swales and ditch lines where the drainage area is smaller than 2 acres and/or where effectiveness is required for less than 3 months.

Bio Bags Bio bags are woven nylon net bags filled with bark chips. They are about the size of straw bales and can be used as an alternative to straw bales for erosion control. Bio bags are much lighter than straw

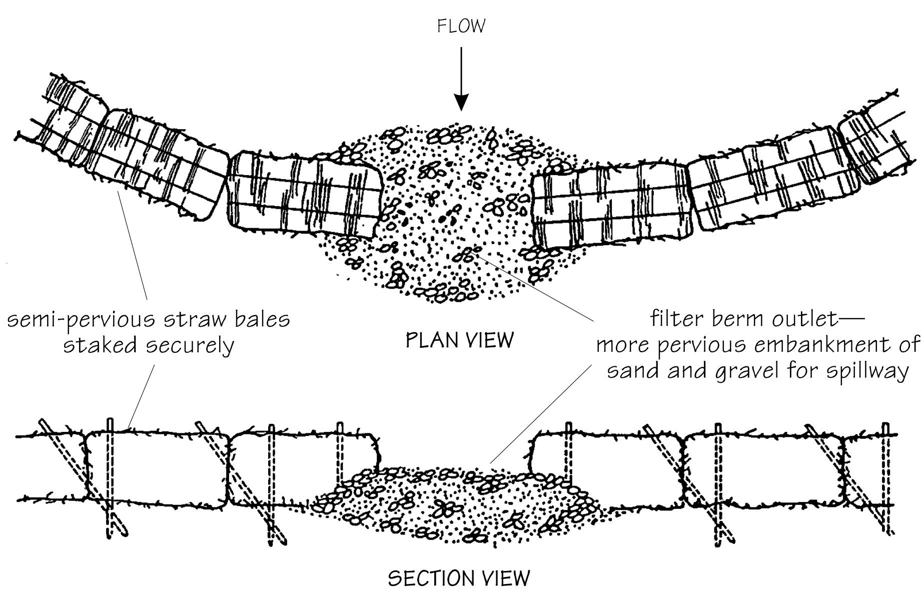


Figure 2.16. Details of construction for a straw-bale barrier combined with a gravel check dam. (Adapted from Idaho Department of Lands, 1992.)

bales; they must be staked down to keep them in place. They are more permeable, but slow water sufficiently to cause sand, silt, and clay to drop out. They fit the contours of the land, avoiding the bridging problem of straw bales. They hold together better and can therefore be removed more easily when saturated. Wildlife won't tear them apart to eat them, and they will not introduce grass and weed seeds to the site.

Bio bags may not be as readily available as straw bales. Their unit price is comparable to that of straw bales, but because they are smaller, more units are needed per application, making them slightly more expensive. They are not as biodegradable as straw bales.

Burlap Bags Filled with Drain Rock

Woven burlap bags filled with drain rock can be used as an alternative to bio bags. They conform well to irregular ground and are easily installed. They do not need to be staked down and are less prone to washing away than bio bags. They can easily be created using recycled burlap bags and the aggregate that is already present on most mine sites.

Silt Fences

A silt fence is made of filter fabric that allows water to pass through. Woven fabric is generally best. Depending on its pore size, filter fabric will trap different particle sizes. The fence is placed perpendicular to the flow direction and is held upright by stakes (Fig. 2.17). A more durable construction uses chicken wire and T-posts to support the fabric vertically.

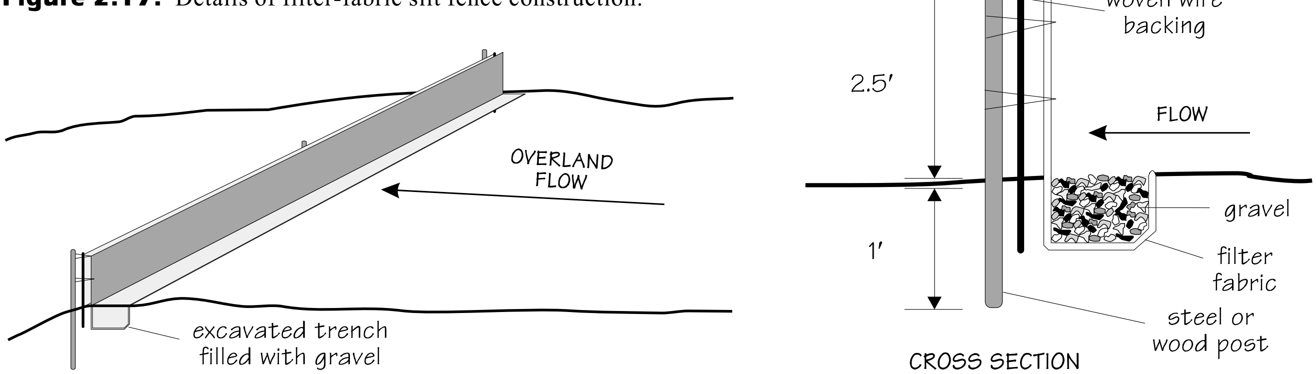
It is essential to bury the bottom of the filter fabric to prevent flow under or around the fence. Maintenance is required to keep the fence functioning properly. Rock check dams or other methods may be needed to slow water enough to allow it to pass through the fence. Although silt fences are more complicated and expensive to install than straw bales, they provide better erosion control in some situations, for example, in coastal climates where hay bales decay rapidly or in locations that are difficult to access with vehicles.

☛ Silt fences should be used below disturbed areas where runoff may occur in the form of sheet and rill erosion.

Erosion-Control Blankets

Erosion-control blankets are made of a variety of artificial and natural materials, including jute, coconut husk fibers, straw, synthetic

Figure 2.17. Details of filter-fabric silt fence construction.



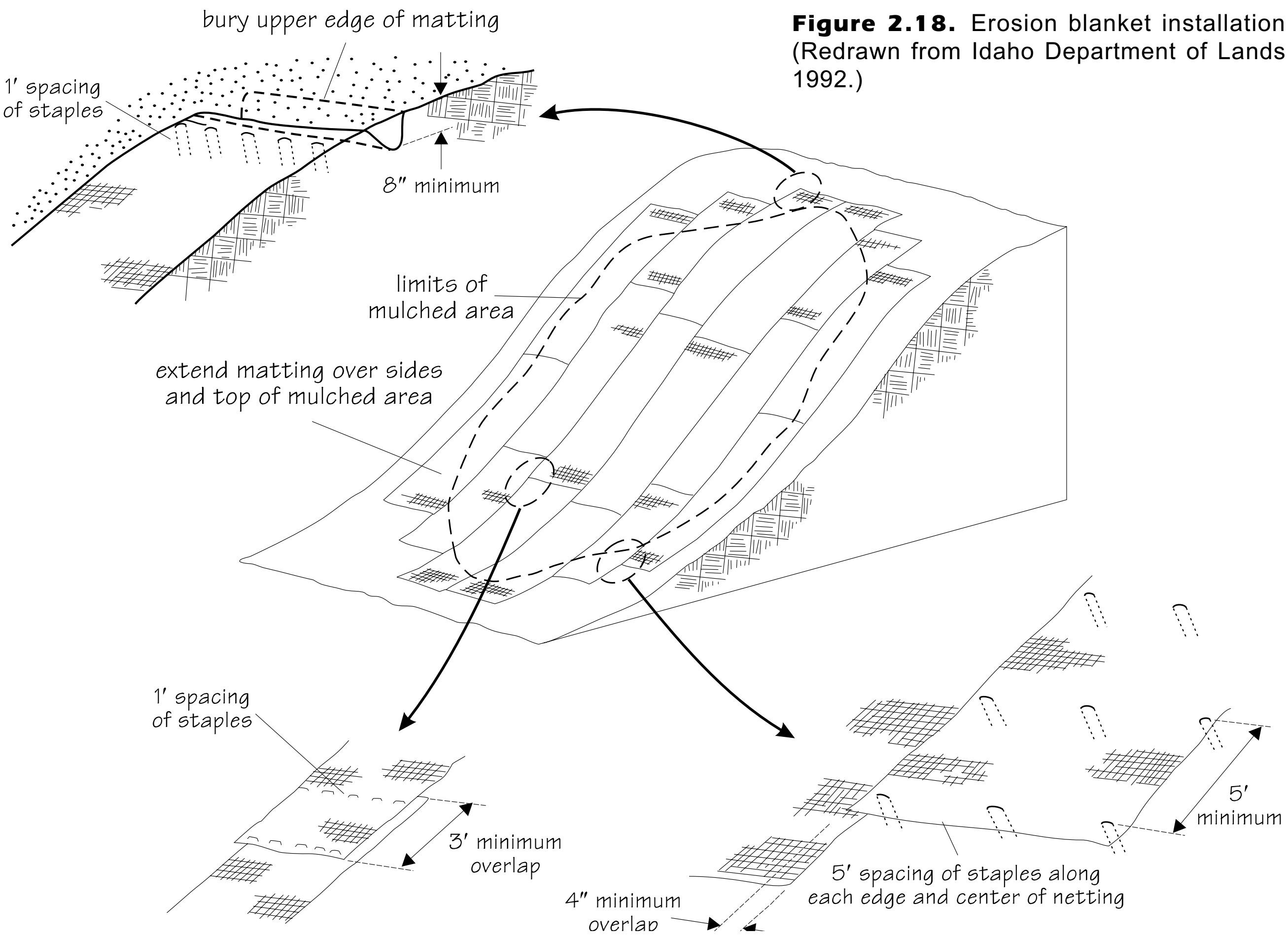


Figure 2.18. Erosion blanket installation. (Redrawn from Idaho Department of Lands, 1992.)

fabrics, plastic, or combinations (Fig. 2.18). Applying erosion blankets over large areas can be prohibitively expensive. However, small applications in areas that are oversteepened and/or prone to erosion, in conjunction with cheaper methods such as hydromulching and/or hay mulch and netting, can be very effective. The effectiveness of jute netting and mulch fabrics is greatly reduced if rills and gullies form beneath these fabrics. Therefore, proper anchoring and ground preparation are essential.

☛ Erosion-control blankets can be used on steep slopes where severe erosion-control problems are anticipated.

Where water infiltration is not desirable, for example, on the surface of an active landslide, an impermeable erosion blanket may be appropriate. In this situation, special care must be taken to provide a place where the energy the water has gained can dissipate, such as a slash windrow, brush sediment barrier, or rock blanket at the base of the slope.

Vegetation

Vegetation absorbs some of the energy of falling rain, hold soils in place, maintains the moisture-holding capacity of the soil, and reduces surface flow velocities (Fig. 2.19).

☛ The most effective way to use vegetation is to leave it undisturbed to prevent erosion and reduce the speed of surface water flows.

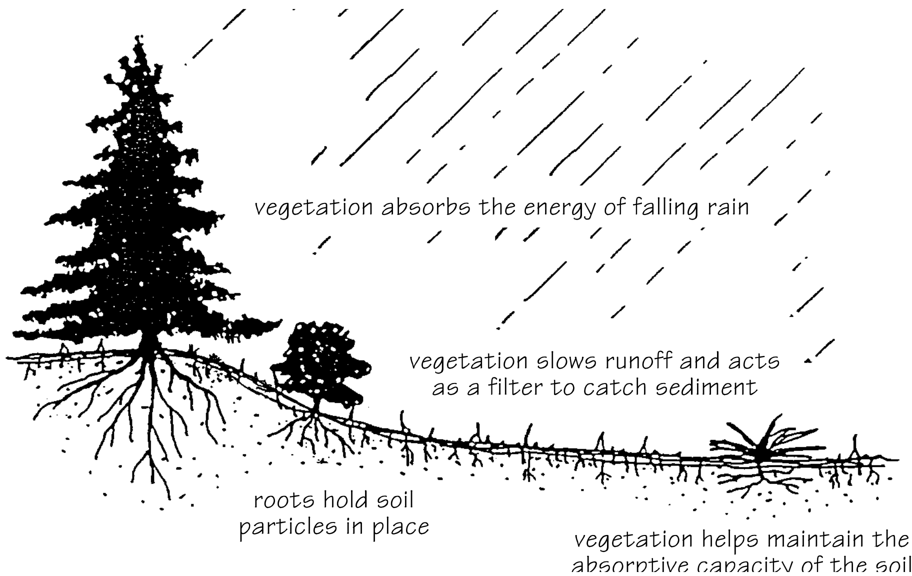


Figure 2.19. Effect of vegetation on storm-water runoff. (Modified from Washington State Department of Ecology, 1992.)

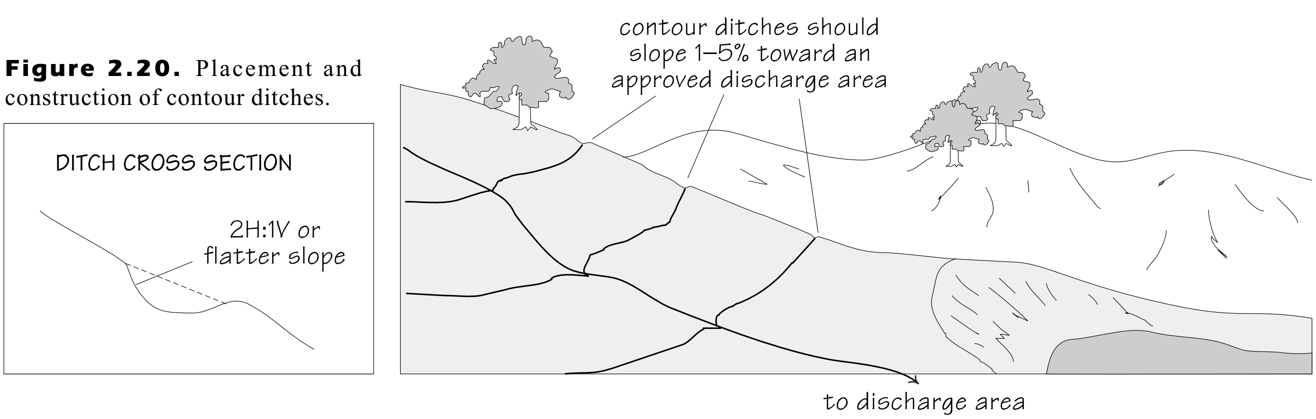
- ☛ If a new area must be cleared for mining, clear only the amount needed for expansion within one year.
- ☛ As an area is cleared of vegetation, save the sod or slash and stake it down across the cleared slopes to temporarily reduce storm-water runoff until the area is mined.
- ☛ Replace topsoil and replant mined areas as soon as possible.
- ☛ Revegetate overburden and topsoil stockpiles over the winter or when they will remain unused for more than six months. (Topsoil should not be replaced in this situation; see Interim Reclamation, p. 3.1.)

Contour and Diversion Ditches

Contour ditches are constructed along a line of approximately equal elevation across the slope (Fig. 2.20). Diversion ditches guide water around unstable areas to prevent both erosion and saturation with water (Fig. 2.21), reducing the likelihood of slope failure. Both types of ditches should have a 1 to 5 percent grade directed away from steep slopes to the appropriate drainage or vegetated areas.

Ditch channels may need to be lined to prevent scouring and minimize sediment transport. When their slope is greater than 5 per-

Figure 2.20. Placement and construction of contour ditches.



cent, ditches are typically lined with rock. Where slope stability is of concern, impermeable liners may be used. Rock check dams, described below, should be placed in diversion and contour ditches at decreasing intervals as the slope increases.

☛ Contour and diversion ditches should be used to direct surface runoff away from disturbed areas and prevent rills and gullies from forming.

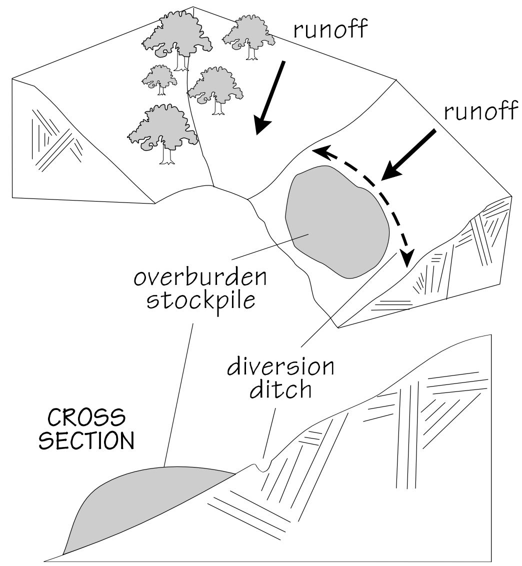


Figure 2.21. A diversion ditch can be placed upslope from an overburden pile to prevent saturation of the pile.

Rock and Log Check Dams

Check dams are typically constructed from coarse crushed rock ranging from about 2 to 4 inches in diameter, depending on the water velocities anticipated. A check dam can generally withstand higher velocity flows than a silt fence, and the integrity of the structure will not be affected if it is overtopped in a large storm event. The tops of check dams are lower than the channel margins so that water can spill over (instead of around the sides) during heavy storms (Fig. 2.22).

The effectiveness of rock check dams for trapping sediment can be improved by applying filter fabric on the upstream side. The bottom of the fabric must be anchored by excavating a trench, applying the fabric, and then filling the trench with coarse rock. This structure functions like a silt fence, but it is more durable. Choosing the proper size of filter fabric mesh is important to minimize clogging.

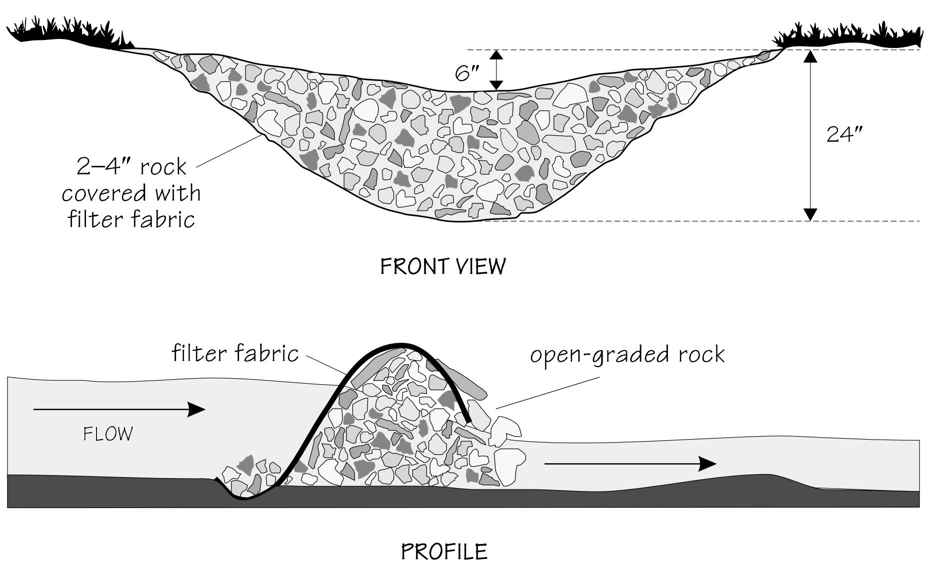
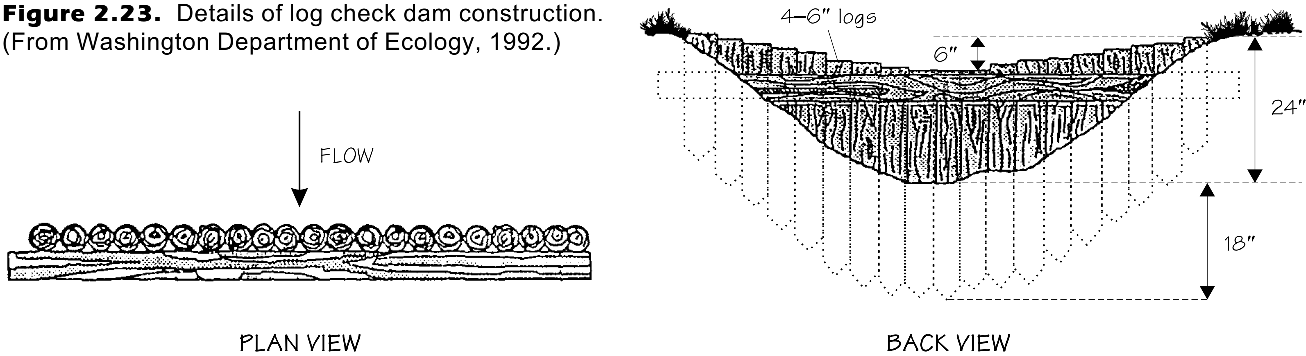


Figure 2.22. Details of rock check dam construction.

Figure 2.23. Details of log check dam construction. (From Washington Department of Ecology, 1992.)



The filter fabric must be replaced when it becomes clogged. Gabions (wire baskets filled with coarse rock) and filter fabric would function in the same manner.

Where they are readily available, logs can be used to construct check dams instead of rock (Fig. 2.23).

☛ Check dams can be used to slow surface flow in ditches.

☛ Check dams are a common means of establishing grade control in a drainage to minimize downcutting.

Concrete Check Dams

Concrete check dams (Fig. 2.24) can be an effective long-term alternative to straw bales, bio bags, and rock-filled burlap bags. They can often be constructed from waste concrete that is cleaned out of mixer trucks, but time constraints may prevent this. Concrete check dams are most appropriate along ditches that are relatively permanent.

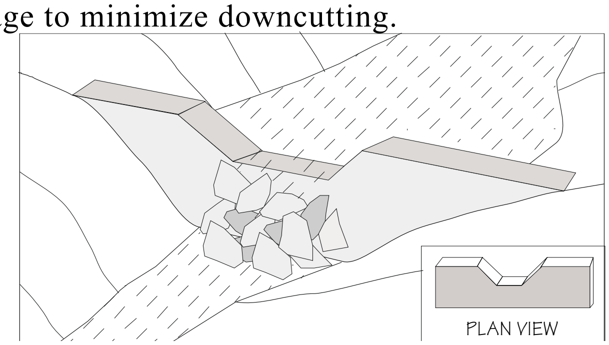


Figure 2.24. Waste concrete check dam. It should be a minimum of 4 inches thick; length and width vary to fit application.

Filter Berm

A filter berm (Fig. 2.25) allows the passage of water but not soil particles. It can be constructed of sand and gravel or crushed and screened quarry rock free of 200-mesh or smaller material. Using pit-run sand and gravel or quarry rock is not recommended because silt and clay will be present. In the ideal berm, fine sand, coarse sand, and gravel are placed sequentially from the upstream side to

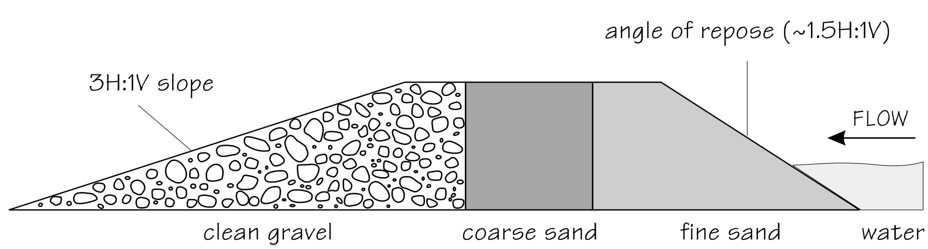


Figure 2.25. Idealized cross section of a filter berm showing details of construction.

the downstream end of the berm. The sand may need periodic replacement as it becomes clogged with sediment.

☛ Filter berms should be used in channels with low flow.

Trench Subdrains and French Drains

The terms 'trench subdrain' and 'French drain' are sometimes used interchangeably. A French drain is a ditch partially backfilled with loose, coarse rock to provide quick subsurface drainage and covered with a compacted clay cap. A trench subdrain is a ditch backfilled all the way to the top with loose, coarse rock, which allows water to enter more freely (Fig. 2.26). Both types of drains are designed to allow the movement of water while preventing or minimizing the movement of soil particles, and both require an outlet to remove water. Either can be improved by placing perforated pipe in the drain. (See also Figs. 3.11 and 6.6.)

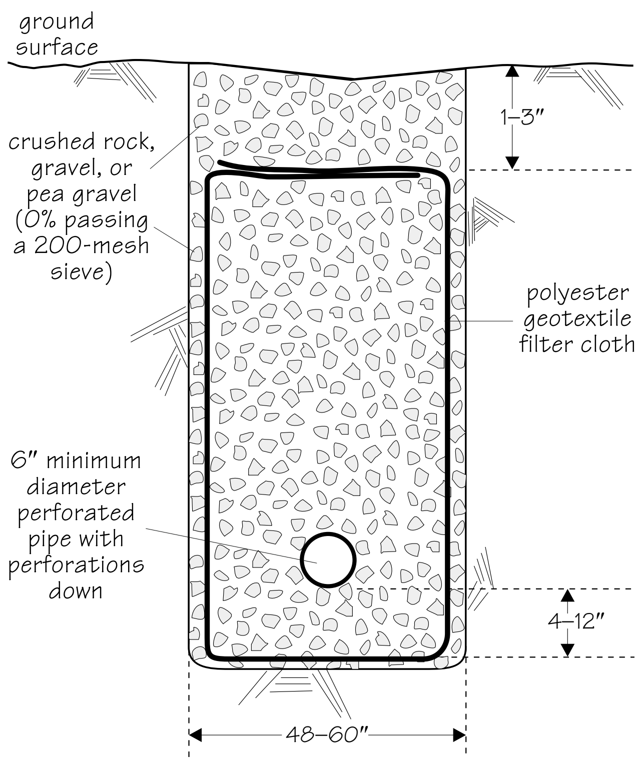


Figure 2.26. Details of trench subdrain construction.

Several filtering methods can improve the long-term effectiveness of these drains. Early applications relied on open-graded aggregate free of 200-mesh or smaller material, but this may eventually become clogged. Current practice is to wrap the perforated pipe in filter fabric so that sediment is trapped on the surface of the fabric rather than in the pore spaces. Because maintenance may eventually be required for subdrains, placement of clean-outs along the pipes is recommended.

☛ Drains are used for dewatering landslides and agricultural lands and stabilizing highway road cuts.

☛ Drains are also well suited for storm-water control.

Infiltration Galleries and Dry Wells

Infiltration galleries (or dry wells) are similar to trench subdrains and French drains except that there is no direct outlet for the water that enters them. These drains are deeper than they are long.

Infiltration galleries are created by excavating a hole— the deeper the better—which is then backfilled with coarse rock (Fig. 2.27). Typically, the holes are dug to the maximum reach (≈20′ of the backhoe used. If possible, water percolation should be improved by fracturing the bottom of the hole. This may require drilling and shooting. Backfilling to the sur-

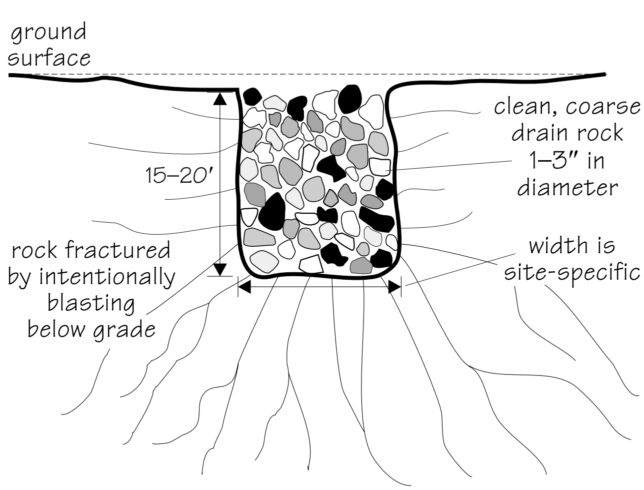


Figure 2.27. Details of infiltration gallery construction. (See also Fig. 2.9.)

face with coarse rock allows heavy equipment to pass safely over these structures, making them well suited for installation around a crusher or screening plant. Because there is no outlet for water, these galleries should be located where fines and storm water accumulate. Grading should direct storm-water runoff to them. The exact size and number of infiltration galleries needed is site specific. Maintenance is typically limited to periodic replacement of the fill with clean rock.

☛ Infiltration galleries are best suited for quarry sites or areas where natural infiltration of storm water is minimal and the water table is low enough to allow drainage. They should be used alone only where grades prevent connection to a gravity-flow subdrain or where volumes of storm water are small.

☛ Infiltration galleries should not be used if oil and grease are present to contaminate the ground water.

Wheel Washes

Tracking of mud and rocks onto roads can become a problem at many mine sites during the winter. A permanent wheel wash can be installed near the exit to wash excess dirt and mud off truck tires. A series of railroad rails spaced 2 to 8 inches apart can be used to shake loose rocks and dirt while the vehicle is driving through the wheel wash (Fig. 2.28). Make sure that water used to wash trucks is treated to remove solids and turbidity before being discharged from the site.

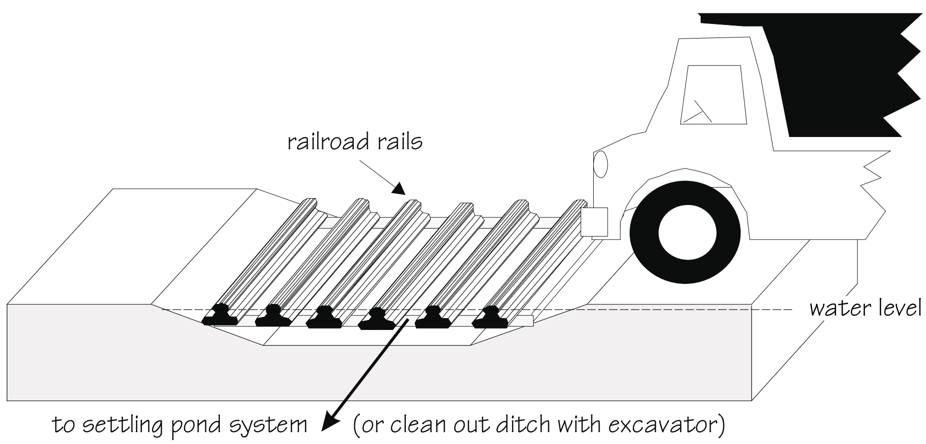


Figure 2.28. Wheel washes can be used to keep mud and rocks from being tracked onto roads. Dirty water can be sent to a settling pond, or the wheel wash can be cleaned out with an excavator.

STORM-WATER SETTLING PONDS

Most mine operations cannot rely solely on passive storm-water control methods and must employ settling ponds as an integral part of their storm-water system. These flat-bottomed excavations can range from small hand-dug sumps to ponds covering several acres. They slow water velocities enough to allow sediment to settle out of suspension. The number and size of ponds needed will depend on the site conditions. Construction of numerous ponds in the upper part of the drainage systems enhances effective trapping of sediments. For example, upper quarry benches and floors can be bermed so that they function as sediment basins during the rainy season.

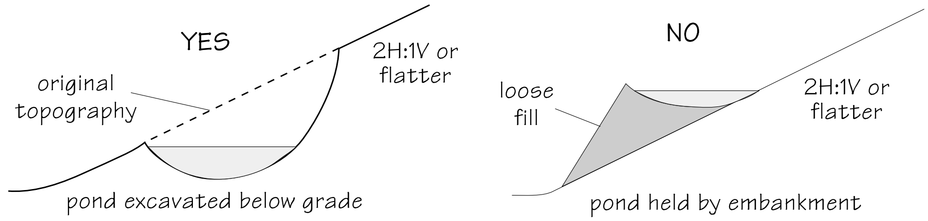


Figure 2.29. Details of settling-pond construction. The excavation method on the left is preferred because it is less likely to fail and cause flooding than an constructed embankment (right).

Two types of ponds are commonly used—detention and retention. Detention ponds reduce the velocity of storm water, allowing sediment to settle before it moves off-site. Retention ponds are large enough to accept all storm water without surface discharge.

Ponds can be developed by building embankments or by excavating below grade. Excavated ponds are preferable because they are less likely to fail than embankments (Fig. 2.29). Embankments have to be carefully constructed using the same techniques that would be used for constructing waste and overburden dumps and stockpiles (see p. 3.15). Ideally, ponds should be situated at the bottom of a slope. Soil or geotextile liners may be required where stability is a concern. Many ponds are designed for the life of the operation, whereas others are used for only a short time.

☛ Settling ponds are the best method of gathering turbid water to allow sediment to settle out.

In Washington, water impoundments that contain more than 10 acre-feet of water must be approved by the Dam Safety Section of the Department of Ecology.

In Oregon, water impoundments with dams more than 10 feet high or with a capacity of more than 9.2 acre-feet of water must be approved by the Dam Safety Section of the Oregon Water Resources Department.

Configuration, Location, and Size

Storm-water detention ponds should be designed to maximize both velocity reduction and storage time. That is, storm water entering a pond should spread out and migrate as slowly as possible toward the discharge point. Baffles constructed across the pond (Fig. 2.30) can reduce flow rates. A good rule of thumb is that the flow path of the pond should be at least five times the length of the pond. The inlet and outlet should be located so as to minimize the velocity and maximize the residence time.

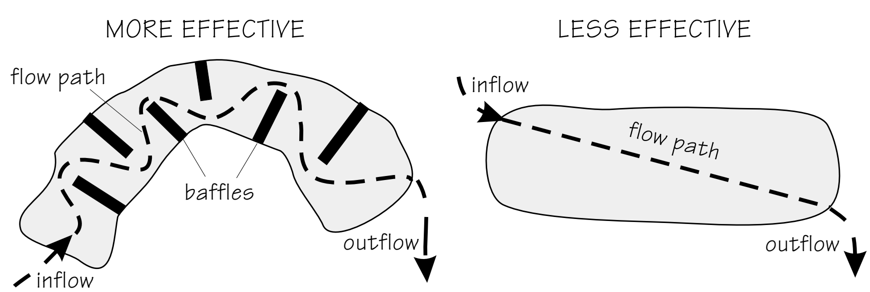


Figure 2.30. Details of detention pond design. The pond on the left, which maximizes the length of the flow path, is preferable to the pond on the right, which does not keep water in the pond long enough for optimum settling.

If ponds are to be placed in the lowest area of the watershed, several should be constructed in a series. This will enable the first pond to slow the high-velocity waters coming into it and allow subsequent ponds to settle out sediments more effectively. For maximum treatment effectiveness, ponds should be placed as close as possible to those areas most likely to contribute sediment, such as the pit floor, the processing plant, and other areas of heavy equipment activity.

There are several widely used methods for determining the appropriate size of storm-water ponds for a given site. Most methods begin with estimating the size of the watershed and estimating runoff using infiltration rates. This information is then used to calculate the amount of runoff on the basis of annual precipitation or a storm event of a certain size. Observations of flow characteristics and locations made near the mine during storm events can be invaluable in developing a good storm-water pond system.

However, choosing an appropriate size for storm-water ponds can be difficult without site-specific information such as a storm hydrograph—a graph of the volume of water flowing past a certain point during a storm event. When hydrographic information is not available, theoretical calculations are used to estimate the flow volume for a given storm event. The calculations quickly become complicated because storm intensity and duration can have a significant effect on the amount of runoff. Also important, but even more complicated, are determining the influence of road systems, vegetative cover, and amount of compaction on runoff volumes.

The Natural Resources Conservation Service (formerly Soil Conservation Service) has developed a simplified method for estimating storm-water runoff. This method can work well if the limitations are understood, and it yields a good starting point for determining pond size. For more information, contact the local office of the Natural Resources Conservation Service.

There are many resources for information on designing storm-water ponds. (See the list of references at the end of the chapter.) For determining spillway designs and diversion ditch liner specifications, Urban Hydrology for Small Watersheds (Soil Conservation Service, 1986) is a good resource.

☛ For most mining situations, storm-water ponds should be designed to handle at least a 25-year/24-hour event or larger.

In Washington, RCW 78.44 sets a standard for water control: "Diversion ditches, including but not limited to channels, flumes, tightlines and retention ponds, shall be capable of carrying the peak flow at the mine site that has the probable recurrence frequency of once in 25 years as determined from data for the 25-year, 24-hour precipitation event published by the National Oceanic and Atmospheric Administration." The data for 25-year, 24-hour precipitation events can be found in Miller and others, 1973. Furthermore, if the site is located in a watershed that is prone to erosion, heavy storms, and/or flooding, design specifications may require planning for a 100-year storm event.

Maintenance Settling ponds must be cleaned out regularly to remain effective. Spillways should be kept open and ready to receive overflow during large storms. Settling ponds should be constructed and placed so that onsite equipment can be used to maintain them. In some situations, sediment can be pumped out of settling ponds as a slurry instead of being removed with heavy equipment. Regardless of the method of sediment removal, all sediment removed should be placed in a stable location so that it will not enter waterways.

Drainage The method of releasing water from storm-water ponds can be critical in determining their efficiency. Standpipes, spillways, and infiltration are the most common release methods.

Standpipes are vertical pipes rising from the bottom of the pond and connected to a gently sloping pipe that passes through the side of the pond to the discharge point (Fig. 2.31). Antiseep collars must be attached to the pipe where it passes through the dam or settling pond wall to prevent water from flowing along the outside of the pipe. A grate or screen should be placed over the standpipe intake to prevent debris from clogging it.

Spillways are overflow channels that are part of the construction of all water impoundments. For small settling ponds used intermittently and designed for low maintenance, spillways may handle all water discharged from the pond. Where water is recirculated to the processing plant or where discharge is through a standpipe or subdrain, a spillway allows overflow during extremely wet weather or when the primary drain system becomes clogged.

Spillways should be located in undisturbed material and not over the face of a constructed dam. If the spillway is placed on erodible material, it must be rock lined to limit erosion that would compromise the safety of the dam.

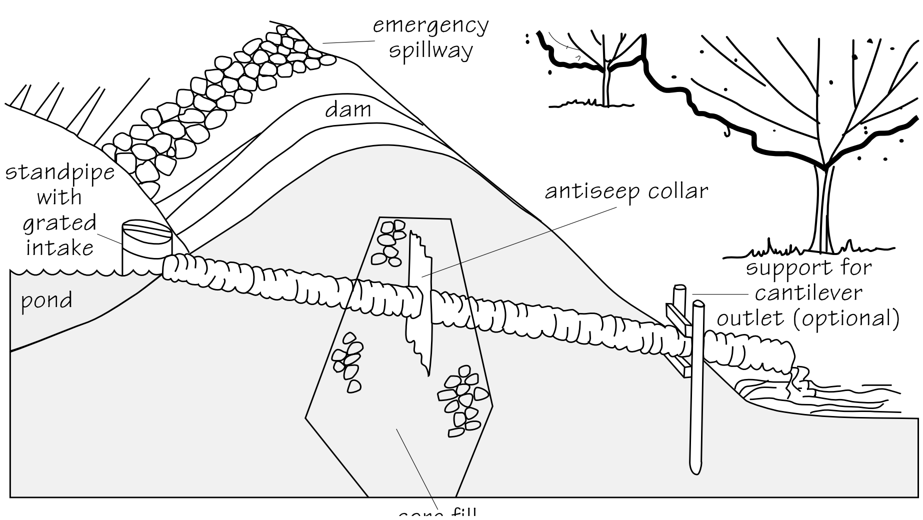


Figure 2.31. Section through a berm showing standpipe with antiseep collar. (Modified from U.S. Soil Conservation Service, 1982.)

STORM-WATER TREATMENT

In some places, additional treatment is required to reduce the turbidity of storm water prior to discharge to public waters. (See p. 2.3.) When storm water contains abundant clay-size particles too fine to settle using conventional pond treatment, land application is the treatment of choice. Alternative treatment methods include the addition of flocculants or the use of water clarifiers.

Land Application

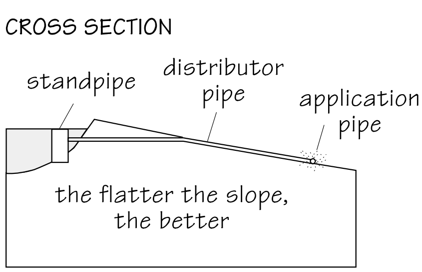
Land application involves sending storm water through dispersal systems that allow the turbid water to slowly soak into vegetated areas. Land application may be a feasible technique to handle all sediment-laden water, or it may just increase storm-water storage capacity. Some of the most common distribution systems are perforated pipe laid across a slope, level spreaders, and sprinkler systems. Where large flat areas are available and water dispersal is not an issue, water can be discharged directly from the distributor pipe, eliminating the need for a perforated application pipe. Turbid water must not be allowed to enter wetlands or creeks.

Perforated Pipe. Plastic pipe with holes drilled in it can disperse a fine spray of water over a large surface area (Fig. 2.32). This method works well if the pipes are laid along slope contours; pipes laid perpendicular to slope contours develop excessive hydraulic head at the lower perforations, resulting in uneven distribution of water and increased erosion potential.

Level Spreader. A level spreader is a trench excavated along the contour and filled with gravel or other permeable material that will allow turbid water to percolate into the ground. Level spreaders work best where the surrounding soil is fairly permeable.

Sprinkler Systems. Sprinkler systems use commercially available sprinklers to apply storm water. Sprinkler systems work well where:

- There is sufficient hydraulic head to distribute the storm water from sprinkler heads.



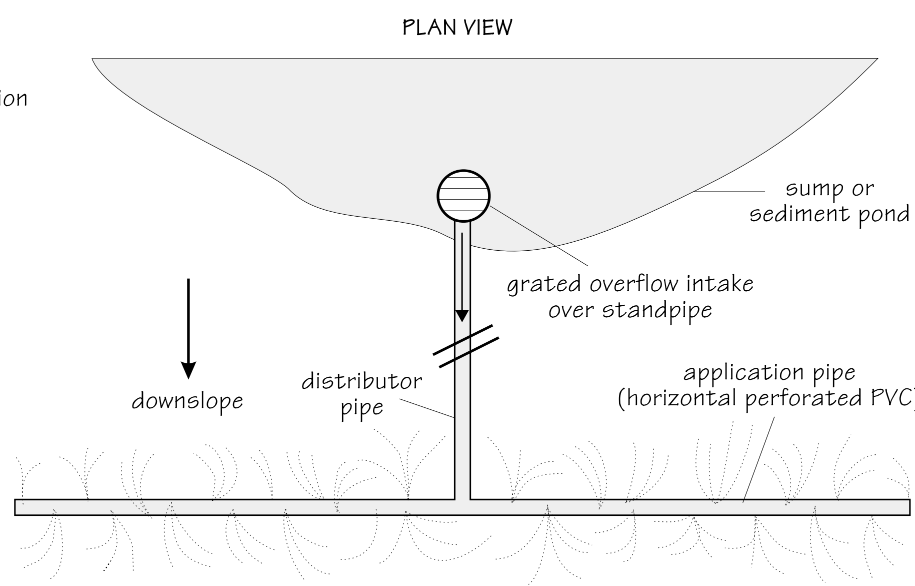


Figure 2.32. Typical land application system for storm water using a perforated pipe laid along a slope contour as a delivery system. The length of the distributor pipe is not to scale. The application area should be a reasonable distance from the pond in a stable vegetated area that can handle the extra water.

- The storm water contains only fine clays that will not clog sprinkler heads.
- There is sufficient vegetation to prevent erosion at the sprinkler heads.

Land application systems generally cannot handle the surges in water volume during a large storm because the storms often occur in winter when the soils may already be saturated. Assuming that soils will always accept the storm water can be a serious error. A simple infiltration analysis can determine the capacity and infiltration rate of a site's soils. The design of a land application system should assume that soils are saturated and that existing or planted vegetation will filter sediments. Concentration of the outflows from a land application system should be avoided because it may cause soil erosion and create problems elsewhere.

Flocculants

Flocculants are most commonly used to clean storm-water discharges or water recycled from rock-washing operations. Proper use of chemical flocculants can reduce the size of settling ponds required for a given site. Most flocculants are not toxic to aquatic organisms and fish. However, the supplier or manufacturer and the state water quality agency should be asked about the environmental effects of the flocculant chosen.

Most flocculants are composed of high-density (heavy) organic polymers with a strong positive charge. The positively charged particles act like a magnet to attract negatively charged clay particles. The adsorption of clay onto the flocculant speeds settling of smaller and lighter clay particles. Alum is an inorganic flocculant that works in much the same way as the organic flocculants.

Chemical flocculants are designed for use with specific types of clay. The key to using a chemical flocculant is maintaining the proper mixture of flocculant and pond water and thoroughly mixing and agitating the flocculant mixture in the pond, making sure not to overagitate. Flocculants are commonly diluted in a large container before they are added to the settling pond.

At least two ponds should be used to remove suspended solids. The first pond should allow slow mixing of the flocculant and the water to be treated, with a retention time of 20 minutes. The second pond should ideally retain water for 3 to 8 hours. Alternatively, the flocculant mixture can be injected into the waste-water stream before it enters the settling ponds. Ponds must be situated where they can easily be cleaned on a frequent basis.

In Washington, a National Pollution Discharge Elimination System (NPDES) permit from the Department of Ecology is required if flocculant-treated storm water is to be discharged offsite.

Water Clarifiers

Water clarifiers are a mechanical method of separating solids and water. They consist of a series of closely spaced inclined plates. A flocculant is injected to assist in separation. These systems are

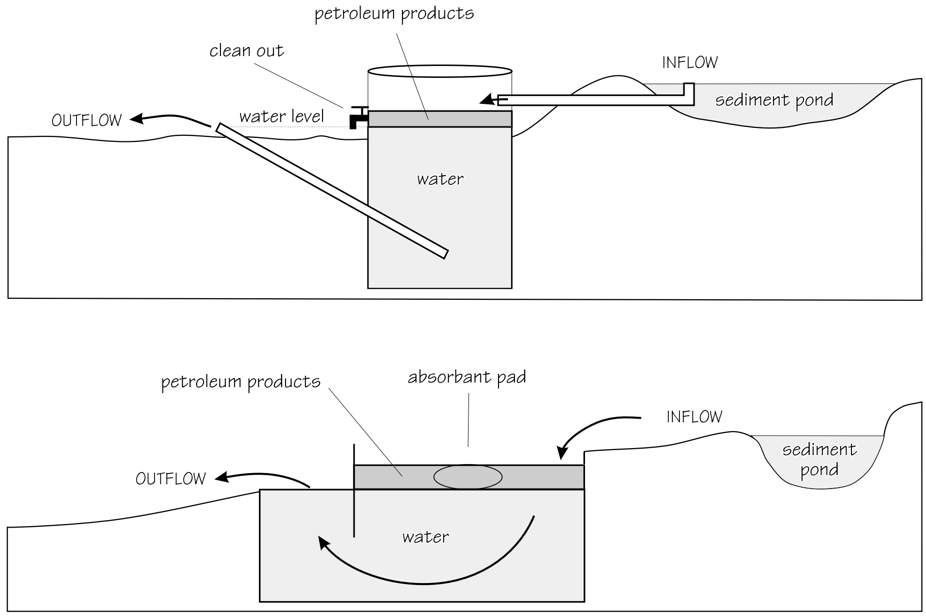


Figure 2.33. Two different types of oil/water separators used to remove petroleum products from storm water. The inflow must be free of sediment or frequent cleaning out will be necessary. The top system uses a clean-out spigot to remove the oil as it floats atop the water. The bottom system uses an absorbent pad to soak up the oil. Design specifications will depend on site conditions and storm-water volumes.

widely used as a final treatment for sewage effluent prior to discharge. In some situations, it may be possible to rely on smaller storm- and process-water ponds if a water clarifier is used. Due to their initial capital costs, however, clarifiers are not used extensively in the aggregate industry.

Oil Separators

Petroleum products can be removed from storm water through the use of oil/water separators. The precise layout and design is usually site-specific but two examples are depicted in Figure 2.33. Oil/water separators take advantage of the fact that oil floats on water. They collect the oil on the surface of the water while allowing the water to flow through. The oil collected can be removed by absorbent pads or skimmed with a bucket. Contaminated absorbent pads and water should be disposed of according to DEQ rules in Oregon and DOE rules for Washington.

Keys to effective oil/water separators:

- ❑ There must be sufficient surface area to allow the petroleum to remain on the surface.
- ❑ The water velocity and volume must be low enough to prevent oil/water mixing or overspillage.
- ❑ The majority of settleable solids must be removed from the storm-water stream before it reaches the oil/water separator or the separator will quickly become filled with sediment.

STREAM BUFFERS

Vegetated stream buffer zones (areas that will not be mined, disturbed, or developed) vary in width from site to site. (See Permanent Setbacks or Buffers, p. 3.4.) Factors usually considered in establishing buffers are the purpose of the buffer, the size of the stream, and the rate of meander of a stream. The primary reasons to establish and maintain buffers are to:

- Preserve water quality in the stream by filtering sediments through a vegetated buffer.
- Protect the existing stream or river channel.
- Protect riparian habitat.
- Minimize the potential for turbid water/sediment discharges into public waters.
- Maintain tree cover over streams to moderate water temperature to insure fish survival.
- Prevent stream capture or avulsion because of lateral migration of a river into a pit.
- Protect the habitat of threatened or endangered riparian and aquatic species.

STREAM DIVERSION

Stream diversion can be beneficial to water quality and mine operations by isolating public waters from the mine activity. To insure the long-term stability of landforms, a highly technical approach to stream diversion has been required at large open-pit mines in the western states where numerous sections of land are being affected. For aggregate sites in the Pacific Northwest where the scale is significantly smaller, a less technical approach is appropriate because typically only a small portion of the total watershed is being impacted.

Streams can be classified as perennial or permanent (containing water all year round), intermittent (containing water only at certain times of the year), or ephemeral (containing water only when it rains). Technical discussions and research on classification of drainages, drainage density, and reconstruction techniques for reclaimed mine sites are ongoing and complex.

IMPORTANT: Before diverting any perennial, ephemeral, or intermittent streams, check to see if a permit is needed.

In Washington, contact the Departments of Ecology, Fish and Wildlife, and Natural Resources.

In Oregon, contact the Departments of Environmental Quality, Fish and Wildlife, and Geology and Mineral Industries and the Division of State Lands.

Perennial or Permanent Streams

Diversion of perennial streams is beyond the scope of this manual and will not be covered. If a perennial stream must be diverted, the proper state and local agencies should be consulted.

Intermittent or Ephemeral Streams

Diversion of intermittent or ephemeral streams is not as critical as for perennial streams but may still require permits. The basic rule of thumb is to replace existing drainages and drainage conditions. In some mines, segments of drainages may be significantly altered, particularly those located in an upland quarry site. The same channel carrying capacity, length, characteristics, and gradient as the original stream should be maintained in the diversion.

On quarry sites after mining, channel length may be shortened if streams are directed over the highwall to enhance reclamation diversity. Channel stability is not generally affected by steepening the gradient or shortening the channel if the channel foundation is hard rock. Decreasing channel length or increasing channel gradient on alluvial or colluvial materials should not be undertaken without thorough analysis.

If the drainage diversion will be short term, a rock-lined diversion channel may be all that is needed. For diversions that will be in place for several years, the diverted stream should be shaded, habitat areas, such as pools and riffles, rootwads or logs (see Fig. 4.12), should be created, and vegetation should be used to stabilize the banks (see Biotechnical Stabilization, p. 7.13).

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3 Operation and Reclamation Strategies

INTRODUCTION

Four general strategies can be used in surface-mine reclamation. Some mines may use all four of these strategies:

Post-mining reclamation – reclamation only after all resources have been depleted from the entire mine.

Interim reclamation – temporary reclamation to stabilize disturbed areas.

Concurrent (progressive or continuous) reclamation – reclamation as minerals are removed; overburden and soil are immediately replaced.

Segmental reclamation – reclamation following depletion of minerals in a sector of the mine (Norman and Lingley, 1992).

In Washington, the Department of Natural Resources (DNR) encourages segmental reclamation wherever site conditions permit.

In Oregon, segmental reclamation is considered a variant of concurrent reclamation. The Department of Geology and Mineral Industries (DOGAMI) encourages concurrent reclamation wherever possible.

POST-MINING RECLAMATION

Reclaiming after all resources have been depleted from the entire mine is generally discouraged by regulating agencies because it results in large areas being left unreclaimed for long periods, but it may be necessary at many quarries and metal mines and at some sand and gravel deposits (Fig. 3.1).

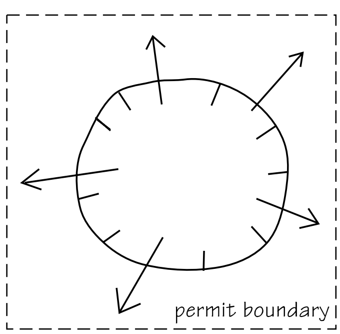

Figure 3.1. For a mine site beginning a center-outward excavation, the preferred segmental reclamation method is not possible, and post-mining reclamation then becomes the method by default.

Advantage

- Complete resource depletion is more easily attainable in some instances.

Disadvantages

- Stockpiled soils will have deteriorated during the mine's life and will not be as fertile as the soils in place.
- Revegetation will probably be more expensive and take longer.
- The site generates negative public opinion for a long period.
- The land is not providing a beneficial use while unreclaimed.
- No reclaimed segments are available as test plots for revegetation.
- Bonding liability is very high.

INTERIM RECLAMATION

Interim reclamation is done seasonally to stabilize mined areas or stockpiles and to prevent erosion. If a mine is to remain inactive for more than 2 years or if a stockpile, excavated slope, or storage area needs rapid stabilization, it may be appropriate to temporarily reclaim it by doing earthwork and using fast-growing vegetation, such as cereal grains or legumes that establish quickly, to stabilize the site. However, topsoil should not be moved for interim reclamation; significant amounts are lost each time topsoil is moved. (See The Soil Resource, p. 3.10.)

Advantages

- Soil viability is maintained.
- Fewer storm-water control structures are needed because the erosion-prone area is vegetated.
- Air and water quality are improved in the short term.
- Sites that use interim reclamation are often easier to convert to final reclamation than those that do not.

Disadvantages

- Areas may be redisturbed as plans change.
- Cost may be greater than when material is moved only once.

CONCURRENT OR PROGRESSIVE RECLAMATION

Concurrent or progressive reclamation typically involves transporting material from the new mining area to the reclamation area in one circuit (Fig. 3.2). This is the method used in strip mining minerals such as coal where a small amount of mineral is mined compared to a large amount of overburden moved.

Concurrent reclamation is viewed by the public as the preferred technique. However, progressively reclaiming land that overlies known mineral resources can be wasteful. Thin soils may render progressive reclamation impractical or impossible on some sites. It is also impractical for those operations that must blend different sand and gravel sizes from various parts of the mine site to achieve product specifications.

Advantages

- Soil is immediately moved to the reclamation area.
- Soil and subsoil profile are more easily reproduced than in other types of reclamation.
- Materials are moved only once.

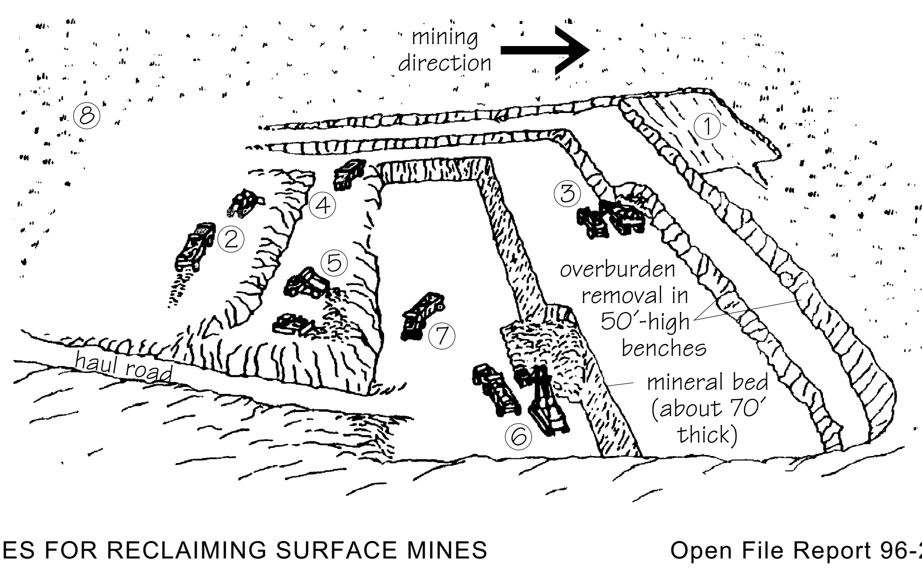

Figure 3.2. Concurrent or progressive extraction and reclamation of a shallow dry pit.
1, removal of topsoil;
2, spreading topsoil on graded wastes;
3, loading of overburden;
4, hauling of overburden;
5, dumping of overburden;
6, loading of product;
7, hauling of product;
8, reclaimed land.

(Modified from U.S. Bureau of Land Management, 1992.)

- Disturbance at any given time is minimized.
- Offsite impacts are minimized in any given area.
- Mined land can be reclaimed earlier for agriculture or grazing.
- Bond liability tends to be low.

Disadvantages

- Progressive reclamation is generally not feasible in quarries or deep gravel deposits.
- Progressive reclamation typically does not work if the water table is above the excavation depth.

SEGMENTAL RECLAMATION

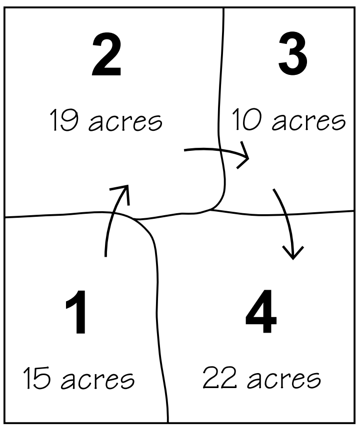

Figure 3.3. A segmental reclamation plan with four segments showing segment size and direction of working.

In segmental reclamation, the mine is divided into segments with fairly uniform characteristics and the order of mining and reclaiming these segments is determined (Fig. 3.3). Prior to mining, soil in the first segment is stockpiled to minimize handling and protect the resource. After resource extraction from the first segment, its slopes are reshaped according to the reclamation plan. Soil is then stripped from the second segment and spread on the slopes of the first segment.

Revegetation of the floor of the first segment does not occur until the area is no longer needed for mineral processing or maneuvering trucks. Immediately prior to replacing topsoil and planting, the pit floor is plowed or ripped because most plants cannot grow in soils that have been overcompacted by heavy machinery. Prompt planting in the correct season with grasses, legumes, and trees will quickly produce a cover that reduces erosion, retains moisture, and moderates soil temperature.

Segmental reclamation works best in homogenous deposits where aggregate mining proceeds in increments. Typical working cells or segments will be larger in heterogeneous deposits (for example, fluvial deposits) where blending minerals from many places in the mine may be required (Norman and Lingley, 1992).

Advantages

- Topsoil for most segments is handled only once and is not stored. This reduces reclamation cost and preserves soil quality.
- Final slope angles and shapes can be established during excavation rather than as a separate operation.
- Clay and silt, which are critical for retaining the moisture and nutrients essential for vegetation, are less likely to be washed away because they are immediately revegetated.
- The potential for establishing a diverse self-sustaining soil/plant ecosystem is enhanced because revegetation of reclaimed segments will be monitored as mining continues.
- Restoration of chemical, physical, and biological processes is less expensive when reclamation is started as soon as possible and spread over the life of the mine.

- Reclamation is less expensive because it does not require mobilization of personnel or equipment for the sole purpose of reclamation.
- Short-term environmental impacts are reduced.
- Bonding liability at any given time is minimized.

Disadvantages

- Thin soils may render this technique impractical.
- It is impractical for those operations that must blend different sand and gravel sizes from various parts of the mine site in order to achieve product specifications.
- Poorly planned segmental reclamation may result in disturbing more land per unit of mineral produced.

By law (RCW 78.44) in Washington, a segment is defined as a 7-acre area with more than 500 linear feet of working face. Larger segments must be approved by DNR in a segmental reclamation agreement.

MINING TO RECLAIM

Mining the slope to the final contours reduces reclamation costs by eliminating some of the earthwork necessary for final reclamation. This can result in reclamation being completed earlier, the performance security being reduced, and operating costs being lower in the long run.

SITE PREPARATION

Before mining begins, steps must be taken to mark permit boundaries, setbacks, buffers, segments, and storage and processing areas. Setbacks, buffers, and storage areas should remain undisturbed until reclamation. Keeping equipment and stockpiled materials out of these areas will help preserve them. Flagging, fences, or monuments will alert operators to areas to be avoided. If vegetation is present on slopes that might be unstable if bare, then those plants should be protected. Activity near trees and shrubs should be kept outside the area below the longest branches (or drip line).

Permit and Disturbed Area Boundaries

Permit boundaries and the limits of the area to be disturbed (permit boundary minus setbacks and buffers) should be identified with clearly visible permanent markers. Markers should be maintained until the reclamation permit is terminated.

Permanent Setbacks or Buffers

Permanent setbacks or buffers are necessary at many mines (Fig. 3.4). They are lands (that may or may not have vegetation) that remain undisturbed during mining to provide habitat and/or visual and noise screening.

In Washington, the minimum permanent setback for quarries (mines in consolidated deposits) permitted after June 30, 1993, is 30 feet. This area cannot be mined, and the material cannot be used for reclamation. Permanent setbacks are not required for gravel pits (unconsolidated deposits) but may still be useful if the mine has close

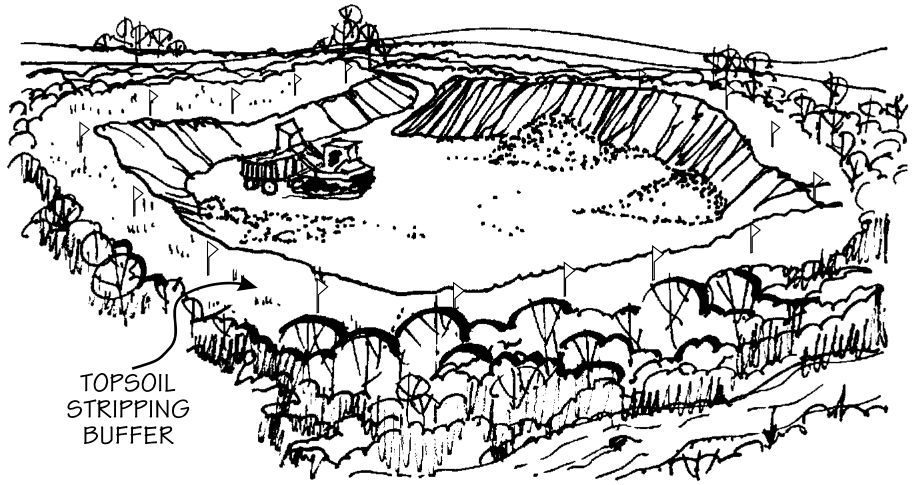

Figure 3.4. Buffer strips of native vegetation protect adjacent land and water and visually screen the operation. Note that the flags marking the limits of the disturbed area show employees where to stop mining. (Modified from Green and others, 1992.)

neighbors or adjacent scenic resources. However, setbacks may still be required by local government.

In Oregon, mine setbacks are site-specific and designed to provide lateral support for adjacent lands. Setbacks for the purpose of minimizing conflicting land uses are determined by the local land-use authority.

Reclamation Setbacks

Reclamation setbacks are lands along the margins of surface mines that must be preserved to provide enough material to accomplish reclamation. If the cut-and-fill method will be used to restore slopes (rather than mining to a final slope), the reclamation setback from the property boundary (or permanent setback, where used) should be wide enough to ensure that sufficient material is available for reclamation.

In Washington, the width of the reclamation setback for pits (mines in unconsolidated deposits) permitted after June 30, 1993, must equal or exceed the maximum anticipated height of the adjacent working face.

Note: A setback equal to the working face will provide only enough material for a 2:1 slope. To meet the standards of the law for slopes of between 2:1 and 3:1, a setback of 1.5 times the vertical height of the working face is required.

Setbacks to Protect Streams and Flood Plains

Streams and flood plains are dynamic locations that frequently experience dramatic changes during flooding. They are prone to damage by, and slow to fully recover from, improperly planned and executed mining operations. Mining in or near streams and flood plains requires greater care on the part of the operator and is subject to closer regulation than mining in less sensitive areas.

In Washington, no mine, including haul roads, stockpiles, and equipment storage, may be located within 200 feet of or on the 100-year flood plain of a stream that has a flow greater than 20 cubic feet per second unless a Shoreline Permit is issued by the local jurisdiction (Washington Department of Ecology, 1992). Wide setbacks may be necessary for stream and flood-plain stability to preserve riparian zones and to prevent breaching of the pit at a later date. The depth of excavation and pit size may be limited in these areas.

In Oregon, mining is not explicitly prohibited on the 100-year flood plain. Setbacks are site-specific to protect riparian areas and stream integrity. Depending on flood frequency, bank stability, and the potential for lateral migration of the river channel, wider setbacks may be required or depth of excavation may be limited.

Conservation Setbacks

In special instances, setbacks that will not be mined or disturbed may be necessary to protect unstable slopes, wildlife habitat, riparian zones, wetlands, or other sensitive areas or to limit turbid water discharge from areas that will be disturbed.

Topsoil and Overburden Storage Areas

Prior to mining a segment, all available topsoil and overburden should be stockpiled in separate, stable storage areas for later use in reclamation or immediately moved to reclaim adjacent depleted segments. Topsoil needed for reclamation cannot be sold, removed from the site or mixed with sterile soils.

In Washington, topsoil should not be used to create screening berms required by local government because this may preclude its timely use for reclamation.

VISUAL AND NOISE SCREENS

The value of visual and noise screens cannot be overstated. The adage ‘out of sight, out of mind’ is particularly applicable to mine sites. The more the public can be screened from the unpleasant aspects of mining, such as dust, noise, and an unsightly view, the less likely they are to aggressively oppose mining operations.

The following are some ways to reduce the noise and visual impacts of mining (Figs. 3.5 and 3.6):

☛ Plan mine development to minimize offsite impacts.

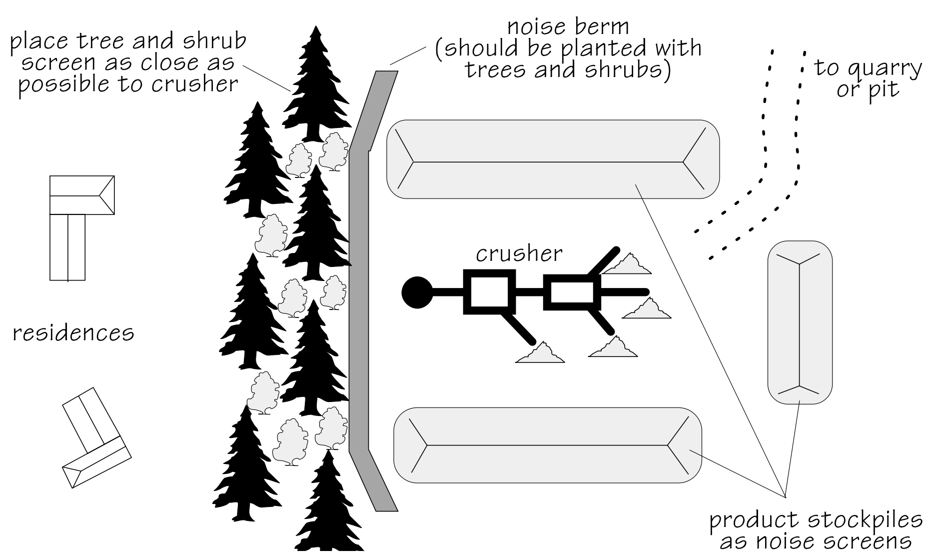

Figure 3.5. Visual and noise screening techniques used at a processing area.

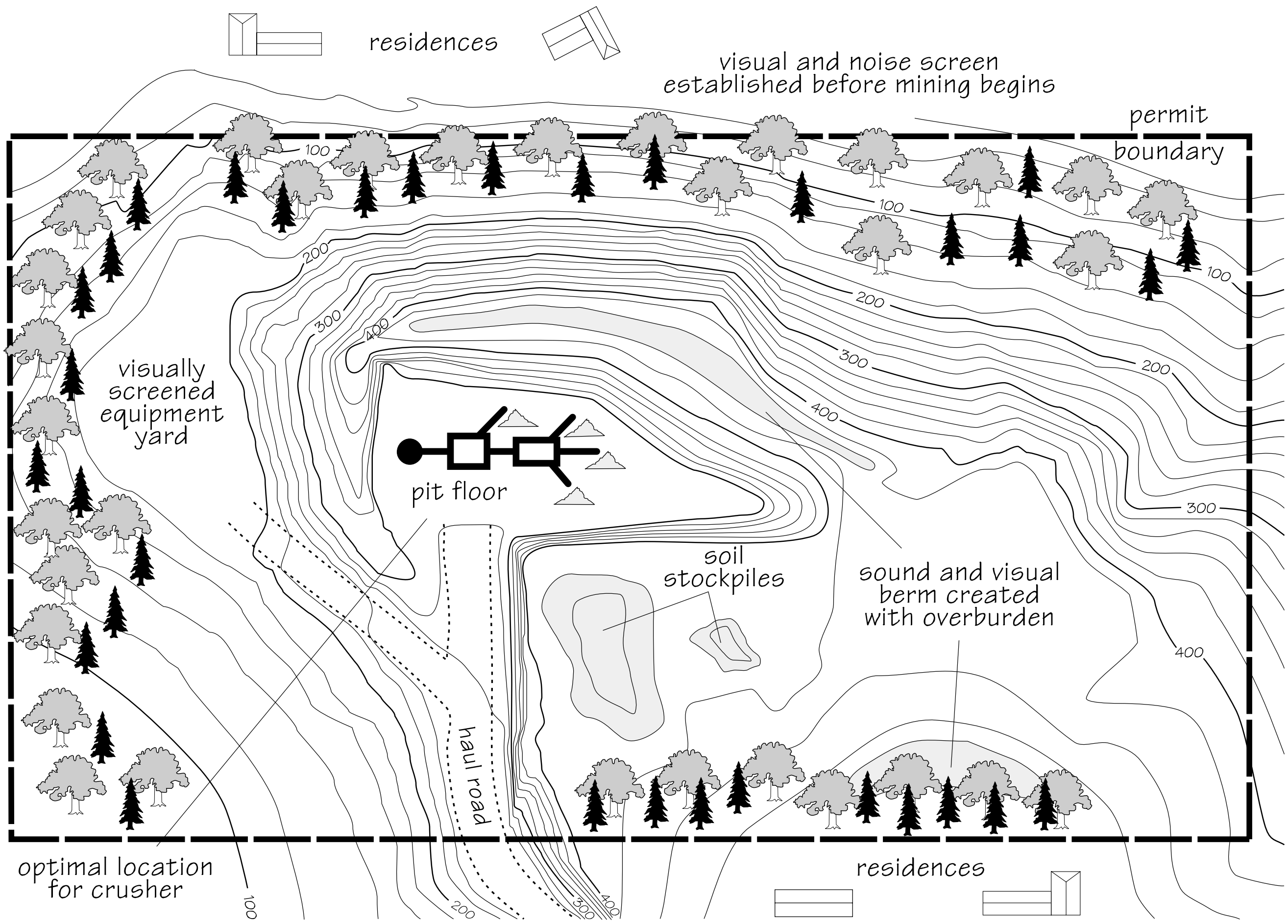

Figure 3.6. Visual and noise screening techniques used at a quarry site. Figure 2.11 shows the storm-water control system at the same site.

- ☛ Use existing topography as a noise and visual screen.
- ☛ Store overburden in berms along the site perimeter. Plant vegetation on them immediately to reduce noise.
- ☛ Plant trees and other visual screens—the denser and wider the better—well ahead of the mining to give them time to establish before they are needed.
- ☛ Plant tree barriers as close to the noise source as possible and between noise sources and the neighbors.
- ☛ Plant trees that will quickly grow tall enough to screen the mine. Plant shrubs to fill in the gaps, particularly if the foliage is sparse on the lower parts of the trees. Use evergreens if the site will be operated year round.
- ☛ Reduce noise by placing loud stationary equipment, such as the crusher, in an excavated area below the surrounding terrain.
- ☛ Surround the crusher with product stockpiles to reduce noise.
- ☛ Enclose the crusher in a building.
- ☛ Muffle the exhaust systems on trucks and other equipment.

Figure 3.7. Noise levels and human response for some common noise sources. (Modified from Barksdale, 1991.)

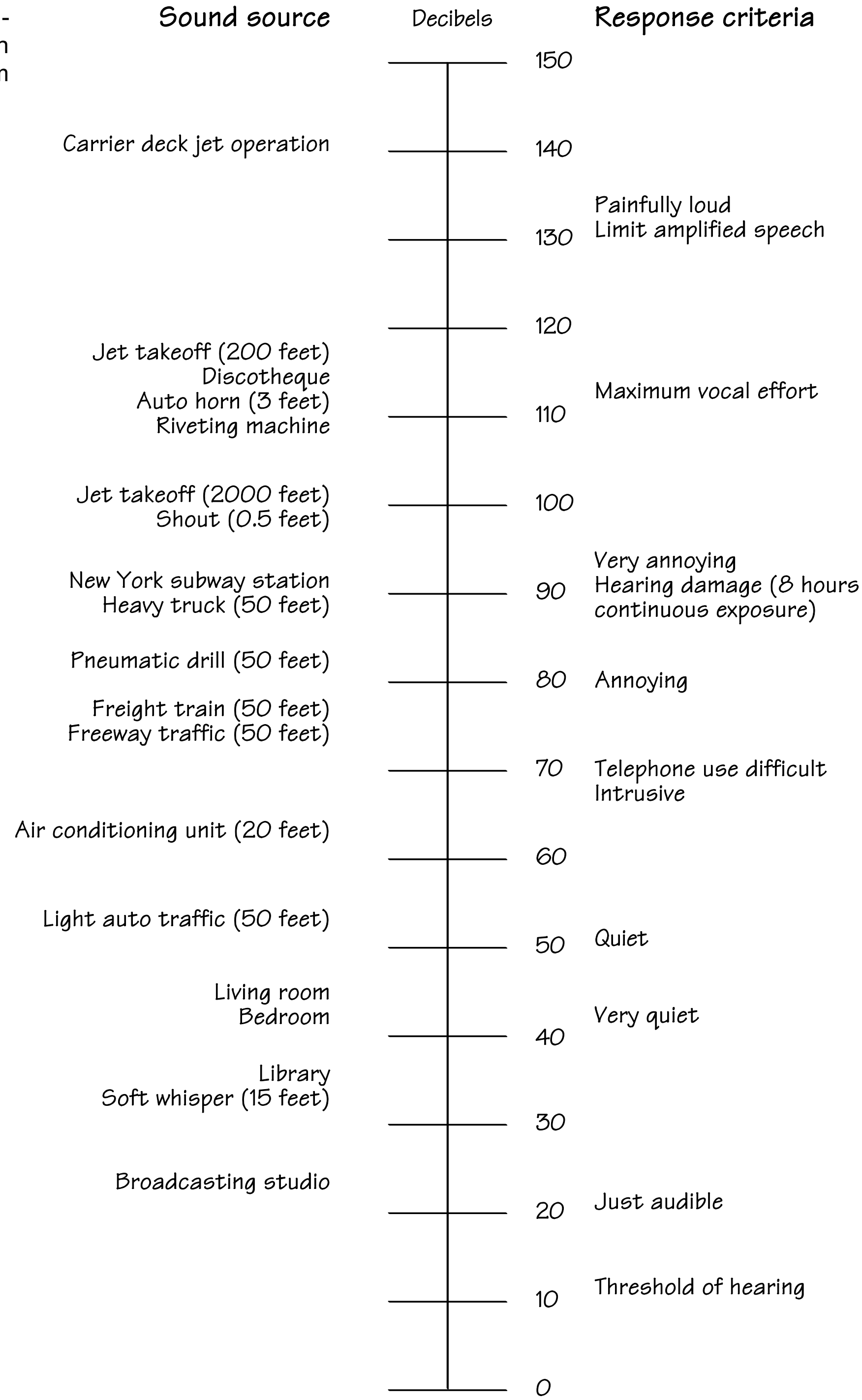

Table 3.1. Summary of noise measurements and projected noise levels in decibels (dBA) for common mining equipment (Barksdale, 1991)

		Projected noise levels		
Noise source	**Measurements**	**1,000 ft**	**2,000 ft**	**3,000 ft**
Primary and secondary crusher	89 dBA at 100 ft	69.0 dBA	63.0 dBA	59.5 dBA
Hitachi 501 shovel, loading	92 dBA at 50 ft	66.0 dBA	60.0 dBA	56.5 dBA
Euclid R-50 pit truck, loaded	90 dBA at 50 ft	64.0 dBA	58.0 dBA	54.4 dBA
Caterpillar 988 loader	80 dBA at 300 ft	69.5 dBA	63.5 dBA	60.0 dBA

- Use screens coated with rubber in the crusher, and line dump trucks beds with rubber.

How Noisy Is It?

Figure 3.7 summarizes the noise level, in decibels (dBA), from some common sources. Table 3.1 summarizes noise measurements for common mining equipment.

Noise-Control Methods

Noise-control measures, such as berms and tree barriers, can reduce the noise experienced by adjacent landowners by as much as 12 dBA, whereas earthen berms with vegetation can reduce noise up to 15 dBA, depending on the size and configuration of the berms, the type and density of vegetation, and the distance to the listener.

Visual Screens

The least expensive visual screen is the existing topography and vegetation on the site. Plan to leave large buffer zones of trees and vegetation between the mining site and nearby roads and buildings. Narrower buffer screens can be created with vegetation (preferably native evergreens), walls, fences, or berms, although they are generally less effective than buffer zones, which rely on distance for their effectiveness.

REMOVING VEGETATION

In a well-planned operation, vegetation is removed from areas to be mined only as needed and is preserved when possible to screen the site and limit erosion that may result in turbid water discharge.

Disposing of Vegetation

Grass and small shrubs can be incorporated into the topsoil stockpile, and larger material can be chipped and used as mulch or to add organic matter to the soil. Burial of large volumes of woody debris is permissible only in areas above the water table because anaerobic decomposition of woody debris produces nitrates, which can degrade water quality. Vegetation should not be buried in areas where building construction is planned because the soil may settle as the vegetation decays.

In Washington, a permit from the county health district is required for burial of more than 2,000 cubic yards of debris. If burning will take place, a burning permit may be necessary.

In Oregon, a permit from the Department of Environmental Quality is generally required for burial of debris and may be required for burning.

Transplanting Vegetation

Bushes and small trees, together with some surrounding soil, can be scooped up using backhoes or front-end loaders with tree spades and transplanted to mined-out segments or areas to be used as screens. (See p. 7.9.) This technique is a cost-effective means of quickly establishing a natural appearance in reclaimed segments, introducing seed trees, and providing screening. These plants are already adapted to the area. Moving the soil along with the plant protects rootlets and microorganisms that are important to plant health. Additionally, the soil may contain seeds or shoots of other vegetation, which may spread across nearby areas.

Using Vegetation for Habitat

Vegetation that cannot be transplanted live can be set aside (with leaves, needles, and roots intact) for future use as fish and wildlife habitat. Placed in ponds, it can provide shelter for small fish, and collected into piles, it can provide shelter for small animals. (See Structures That Enhance Habitat, p. 4.12.) Salvaged coarse woody material, such as logs, should be distributed across a regraded area at the rate of about 8 tons per acre.

THE SOIL RESOURCE

Soil is one of the most important components of successful reclamation. Without soil, vegetation cannot be established. A typical soil is composed of approximately 45 percent minerals (sand, silt, and clay particles), 5 percent organic matter, and 50 percent pore space for air and water. Organic matter, air, and water in a soil allow it to support a tremendous amount of animal and plant life, most of which is invisible to the naked eye.

The word ‘topsoil’ is often used to describe a broad range of soil types. It may refer to high-quality river-bottom loams suitable for intensive agriculture or to the top layer of the soil resource, generally the most fertile slice.

In Washington, topsoil is defined in the reclamation law [RCW 78.44] as the “naturally occurring upper part of a soil profile, including the soil horizon that is rich in humus and capable of supporting vegetation together with other sediments within four vertical feet of the ground surface”.

In Oregon, soil salvage requirements are determined on a site- specific basis.

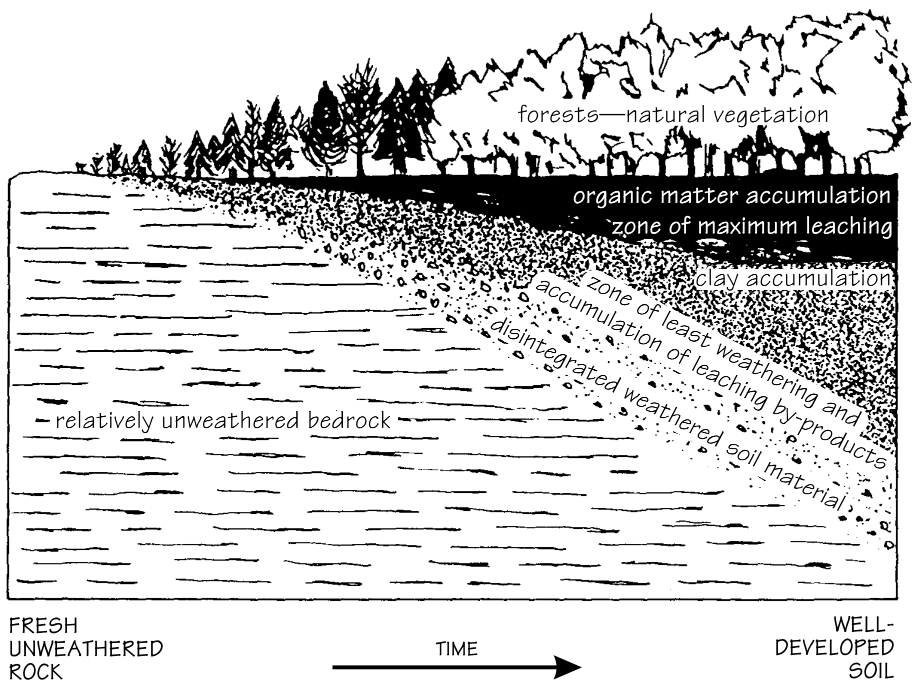

Figure 3.8. Soil profile development over time. Organic matter accumulates in the upper horizons, and the rate of accumulation is dependent on the type and amount of vegetation present. Clay and the by-products of chemical leaching accumulate in the lower horizons. (Modified from THE NATURE AND PROPERTIES OF SOILS, 8/E by Brady, ©1974. Reprinted by permission of Prentice-Hall, Inc., Upper Saddle River, NJ.)

Soil Development

Soils may be defined in terms of soil profile development (Fig. 3.8). Weathering creates chemical and physical changes in bedrock or other parent material. Over time, layers or soil horizons develop. A soil horizon is chemically and/or physically different from the soil horizons above or below. A soil horizon may be leached of certain minerals, or it may be altered by the deposition or formation of other minerals.

Plants decay and contribute organic matter to the top of the soil profile (topsoil). This is where organic matter accumulates and the maximum leaching of minerals occurs. Water moving through the upper soil carries clay and dissolved minerals to deeper layers (subsoil).

The conceptual soil profile in Figure 3.9 shows the major horizons in a soil weathered from bedrock. Climate is the most influential factor in soil formation because it determines the degree of weathering that occurs. Thin, poorly developed soils are common in

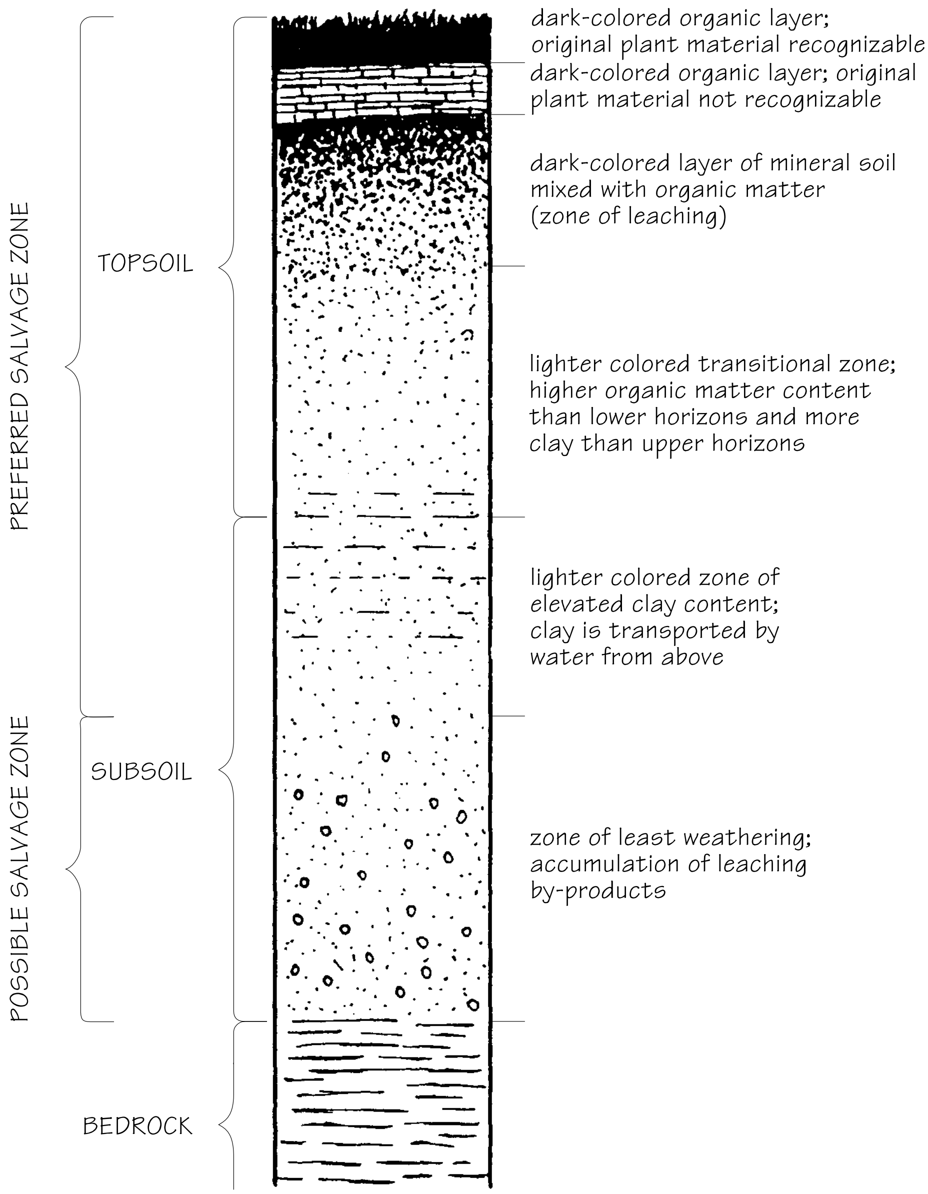

Figure 3.9. A diagrammatic sketch of the residual soil profile that develops over time on a bedrock surface. The thickness of the layers can vary widely within a mine site and between nearby sites. No scale is intended here. (Modified from THE NATURE AND PROPERTIES OF SOILS, 8/E by Brady, ©1974. Reprinted by permission of Prentice-Hall, Inc., Upper Saddle River, NJ.)

arid areas, whereas thick, well-developed soils are common in wetter areas.

Topsoil can be identified by its dark color and organic content. It also has a high water-retention capacity. Subsoils commonly contain fewer nutrients. Overburden is the material removed to allow access to the material that is being mined. At most aggregate operations, overburden consists of clay and silt that is poorly drained. Examples include volcanic ash overlying basalt or decomposed rock that overlies an unweathered rock.

Soil Fertility

Soil fertility is created by the recycling of organic matter and the weathering of minerals. Soil systems continually produce and recycle organic matter through the vegetative cover they support. Organisms in soil convert organic matter (through decomposition) to a form plants can use. Decomposition of organic matter also produces fairly strong acids that can react with minerals in the soil to extract base cations such as Ca^{++}, Mg^{++} and K^{+}, which are essential for plant growth.

Unweathered geologic materials and subsoils are typically less desirable as reclamation media for mined lands because they lack the organic matter and elevated concentrations of dissolved minerals found in more fertile soils.

Soil Types

Rocks weathering in place form residual soils. Eolian, alluvial, or colluvial soils form from weathering of materials deposited by wind, water, or gravity, respectively. Alluvial soils, although they are generally young soils with poorly developed soil profiles, are typically fertile because they include silts and flood deposits containing abundant organic debris.

Soil Inventories

The Natural Resource Conservation Service (NRCS, formerly the Soil Conservation Service) is responsible for classifying, naming, and mapping the nation's soil resources. Traditionally the mapping focus has been on the agricultural suitability and fertility of soils. NRCS soil surveys also provide information about erosion hazards, flooding potential, soil stability, and suitability for various uses, including drain fields, road building, timber harvesting, and housing development, as well as information on suitable trees to plant and potential wildlife habitat and recreational development.

For most areas, Order III soil surveys are available as published or unpublished maps on a countywide basis. Unpublished surveys may be available at the local NRCS office; published surveys should be available at the local library. Order III maps are at a scale of 1:20,000. Boundaries are field checked, but most of the mapping is done in the office from aerial photographs.

In an Order III survey, soils are grouped into 'associations' and 'complexes' on the basis of genetic similarities. That is, if soils have the same parent material and have been subjected to the same soil-forming processes, they may be grouped together on an Order III

Survey map, even though the depth of the individual soils in the group may be significantly different.

For mine development and reclamation, it is important to know how much soil is present and where it is in the project area. Order I and Order II soil surveys can provide this information. They are commonly available for areas of intensive agricultural production and can be obtained from the NRCS, DOGAMI, or DNR.

On-site soils investigations can be accomplished with a backhoe or a shovel and a hand auger. If the mine operator is doing the soil investigation, the NRCS, DOGAMI, or DNR should be contacted for information about soil types at the mine site and for recommendations on how to handle them. Understanding the approximate fertility level of each soil type and different soil horizons will contribute to wise use of the resource.

REMOVING AND STORING TOPSOIL AND SUBSOILS

Topsoil, subsoil, and overburden should be removed separately before mining and retained for reclamation. Placing several inches of soil with elevated organic matter over a lower quality subsoil material can make a dramatic difference in revegetation success. If adequate soils are not reserved to accomplish the approved reclamation plan, miners may need to import soil—often at considerable expense. It is important to ensure that soil resources are protected and used to their maximum potential, because few mine operations can afford to import soils.

The pore space in soil is essential for the proliferation of bacteria, fungi, algae, and soil-dwelling insects and worms. One gram of soil may contain as many as 3 billion soil bacteria. Consequently, soils must be properly handled and stored to protect both the pore spaces and soil organisms. Porosity, or structure, can be permanently damaged if soils are stripped when they are excessively wet or dry. This is a particular problem with clay-rich soils and loams. Stockpiling aggregate on top of a soil stockpile, compaction caused by the passage of heavy equipment, burial by overburden, or creation of large soil stockpiles can destroy the dynamic qualities of a soil.

Live Topsoiling

'Live topsoiling' means placing stripped soil directly onto an area that has been mined out or backfilled or on a reshaped surface for reclamation (Fig. 3.10). Soil should be spread with a minimum of equipment traffic to avoid compaction and protect pore spaces. Because the soil contains viable seeds and the soil organisms are relocated to the same ecological niche, revegetation can occur within a short time (Munshower, 1994).

In both Washington and Oregon, live topsoiling is recommended wherever possible. However, live topsoiling may not be practical, particularly in quarry operations where concurrent reclamation opportunities are limited or where the soil contains noxious or undesirable weeds and the site is being reclaimed to cultivated cropland.

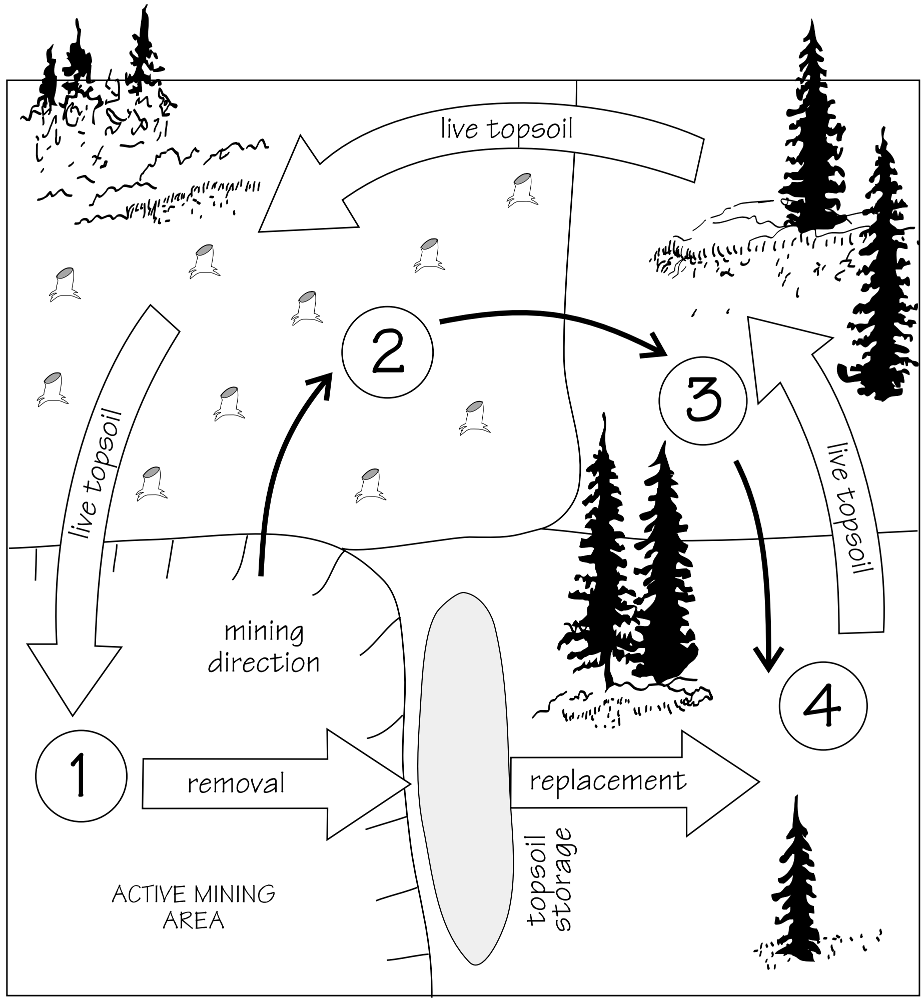

Figure 3.10. Topsoil handling in a four-segment mine. Segment 1 is the first to be mined. Its topsoil is removed and stored just inside segment 4. When mining of segment 1 is finished, topsoil is taken from segment 2 and place directly on segment 1 (live topsoiling). The topsoil from segment 3 is placed on segment 2. The topsoil from segment 4 is placed on segment 3. When mining is completed, the stockpiled topsoil from segment 1 is used to reclaim segment 4.

Stripping and Salvage

Before soils can be stripped and stockpiled, areas to be stripped and storage areas should be marked. (See Fig. 1.3.) Equipment operators who are stripping soils by horizon or separating soils from subsoils should have enough information to identify and segregate topsoil, subsoil, and overburden. A color change is typically the most obvious indicator of a change in soil horizons. Soil horizons that contain a fairly large amount of organic matter can generally be recognized in the field by their darker color and position at the top of the soil profile. Another technique is to identify stripping depths on survey stakes placed on 100 to 200 foot centers. It is best to move the soil only once. This also reduces operating costs.

By law in Washington [RCW 78.44], topsoil needed for reclamation cannot be sold or mixed with sterile soil unless specific authority has been granted in the permit documents. Subsoils capable of supporting vegetation must be salvaged to a depth of 4 feet and stored in a stable area if not immediately used for reclamation.

In Oregon, subsoil salvage depth must be adequate to accomplish reclamation according to the approved plan.

Constructing Storage Piles

Choosing an appropriate method for storage pile construction is also important. Continually driving heavy equipment over the soil while constructing scraper-built or end-dump piles can permanently damage soil structure and reduce the pore space essential for micro- organisms. This type of construction should be avoided.

Soil storage piles should be constructed to minimize size and compaction so soil organisms can 'breathe'. Extensive experience and research have shown that the size of soil storage piles can significantly affect soil viability (Allen and Friese, 1992). Soil storage piles should be no more than 25 feet in height. Available plant material such as grasses, shrubs, and chipped tree limbs should be incorporated into the piles. However, if large amounts of woody material are added, soil may become nitrogen deficient.

Soil storage piles should be revegetated. They are good areas to do test seedings to prepare for final revegetation. To retain soil microbes deep in the soil pile, it can be aerated by deep ripping, discing, and tilling every 2 or 3 years.

Recent research (Allen and Friese, 1992) has shown that soil microbes can be regenerated in sterile soils by spotting live soil throughout the area and by using inoculated trees and shrubs. Microbes will spread to other areas in a relatively short time (weeks to a few months).

WASTE AND OVERBURDEN DUMPS AND STOCKPILES

Large amounts of overburden exist at many mine sites, and operations frequently create large volumes of waste rock. Dumps and stockpiles are created to temporarily or permanently store both overburden and unwanted material separated from the salable product on the site, for example, crusher scalpings, oversize material, and reject fines. During reclamation, overburden and waste can be used to create landscape diversity. It is important to plan the location of overburden or waste piles so they can be used in reclamation.

Site Selection

Dumps and stockpiles can result in landslides and increased sediment load that may pollute nearby waters if they are not properly designed and maintained. Careful planning is necessary to ensure that dumps and stockpiles are placed in a geologically stable location, and that they can be revegetated successfully. Locations next to waterways or springs or seeps will probably not be acceptable. Ideally, from both construction and water-quality protection standpoints, these materials should be removed and placed only during dry periods.

Site Preparation

Storage sites for overburden and waste rock should be properly prepared. All vegetation, soil, and subsoil must be stripped from the site prior to dump construction. Any buried vegetation will rot; this soft material provides little resistance to sliding and increases the potential for downslope movement. Slash cleared from the stockpile area can be used below the stockpile to filter runoff. (See Slash Windrows and Brush Sediment Barriers, p. 2.12.)

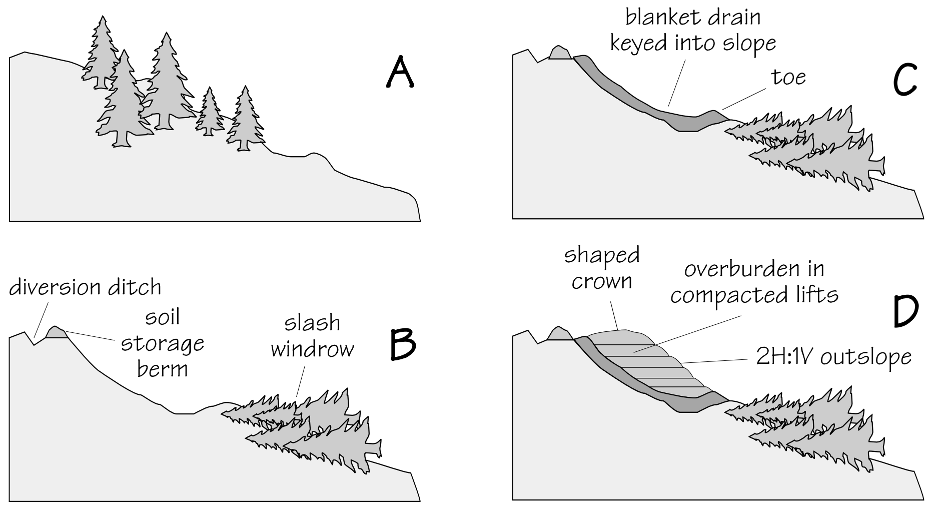

Figure 3.11. Proper procedures for waste dump construction. Trees removed from the site are used to construct a slash windrow to filter runoff. A blanket drain (a French drain that covers a slope instead of being confined to a trench; see Trench Subdrains and French drains, p. 2.20 and Fig. 6.6.) is laid down first to prevent the buildup of water, and the dump itself is constructed of thin, compacted layers.

Before overburden is stockpiled, all vegetation should be cleared, and the drainage for the pile must be prepared. Undrained and uncompacted fill dumped over vegetation without drainage is prone to mass wasting and landslides that waste topsoil. Soil placed over permanent waste piles will promote self-sustaining vegetation. (See Topsoil and Overburden Storage Areas, p. 3.6.)

Large dumps and stockpiles or those located on steep ground should have diversion ditches constructed above them (Fig. 3.11B). (See Contour and Diversion Ditches, p. 2.17.) A blanket drain should be installed on any slopes where drainage problems are anticipated (Fig. 3.11C). (See also Trench Subdrains and French Drains, p. 2.20.)

Dump and Stockpile Construction

Stability is important, particularly for dumps that will become permanent features. Both dumps and stockpiles should be constructed using thin, compacted layers (Fig. 3.11D). Before compaction, layers may be as thin as 12 to 18 inches. When compacted by rubber-tired equipment, they will result in a much more stable dump than one prepared by simply end-dumping or pushing with a bulldozer.

Dumps and stockpiles on hillsides or filling ravines need a properly constructed toe to key the pile into competent material. The toe should have a blanket drain to prevent the buildup of water. (See Fig. 6.6.)

Dumps and stockpiles should be shaped to prevent water from ponding. The top should be sloped to direct runoff to a drainage system and to avoid critical areas, or it should be crowned to disperse runoff around the perimeter. The slopes of the dump or stockpile should be constructed with appropriate runoff control structures. The top and overall shape should be rounded off to blend into the natural topography. (See Slope Stabilization, p. 6.6.)

Most final slopes should be between 2H:1V and 3H:1V. Generally, the flatter the slope, the more stable it will be and the easier to access for reclamation. Terraces should be constructed at 30-foot intervals vertically, or other methods of slope shaping should be use to reduce water velocities.

When shaping is complete, the dump or stockpile should be seeded and mulched to establish vegetation.

DUST CONTROL

Neighbors often complain about dust from mining operations. Dust is generated by the crusher, rock drills, and other mining equipment, and from disturbed areas, including haul roads and stockpiles.

In Washington, the Department of Ecology or the local air pollution control authority has review and permit authority over rock crushers, batch plants, fugitive dust emissions from mining operations, and haul roads. Contact these agencies for further information.

In Oregon, emissions from on-site processing require a permit from the Department of Environmental Quality.

Controlling Dust with Water

Controlling fugitive dust is usually a matter of frequent application of water or chemicals. Water trucks are typically used for conveying these liquids. However, sprinklers and irrigation pipe installed in the berms alongside haul roads can significantly decrease dust without the expense of using a water truck several times a day.

Controlling Dust with Chemicals

Chemical dust suppressants, such as magnesium chloride, are appropriate where water is in short supply. Most chemical dust suppressants require repeated application. There are numerous chemical dust suppressants designed for a variety of uses. The local and state water-quality agency can provide information about appropriate chemicals and how to apply them.

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4 Restoring Landforms

INTRODUCTION

Land shaping is an important but often underemphasized part of the reclamation process. Common objectives for land shaping include:

- minimizing erosion,
- reducing slope angles to provide stability for post-mining development,
- contouring aesthetically pleasing landforms to blend with the surrounding area,
- forming shapes and slopes consistent with the subsequent use planned for the site (Fig. 4.1),
- increasing revegetation success, and
- providing diverse wildlife and fish habitat.

SUBSEQUENT USE

Reclamation of a mine site, and thus its subsequent use, can be driven by high land values, zoning, and/or environmental protection and the state regulations that set minimum standards for reclamation and water quality.

In urban areas, high land values motivate miners to reclaim for intensive use. For example, in Portland, Oregon, gravel pits are typically backfilled with construction waste and developed as building sites. Building sites can also be developed directly without backfilling. Government-owned sites where the water table is high often become parks with ponds. In rural areas, less intensive uses such as wildlife habitat, agriculture, or timber production can also be profitable. (See Agricultural and Forestry Subsequent Uses, p. 7.17.)

Imagination and careful planning can yield a wide variety of landforms that make the site better for a specific use than it was prior to mining. For example, wetlands and fishing ponds can be created from rock quarries and gravel pits if proper water conditions exist. Many agricultural sites have been enhanced by selective gravel removal, making them easier to irrigate or till after gravel-rich knobs

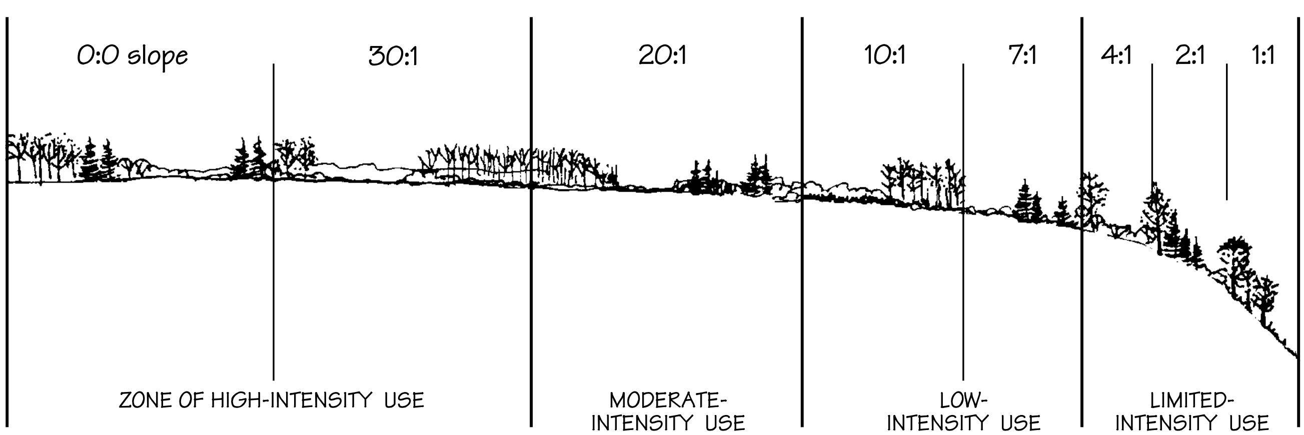
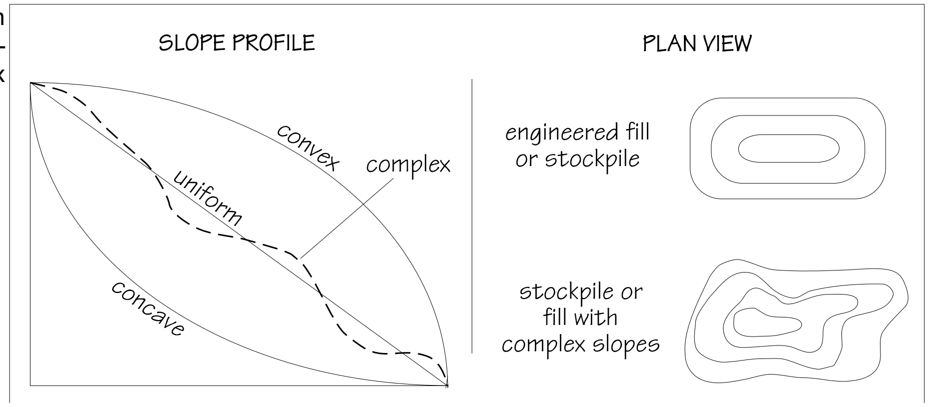


Figure 4.1. The steepness of the final slope strongly influences the intensity of proposed land use for reclaimed mine sites. Fewer options are available on steeper slopes. (From Green and others, 1992.)

Figure 4.2. **A**, profile of common slope types. **B**, plan view of different stockpile designs. Complex slopes are preferred.



have been selectively removed from the fields. Mining can level areas of hilly topography making them more suitable for agricultural or industrial uses. In eastern Oregon and Washington, many of the mine sites developed on rangeland are returned to their previous condition by revegetation, generally with native species.

In Washington, RCW 78.44.031 identifies subsequent use as a criterion for guiding the reclamation scheme, while RCW 78.44.141 sets forth reclamation standards that must be met for various uses.

In Oregon, the subsequent use of the mined land must be compatible with the local comprehensive land-use plan.

SLOPE TYPES

Profiles of four basic slope types are shown in Figure 4.2. Convex slopes erode rapidly and yield the most sediment. Concave slopes are less affected by erosion and typically yield less sediment than convex slopes. The steepness of the slope is a major factor influencing the amount of sediment production. Surface-water runoff velocities are higher on longer, steeper slopes, and more soil particles are typically dislodged and transported. Sediment production on uniform slopes is intermediate between concave and convex slopes. Long uniform slopes should be avoided because they can be severely eroded in a single storm event.

☛ Complex slopes generally produce the least sediment and are the most stable. Complex slopes are preferred for mine site reclamation.

CREATING SLOPES

Where the goal of reclamation is to restore natural slopes that blend with surrounding landforms, sinuous slopes that are curved in plan and section and irregular in profile should be created (Fig. 4.3). Irregular slopes will intercept more runoff and reduce its velocity, trap seeds, and speed revegetation. Rectilinear slopes should be avoided because they are prone to sheet erosion and gullying and because they look unnatural.

Natural-looking topography can be achieved early on through a well-planned extraction operation and equipment operators who fully understand the post-mining use of the site. Sinuous slopes can be formed by mining to the prescribed angles (generally the most

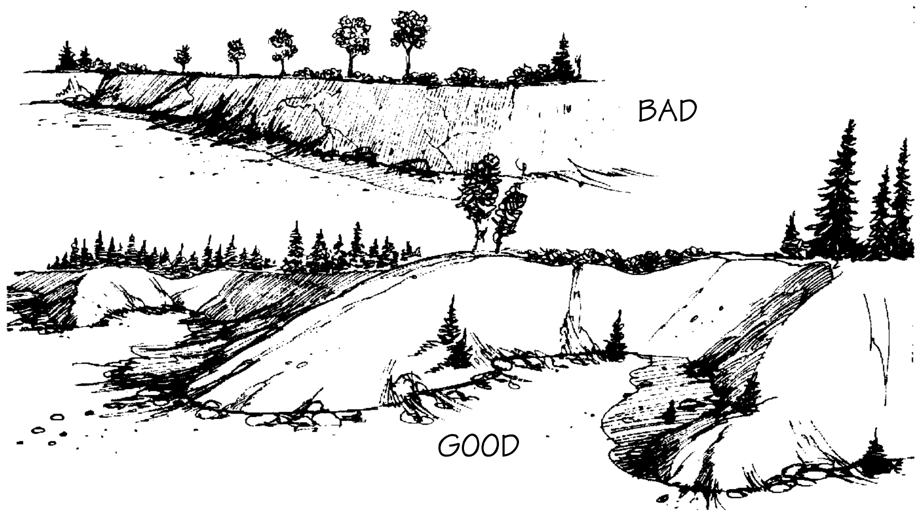


Figure 4.3. A key element in restoring topography is creating natural-looking slopes that blend with the surrounding landforms. Rectilinear slopes (top) are inappropriate for reclamation in unconsolidated materials. Slopes should be curved in plan and section and irregular in profile (bottom). (Redrawn from Green and others, 1992.)

inexpensive means of reclamation) or by using the cut-and-fill method, which requires a reclamation setback or material from overburden stockpiles. (See Reclamation Setbacks, p. 3.5.) Backfilling to create appropriate slopes can be the most expensive reclamation technique when it is done after mining.

A reclaimed site should consist entirely of stable slopes. A rule of thumb is that slopes are unstable if pioneer plants cannot establish themselves naturally, if the slopes ravel or show signs of soil creep and tension cracks, or if landsliding is noted. (See Identifying Unstable Slope Conditions, p. 6.3.) In general, unconsolidated materials are stable and can sustain vegetation at slopes of 3 feet horizontal to 1 foot vertical (commonly expressed as 3H:1V) (Fig. 4.4) (Norman and Lingley, 1992).

For variety, a few locally steeper areas (1.5H:1V to 2H:1V) may be created (if stable), especially if they mimic locally steeper slopes nearby. However, steep slopes greatly increase the potential for erosion. Long, steep slopes produce more and faster runoff and allow less infiltration than a series of short, gentle slopes separated by

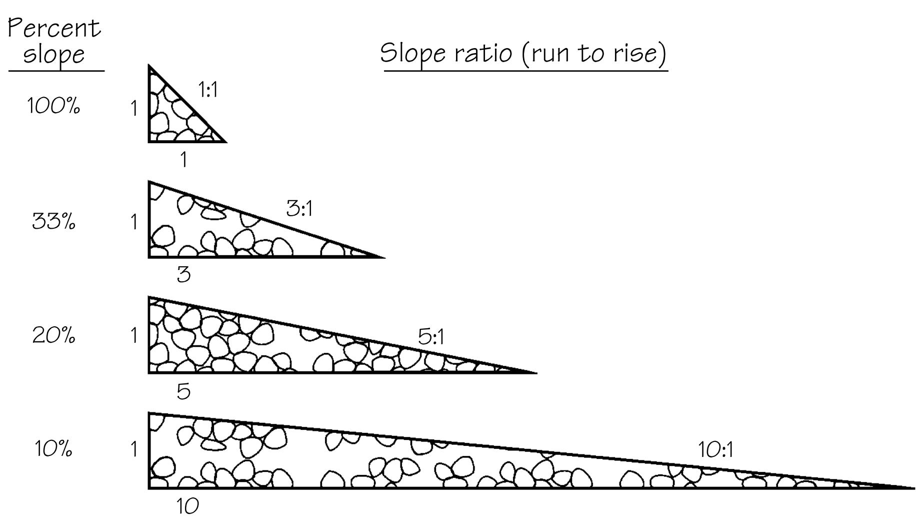


Figure 4.4. Slopes are expressed as the ratio of horizontal run to vertical rise. This diagram shows the percent slope of several common ratios. (Redrawn from Green and others, 1992.)

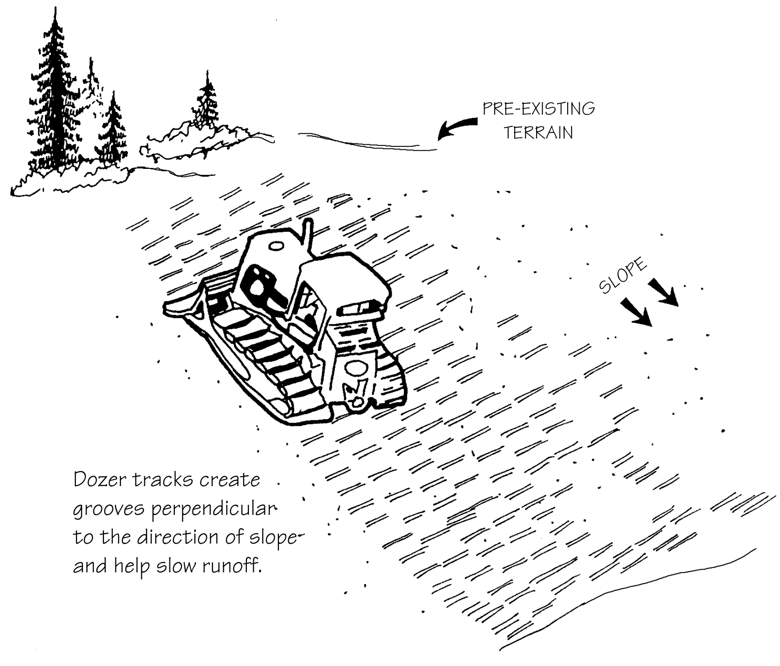


Figure 4.5. Dozer tracking can reduce runoff and enhance revegetation. Tracked equipment should be run up and down a slope, not across, to increase slope roughness. (Modified from Law, 1984.)

benches or terraces. New drainages or contour ditches should be established within the reclaimed area to contain the expected surface water runoff. Any water diverted during reclamation or land shaping should be directed to the drainage it occupied before mining to prevent drying up or flooding of areas downstream. This water should have approximately the same velocity, volume, and quality as the drainage it is entering.

Some guidelines for slope shaping are:

- ☛ Slopes steeper than 3H:1V should be kept shorter than 75 feet by creating breaks in slope, such as irregular terraces, berms, or basins. (See Figs. 2.3 and 2.4.)
- ☛ If the site is to be dry after mining, then pit floors should be graded to a slope of 2 to 5 percent to promote drainage.
- ☛ Some mounds, hills, and depressions can be left on pit floors to vary the topography for subsequent use (Norman and Lingley, 1992).
- ☛ In the final grading, bulldozers or other tracked equipment should be run up and down a slope, not across it, to increase slope roughness (Fig. 4.5). (Older bulldozers are generally unable to back up sand and gravel slopes steeper than 3H:1V.)
- ☛ Final slopes should be revegetated immediately to minimize erosion.

REGRADING

After the land has been shaped, it should be regraded to produce a rough, irregular surface, particularly on slopes (Fig. 4.5). This ensures that replaced soil is keyed into the substrate to slow erosion.

Roads, pit floors, and stockpile areas should be ripped at close intervals to provide drainage prior to replacing the soil. Placing a loose, friable soil over a compacted base does not increase soil moisture-holding capacity, drainage, or slope stability and will result in inadequate root development and penetration. A good rule of thumb is that ripper spacing should be less than or equal to the depth of ripping.

REPLACING TOPSOIL AND SUBSOIL

Understanding the soil resources of a site and the post-mining land use will lead to effective site development, using the best management practices for soil replacement. The type of vegetation planned for reclamation may dictate soil replacement depth. Deeper soils will be needed for agricultural production or establishing trees, particularly for timber production. More important than the depth of the replaced soil is how replacement is done. Soils should not be compacted. The less equipment is run over soils, the better. The most skilled and experienced equipment operators should be used for soil replacement—their skill will pay off.

Topsoil should be replaced on slopes as soon as possible after restoring topography. Soil horizons from stockpiles should be replaced separately in the proper order for best use of the resource. After the topsoil is spread, it should be tilled to construct a proper seed bed.

A minimum soil replacement depth of 12 inches of topsoil is recommended for reclamation for most post-mine uses. Upland sites may have soil depths, prior to mining, of 6 inches or less. On these sites, reject soil fines and rock fines produced during rock processing may be used to supplement pre-existing soil resources as a growth medium. Generally fines would be mixed with organic material and put in place before the topsoil is added.

The minimum recommended soil depth for timber production is 4 feet over rock and 2 feet over gravel or soft overburden to establish an effective rooting depth of 4 feet. Timber growth rates are generally directly related to the depth of the soil available.

A common problem in reapplying topsoil and subsoil is spreading them too thickly initially so that little is left for remaining areas. If the volume of topsoil at the site is limited, its application should be restricted to low areas or excavated depressions that will conserve soil, retain moisture, and catch wind-blown pioneer seeds. These low areas are also ideal sites for planting trees.

Varied soil replacement depths mimic natural soil-forming processes and should be incorporated into reclamation strategies where possible. Thinner layers of soil on the upslope areas and thicker layers on the lower slopes may naturally encourage different vegetation types. These parts of the slopes should be planted differently to encourage post-mining vegetation diversity.

In Washington, topsoil is defined as the naturally occurring upper part of a soil profile, including the soil horizon that is rich in humus and capable of supporting vegetation, together with other sediments within 4 vertical feet of the ground surface [RCW 78.44].

In Oregon, topsoil is not defined by law; however, sufficient soil must be retained onsite for reclamation.

AMENDING OR MANUFACTURING SOIL

Where little or no topsoil exists prior to mining, it may be necessary to amend or even manufacture soils. Amending soil can significantly reduce the time required for revegetation and performance security release. (See The Soil Resource, p. 3..)

Reconstructed soils should have the same soil characteristic as topsoil. Soil characteristics that have the greatest effect on plant growth are the amount of organic matter, moisture-holding capacity, drainage, and available nutrients.

Adding Organic Matter

Organic matter improves both the fertility and physical condition of a soil. The chief problem with using subsoils for reclamation is usually a lack of organic matter. Subsoils can be used in place of topsoils if they are combined with organic products, such as wood chips, paper sludge, rice hulls, mushroom compost, mint clippings, farm manure, processed municipal biosolids, straw, or native hay. In some instances, trading loads of rock for manure and straw from local dairies, farms, and ranches may be mutually beneficial. However, weeds should not imported with the manure or straw. Knowing the quality of the hay can prevent this from happening.

Quarry sites are generally developed where mineable rock is at or very near the surface. In these cases, reject fines, scalpings, or other fine-grained materials can be used to replace topsoil, provided they are amended with organic matter.

Biosolids and some other soil amendments may not be appropriate at sites near sensitive aquifers or waterways.

A solid waste permit from the local health district may be needed for application of biosolids, paper mill sludge, manure, etc. In Washington, contact the Department of Ecology. In Oregon, contact the Department of Environmental Quality or the local health department.

Improving Moisture-Holding Capacity

In the arid regions of the Pacific Northwest, the moisture-holding capacity of a soil is often the factor limiting planting success. A thick soil will hold more water than a thin one, and clay soils will hold more water than sandy soils. Moisture-holding capacity can be increased by adding large amounts clay or other fine-grained geologic material or by increasing the thickness of the subsoil. A mulch layer at the surface also helps conserve water by insulating the soil against evaporation.

Improving Drainage

In areas that are not being developed as wetlands, soils that do not drain well can cause plants to rot. Adding organic matter, sand, or other coarse materials improves drainage by modifying the structural characteristics of a soil. Adding lime or gypsum neutralizes acidic soils, which usually develop in wet areas.

Using Fertilizers

Natural Fertilizers. Adding organic matter can improve both the fertility and physical condition of a soil or fine-grained substitute. However, it may not provide any short-term fertility benefits and possibly no long-term benefits unless it is worked into the top 6 inches of soil. The smaller the particle size and the greater the surface area of the fertilizer, the faster it will be broken down by soil microbes.

The natural range of carbon to nitrogen in soils is 8:1 to 15:1. Organic amendments that help reclaimed soil achieve this ratio provide significant benefits. For example, amendments high in carbon and low in nitrogen, such as wood chips, may require additions of nitrogen-rich fertilizers (Table 4.1). This is because when an organic amendment rich in carbon is added to the soil, all the nitrogen available to plants will be tied up by soil microbes trying to consume the carbon. Soil microbes need nitrogen to consume the carbon and can preferentially absorb nitrogen before plant roots can use it. This means that there will be no nitrogen available to plants until the carbon: nitrogen ratio has dropped to 8:1–15:1. Therefore, adding amendments high in nitrogen will help plants grow under these conditions. Amendments in which carbon greatly exceeds nitrogen should be used sparingly.

Table 4.1. Nitrogen and carbon content of common organic soil amendments. The natural range of carbon to nitrogen in soils is 8:1 to 15:1. Organic amendments that help reclaimed soil achieve this ratio provide significant benefits. (Modified from FERTILIZERS AND SOIL AMENDMENTS by Follett, Murphy, and Donahue, © 1981. Reprinted by permission of Prentice-Hall, Inc., Upper Saddle River, NJ.)

Material	Organic Carbon (C) (%)	Total Nitrogen (N) (%)	Carbon: Nitrogen (C:N) Ratio
Sewage sludge (dry weight basis)			
Aerobic	35	5.60	6:1
Anaerobic	30	1.90	16:1
Alfalfa hay	43	2.40	18:1
Grass clippings, fresh	43	2.20	20:1
Leaves, freshly fallen	20–80	.50–1.00	40:1–80:1
Peat moss	48	.83	58:1
Corncobs	47	.45	104:1
Red alder sawdust	50	.37	135:1
Paper, mostly newspaper	43	.26	172:1
Hardwood sawdust	50	.20	250:1
Douglas fir			
Old bark	59	.20	295:1
Sawdust	51	.07	728:1
Wheat straw	45	.12	375:1

Chemical Fertilizers. If a quick cover of vegetation is needed to provide erosion control or if the soil or manufactured soil substitute is of poor quality, applying a fertilizer is recommended. Organic matter should be added to achieve a long-term response before seeding directly into soil substitutes. Avoid applying fertilizers in areas where runoff into streams could occur.

Some research shows that native plants do not respond well to chemical fertilization, and fertilizers are not generally needed for the long-term survival of these species. Fertilization tends to depress plant community diversity by indirectly decreasing desirable native plant populations such as warm season grasses and legumes. Fertilizers tend to give a competitive advantage to opportunistic species such as annual grasses and herbaceous plants, many of which are weeds.

RESTORING DRAINAGE

Where the pit or quarry is mined below the water table or surface drainage collects on the mined property, productive ponds and wetlands can be formed with careful water management.

Where appropriate to the subsequent use, a pond creates additional plant and habitat diversity, even though it may contain water only on a seasonal basis. Shallow process-water ponds, as well as low places on excavation floors and in stockpile areas at upland sites, can be developed as seasonal wetlands, even in arid areas east of the Cascades.

Extraction ponds (ponds being mined for gravel) and some upland rock pits with a permanent water source make ideal sites for constructing wetlands if the water table is shallow. Sediment from washing and screening rock can be deposited to form shallow deltas that, when combined with the permanent water supply, can easily be revegetated with wetlands species.

CREATING PONDS FOR WILDLIFE

Ponds for wildlife habitat should have irregular outlines (Fig. 4.6). The bottom of the pond should also be irregular so as to offer a variety of habitat possibilities for plants, bottom dwellers, and fish (Fig. 4.7). Both water deeper than 10 feet and benches and bars with water depths less than 2 feet should be provided. As a general rule, 25 percent of the pond should be less than 2 feet deep, 25 percent 2–6 feet deep, and 50 percent deeper than 10 feet. Water deeper than 15

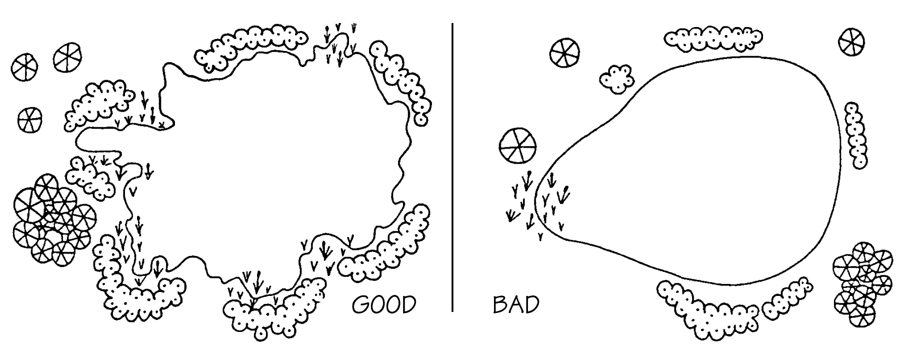


Figure 4.6. The shoreline of ponds used for wildlife habitat should be irregular and planted for cover with a mixture of open meadows and shrubs in the surrounding area. The shape of the pond on the left is better suited to supporting wildlife than that of the pond on the right. (Redrawn from Szafoni, 1982.)

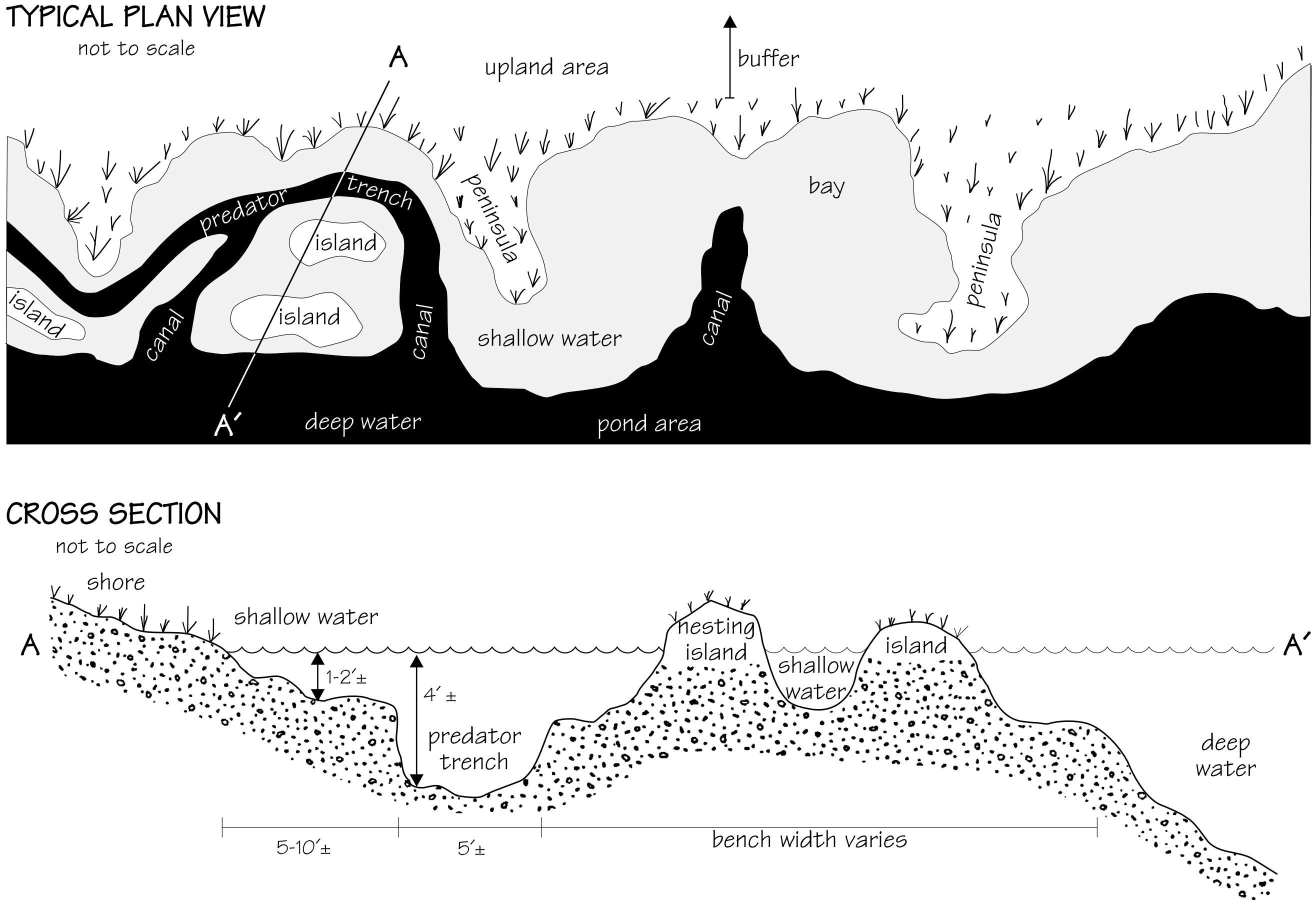


Figure 4.7. Plan view and cross section of a well-designed irregular wetland or pond shoreline. Note the large areas of shallow water. Steep slopes along parts of the shore will discourage the growth of wetland plants and provide clear access to the pond. Bird nesting sites are provided. The trench discourages predators, but the shallow water offers sites for food for fish and cover plantings. Islands can be constructed from fill, unmined material, or sediments saved from digging the trench.

feet can provide a cool summer refuge for fish (Norman and Lingley, 1992).

In-Water Slopes

Slopes should be very gentle, 5H:1V or flatter, to allow development of wetland plant species (Fig. 4.8). In general, the more shallow areas, the better. Slope variations will enhance the plant diversity in created wetlands.

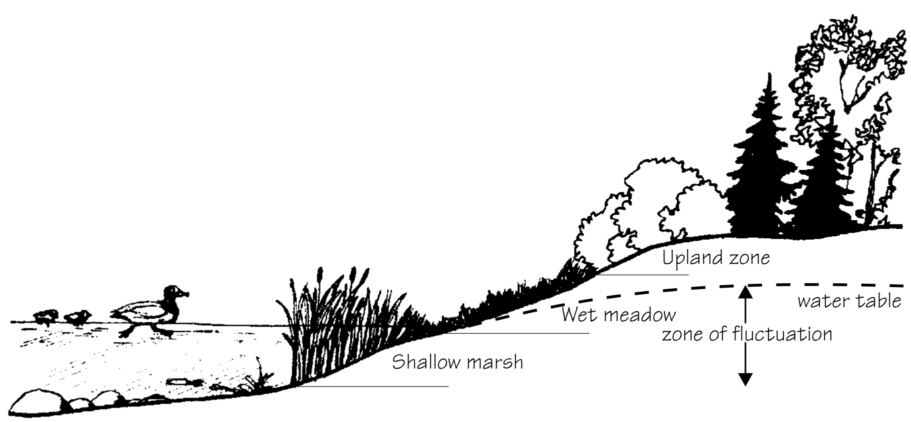


Figure 4.8. Slope variations will enhance the habitat diversity of created wetlands. To successfully establish wetland vegetation, seeds and transplants must be placed in sites with the correct water depth. (Modified from Green and others, 1992.)

The most economical means of shaping final pond slopes is to create them as material is excavated (Fig. 4.9). In mines that are being dewatered while operations proceed, resloping must be done before allowing the pits to fill with water.

Windward pond shores can be protected from wave erosion by placing boulders at the range of pond levels.

In Washington, slopes in unconsolidated materials (sand, gravel, or soil) below the permanent water table should not be steeper than 1.5H:1V. Slopes at the water/land interface should be between 2H:1V to 3H:1V. Solid rock banks must be shaped so that a person can escape from the water in those places.

Oregon statutes require a 3H:1V slope to 6 feet below the low-water mark of a pond to provide a means of escape in the event that someone were to fall in.

Special Considerations Near Rivers

Mining sand and gravel near a river can eliminate wetlands and fish and wildlife habitat, cause channelization of the river, and may even result in channel capture, if not planned properly. If mining is allowed by local jurisdictions, leaving ponds and depressions can replace lost fish and wildlife habitat and wetlands. By locating mining sites in relatively stable areas of the flood plain and not excavating overly deep or large pits, reclamation of fish and wildlife habitat can be done without extensive engineering to ensure river stability.

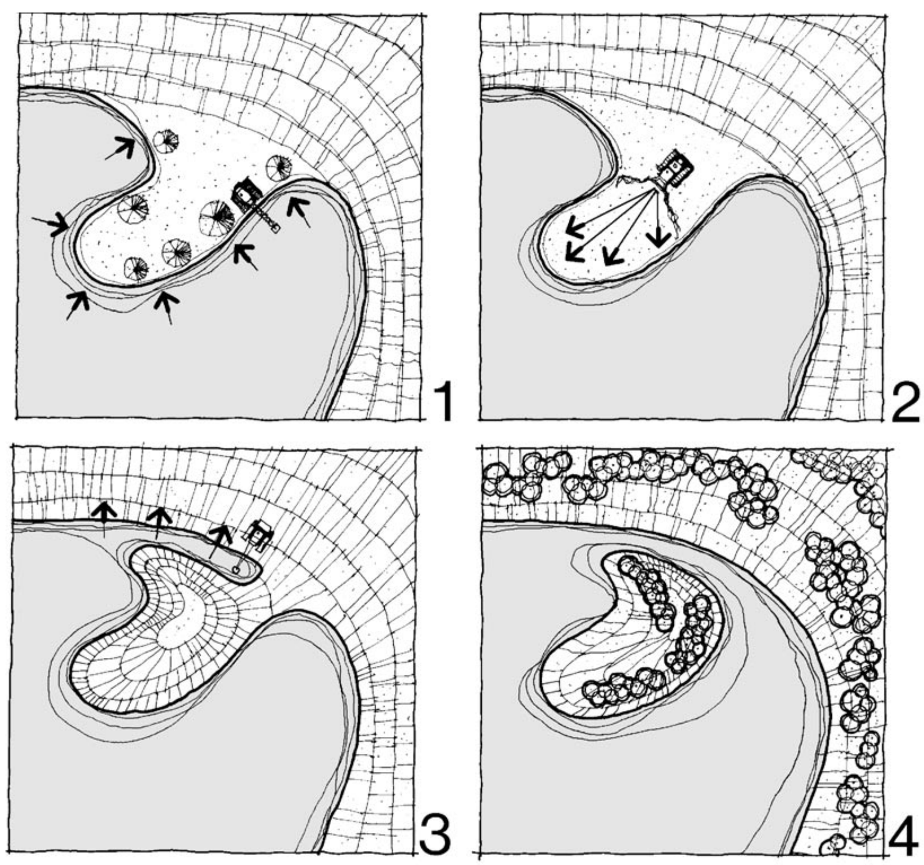


Figure 4.9. Islands can be developed in undrained pits during operations. They start as peninsulas (1), which are then graded to provide the appropriate final shapes and slopes (2). Channels can then be dredged to separate the tips of the peninsulas from the mainland (3). Step 3 should not be undertaken until final water levels are known. (4) Final configuration of constructed island. (Redrawn from Michalski and others, 1987.)

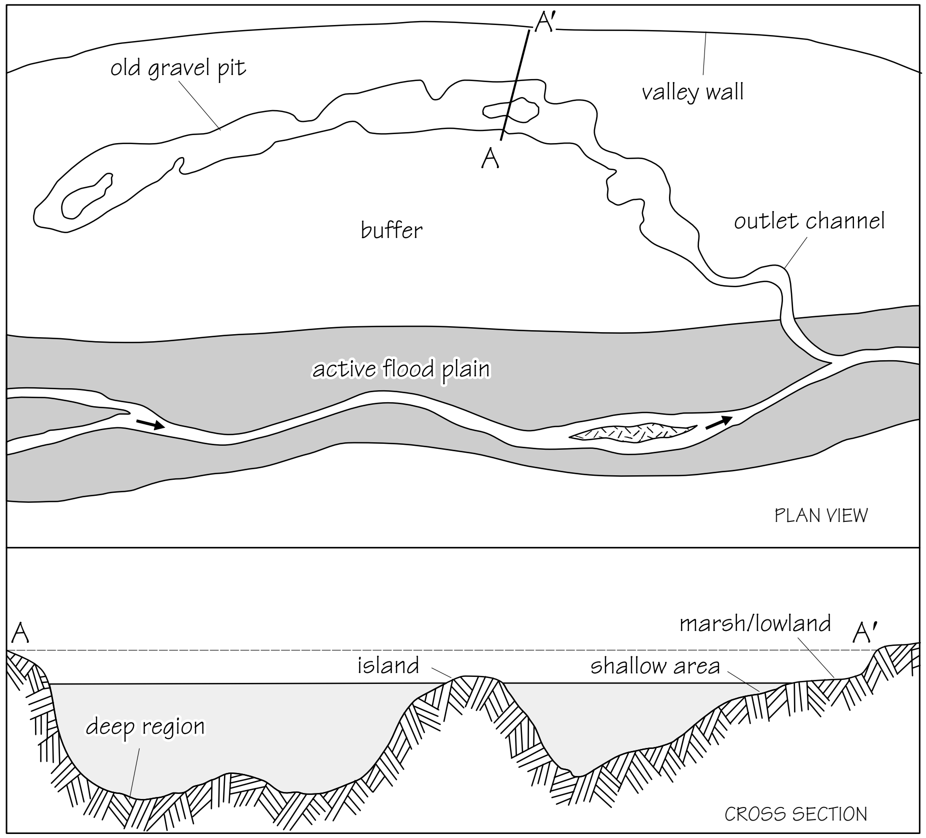


Figure 4.10. Plan view and cross section of a reclaimed gravel pit with pond shape that mimics a natural river system. Not to scale. (Modified from Woodward-Clyde, 1980.)

A desirable post-mining pond configuration for a gravel pit near a river is long, narrow, and moderately deep, with irregular islands and peninsulas. It should be connected to the river on the down-stream side (Fig. 4.10) (Woodward-Clyde, 1980) to mimic a natural river system on a flood plain.

BUILDING HABITAT

Subsoils, mine waste rock, construction fill, or boulders can be used to create rock reefs, islands, and other features to provide habitat.

Islands

Islands can be formed as part of the mining process or made after the basic mine shape is in place (Fig. 4.9). If the mine itself consists of individual cells separated by dikes, portions of the dikes can be re-moved to create post-mining peninsulas or islands for use as habitat. If the excavation is dewatered, silt and sand can be compacted or boulders can be placed on the floor of the excavation to create is-lands for bird and turtle loafing.

Many small islands are better than a few large islands. They should range from 0.1 to 0.5 acres if they are meant to provide water-fowl nesting sites. Smaller islands may provide only resting sites, and larger islands may encourage predators to take up residence. Adequate separation of the island from the mainland, with water depths between them exceeding 30 inches, will discourage preda-tors. Soil, logs, and rocks should be placed on the island to enhance habitat diversity.

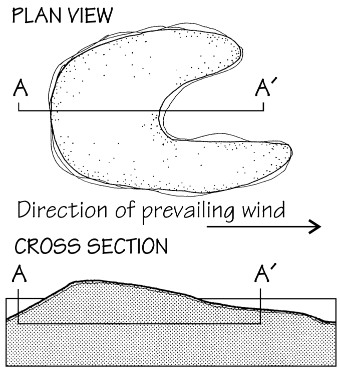


Figure 4.11. Plan view and cross section of a horseshoe island. (Redrawn from Michalski and others, 1987.)

Irregular islands are better than round islands (Fig. 4.8). Horseshoe-shaped islands are ideal for waterfowl (Fig. 4.11). The opening of the horseshoe should be in the lee of the prevailing wind to provide shelter for young birds. The banks between the prongs of the horseshoe should be more gently sloped than the outer banks to increase the sheltering effect.

Structures That Enhance Habitat

To create cover for fish and habitat for aquatic insects, submerged and anchored tree crowns can be placed along steep banks (Fig. 4.12). Where possible, logs and stumps should be lashed together and anchored to form reefs (Fig. 4.13). These lashed materials can be either placed by helicopter or dragged into place by bulldozer. Root wads with soil attached also provide ideal cover (Cederholm and Scarlett, 1991; Cederholm and others, 1988).

Depending on the plan's habitat objective, branches that stick out of the water may be removed to minimize roosting by predatory birds until a robust fishery is established. Alternatively, protruding branches and logs just breaking the surface may be left to provide sunning areas for turtles and other amphibians.

Structures that can be constructed in or near ponds to enhance habitat for wildlife include:

- trees, logs, and root wads lashed together, submerged, and anchored (Fig. 4.12),

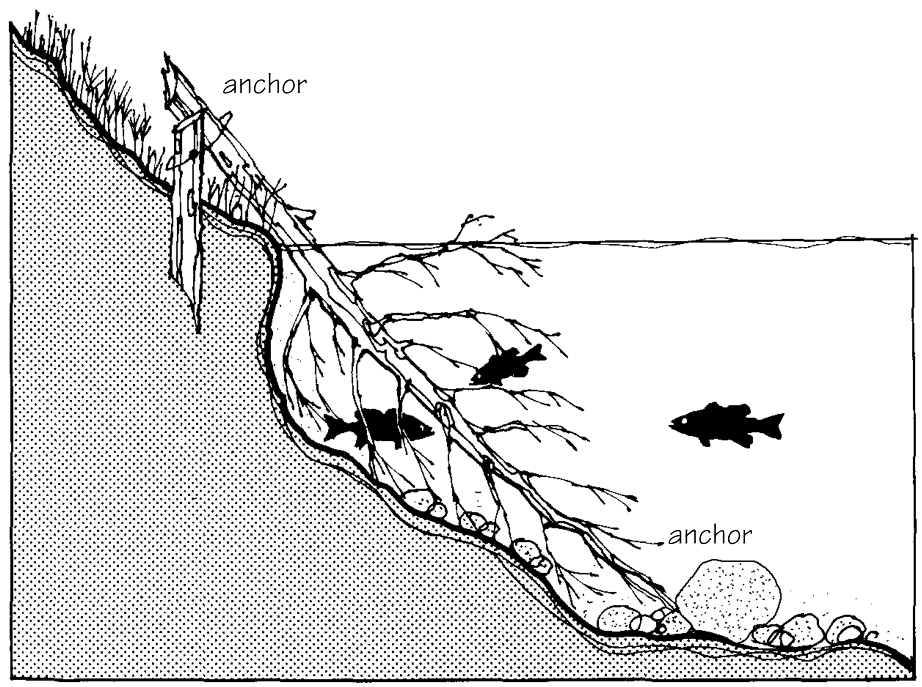


Figure 4.12. A submerged tree crown, anchored top and bottom, provides cover where the bank drops off steeply in some parts of the pit. (Modified from Michalski and others, 1987.)

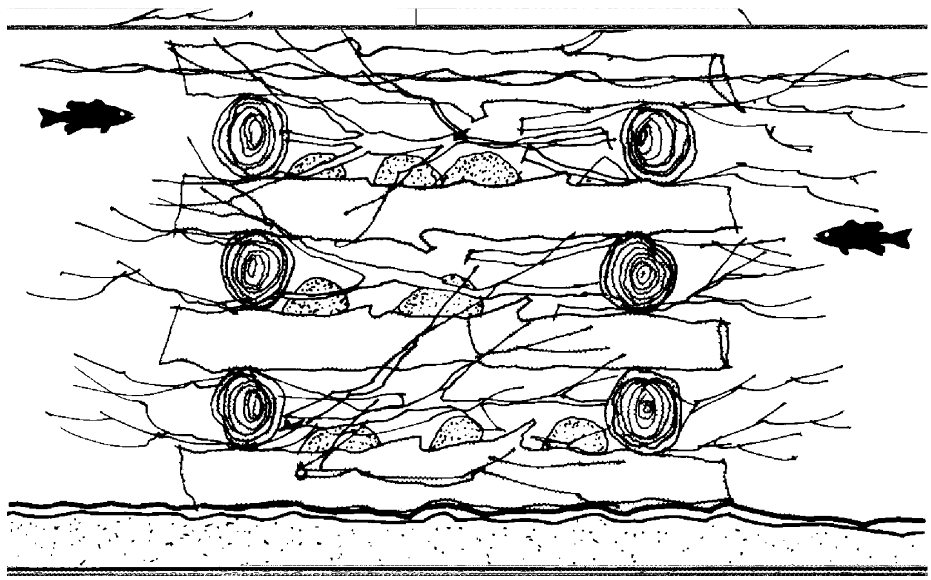


Figure 4.13. A submerged crib structure provides habitat for aquatic insects and cover for fish that feed on them. Rocks are used to anchor the crib in place. (Modified from Michalski and others, 1987.)

- submerged crib structures (Fig. 4.13),
- piles of angular rock (Fig. 4.14),
- nesting boxes (Fig. 4.15), and
- nesting poles and snags for osprey and cavity-dwelling birds (Fig. 4.16).

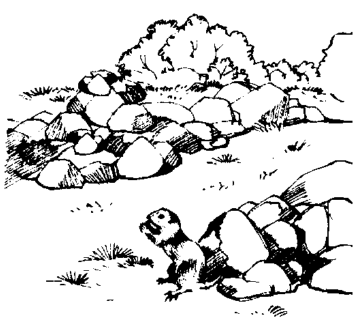


Figure 4.14. Piles of rock provide homes for small mammals. (From Green and others, 1992.)

Groups interested in wildlife or fish habitat enhancement, such as Ducks Unlimited or Trout Unlimited, the Boy Scouts (and similar groups), or schools, can be invited to help in enhancing reclamation of a pond by constructing nesting boxes, planting willows, or other activities. U.S. Fish and Wildlife staff may provide technical assistance, and the agency may be a source of potential grants.

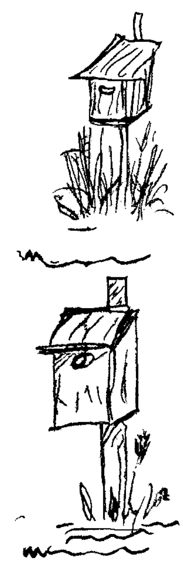


Figure 4.15. Typical nesting boxes.

Off-Channel Ponds for Salmon

At mine sites near rivers, off-channel ponds can be connected to the river by a stable outlet channel that allows access for fish (Fig. 4.10). The channel, excavated after mining, must be shown on the reclamation plan. Ponds like these can provide valuable habitat for salmon (Cederholm and Scarlett, 1991; Cederholm and others, 1988).

The following questions should be addressed in selecting sites for creating off-channel salmon habitat:

Figure 4.16. Snags make good nesting sites for cavity- dwelling birds. (From DeGraaf and Shigo, 1985.)

- Is the section of river or stream near a site used in any way by salmon? Is any part of the whole river or stream used for spawning, travel to spawning areas, or for rearing of fry?
- Will the depth of excavation be compatible with final off-channel habitat (that is, not too deep for spawning, but deep enough to provide cold-water habitat)?
- Is the potential mine site stable? Or is it prone to capture during floods and by lateral migration of the river?
- Is the substrate of the excavation going to be suitable for the habitat desired?
- Is there sufficient water circulation to provide oxygen and keep the water cool?
- Can an outlet channel be connected to the river where it can be easily found by migrating fish?

The Oregon or Washington Department of Fish and Wildlife should be consulted before undertaking any off-channel pond creation project.

Outlet Channels

Outlet channels allow fish to enter and leave the off-channel ponds. They are integral parts of off-channel habitat and should mimic natural river sloughs whenever possible. In some situations, a weir is necessary to control the water level in the outlet channel and ponds.

Outlet channels should join the river system where fish are likely to notice them—for example, near a pool or eddy where fish tend to rest. Riffles or fast water areas are less desirable outlet sites because fish may not find the outlet, and it may be left high and dry during low water. Joining an outlet channel to an existing tributary or slough instead of the river is a good strategy where feasible.

FORMING WETLANDS

Natural wetlands can be defined in terms of three broad environmental indicators: soils, hydrology, and vegetation. The viability of created wetlands can be enhanced by addressing these three elements in the reclamation plan.

Soils

Soils are essential to vegetation, both above and below the water surface. In creating wetlands, pond banks and bottoms should be covered with at least 12 inches of fine materials that have a large clay component to help seal the bottom of the pond. In some places, process fines can be substituted for soils; however, they are less desirable than native soil because they are less fertile. Material routinely removed from roadside ditches may be a good source of wetland soil and vegetation if it is not contaminated with oil and grease. If any wetlands on the project are disturbed, that soil should be used in new wetland creation.

In Washington, a solid waste permit from local jurisdictions may be necessary for disposing of material acquired from ditch cleaning.

Hydrology

A wetland must have water present at least seasonally. A common reclamation challenge at many mine sites is the seasonal fluctuation of the water table. The highly permeable nature of sand and gravel creates a situation where vegetation on pond banks is inundated during the wet season and high and dry during the summer. This results in a zone, similar to that found along reservoirs, in which upland and wetland plants will not readily grow. Here are some ways to reduce water fluctuation and the related adverse effects:

- Seal the bottom of the pond and the downstream banks with clay-rich material. This can happen naturally over time, but it may take many years.
- Reduce bank slopes to 5H:1V or flatter to allow a more gradual transition from the wetland to upland environment.
- Install a head-gate or weir at the outlet of the pond to retain water.
- Anchor jute netting or some other organic mulch fabric over the bank slopes to capture fines and retain soil moisture.

Vegetation

Wetlands are characterized by many plant species that do not grow in upland areas. Most created wetlands in western Washington and Oregon will develop a wetland community on their own if conditions are hospitable and given enough time. Willows, cattails, and other wetland plants will often volunteer on the site in a year or two. To speed the reclamation process, however, suitable species can be obtained from nearby sources or purchased for planting.

Propagating wetland species can be difficult and can, in some places, produce a plant community composed of only a few species, that is, far less diverse than natural populations on undisturbed sites. The best way to establish a diverse community is to transplant soils and plants from an existing wetland, particularly one that is being eliminated by mining. Care must be taken when planting nursery stock to replicate as nearly as possible the plant community surrounding the site being reclaimed.

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5 Reclamation Techniques for Quarries

HIGHWALL AND BENCH RECLAMATION

Many quarry operations create benches and highwalls composed of solid rock. Shaping the tall rock faces and engineered benches created during production blasting can be difficult. Vertical cliffs may be incorporated in the reclamation landscape if natural cliffs exist in the area of the mine. The extent and types of cliffs present should be shown on maps and cross sections submitted in the permit application.

Primary reclamation concerns for these areas are stability and aesthetics. Some post-production blasting may be necessary to break up linear features. The effects of blasting the highwall should be carefully considered when preparing both the operating and reclamation plans. If blasting is contemplated, seek the help of a qualified professional before proceeding. A poorly designed blasting plan can result in unsafe conditions that are difficult and expensive to fix.

Public access and safety should also be addressed as part of the reclamation plan wherever steep cliffs are to be left. After mining, a bench or berm may be needed at the base or top of steep highwalls to catch falling rock. Placing a berm at the top of the quarry or a 10-foot-high by 15-foot-wide bench near the top will improve safety by discouraging access and reducing the likelihood of injury due to falling.

Where adequate moisture is present (west of the Cascade Range), wide benches may be revegetated. Benches to be revegetated should slope toward the highwall to trap moisture and soil. (See Fig. 2.4.) They should also slope gently to the side to promote drainage. Enough soil should be placed on the bench to support the proposed vegetation.

West of the Cascades, trees planted on benches may eventually break up the line of the face, although it may take years before benches are screened from view, even in smaller quarries. Revegetation may not be a viable reclamation technique in dry areas, larger quarries, and open pits unless combined with other methods discussed in this chapter. In arid areas east of the Cascades, bench revegetation will probably not obscure linear features.

Several methods of reclaiming quarry walls are effective in achieving stable slopes and preparing the site for the proposed subsequent land use. Excavated quarry slopes are generally more stable than fill slopes. However, once a material is blasted, it is no longer considered consolidated and must be reclaimed to a shallower angle, depending on the nature of the rock.

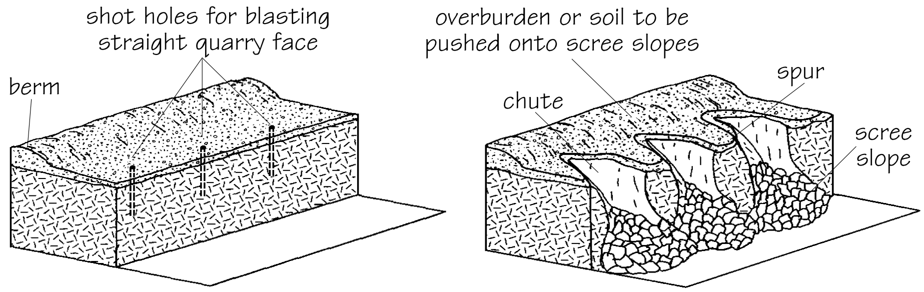


Figure 5.1. Blasting at the holes shown in the left sketch can create scree slopes (right), which may then be stabilized by plantings.

RECLAMATION BLASTING

Reclamation blasting is a fairly new technique. The amount of fracture desired often differs from that for production blasting. Chutes, spurs, scree slopes, and rough cliff faces can be intentionally created by strategically placed blast holes (Fig. 5.1) (Norman, 1992; Coppin and Bradshaw, 1982). Because few people have the field experience necessary for this type of blasting, the use of a contractor familiar with this technique is recommended.

Highwalls

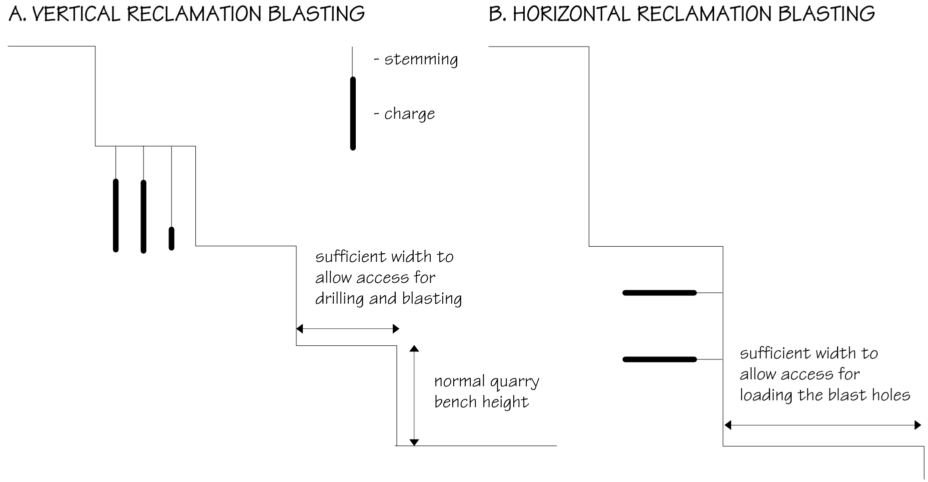
Selective blasting produces a natural appearance and stabilizes a site. Selective blasting can be used to modify benches, break up linear features, and blend highwalls with their natural surroundings. Proper blasting of highwalls leaves rough surfaces that can provide nesting and perching habitat for birds (Fig. 5.2). However, the rough surface should be free of loose rock.

Reclamation blasting that reduces the entire highwall to a scree or overburden slope is essentially a cut-and-fill method. This technique can be used only where there is sufficient material remaining in a setback behind the quarry face to create the desired slope. Blasting for this purpose will not be possible if the operator has mined to the permit boundaries.

The highwall profiles of Figure 5.3 show two conceptual blasting patterns for reclamation. In 5.3A, vertical holes are drilled across the bench floor. The outermost row of holes is only lightly charged to minimize flyrock and keep the blasted material on the slope. Most of the rock fracturing is done by the explosives in the

Figure 5.2. Proper blasting of highwalls leaves rough surfaces that can provide nesting and perching habitat for birds. (From Green and others, 1992.)

Figure 5.3. Conceptual blasting patterns for obliterating quarry benches.



rows farther back from the face. The blasthole design of Figure 5.3B uses horizontal blast holes. PVC pipe can be inserted into the drilled holes to keep them open and serve as a water drain. The final pit configuration must allow for access to the drilled holes for loading with explosives.

The final choice of blast pattern, delays, stemming depth, etc. depends upon the rock type, structural geology, blasting agent, and other highly variable conditions that cannot be addressed in this manual. Although this method can be less expensive than backfilling (Thorne, 1991; Petrunyak, 1986), the operator has only one chance to get it done right. Doing proper research and consulting appropriate experts before starting reclamation blasting cannot be stressed enough.

After the blasting is completed, topsoil and overburden stored above the final slope can be pushed onto the blasted rubble to promote revegetation. For quarries in which there are multiple benches, the final slope will approximate the overall slope of the benches. Proper setback must be accounted for from the lowermost bench to the uppermost one.

Benches

If selective blasting of benches is impractical or dangerous, other reclamation methods may be necessary, such as leaving wide benches that can be revegetated or pushing rock over the side of the pit to hide the benches (Fig. 5.4).

MINIMIZING OFFSITE IMPACTS

Minimizing offsite impacts from blasting is in the best interest of both neighboring landowners *and* mine operators. It can reduce litigation and negative publicity for a project. All blasting should be done by professionally trained and certified experts. Blasting techniques have improved dramatically since the days of black powder fuses and dynamite. Vibrations, noise, and fly rock can be greatly reduced when proper techniques are employed.

Causes of Damage

Vibrations from the blast may damage nearby structures and residences. A blast creates a wave that travels through rock and uncon-

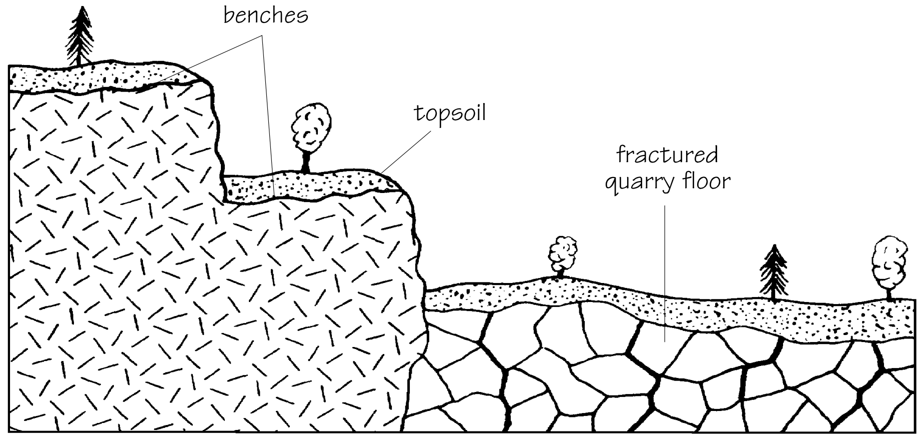


Figure 5.4. Topsoil placed on benches and on a fractured quarry floor will prepare the site for revegetation.

solidated materials. When the wave arrives at nearby structures, it can cause them to vibrate. Sound waves from the blast, transmitted through the air, are usually more detectable by humans, but it is the back and forth movement of the ground wave that causes the damage, not the accompanying sound. The amplitude and intensity of the ground wave are determined by the number of pounds of explosive detonated at one time. Most problems can be avoided when the amount of explosive is minimized and the blast is properly timed.

Vibration Effects Under Various Conditions

Unconsolidated material will vibrate more strongly in response to the ground wave than will competent rock. All other factors being equal, the potential for vibration damage is greater if a structure is built on fill, sand, dirt, or other unconsolidated material than if it is built on compacted material or competent rock. The more competent the material, the less movement will occur.

The way the structure is built can also have an effect on the kind and amount of damage. A structure with a concrete slab floor usually develops more cracks than one with a perimeter foundation built on solid rock.

Pre-Blast Survey

In order to establish pre-blast conditions at nearby residences, a pre-blast survey should be performed by an outside specialist rather than by a member of the organization doing the blasting. Typically, after a blast has taken place, owners of nearby structures will find cracks, settlement, and displacement, all of which were pre-existing, but never noticed. All structures within any possible damage range must be thoroughly surveyed before any blasting is done.

The importance of a pre-blast survey of all surrounding structures cannot be overstated. The lack of a proper survey by a qualified specialist is an open invitation to lawsuits. Without a survey, the damage could be real or imagined, but an expensive lawsuit will be required to establish liability.

Use and Placement of Vibration-Measuring Equipment

The blast contractor should monitor the blasting with vibration-measuring equipment, but the equipment should be placed and the results read by a qualified independent third party. Monitoring equipment that provides an immediate printout is generally better than equipment requiring post-blast data manipulation and interpretation because the results are available immediately and cannot be changed once recorded.

Blasting Plans and Logs

The mine operator should require a blasting plan and blasting logs. Blasting plans are prepared before the blast. Blasting logs are made on the site as each hole is primed, loaded, stemmed, wired, and connected to the circuit. Blasting logs must accurately describe the work on each hole and must be kept for 2 years after the work is completed in case they need to be referred to later.

BACKFILLING

Quarries located in populated areas should consider total or partial backfilling when it is economically feasible (Fig. 5.5). Advantages of backfilling include reducing slopes, increasing post-mining property values, and reducing safety hazards. (See Chapter 4.) In urban areas, many quarry sites are backfilled. If buildings or other structural improvements are to be placed on top of the old excavation, the backfill material must be structurally sound and stable. Dumping fill material over the highwall can also help disguise the linear benches. If overburden or waste rock is strategically placed, backfilling may be done with a short push or haul.

Fill Materials

In some quarries, operators will decide to rebuild slopes after all rock is removed by:

- concurrent backfilling using overburden mined elsewhere on the site,
- bringing in fill material from construction projects offsite, and
- retaining enough overburden or mine waste for resloping after completion of mining.

Overburden should be stored where it can be readily and economically moved into position during reclamation. Mining plans should take the backfill process into account. Operators need to be sure there is enough onsite material or identify a likely source.

If fill is accepted from construction sites, a monitoring plan should be established by the operator to prevent disposing of hazard-

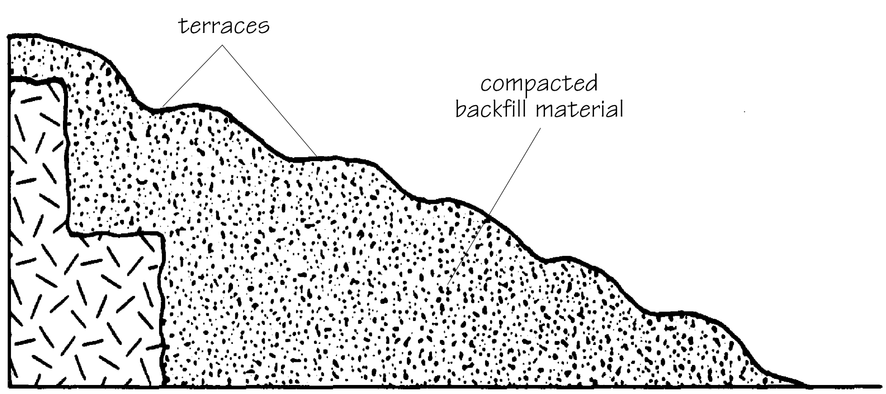


Figure 5.5. Quarry slopes that are backfilled should be compacted so that the final slope is stable; a 3H:1V angle (with terraces, if it is long) generally results in a stable slope. Topsoil should be spread over the compacted slope to make revegetation possible.

ous or unapproved material on the site. Local permits from health departments may be necessary before importing fill.

Fill Slopes

Stability and erosion control are primary concerns for slopes created by backfilling. Backfilled slopes may be prone to erosion and gullying if they are smooth, planar, and long. (See Creating Slopes, p. 4..) As slope length and steepness increase, runoff velocity and soil erosion also increase, and infiltration decreases. Careful location of drainages and water-control features enhances slope stability and revegetation potential (Banks and others, 1981; Washington Department of Ecology, 1992). (See Chapter 2.)

Temporary protection of bare slopes from rain or snow-melt runoff may be necessary if backfilling occurs over a long period and if establishing permanent vegetation must be delayed. Temporary protection can include covering the slope with plastic sheeting or mulches or matting and seeding with grasses. (See Chapter 2.)

A final slope angle of 2H:1V to 3H:1V is recommended. The gentler the slope, the easier soil application will be and the more quickly vegetation will establish. Backfilled slopes may require compaction to ensure stability.

DRAINING PIT FLOORS

If wetland creation is not part of the reclamation plan, pit floors can present special drainage problems. There are two basic ways to improve drainage in quarry floors: blasting and ripping.

Blasting

Impermeable pit floors of solid rock can be blasted to fracture the rock so that water can drain slowly from the site and roots can penetrate (Fig. 5.4). The least expensive way to blast the pit floor is to

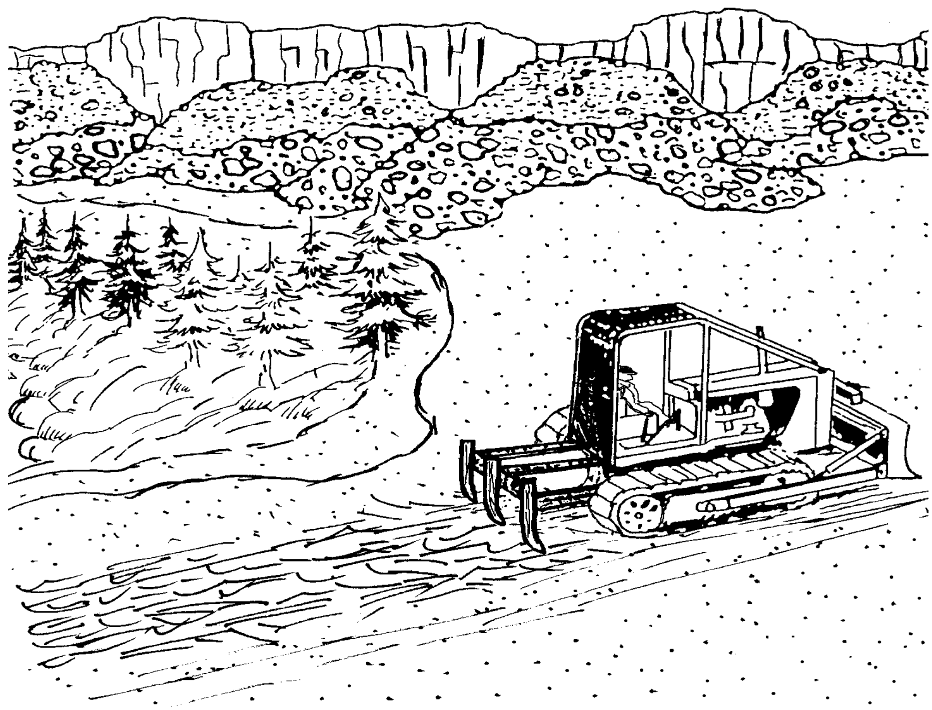


Figure 5.6. Ripping or decompaction of pit floors is typically accomplished with rippers mounted on heavy equipment.

drill an extra 10 feet on the last production shot and leave some of the fractured material in place.

Ripping

Ripping or decompaction is typically accomplished with rippers mounted on heavy equipment (Fig. 5.6). Rippers consist of a vertical shank or shanks that can shatter compacted or hard areas to depths of 7 feet. Before ripping or tilling compacted mine wastes or soils, at least one backhoe pit should be dug on the site to determine the thickness of the compacted zone, thus the depth of tilling. As a rule of thumb, ripper spacing should be less than the depth of ripping.

If soil is replaced using equipment with rubber tires, discing, plowing, or shallow ripping may be necessary to loosen the soil to create seedbeds and suitable substrate for ground cover or trees.

In locations where topsoil is minimal or absent and ripping is not possible, selective drilling and blasting may improve revegetation success. A basalt quarry in Australia achieved 85 percent survival of tree seedlings after four years by blasting 7-foot-deep holes into the pit floor (Rock Products, 1995). This technique fractures the rock, provides a moisture trap where roots are able to penetrate, and, if ammonium nitrate explosives are used, may provide some residual nitrate to stimulate plant growth.

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6 Landslides and Slope Failures

Many upland mining sites are situated in terrain that has potentially unstable slopes or is already unstable. Construction of spoil dumps, stockpiles, and mine cuts can destabilize areas that were stable prior to mining. If mines are located in potentially unstable areas, such areas should be identified before mining, and the mine plan should be developed so as to minimize risk to the environment. Common mining-related causes of landsliding are:

- removing the toe (support) of the slope,
- saturation of unstable slopes due to poor water management (such as constructing a pond on a slope),
- placing waste rock over vegetation on steep slopes causing failure as the vegetation rots,
- adding weight to an unstable slope, and
- placing weight (generally overburden) on an unstable area.

Landslides do not recognize property lines. Conditions on adjacent property may be 'causing' the slide on the mine site, and slides occurring on the mine site may damage adjacent properties. If stability is a concern or major faulting is encountered, a geotechnical consultant should be involved in mine planning.

TYPES OF SLOPE FAILURES

The movement of soil and rock under the influence of gravity is called mass movement or mass wasting. Rockfalls, slides, earthflows, slumps, soil creep, raveling, and (more commonly) combinations of flow types are all forms of mass movement that can occur at mine sites.

Rockfalls

Rockfalls travel most of the distance through the air (Fig. 6.1). Movement is extremely rapid and includes free fall, tumbling, and rolling of fragments of bedrock or soil. Rockfalls may occur in a mine as pressure is released on the free face.

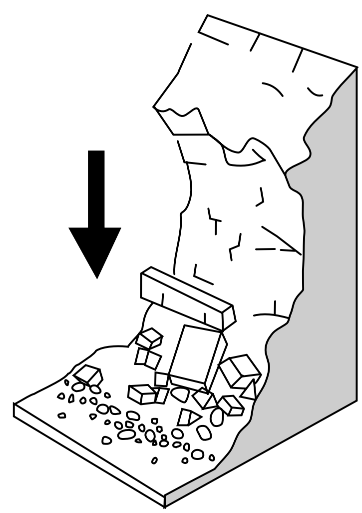

Figure 6.1. Rockfall on a steep or overhanging face. (Redrawn from Chatwin and others, 1991.)

Slides

Slides move along one or more zones of weakness. Movement along the failure surface may be rotational, as in a slump, or translational along a more or less planar surface (Fig. 6.2).

Live tree roots contribute to holding the soil together and help tie the upper soil horizon to the subsoil. Runoff and surface erosion, when combined with a decrease in tree-root tensile strength caused by stripping vegetation and soil, have contributed to many land-

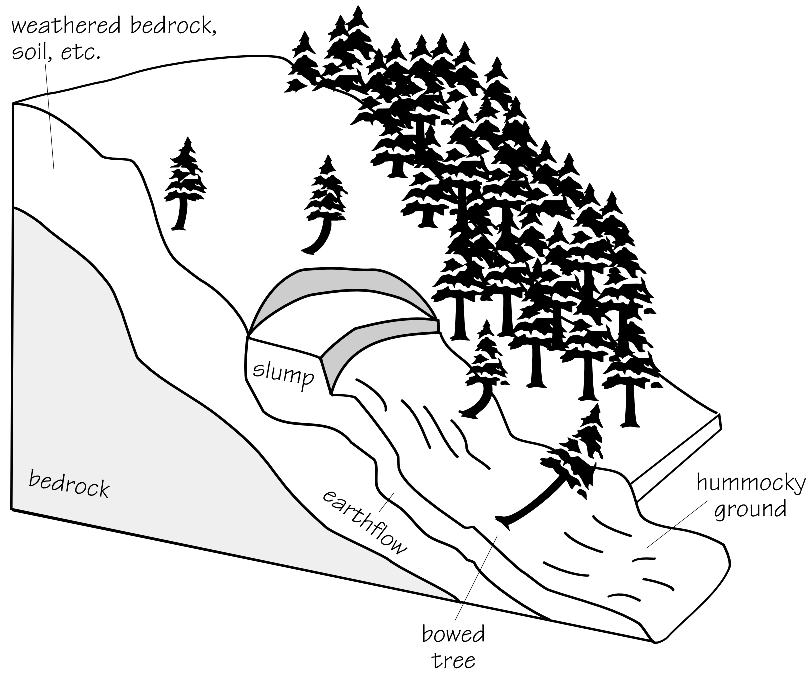

Figure 6.2. A complex slide called a slump-earthflow. (Modified from Chatwin and others, 1991.)

slides by removing the slope support. Scars from debris slides (shallow soil slips) may commonly be seen on steep slopes that have been stripped of vegetation. Removing the toes from steep slopes such as on talus, sand and gravel, or clay deposits can result in a landslide.

Earthflows Earthflows, composed of soil and rock, move slowly downslope as a viscous fluid. The amount and rate of movement vary according to the particle size and water content of the earthflow. Clay-rich zones are especially vulnerable to plastic flow when saturated. If enough water is present, the material can 'liquefy', causing an earthflow.

Slumps In a slump, the movement is rotational, producing a bowl-shaped failure surface. Slumps and slump-earthflows typically leave behind a steep scarp that is itself vulnerable to further slumping. Slumps also commonly occur in areas underlain by till and/or glacial lake deposits, both of which are vulnerable to failure when they are saturated.

Soil Creep Soil creep is the very slow (inches per year) downslope movement of surface materials (Fig. 6.3).

Raveling Raveling is downslope movement of particles and commonly occurs on sand and gravel slopes that are too steep. Reclaimed slopes of 2H:1V to 3H:1V usually do not ravel.

ANATOMY OF A LANDSLIDE

Most landslides are combinations of several kinds of slope failure. The method of failure may be different in different parts of the slope. A landslide, in this case a slump-earthflow (Fig. 6.4), has the following parts (Varnes, 1978):

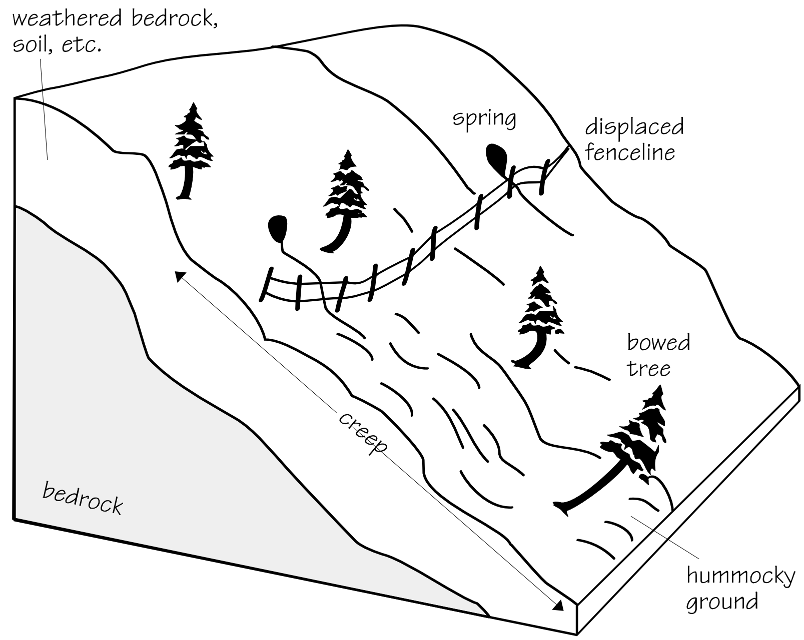

Figure 6.3. Conditions that lead to and indications of soil creep. (Modified from Chatwin and others, 1991.)

Main scarp – A steep surface separating the undisturbed ground from the slide mass, caused by the movement of slide material away from undisturbed ground. The projection of the scarp surface under the displaced material becomes the surface of the rupture.

Minor scarp – A steep surface in the displaced material produced by differential movements within the sliding mass.

Head – The upper part(s) of the slide material along the contact between the displaced material and the main scarp.

Toe – The lower margin of displaced material most distant from the main scarp.

Crown – The material that is practically undisplaced and adjacent to the highest parts of the main scarp.

IDENTIFYING UNSTABLE SLOPE CONDITIONS

Regardless of the cause, instability can often be identified in the field through careful observation. Tension cracks, hummocky topography, springs and seeps, bowed trees, abrupt scarps, and toe bulges are all readily observable indicators.

Tension Cracks

Tension cracks, also known as transverse cracks, are openings that can extend deep below the ground surface (Fig. 6.4). Tension cracks near the crest of an embankment or hillside can indicate mass movement. However, cracks may occur anywhere on the slide. They are perpendicular to the direction of movement and are typically continuous in a pattern across the width of the landslide. Tension cracks can fill with water, which lubricates the slide mass and may cause additional movement. Correction of slope failures must include preventing surface water from reaching tension cracks.

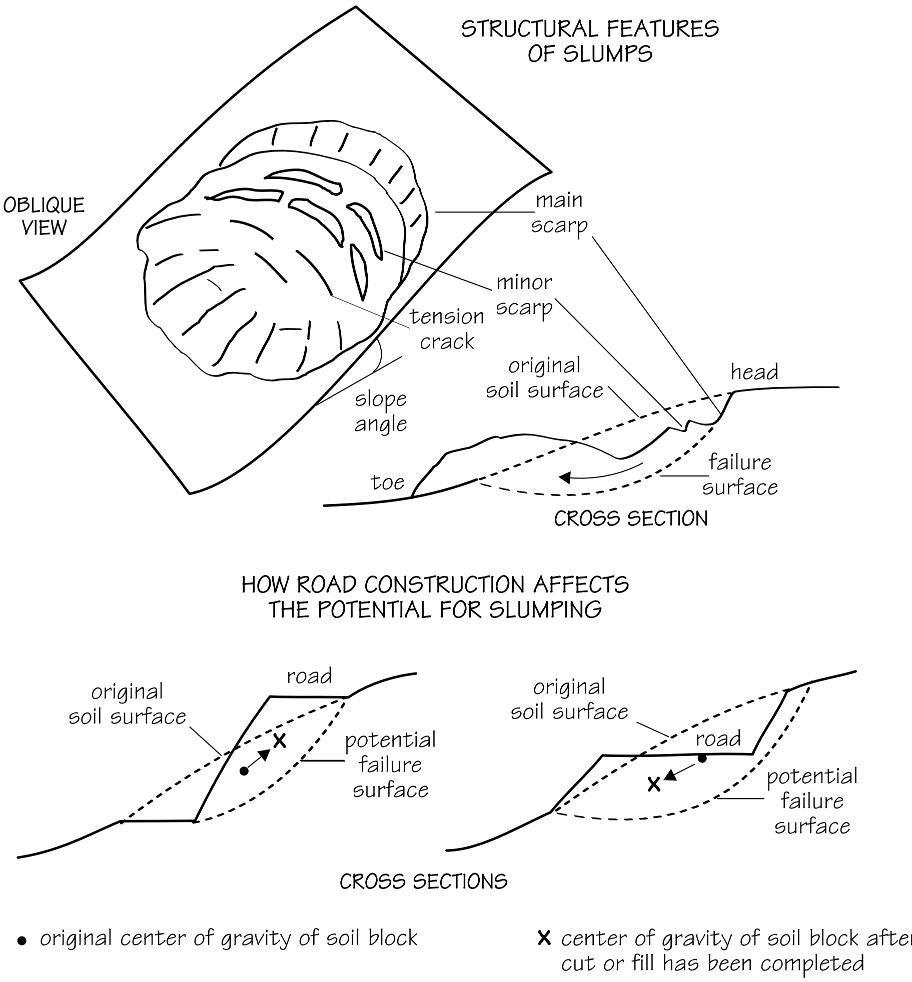

Figure 6.4. Structural features of slumps and the effect of cutting and filling on the stability of short slopes. (Redrawn from Burroughs and others, 1976.)

Hummocky Ground

Hummocky ground can indicate past or active slide movement. A slide mass has an irregular, undulating surface (Figs. 6.2 and 6.3).

Displaced and Distorted Trees

Vegetation, particularly trees, records the downslope movement of soil. Trees may be uprooted and may lean in a variety of directions (jackstrawed trees) as their roots are broken or moved in a rapid slide movement (Fig. 6.5). Bowed tree trunks may indicate soil creep; trees attempt to remain upright as the soil moves slowly downslope (Figs. 6.2 and 6.3).

Springs and Seeps

Ground water that collects at the contact between permeable layers that overlie relatively impermeable layers or rock strata dipping with the slope can cause instability. Carefully investigate springs, seeps, and areas of lush vegetation. Alder, horsetail, devils club, cow parsnip, and skunk cabbage typically grow in wet sites.

Scarps

Fresh scarps are a clear sign of recent slope failure (Fig. 6.4). Older scarps may be covered by vegetation and hard to identify. The presence of several scarps can indicate several active failure surfaces or movement downslope along a larger failure surface.

Toe Bulge The toe of a slide commonly bulges out onto the more stable ground surface below the slide (Fig. 6.4). A toe bulge often gives the appearance of a mud wave displacing trees and vegetation in its path. The bulged toe should be noted in the site inventory along with the other slide features to define the size of the failed area. Removing the toe may reactivate the slide mass.

SURFACE DRAINAGE CONTROL IN UNSTABLE AREAS

The quantity and distribution of water in a slope, whether it is a slide mass, overburden, or soil stockpile, greatly influences its stability. Water saturation builds up pore pressure, which causes an increase in downhill-directed forces (Fig. 6.5). This increases the weight (increases driving force) and particle lubrication (decreases resisting forces). Slope failure can occur when more water is present in the soil than the pore spaces can accommodate.

If motion on a slide at the mine site responds directly to rainfall, surface drainage improvements may decrease slide activity. Control of surface drainage, by itself, is seldom sufficient to stop landslides, because rainfall from outside the site can eventually show up as ground water in the slide. Surface drainage improvements are typically combined with other abatement techniques. (See Chapter 2.)

When soils, subsoils, and geologic material are excavated, drainage paths through the pore spaces are disrupted. Therefore, drainage control may be needed for constructed permanent and temporary storage or disposal piles and reclaimed slopes that are created by backfilling.

Listed below are techniques for improving slope drainage. (See Chapter 2 for specifics.) These techniques may not stop landsliding altogether, but they may prevent a slide from becoming worse:

☛ To improve slope stability, lower the water table by providing more drainage. Adequate drainage prevents water saturation and the build up of pore pressure.

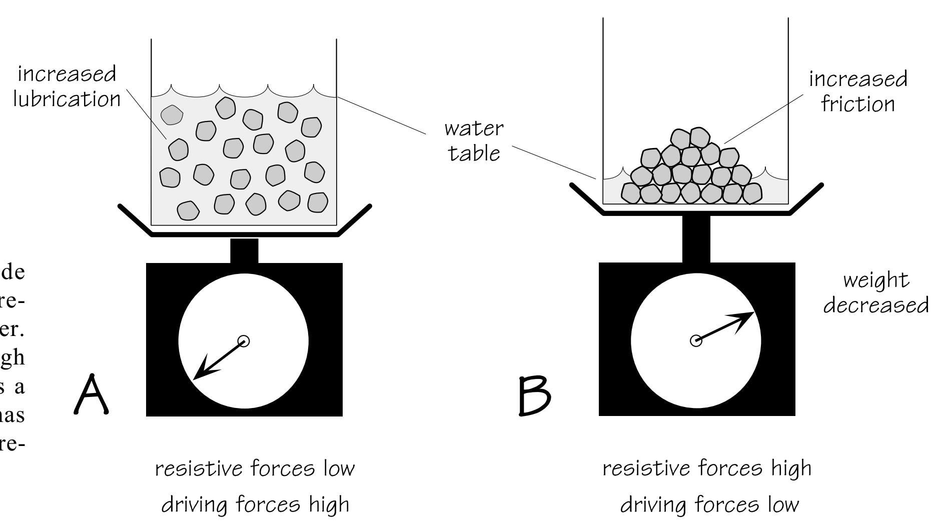

Figure 6.5. Forces acting on slide masses and large stockpiles. **A** represents a slide mass saturated with water. It has both low resisting force and high driving forces (weight). **B** represents a stabilized slope after the water table has been lowered or the water has been removed using drainage methods.

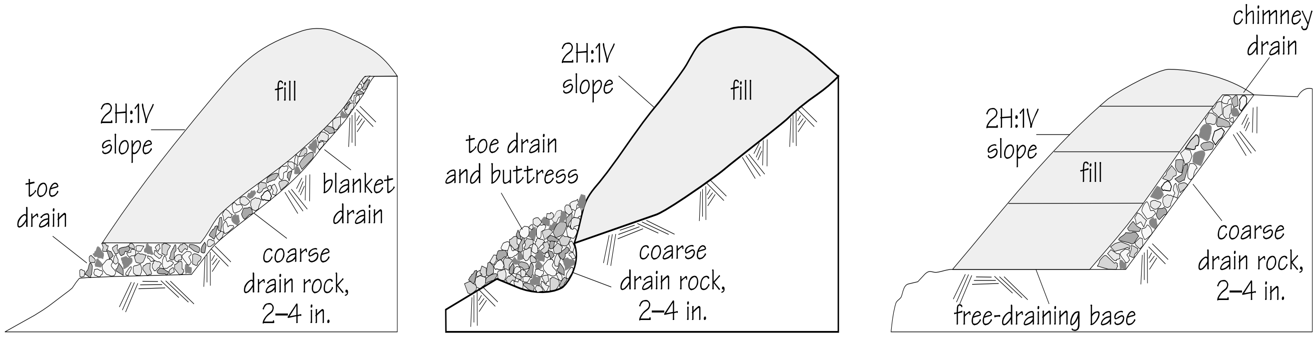

Figure 6.6. Details of toe, blanket, and chimney drain construction shown in cross section. (See also p. 2..)

- Berms and ditches should be built above and along the unstable slope to intercept and divert overland flow. They should be lined or sealed to prevent infiltration.
- Slopes adjacent to the slide mass should be graded to direct overland flow away from the slide area.
- The area above a slide should be crowned or sloped so that surface water is directed away from the slide and graded so water does not pond.
- Where drainage must cross an unstable slope, using a pipe should be considered.
- Avoid concentrating water on spoil dumps or natural slopes, thereby reducing their stability. Concentrated surface flows near slides should be handled in ditches lined with impermeable fabric, if necessary. (See Fig. 2.16.)
- If a slide area is to be regraded, the regrading should not produce a depression in the slope that could pond or concentrate water.
- If a slide is triggered, benches or cross-slope ditches should be used. They should be sloped and lined to move water away from the slide area.
- As part of grading operations, any exposed tension cracks should be sealed and compacted to prevent infiltration, then seeded to prevent erosion.

SLOPE STABILIZATION

Toe, blanket, chimney, and other types of permanent drains (Fig. 6.6) can help prevent saturation of a constructed slope. The minimum thickness of an underdrain or rock blanket should be 3 feet, because fines will eventually migrate into this zone. The drains should be thick enough to keep running freely for a long time. In some cases, a geotextile liner should be used to insure that the integrity of the drain is not compromised by soil movement.

Slope length and height may require construction of cross-slope drains to intercept runoff without creating gullies and erosion. Grading to break up long slopes and creating berms, furrows, and terraces will compartmentalize the runoff. The more landscape diversity that is incorporated into the final grading, the less a site will need cross-slope drains to ensure stability.

SLOPE FAILURES ABOVE THE MINE

Overburden failures above mine cuts can be a problem if proper slope angles are not maintained above the rock face. If the contact between the overburden and the rock dips toward the highwall or open face and the overburden slope is near vertical or steep (1V:1H), a failure is likely. To prevent this from occurring, operators should make sure the overburden cut has a gentle slope and is well drained.

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7 Revegetation

INTRODUCTION

Mines west of the Cascades in Washington and Oregon are fairly easy to reclaim because they typically have deeper soil horizons due to abundant precipitation. Mined areas east of the Cascades are more difficult to reclaim because soils are thinner, the region is drier, and temperatures are more extreme. Therefore, successful revegetation in the eastern part of the state is more dependent on proper plant selection, appropriate timing of planting, adequate fertilization, presence of organic matter in the soil, and irrigation.

West of the Cascades, even though revegetation can be accomplished without separately salvaging and replacing the soil because of the abundant moisture, species diversity will be limited until a soil horizon rebuilds, and this may take decades. Additionally, plant vigor may quickly decline after the first planting if ample amounts of organic matter are not provided or supplemental chemical fertilizers are not added to initiate the cycle of plant growth, decomposition, and nutrient recycling. Amounts of fertilizer should be based on site-specific needs determined by soil tests. (See Amending or Manufacturing Soil, p. 4.6.)

Natural plant communities develop through a succession from pioneer species to climax species (Fig. 7.1). Pioneer species are aggressive and tend to grow rapidly to fill disturbed areas, whereas climax plant communities develop over longer periods and tend to be slower growing. Each phase in the plant succession prepares the ground for the next. Nitrogen-fixing legumes, shrubs, and trees may play a crucial role in soil reconstruction.

It is tempting, particularly with trees, to plant only climax species (for example, Douglas fir) even if the ground is not fully prepared. However, natural communities develop slowly in a succes-

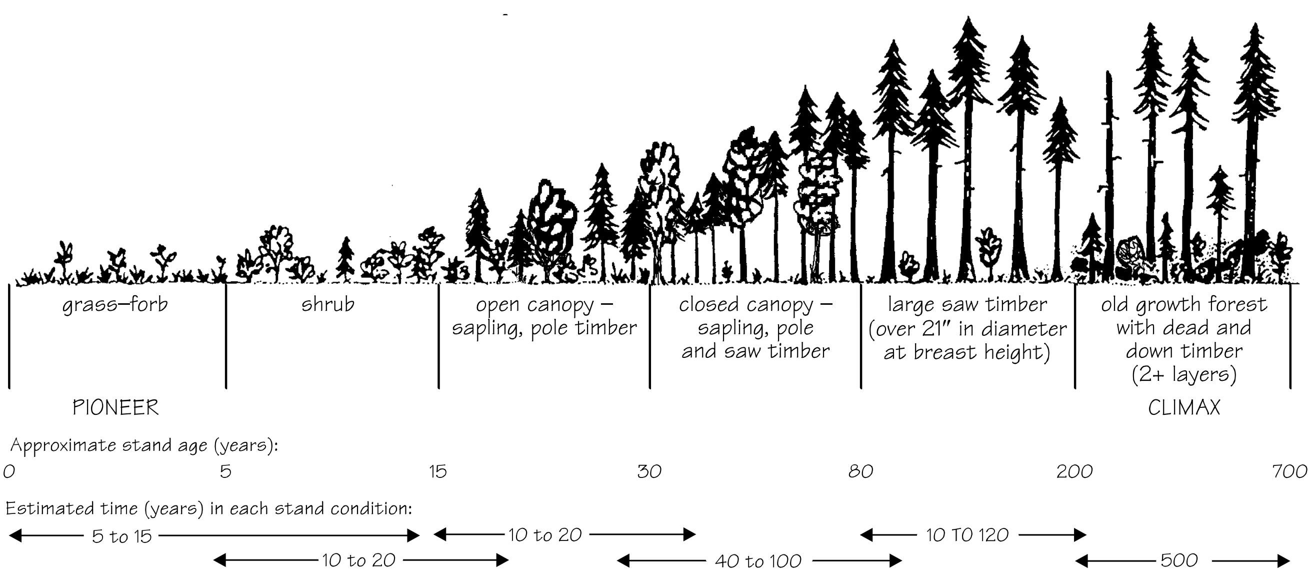

Figure 7.1. Sequence from pioneer to climax vegetation for a Douglas fir forest after clear cutting. The same recovery process occurs naturally in mined areas. (From Brown, 1985.)

sion. Mimicking this progression during reclamation is impractical, but planning a phased succession for both ground cover and trees will establish a good climax mix (Norman and Lingley, 1992).

Grasses may be appropriate as either quick pioneer soil builders under developing woodland or as climax species for rangeland. Pioneer trees will act as fast-growing nurse trees for slowly maturing forest trees that find it difficult to establish in disturbed ground or in areas with no canopy.

Revegetation is important because it:

- reduces erosion,
- reduces storm-water runoff,
- provides habitat and forage for animals,
- reduces visual and noise impacts,
- reduces reclamation liability, and
- increases the value of property by returning it to agriculture, forestry, or other beneficial use.

Note: While vegetation significantly reduces erosion, it cannot prevent slippage of a soil that is not stable due to improper placement techniques. For example, soil placed on steep slopes requires additional stabilization techniques to ensure revegetation success. (See Chapter 6.)

SPECIAL PROBLEMS AT MINE SITES

Plants need fertile soil, sunlight or protection from the sun, and water to thrive. Mining often removes fertile soil. (Salvaging and replacing soil is discussed in The Soil Resource, p. 3..) Even in the best of conditions, plant growth cannot be guaranteed immediately after mining. Mine sites generally offer harsh conditions that make it difficult to establish vegetation. Some common problems affecting revegetation are:

- high surface temperature (especially on south-facing slopes),
- steep slopes,
- poor water retention,
- lack of adequate soil,
- erosion before seedlings establish,
- only limited periods during the year suitable for seeding,
- lack of water
- poor conditions for germination,
- slopes inaccessible to equipment, and
- grazing impacts.

By being aware of these potential problems, an operator can improve the quality of reclamation and save money by being successful on the first attempt. Revegetation early in the reclamation process is critical because it may take several seasons to establish widespread healthy vegetation. For example, by planning ahead and choosing appropriate techniques, an operator can place young trees in strategic locations to provide a significant visual screen within a few years.

SUCCESSFUL REVEGETATION STRATEGIES

Trial-and-error revegetation that relies on natural precipitation and hardier natural pioneer species (such as alder) is generally less expensive, uses less labor, and is more effective than waiting until mining is complete to plant the entire site with commercial plants. Segmental mining results in fairly small areas on which to begin this process. Test plots can be used to determine which species will be successful. Areas in which plants fail to establish can be reseeded with more appropriate vegetation (Norman and Lingley, 1992).

Steps to successful revegetation of mined land can be summarized as follows:

- ☛ *Plan before you start.* Know in advance what has to be done, but allow for modification if necessary.
- ☛ *Strip and store the topsoil, subsoil, and overburden separately.* Minimize handling and storage.
- ☛ *Strip a small area at a time.* Strip only the area that can be revegetated within a reasonable time to minimize erosion.
- ☛ *Move soil materials under dry conditions (June–September).* Wet soils are easily damaged.
- ☛ *Carefully calculate volumes of soils necessary for reclamation to ensure that sufficient amounts are retained.*
- ☛ *Reclaim the mine in segments.* Segmental reclamation allows for 'live topsoil' replacement, which often enhances revegetation.
- ☛ *Shape slopes for subsequent use.* Slopes between 40H:1V and 20H:1V are desirable for agriculture purposes. For forestry, the slopes can be steeper.
- ☛ *Replace overburden (if any), subsoil, and topsoil in the correct sequence.*
- ☛ *Eliminate compacted soil.* Where compaction has occurred, rip the mine floor as deeply as possible before reapplication of stored overburden, subsoil, and topsoil.
- ☛ *Develop a post-reclamation management program.* Choose plants that increase soil fertility and improve soil structure, such as deep-rooted nitrogen-fixing legumes, for the first plantings. Monitor progress and determine why plants did not thrive.
- ☛ *Get good advice from the experts.* Take advantage of the expertise available in various government agencies and though local farmers.
- ☛ *Be patient.* Successful revegetation may be a slow process taking several seasons or years.

CLASSES OF VEGETATION

Four basic classes of vegetation—grasses, forbs, shrubs, and trees—are important for reclamation. Forbs, which include legumes such as alfalfa, clover, and lupines, are any herbaceous plant that is not grass or grasslike. Forbs and shrubs have many similarities but differ in that shrubs have a woody stem. They will be considered together in this discussion. Many sites naturally support a mixture of two, three, or all four types of vegetation.

Grasses

Grasses are either perennial or annual. Annual grasses start from seed every year, whereas perennial grasses die back but start from the same root mass each year. Annual grasses green up and establish quickly, but put most of their energy into seed production. Perennial grasses put significant energy into root development and foliage; individual plants persist for many years.

Grasses typically are shallow rooted (6 inches to 2 feet) but, because of their ability to provide complete ground cover, are effective for erosion control. Grasses provide significant nutrition to both livestock and wildlife and provide cover for small animals and birds. Newly established grasses, freshly fertilized, are a favorite food for grazing animals. Therefore, such areas should be fenced for optimum revegetation success.

Forbs and Shrubs

Forbs and shrubs include everything from small wildflowers (forbs) to sagebrush plants (shrubs) that may reach 6 feet in height. They are nutritious and provide significant cover. Many plants of this class have a single taproot with a shallow fibrous root system around it. Although mature forbs and shrubs can establish significant root wads, they typically provide only minimal erosion protection for several years.

Trees

Trees are generally the slowest of the three classes to establish themselves and mature. They typically have a deep, extensive root system. Evergreens or conifers (except larch) keep their leaves or needles all year long. Deciduous trees lose their leaves every fall and, compared to conifers, grow faster and add leaf litter to the ground.

SELECTING PLANTS FOR A SITE

Wherever possible, native species should be used in revegetation. Native plants often out-compete introduced (exotic) species over time and are the most useful to wildlife, although some introduced species can out-compete some native species, especially in arid environments. The vegetation surrounding a mine site can be used as a guide when selecting native species. Re-establishing native species can be greatly accelerated by using native seed mixes and locally transplanted species.

If sufficient preplanning is done, soil and native vegetation can be transferred from areas being stripped for new mining to areas in the final stage of reclamation. This approach is less expensive and often more successful than long-term soil storage. Soil hauled directly from a new mining area to a reclamation area carries with it viable seeds of native vegetation that can rapidly establish on the reclaimed area. This typically reduces the need for added seed and plant material.

Commercial sources typically sell native and non-native bareroot and container plant stock, as well as native grass seed mixtures. Bareroot stock should be planted during the winter and is typically less expensive than plants sold in containers. Generally, plants in containers have a better survival rate than bareroot plants. A plant-selection guide is given in Tables 7.1 through 7.4.

The best source of native shrubs and trees is in or near the site to be revegetated. Avoid transplanting native species from an elevation significantly higher or lower than the area in which they will be planted.

Weeds (imported or local) can render reclamation ineffective. Local extension agents can provide lists of noxious weeds and suggest methods for their control.

Information on plant availability and nurseries carrying suitable plants can be obtained from Hortus Northwest, PO Box 955, Canby, OR 97013, *Phone:* 503-570-0859, *Fax:* 503-399-6173.

Grasses and Legumes

Grasses and legumes are very effective at stabilizing disturbed areas because of their extensive root systems. They also increase water infiltration, contribute organic matter to the soil, and, in the case of legumes, fix atmospheric nitrogen into the soil.

In determining what mix of grasses and legumes is best for a given site, the climate, soil conditions, sun exposure, and objective of the seeding must be considered. The Oregon Department of Geology and Mineral Industries (DOGAMI), The Washington Department of Natural Resources (DNR), and the local Natural Resource Conservation Service (NRCS) offices can provide valuable information about seed mixes that are suited to various site conditions. The Washington or Oregon *Interagency Guide for Conservation and Forage Plantings* is also a useful resource for determining seed mixes. Tables 7.1 through 7.4 contain descriptions of some of the most common grasses, legumes, and woody plants.

Some grasses, such as annual rye, grow quickly, while others, such as many of the perennial bunch grasses or sod-formers, grow rather slowly. Cereal grains, the same as those cultivated for food, can be very effective in establishing a rapid vegetative cover that will still allow native species to establish. Cereal grains help protect against soil erosion because they possess 50 percent more below-ground biomass (roots) than grasses.

The success of legume plantings can be greatly improved by treating the seeds with legume inoculant, available from many seed suppliers.

Forbs and Shrubs

Many forbs establish easily from seed and can be just as important as grasses and trees for reclamation. Some shrubs do well from seed, many do not. Bareroot plants, which can often be purchased inexpensively and easily from nurseries, are an effective way to establish shrubs. Young plants in containers are generally easiest to establish but are the most expensive to purchase.

Trees

A variety of species suitable for revegetation projects are available in containers at nurseries. Tublings (plants grown in narrow, deep containers) may be useful on rocky areas and steep slopes. Bareroot transplants are successful for many species and are more economical to purchase than containerized plants. Nurseries can provide both tublings and bareroot stock.

Native Plants for Arid Regions

For the high desert areas of Washington and Oregon, a selection of the following species are recommended when native plants are specified in the reclamation plan:

- basin big sagebrush (*Artemisia tridentata tridentata*)
- Wyoming big sagebrush (*Artemisia tridentata wyomingensis*)
- mountain sagebrush (*Artemisia tridentata vayseyana*)
- fourwing saltbush (*Atriplex canescens*)
- antelope bitterbrush (*Pershia tridentata*)
- Lewis flax (*Linium lewisii*)
- white yarrow (*Achillea millefolium*)
- annual sunflower (*Helianthus annuus*)

In the higher areas of eastern Washington and Oregon where sites will be reforested, the following seed mix of non-pervasive exotics has been used to control erosion and noxious weed invasion in the short term. These plants die out as long-term native plants take over from nearby natural areas when the sites are relatively small (less than 15 acres or long, narrow sites):

Sheep fescue	4 pounds/acre
Kentucky bluegrass	4 pounds/acre
Dutch white clover	2 pounds/acre

(the clover should be inoculated)

SOWING SEEDS

Grasses and cover crops such as legumes are relatively easy to establish from seed. In most places, grass and legume seeds should be planted no deeper than 1/4 inch. For the best chance at revegetation success, topsoil should be spread between September 15 and October 15. Seeding with grasses and legumes should be done within 3 days after final shaping (R. Shinbo, personal commun., 1995). However, if proper conditions of soil moisture and temperature are present, revegetation can also be successful at other times of the year. Proper conditions for reclamation and revegetation exist between March 1 and November 1 for sites west of the Cascades in some years. During the winter, bare slopes should be protected with mulch or other erosion-control techniques until the next seeding period.

Summer plantings should be avoided unless irrigation is planned. Fall plantings may be preferable in areas with long growing seasons, winter rains, or summer drought; they allow plants to establish themselves over the winter. Optimal planting dates will vary slightly from year to year and with weather conditions. The local county extension service can provide information on planting dates.

Seed Drills

Seed drills are used extensively in agricultural applications where soil has been tilled and is free of rocks. Range drills are used in irregular terrain or on rocky soils. In arid areas with coarse-textured soils, improved success with drilling may be obtained by placing the seeds 1 inch deep.

Range drills may be available for use from some federal agencies, such as the NRCS and the Bureau of Land Management. Agricultural seed drills are commonly not suited for reclamation seeding because of the rocky soil. Neither type of drill is suitable for the rough and steep terrain found on many mine sites.

Broadcast Seeding

Seeds can be broadcast using many different methods. Spreading handfuls of seed by hand is quick and easy but produces incomplete coverage in many cases. The use of hand-operated mechanical spreaders is a far more effective way to spread adequate amounts of seed evenly. Hand-operated mechanical spreaders come in many different sizes and styles, but most are relatively cheap. In many cases, they can be rented from a local shop. Regardless of the method of broadcast, the seeds must be covered with mulch and/or soil to germinate successfully. Broadcast seeding in arid environments should be followed by dragging a meadow or flex harrow (a bar or chains in rocky areas) over the seeded area to insure adequate seed/soil contact.

Hydroseeding

Hydroseeding can effectively convey, in one application, seed, fertilizer, and mulch onto steep slopes and other areas inaccessible to other seeding equipment. The mulch blanket retains moisture; a tackifier or binder added to the hydromulch slurry can prevent it from eroding away. Revegetation success can often be increased by using a two-step hydromulching process in which seed, mulch, and fertilizer are applied with the first application. Then the entire area is remulched with another application of mulch only. The two-step technique is especially useful in arid areas where the seed germinating in the mulch may dry out before roots become established enough to provide water.

Seedbed Preparation

Seedbed preparation establishes conditions conducive to seed germination and seedling growth. Seedbed preparation on mining sites is especially important because the heavy equipment commonly compacts the soil, which inhibits seed germination. In order for a seed to germinate and thrive, there must be contact between seed and soil, adequate moisture, and moderate soil temperature. The soil must be loose enough to allow root penetration once the seed has

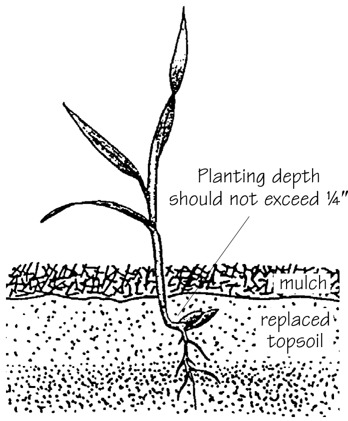

Figure 7.2. Cross section of seed germination.

germinated (Fig. 7.2). A soil or mulch covering of 1/4 inch moderates temperature and prevents seed loss to birds. Mulching also conserves the much-needed moisture for continued seedling development.

Depressions, small pits, and irregularities in the seedbed can greatly enhance the ability of seeds to germinate and thrive. A sheepsfoot roller, land imprinter, or bulldozer can be used to create micro-depressions. Bulldozer tracks parallel to the contours can enhance seed germination and reduce runoff (see Fig. 4.5).

Mulching

The primary purposes of mulch are to retain moisture, prevent erosion, and moderate soil temperature fluctuations. Among materials that can be used as mulch are:

- hay or straw,
- processed mint clippings,
- wood chips,
- grass clippings, and
- wood fiber.

Mulches can be applied with blowers, hydromulching equipment, or manually. Mulch may be anchored to prevent water or wind erosion by crimping it, adding tackifiers or binders, or by covering it with natural or synthetic netting.

Hay or straw mulch can be anchored using a modified agricultural disc implement that crimps the hay into the soil.

Logs and other woody debris, placed perpendicular to the slope in seeded areas, will help stabilize mulch and can provide valuable shade and microhabitat for the emerging seedlings.

Cattle as a Reclamation Tool

Using cattle to control erosion and enhance revegetation of tailings dams and waste rock dumps is now a relatively widespread activity in Arizona and Nevada. Judging by the success in these states, cattle can be a valuable reclamation alternative for some hard-to-reclaim sites in Washington and Oregon, especially those in arid areas with steep slopes.

Carefully monitored and controlled cattle grazing can dramatically reduce wind and water erosion on slopes and accomplish many of the tasks required for successful revegetation. The hooves of the cattle compact and blend soil materials and, at the same time, create abundant depressions that catch moisture and prevent erosion. Cattle urine and excrement provide fertilizer that is generally well distributed and mixed into the slope by grazing activity, and the microbes in the manure are an important ingredient in building a healthy soil.

In order for cattle to be used for reclamation, they must be restricted to relatively small areas using easily moveable fences, such as an electric tape fence. Cattle must be moved from one area to another regularly to prevent overgrazing. Salt blocks, water, and feed must also be periodically moved to insure that the entire slope being treated is covered. A pilot project was started in Lake County, Oregon, in 1997. The results are not yet available. Contact DOGAMI-MLR for the latest information on this technique.

TRANSPLANTING

Transplanting is the technique used for relocating containerized stock, bareroot stock, or plants from elsewhere on site and planting them in another.

Planting Times

Containerized plants have an advantage over bareroot stock in that they can be successfully transplanted almost any time of year. However, transplanting should not be done during the summer unless irrigation is provided.

Trees and shrubs should be planted while they are dormant, generally from November 1 through March 1. Bareroot stock and transplants are usually planted in the spring because the plants have to be dormant before they can be dug. Bareroot plants may not be shipped from the nursery until late fall or mid-winter. Spring planting may be appropriate for bareroot stock if the site is subject to frost heaving in the late fall or winter.

Spring plantings should be done as soon as site conditions allow. Typically plants should be placed in the ground just before or just after shrubs at the site break dormancy. That can be determined by looking at buds. Buds begin to swell when the plants are ‘breaking’ their dormant condition.

Plants should be adequately acclimatized. This is particularly critical when the environment of the growing nursery or location is different from the planting site. Plants can be acclimatized by moving them to the site before the planting date. Bareroot materials should be kept under refrigeration or the roots should be buried in a shallow trench and kept moist until planting.

Planting Techniques

If moisture conservation is important, planting should be done immediately after digging the planting holes to reduce drying of the backfill.

When transplanting, keep the majority of the root mass intact (Fig 7.3). Even if care is taken in transplanting, some roots will break. Often the damage is to the fine roots that are essential for providing nutrients and moisture. Pruning the above-ground stem(s) reduces evapotranspiration and increases the likelihood of survival by reducing the plant’s demand for nutrients and moisture.

It may be helpful to construct berms 2 to 6 inches high around the planting holes to concentrate rainfall and runoff. On sloping ground, leaving the berm open on the uphill side of a planting can be beneficial (Fig. 7.4).

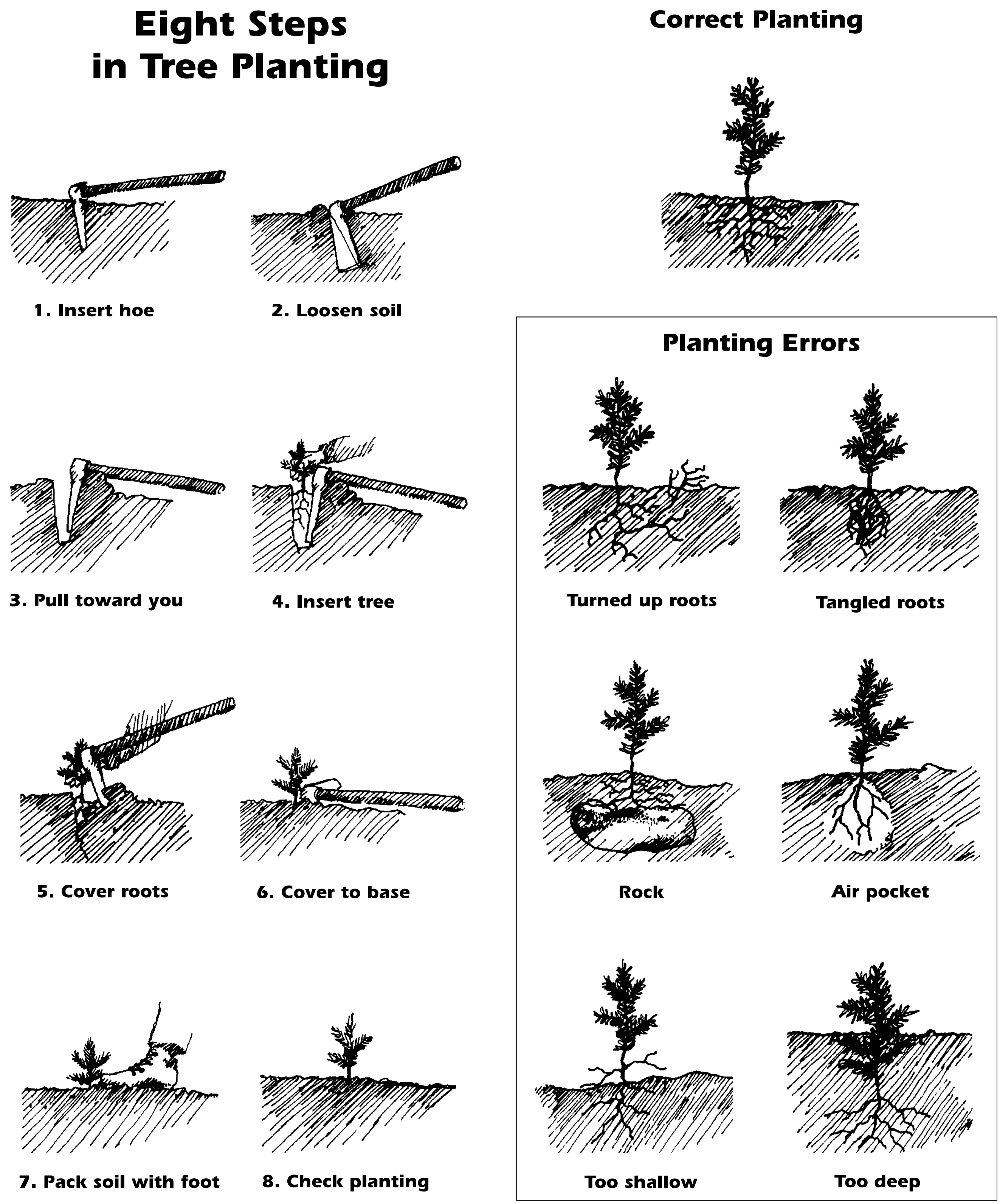

Figure 7.3. Steps in transplanting bareroot or container plants.

Mulch will help retain moisture. However, it must be anchored to prevent erosion by water or wind. Mulch is of little use on sites that flood since the mulch washes away.

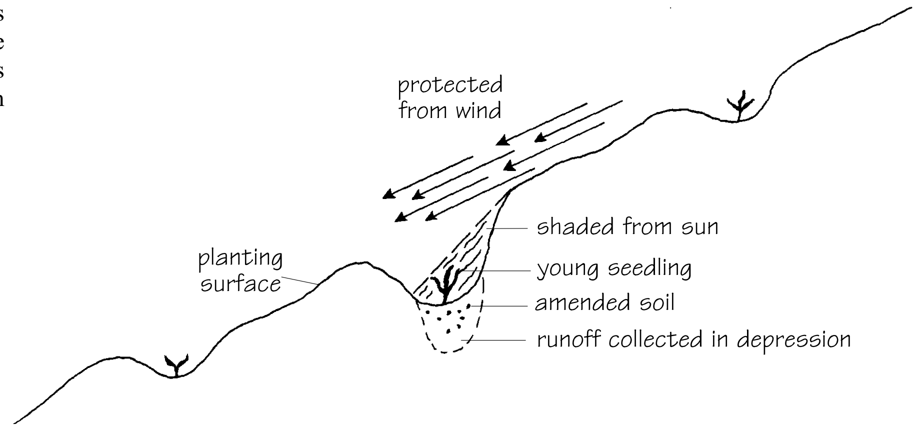

Figure 7.4. Transplanted seedlings on a slope. Small berms on the downslope side of the planting holes help retain runoff. (Redrawn from Banks, 1981.)

Tools Required

Choice of planting tools will depend upon the revegetation plan, the size of plant materials, and planting conditions. Shovels, picks, pry bars, posthole diggers, hand or power augers, front-end loaders, or backhoes may be needed to develop the planting site. For gathering plant materials from the site, chainsaws, lopping and pruning shears, buck saws, mechanical tree spades, and backhoes or front-end loaders are useful. Straw or hay for mulch for moisture retention, fencing and wire for plant protection, and cages and stakes for support may also be required. Fencing or cages are highly recommended if deer, beavers, or other plant 'predators' are in the area. They appear to seek out recently established trees and shrubs.

PROPAGATING FROM CUTTINGS

The easiest and most economical method for propagation of some species of woody plants is the use of cuttings. Willows and cottonwoods are the two most common plants propagated from cuttings (Fig. 7.5). The best time to collect cuttings is while the plants are dormant, typically between November 1 and March 1. Cuttings taken near or at the planting site or from a similar elevation zone will have a good chance of surviving on the site.

Determining Cutting Length

Cuttings should be at least 3 feet long, but the length of the cutting depends on the planting depth required. At least two-thirds of the cutting length should be placed in the ground. The planting depth depends on the mid-summer water table and the potential for erosion in the planting area. Where erosion potential is high or the water table is deep, planting depth and cutting length should be increased. The above-ground stem should have at least three buds exposed. The minimum stem diameter for cuttings should be 3/4 inch.

Collecting Cuttings

Healthy-looking plants should be used. Willows are particularly susceptible to willow bore—avoid plants with burls, lumps, or scabs surrounded by smooth bark. Several years of drought conditions or other plant stresses will diminish the reserves in the plant and may affect the survival rate. Transplant stock should be selected from wetter areas. Avoid suckers (the current year's growth) because they may not contain adequate stored energy reserves. Trim off all side branches and remove the apical (top) bud; the apical bud draws too much energy and may affect survival.

Storing Cuttings

If cuttings need to be stored longer than several days, they should be kept in a cooler at 24° to 32°F. A mixture of 50 percent latex paint and 50 percent water can be used to mark and seal the top of the cuttings and reduce moisture loss. All cuttings should be soaked prior to planting for at least 24 hours to initiate root growth. At a minimum, the bottom third of the cutting should be submerged. The entire cutting may be soaked once the paint has dried. Rooting hormone added to the water may improve the survival rate. A diagonal cut should be made on the bottom for ease of planting and a straight cut on the top.

Planting Cuttings

Cuttings can be placed either in the spring or fall, preferably when the plants are dormant. If cuttings are taken in the fall before dormancy, the leaves should be stripped. (A general rule of thumb is

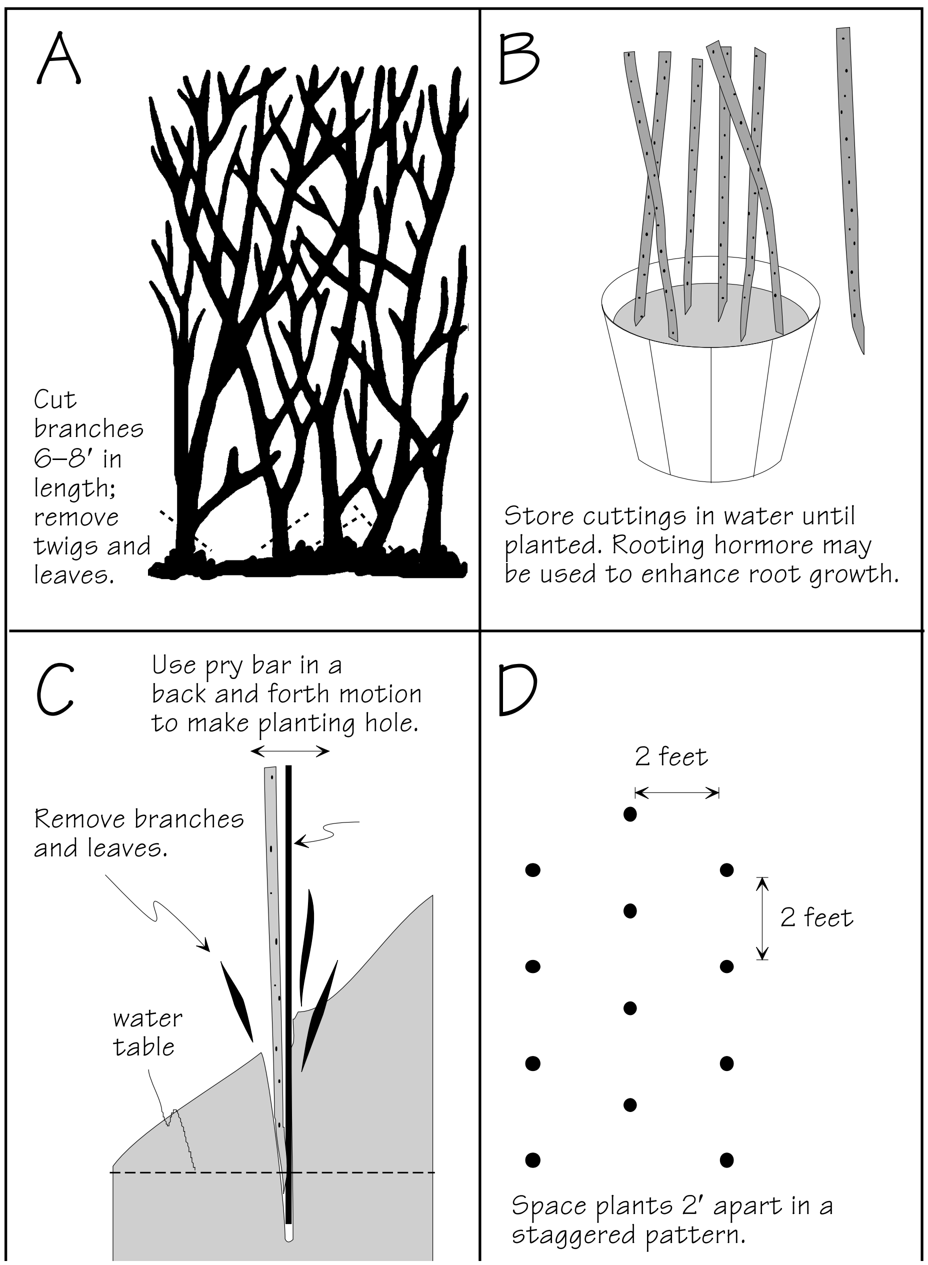

Figure 7.5. Steps in propagation by cuttings.

that cuttings should be taken in the late fall or early winter and that rooted plants should be taken in the spring.)

Cuttings must be planted with the buds facing up. Be sure to keep track of which end of the cutting is the top—a cutting planted upside down is not likely to survive.

For successful plantings, the following guidelines are suggested:

- Select cutting stock from a nearby plant source.
- Cut when plant is dormant (usually late fall or winter).
- Use cutting of proper diameter and length.
- Properly store and maintain the cuttings before planting.
- Add root hormones to storage water.
- Use good planting techniques.

Optimum spacing of the cuttings will depend on the site and the purpose of the planting. To achieve good density, plant cuttings 2 feet apart in rows offset by 1 foot (Fig. 7.5D). Cuttings can be planted wiggling a pry bar or a piece of rebar back and forth to develop the planting hole (Fig. 7.5C). Critical factors are preventing damage to the bark and ensuring good contact between the cutting and the soil. Pack the soil around the cutting; air pockets around the cuttings will kill the roots. Driving the cutting directly into the ground with a hammer is not recommended because it causes the cutting to split.

BIOTECHNICAL STABILIZATION

The term 'biotechnical stabilization' refers to the use of plants to revegetate and stabilize slopes and stream banks instead of engineered structures, such as gabions, retaining walls, or riprap. The planting techniques discussed above may also be used as components of a system where biotechnical methods are employed. Rock or other structures can be incorporated in the design where planting alone is not enough to stabilize an eroding bank. For a comprehensive review of this subject, the *Soil Conservation Service Engineering Field Book*, Chapter 18, Soil Bioengineering for Upland Slope Protection and Erosion Reduction, is recommended.

Brush Layering

In brush layering, live woody plant materials, such as willow, cottonwood, and dogwood, are placed in layers on a slope to reinforce the soil and prevent shallow slope failures (Figs. 7.6 and 7.7). The layers also act as a living fence to trap sediment and debris. Brush layering has been successfully used to repair partial fill-slope failures, increase streambank stability, and enhance riparian vegetation. However, brush layering will not correct a deep unstable slope condition where mechanical methods of control are needed. If brush layering is used to stabilize an eroded bank, place a blanket of large rock from just above the ordinary high-water mark to just below the ordinary low-water mark.

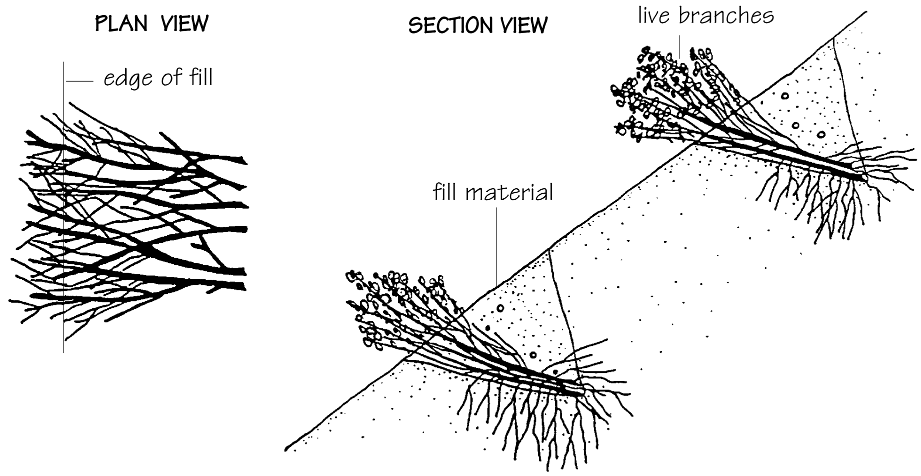

Figure 7.6. Details of brush layering in trenches. Start this process from the top down. (Modified from Bellevue Storm and Surface Water Utility, 1989.)

Starting at the top of a slope, brush layering is installed by trenching along the contour and then placing the live plant materials prior to backfilling the trench (Fig. 7.6). It may be appropriate to mix species of brush in the trench. Generally the brush-layer branches should be 6 to 8 feet in length, but they can be longer. The number of contour trenches opened at any one time should be limited to prevent destabilization of the slope.

Trenches should be excavated so that three-fourths of the live plant material can be buried in the trench, leaving one-fourth of the plant above the ground surface. Once the materials are placed, the excavated soil is then pulled down into the trench to reshape the slope.

Brush layering can also be used on fill slopes. In this situation, live plant materials can be placed on successive lifts of backfill. If this method is used, grading equipment can be used for hauling and

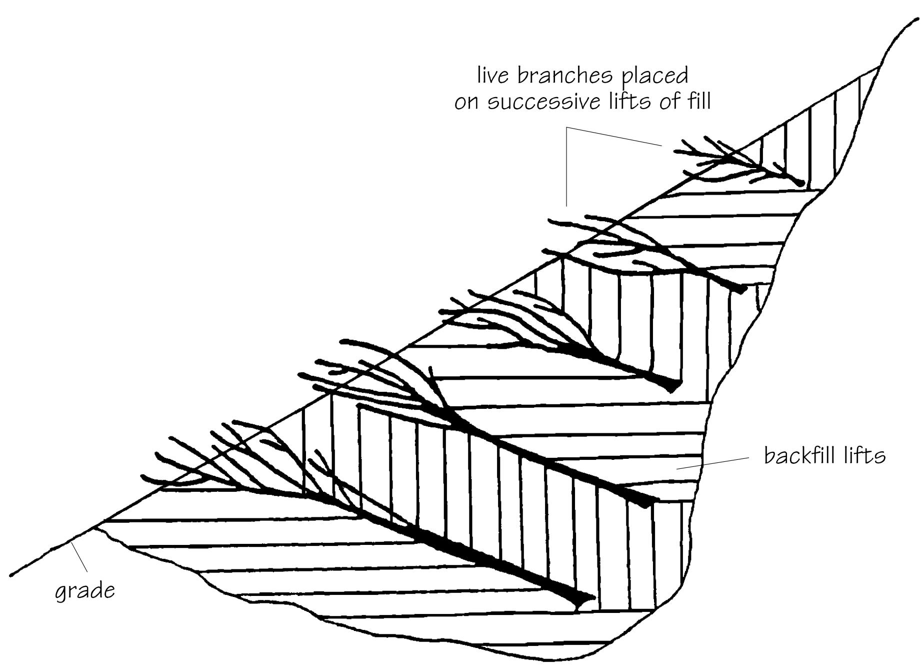

Figure 7.7. Brush layering of live plant materials on successive lifts of fill. Grading equipment can move and place the vegetation. (Modified from Bellevue Storm and Surface Water Utility, 1989.)

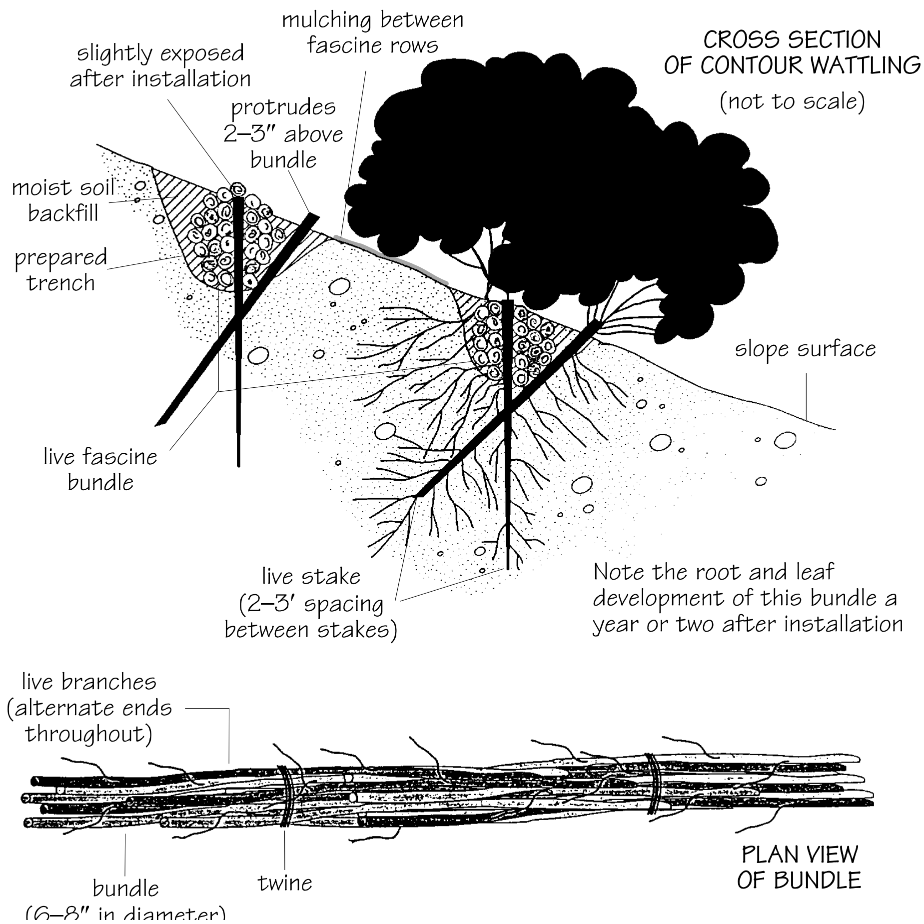

Figure 7.8. Wattle construction and placement. Wattles are bundles of live plant material, 6–8 inches thick, tied with twine. The butt ends and the tops are alternated and tied together, repeating this process until the necessary length is created. The bundles are then placed in shallow trenches along the contour and partially covered with soil so that about 10–20 percent of the bundle is exposed. (Modified from U.S. Soil Conservation Service, 1992.)

placing the vegetation (Fig. 7.7). Brush layering is less labor intensive than wattling.

Contour Wattling

The first recognized use of contour wattling was in the 1930s. Wattling controls erosion by stabilizing surface soils, reducing erosive runoff velocities, increasing infiltration, and trapping sediments. It can be very effective in stabilizing gullies. The bundles are placed across the gully.

Wattles are cigar-shaped bundles of live plant material, sometimes called 'live fascines'. The bundles are 8 to 10 inches thick and are compressed by tying with twine. The butt ends and the tops of plants are alternated and tied together, repeating this process until the necessary length is created (Fig. 7.8).

Wattles are placed in shallow trenches along the contour. On riparian sites, they can be placed diagonally to the water flow or wave action. After placement, the wattles are partially covered with soil so that approximately 10 to 20 percent of the bundle is exposed. Either live or dead stakes will secure the wattles on the slope.

Woody plants that work well with this technique are willow, red-osier dogwood, and snowberry. Over time, the planted wattles may be crowded out by more dominant species.

RIPARIAN AND WETLAND AREAS

Riparian areas are those on or near the banks of streams or other bodies of water. They are the zone of direct interaction between terrestrial and aquatic environments. Wetlands are areas that are permanently wet or intermittently water covered. (See Forming Wetlands, p. 4..) Vegetation in both areas requires water in the rooting zone on a permanent or seasonal basis. Classification of an area as riparian or wetland is based on factors such as vegetation type, surface and subsurface hydrology, topography, and ecosystem function.

Ecological Functions

Restoring or creating vegetated riparian areas or wetlands can:

- increase plant species diversity for habitat reconstruction,
- enhance erosion control and stream bank and/or slope stabilization,
- help to moderate water temperatures,
- improve water quality by filtering sediments and other contaminants,
- provide food for wildlife,
- provide leaf litter for worms and insects,
- slow floodwater, and
- disperse floodwater.

Alluvial mining operations or those with intermittent or perennial streams in the disturbed area should plan to revegetate wetlands and riparian areas. The woody and herbaceous vegetation that grows in the riparian zone is important in maintaining the health of streams, lakes, and wetlands.

Plant Selection

Knowing which riparian species are best suited for a particular planting technique is essential for successful revegetation. Species such as willow, cottonwood, and red-osier dogwood can be propagated by cuttings, while others, such as red alder, salmonberry, snowberry, thimbleberry, Douglas' spiraea, vine maple, and Pacific ninebark, can only be propagated by transplanting the root mass with the above-ground stem. Those species that have a fibrous, spreading root system can generally be propagated by root division.

Planting riparian areas with native trees (cottonwoods, poplar, alders, willows, fir, pines, maples), grasses, legumes (lupine), and forbs can provide nesting cover and accelerate the restoration of productive habitat. Planting willow, poplar, and cottonwood cuttings is an effective method of building a root matrix and slowing erosion. (See Chapter 2.) In ponds, aquatic grasses, sedges, rushes, and tubers should be planted to provide cover and food for insects and fish. Generally, non-native species should be avoided unless rapid stabilization is required. Aggressive native species such as common cattail and Douglas' spiraea should be used cautiously, because they may crowd out other plants.

To insure good growth and survival, species should be planted in environments they are adapted to. Some species are more tolerant of constant inundation than others. For example, big leaf maple and Oregon ash should be planted high enough up the bank so that the roots are above the water table. Table 7.1 is a plant selection guide listing plant growth characteristics, requirements, and planting conditions necessary for propagation. (For more information on wetlands vegetation selection, see Vegetation, p. 4.15.)

AGRICULTURAL AND FORESTRY SUBSEQUENT USES

Often the post-mining use calls for commercial agriculture or reforestation. For those situations, the operator may want to plan reclamation with a professional forester or an extension service agent. The Oregon Departments of Forestry or Agriculture and the Washington Department of Natural Resources are other good sources of information.

Topsoil

For a mine site to be reclaimed for agriculture or forestry, topsoil must be replaced. Operators who have not saved topsoil and subsoil for reclamation will generally not be able to use the site for agriculture or forestry because topsoil replacement would be too costly.

Other conditions to avoid are excessively stony soils resulting from mixing soils and subsoils with the sand and gravel deposit, compacted pit floors, and inadequate treatment of applied topsoil and subsoil to ameliorate compaction problems. In addition, slopes steeper than 3H:1V will not be as productive for agriculture or forestry.

Segmental reclamation and live topsoiling increase the chances of productive agricultural and forestry land after mining. Detailed knowledge of the sand and gravel deposit is also necessary. The composition of the pit floor is an important component in developing a reclamation plan. For example, if the pit floor is on impermeable or compressible silty and clayey material, severe soil compaction will occur, soil drainage will be impeded, and a perched water table causing excessive wetness will result.

Factors to Consider

From an agricultural standpoint, at least 8 inches of topsoil with suitable subsoils or a minimum of 3 feet of combined topsoil and subsoil overlying a zone saturated with water is needed for most plants during the growing season. Therefore mineral extraction should not occur below the water table. Knowledge of the hydrologic conditions of the site is necessary for reclamation to be successful.

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Table 7.1. A partial listing of appropriate native plants suitable for erosion control and slope stabilization. Water requirements: dry—once established, tolerates dry soil conditions during the growing season; moist—requires moist soil throughout the growing season; wet—tolerates saturated soil year-round; usage—relative water uptake by plant. Light requirements: full sun—requires sun throughout the day; sun/shade—requires shade for about half the day; full shade—-requires shade throughout the day. Rooting characteristics: fibrous—lacks a central root, root mass composed of fibrous lateral roots; tap—with a stout, central main root; shallow, moderate, or deep refers to relative rooting depth (influenced by soil and ground-water conditions). Planting: sizes given are those that are generally found in nurseries; other sizes may also be available. (Modified from Myers Biodynamics Inc., 1993, with additional data from Ken Thacker, Bureau of Land Management)

Scientific name **common name**	**Form and habit**	**Water requirements**	**Light requirements**	**Soil**	**Rooting characteristics**	**Planting**	**Comments**
Acer circinatum vine maple	deciduous shrub; may spread aggressively	moist	sun/shade full shade	any soil; tolerates shallow flooding during the growing season	moderate fibrous	to 4′ tall in containers; balled and burlapped plants to 10′ tall	Large specimens widely available; spreads by root and seed
Achillea millefolium white yarrow	perennial forb	dry	full sun	any well drained soil	fibrous	direct seeding in fall or spring	Aggressive once established
Alnus rubra red alder	deciduous tree; seeds prolifically on bare soil	moist	full sun	any soil	moderately deep fibrous	bareroot seedlings up to 3′ tall; larger plants in containers	Fast grower in poor mineral soils; typical 40–50-yr lifespan; large limbs become brittle; provides food for birds
Arctostaphylos uva-ursi kinnikinnick	low-growing shrub; spreads to form dense evergreen carpet	dry	full sun	any slightly acid soil	shallow fibrous	rooted plants in containers	Widely available evergreen ground cover; tolerates salt spray
Artemisia tridentata tridentata basin big sagebrush	tall evergreen shrub	dry	full sun	deep, well drained soil along drainages	very deep fibrous with taproot	direct seeding in fall; containerized stock in early spring	Variable ecotypes
Artemisia tridentata vayseyana mountain sagebrush	evergreen shrub	dry	full sun	deep, well drained, cool soils	very deep fibrous	containerized stock in early spring	Variable ecotypes, variety ‘Hobble Creek’
Artemisia tridentata wyomingensis Wyoming big sagebrush	evergreen shrub	dry	full sun	deep, well drained soil	very deep fibrous with taproot	containerized stock in early spring	Variable ecotypes
Atriplex canescens fourwing saltbush	evergreen shrub	dry	full sun	salt and alkaline tolerant	deep fibrous	cuttings or containerized stock in early spring	Tolerant of grazing; nitrogen fixer
Cornus stolonifera or *Cornus sericea* red-osier dogwood	deciduous shrub; does not spread	moist to wet	full sun sun/shade	any soil; tolerates shallow flooding during the growing season	shallow fibrous	rooted plants to 6′ tall in containers; bareroot and cuttings 18–24″ tall	Produces bright red stems
Elymus elymoides bottlebrush squirreltail	perennial bunchgrass	dry	full sun	well drained, disturbed sites	deep fibrous	direct seeding in fall or early spring	Very drought tolerant
Elymus lanceolatus streambank wheatgrass	perennial sod former	dry	full sun	well drained soils	fibrous, spreading with rhizomes	direct seeding in fall or early spring	Variety ‘Sodar’
Elymus trachycaulus slender wheatgrass	perennial bunchgrass	dry	full sun	well drained, cold soils	deep fibrous	direct seeding in fall or early spring	Variety ‘Primar’

Table 7.1. A partial listing of appropriate native plants suitable for erosion control and slope stabilization *(continued)*

Scientific name **common name**	**Form and habit**	**Water requirements**	**Light requirements**	**Soil**	**Rooting characteristics**	**Planting**	**Comments**
Gaultheria shallon salal	evergreen shrub; forms thickets by underground runners	dry to moist	sun/shade full shade	any soil; tolerates shallow flooding during the growing season	shallow fibrous	rooted plants 4–12″ tall	Widely available; difficult to establish; slow growing; tolerates salt spray
Helianthus annuus annual sunflower	annual forb	dry	full sun	any; prefers disturbed areas	fibrous	direct seeding in fall or spring	Easily established; food value for birds
Holodiscus discolor ocean spray	deciduous shrub; does not always spread aggressively	dry to moist	full sun sun/shade	any soil; tolerates shallow flooding during the growing season	moderate fibrous	to 2′ tall in containers; bareroot 6–12″ tall	Produces attractive sprays of creamy white flowers; will root spread
Linium lewisii Lewis flax	perennial forb	dry	full sun	well drained; tolerant of rocky conditions	deep fibrous	direct seeding in fall or spring	Variety ‘Appar’
Lonicera involucrata black twinberry	deciduous shrub; does not spread	moist	full sun	any soil	shallow fibrous	to 6′ tall in containers; bareroot 18–24″ tall	Produces yellow twin flowers and black twin berries; some success reported from cuttings
Myrica californica wax myrtle	evergreen shrub; does not spread	dry to moist	full sun sun/shade	slightly acid soil with organic matter	moderate fibrous	rooted plants to 10′	Tolerates salt spray; high wildlife usage
Oemleria cerasiformis indian plum	deciduous shrub; forms open stands by underground runners	moist	full sun to full shade	any soil	shallow fibrous with underground runners	to 4′ in containers; bareroot 6–8′ tall	Male and female flowers are on separate plants; only female flowers produce the ‘plums’
Oryzopsis hymenoides indian ricegrass	perennial bunchgrass	dry	full sun	well drained; prefers sandy soil	deep fibrous	direct seeding in fall or early spring	Variety ‘Nezpar’; very drought tolerant
Pershia tridentata antelope bitterbrush	perennial shrub	dry	moderately shade tolerant	deep, well drained soil; best on coarse soils	deep fibrous	containerized stock in early spring	Variable ecotypes; high forage value; nitrogen fixer
Physocarpus capitatus Pacific ninebark	deciduous shrub; does not spread	moist	full sun sun/shade	any soil	shallow fibrous	to 6′ tall in containers; bareroot 18–24″ tall	Produces masses of tiny white flowers that change to reddish seed clumps
Populus trichocarpa northern black cottonwood	deciduous tree; does not spread	moist; usage high	full sun	any soil; tolerates shallow flooding during the growing season	fibrous, shallow to deep and widespread	to 10′ tall in containers; cuttings 18–24″; whips 4′ tall	Fast grower in moist to saturated soils; also widely used for streambank stabilization; potential for wind throw
Pseudoroegneria spicata bluebunch wheatgrass	perennial bunchgrass	dry	full sun	well drained soil	deep fibrous	direct seeding in fall or early spring	Varieties ‘Secar’ and ‘Goldar’
Pseudotsuga menziesii Douglas fir	coniferous tree; does not spread	dry to moist; usage moderate	full sun	any soil	tap, modified tap; shallow to deep and widespread	12–18″ bareroot seedlings; larger plants in containers	Not good for slope stabilization; high root strength but typical shallow; can be planted in stands; good eagle and osprey perch and nest trees; potential for wind throw in thin or disturbed soils
Rhamnus purshiana cascara, buckthorn	deciduous tree/shrub; does not spread	moist	full sun	any soil	moderately deep tap	to 6′ tall in containers; bareroot 18–24″ tall	Shiny black berries are favored by cedar waxwings
Ribes sanguineum red currant	deciduous shrub; does not spread	dry to moist	full sun sun/shade	any soil	shallow fibrous (not extensive)	to 4′ tall in containers; bareroot to 18″ tall	Ornamental native; produces clusters of white to red flowers

Table 7.1. A partial listing of appropriate native plants suitable for erosion control and slope stabilization *(continued)*

Scientific name **common name**	**Form and habit**	**Water requirements**	**Light requirements**	**Soil**	**Rooting characteristics**	**Planting**	**Comments**
Rosa nutkana Nootka rose	deciduous shrub; forms thickets by underground runners	moist	full sun	any soil, prefers rich soils	shallow fibrous (not extensive)	rooted plants to 2′ tall in containers; bareroot to 18″ tall; cuttings 12–18″	Thickets of stems create a formidable barrier; pink flowers followed by large red hips; tolerates salt spray
Rubus parviflorus thimbleberry	deciduous shrub; forms thickets by underground runners	moist	full sun sun/shade	any soil	shallow fibrous	rooted plants in containers	May be difficult to find in some native plant nurseries
Rubus spectabilis salmonberry	deciduous shrub; forms thickets by underground runners	moist	sun/shade full shade	any soil	shallow fibrous	to 4′ tall in containers; bareroot 6-8″ tall; cuttings 18-24″	Spreads quickly once established; berries provide food for a variety of songbirds
Salix hookeriana Hooker willow	deciduous shrub; does not spread	moist to wet	full sun	any soil	moderately deep fibrous	to 6′ tall in containers; bareroot and cuttings 18–24″ tall; whips 4′; whips not recommended	Variety, 'Clatsop', has vigor, disease resistance, and attractive foliage; salt spray tolerant
Salix lasiandra Pacific willow	deciduous multi-stemmed tree; does not spread	wet; usage high?	full sun	any soil; tolerates shallow flooding during the growing season	fibrous, moderately deep and widespread	to 10′ tall in containers; cuttings 18–24″; whips 4′	Fast grower in saturated or shallowly flooded areas; 25-year lifespan; large limbs become brittle
Salix scouleriana scouler willow	deciduous tree/shrub; does not spread	dry to moist; usage high?	full sun	any soil	fibrous, moderately deep and widespread	to 10′ tall in containers; cuttings 18–24″; whips 4′; whips not recommended	Of the willows listed here, this species tolerates the driest conditions
Salix sitchensis Sitka willow	deciduous tree or shrub; does not spread	moist; usage high?	full sun	any soil	fibrous, moderately deep and widespread	to 10′ tall in containers; cuttings 18–24″; whips 4′; whips not recommended	Fast grower in moist to saturated soils; widely used for streambank stabilization
Sambucus racemosa red elderberry	deciduous shrub; does not spread	moist	full sun to full shade	any soil	shallow fibrous	to 6′ tall in containers; bareroot 18–24″ tall	Produces red nonedible berries; some success reported from woody cuttings
Spiraea douglasii Douglas' spiraea	deciduous shrub; spreads by seed and underground runners	moist to wet	full sun	any soil; tolerates shallow flooding during the growing season	shallow fibrous	to 6′ tall in containers; bareroot and cuttings 18–24″ tall	Spreads quickly and aggressively in most sites
Symphoricarpus albus snowberry	deciduous shrub; forms thickets by underground runners	dry to moist	full sun to full shade	any soil; tolerates shallow flooding during the growing season	shallow fibrous	rooted plants to 24″ tall; bareroot 6–18″ tall	Tolerates high winds and often grows on vegetated slopes overlooking salt water
Vaccinium ovatum evergreen huckleberry	evergreen shrub; does not spread	dry to moist	sun/shade full shade	slightly acid	shallow fibrous	rooted plants to 2′ tall in containers	Attractive, but slow-growing; difficult to establish; tolerates salt spray
native plant seed mixes	annual and perennial grass and forb mixes available	dry to wet; usage medium to high	species dependent	species dependent	shallow fibrous	seed; seeds of woody plants also available (success typically low); very slow to establish	Avoid exotic commercial mixes; seed mixes typically used in conjunction with other types of plantings; typically short-term erosion control technique

Table 7.2. Plant selection guide for legumes, except for lupines—Species characteristics, adaptations, and seeding rates. (See Table 7.3 for lupines.) PLS, pure live seed. (Modified from Grassland West, 1994)

Scientific name **common name**	**Adaptation**	**Minimum precipitation (inches/year)**	**Bloat/nonbloat**	**PLS pounds/acre**	**Seeds/pound**	**Varieties**
Astragalus cicer cicer milkvetch	best on medium to clayey textures	12 to 18	NB	20 to 25	145,000	Lutana, Monarch
Coronilla veria crownvetch	well-drained, most soil, neutral pH	20 to 25	B	15 to 20	110,000	Emerald, Penngift, Chemung
Hedysarum boreale northern sweetvetch	drought-tolerant native legume	12	NB	10 to 15	30,000	Timp
Lotus corniculatus birdsfoot trefoil	medium to clay soils	18 to 24	NB	4 to 6	418,000	Dawn, Empire
Medicago sativa alfalfa	deep, well-drained soils, all textures	15 to 18	B	8 to 15	210,000	Legacy, Cimarron, Vector, Angler, Cody
Melilotus alba white sweetclover	drought, saline, and alkaline tolerant	12	B	10 to 15	260,000	
Melilotus officinalis yellow sweetclover	wide range of soils	12	B	10 to 15	260,000	Madrid
Onobrychis viciaefolia sainfoin	deep, well-drained soils of all textures	15 to 18	NB	35 to 45	30,000	Eski, Remont, Renumex
Trifolium fragiferum strawberry clover	wet, saline and alkaline tolerant; shade	15 to 18	B	5 to 15	300,000	O'Connors, Salina, Fresa
Trifolium hirtum rose clover	warm winter ranges, green crop	15 to 20	B	20	140,000	Hykon
Trifolium hybridum alsike clover	heavy silt to clay soils, alkaline sites	18 to 20	B	6 to 8	680,000	
Trifolium pratense red clover	heavy, fertile, well-drained soils	18 to 20	B	8 to 10	275,000	Kenland, Redland, Arlington, Mammoth
Trifolium repens white dutch clover	medium to clayey, shallow soils	18 to 20	B	2 to 6	850,000	
Trifolium repens latum ladino clover	medium to clayey, shallow soils	18 to 20	B	2 to 6	800,000	
Vicia americana American vetch	wide range of soils, best in meadows	18 to 20	NB	10 to 20	75,000	
Vicia dasycarpa woolly pod vetch	wide range of soils, best on rich loam	18 to 20	NB	35 to 40	100,000	Lana
Vicia villosa hairy vetch	wide range of soils, tolerates poor sandy sites	18 to 20	NB	25 to 35	20,000	

Table 7.3. Plant selection guide for lupines—Species characteristics, adaptations, and seeding rates. PLS, pure live seed. (Modified from Grassland West, 1994)

Scientific name **common name**	**Adapted range**	**Annual/ perennial**	**Color**	**Height (inches)**	**Native/ introduced**	**Seeding rate (PLS pounds/acre)**	**Seeds/ pound**
Lupinus alpestris mountain lupine	Rocky Mountains and western North America	perennial	blue	12 to 20	N	25	12,500
Lupinus arizonicus desert lupine	southwest deserts	annual	blue	12 to 48	N	3	135,000
Lupinus caudatus tailcup lupine	Rocky Mountains and western North America	perennial	blue	12 to 24	N	12	27,600
Lupinus densiflorus aureus golden lupine	Pacific coast	annual	yellow	24 to 36	N	35	13,500
Lupinus perennis wild lupine	throughout North America	perennial	purplish-blue	12 to 24	N	11	21,000
Lupinus sericeus silky lupine	Rocky Mountains and western North America	perennial	blue	12 to 24	N	10 to 25	12,900
Lupinus succulentus arroyo lupine	Pacific coast and northwestern North America	annual	blue	24 to 28	N	20	15,600
Lupinus texensis Texas bluebonnet	southcentral and southwestern North America	annual	blue and white	16 to 20	N	16 to 20	16,000

Table 7.4. Plants for special-use situations. PLS, pure live seed. (Modified from Grassland West, 1994. Copyright ©1994 by Grassland West. Used by permission of the publisher)

Scientific name Common name	Cool/warm season	Minimum precip. (in./yr)	Bunch/sod former	Native/ introduced	PLS lb/acre	Planting dates
DROUGHT-TOLERANT BUNCHGRASSES						
Agropyron inerme beardless bluebunch wheatgrass	C	8	B	N	7 to 8	spring or fall
Agropyron desertorum standard crested wheatgrass	C	10	B	I	6 to 8	spring or fall
Agropyron elongatum tall wheatgrass	C	8	B	I	6 to 8	spring or fall
Agropyron sibiricum Siberian wheatgrass	C	6	B	I	6 to 8	fall
Agropyron spicatum bluebunch wheatgrass	C	8	B	N	6 to 8	spring or fall
Bouteloua curtipendula sideoats grama	W	8	B	N	3 to 6	spring or fall
Elymus cinereus Great Basin wildrye	C	12	B	N	9	spring or fall
Elymus elymoides bottlebrush squirreltail	C	6	B	N	8 to 10	spring or fall
Elymus junceus Russian wildrye	C	12	B	I	8 to 10	spring or fall
Eragrostis curvula weeping lovegrass	W	16	B	I	2	April to August 15
Festuca longifolia hard fescue	C	16	B	I	10	spring or fall
Festuca ovina sheep fescue	C	10	B	N	10	spring or fall
Oryzopsis hymenoides Indian ricegrass	C	9	B	N	6 to 8	spring or fall
Poa nevadensis Nevada bluegrass	C	10	B	N	3	spring or fall
Sporobolus cryptandrus sand dropseed	W	10	B	N	1	April to May 31
Stipa comata needle and thread	C	10	B	N	8	spring or fall
DROUGHT-TOLERANT SOD-FORMING GRASSES						
Agropyron dasystachyum thickspike wheatgrass	C	8	S	N	6 to 8	spring or fall
Agropyron riparium streambank wheatgrass	C	8	S	N	6 to 8	spring or fall
Agropyron smithii western wheatgrass	C	10	S	N	10	spring or fall
Bouteloua gracilis blue grama	W	12	S	N	2 to 3	spring or fall
Buchloe dactyloides buffalograss	W	12	S	N	4 to 8	June to August 15
Cynodon dactylon Bermuda grass	W	10	S	I	15	April to August
Elytrigia intermedia intermedia intermediate wheatgrass	C	14	S	I	15	spring or fall
Elytrigia intermedia trichophorum pubescent wheatgrass	C	14	S	I	10 to 12	fall
Festuca rubra red fescue	C	18	S	I	10	spring or fall
Poa compressa Canada bluegrass	C	18	S	I	1 to 2	spring or fall
Schizachyrium scoparium little bluestem	W	14	S	N	3 to 4	spring or fall

ACID-TOLERANT GRASSES						
Scientific name **common name**	**Cool/warm season**	**Minimum precip. (in./yr)**	**Bunch/sod former**	**Native/ introduced**	**PLS lb/acre**	**Planting dates**
Agrostis alba redtop	C	20	S	I	1	spring or fall
Agrostis palustris creeping bentgrass	C	20	S	I	.5 to 1	spring or fall
Agrostis tenuis colonial bentgrass	C	18	S	I	2	spring or fall
Alopecurus arundinaceus creeping foxtail	C	25	S	I	3 to 4	spring or fall
Alopecurus pratensis meadow foxtail	C	25	B	I	4 to 5	spring or fall
Cynodon dactylon Bermuda grass	W	10	S	I	15	April to August
Eragrostis curvula weeping lovegrass	W	16	B	I	2	spring or fall
Festuca longifolia hard fescue	C	16	B	I	10	spring or fall
*Festuca rubr*a red fescue	C	18	S	I	10	spring or fall
Festuca rubra, var. *commutata* Chewings fescue	C	18	B	I	4 to 5	spring or fall
Lolium perenne perennial ryegrass	C	12	B	I	25 to 35	spring or fall
*P*anicum virgatum switchgrass	W	18	S	N	5 to 8	June to August
*P*oa compressa Canada bluegrass	C	18	S	I	1 to 2	spring or fall
ALKALINE-TOLERANT GRASSES						
Agropyron desertorum standard crested wheatgrass	C	10	B	I	7 to 10	spring or fall
Agropyron elongatum tall wheatgrass	C	8	B	I	6 to 20	spring
Agropyron riparium streambank wheatgrass	C	8	S	N	6 to 8	spring or fall
Agropyron smithii western wheatgrass	C	10	S	N	10	spring or fall
Agropyron trachycaulum slender wheatgrass	C	16	B	N	6 to 8	fall
Cynodon dactylon Bermuda grass	W	10	S	I	15	April to August
Distichlis stricta inland saltgrass	W	8	S	N	10	June to August
Elymus canadensis Canada wildrye	C	12	B	N	7	spring or fall
Elymus cinereus Great Basin wildrye	C	8	B	N	9	spring or fall
Elymus junceus Russian wildrye	C	12	B	I	8 to 10	fall
Lolium perenne perennial ryegrass	C	12	B	I	25 to 35	spring or fall
Puccinellia distans alkaligrass	C	15	B	N	2 to 3	spring or fall
Sporobolus airoides alkali sacaton	W	6	B	N	2 to 3	July to October

GRASSES AND LEGUMES TOLERANT OF OCCASIONALLY SATURATED SOILS

Scientific name **Common name**	**Cool/warm season**	**Minimum precip. (in./yr)**	**Bunch/sod former**	**Native/ introduced**	**PLS lb/acre**	**Planting dates**
Agrostis alba redtop	C	20	S	I	1	spring or fall
Agrostis palustris creeping bentgrass	C	20	S	I	.5 to 1	spring or fall
Alopecurus arundinaceus creeping foxtail	C	25	S	I	3 to 4	spring or fall
Alopecurus pratensis meadow foxtail	C	25	B	I	4 to 5	spring or fall
Festuca elatior meadow fescue	C	25	B	I	6	spring or fall
Lolium perenne perennial ryegrass	C	12	B	I	25 to 35	spring or fall
Phalaris arundinacea reed canarygrass	C	16	S	N	5 to 10	spring or fall
Poa trivialis Poa trivialis	C	25	S	I	4	spring or fall
Trifolium hybridum alsike clover	C	35	B	H	6 to 8	spring
COLD-TOLERANT GRASSES						
Deschampia caespitosa tufted hairgrass	C	20	B	N	1 to 2	spring or fall
Elymus cinereus Great Basin wildrye	C	12	B	N	9	spring or fall
Elymus elymoides bottlebrush squirreltail	C	6	B	N	8 to 10	spring or fall
Festuca elatior meadow fescue	C	25	B	I	6	spring or fall
Festuca longifolia hard fescue	C	16	B	I	10	spring or fall
Festuca ovina sheep fescue	C	10	B	N	10	spring or fall
Festuca rubra red fescue	C	18	S	I	10	spring or fall
Festuca rubra, var. *commutata* Chewings fescue	C	18	B	I	4 to 5	spring or fall
Poa alpinum alpine bluegrass	C	20	B	N	1	spring or fall
Poa pratensis Kentucky bluegrass	C	18	S	N	2 to 3	spring or fall

GRASSES PROVIDING TEMPORARY COVER

(These grasses are generally planted in the spring for temporary cover. They should not be used for permanent revegetation.)

Arrhenatherum elatius
tall oatgrass

Avena sativa
oats

Bromus arvensis
field brome

Hordeum vulgare
barley

Lolium multiflorum
annual ryegrass

Secale cereale
winter rye

Sorghum vulgare, var. *sudanense*
Sudangrass

Technical Memorandum

TO: Queen City Farms

FROM: Katherine Saltanovitz, PE and Meghan Veilleux, EIT

DATE: August 31, 2019

RE: **Queen City Lake Basin – Western Washington Hydrology Model**
Queen City Farms Phase III Refill
Maple Valley, Washington
Project No. 0992002.050.051

Introduction

Stormwater management at the Queen City Farms (QCF) site comprises several components including infiltration areas, constructed detention ponds, and enhanced storage in Queen City Lake. The currently permitted refill plan for QCF includes adding an outlet structure to the lake to increase the stormwater storage available within the lake, while allowing for emergency overflow of large storm events. W&H Pacific completed a study in December 2006 that calculated the peak stormwater runoff release rates and peak stage/elevation within the lake to model changes to lake wetland hydrology and provide recommendations for the proposed lake outlet structure (W&H Pacific 2006).

The next phase of the QCF refill (referred to as the Phase III Refill) is proposed to include rerouting the existing Tributary 316A from its current infiltration location into Queen City Lake. The purpose of this current technical memorandum is to update the December 2006 study to model the effect on water levels in Queen City Lake if Tributary 316A is redirected into the lake and to determine if any changes are needed to the proposed outlet structure.

Queen City Lake Hydrology

Queen City Lake has no natural surface water outlet. Water infiltrates into the underlying sediments and eventually to an aquifer that underlies the lake bottom (LAI 2007). Water infiltration rate within the lake is dependent on the water elevation. Landau Associates, Inc. (LAI) estimated the infiltration rates by using the relationship of the lake level with lake volume, as presented in a technical memorandum dated January 29, 2007 (LAI 2007).

The infiltration rates for the varying lake depths are presented in Table 1, which is reprinted from Table 2 of the January 29, 2007 LAI memorandum (LAI 2007).

Table 1 – Queen City Lake Estimated Infiltration Rates (reprinted from 2007 TIR Figure 2)

Lake Depth (ft)	Estimated Infiltration Rate (cfs)	Estimated Lake Surface Area (sf)	Estimated Infiltration Rate per Unit Area (in/hr)
0 to 1	0.3	100,000	0.13
1 to 2	0.8	240,000	0.14
2 to 3	1.2	320,000	0.16
3 to 4	1.6	420,000	0.16
4 to 5	1.8	470,000	0.17
5 to 6	2.0	520,000	0.17
6 to 7	3.0	640,000	0.20
7 to 8	7.0	675,000	0.45
8 to 9	15.7	750,000	0.90

Notes:

1. Bottom of the lake elevation assumed to be 444 ft MSL.
2. Queen City Lake surface area estimated from Figure 3-10 of the Queen City Farms RI Report (Landau Associates 1990).

Abbreviations:

ft = feet/foot
cfs = cubic feet per second
sf = square feet
in/hr = inch per hour

Queen City Lake does have an existing 36-inch outlet pipe that was installed in 1991 to control erosion of the gravel pit face. This pipe drains to another on-site infiltration area. This pipe will be removed as part of the currently permitted refill and replaced with a new engineered outlet structure.

Surface water enters Queen City Lake as runoff from the Cedar Hills Sub-basin, which covers approximately 340 acres (prior to the development of Phase III) and includes a portion of the Cedar Hills Regional Landfill (CHRL; LAI 2007). In the December 2006 study by W&H Pacific, this basin was modeled as till forest conditions. The surface area of the lake is considered an impervious surface for all analyses. W&H Pacific delineated the Cedar Hills Sub-basin from Attachment 1 provided in a stormwater report written by King County Solid Waste Division and LiDAR topographic mapping data from 2000.

Drainage Basins

Tributary 316A, which drains the Maple Hills Sub-basin, currently flows through the Queen City Farms site and infiltrates in the Main Infiltration Area. As part of the Phase III Refill, it is proposed to reroute Tributary 316A into Queen City Lake. The boundary of the Maple Hills sub-basin is shown on Figure 3A of the TIR. The entire Maple Hills sub-basin was modeled to represent the runoff that will be directly flowing into the lake from Tributary 316A.

The lake will continue to receive runoff from the Cedar Hills Sub-basin as well. The Cedar Hills sub-basin includes CHRL property, some buffer area east and west of the CHRL boundary limits, as well as Queen City Farm's property. The boundary of this sub-basin was updated based on site development that has occurred since 2007, and on recent aerial photographs; it is shown in Figure 3A of the TIR.

The lake will also receive approximately 32 acres of runoff from the gravel refill mound, which currently flow to Main Gravel Pit Lake and Cedar Shores Stormwater Ponds. The drainage area that will be redirected to Queen City Lake from Main Gravel Pit Lake and from the Cedar Shores Stormwater Ponds is shown on Figure 3B of the TIR.

Outlet Structure Design

Consistent with the 2006 W&H Pacific study, there are two design parameters for the outlet structure:

1. It is assumed that wetland species have been established from the bottom portion of the lake to an elevation that is approximately equal to the invert of the existing 36-inch corrugated metal outlet pipe. The new outlet structure will maintain this minimum water level (5 feet above the lake bottom).
2. The maximum lake water level will be maintained at a depth less than or equal to 9 feet (ft) above the lake bottom to protect the adjacent superfund site barrier wall. Major (greater than 100-year return frequency) storm events will be routed through the lake with a high flow overflow structure.

Western Washington Hydrology Model

Due to the 2016 update of the King County Surface Water Design Manual (KCSWDM) that occurred between the original W&H Pacific study and this Phase III project, the Queen City Lake modeling has been updated using an approved model, the Western Washington Hydrology Model (WWHM), to meet the requirements of the KCSWDM, which no longer accepts the use of the King County Runoff Time Series (KCRTS) modeling. Additionally, because the drainage entering Queen City Lake has changed significantly, including the addition of a new basin, the modeling previously performed by W&H Pacific has been updated to reflect these changes.

Drainage Basins and Land Cover

The previous W&H Pacific study included only the Cedar Hills Sub-basin as a surface water source for Queen City Lake, with the following land cover (Attachment 2):

Till Forest:	331.5 acres
Impervious:	8.5 acres
Total:	340.0 acres

Since 2006, the Cedar Hills Sub-basin has changed slightly. The boundary was modified and a compost pad was added that allows infiltration, decreasing some forested area. The land cover in this basin has not changed significantly since the previous study; therefore, in WWHM, it is modeled as the same land cover (till forest).

With the plan to reroute Tributary 316A to Queen City Lake, the Maple Hills Sub-basin is added to the modeling as a contributing upstream basin. Based on aerial base maps (Terrain Navigator Pro 2000), this sub-basin appears to be mostly forested, with some developed and residential areas in the upper northwest corner of the basin. Additionally, a portion of the Cedar Grove Composting property overlaps with the basin. Cedar Grove collects and sends runoff from their property to a King County sanitary sewer line; therefore this area is not included in the modeling. The developed and residential area in the northwest corner has been conservatively modeled as impervious, with the rest of the drainage basin modeled as “Outwash Forest”. A portion of the new compost pad is contained in the Maple Hills Sub-basin, but is not included in the drainage area since it is routed to its own infiltration basin rather than entering the lake. Additionally, due to the Phase III Refill project, the drainage patterns have changed, resulting in a portion of the Maple Hills Sub-basin that will no longer drain to Tributary 316A, and subsequently Queen City Lake.

The Phase III Refill project also results in a portion of the Main Gravel Pit Sub-Basin and a portion of the area draining to the Cedar Shores Stormwater Ponds to be routed to Queen City Lake. Consistent with modeling used for sizing other stormwater facilities for QCF, the refill area is modeled as “Till Pasture”. A summary of the sub-basins that were modeled in WWHM are shown in Table 2. For the purposes of modeling, it is assumed that the average slope of the pervious land types are moderate (5-15%) and the average slope of the impervious land types are flat (0-5%).

Table 2 – Queen City Lake Sub-Basin Summary

Sub-Basin					
Cedar Hills		**Maple Hills**		**Phase III**	
Till Forest: Till Pasture: Lake (Impervious):	333 ac 11 ac 8.5 ac	Outwash Forest: Till Pasture: Impervious:	128 ac 8 ac 38 ac	Till Pasture	13 ac
TOTAL:	**352.5 ac**	**TOTAL:**	**174 ac**	**TOTAL:**	**13 ac**
Not included in modeling: - 6.5 ac (infiltrates via compost pad/infiltration pond)		**Not included in modeling:** - 19 ac (Cedar Grove Compost Facility – to SS) - 12 ac (infiltrates via compost pad/infiltration pond or conveyed via french drain to Main Infiltration Area) - 19 ac (areas that infiltrate or area re-routed to other stormwater facilities)		**NOTE:** This area originally drained to the Cedar Shores Stormwater Ponds.	

Therefore, the total contributing drainage area to Queen City Lake in the Phase III Refill is 539.5 ac.

Lake Volume

Lake stage-storage volume was calculated from topographic survey presented in the W&H Pacific 2006 study. Attachment 3 shows the resulting contours generated from the surveyed data, the location of the existing 36-inch corrugated metal outlet pipe, and the location of the existing overflow point. Together with the cross-sections and LiDAR, a volume of a reservoir calculation was performed. The stage-storage volume calculation and chart for the existing Queen City Lake were shown in Figures A2 and A3 of the 2006 study. Figure A2 was used to develop Table 3, which shows stage and storage volume of Queen City Lake up to a maximum depth of 10.9 ft. Figure A3 is reproduced as Table 3 and Figure 2 of this memorandum. As reported in the 2006 study, the total storage volume of Queen City Lake below a depth of 9.0 ft is approximately 62.0 acre-feet (ac-ft), based on the stage-storage curve shown in Figure 2.

Table 3 – Queen City Lake Volume of a Reservoir Calculation

Elevation (ft)	Stage (ft)	Elevation Difference (ft)	Area (sf)	Incremental Volume (cf)	Total Volume (cf)	Total Volume (ac-ft)
446.6	0		0		0	0
		0.4		37		
447	0.4		276		37	0.0008
		1.0		28,829		
448	1.4		81,470		28,866	0.6627
		1.0		126,733		
449	2.4		178,228		155,599	3.5721
		1.0		209,894		
450	3.4		243,242		365,493	8.3906
		1.0		275,747		
451	4.4		309,583		641,240	14.7208
		1.0		340,166		
452	5.4		371,695		981,406	22.5300
		1.0		401,890		
453	6.4		432,861		1,383,296	31.7561
		1.0		471,378		
454	7.4		510,974		1,854,674	42.5774
		1.0		540,975		
455	8.4		571,541		2,395,648	54.9965
		1.0		602,137		
456	9.4		633,261		2,997,786	68.8197
		1.5		971,082		
457.5	10.9		661,618		3,968,867	91.1127

Abbreviation/Acronym:

cf = cubic foot/feet

Outlet Structure

The originally designed outlet structure for Queen City Lake (as permitted for Phase II, which did not include rerouting of Tributary 316A) has a two-stage discharge. The Stage 1 discharge is a 12-inch orifice set at the elevation of the existing 36-inch outlet pipe (451.6 ft), which ensures that design parameter 1 will be satisfied (no impacts to wetland vegetation below this elevation). The Stage 2 discharge is an overflow weir set just below the maximum water elevation of 9.0 ft above the lake bottom (455.6 ft), which ensures that design parameter 2 will be satisfied. The sizing of the weir was based on the 100-year peak inflow to Queen City Lake, which was originally modeled by W&H Pacific as 40.8 cubic feet per second (cfs). This modeling was updated for Phase III in WWHM to account for the additional basin area, and is attached in Appendix A. The new peak inflow to Queen City Lake, used for overflow weir sizing, is 105.6 cfs.

Model Input

Stage, area, storage, and infiltration values were manually input into WWHM to model actual conditions of Queen City Lake via the stage-storage-discharge (SSD) table tool. See Table 4, below, for WWHM SSD inputs, which incorporates information from Table 3, above. An outlet structure column was also added to simulate the proposed Queen City Lake outlet. The manual outlet structure input parameters are as follows:

- Riser Height (Stage 2): 7.96 ft (corresponds to elevation of 454.56 ft)
- Riser Diameter (Stage 2): 36 in
- Riser Type: Flat
- Orifice Diameter (Stage 1): 12 in
- Orifice Height (Stage 1): 5 ft (corresponds to elevation of 451.60 ft)

The WWHM Report for the Queen City Lake outlet structure can be found in Appendix C.

Table 4 – SSD Table Inputs

Stage (ft)	Area (acres)	Storage (ac-ft)	Infiltration (cfs)
0	0	0	0
0.4	0.006	0.0008	0.3
1.4	1.870	0.6627	0.8
2.4	4.092	3.5721	1.2
3.4	5.584	8.3906	1.6
4.4	7.107	14.7208	1.8
5.4	8.533	22.5300	2.0
6.4	9.937	31.7561	3.0
7.4	11.730	42.5774	7.0
8.4	13.121	54.9965	15.7
9.4	14.538	68.8197	15.7
10.9	15.189	91.1127	15.7

Table 5 – Summary of 2- and 100-year Peaks for All Scenarios

Scenario	Total drainage area (ac)	Orifice Diameter	2-year infiltration rate (cfs)[1]	2-year Peak Release Rate (cfs)	2-year Stage Elevation (ft)	100-year infiltration rate (cfs)[1]	100-year Peak Release Rate (cfs)	100-year Stage Elevation (ft)
Phase II	340	12"	2.93	3.38	6.33	15.70	7.02	8.96
Phase III	539.5	12"	4.15	5.18	6.68	15.70	13.69	8.76

[1] 2-year and 100-yr infiltration rates are found by analyzing the "infiltration outlet" (Outlet 2) of the SSD element in WWHM as the "point of compliance" (POC), rather than the outlet structure (Outlet 1). The WWHM report only reports the outlet structure (Outlet 1) peak release rates.

Conclusions

Rerouting Tributary 316A to Queen City Lake will affect the peak inflow to the lake as well as the discharge rates from the proposed outlet structure. Compared to the permitted Phase II design, Queen City Lake inflows will increase from 40.76 cfs to 105.62 cfs. Peak 2-year outflows will increase from 3.38 cfs to 5.18 cfs, and 100-year peak outflows will increase from 7.02 cfs to 13.69 cfs. To account for the higher outflow from the lake, the Stage 2 outlet elevation will be decreased from 455.6 ft to 454.6 ft to prevent the lake level from exceeding 9 ft. The Stage 2 outlet dimensions and both the Stage 1 outlet elevation and diameter remain unchanged.

LANDAU ASSOCIATES, INC.

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Katherine Saltanovitz, PE
Senior Associate

MDV/KMS/kjg
P:\projects\992\002\FileRm\R\Revised Ph3 TIR\Sec 6 TMs\6.10 Queen City Lakes\Queen City Lake Basin-WWHM Modeling.docx

Attachments: Figure 1: VOID
Figure 2: Queen City Lake – Stage Storage, Existing Condition Chart

Appendix A: VOID
Appendix B: VOID

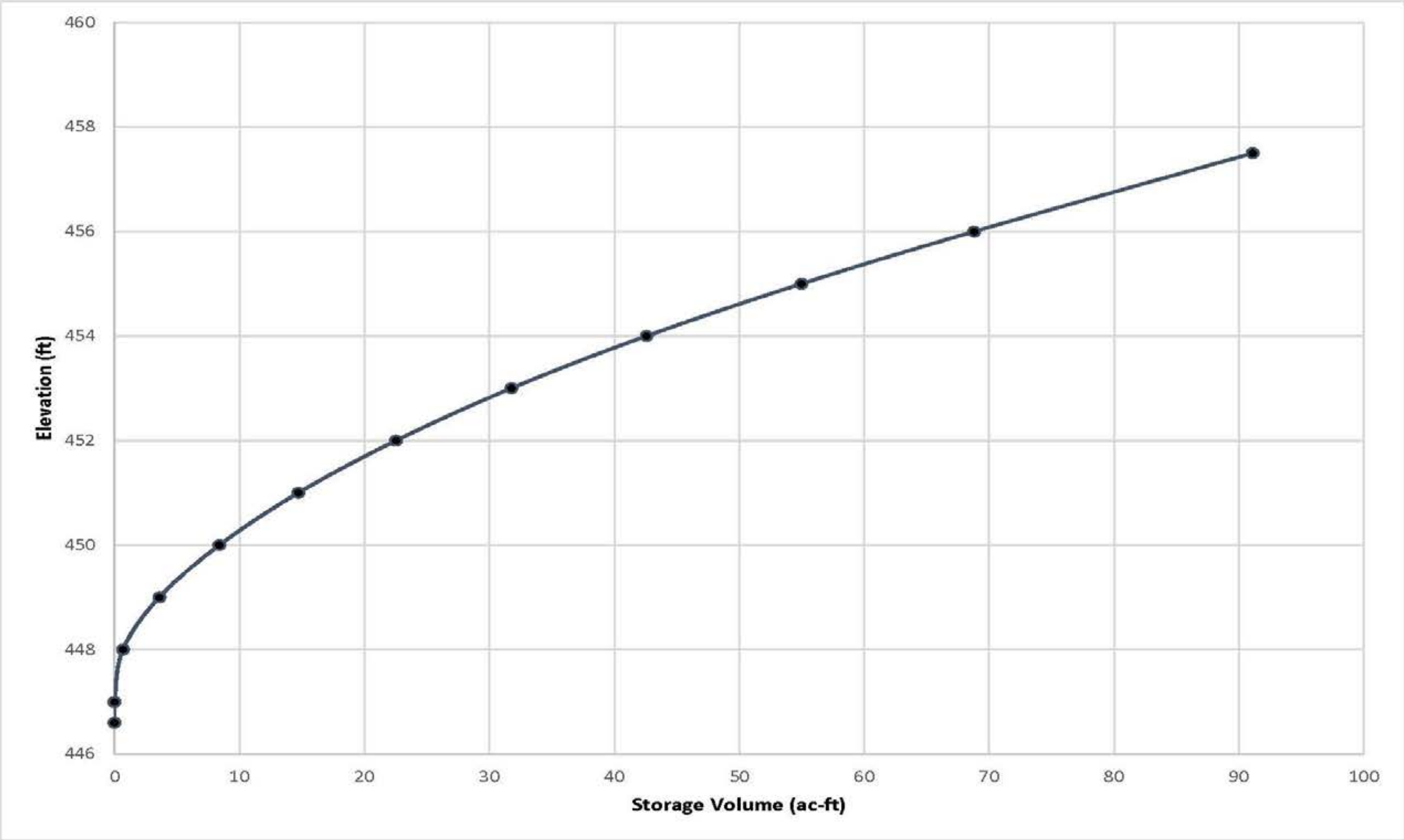
Appendix C: Queen City Lake – WWHM Report

Attachment 1: Cedar Hills Regional Landfill – KCRTS Analysis Final Development Scenario Sub-Basin Boundaries Based on 2000 Topography
Attachment 2: Cedar Hills Regional Landfill 2000 Basin Topo – Queen City Lake
Attachment 3: Queen City Lake – Storage Routing and Water Level, Existing Conditions

References

LAI. 2007. Technical Information Report, Queen City Farms Refill Project, Cedar Grove Road SE, King County, Washington. Landau Associates, Inc. August 13, 2007.

W&H Pacific. 2006. Letter: Queen City Farms - Storage Routing and Water Level Analysis, Phase 3 Outlet Structure Sizing. From Michael Gomez, W&H Pacific, to Brian Butler, Senior Associate Geologist, Landau Associates, Inc. December 13.



Note: Figure is reprinted from the W&H Pacific technical memorandum "Queen City Farms - Storage Routing and Water Level Analysis Phase 3 Outlet Structure Sizing" dated December 13, 2006.

Landau Associates

Queen City Farms Phase III Refill Maple Valley, Washington	**Queen City Lake Stage Storage, Existing Condition Chart**	Figure 2

APPENDIX C

Queen City Lake – WWHM Report

WWHM2012

PROJECT REPORT

General Model Information

Project Name:	QCF Phase III - Queen City Lake Outlet Sizing
Site Name:	Queen City Farms
Site Address:	17825 Cedar Grove Rd SE
City:	Maple Valley
Report Date:	8/26/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.167
Version Date:	2018/10/10
Version:	4.2.16

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Cedar Hills dev sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	333
C, Pasture, Mod	11
Pervious Total	344
Impervious Land Use	acre
POND	8.5
Impervious Total	8.5
Basin Total	352.5

Element Flows To:

Surface	Interflow	Groundwater

Maple Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
C, Pasture, Mod	8
Pervious Total	136
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	174

Element Flows To:

Surface	Interflow	Groundwater

Phase III to QCL

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	13
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Maple Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
C, Pasture, Mod	8
Pervious Total	136
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	174

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Cedar Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	333
C, Pasture, Mod	11
Pervious Total	344
Impervious Land Use	acre
POND	8.5
Impervious Total	8.5
Basin Total	352.5

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Phase III Refill to QCL

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	13
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Routing Elements
Predeveloped Routing

Mitigated Routing

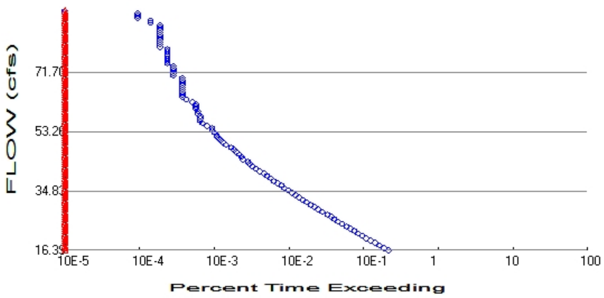
Queen City Lake

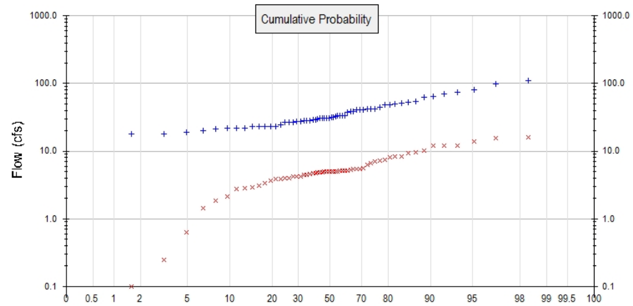
Depth: 10.9 ft.
Discharge Structure: 1
Riser Height: 7.96 ft.
Riser Diameter: 36 in.
Orifice 1 Diameter: 12 in. Elevation:5 ft.
Element Flows To:
Outlet 1 Outlet 2

SSD Table Hydraulic Table

Stage (feet)	Area (ac.)	Volume (ac-ft.)	Outlet Struct	Infilt (cfs)	NotUsed	NotUsed	NotUsed
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.400	0.006	0.001	0.000	0.300	0.000	0.000	0.000
1.400	1.870	0.663	0.000	0.800	0.000	0.000	0.000
2.400	4.092	3.572	0.000	1.200	0.000	0.000	0.000
3.400	5.584	8.391	0.000	1.600	0.000	0.000	0.000
4.400	7.107	14.72	0.000	1.800	0.000	0.000	0.000
5.400	8.533	22.53	2.471	2.000	0.000	0.000	0.000
6.400	9.937	31.76	4.624	3.000	0.000	0.000	0.000
7.400	11.73	42.58	6.054	7.000	0.000	0.000	0.000
8.400	13.12	55.00	16.33	15.70	0.000	0.000	0.000
9.400	14.54	68.82	41.95	15.70	0.000	0.000	0.000
10.90	15.19	91.11	58.10	15.70	0.000	0.000	0.000

Analysis Results
POC 1





+ Predeveloped   x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 493
Total Impervious Area: 46.5

Mitigated Landuse Totals for POC #1
Total Pervious Area: 493
Total Impervious Area: 46.5

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	32.775625
5 year	47.768722
10 year	59.378734
25 year	76.101607
50 year	90.141409
100 year	105.616188

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	5.179732
5 year	8.281944
10 year	9.998736
25 year	11.764039
50 year	12.820834
100 year	13.693677

Annual Peaks
Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	50.858	3.890
1950	48.944	6.249
1951	40.610	16.199
1952	22.991	3.092
1953	19.226	3.965
1954	26.830	5.092
1955	31.257	5.134
1956	30.418	6.543
1957	37.020	4.814
1958	23.401	5.401

1959	21.848	4.575
1960	41.900	10.124
1961	29.078	4.819
1962	17.999	1.873
1963	28.627	4.970
1964	28.480	4.499
1965	33.299	5.161
1966	23.074	4.242
1967	50.189	5.376
1968	32.405	4.718
1969	28.592	4.177
1970	30.871	3.965
1971	33.688	4.771
1972	41.832	9.656
1973	21.813	4.966
1974	33.485	5.193
1975	40.273	4.507
1976	30.365	5.000
1977	21.235	0.000
1978	27.562	4.886
1979	30.632	2.158
1980	70.258	8.430
1981	29.868	4.221
1982	54.614	5.434
1983	26.534	5.044
1984	22.009	2.904
1985	23.259	0.248
1986	38.988	7.022
1987	39.060	8.275
1988	17.832	2.866
1989	24.169	1.433
1990	111.627	12.012
1991	63.831	12.140
1992	27.125	4.988
1993	19.894	3.349
1994	15.454	0.628
1995	26.805	5.551
1996	63.304	15.479
1997	41.862	12.152
1998	28.411	2.780
1999	73.944	8.036
2000	30.697	3.663
2001	23.447	0.000
2002	40.328	4.981
2003	48.488	3.872
2004	53.182	7.201
2005	33.648	5.023
2006	31.518	5.217
2007	98.549	9.439
2008	79.507	13.868
2009	44.157	7.458

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	111.6270	16.1991
2	98.5492	15.4790
3	79.5073	13.8682

4	73.9443	12.1519
5	70.2581	12.1396
6	63.8307	12.0116
7	63.3038	10.1244
8	54.6140	9.6563
9	53.1816	9.4391
10	50.8580	8.4301
11	50.1890	8.2749
12	48.9439	8.0362
13	48.4878	7.4579
14	44.1568	7.2010
15	41.9000	7.0218
16	41.8620	6.5430
17	41.8315	6.2487
18	40.6099	5.5505
19	40.3283	5.4336
20	40.2733	5.4014
21	39.0604	5.3760
22	38.9884	5.2171
23	37.0198	5.1933
24	33.6880	5.1608
25	33.6479	5.1344
26	33.4852	5.0915
27	33.2994	5.0440
28	32.4045	5.0231
29	31.5179	4.9996
30	31.2571	4.9884
31	30.8712	4.9811
32	30.6971	4.9698
33	30.6323	4.9658
34	30.4175	4.8856
35	30.3647	4.8193
36	29.8675	4.8143
37	29.0776	4.7707
38	28.6270	4.7179
39	28.5924	4.5750
40	28.4797	4.5066
41	28.4106	4.4988
42	27.5616	4.2424
43	27.1253	4.2205
44	26.8298	4.1775
45	26.8052	3.9649
46	26.5342	3.9647
47	24.1692	3.8903
48	23.4469	3.8723
49	23.4009	3.6630
50	23.2587	3.3487
51	23.0736	3.0923
52	22.9911	2.9044
53	22.0086	2.8662
54	21.8475	2.7801
55	21.8128	2.1580
56	21.2348	1.8727
57	19.8937	1.4330
58	19.2256	0.6277
59	17.9993	0.2481
60	17.8317	0.0000
61	15.4538	0.0000

Duration Flows

The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
16.3878	4552	0	0	Pass
17.1328	3974	0	0	Pass
17.8778	3454	0	0	Pass
18.6228	3018	0	0	Pass
19.3678	2620	0	0	Pass
20.1127	2289	0	0	Pass
20.8577	2032	0	0	Pass
21.6027	1805	0	0	Pass
22.3477	1606	0	0	Pass
23.0927	1417	0	0	Pass
23.8377	1257	0	0	Pass
24.5827	1102	0	0	Pass
25.3276	982	0	0	Pass
26.0726	859	0	0	Pass
26.8176	783	0	0	Pass
27.5626	698	0	0	Pass
28.3076	602	0	0	Pass
29.0526	520	0	0	Pass
29.7976	475	0	0	Pass
30.5425	419	0	0	Pass
31.2875	373	0	0	Pass
32.0325	328	0	0	Pass
32.7775	288	0	0	Pass
33.5225	261	0	0	Pass
34.2675	235	0	0	Pass
35.0125	210	0	0	Pass
35.7574	186	0	0	Pass
36.5024	169	0	0	Pass
37.2474	150	0	0	Pass
37.9924	134	0	0	Pass
38.7374	122	0	0	Pass
39.4824	108	0	0	Pass
40.2274	99	0	0	Pass
40.9723	86	0	0	Pass
41.7173	77	0	0	Pass
42.4623	69	0	0	Pass
43.2073	64	0	0	Pass
43.9523	60	0	0	Pass
44.6973	52	0	0	Pass
45.4423	50	0	0	Pass
46.1872	47	0	0	Pass
46.9322	42	0	0	Pass
47.6772	39	0	0	Pass
48.4222	36	0	0	Pass
49.1672	31	0	0	Pass
49.9122	28	0	0	Pass
50.6572	26	0	0	Pass
51.4021	24	0	0	Pass
52.1471	23	0	0	Pass
52.8921	22	0	0	Pass
53.6371	20	0	0	Pass
54.3821	20	0	0	Pass
55.1271	17	0	0	Pass

55.8721	15	0	0	Pass
56.6170	14	0	0	Pass
57.3620	14	0	0	Pass
58.1070	14	0	0	Pass
58.8520	13	0	0	Pass
59.5970	13	0	0	Pass
60.3420	12	0	0	Pass
61.0870	12	0	0	Pass
61.8319	12	0	0	Pass
62.5769	11	0	0	Pass
63.3219	9	0	0	Pass
64.0669	8	0	0	Pass
64.8119	8	0	0	Pass
65.5569	8	0	0	Pass
66.3019	8	0	0	Pass
67.0468	8	0	0	Pass
67.7918	8	0	0	Pass
68.5368	8	0	0	Pass
69.2818	8	0	0	Pass
70.0268	8	0	0	Pass
70.7718	6	0	0	Pass
71.5168	6	0	0	Pass
72.2617	6	0	0	Pass
73.0067	6	0	0	Pass
73.7517	6	0	0	Pass
74.4967	5	0	0	Pass
75.2417	5	0	0	Pass
75.9867	5	0	0	Pass
76.7317	5	0	0	Pass
77.4767	5	0	0	Pass
78.2216	5	0	0	Pass
78.9666	5	0	0	Pass
79.7116	4	0	0	Pass
80.4566	4	0	0	Pass
81.2016	4	0	0	Pass
81.9466	4	0	0	Pass
82.6916	4	0	0	Pass
83.4365	4	0	0	Pass
84.1815	4	0	0	Pass
84.9265	4	0	0	Pass
85.6715	4	0	0	Pass
86.4165	4	0	0	Pass
87.1615	3	0	0	Pass
87.9065	3	0	0	Pass
88.6514	2	0	0	Pass
89.3964	2	0	0	Pass
90.1414	2	0	0	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0 acre-feet
On-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.
Off-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Queen City Lake POC	☐	29242.24			☐	77.26			
Total Volume Infiltrated		29242.24	0.00	0.00		77.26	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

POC 2

POC #2 was not reported because POC must exist in both scenarios and both scenarios must have been run.

Model Default Modifications

Total of 0 changes have been made.

PERLND Changes

No PERLND changes have been made.

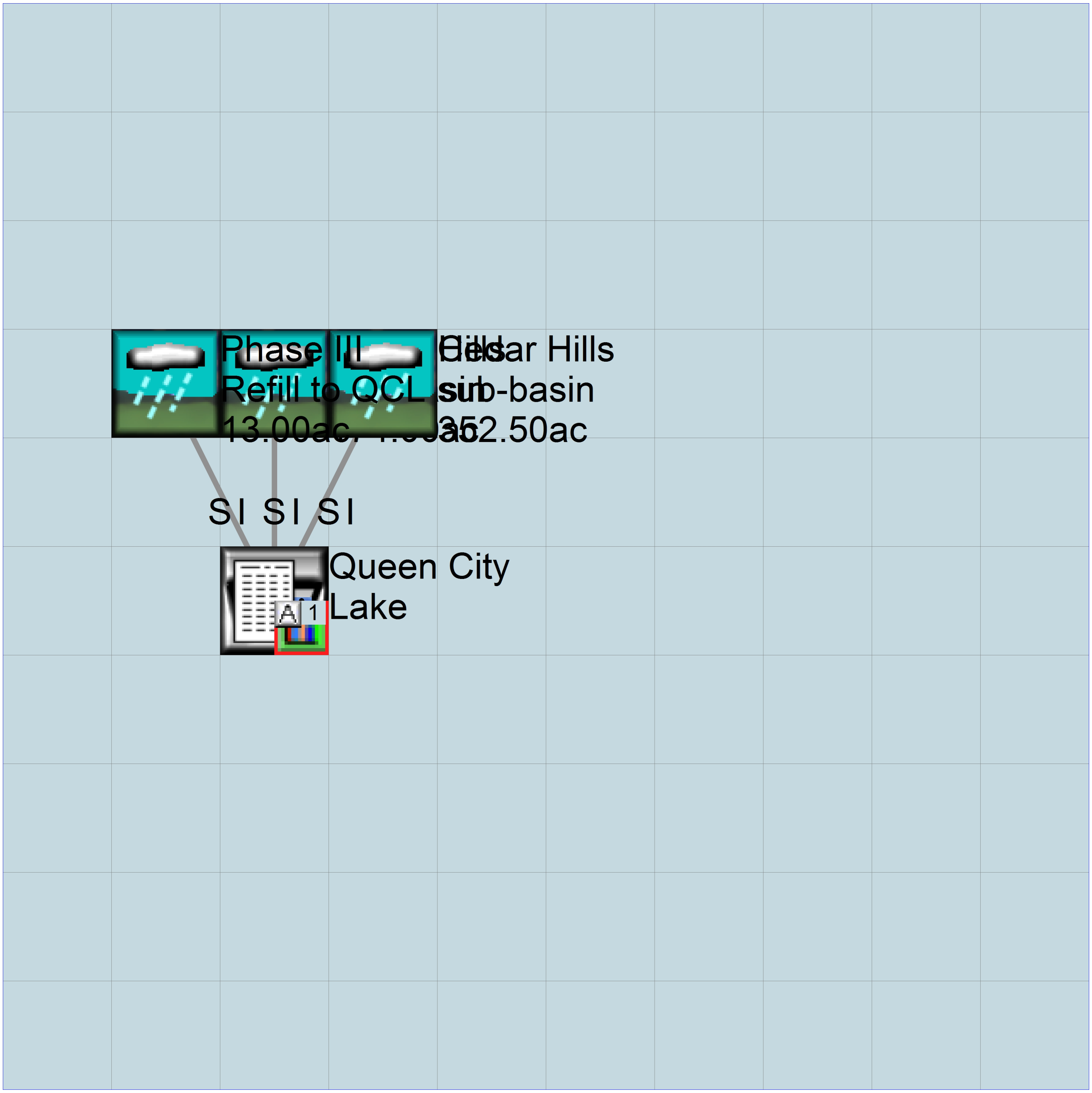
IMPLND Changes

No IMPLND changes have been made.

Appendix

Predeveloped Schematic

Mitigated Schematic



Predeveloped UCI File

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RUN

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END GLOBAL

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MESSU      25   PreQCF Phase III - Queen City Lake Outlet Sizing.MES
           27   PreQCF Phase III - Queen City Lake Outlet Sizing.L61
           28   PreQCF Phase III - Queen City Lake Outlet Sizing.L62
           30   POCQCF Phase III - Queen City Lake Outlet Sizing1.dat
END FILES

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      PERLND       2
      IMPLND       1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Cedar Hills dev sub-basin   MAX                   1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                            User  t-series Engl Metr ***
                                             in  out          ***
   11      C, Forest, Mod          1    1    1    1   27    0
   14      C, Pasture, Mod         1    1    1    1   27    0
    2      A/B, Forest, Mod        1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
   11         0    0    1    0    0    0    0    0    0    0    0    0
   14         0    0    1    0    0    0    0    0    0    0    0    0
    2         0    0    1    0    0    0    0    0    0    0    0    0
```

```
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
   11         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
   11         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
    2         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
   11              0       4.5      0.08       400       0.1       0.5     0.996
   14              0       4.5      0.06       400       0.1       0.5     0.996
    2              0         5         2       400       0.1       0.3     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
   11              0         0         2         2         0         0         0
   14              0         0         2         2         0         0         0
    2              0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
   11            0.2       0.5      0.35         6       0.5       0.7
   14           0.15       0.4       0.3         6       0.5       0.4
    2            0.2       0.5      0.35         0       0.7       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
   11              0         0         0         0       2.5         1         0
   14              0         0         0         0       2.5         1         0
    2              0         0         0         0         3         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name-------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                       in  out           ***
   14      POND                   1    1    1   27    0
    1      ROADS/FLAT             1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
   14         0    0    1    0    0    0
    1         0    0    1    0    0    0
  END ACTIVITY

  PRINT-INFO
```

```
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
   14         0    0    4    0    0    0    1    9
    1         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
   14         0    0    0    0    0
    1         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
   14            400      0.01       0.1       0.1
    1            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
   14              0         0
    1              0         0
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
   14              0         0
    1              0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Cedar Hills dev sub-basin***
PERLND  11                         333     COPY   501     12
PERLND  11                         333     COPY   501     13
PERLND  14                          11     COPY   501     12
PERLND  14                          11     COPY   501     13
IMPLND  14                         8.5     COPY   501     15
Maple Hills sub-basin***
PERLND   2                         128     COPY   501     12
PERLND   2                         128     COPY   501     13
PERLND  14                           8     COPY   501     12
PERLND  14                           8     COPY   501     13
IMPLND   1                          38     COPY   501     15
Phase III to QCL***
PERLND  14                          13     COPY   501     12
PERLND  14                          13     COPY   501     13

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
```

```
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                  ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG             ***
                                        in  out                               ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                        ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *    *  *  *  *  *       *  *  *  *  *        ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50        ***
  <------><--------><--------><--------><--------><--------><-------->        ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                          ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
         *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    501 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK       15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
```

```
  END MASS-LINK   15

END MASS-LINK

END RUN
```

Mitigated UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                             ***
WDM        26   QCF Phase III - Queen City Lake Outlet Sizing.wdm
MESSU      25   MitQCF Phase III - Queen City Lake Outlet Sizing.MES
           27   MitQCF Phase III - Queen City Lake Outlet Sizing.L61
           28   MitQCF Phase III - Queen City Lake Outlet Sizing.L62
           30   POCQCF Phase III - Queen City Lake Outlet Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      PERLND      14
      IMPLND       1
      PERLND      11
      IMPLND      14
      RCHRES       1
      COPY         1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Queen City Lake             MAX                    1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                          User  t-series Engl Metr ***
                                          in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
   11     C, Forest, Mod          1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
    2         0    0    1    0    0    0    0    0    0    0    0    0
```

```
  14         0    0    1    0    0    0    0    0    0    0    0    0
  11         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   11         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
   11         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2              0         5         2       400       0.1       0.3     0.996
   14              0       4.5      0.06       400       0.1       0.5     0.996
   11              0       4.5      0.08       400       0.1       0.5     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2              0         0         2         2         0         0         0
   14              0         0         2         2         0         0         0
   11              0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2            0.2       0.5      0.35         0       0.7       0.7
   14           0.15       0.4       0.3         6       0.5       0.4
   11            0.2       0.5      0.35         6       0.5       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2              0         0         0         0         3         1         0
   14              0         0         0         0       2.5         1         0
   11              0         0         0         0       2.5         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                      in  out           ***
    1      ROADS/FLAT             1    1    1   27    0
   14      POND                   1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
    1         0    0    1    0    0    0
   14         0    0    1    0    0    0
  END ACTIVITY
```

```
  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
    1         0    0    4    0    0    0    1    9
   14         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
    1         0    0    0    0    0
   14         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
    1            400      0.01       0.1       0.1
   14            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
    1              0         0
   14              0         0
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
    1              0         0
   14              0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Maple Hills sub-basin***
PERLND   2                         128     RCHRES   1      2
PERLND   2                         128     RCHRES   1      3
PERLND  14                           8     RCHRES   1      2
PERLND  14                           8     RCHRES   1      3
IMPLND   1                          38     RCHRES   1      5
Cedar Hills sub-basin***
PERLND  11                         333     RCHRES   1      2
PERLND  11                         333     RCHRES   1      3
PERLND  14                          11     RCHRES   1      2
PERLND  14                          11     RCHRES   1      3
IMPLND  14                         8.5     RCHRES   1      5
Phase III Refill to QCL***
PERLND  14                          13     RCHRES   1      2
PERLND  14                          13     RCHRES   1      3

******Routing******
PERLND   2                         128     COPY     1     12
PERLND  14                           8     COPY     1     12
IMPLND   1                          38     COPY     1     15
PERLND   2                         128     COPY     1     13
PERLND  14                           8     COPY     1     13
PERLND  11                         333     COPY     1     12
PERLND  14                          11     COPY     1     12
IMPLND  14                         8.5     COPY     1     15
PERLND  11                         333     COPY     1     13
PERLND  14                          11     COPY     1     13
PERLND  14                          13     COPY     1     12
PERLND  14                          13     COPY     1     13
```

```
RCHRES   1                           1     COPY   501     17
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                 ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                           in  out                           ***
    1     Queen City Lake         2    1    1    1   28    0    1
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
    1         1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
    1         4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                      ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *     *  *  *  *  *       *  *  *  *  *        ***
    1        0  1  0  0    4  5  0  0  0       0  0  0  0  0       2  2  2  2  2
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50       ***
  <------><--------><--------><--------><--------><--------><-------->       ***
    1            1      0.01       0.0       0.0       0.5       0.0
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                         ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
    1           0         4.0  5.0  0.0  0.0  0.0       0.0  0.0  0.0  0.0  0.0
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
  FTABLE      1
   12    5
     Depth      Area    Volume  Outflow1  Outflow2  Velocity  Travel Time***
      (ft)   (acres) (acre-ft)   (cfs)      (cfs)   (ft/sec)    (Minutes)***
  0.000000  0.000000  0.000000  0.000000  0.000000
  0.400000  0.006300  0.000800  0.000000  0.300000
  1.400000  1.870300  0.662700  0.000000  0.800000
  2.400000  4.091600  3.572100  0.000000  1.200000
  3.400000  5.584100  8.390600  0.000000  1.600000
  4.400000  7.107000  14.72080  0.000000  1.800000
```

```
  5.400000  8.532900  22.53000  2.471446  2.000000
  6.400000  9.937100  31.75610  4.623653  3.000000
  7.400000  11.73030  42.57740  6.053783  7.000000
  8.400000  13.12080  54.99650  16.33172  15.70000
  9.400000  14.53770  68.81970  41.95013  15.70000
  10.90000  15.18870  91.11270  58.09619  15.70000
  END FTABLE  1
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP
WDM      2 PREC     ENGL    1.167          RCHRES   1     EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           RCHRES   1     EXTNL  POTEV

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
RCHRES   1 HYDR   RO     1 1        1      WDM   1002 FLOW     ENGL      REPL
RCHRES   1 HYDR   O      1 1        1      WDM   1004 FLOW     ENGL      REPL
RCHRES   1 HYDR   O      2 1        1      WDM   1005 FLOW     ENGL      REPL
RCHRES   1 HYDR   STAGE  1 1        1      WDM   1003 STAG     ENGL      REPL
COPY     1 OUTPUT MEAN   1 1     48.4      WDM    701 FLOW     ENGL      REPL
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    801 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       2
PERLND     PWATER SURO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    2

  MASS-LINK       3
PERLND     PWATER IFWO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    3

  MASS-LINK       5
IMPLND     IWATER SURO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    5

  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK       15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   15

  MASS-LINK       17
RCHRES     OFLOW  OVOL   1                 COPY           INPUT  MEAN
  END MASS-LINK   17


END MASS-LINK

END RUN
```

Predeveloped HSPF Message File

Mitigated HSPF Message File

Disclaimer

Legal Notice

Clear Creek Solutions, Inc.
6200 Capitol Blvd. Ste F
Olympia, WA. 98501
Toll Free 1(866)943-0304
Local (360)943-0304

www.clearcreeksolutions.com

ATTACHMENT 1

Cedar Hills Regional Landfill – KCRTS Analysis Final Development Scenario Sub-Basin Boundaries Based on 2000 Topography

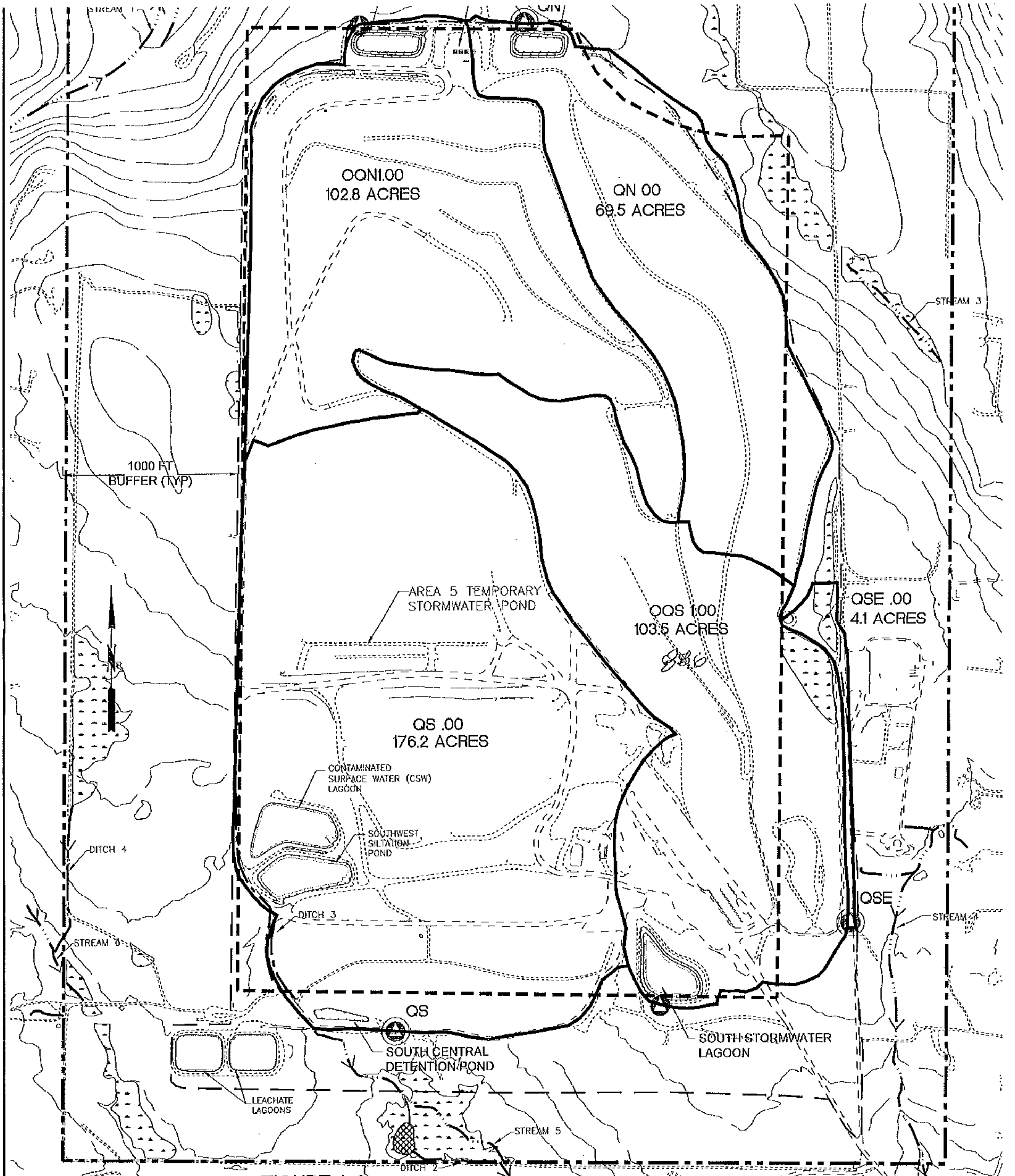


FIGURE 1-2

CEDAR HILLS REGIONAL LANDFILL

KCRTS ANALYSIS FINAL DEVELOPMENT SCENARIO

SUB-BASIN BOUNDARIES BASED ON 2000 TOPOGRAPHY

BUFFER EDGE

PROPERTY LINE

STUDY AREA

Approximate location for discharge point
See Appendix "B" for exact locations

Designated discharge point

NOTE:
AS THE LANDFILL IS CLOSED OUT (FINAL COVER ANTICIPATED 2013), SUB-BASIN BOUNDARIES MAY CHANGE SLIGHTLY. THE EFFECT WILL BE TO RE-ALLOCATE FLOWS BETWEEN SUB-BASINS. TOTAL DISCHARGE WILL NOT BE AFFECTED.

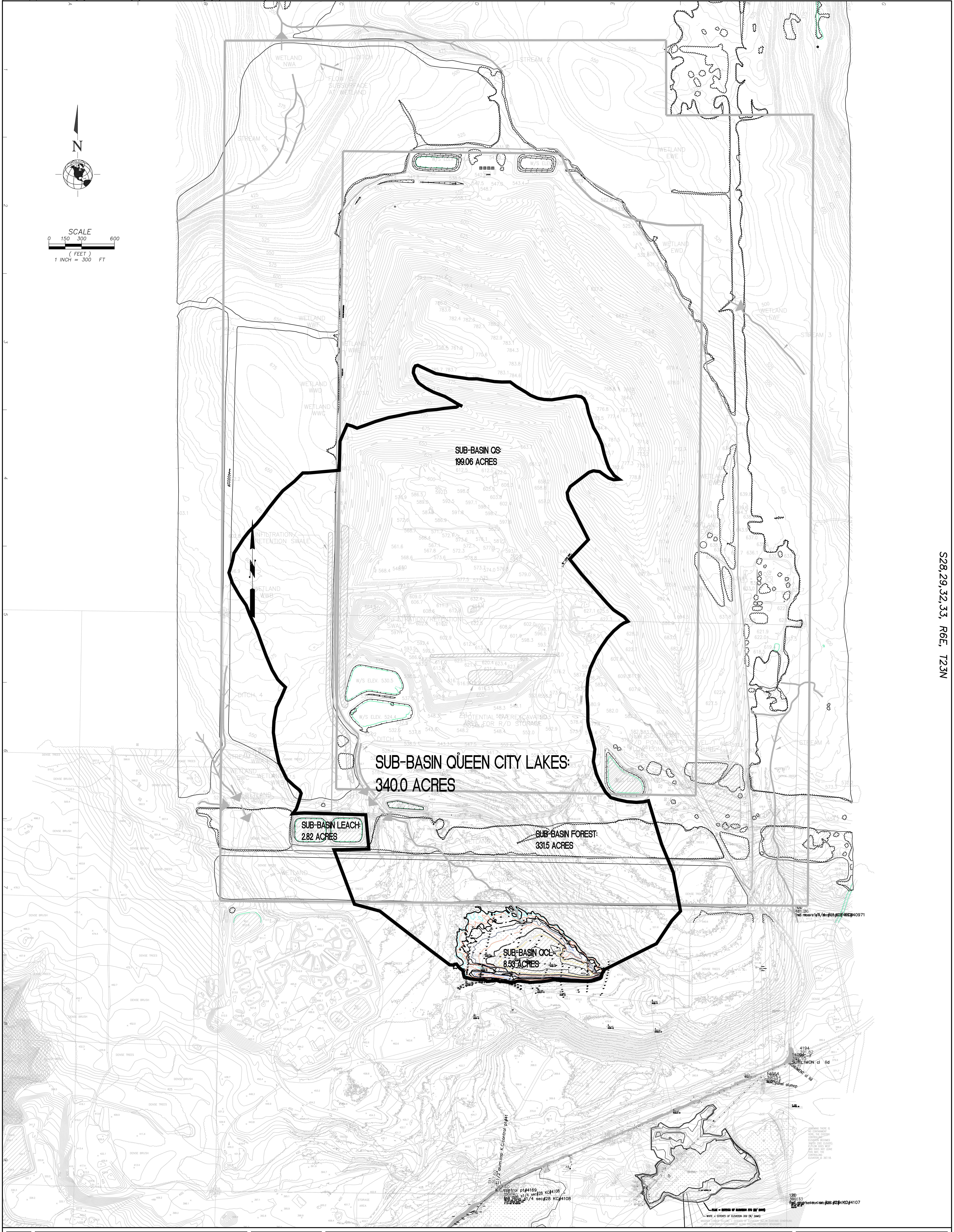
Department of Natural Resources and Parks

300 0 300 600 900
SCALE IN FEET

ATTACHMENT 2

Cedar Hills Regional Landfill 2000 Basin Topo – Queen City Lake

[Path: P:\Landau Associates\035144\Design\Drawings\Civil\035144-QCL-FIG-I1.dwg] [CHRL-2K-USE]
[Date: 12/5/2006 3:31 PM] [Author: MGomez] [Plotter: CutePDF Writer] [Style: Full Size.ctb]



WHP	WHP	CHECKED BY:	
DRAWN BY:	MAG	APPROVED BY:	
LAST EDIT:	12/5/2006	PLOT DATE:	12/05/06

DATE	BY	REV#	REVISION	CK'D	APPR

SHEET FIG I1

QUEEN CITY FARMS
CHRL 2000 BASIN TOPO
QUEEN CITY LAKE

KING COUNTY WA

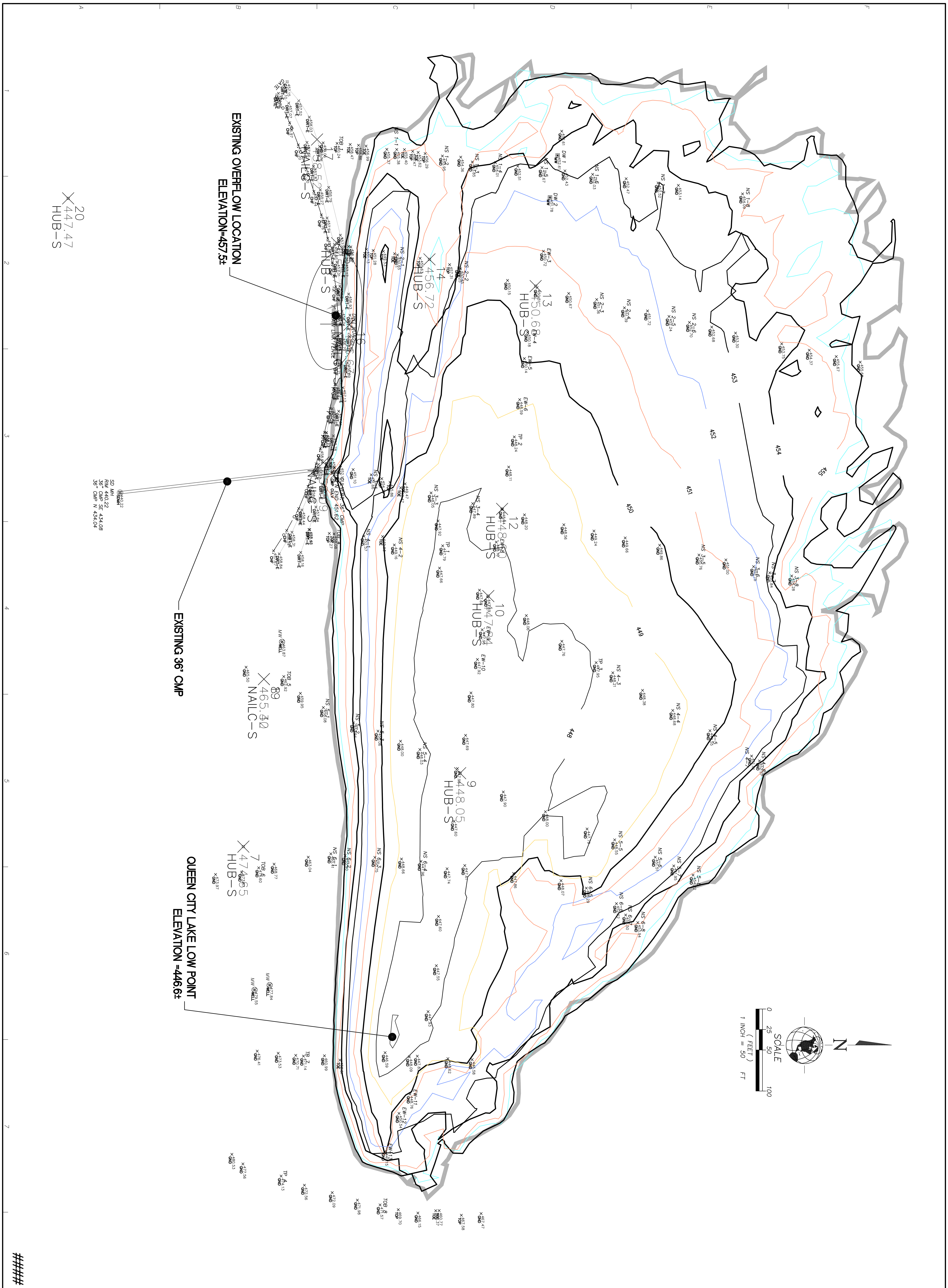
SCALE: 1"=300'
PROJECT NO. 035144
DRAWING FILE NAME: 035144-QCL-FIG-I1

W&H PACIFIC
3350 Monte Villa Parkway
Bothell, Washington 98021-8972
(425)951-4800
(425)951-4808
whpacific.com
Planners • Engineers • Surveyors • Landscape Architects

ATTACHMENT 3

Queen City Lake – Storage Routing and Water Level, Existing Conditions

P:\Landau Associates\035144\Design\Drawings\Civil\035144-QCL-FIG-A1.dwg, FIG A1, 12/5/2006 3:21:27 PM, MGomez



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WHP	MG/SS	CHECKED BY:	MG/SS
DRAWN BY:	MAG	APPROVED BY:	MAG
LAST EDIT:	12/4/2006	PLOT DATE:	12/04/06

DATE	BY	REV#	REVISION	CK'D	APPR

PRELIMINARY
MICHAEL A. GOMEZ
STATE OF WASHINGTON
REGISTERED PROFESSIONAL ENGINEER
EXPIRES 02/20/07

QUEEN CITY LAKE
STORAGE ROUTING AND WATER LEVEL EXISTING CONDITIONS

KING COUNTY WA

SCALE:
PROJECT NO. 035144
DRAWING FILE NAME: 035144-QCL-FIG-A1

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Bothell, Washington 98021-8972
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(425)951-4808
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SHEET 1/1
FIG A1

Objectives

1 Determine the dimensions of a proposed stilling (rip rap) basin at the Queen City Lake outlet where it enters the East stormwater basin.

1. Determine peak design flow rate

Approach Use the design procedure for the rip rap basin from Publication No. FHWA-NHI-06-086, July 2006, "Hydraulic Engineering Circular No. 14, Third Edition, Hydraulic Design of Energy Dissipators for Culverts and Channels."

Assumptions Rip rap D_{50} will be 6 inches
Rip rap D_{max} will be 12 inches
Inlet velocity is 23.8 fps - for 13.69 cfs design flow (100-yr) at 17% slope in a 24" culvert

Conversions 1 m = 3.28 ft

Calculation

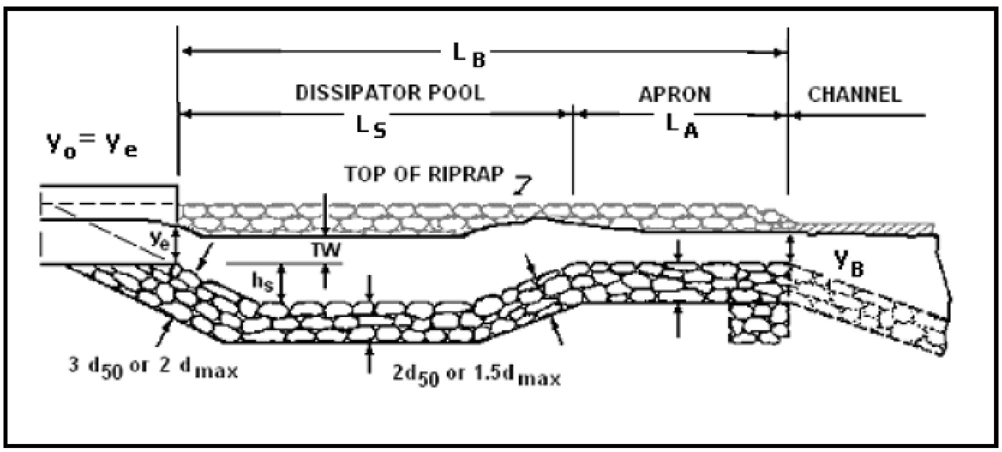


Figure 10.1. Profile of Riprap Basin

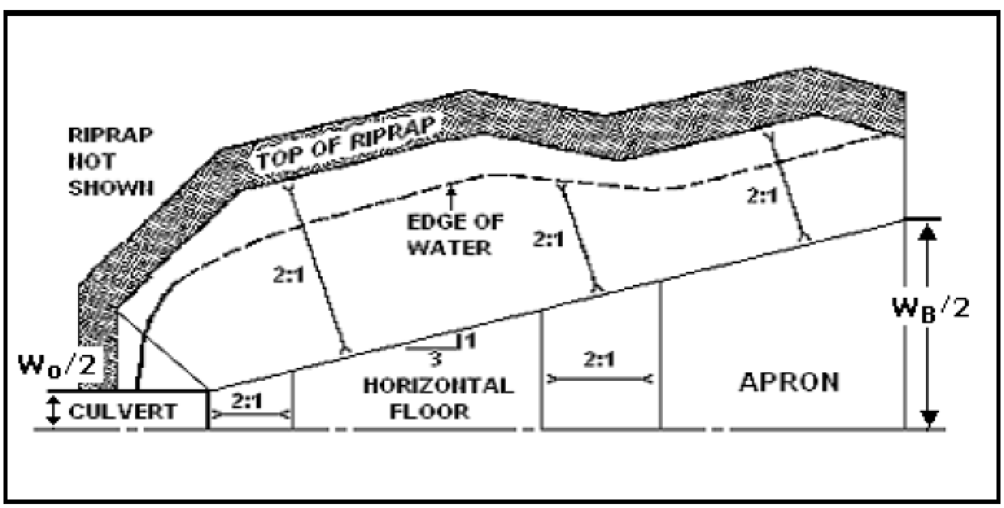


Figure 10.2. Half Plan of Riprap Basin

Given:

rip rap D50	0.50	ft	Assumption from above
riprap Dmax	1.00	ft	Assumption from above
culvert width, W0	3	ft	Assumption from above
brink depth, y0 or ye	0.5	ft	maximum 100-year water level in pipe at 17% slope (see QCL Outlet Sizing calcs)
tailwater, TW	1.0	ft	set by max water level in East Basin
outlet velocity, V0	23.8	ft/s	100-year discharge from QCL

to metric units:

riprap D50	0.15	m
riprap Dmax	0.30	m
culvert width, W0	0.91	m
brink depth, y0 or ye	0.15	m
tailwater, TW	0.30	m
outlet velocity, V0	7.26	m/s
gravitational acceleration, g	9.81	m^2/s

Calculate:

TW/ye	2.08		
C0	2.40		based on TW/ye > 0.75
D50/ye	1.04		should be > 0.1
sqrt(g*ye)	1.20		
hs/ye	2.69		
hs/D50	2.58		Should be > 2
scour depth, hs	1.29	ft	
length of pool, Ls	13	ft	should be > 3*W0
3*W0	9	ft	
length of apron, La	6	ft	should be > W0
length of basin, Lb	19	ft	should be > 4*W0
width at basin exit, Wb	16	ft	
side slope length	2.58	ft	based on 2:1 slope

Conclusion

A rip rap basin with D50 of 6 inches and Dmax of 12 inches can be used for energy dissipation at the 24" Queen City Lake culvert outlet. The basin length will be 27 feet (minimum), and it will be 21 feet wide (minimum) at the widest point. The rip rap basin depth will be 1.3 feet (minimum). Riprap thickness will be min. 18" at the headwall and 12" for the remainder of the basin.

LANDAU ASSOCIATES

JOB NO.	0992002.050.051		
JOB NAME	Queen City Farms - Phase III Refill		
SUBJECT	East Pond Sizing		
CALC BY	MDV	DATE	8/20/2019
CHK BY	KMS	DATE	8/27/2019

Objective Determine the minimum East Pond footprint area to provide adequate retention and infiltration for Queen City Lake overflow.

Approach Use WWHM to find the minimum bottom footprint of the East Pond to achieve 100% infiltration of the runoff (i.e. Queen City Lake outlet pipe discharge), by varying the bottom length and keeping all other parameters fixed.

Assumptions *Parameters used in WWHM:*

Element	Trapezoidal Pond with Infiltration		
Facility Bottom Elevation	386	ft	
Bottom Length	300	ft	(variable)
Bottom Width	100	ft	
Bottom Pond Area	30,000	sf	
	0.7	ac	

NOTE: Bottom length and width are approximate. Total bottom pond area is the same.

Effective Depth	4	ft

NOTE: Includes maximum water depth (3 ft) plus freeboard (1 ft)

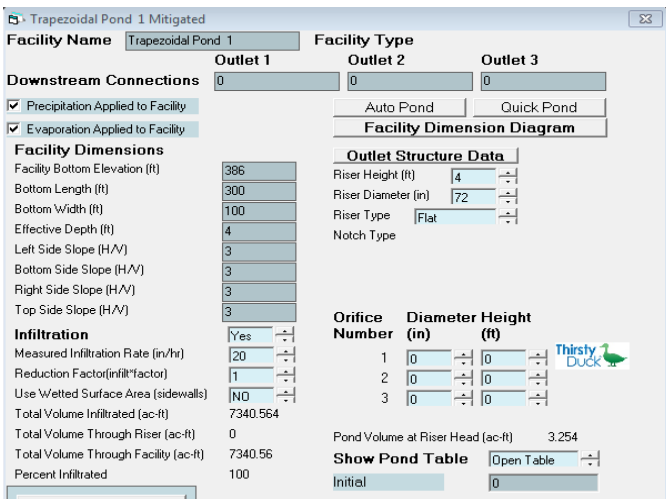
Slopes (H/V)	3	
Measured Infiltration Rate	20	in/hr
Riser Height	4	ft

NOTE: Corresponds to level of overflow spillway at top of berm

Riser Diameter	72	in

NOTE: Corresponds to width (6 ft) of overflow spillway at top of berm

Solution



Conclusion A 0.7-acre infiltration pond with a 3-foot effective storage depth (1 foot freeboard) would provide retention and 100% infiltration for the modeled Queen City Lake outflows. The infiltration pond passes the Stream Protection and LID Duration standards, as well as the Flow Control standard (developed/mitigated flows less than the predeveloped flows). WWHM results are attached.

WWHM2012

PROJECT REPORT

General Model Information

Project Name:	QCF Phase III - East Pond Sizing
Site Name:	Queen City Farms
Site Address:	17825 Cedar Grove Rd SE
City:	Maple Valley
Report Date:	8/26/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.167
Version Date:	2018/10/10
Version:	4.2.16

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Cedar Hills dev sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	333
C, Pasture, Mod	11
Pervious Total	344
Impervious Land Use	acre
POND	8.5
Impervious Total	8.5
Basin Total	352.5

Element Flows To:

Surface	Interflow	Groundwater

Maple Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
C, Pasture, Mod	8
Pervious Total	136
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	174

Element Flows To:

Surface	Interflow	Groundwater

Phase III to QCL

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	13
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Maple Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
C, Pasture, Mod	8
Pervious Total	136
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	174

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Cedar Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	333
C, Pasture, Mod	11
Pervious Total	344
Impervious Land Use	acre
POND	8.5
Impervious Total	8.5
Basin Total	352.5

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Phase III Refill to QCL

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	13
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Routing Elements
Predeveloped Routing

Mitigated Routing

Queen City Lake

Depth: 10.9 ft.
Discharge Structure: 1
Riser Height: 7.96 ft.
Riser Diameter: 36 in.
Orifice 1 Diameter: 12 in. Elevation:5 ft.
Element Flows To:
Outlet 1 Outlet 2
Trapezoidal Pond 1

SSD Table Hydraulic Table

Stage (feet)	**Area (ac.)**	**Volume (ac-ft.)**	**Outlet Struct**	**Infilt (cfs)**	**NotUsed**	**NotUsed**	**NotUsed**
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.400	0.006	0.001	0.000	0.300	0.000	0.000	0.000
1.400	1.870	0.663	0.000	0.800	0.000	0.000	0.000
2.400	4.092	3.572	0.000	1.200	0.000	0.000	0.000
3.400	5.584	8.391	0.000	1.600	0.000	0.000	0.000
4.400	7.107	14.72	0.000	1.800	0.000	0.000	0.000
5.400	8.533	22.53	2.471	2.000	0.000	0.000	0.000
6.400	9.937	31.76	4.624	3.000	0.000	0.000	0.000
7.400	11.73	42.58	6.054	7.000	0.000	0.000	0.000
8.400	13.12	55.00	16.33	15.70	0.000	0.000	0.000
9.400	14.54	68.82	41.95	15.70	0.000	0.000	0.000
10.90	15.19	91.11	58.10	15.70	0.000	0.000	0.000

Trapezoidal Pond 1

Bottom Length:	300.00 ft.	
Bottom Width:	100.00 ft.	
Depth:	4 ft.	
Volume at riser head:	3.2543 acre-feet.	
Infiltration On		
Infiltration rate:	20	
Infiltration safety factor:	1	
Total Volume Infiltrated (ac-ft.):		7340.564
Total Volume Through Riser (ac-ft.):		0
Total Volume Through Facility (ac-ft.):		7340.564
Percent Infiltrated:		100
Total Precip Applied to Facility:		34.083
Total Evap From Facility:		2.499
Side slope 1:	3 To 1	
Side slope 2:	3 To 1	
Side slope 3:	3 To 1	
Side slope 4:	3 To 1	
Discharge Structure		
Riser Height:	4 ft.	
Riser Diameter:	72 in.	
Element Flows To:		
Outlet 1	Outlet 2	

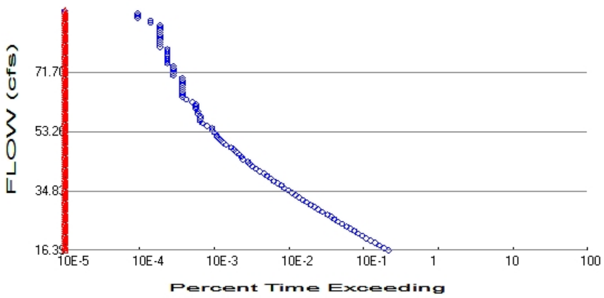
Pond Hydraulic Table

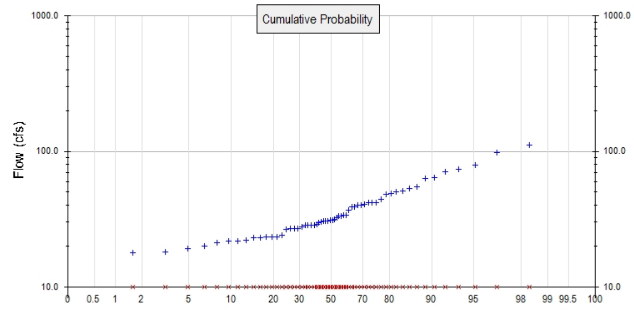
Stage(feet)	Area(ac.)	Volume(ac-ft.)	Discharge(cfs)	Infilt(cfs)
386.00	0.688	0.000	0.000	0.000
386.04	0.691	0.030	0.000	13.88
386.09	0.693	0.061	0.000	13.88
386.13	0.696	0.092	0.000	13.88
386.18	0.698	0.123	0.000	13.88
386.22	0.701	0.154	0.000	13.88
386.27	0.703	0.185	0.000	13.88
386.31	0.705	0.216	0.000	13.88
386.36	0.708	0.248	0.000	13.88
386.40	0.710	0.279	0.000	13.88
386.44	0.713	0.311	0.000	13.88
386.49	0.715	0.343	0.000	13.88
386.53	0.718	0.375	0.000	13.88
386.58	0.720	0.407	0.000	13.88
386.62	0.723	0.439	0.000	13.88
386.67	0.725	0.471	0.000	13.88
386.71	0.728	0.503	0.000	13.88
386.76	0.730	0.536	0.000	13.88
386.80	0.733	0.568	0.000	13.88
386.84	0.735	0.601	0.000	13.88
386.89	0.738	0.634	0.000	13.88
386.93	0.740	0.667	0.000	13.88
386.98	0.743	0.700	0.000	13.88
387.02	0.745	0.733	0.000	13.88
387.07	0.748	0.766	0.000	13.88
387.11	0.750	0.799	0.000	13.88
387.16	0.753	0.833	0.000	13.88
387.20	0.756	0.866	0.000	13.88
387.24	0.758	0.900	0.000	13.88

387.29	0.761	0.934	0.000	13.88
387.33	0.763	0.967	0.000	13.88
387.38	0.766	1.001	0.000	13.88
387.42	0.768	1.036	0.000	13.88
387.47	0.771	1.070	0.000	13.88
387.51	0.773	1.104	0.000	13.88
387.56	0.776	1.139	0.000	13.88
387.60	0.779	1.173	0.000	13.88
387.64	0.781	1.208	0.000	13.88
387.69	0.784	1.243	0.000	13.88
387.73	0.786	1.278	0.000	13.88
387.78	0.789	1.313	0.000	13.88
387.82	0.791	1.348	0.000	13.88
387.87	0.794	1.383	0.000	13.88
387.91	0.797	1.418	0.000	13.88
387.96	0.799	1.454	0.000	13.88
388.00	0.802	1.489	0.000	13.88
388.04	0.804	1.525	0.000	13.88
388.09	0.807	1.561	0.000	13.88
388.13	0.810	1.597	0.000	13.88
388.18	0.812	1.633	0.000	13.88
388.22	0.815	1.669	0.000	13.88
388.27	0.817	1.705	0.000	13.88
388.31	0.820	1.742	0.000	13.88
388.36	0.823	1.778	0.000	13.88
388.40	0.825	1.815	0.000	13.88
388.44	0.828	1.852	0.000	13.88
388.49	0.831	1.889	0.000	13.88
388.53	0.833	1.926	0.000	13.88
388.58	0.836	1.963	0.000	13.88
388.62	0.838	2.000	0.000	13.88
388.67	0.841	2.037	0.000	13.88
388.71	0.844	2.075	0.000	13.88
388.76	0.846	2.112	0.000	13.88
388.80	0.849	2.150	0.000	13.88
388.84	0.852	2.188	0.000	13.88
388.89	0.854	2.226	0.000	13.88
388.93	0.857	2.264	0.000	13.88
388.98	0.860	2.302	0.000	13.88
389.02	0.862	2.340	0.000	13.88
389.07	0.865	2.379	0.000	13.88
389.11	0.868	2.417	0.000	13.88
389.16	0.870	2.456	0.000	13.88
389.20	0.873	2.495	0.000	13.88
389.24	0.876	2.533	0.000	13.88
389.29	0.878	2.572	0.000	13.88
389.33	0.881	2.612	0.000	13.88
389.38	0.884	2.651	0.000	13.88
389.42	0.886	2.690	0.000	13.88
389.47	0.889	2.730	0.000	13.88
389.51	0.892	2.769	0.000	13.88
389.56	0.895	2.809	0.000	13.88
389.60	0.897	2.849	0.000	13.88
389.64	0.900	2.889	0.000	13.88
389.69	0.903	2.929	0.000	13.88
389.73	0.905	2.969	0.000	13.88
389.78	0.908	3.009	0.000	13.88
389.82	0.911	3.050	0.000	13.88

389.87	0.914	3.090	0.000	13.88
389.91	0.916	3.131	0.000	13.88
389.96	0.919	3.172	0.000	13.88
390.00	0.922	3.213	0.000	13.88
390.04	0.925	3.254	0.596	13.88

Analysis Results
POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 493
Total Impervious Area: 46.5

Mitigated Landuse Totals for POC #1
Total Pervious Area: 493
Total Impervious Area: 46.5

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	32.775625
5 year	47.768722
10 year	59.378734
25 year	76.101607
50 year	90.141409
100 year	105.616188

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0
5 year	0
10 year	0
25 year	0
50 year	0
100 year	0

Annual Peaks
Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	50.858	0.000
1950	48.944	0.000
1951	40.610	0.000
1952	22.991	0.000
1953	19.226	0.000
1954	26.830	0.000
1955	31.257	0.000
1956	30.418	0.000
1957	37.020	0.000
1958	23.401	0.000

1959	21.848	0.000
1960	41.900	0.000
1961	29.078	0.000
1962	17.999	0.000
1963	28.627	0.000
1964	28.480	0.000
1965	33.299	0.000
1966	23.074	0.000
1967	50.189	0.000
1968	32.405	0.000
1969	28.592	0.000
1970	30.871	0.000
1971	33.688	0.000
1972	41.832	0.000
1973	21.813	0.000
1974	33.485	0.000
1975	40.273	0.000
1976	30.365	0.000
1977	21.235	0.000
1978	27.562	0.000
1979	30.632	0.000
1980	70.258	0.000
1981	29.868	0.000
1982	54.614	0.000
1983	26.534	0.000
1984	22.009	0.000
1985	23.259	0.000
1986	38.988	0.000
1987	39.060	0.000
1988	17.832	0.000
1989	24.169	0.000
1990	111.627	0.000
1991	63.831	0.000
1992	27.125	0.000
1993	19.894	0.000
1994	15.454	0.000
1995	26.805	0.000
1996	63.304	0.000
1997	41.862	0.000
1998	28.411	0.000
1999	73.944	0.000
2000	30.697	0.000
2001	23.447	0.000
2002	40.328	0.000
2003	48.488	0.000
2004	53.182	0.000
2005	33.648	0.000
2006	31.518	0.000
2007	98.549	0.000
2008	79.507	0.000
2009	44.157	0.000

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	111.6270	0.0000
2	98.5492	0.0000
3	79.5073	0.0000

4	73.9443	0.0000
5	70.2581	0.0000
6	63.8307	0.0000
7	63.3038	0.0000
8	54.6140	0.0000
9	53.1816	0.0000
10	50.8580	0.0000
11	50.1890	0.0000
12	48.9439	0.0000
13	48.4878	0.0000
14	44.1568	0.0000
15	41.9000	0.0000
16	41.8620	0.0000
17	41.8315	0.0000
18	40.6099	0.0000
19	40.3283	0.0000
20	40.2733	0.0000
21	39.0604	0.0000
22	38.9884	0.0000
23	37.0198	0.0000
24	33.6880	0.0000
25	33.6479	0.0000
26	33.4852	0.0000
27	33.2994	0.0000
28	32.4045	0.0000
29	31.5179	0.0000
30	31.2571	0.0000
31	30.8712	0.0000
32	30.6971	0.0000
33	30.6323	0.0000
34	30.4175	0.0000
35	30.3647	0.0000
36	29.8675	0.0000
37	29.0776	0.0000
38	28.6270	0.0000
39	28.5924	0.0000
40	28.4797	0.0000
41	28.4106	0.0000
42	27.5616	0.0000
43	27.1253	0.0000
44	26.8298	0.0000
45	26.8052	0.0000
46	26.5342	0.0000
47	24.1692	0.0000
48	23.4469	0.0000
49	23.4009	0.0000
50	23.2587	0.0000
51	23.0736	0.0000
52	22.9911	0.0000
53	22.0086	0.0000
54	21.8475	0.0000
55	21.8128	0.0000
56	21.2348	0.0000
57	19.8937	0.0000
58	19.2256	0.0000
59	17.9993	0.0000
60	17.8317	0.0000
61	15.4538	0.0000

Duration Flows
The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
16.3878	4552	0	0	Pass
17.1328	3974	0	0	Pass
17.8778	3454	0	0	Pass
18.6228	3018	0	0	Pass
19.3678	2620	0	0	Pass
20.1127	2289	0	0	Pass
20.8577	2032	0	0	Pass
21.6027	1805	0	0	Pass
22.3477	1606	0	0	Pass
23.0927	1417	0	0	Pass
23.8377	1257	0	0	Pass
24.5827	1102	0	0	Pass
25.3276	982	0	0	Pass
26.0726	859	0	0	Pass
26.8176	783	0	0	Pass
27.5626	698	0	0	Pass
28.3076	602	0	0	Pass
29.0526	520	0	0	Pass
29.7976	475	0	0	Pass
30.5425	419	0	0	Pass
31.2875	373	0	0	Pass
32.0325	328	0	0	Pass
32.7775	288	0	0	Pass
33.5225	261	0	0	Pass
34.2675	235	0	0	Pass
35.0125	210	0	0	Pass
35.7574	186	0	0	Pass
36.5024	169	0	0	Pass
37.2474	150	0	0	Pass
37.9924	134	0	0	Pass
38.7374	122	0	0	Pass
39.4824	108	0	0	Pass
40.2274	99	0	0	Pass
40.9723	86	0	0	Pass
41.7173	77	0	0	Pass
42.4623	69	0	0	Pass
43.2073	64	0	0	Pass
43.9523	60	0	0	Pass
44.6973	52	0	0	Pass
45.4423	50	0	0	Pass
46.1872	47	0	0	Pass
46.9322	42	0	0	Pass
47.6772	39	0	0	Pass
48.4222	36	0	0	Pass
49.1672	31	0	0	Pass
49.9122	28	0	0	Pass
50.6572	26	0	0	Pass
51.4021	24	0	0	Pass
52.1471	23	0	0	Pass
52.8921	22	0	0	Pass
53.6371	20	0	0	Pass
54.3821	20	0	0	Pass
55.1271	17	0	0	Pass

55.8721	15	0	0	Pass
56.6170	14	0	0	Pass
57.3620	14	0	0	Pass
58.1070	14	0	0	Pass
58.8520	13	0	0	Pass
59.5970	13	0	0	Pass
60.3420	12	0	0	Pass
61.0870	12	0	0	Pass
61.8319	12	0	0	Pass
62.5769	11	0	0	Pass
63.3219	9	0	0	Pass
64.0669	8	0	0	Pass
64.8119	8	0	0	Pass
65.5569	8	0	0	Pass
66.3019	8	0	0	Pass
67.0468	8	0	0	Pass
67.7918	8	0	0	Pass
68.5368	8	0	0	Pass
69.2818	8	0	0	Pass
70.0268	8	0	0	Pass
70.7718	6	0	0	Pass
71.5168	6	0	0	Pass
72.2617	6	0	0	Pass
73.0067	6	0	0	Pass
73.7517	6	0	0	Pass
74.4967	5	0	0	Pass
75.2417	5	0	0	Pass
75.9867	5	0	0	Pass
76.7317	5	0	0	Pass
77.4767	5	0	0	Pass
78.2216	5	0	0	Pass
78.9666	5	0	0	Pass
79.7116	4	0	0	Pass
80.4566	4	0	0	Pass
81.2016	4	0	0	Pass
81.9466	4	0	0	Pass
82.6916	4	0	0	Pass
83.4365	4	0	0	Pass
84.1815	4	0	0	Pass
84.9265	4	0	0	Pass
85.6715	4	0	0	Pass
86.4165	4	0	0	Pass
87.1615	3	0	0	Pass
87.9065	3	0	0	Pass
88.6514	2	0	0	Pass
89.3964	2	0	0	Pass
90.1414	2	0	0	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0 acre-feet
On-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.
Off-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Trapezoidal Pond 1 POC	☐	6679.91			☐	100.00			
Queen City Lake	☐	29242.24			☐	77.26			
Total Volume Infiltrated		35922.15	0.00	0.00		81.49	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

POC 2

POC #2 was not reported because POC must exist in both scenarios and both scenarios must have been run.

Model Default Modifications

Total of 0 changes have been made.

PERLND Changes

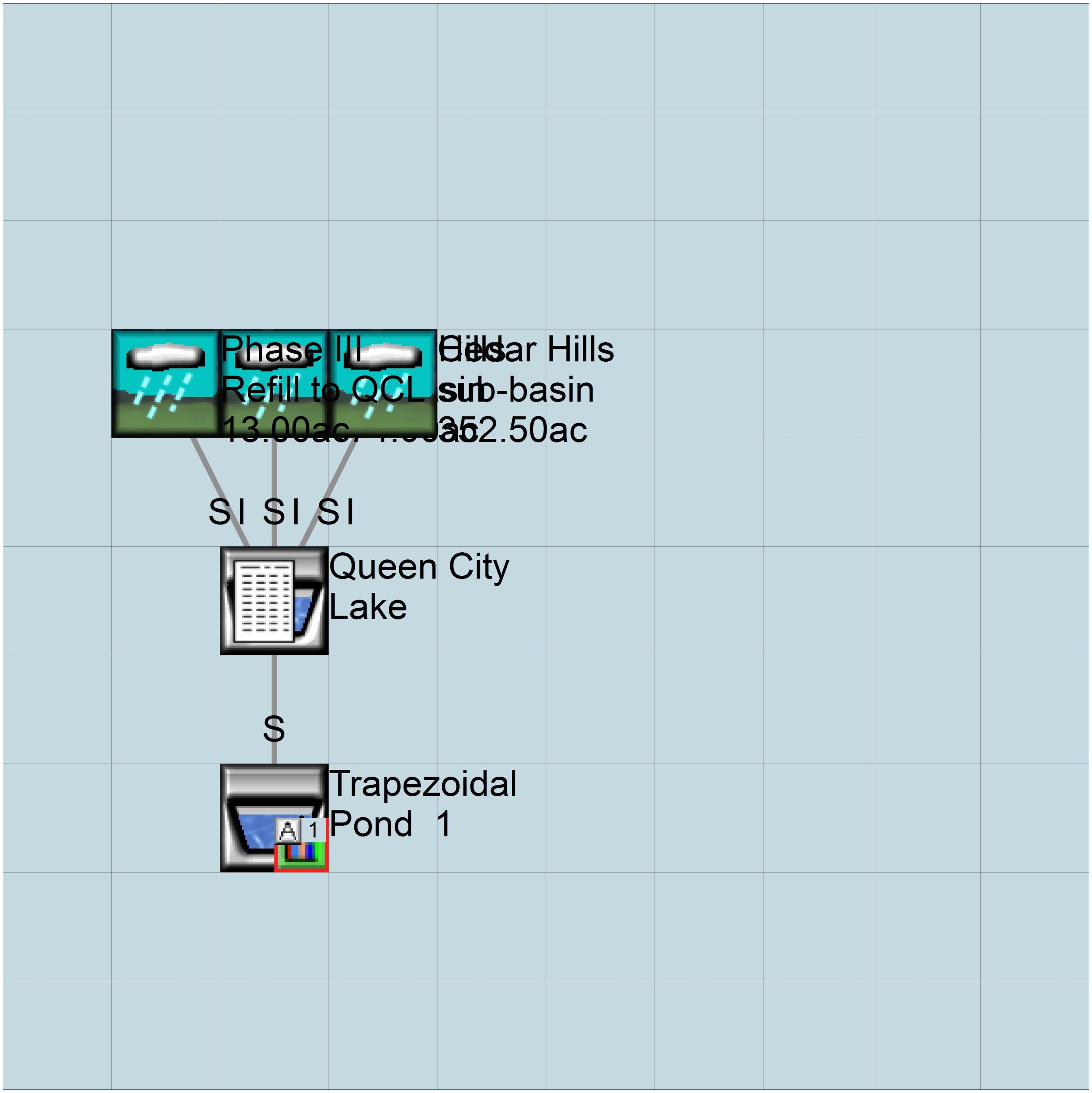
No PERLND changes have been made.

IMPLND Changes

No IMPLND changes have been made.

Appendix
Predeveloped Schematic

Mitigated Schematic



Predeveloped UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                              ***
WDM        26   QCF Phase III - East Pond Sizing.wdm
MESSU      25   PreQCF Phase III - East Pond Sizing.MES
           27   PreQCF Phase III - East Pond Sizing.L61
           28   PreQCF Phase III - East Pond Sizing.L62
           30   POCQCF Phase III - East Pond Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND      11
      PERLND      14
      IMPLND      14
      PERLND       2
      IMPLND       1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Cedar Hills dev sub-basin   MAX                   1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                          User  t-series Engl Metr ***
                                          in  out           ***
   11     C, Forest, Mod          1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
    2     A/B, Forest, Mod        1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
   11         0    0    1    0    0    0    0    0    0    0    0    0
   14         0    0    1    0    0    0    0    0    0    0    0    0
    2         0    0    1    0    0    0    0    0    0    0    0    0
```

```
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
   11         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
   11         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
    2         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
   11              0       4.5      0.08       400       0.1       0.5     0.996
   14              0       4.5      0.06       400       0.1       0.5     0.996
    2              0         5         2       400       0.1       0.3     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
   11              0         0         2         2         0         0         0
   14              0         0         2         2         0         0         0
    2              0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
   11            0.2       0.5      0.35         6       0.5       0.7
   14           0.15       0.4       0.3         6       0.5       0.4
    2            0.2       0.5      0.35         0       0.7       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
   11              0         0         0         0       2.5         1         0
   14              0         0         0         0       2.5         1         0
    2              0         0         0         0         3         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                       in  out           ***
   14      POND                   1    1    1   27    0
    1      ROADS/FLAT             1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
   14         0    0    1    0    0    0
    1         0    0    1    0    0    0
  END ACTIVITY

  PRINT-INFO
```

```
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
   14         0    0    4    0    0    0    1    9
    1         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
   14         0    0    0    0    0
    1         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
   14            400      0.01       0.1       0.1
    1            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
   14              0         0
    1              0         0
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
   14              0         0
    1              0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Cedar Hills dev sub-basin***
PERLND  11                         333     COPY   501     12
PERLND  11                         333     COPY   501     13
PERLND  14                          11     COPY   501     12
PERLND  14                          11     COPY   501     13
IMPLND  14                         8.5     COPY   501     15
Maple Hills sub-basin***
PERLND   2                         128     COPY   501     12
PERLND   2                         128     COPY   501     13
PERLND  14                           8     COPY   501     12
PERLND  14                           8     COPY   501     13
IMPLND   1                          38     COPY   501     15
Phase III to QCL***
PERLND  14                          13     COPY   501     12
PERLND  14                          13     COPY   501     13

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
```

```
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                   ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG              ***
                                       in  out                                ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                       ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *    *  *  *  *  *       *  *  *  *  *          ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50        ***
  <------><--------><--------><--------><--------><--------><-------->        ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                          ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    501 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK       15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
```

```
  END MASS-LINK   15

END MASS-LINK

END RUN
```

Mitigated UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                              ***
WDM        26   QCF Phase III - East Pond Sizing.wdm
MESSU      25   MitQCF Phase III - East Pond Sizing.MES
           27   MitQCF Phase III - East Pond Sizing.L61
           28   MitQCF Phase III - East Pond Sizing.L62
           30   POCQCF Phase III - East Pond Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      PERLND      14
      IMPLND       1
      PERLND      11
      IMPLND      14
      RCHRES       1
      RCHRES       2
      COPY         1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Trapezoidal Pond  1         MAX                    1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                          User  t-series Engl Metr ***
                                           in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
   11     C, Forest, Mod          1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
```

```
    2         0    0    1    0    0    0    0    0    0    0    0    0
   14         0    0    1    0    0    0    0    0    0    0    0    0
   11         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   11         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
   11         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2              0         5         2       400       0.1       0.3     0.996
   14              0       4.5      0.06       400       0.1       0.5     0.996
   11              0       4.5      0.08       400       0.1       0.5     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2              0         0         2         2         0         0         0
   14              0         0         2         2         0         0         0
   11              0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2            0.2       0.5      0.35         0       0.7       0.7
   14           0.15       0.4       0.3         6       0.5       0.4
   11            0.2       0.5      0.35         6       0.5       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2              0         0         0         0         3         1         0
   14              0         0         0         0       2.5         1         0
   11              0         0         0         0       2.5         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                       in  out          ***
    1      ROADS/FLAT             1    1    1   27    0
   14      POND                   1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
    1         0    0    1    0    0    0
   14         0    0    1    0    0    0
```

```
  END ACTIVITY

  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
    1         0    0    4    0    0    0    1    9
   14         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
    1         0    0    0    0    0
   14         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
    1            400      0.01       0.1       0.1
   14            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
    1             0         0
   14             0         0
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
    1             0         0
   14             0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Maple Hills sub-basin***
PERLND   2                         128     RCHRES   1      2
PERLND   2                         128     RCHRES   1      3
PERLND  14                           8     RCHRES   1      2
PERLND  14                           8     RCHRES   1      3
IMPLND   1                          38     RCHRES   1      5
Cedar Hills sub-basin***
PERLND  11                         333     RCHRES   1      2
PERLND  11                         333     RCHRES   1      3
PERLND  14                          11     RCHRES   1      2
PERLND  14                          11     RCHRES   1      3
IMPLND  14                         8.5     RCHRES   1      5
Phase III Refill to QCL***
PERLND  14                          13     RCHRES   1      2
PERLND  14                          13     RCHRES   1      3

******Routing******
RCHRES   1                           1     RCHRES   2      7
RCHRES   1                                 COPY     1     17
RCHRES   2                           1     COPY   501     17
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1
```

```
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                  ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                         in  out                              ***
    1     Queen City Lake         2    1    1    1   28    0    1
    2     Trapezoidal Pond-019    2    1    1    1   28    0    1
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
    1         1    0    0    0    0    0    0    0    0    0
    2         1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
    1         4    0    0    0    0    0    0    0    0    0    1    9
    2         4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                       ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *    *  *  *  *  *       *  *  *  *  *        ***
    1        0  1  0  0    4  5  0  0  0       0  0  0  0  0       2  2  2  2  2
    2        0  1  0  0    4  5  0  0  0       0  0  0  0  0       2  2  2  2  2
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50        ***
  <------><--------><--------><--------><--------><--------><-------->        ***
    1             1      0.01       0.0       0.0       0.5       0.0
    2             2      0.06       0.0     386.0       0.5       0.0
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                          ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
    1          0          4.0  5.0  0.0  0.0  0.0       0.0  0.0  0.0  0.0  0.0
    2          0          4.0  5.0  0.0  0.0  0.0       0.0  0.0  0.0  0.0  0.0
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
  FTABLE      1
   12    5
     Depth      Area    Volume  Outflow1  Outflow2  Velocity  Travel Time***
      (ft)   (acres) (acre-ft)   (cfs)     (cfs)    (ft/sec)    (Minutes)***
  0.000000  0.000000  0.000000  0.000000  0.000000
  0.400000  0.006300  0.000800  0.000000  0.300000
  1.400000  1.870300  0.662700  0.000000  0.800000
  2.400000  4.091600  3.572100  0.000000  1.200000
  3.400000  5.584100  8.390600  0.000000  1.600000
  4.400000  7.107000  14.72080  0.000000  1.800000
  5.400000  8.532900  22.53000  2.471446  2.000000
  6.400000  9.937100  31.75610  4.623653  3.000000
  7.400000  11.73030  42.57740  6.053783  7.000000
```

8.400000	13.12080	54.99650	16.33172	15.70000		
9.400000	14.53770	68.81970	41.95013	15.70000		
10.90000	15.18870	91.11270	58.09619	15.70000		

END FTABLE 1
FTABLE 2
91 5

Depth (ft)	Area (acres)	Volume (acre-ft)	Outflow1 (cfs)	Outflow2 (cfs)	Velocity (ft/sec)	Travel Time*** (Minutes)***
0.000000	0.688705	0.000000	0.000000	0.000000		
0.044444	0.691156	0.030664	0.000000	13.88889		
0.088889	0.693609	0.061436	0.000000	13.88889		
0.133333	0.696066	0.092318	0.000000	13.88889		
0.177778	0.698526	0.123309	0.000000	13.88889		
0.222222	0.700990	0.154409	0.000000	13.88889		
0.266667	0.703456	0.185619	0.000000	13.88889		
0.311111	0.705926	0.216939	0.000000	13.88889		
0.355556	0.708400	0.248368	0.000000	13.88889		
0.400000	0.710876	0.279908	0.000000	13.88889		
0.444444	0.713356	0.311557	0.000000	13.88889		
0.488889	0.715839	0.343317	0.000000	13.88889		
0.533333	0.718325	0.375187	0.000000	13.88889		
0.577778	0.720815	0.407168	0.000000	13.88889		
0.622222	0.723307	0.439260	0.000000	13.88889		
0.666667	0.725803	0.471462	0.000000	13.88889		
0.711111	0.728303	0.503776	0.000000	13.88889		
0.755556	0.730805	0.536200	0.000000	13.88889		
0.800000	0.733311	0.568736	0.000000	13.88889		
0.844444	0.735820	0.601384	0.000000	13.88889		
0.888889	0.738333	0.634143	0.000000	13.88889		
0.933333	0.740848	0.667013	0.000000	13.88889		
0.977778	0.743367	0.699996	0.000000	13.88889		
1.022222	0.745890	0.733091	0.000000	13.88889		
1.066667	0.748415	0.766297	0.000000	13.88889		
1.111111	0.750944	0.799616	0.000000	13.88889		
1.155556	0.753476	0.833048	0.000000	13.88889		
1.200000	0.756011	0.866592	0.000000	13.88889		
1.244444	0.758550	0.900249	0.000000	13.88889		
1.288889	0.761091	0.934019	0.000000	13.88889		
1.333333	0.763636	0.967902	0.000000	13.88889		
1.377778	0.766185	1.001898	0.000000	13.88889		
1.422222	0.768736	1.036007	0.000000	13.88889		
1.466667	0.771291	1.070230	0.000000	13.88889		
1.511111	0.773849	1.104566	0.000000	13.88889		
1.555556	0.776411	1.139016	0.000000	13.88889		
1.600000	0.778975	1.173581	0.000000	13.88889		
1.644444	0.781543	1.208259	0.000000	13.88889		
1.688889	0.784114	1.243051	0.000000	13.88889		
1.733333	0.786689	1.277958	0.000000	13.88889		
1.777778	0.789266	1.312979	0.000000	13.88889		
1.822222	0.791847	1.348115	0.000000	13.88889		
1.866667	0.794432	1.383366	0.000000	13.88889		
1.911111	0.797019	1.418731	0.000000	13.88889		
1.955556	0.799610	1.454212	0.000000	13.88889		
2.000000	0.802204	1.489808	0.000000	13.88889		
2.044444	0.804801	1.525519	0.000000	13.88889		
2.088889	0.807402	1.561346	0.000000	13.88889		
2.133333	0.810006	1.597288	0.000000	13.88889		
2.177778	0.812613	1.633346	0.000000	13.88889		
2.222222	0.815223	1.669520	0.000000	13.88889		
2.266667	0.817837	1.705811	0.000000	13.88889		
2.311111	0.820453	1.742217	0.000000	13.88889		
2.355556	0.823074	1.778740	0.000000	13.88889		
2.400000	0.825697	1.815379	0.000000	13.88889		
2.444444	0.828324	1.852135	0.000000	13.88889		
2.488889	0.830954	1.889008	0.000000	13.88889		
2.533333	0.833587	1.925998	0.000000	13.88889		
2.577778	0.836223	1.963105	0.000000	13.88889		
2.622222	0.838863	2.000329	0.000000	13.88889		
2.666667	0.841506	2.037670	0.000000	13.88889		
2.711111	0.844152	2.075129	0.000000	13.88889		

```
  2.755556  0.846802  2.112706  0.000000  13.88889
  2.800000  0.849455  2.150401  0.000000  13.88889
  2.844444  0.852111  2.188213  0.000000  13.88889
  2.888889  0.854770  2.226144  0.000000  13.88889
  2.933333  0.857433  2.264193  0.000000  13.88889
  2.977778  0.860098  2.302360  0.000000  13.88889
  3.022222  0.862767  2.340646  0.000000  13.88889
  3.066667  0.865440  2.379051  0.000000  13.88889
  3.111111  0.868115  2.417574  0.000000  13.88889
  3.155556  0.870794  2.456217  0.000000  13.88889
  3.200000  0.873477  2.494978  0.000000  13.88889
  3.244444  0.876162  2.533859  0.000000  13.88889
  3.288889  0.878851  2.572859  0.000000  13.88889
  3.333333  0.881543  2.611979  0.000000  13.88889
  3.377778  0.884238  2.651219  0.000000  13.88889
  3.422222  0.886936  2.690578  0.000000  13.88889
  3.466667  0.889638  2.730058  0.000000  13.88889
  3.511111  0.892343  2.769657  0.000000  13.88889
  3.555556  0.895052  2.809377  0.000000  13.88889
  3.600000  0.897763  2.849218  0.000000  13.88889
  3.644444  0.900478  2.889178  0.000000  13.88889
  3.688889  0.903196  2.929260  0.000000  13.88889
  3.733333  0.905917  2.969463  0.000000  13.88889
  3.777778  0.908642  3.009786  0.000000  13.88889
  3.822222  0.911370  3.050231  0.000000  13.88889
  3.866667  0.914101  3.090797  0.000000  13.88889
  3.911111  0.916835  3.131484  0.000000  13.88889
  3.955556  0.919573  3.172293  0.000000  13.88889
  4.000000  0.922314  3.213224  0.000000  13.88889
  END FTABLE  2
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP
WDM      2 PREC     ENGL    1.167          RCHRES   1     EXTNL  PREC
WDM      2 PREC     ENGL    1.167          RCHRES   2     EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           RCHRES   1     EXTNL  POTEV
WDM      1 EVAP     ENGL    0.76           RCHRES   2     EXTNL  POTEV

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
RCHRES   2 HYDR   RO     1 1        1      WDM   1006 FLOW     ENGL      REPL
RCHRES   2 HYDR   O      1 1        1      WDM   1007 FLOW     ENGL      REPL
RCHRES   2 HYDR   O      2 1        1      WDM   1008 FLOW     ENGL      REPL
RCHRES   2 HYDR   STAGE  1 1        1      WDM   1009 STAG     ENGL      REPL
COPY     1 OUTPUT MEAN   1 1     48.4      WDM    701 FLOW     ENGL      REPL
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    801 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK        2
PERLND     PWATER SURO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    2

  MASS-LINK        3
PERLND     PWATER IFWO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    3

  MASS-LINK        5
IMPLND     IWATER SURO       0.083333      RCHRES         INFLOW IVOL
```

```
  END MASS-LINK     5

  MASS-LINK         7
RCHRES     OFLOW  OVOL   1                      RCHRES          INFLOW IVOL
  END MASS-LINK     7

  MASS-LINK        17
RCHRES     OFLOW  OVOL   1                      COPY            INPUT  MEAN
  END MASS-LINK    17

END MASS-LINK

END RUN
```

Predeveloped HSPF Message File

Mitigated HSPF Message File

Disclaimer

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www.clearcreeksolutions.com

JOB NO.	0992002.050.051		
JOB NAME	Queen City Farms - Phase III Refill		
SUBJECT	French Drain Sizing		
CALC BY	MDV	DATE	8/23/2019
CHK BY	KMS	DATE	8/27/2019

Objectives

1 Determine the peak design flow rate for the French drain within the Tributary 316A engineered reach, using 100-year peak flow from WWHM.
2 Determine the flow capacity of the proposed French drain.
3 Determine the required flow capacity of an internal subdrain pipe within the French drain

1. Determine peak design flow rate

Approach Use the flow frequency distribution modeled in WWHM to obtain the design flow for the French drain, using the drainage area delineated on Sheet 5 of the TIR site improvement plans.

Assumptions Design flow rate will be for 100-year return period from the 13-acre drainage area (a portion of the drainage area is within the Maple Hills and Cedar Hills sub-basins)
Drainage area does not include the existing compost pad stormwater infiltration pond
11.5 acres are outwash (A/B), forest (undeveloped forested area from aerial maps)
1.5 acres are gravel refill; modeled as till (C), pasture
Land slopes are moderate (5-15%)

Solution From WWHM model results (see attached):
The design flow (Q_d) is 0.84 cfs

2. Determine flow capacity of French drain

Approach Use Darcy's Law with assumed porosity of drainage gravel and design parameters of the existing Tributary 316A engineered reach.

Assumptions Gravel for drain will have permeability of 10 cm/s.
Tributary 316A engineered reach dimensions:
hydraulic gradient = 0.018 (min)
min. backfill depth 2.5 ft
min. backfill top width at
3:1 side slopes 15 ft

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JOB NAME	Queen City Farms - Phase III Refill		
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Calculation Q = kiA

k, coefficient of permeability for drain gravel	10	cm/s
k	0.33	ft/s
I, hydraulic gradient	0.018	ft/ft
A, flow area	19	sq ft
Q, flow capacity of french drain	0.11	cfs
Qd, 100-year design flow	0.84	cfs
remaining capacity to be carried by pipe	0.73	cfs

Conclusion The French drain will not have adequate flow capacity with a minimum backfill depth of 2.5 ft to carry the entire 100-year flow. A subdrain pipe will be required.

3. Determine required pipe diameter to convey peak flow

Approach Use the Manning Equation for a partially full pipe to determine the maximum pipe flow capacity.

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad \text{(equation 8)}$$

Where Q = discharge (cfs)

V = velocity (fps)

A = area (sf)

n = Manning's roughness coefficient; see Table D.4- 2 Manning's "n" Values for Pipes

R = hydraulic radius = area/wetted perimeter

S = slope of the energy grade line (ft/ft)

Assumptions

1 Maximum pipe capacity occurs when ratio of flow depth to diameter is 0.94.

2	Pipe Slope	0.018	ft/ft
3	Manning's n for PVC pipe	0.011	
4	pipe diameter	6	in

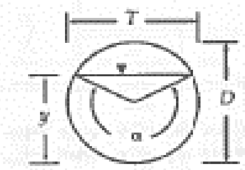
Conversions

1	ft	=	12	in
	π	=	3.14159	

JOB NO.	0992002.050.051
JOB NAME	Queen City Farms - Phase III Refill
SUBJECT	French Drain Sizing
CALC BY	MDV
DATE	8/23/2019
CHK BY	KMS
DATE	8/27/2019

Calculation

Circular



	Area =	Wetted Perimeter =		
$\alpha = 2\sin^{-1}\left[\frac{2\sqrt{y(D-y)}}{D}\right]$	$(\alpha - \sin\alpha)D^2/8$	$\frac{\alpha D}{2}$	$\frac{D}{4}\left[1 - \frac{\sin\alpha}{\alpha}\right]$	$(\sin \tfrac{1}{2}\alpha)D$ or $2\sqrt{y(D-y)}$

?
not sure if this is correct

$$\alpha = 2\left[\pi - \cos^{-1}\left[\frac{y - 0.5D}{0.5D}\right]\right]$$

See above for equations.

Ratio of Depth to Diameter, y/D	Depth, y ft	Alpha, α radians	Area, A ft^2	Wetted Perimeter, P ft	Hydraulic Radius, R ft	Q cfs
0.940	0.470	5.29	0.192	1.323	0.145	0.96

Conclusion

A French drain as designed above, with 6-inch subdrain pipe, will have sufficient capacity to carry the 100-year flow beneath the Phase III refill.

WWHM2012

PROJECT REPORT

General Model Information

Project Name:	QCF Phase III - French Drain Sizing
Site Name:	Queen City Farms
Site Address:	17825 Cedar Grove Rd SE
City:	Maple Valley
Report Date:	8/23/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.167
Version Date:	2018/10/10
Version:	4.2.16

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

French Drain Drainage Area

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	11.5
C, Pasture, Mod	1.5
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

French Drain Drainage Area

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	11.5
C, Pasture, Mod	1.5
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

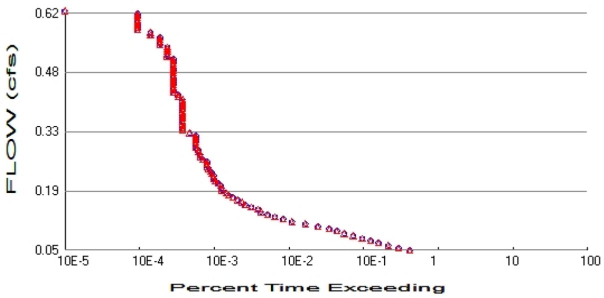
Surface	Interflow	Groundwater

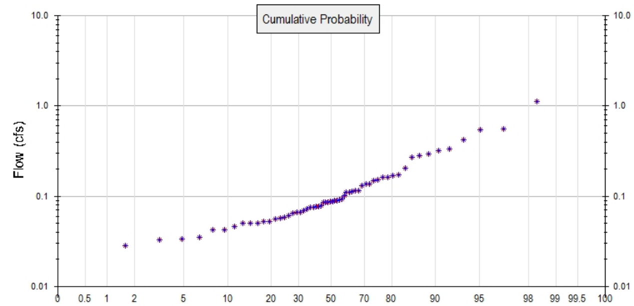
Routing Elements
Predeveloped Routing

Mitigated Routing

Analysis Results

POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 13
Total Impervious Area: 0

Mitigated Landuse Totals for POC #1
Total Pervious Area: 13
Total Impervious Area: 0

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	0.091544
5 year	0.18407
10 year	0.277647
25 year	0.446163
50 year	0.618501
100 year	0.841228

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	0.091544
5 year	0.18407
10 year	0.277647
25 year	0.446163
50 year	0.618501
100 year	0.841228

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	0.162	0.162
1950	0.426	0.426
1951	0.170	0.170
1952	0.047	0.047
1953	0.042	0.042
1954	0.077	0.077
1955	0.089	0.089
1956	0.152	0.152
1957	0.086	0.086
1958	0.076	0.076

1959	0.056	0.056
1960	0.149	0.149
1961	0.078	0.078
1962	0.043	0.043
1963	0.065	0.065
1964	0.113	0.113
1965	0.088	0.088
1966	0.050	0.050
1967	0.203	0.203
1968	0.086	0.086
1969	0.073	0.073
1970	0.066	0.066
1971	0.085	0.085
1972	0.268	0.268
1973	0.053	0.053
1974	0.092	0.092
1975	0.102	0.102
1976	0.075	0.075
1977	0.057	0.057
1978	0.061	0.061
1979	0.033	0.033
1980	0.280	0.280
1981	0.058	0.058
1982	0.163	0.163
1983	0.088	0.088
1984	0.051	0.051
1985	0.034	0.034
1986	0.111	0.111
1987	0.110	0.110
1988	0.050	0.050
1989	0.035	0.035
1990	0.539	0.539
1991	0.294	0.294
1992	0.069	0.069
1993	0.052	0.052
1994	0.028	0.028
1995	0.114	0.114
1996	0.561	0.561
1997	0.135	0.135
1998	0.077	0.077
1999	0.323	0.323
2000	0.066	0.066
2001	0.022	0.022
2002	0.116	0.116
2003	0.171	0.171
2004	0.136	0.136
2005	0.092	0.092
2006	0.089	0.089
2007	1.119	1.119
2008	0.331	0.331
2009	0.131	0.131

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	1.1193	1.1193
2	0.5606	0.5606
3	0.5394	0.5394

4	0.4260	0.4260
5	0.3313	0.3313
6	0.3230	0.3230
7	0.2937	0.2937
8	0.2804	0.2804
9	0.2677	0.2677
10	0.2031	0.2031
11	0.1714	0.1714
12	0.1700	0.1700
13	0.1632	0.1632
14	0.1621	0.1621
15	0.1522	0.1522
16	0.1487	0.1487
17	0.1364	0.1364
18	0.1355	0.1355
19	0.1313	0.1313
20	0.1163	0.1163
21	0.1143	0.1143
22	0.1128	0.1128
23	0.1115	0.1115
24	0.1097	0.1097
25	0.1022	0.1022
26	0.0924	0.0924
27	0.0918	0.0918
28	0.0891	0.0891
29	0.0887	0.0887
30	0.0881	0.0881
31	0.0880	0.0880
32	0.0857	0.0857
33	0.0856	0.0856
34	0.0850	0.0850
35	0.0782	0.0782
36	0.0769	0.0769
37	0.0766	0.0766
38	0.0759	0.0759
39	0.0745	0.0745
40	0.0725	0.0725
41	0.0689	0.0689
42	0.0662	0.0662
43	0.0658	0.0658
44	0.0653	0.0653
45	0.0608	0.0608
46	0.0580	0.0580
47	0.0573	0.0573
48	0.0559	0.0559
49	0.0526	0.0526
50	0.0521	0.0521
51	0.0507	0.0507
52	0.0501	0.0501
53	0.0500	0.0500
54	0.0466	0.0466
55	0.0428	0.0428
56	0.0420	0.0420
57	0.0347	0.0347
58	0.0335	0.0335
59	0.0328	0.0328
60	0.0283	0.0283
61	0.0217	0.0217

Duration Flows
The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
0.0458	8846	8846	100	Pass
0.0516	6205	6205	100	Pass
0.0573	4564	4564	100	Pass
0.0631	3397	3397	100	Pass
0.0689	2663	2663	100	Pass
0.0747	2045	2045	100	Pass
0.0805	1543	1543	100	Pass
0.0863	1171	1171	100	Pass
0.0921	929	929	100	Pass
0.0978	738	738	100	Pass
0.1036	519	519	100	Pass
0.1094	356	356	100	Pass
0.1152	235	235	100	Pass
0.1210	178	178	100	Pass
0.1268	138	138	100	Pass
0.1325	111	111	100	Pass
0.1383	90	90	100	Pass
0.1441	82	82	100	Pass
0.1499	67	67	100	Pass
0.1557	55	55	100	Pass
0.1615	50	50	100	Pass
0.1673	44	44	100	Pass
0.1730	38	38	100	Pass
0.1788	33	33	100	Pass
0.1846	31	31	100	Pass
0.1904	27	27	100	Pass
0.1962	27	27	100	Pass
0.2020	26	26	100	Pass
0.2078	24	24	100	Pass
0.2135	22	22	100	Pass
0.2193	21	21	100	Pass
0.2251	21	21	100	Pass
0.2309	20	20	100	Pass
0.2367	19	19	100	Pass
0.2425	18	18	100	Pass
0.2483	17	17	100	Pass
0.2540	17	17	100	Pass
0.2598	17	17	100	Pass
0.2656	15	15	100	Pass
0.2714	14	14	100	Pass
0.2772	14	14	100	Pass
0.2830	13	13	100	Pass
0.2887	13	13	100	Pass
0.2945	12	12	100	Pass
0.3003	12	12	100	Pass
0.3061	12	12	100	Pass
0.3119	12	12	100	Pass
0.3177	12	12	100	Pass
0.3235	12	12	100	Pass
0.3292	10	10	100	Pass
0.3350	8	8	100	Pass
0.3408	8	8	100	Pass
0.3466	8	8	100	Pass

0.3524	8	8	100	Pass
0.3582	8	8	100	Pass
0.3640	8	8	100	Pass
0.3697	8	8	100	Pass
0.3755	8	8	100	Pass
0.3813	8	8	100	Pass
0.3871	8	8	100	Pass
0.3929	8	8	100	Pass
0.3987	8	8	100	Pass
0.4045	8	8	100	Pass
0.4102	8	8	100	Pass
0.4160	7	7	100	Pass
0.4218	7	7	100	Pass
0.4276	6	6	100	Pass
0.4334	6	6	100	Pass
0.4392	6	6	100	Pass
0.4449	6	6	100	Pass
0.4507	6	6	100	Pass
0.4565	6	6	100	Pass
0.4623	6	6	100	Pass
0.4681	6	6	100	Pass
0.4739	6	6	100	Pass
0.4797	6	6	100	Pass
0.4854	6	6	100	Pass
0.4912	6	6	100	Pass
0.4970	6	6	100	Pass
0.5028	6	6	100	Pass
0.5086	6	6	100	Pass
0.5144	5	5	100	Pass
0.5202	5	5	100	Pass
0.5259	5	5	100	Pass
0.5317	5	5	100	Pass
0.5375	5	5	100	Pass
0.5433	4	4	100	Pass
0.5491	4	4	100	Pass
0.5549	4	4	100	Pass
0.5606	4	4	100	Pass
0.5664	3	3	100	Pass
0.5722	3	3	100	Pass
0.5780	2	2	100	Pass
0.5838	2	2	100	Pass
0.5896	2	2	100	Pass
0.5954	2	2	100	Pass
0.6011	2	2	100	Pass
0.6069	2	2	100	Pass
0.6127	2	2	100	Pass
0.6185	2	2	100	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0 acre-feet
On-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.
Off-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Total Volume Infiltrated		0.00	0.00	0.00		0.00	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

Model Default Modifications

Total of 0 changes have been made.

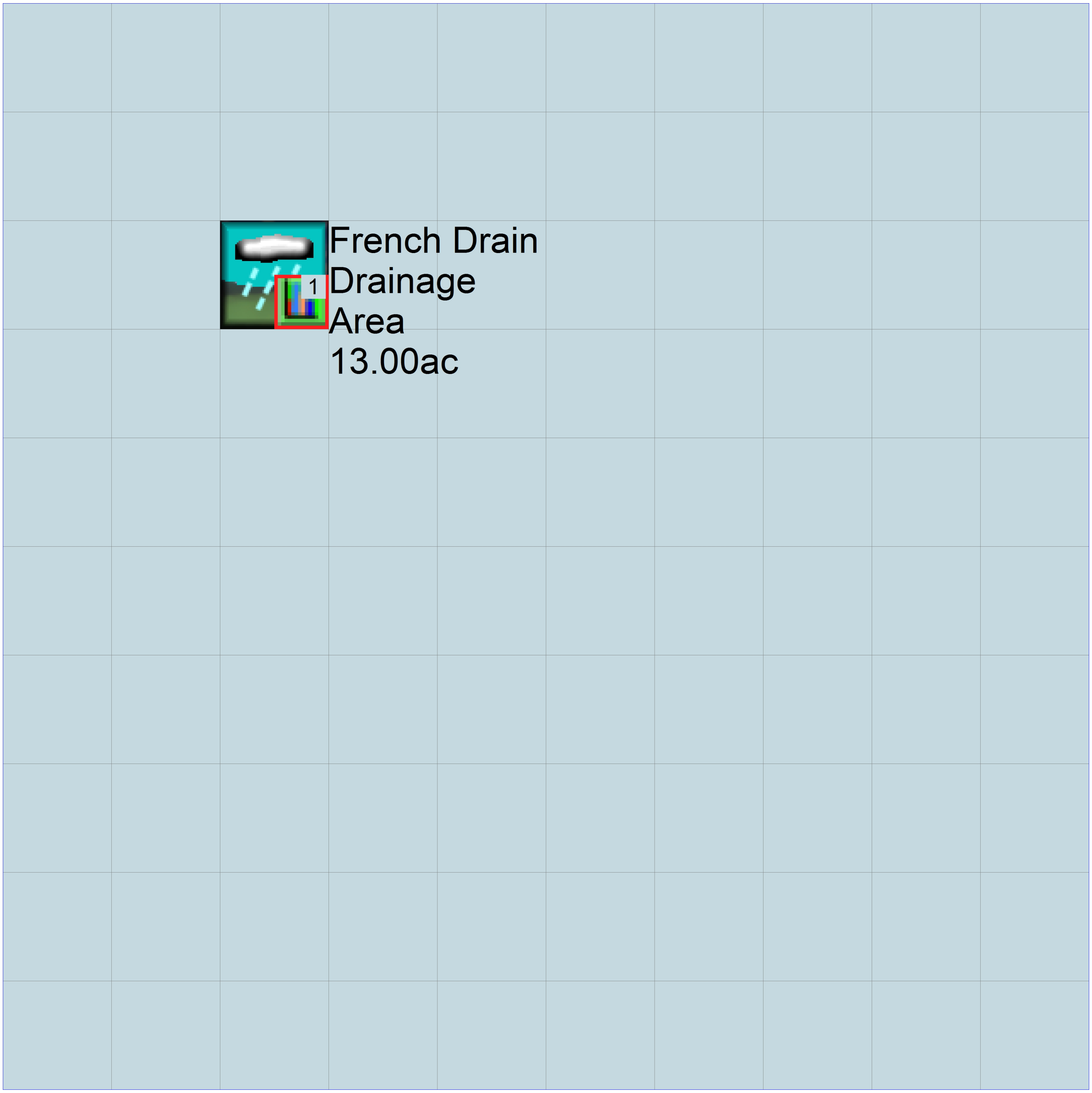
PERLND Changes

No PERLND changes have been made.

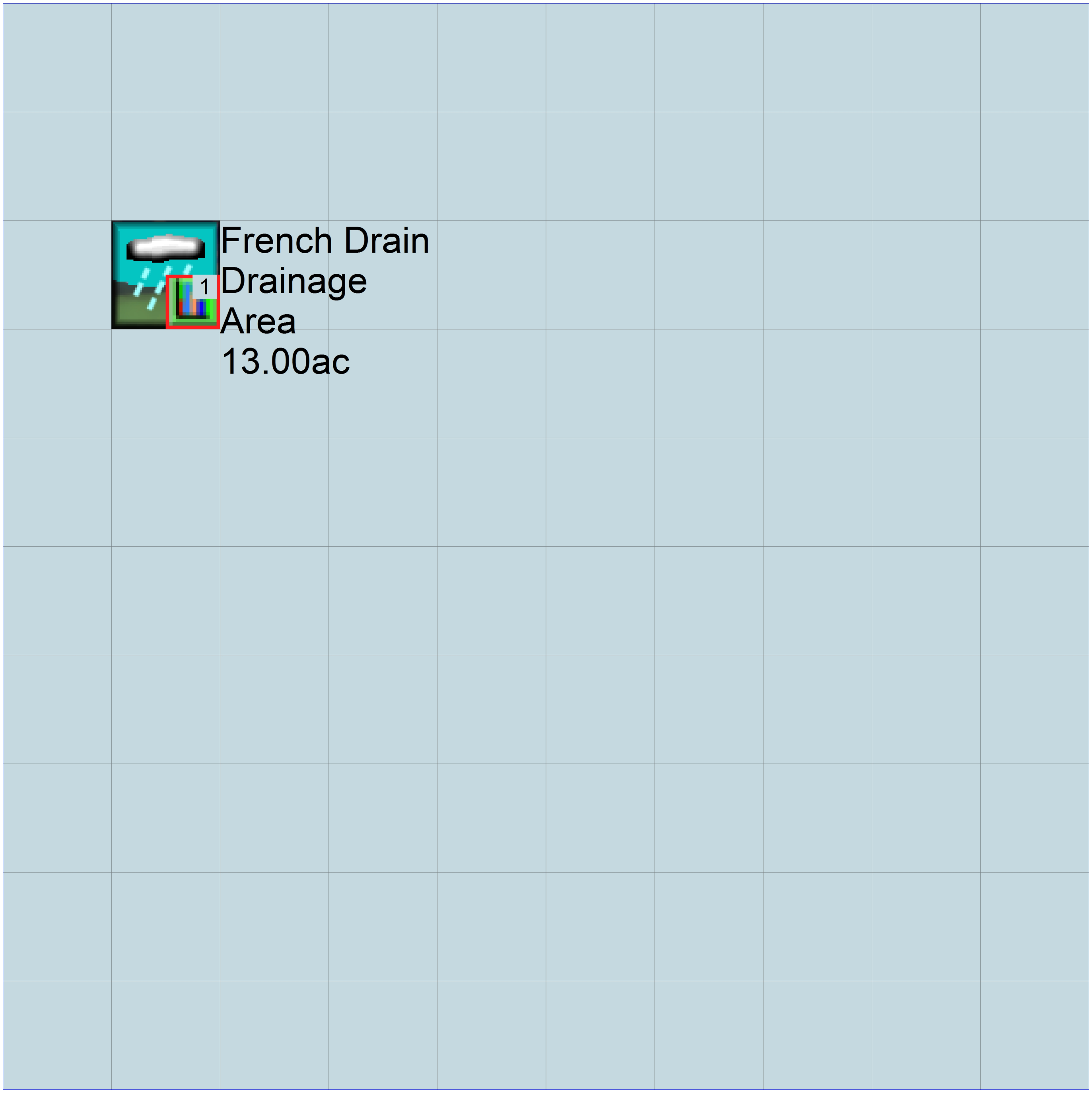
IMPLND Changes

No IMPLND changes have been made.

Appendix
Predeveloped Schematic



Mitigated Schematic



Predeveloped UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                              ***
WDM        26   QCF Phase III - French Drain Sizing.wdm
MESSU      25   PreQCF Phase III - French Drain Sizing.MES
           27   PreQCF Phase III - French Drain Sizing.L61
           28   PreQCF Phase III - French Drain Sizing.L62
           30   POCQCF Phase III - French Drain Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      PERLND      14
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        French Drain Drainage Are   MAX                     1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                          User  t-series Engl Metr ***
                                          in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
    2         0    0    1    0    0    0    0    0    0    0    0    0
   14         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
```

```
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2              0         5         2       400       0.1       0.3     0.996
   14              0       4.5      0.06       400       0.1       0.5     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2              0         0         2         2         0         0         0
   14              0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2            0.2       0.5      0.35         0       0.7       0.7
   14           0.15       0.4       0.3         6       0.5       0.4
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2              0         0         0         0         3         1         0
   14              0         0         0         0       2.5         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name-------->   Unit-systems   Printer ***
    # -  #                      User  t-series Engl Metr ***
                                       in  out           ***
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
  END ACTIVITY

  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
  END IWAT-PARM2

  IWAT-PARM3
```

```
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
French Drain Drainage Area***
PERLND   2                        11.5     COPY   501     12
PERLND   2                        11.5     COPY   501     13
PERLND  14                         1.5     COPY   501     12
PERLND  14                         1.5     COPY   501     13

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                 ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                         in  out                             ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                      ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *    *  *  *  *  *       *  *  *  *  *         ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50       ***
  <------><--------><--------><--------><--------><--------><-------->       ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                         ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES
```

```
SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    501 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>      <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>               <Name> # #***
  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY          INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY          INPUT  MEAN
  END MASS-LINK   13


END MASS-LINK

END RUN
```

Mitigated UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                              ***
WDM        26   QCF Phase III - French Drain Sizing.wdm
MESSU      25   MitQCF Phase III - French Drain Sizing.MES
           27   MitQCF Phase III - French Drain Sizing.L61
           28   MitQCF Phase III - French Drain Sizing.L62
           30   POCQCF Phase III - French Drain Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      PERLND      14
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        French Drain Drainage Are   MAX                    1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                            User  t-series Engl Metr ***
                                             in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
    2         0    0    1    0    0    0    0    0    0    0    0    0
   14         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
```

```
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2             0         5         2       400       0.1       0.3     0.996
   14             0       4.5      0.06       400       0.1       0.5     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2             0         0         2         2         0         0         0
   14             0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2           0.2       0.5      0.35         0       0.7       0.7
   14          0.15       0.4       0.3         6       0.5       0.4
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2             0         0         0         0         3         1         0
   14             0         0         0         0       2.5         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                     in  out            ***
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
  END ACTIVITY

  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
  END IWAT-PARM2

  IWAT-PARM3
```

```
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
French Drain Drainage Area***
PERLND   2                        11.5     COPY   501     12
PERLND   2                        11.5     COPY   501     13
PERLND  14                         1.5     COPY   501     12
PERLND  14                         1.5     COPY   501     13

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                 ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                           in  out                           ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                      ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *    *  *  *  *  *       *  *  *  *  *         ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50       ***
  <------><--------><--------><--------><--------><--------><-------->       ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                         ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES
```

```
SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY     1 OUTPUT MEAN   1 1     48.4      WDM    701 FLOW     ENGL      REPL
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    801 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13


END MASS-LINK

END RUN
```

Predeveloped HSPF Message File

Mitigated HSPF Message File

Disclaimer

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Objectives

1 Determine the peak design flow rate from Queen City Lake (QCL) Outlet to East Pond for a pipe with 0.5% slope.
2 Determine the pipe size required to convey the peak flows calculated from Objective 1.
3 Determine the full flow pipe velocity and design pipe velocity for the selected pipe size from Objective 2 at a 17% slope.

1. Determine peak design flow rate

Approach Use the flow frequency distribution modeled in WWHM to obtain the design flow for the pipe.

Assumptions Design flow rate will be for 100-year return period

From WWHM results (see 6.10 TM):

Solution The design flow (Q_d) is 13.69 cfs
with 1.2 factor of safety (FS) 16.43 cfs

2. Determine required pipe diameter to convey peak flow

Approach Use the Manning Equation for a partially full pipe to determine the maximum pipe flow capacity for a range of pipe diameters.

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \quad \text{(equation 8)}$$

Where Q = discharge (cfs)

V = velocity (fps)

A = area (sf)

n = Manning's roughness coefficient; see Table D.4- 2 Manning's "n" Values for Pipes

R = hydraulic radius = area/wetted perimeter

S = slope of the energy grade line (ft/ft)

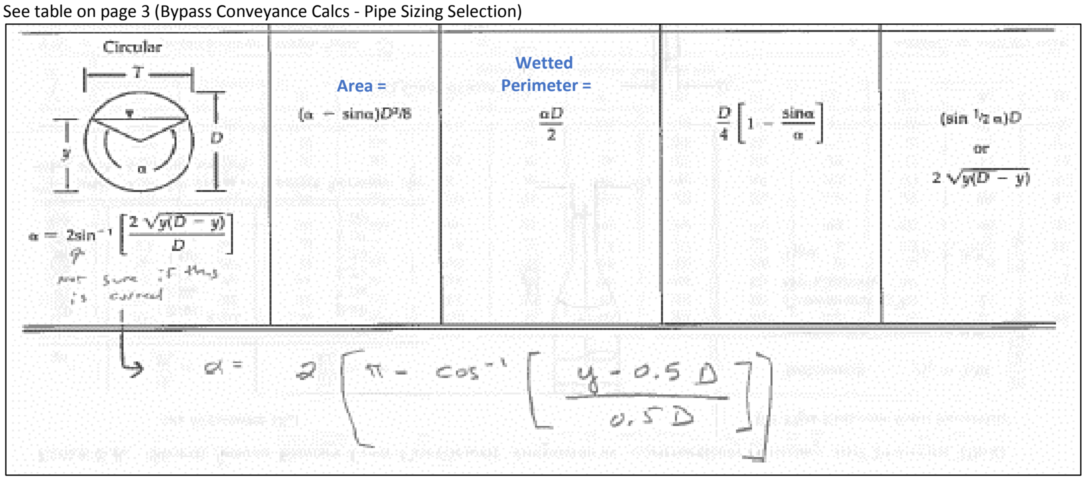
Assumptions

1 Maximum pipe capacity occurs when ratio of flow depth to diameter is 0.94.
2 Pipe Slope 0.005 ft/ft
3 Manning's n for PVC pipe 0.011

Conversions

1 ft	=	12	in
π	=	3.14159	

Calculation See table on page 3 (Bypass Conveyance Calcs - Pipe Sizing Selection)



Solution

20.39 cfs (maximum pipe capacity - 24" full pipe, Q_f)
16.43 cfs (design peak flow rate, 100-year storm, Q_d, with FS)
1.24 Ratio, Q_f/Q_d

Conclusion A 24" pipe has sufficient capacity to convey the design peak flow rate for the 0.5% sloped segment of pipe. Therefore a 24" pipe

A 24" pipe has sufficient capacity to convey the design peak flow rate for the 0.5% sloped segment of pipe. Therefore a 24" pipe will also have sufficient capacity for the 3.5% and 17% sloped segments of pipe.

<u>3. Determine pipe velocities for selected pipe diameter at a 17% slope</u>

Approach Use the Manning Equation (above) for a partially full pipe to determine the maximum pipe flow capacity and corresponding pipe velocity.
Using the Manning Equation and "Goal Seek" Excel function (by varying flow depth, y), determine the design flow pipe velocity.
Average flow velocity = flow divided by cross sectional area.

Assumptions

1	Pipe Diameter	24	in
		2.0	ft
2	Pipe Slope	0.17	ft/ft
3	Manning's n for pipe	0.011	

Conversions

1	ft	=	12	in
	π	=	3.14159	

Calculation See above for equations.

Ratio of Depth to Diameter, y/D	Depth, y	Alpha, α	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q	
	ft	radians	ft^2	ft	ft	cfs	
0.940	1.880	5.29	3.065	5.293	0.579	118.1	*Partially full (Q_f)*
--	0.477	2.04	0.575	2.041	0.282	13.7	*For solving V_d*

V = Q/A

Solution

38.5 fps (maximum pipe velocity, full pipe, V_f)
23.8 fps (design pipe velocity, V_d)

Conclusion A 24" pipe at 17% slope will reach 23.8 fps at the design flow (100-year storm).
Because this velocity exceeds 20 fps, an engineered energy dissipater (rip rap basin) will be required, per the KCSWDM Table 4.2.2.A.

Landau Associates

JOB NO.	0992002.050.051		
JOB NAME	Queen City Farms - Phase III Refill		
SUBJECT	Bypass Conveyance Calcs - Pipe Sizing Selection		
CALC BY	MDV	DATE	8/22/2019
CHK BY	KMS	DATE	8/27/2019

Q_d cfs	D in	s %	n	y/d	y ft	α radians	A ft^2	P ft	R ft	Q_f Qf	F.S.	V ft/s
16.4	18	0.50	0.011	0.94	1.41	5.29	1.724	3.970	0.434	9.47	0.58	na
16.4	24	0.50	0.011	0.94	1.88	5.29	3.065	5.293	0.579	20.39	1.24	na
16.4	30	0.50	0.011	0.94	2.35	5.29	4.788	6.617	0.724	36.97	2.25	na
16.4	36	0.50	0.011	0.94	2.82	5.29	6.895	7.940	0.868	60.12	3.66	2.38
16.4	48	0.50	0.011	0.94	3.76	5.29	12.259	10.587	1.158	129.47	7.88	1.34

Q_d	design flow (100-year x 1.2 multiplication factor)
D	diameter of pipe
s	slope of pipe
n	Manning's roughness coefficient
y	depth
A	area
P	wetted perimeter
R	hydraulic radius = A/P
Q_f	full capacity flow (94%)
V	average flow velocity
F.S.	Factor of Safety

Conversions

ft = 12 in

π = 3.1415927

WWHM2012

PROJECT REPORT

General Model Information

Project Name:	QCF Phase III - Queen City Lake Outlet Sizing
Site Name:	Queen City Farms
Site Address:	17825 Cedar Grove Rd SE
City:	Maple Valley
Report Date:	8/26/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.167
Version Date:	2018/10/10
Version:	4.2.16

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Cedar Hills dev sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	333
C, Pasture, Mod	11
Pervious Total	344
Impervious Land Use	acre
POND	8.5
Impervious Total	8.5
Basin Total	352.5

Element Flows To:

Surface	Interflow	Groundwater

Maple Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
C, Pasture, Mod	8
Pervious Total	136
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	174

Element Flows To:

Surface	Interflow	Groundwater

Phase III to QCL

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	13
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Maple Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
C, Pasture, Mod	8
Pervious Total	136
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	174

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Cedar Hills sub-basin

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Forest, Mod	333
C, Pasture, Mod	11
Pervious Total	344
Impervious Land Use	acre
POND	8.5
Impervious Total	8.5
Basin Total	352.5

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Phase III Refill to QCL

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
C, Pasture, Mod	13
Pervious Total	13
Impervious Land Use	acre
Impervious Total	0
Basin Total	13

Element Flows To:

Surface	Interflow	Groundwater
Queen City Lake	Queen City Lake	

Routing Elements
Predeveloped Routing

Mitigated Routing

Queen City Lake

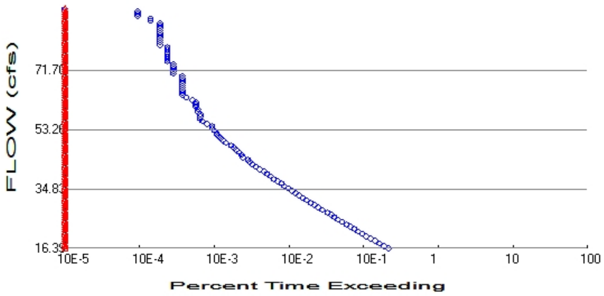
Depth: 10.9 ft.
Discharge Structure: 1
Riser Height: 7.96 ft.
Riser Diameter: 36 in.
Orifice 1 Diameter: 12 in. Elevation:5 ft.
Element Flows To:
Outlet 1 Outlet 2

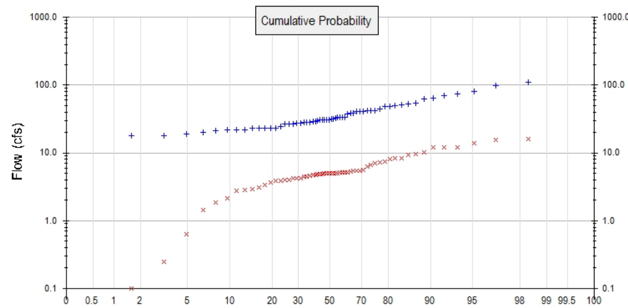
SSD Table Hydraulic Table

Stage (feet)	Area (ac.)	Volume (ac-ft.)	Outlet Struct	Infilt (cfs)	NotUsed	NotUsed	NotUsed
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.400	0.006	0.001	0.000	0.300	0.000	0.000	0.000
1.400	1.870	0.663	0.000	0.800	0.000	0.000	0.000
2.400	4.092	3.572	0.000	1.200	0.000	0.000	0.000
3.400	5.584	8.391	0.000	1.600	0.000	0.000	0.000
4.400	7.107	14.72	0.000	1.800	0.000	0.000	0.000
5.400	8.533	22.53	2.471	2.000	0.000	0.000	0.000
6.400	9.937	31.76	4.624	3.000	0.000	0.000	0.000
7.400	11.73	42.58	6.054	7.000	0.000	0.000	0.000
8.400	13.12	55.00	16.33	15.70	0.000	0.000	0.000
9.400	14.54	68.82	41.95	15.70	0.000	0.000	0.000
10.90	15.19	91.11	58.10	15.70	0.000	0.000	0.000

Analysis Results

POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 493
Total Impervious Area: 46.5

Mitigated Landuse Totals for POC #1
Total Pervious Area: 493
Total Impervious Area: 46.5

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	32.775625
5 year	47.768722
10 year	59.378734
25 year	76.101607
50 year	90.141409
100 year	105.616188

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	5.179732
5 year	8.281944
10 year	9.998736
25 year	11.764039
50 year	12.820834
100 year	13.693677

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	50.858	3.890
1950	48.944	6.249
1951	40.610	16.199
1952	22.991	3.092
1953	19.226	3.965
1954	26.830	5.092
1955	31.257	5.134
1956	30.418	6.543
1957	37.020	4.814
1958	23.401	5.401

1959	21.848	4.575
1960	41.900	10.124
1961	29.078	4.819
1962	17.999	1.873
1963	28.627	4.970
1964	28.480	4.499
1965	33.299	5.161
1966	23.074	4.242
1967	50.189	5.376
1968	32.405	4.718
1969	28.592	4.177
1970	30.871	3.965
1971	33.688	4.771
1972	41.832	9.656
1973	21.813	4.966
1974	33.485	5.193
1975	40.273	4.507
1976	30.365	5.000
1977	21.235	0.000
1978	27.562	4.886
1979	30.632	2.158
1980	70.258	8.430
1981	29.868	4.221
1982	54.614	5.434
1983	26.534	5.044
1984	22.009	2.904
1985	23.259	0.248
1986	38.988	7.022
1987	39.060	8.275
1988	17.832	2.866
1989	24.169	1.433
1990	111.627	12.012
1991	63.831	12.140
1992	27.125	4.988
1993	19.894	3.349
1994	15.454	0.628
1995	26.805	5.551
1996	63.304	15.479
1997	41.862	12.152
1998	28.411	2.780
1999	73.944	8.036
2000	30.697	3.663
2001	23.447	0.000
2002	40.328	4.981
2003	48.488	3.872
2004	53.182	7.201
2005	33.648	5.023
2006	31.518	5.217
2007	98.549	9.439
2008	79.507	13.868
2009	44.157	7.458

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	111.6270	16.1991
2	98.5492	15.4790
3	79.5073	13.8682

4	73.9443	12.1519
5	70.2581	12.1396
6	63.8307	12.0116
7	63.3038	10.1244
8	54.6140	9.6563
9	53.1816	9.4391
10	50.8580	8.4301
11	50.1890	8.2749
12	48.9439	8.0362
13	48.4878	7.4579
14	44.1568	7.2010
15	41.9000	7.0218
16	41.8620	6.5430
17	41.8315	6.2487
18	40.6099	5.5505
19	40.3283	5.4336
20	40.2733	5.4014
21	39.0604	5.3760
22	38.9884	5.2171
23	37.0198	5.1933
24	33.6880	5.1608
25	33.6479	5.1344
26	33.4852	5.0915
27	33.2994	5.0440
28	32.4045	5.0231
29	31.5179	4.9996
30	31.2571	4.9884
31	30.8712	4.9811
32	30.6971	4.9698
33	30.6323	4.9658
34	30.4175	4.8856
35	30.3647	4.8193
36	29.8675	4.8143
37	29.0776	4.7707
38	28.6270	4.7179
39	28.5924	4.5750
40	28.4797	4.5066
41	28.4106	4.4988
42	27.5616	4.2424
43	27.1253	4.2205
44	26.8298	4.1775
45	26.8052	3.9649
46	26.5342	3.9647
47	24.1692	3.8903
48	23.4469	3.8723
49	23.4009	3.6630
50	23.2587	3.3487
51	23.0736	3.0923
52	22.9911	2.9044
53	22.0086	2.8662
54	21.8475	2.7801
55	21.8128	2.1580
56	21.2348	1.8727
57	19.8937	1.4330
58	19.2256	0.6277
59	17.9993	0.2481
60	17.8317	0.0000
61	15.4538	0.0000

Duration Flows

The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
16.3878	4552	0	0	Pass
17.1328	3974	0	0	Pass
17.8778	3454	0	0	Pass
18.6228	3018	0	0	Pass
19.3678	2620	0	0	Pass
20.1127	2289	0	0	Pass
20.8577	2032	0	0	Pass
21.6027	1805	0	0	Pass
22.3477	1606	0	0	Pass
23.0927	1417	0	0	Pass
23.8377	1257	0	0	Pass
24.5827	1102	0	0	Pass
25.3276	982	0	0	Pass
26.0726	859	0	0	Pass
26.8176	783	0	0	Pass
27.5626	698	0	0	Pass
28.3076	602	0	0	Pass
29.0526	520	0	0	Pass
29.7976	475	0	0	Pass
30.5425	419	0	0	Pass
31.2875	373	0	0	Pass
32.0325	328	0	0	Pass
32.7775	288	0	0	Pass
33.5225	261	0	0	Pass
34.2675	235	0	0	Pass
35.0125	210	0	0	Pass
35.7574	186	0	0	Pass
36.5024	169	0	0	Pass
37.2474	150	0	0	Pass
37.9924	134	0	0	Pass
38.7374	122	0	0	Pass
39.4824	108	0	0	Pass
40.2274	99	0	0	Pass
40.9723	86	0	0	Pass
41.7173	77	0	0	Pass
42.4623	69	0	0	Pass
43.2073	64	0	0	Pass
43.9523	60	0	0	Pass
44.6973	52	0	0	Pass
45.4423	50	0	0	Pass
46.1872	47	0	0	Pass
46.9322	42	0	0	Pass
47.6772	39	0	0	Pass
48.4222	36	0	0	Pass
49.1672	31	0	0	Pass
49.9122	28	0	0	Pass
50.6572	26	0	0	Pass
51.4021	24	0	0	Pass
52.1471	23	0	0	Pass
52.8921	22	0	0	Pass
53.6371	20	0	0	Pass
54.3821	20	0	0	Pass
55.1271	17	0	0	Pass

55.8721	15	0	0	Pass
56.6170	14	0	0	Pass
57.3620	14	0	0	Pass
58.1070	14	0	0	Pass
58.8520	13	0	0	Pass
59.5970	13	0	0	Pass
60.3420	12	0	0	Pass
61.0870	12	0	0	Pass
61.8319	12	0	0	Pass
62.5769	11	0	0	Pass
63.3219	9	0	0	Pass
64.0669	8	0	0	Pass
64.8119	8	0	0	Pass
65.5569	8	0	0	Pass
66.3019	8	0	0	Pass
67.0468	8	0	0	Pass
67.7918	8	0	0	Pass
68.5368	8	0	0	Pass
69.2818	8	0	0	Pass
70.0268	8	0	0	Pass
70.7718	6	0	0	Pass
71.5168	6	0	0	Pass
72.2617	6	0	0	Pass
73.0067	6	0	0	Pass
73.7517	6	0	0	Pass
74.4967	5	0	0	Pass
75.2417	5	0	0	Pass
75.9867	5	0	0	Pass
76.7317	5	0	0	Pass
77.4767	5	0	0	Pass
78.2216	5	0	0	Pass
78.9666	5	0	0	Pass
79.7116	4	0	0	Pass
80.4566	4	0	0	Pass
81.2016	4	0	0	Pass
81.9466	4	0	0	Pass
82.6916	4	0	0	Pass
83.4365	4	0	0	Pass
84.1815	4	0	0	Pass
84.9265	4	0	0	Pass
85.6715	4	0	0	Pass
86.4165	4	0	0	Pass
87.1615	3	0	0	Pass
87.9065	3	0	0	Pass
88.6514	2	0	0	Pass
89.3964	2	0	0	Pass
90.1414	2	0	0	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0 acre-feet
On-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.
Off-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Queen City Lake POC	☐	29242.24			☐	77.26			
Total Volume Infiltrated		29242.24	0.00	0.00		77.26	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

POC 2

POC #2 was not reported because POC must exist in both scenarios and both scenarios must have been run.

Model Default Modifications

Total of 0 changes have been made.

PERLND Changes

No PERLND changes have been made.

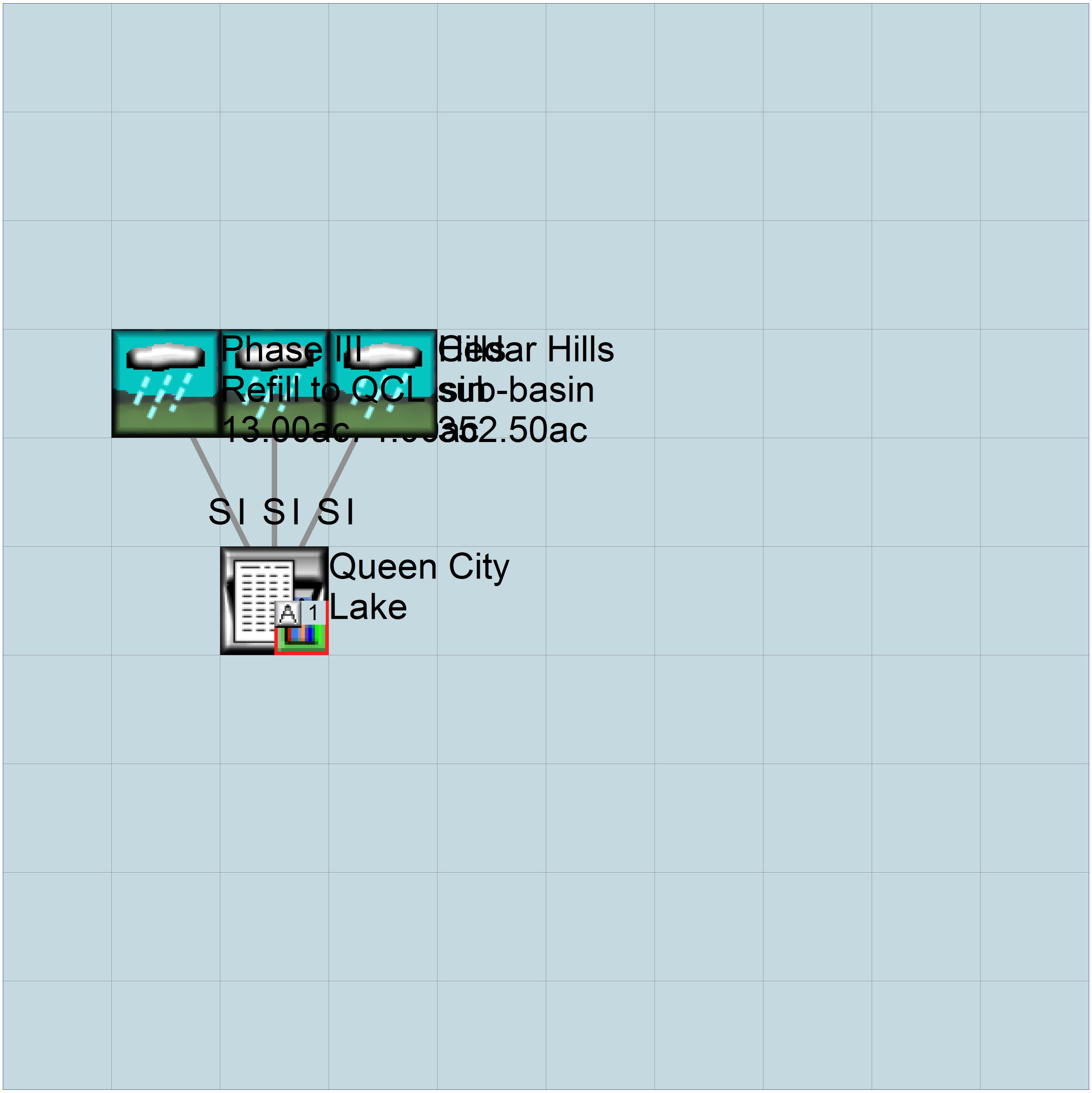
IMPLND Changes

No IMPLND changes have been made.

Appendix

Predeveloped Schematic

Mitigated Schematic



Predeveloped UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                             ***
WDM        26   QCF Phase III - Queen City Lake Outlet Sizing.wdm
MESSU      25   PreQCF Phase III - Queen City Lake Outlet Sizing.MES
           27   PreQCF Phase III - Queen City Lake Outlet Sizing.L61
           28   PreQCF Phase III - Queen City Lake Outlet Sizing.L62
           30   POCQCF Phase III - Queen City Lake Outlet Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND      11
      PERLND      14
      IMPLND      14
      PERLND       2
      IMPLND       1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Cedar Hills dev sub-basin   MAX                    1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                          User  t-series Engl Metr ***
                                          in  out           ***
   11     C, Forest, Mod          1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
    2     A/B, Forest, Mod        1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
   11         0    0    1    0    0    0    0    0    0    0    0    0
   14         0    0    1    0    0    0    0    0    0    0    0    0
    2         0    0    1    0    0    0    0    0    0    0    0    0
```

```
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
   11         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
   11         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
    2         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
   11              0       4.5      0.08       400       0.1       0.5     0.996
   14              0       4.5      0.06       400       0.1       0.5     0.996
    2              0         5         2       400       0.1       0.3     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
   11              0         0         2         2         0         0         0
   14              0         0         2         2         0         0         0
    2              0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                                ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
   11            0.2       0.5      0.35         6       0.5       0.7
   14           0.15       0.4       0.3         6       0.5       0.4
    2            0.2       0.5      0.35         0       0.7       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
   11              0         0         0         0       2.5         1         0
   14              0         0         0         0       2.5         1         0
    2              0         0         0         0         3         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                      in  out           ***
   14      POND                   1    1    1   27    0
    1      ROADS/FLAT             1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
   14         0    0    1    0    0    0
    1         0    0    1    0    0    0
  END ACTIVITY

  PRINT-INFO
```

```
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
   14         0    0    4    0    0    0    1    9
    1         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
   14         0    0    0    0    0
    1         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
   14            400      0.01       0.1       0.1
    1            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
   14              0         0
    1              0         0
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
   14              0         0
    1              0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Cedar Hills dev sub-basin***
PERLND  11                         333     COPY   501     12
PERLND  11                         333     COPY   501     13
PERLND  14                          11     COPY   501     12
PERLND  14                          11     COPY   501     13
IMPLND  14                         8.5     COPY   501     15
Maple Hills sub-basin***
PERLND   2                         128     COPY   501     12
PERLND   2                         128     COPY   501     13
PERLND  14                           8     COPY   501     12
PERLND  14                           8     COPY   501     13
IMPLND   1                          38     COPY   501     15
Phase III to QCL***
PERLND  14                          13     COPY   501     12
PERLND  14                          13     COPY   501     13

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
```

```
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                  ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                        in  out                               ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                       ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *     *  *  *  *  *        *  *  *  *  *        ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50        ***
  <------><--------><--------><--------><--------><--------><-------->        ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                          ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
           *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC      ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC      ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP      ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP      ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY   501 OUTPUT MEAN   1 1     48.4       WDM    501 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK       15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
```

```
  END MASS-LINK   15

END MASS-LINK

END RUN
```

Mitigated UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                              ***
WDM        26   QCF Phase III - Queen City Lake Outlet Sizing.wdm
MESSU      25   MitQCF Phase III - Queen City Lake Outlet Sizing.MES
           27   MitQCF Phase III - Queen City Lake Outlet Sizing.L61
           28   MitQCF Phase III - Queen City Lake Outlet Sizing.L62
           30   POCQCF Phase III - Queen City Lake Outlet Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      PERLND      14
      IMPLND       1
      PERLND      11
      IMPLND      14
      RCHRES       1
      COPY         1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Queen City Lake             MAX                   1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1         1    1
  501         1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                          User  t-series Engl Metr ***
                                          in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
   14     C, Pasture, Mod         1    1    1    1   27    0
   11     C, Forest, Mod          1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
    2         0    0    1    0    0    0    0    0    0    0    0    0
```

```
   14         0    0    1    0    0    0    0    0    0    0    0    0
   11         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   14         0    0    4    0    0    0    0    0    0    0    0    0    1    9
   11         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
   14         0    0    0    0    0    0    0    0    0    0    0
   11         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2             0         5         2       400       0.1       0.3     0.996
   14             0       4.5      0.06       400       0.1       0.5     0.996
   11             0       4.5      0.08       400       0.1       0.5     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2             0         0         2         2         0         0         0
   14             0         0         2         2         0         0         0
   11             0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                                ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2           0.2       0.5      0.35         0       0.7       0.7
   14          0.15       0.4       0.3         6       0.5       0.4
   11           0.2       0.5      0.35         6       0.5       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2             0         0         0         0         3         1         0
   14             0         0         0         0       2.5         1         0
   11             0         0         0         0       2.5         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                       in  out           ***
    1      ROADS/FLAT             1    1    1   27    0
   14      POND                   1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
    1         0    0    1    0    0    0
   14         0    0    1    0    0    0
  END ACTIVITY
```

```
  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
    1         0    0    4    0    0    0    1    9
   14         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
    1         0    0    0    0    0
   14         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
    1            400      0.01       0.1       0.1
   14            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
    1              0         0
   14              0         0
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
    1              0         0
   14              0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Maple Hills sub-basin***
PERLND   2                         128     RCHRES   1      2
PERLND   2                         128     RCHRES   1      3
PERLND  14                           8     RCHRES   1      2
PERLND  14                           8     RCHRES   1      3
IMPLND   1                          38     RCHRES   1      5
Cedar Hills sub-basin***
PERLND  11                         333     RCHRES   1      2
PERLND  11                         333     RCHRES   1      3
PERLND  14                          11     RCHRES   1      2
PERLND  14                          11     RCHRES   1      3
IMPLND  14                         8.5     RCHRES   1      5
Phase III Refill to QCL***
PERLND  14                          13     RCHRES   1      2
PERLND  14                          13     RCHRES   1      3

******Routing******
PERLND   2                         128     COPY     1     12
PERLND  14                           8     COPY     1     12
IMPLND   1                          38     COPY     1     15
PERLND   2                         128     COPY     1     13
PERLND  14                           8     COPY     1     13
PERLND  11                         333     COPY     1     12
PERLND  14                          11     COPY     1     12
IMPLND  14                         8.5     COPY     1     15
PERLND  11                         333     COPY     1     13
PERLND  14                          11     COPY     1     13
PERLND  14                          13     COPY     1     12
PERLND  14                          13     COPY     1     13
```

```
RCHRES   1                            1     COPY   501     17
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                 ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                         in  out                             ***
    1     Queen City Lake         2    1    1    1   28    0    1
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
    1         1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
    1         4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                      ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *     *  *  *  *  *       *  *  *  *  *         ***

    1        0  1  0  0    4  5  0  0  0       0  0  0  0  0       2  2  2  2  2
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50       ***
  <------><--------><--------><--------><--------><--------><-------->       ***
    1             1      0.01       0.0       0.0       0.5       0.0
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                         ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
         *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
    1           0          4.0  5.0  0.0  0.0  0.0       0.0  0.0  0.0  0.0  0.0
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
END SPEC-ACTIONS
FTABLES
  FTABLE      1
   12    5
     Depth      Area    Volume  Outflow1  Outflow2  Velocity  Travel Time***
      (ft)   (acres) (acre-ft)   (cfs)      (cfs)   (ft/sec)    (Minutes)***
  0.000000  0.000000  0.000000  0.000000  0.000000
  0.400000  0.006300  0.000800  0.000000  0.300000
  1.400000  1.870300  0.662700  0.000000  0.800000
  2.400000  4.091600  3.572100  0.000000  1.200000
  3.400000  5.584100  8.390600  0.000000  1.600000
  4.400000  7.107000  14.72080  0.000000  1.800000
```

```
  5.400000  8.532900  22.53000  2.471446  2.000000
  6.400000  9.937100  31.75610  4.623653  3.000000
  7.400000  11.73030  42.57740  6.053783  7.000000
  8.400000  13.12080  54.99650  16.33172  15.70000
  9.400000  14.53770  68.81970  41.95013  15.70000
  10.90000  15.18870  91.11270  58.09619  15.70000
  END FTABLE  1
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC      ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC      ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP      ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP      ENGL    0.76           IMPLND   1 999 EXTNL  PETINP
WDM      2 PREC      ENGL    1.167          RCHRES   1     EXTNL  PREC
WDM      1 EVAP      ENGL    0.76           RCHRES   1     EXTNL  POTEV

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
RCHRES   1 HYDR   RO      1 1        1      WDM   1002 FLOW     ENGL      REPL
RCHRES   1 HYDR   O       1 1        1      WDM   1004 FLOW     ENGL      REPL
RCHRES   1 HYDR   O       2 1        1      WDM   1005 FLOW     ENGL      REPL
RCHRES   1 HYDR   STAGE   1 1        1      WDM   1003 STAG     ENGL      REPL
COPY     1 OUTPUT MEAN    1 1     48.4      WDM    701 FLOW     ENGL      REPL
COPY   501 OUTPUT MEAN    1 1     48.4      WDM    801 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       2
PERLND     PWATER SURO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    2

  MASS-LINK       3
PERLND     PWATER IFWO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    3

  MASS-LINK       5
IMPLND     IWATER SURO       0.083333      RCHRES         INFLOW IVOL
  END MASS-LINK    5

  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK       15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   15

  MASS-LINK       17
RCHRES     OFLOW  OVOL   1                 COPY           INPUT  MEAN
  END MASS-LINK   17


END MASS-LINK

END RUN
```

Predeveloped HSPF Message File

Mitigated HSPF Message File

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JOB NO.	0992002.050.051
JOB NAME	Queen City Farms - Phase III Refill
SUBJECT	Tributary 316A Channel Sizing
CALC BY	MDV — DATE 8/23/2019
CHK BY	KMS — DATE 8/27/2019

Objectives

1 Determine the peak design flow rate for the Tributary 316A channel relocation, using 100-year peak flow from WWHM.
2 Determine maximum and minimum slopes of the relocated channel.
3 Determine the channel dimensions required to convey the peak flow.
4 Determine the minimum and maximum flow velocity in the relocated channel for the 100-year storm.
5 Determine the minimum and maximum flow velocity in the relocated channel for the 2-year storm.
6 Determine the channel dimensions required to reduce the average velocity to 2 fps in the 100-year event.

1. Determine peak design flow rate

Approach Use the flow frequency distribution modeled in WWHM to obtain the design flow for the Tributary 316A channel.

Assumptions Design flow rate will be for 100-year return period from the contributing portion of the Maple Hills Subbasin:
128 acres are outwash (A/B), forest (undeveloped forested area from aerial maps), moderately sloped (5-15%)
38 acres are flat developed areas (impervious)

From WWHM results (see atatched):

Solution The design flow (Q_d) is 36.42 cfs

2. Determine maximum and minimum channel slopes

Approach Create a profile along the proposed channel realignment.

Assumptions Proposed alignment will be as shown on Sheet 8 of the Phase III Site Improvement Plans.

Solution Minimum slope 0.005 ft/ft

3. Determine required channel dimensions to convey 100-year peak flow

Approach Use the Manning Equation to determine the required channel dimensions.

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2}$$ (equation 8)

Where Q = discharge (cfs)

V = velocity (fps)

A = area (sf)

n = Manning's roughness coefficient; see Table D.4- 2 Manning's "n" Values for Pipes

R = hydraulic radius = area/wetted perimeter

S = slope of the energy grade line (ft/ft)

Assumptions

Minimum slope	0.005	ft/ft	
Manning's n	0.04		mountain stream with gravel, cobbles, and few boulders
side slope	2	ft/ft	
bottom width	6	ft	
minimum excavation depth	2	ft	

Reference

Chow, 1959. http://www.fsl.orst.edu/geowater/FX3/help/8_Hydraulic_Reference/Mannings_n_Tables.htm

Conversions

1 ft	=	12	in
π	=	3.14159	

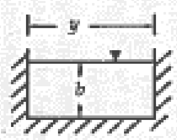
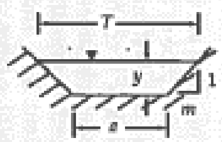
LANDAU ASSOCIATES

JOB NO.	0992002.050.051		
JOB NAME	Queen City Farms - Phase III Refill		
SUBJECT	Tributary 316A Channel Sizing		
CALC BY	MDV	DATE	8/23/2019
CHK BY	KMS	DATE	8/27/2019

Calculation

side slope, m	bottom width, a	Depth, y	Top width, T	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q
ft	ft	ft	ft	ft^2	ft	ft	cfs
2	6	2	14	20	14.94	1.34	63.98

TABLE 6.7. Geometric Relationships for Various Flow Sections.

Section	Area (*A*)	Wetted Perimeter (*P*)	Hydraulic Radius (*R*)	Top Width (*T*)
Rectangular 	by	$b + 2y$	$\frac{by}{b + 2y}$	T
Trapezoidal 	$(a + my)y$	$a + 2y\sqrt{1 + m^2}$	$\frac{(a + my)y}{a + 2y\sqrt{1 + m^2}}$	$a + 2my$

Solution

63.98 cfs (maximum channel capacity at minimum slope)
36.42 cfs (design peak flow rate, 100-year storm, Q_d)
1.76 Calculated Factor of Safety (Q_f/Q_d)

Conclusion This factor of safety is sufficient to carry the 100-year peak flow.

<u>4. Determine flow depth and velocities at the maximum slope.</u>

Approach Average flow velocity = flow divided by cross sectional area. Use Goal Seek to set Q = Qd by varying y.

Assumptions Use same channel side slope and width as above.
Peak flow, Qd 36.42 cfs

Calculation

slope, s	side slope, m	bottom width, a	Depth, y	Top width, T	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q, max slope	V, velocity
ft/ft	ft	ft	ft	ft	ft^2	ft	ft	cfs	ft/s
0.005	2	6	*1.49*	12	13.34	12.65	1.05	36.42	2.73

Conclusion The average channel flow velocity for the minimum channel slope will be 2.7 ft/s in the 100-year storm, with a flow depth of 1.49 feet.

<u>5. Determine flow depths and velocities for the 2-year flow at the minimum slope.</u>

Approach Average flow velocity = flow divided by cross sectional area. Use Goal Seek to set Q = Qd by varying y.

Assumptions Use same channel side slope and width as above.
2-year flow, Qd 17.37 cfs

Calculation

slope, s	side slope, m	bottom width, a	Depth, y	Top width, T	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q, min slope	V, velocity
ft/ft	ft	ft	ft	ft	ft^2	ft	ft	cfs	ft/s
0.005	2	6	*0.99*	10	7.92	10.44	0.76	17.37	2.19

Conclusion The average channel flow velocity for the minimum channel slope will be 2.2 ft/s in the 2-year storm, with a flow depth of 0.99 feet.

6. Determine the channel dimensions required to reduce the average velocity to 2 fps in the 100-year event.

Approach Average flow velocity = flow divided by cross sectional area. Iterate bottom width, a, and use Goal Seek to set Q = Qd by varying y, until v < 2 fps.

Assumptions Use same channel side slope.
Peak flow, Qd 36.42 cfs

Calculation

slope, s	side slope, m	bottom width, a	Depth, y	Top width, T	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q, min slope	V, velocity
ft/ft	ft	ft	ft	ft	ft^2	ft	ft	cfs	ft/s
0.005	2	25	*0.70*	28	18.36	28.11	0.65	36.42	1.98

Conclusion The channel dimensions required to reduce the average velocity to below 2 fps in the 100-year event are a bottom width of 25 ft, 2:1 side slopes, and top width of 28 ft, which corresponds to a flow depth of 0.70 ft.

JOB NO.	0992002.050.051
JOB NAME	Queen City Farms - Phase III Refill
SUBJECT	Tributary 316A Channel Bed Specification
CALC BY	MDV	DATE	8/23/2019
CHK BY	KMS	DATE	8/27/2019

LANDAU ASSOCIATES

Objectives Determine the material and sizing to be used in the Tributary 316A channel bed.

Approach Use the method presented in the Washington Department of Fish and wildlife (WDFW) Water Crossing Design Guidelines, based on flow depth and channel slope.

Reference Barnard, R. J., J. Johnson, P. Brooks, K. M. Bates, B. Heiner, J. P. Klavas, D.C. Ponder, P.D. Smith, and P. D. Powers (2013), Water Crossings Design Guidelines, Washington Department of Fish and Wildlife, Olympia, Washington.
http://wdfw.wa.gov/hab/ahg/culverts.htm

Assumptions Maximum channel slope will be 0.005 between rock check dams
Maximum flow depth will be 1.49 ft at S=0.005

Calculation

Table 3.1. Prediction of water depth for a given maximum particle size that has been moved. Data has been converted to English Units; some values are log-interpolated, adapted from (Costa 1983).

Particle Size (ft)	Slope (ft/ft)										
	0.005	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09	0.1
	Depth (ft)										
0.2	1.2	0.9	0.6	0.5	0.5	0.4	0.4	0.4	0.4	0.4	0.4
0.5	3	2.1	1.5	1.3	1.2	1	1	0.9	0.9	0.9	0.8
1	6	4.1	2.9	2.5	2.2	1.9	1.8	1.8	1.7	1.6	1.5
1.5	8.8	5.9	4.1	3.6	3.1	2.7	2.6	2.5	2.4	2.2	2.1
2	11.3	7.4	5.2	4.5	3.9	3.4	3.2	3.1	2.9	2.8	2.7
2.5	13.6	8.9	6.2	5.4	4.7	4.1	3.9	3.7	3.5	3.3	3.2
3	15.6	10.2	7.1	6.1	5.3	4.6	4.4	4.2	4	3.8	3.6
3.5	17.6	11.4	7.9	6.9	6	5.2	4.9	4.7	4.5	4.3	4.1
4	19.5	12.6	8.7	7.5	6.6	5.7	5.4	5.2	4.9	4.7	4.5
4.5	21.3	13.7	9.4	8.2	7.2	6.2	5.9	5.7	5.4	5.1	4.9
8.1	36.4	23.1	15.6	13.5	11.7	10.1	9.6	9.1	8.6	8.2	7.8
10.5	45.6	28.9	19.4	16.7	14.4	12.5	11.8	11.2	10.6	10	9.5

Based on Table 3.1, with S=0.005 ft/ft and a flow depth between 1.2 and 3 feet, a particle size of 0.5 ft (6 inches) is selected.

WDFW recommends setting the gradation as follows. Select D84 from Table 3.1

D84	6.00	in

Use the following equations to calculate D16, D50, and D100.

D100 = D84/0.4 (Eqn 3.6)	15.00	in
D50 = D84/2.5 (Eqn 3.7)	2.40	in
D16 = D84/8 (Eqn 3.8)	0.75	in

Compare to the WSDOT material specification for 12" Streambed Cobbles, 9-03.11(2):

Percent passing		
99-100	12.00	in
70-90	10.00	in
30-60	5.00	in
10	0.75	in

Conclusion WSDOT 12" Streambed Cobbles are an appropriate material specification for the bed of Tributary 316A.

WWHM2012

PROJECT REPORT

General Model Information

Project Name:	QCF Phase III - Tributary 316A Sizing
Site Name:	Queen City Farms
Site Address:	17825 Cedar Grove Rd SE
City:	Maple Valley
Report Date:	8/23/2019
Gage:	Seatac
Data Start:	1948/10/01
Data End:	2009/09/30
Timestep:	15 Minute
Precip Scale:	1.167
Version Date:	2018/10/10
Version:	4.2.16

POC Thresholds

Low Flow Threshold for POC1:	50 Percent of the 2 Year
High Flow Threshold for POC1:	50 Year

Landuse Basin Data
Predeveloped Land Use

Maple Hill sub-basin (north area only)

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
Pervious Total	128
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	166

Element Flows To:

Surface	Interflow	Groundwater

Mitigated Land Use

Maple Hills sub-basin (north area only)

Bypass:	No
GroundWater:	No
Pervious Land Use	acre
A B, Forest, Mod	128
Pervious Total	128
Impervious Land Use	acre
ROADS FLAT	38
Impervious Total	38
Basin Total	166

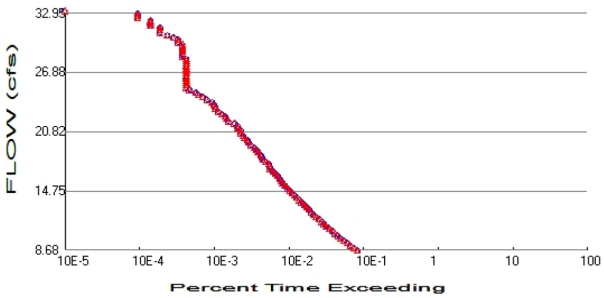
Element Flows To:

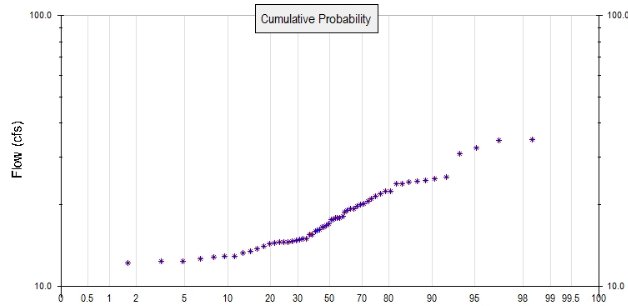
Surface	Interflow	Groundwater

Routing Elements
Predeveloped Routing

Mitigated Routing

Analysis Results
POC 1





+ Predeveloped x Mitigated

Predeveloped Landuse Totals for POC #1
Total Pervious Area: 128
Total Impervious Area: 38

Mitigated Landuse Totals for POC #1
Total Pervious Area: 128
Total Impervious Area: 38

Flow Frequency Method: Log Pearson Type III 17B

Flow Frequency Return Periods for Predeveloped. POC #1

Return Period	**Flow(cfs)**
2 year	17.36959
5 year	22.044684
10 year	25.2983
25 year	29.599134
50 year	32.946245
100 year	36.420295

Flow Frequency Return Periods for Mitigated. POC #1

Return Period	**Flow(cfs)**
2 year	17.36959
5 year	22.044684
10 year	25.2983
25 year	29.599134
50 year	32.946245
100 year	36.420295

Annual Peaks

Annual Peaks for Predeveloped and Mitigated. POC #1

Year	**Predeveloped**	**Mitigated**
1949	22.417	22.417
1950	24.201	24.201
1951	14.502	14.502
1952	12.220	12.220
1953	13.488	13.488
1954	14.049	14.049
1955	16.184	16.184
1956	15.482	15.482
1957	17.593	17.593
1958	14.383	14.383

1959	14.944	14.944
1960	14.460	14.460
1961	14.996	14.996
1962	13.255	13.255
1963	14.831	14.831
1964	14.634	14.634
1965	18.118	18.118
1966	12.071	12.071
1967	21.015	21.015
1968	24.451	24.451
1969	16.534	16.534
1970	16.118	16.118
1971	19.266	19.266
1972	19.966	19.966
1973	12.378	12.378
1974	17.818	17.818
1975	20.109	20.109
1976	13.709	13.709
1977	14.806	14.806
1978	18.824	18.824
1979	25.006	25.006
1980	21.946	21.946
1981	17.871	17.871
1982	25.249	25.249
1983	20.638	20.638
1984	12.902	12.902
1985	17.827	17.827
1986	15.561	15.561
1987	23.976	23.976
1988	14.535	14.535
1989	19.734	19.734
1990	30.835	30.835
1991	24.563	24.563
1992	12.820	12.820
1993	12.401	12.401
1994	12.620	12.620
1995	15.945	15.945
1996	19.290	19.290
1997	16.515	16.515
1998	16.788	16.788
1999	34.498	34.498
2000	16.968	16.968
2001	19.111	19.111
2002	21.530	21.530
2003	17.562	17.562
2004	32.343	32.343
2005	14.530	14.530
2006	12.934	12.934
2007	34.741	34.741
2008	23.923	23.923
2009	22.429	22.429

Ranked Annual Peaks

Ranked Annual Peaks for Predeveloped and Mitigated. POC #1

Rank	Predeveloped	Mitigated
1	34.7413	34.7413
2	34.4978	34.4978
3	32.3426	32.3426

4	30.8345	30.8345
5	25.2486	25.2486
6	25.0060	25.0060
7	24.5630	24.5630
8	24.4508	24.4508
9	24.2009	24.2009
10	23.9764	23.9764
11	23.9227	23.9227
12	22.4292	22.4292
13	22.4174	22.4174
14	21.9461	21.9461
15	21.5295	21.5295
16	21.0153	21.0153
17	20.6376	20.6376
18	20.1092	20.1092
19	19.9657	19.9657
20	19.7336	19.7336
21	19.2895	19.2895
22	19.2657	19.2657
23	19.1108	19.1108
24	18.8242	18.8242
25	18.1184	18.1184
26	17.8708	17.8708
27	17.8272	17.8272
28	17.8184	17.8184
29	17.5930	17.5930
30	17.5621	17.5621
31	16.9679	16.9679
32	16.7875	16.7875
33	16.5341	16.5341
34	16.5154	16.5154
35	16.1842	16.1842
36	16.1175	16.1175
37	15.9447	15.9447
38	15.5609	15.5609
39	15.4818	15.4818
40	14.9959	14.9959
41	14.9437	14.9437
42	14.8311	14.8311
43	14.8058	14.8058
44	14.6341	14.6341
45	14.5346	14.5346
46	14.5300	14.5300
47	14.5022	14.5022
48	14.4602	14.4602
49	14.3828	14.3828
50	14.0492	14.0492
51	13.7093	13.7093
52	13.4880	13.4880
53	13.2546	13.2546
54	12.9339	12.9339
55	12.9016	12.9016
56	12.8200	12.8200
57	12.6200	12.6200
58	12.4006	12.4006
59	12.3776	12.3776
60	12.2201	12.2201
61	12.0712	12.0712

Duration Flows

The Facility PASSED

Flow(cfs)	Predev	Mit	Percentage	Pass/Fail
8.6848	1751	1751	100	Pass
8.9299	1576	1576	100	Pass
9.1749	1412	1412	100	Pass
9.4200	1287	1287	100	Pass
9.6651	1166	1166	100	Pass
9.9101	1052	1052	100	Pass
10.1552	964	964	100	Pass
10.4003	881	881	100	Pass
10.6453	813	813	100	Pass
10.8904	742	742	100	Pass
11.1354	675	675	100	Pass
11.3805	629	629	100	Pass
11.6256	589	589	100	Pass
11.8706	540	540	100	Pass
12.1157	491	491	100	Pass
12.3608	452	452	100	Pass
12.6058	422	422	100	Pass
12.8509	392	392	100	Pass
13.0960	366	366	100	Pass
13.3410	341	341	100	Pass
13.5861	315	315	100	Pass
13.8312	296	296	100	Pass
14.0762	268	268	100	Pass
14.3213	254	254	100	Pass
14.5664	235	235	100	Pass
14.8114	220	220	100	Pass
15.0565	204	204	100	Pass
15.3016	189	189	100	Pass
15.5466	176	176	100	Pass
15.7917	168	168	100	Pass
16.0367	161	161	100	Pass
16.2818	149	149	100	Pass
16.5269	139	139	100	Pass
16.7719	130	130	100	Pass
17.0170	120	120	100	Pass
17.2621	118	118	100	Pass
17.5071	115	115	100	Pass
17.7522	104	104	100	Pass
17.9973	95	95	100	Pass
18.2423	92	92	100	Pass
18.4874	89	89	100	Pass
18.7325	82	82	100	Pass
18.9775	78	78	100	Pass
19.2226	73	73	100	Pass
19.4677	65	65	100	Pass
19.7127	61	61	100	Pass
19.9578	59	59	100	Pass
20.2029	54	54	100	Pass
20.4479	52	52	100	Pass
20.6930	48	48	100	Pass
20.9381	48	48	100	Pass
21.1831	44	44	100	Pass
21.4282	43	43	100	Pass

21.6732	40	40	100	Pass
21.9183	33	33	100	Pass
22.1634	31	31	100	Pass
22.4084	30	30	100	Pass
22.6535	27	27	100	Pass
22.8986	24	24	100	Pass
23.1436	22	22	100	Pass
23.3887	22	22	100	Pass
23.6338	21	21	100	Pass
23.8788	20	20	100	Pass
24.1239	17	17	100	Pass
24.3690	15	15	100	Pass
24.6140	13	13	100	Pass
24.8591	12	12	100	Pass
25.1042	10	10	100	Pass
25.3492	9	9	100	Pass
25.5943	9	9	100	Pass
25.8394	9	9	100	Pass
26.0844	9	9	100	Pass
26.3295	9	9	100	Pass
26.5746	9	9	100	Pass
26.8196	9	9	100	Pass
27.0647	9	9	100	Pass
27.3097	9	9	100	Pass
27.5548	9	9	100	Pass
27.7999	9	9	100	Pass
28.0449	9	9	100	Pass
28.2900	9	9	100	Pass
28.5351	8	8	100	Pass
28.7801	8	8	100	Pass
29.0252	8	8	100	Pass
29.2703	8	8	100	Pass
29.5153	8	8	100	Pass
29.7604	8	8	100	Pass
30.0055	7	7	100	Pass
30.2505	7	7	100	Pass
30.4956	6	6	100	Pass
30.7407	5	5	100	Pass
30.9857	4	4	100	Pass
31.2308	4	4	100	Pass
31.4759	4	4	100	Pass
31.7209	3	3	100	Pass
31.9660	3	3	100	Pass
32.2110	3	3	100	Pass
32.4561	2	2	100	Pass
32.7012	2	2	100	Pass
32.9462	2	2	100	Pass

Water Quality

Water Quality BMP Flow and Volume for POC #1

On-line facility volume:	0 acre-feet
On-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.
Off-line facility target flow:	0 cfs.
Adjusted for 15 min:	0 cfs.

LID Report

LID Technique	Used for Treatment ?	Total Volume Needs Treatment (ac-ft)	Volume Through Facility (ac-ft)	Infiltration Volume (ac-ft)	Cumulative Volume Infiltration Credit	Percent Volume Infiltrated	Water Quality	Percent Water Quality Treated	Comment
Total Volume Infiltrated		0.00	0.00	0.00		0.00	0.00	0%	No Treat. Credit
Compliance with LID Standard 8% of 2-yr to 50% of 2-yr									Duration Analysis Result = Passed

Model Default Modifications

Total of 0 changes have been made.

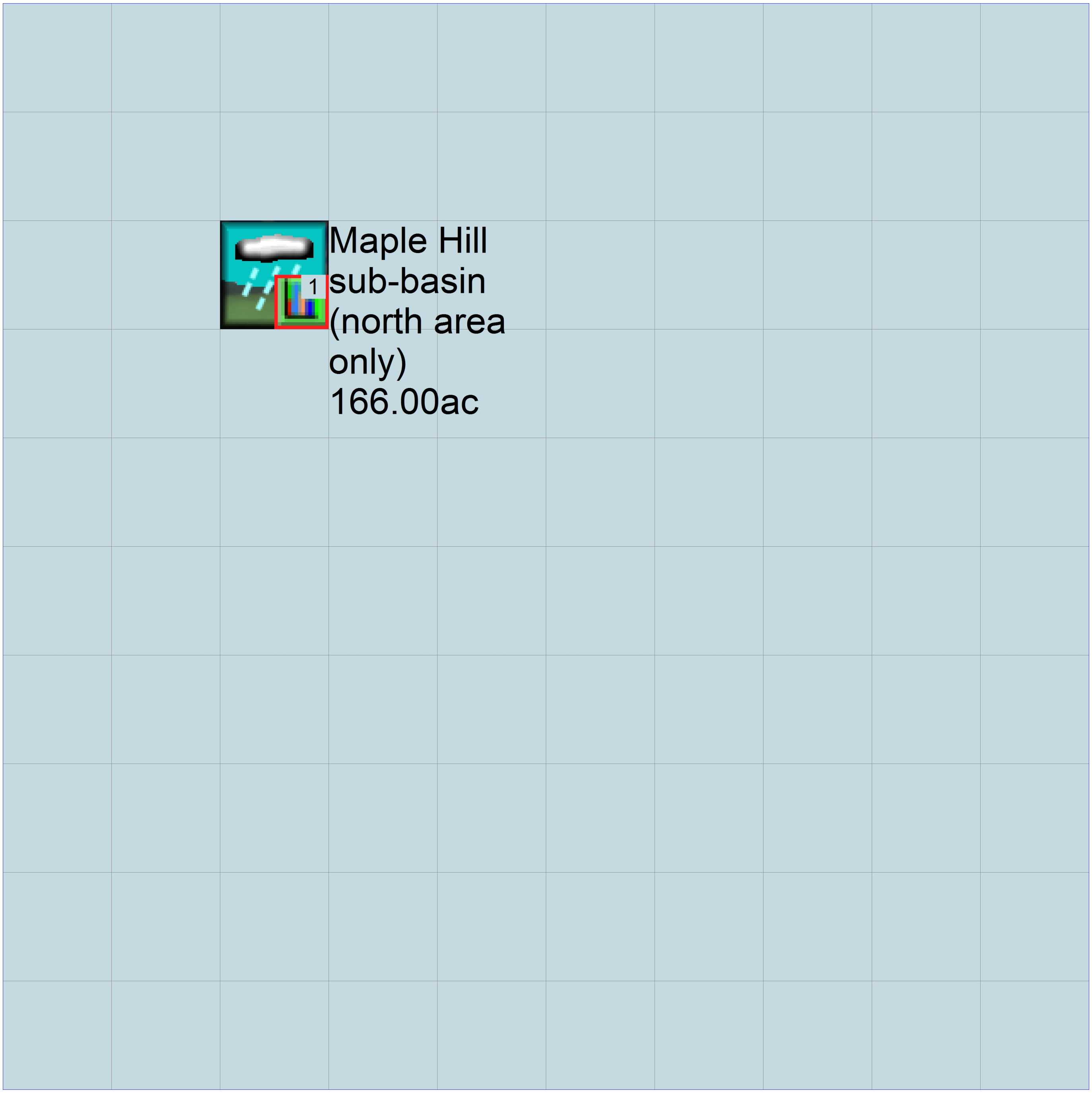
PERLND Changes

No PERLND changes have been made.

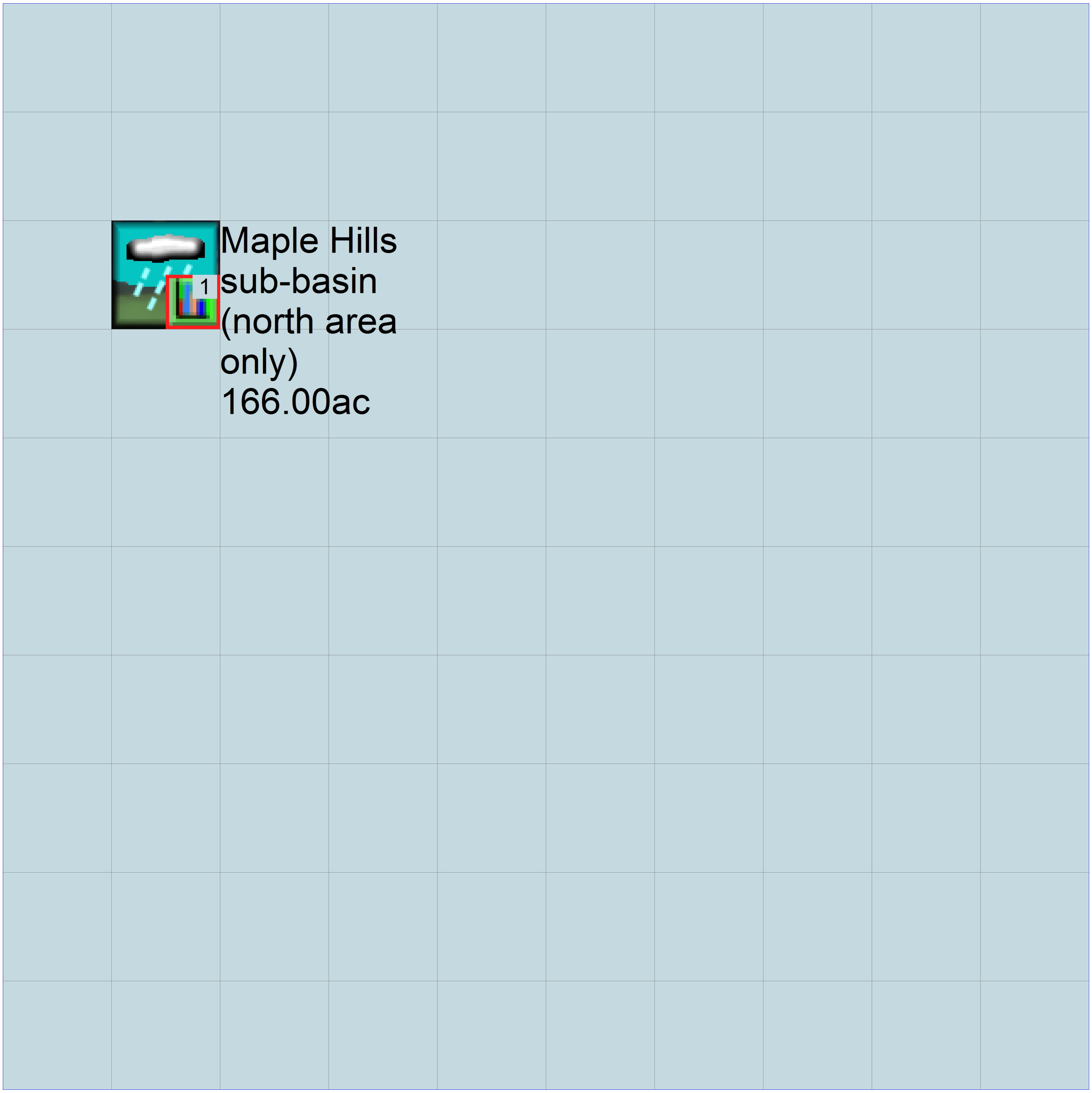
IMPLND Changes

No IMPLND changes have been made.

Appendix
Predeveloped Schematic



Mitigated Schematic



Predeveloped UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                             ***
WDM        26   QCF Phase III - Tributary 316A Sizing.wdm
MESSU      25   PreQCF Phase III - Tributary 316A Sizing.MES
           27   PreQCF Phase III - Tributary 316A Sizing.L61
           28   PreQCF Phase III - Tributary 316A Sizing.L62
           30   POCQCF Phase III - Tributary 316A Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      IMPLND       1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Maple Hill sub-basin (nor   MAX                   1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1        1    1
  501        1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                            User  t-series Engl Metr ***
                                             in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
    2         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO
```

```
  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2             0         5         2       400       0.1       0.3     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2             0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2            0.2       0.5      0.35         0       0.7       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2             0         0         0         0         3         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name-------->   Unit-systems   Printer ***
    # -  #                     User  t-series Engl Metr ***
                                     in  out           ***
    1      ROADS/FLAT             1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
    1         0    0    1    0    0    0
  END ACTIVITY

  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
    1         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
    1         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
    1            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
    1             0         0
```

```
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
    1             0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Maple Hill sub-basin (north area only)***
PERLND   2                       128     COPY   501     12
PERLND   2                       128     COPY   501     13
IMPLND   1                        38     COPY   501     15

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                 ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                           in  out                           ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                      ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *     *  *  *  *  *       *  *  *  *  *         ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50       ***
  <------><--------><--------><--------><--------><--------><-------->       ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                         ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
```

```
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    501 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK       12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK       13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK       15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   15


END MASS-LINK

END RUN
```

Mitigated UCI File

```
RUN

GLOBAL
  WWHM4 model simulation
  START       1948 10 01        END    2009 09 30
  RUN INTERP OUTPUT LEVEL    3    0
  RESUME     0 RUN     1                   UNIT SYSTEM     1
END GLOBAL

FILES
<File>  <Un#>   <-----------File Name------------------------------>***
<-ID->                                                             ***
WDM        26   QCF Phase III - Tributary 316A Sizing.wdm
MESSU      25   MitQCF Phase III - Tributary 316A Sizing.MES
           27   MitQCF Phase III - Tributary 316A Sizing.L61
           28   MitQCF Phase III - Tributary 316A Sizing.L62
           30   POCQCF Phase III - Tributary 316A Sizing1.dat
END FILES

OPN SEQUENCE
    INGRP              INDELT 00:15
      PERLND       2
      IMPLND       1
      COPY       501
      DISPLY       1
    END INGRP
END OPN SEQUENCE
DISPLY
  DISPLY-INFO1
    # -  #<----------Title----------->***TRAN PIVL DIG1 FIL1  PYR DIG2 FIL2 YRND
    1        Maple Hills sub-basin (no   MAX                   1    2   30    9
  END DISPLY-INFO1
END DISPLY
COPY
  TIMESERIES
    # -  #  NPT  NMN ***
    1        1    1
  501        1    1
  END TIMESERIES
END COPY
GENER
  OPCODE
    #    # OPCD ***
  END OPCODE
  PARM
    #    #         K ***
  END PARM
END GENER
PERLND
  GEN-INFO
    <PLS ><-------Name------->NBLKS   Unit-systems   Printer ***
    # -  #                            User  t-series Engl Metr ***
                                            in  out           ***
    2     A/B, Forest, Mod        1    1    1    1   27    0
  END GEN-INFO
  *** Section PWATER***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC ***
    2         0    0    1    0    0    0    0    0    0    0    0    0
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ***************************** PIVL  PYR
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC  *********
    2         0    0    4    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO
```

```
  PWAT-PARM1
    <PLS >  PWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE INFC  HWT ***
    2         0    0    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1

  PWAT-PARM2
    <PLS >      PWATER input info: Part 2         ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    2            0         5         2       400       0.1       0.3     0.996
  END PWAT-PARM2

  PWAT-PARM3
    <PLS >      PWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    2            0         0         2         2         0         0         0
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >     PWATER input info: Part 4                               ***
    # -  #     CEPSC      UZSN      NSUR     INTFW       IRC     LZETP ***
    2            0.2       0.5      0.35         0       0.7       0.7
  END PWAT-PARM4

  PWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
              ran from 1990 to end of 1992 (pat 1-11-95) RUN 21 ***
    # -  # ***  CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    2            0         0         0         0         3         1         0
  END PWAT-STATE1

END PERLND

IMPLND
  GEN-INFO
    <PLS ><-------Name------->   Unit-systems   Printer ***
    # -  #                    User  t-series Engl Metr ***
                                    in  out           ***
    1      ROADS/FLAT            1    1    1   27    0
  END GEN-INFO
  *** Section IWATER***

  ACTIVITY
    <PLS > ************* Active Sections ****************************
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL   ***
    1         0    0    1    0    0    0
  END ACTIVITY

  PRINT-INFO
    <ILS > ******** Print-flags ******** PIVL  PYR
    # -  # ATMP SNOW IWAT  SLD  IWG IQAL    *********
    1         0    0    4    0    0    0    1    9
  END PRINT-INFO

  IWAT-PARM1
    <PLS >  IWATER variable monthly parameter value flags  ***
    # -  # CSNO RTOP  VRS  VNN RTLI     ***
    1         0    0    0    0    0
  END IWAT-PARM1

  IWAT-PARM2
    <PLS >      IWATER input info: Part 2         ***
    # -  # ***  LSUR     SLSUR      NSUR     RETSC
    1            400      0.01       0.1       0.1
  END IWAT-PARM2

  IWAT-PARM3
    <PLS >      IWATER input info: Part 3         ***
    # -  # ***PETMAX    PETMIN
    1            0         0
```

```
  END IWAT-PARM3

  IWAT-STATE1
    <PLS > *** Initial conditions at start of simulation
    # -  # ***  RETS      SURS
    1            0         0
  END IWAT-STATE1

END IMPLND

SCHEMATIC
<-Source->                  <--Area-->     <-Target->   MBLK   ***
<Name>   #                  <-factor->     <Name>   #   Tbl#   ***
Maple Hills sub-basin (north area only)***
PERLND   2                        128      COPY   501     12
PERLND   2                        128      COPY   501     13
IMPLND   1                         38      COPY   501     15

******Routing******
END SCHEMATIC

NETWORK
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
COPY   501 OUTPUT MEAN   1 1   48.4        DISPLY   1     INPUT  TIMSER 1



<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   #        <Name> # #<-factor->strg <Name>   #   #        <Name> # #  ***
END NETWORK

RCHRES
  GEN-INFO
    RCHRES       Name        Nexits   Unit Systems   Printer                 ***
    # -  #<------------------><---> User T-series  Engl Metr LKFG            ***
                                           in  out                           ***
  END GEN-INFO
  *** Section RCHRES***

  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # HYFG ADFG CNFG HTFG SDFG GQFG OXFG NUFG PKFG PHFG ***
  END ACTIVITY

  PRINT-INFO
    <PLS > ***************** Print-flags ****************** PIVL  PYR
    # -  # HYDR ADCA CONS HEAT  SED  GQL OXRX NUTR PLNK PHCB PIVL  PYR  *********
  END PRINT-INFO

  HYDR-PARM1
    RCHRES  Flags for each HYDR Section                                      ***
    # -  #  VC A1 A2 A3  ODFVFG for each *** ODGTFG for each     FUNCT  for each
            FG FG FG FG  possible  exit  *** possible  exit      possible  exit
             *  *  *  *     *  *  *  *  *       *  *  *  *  *          ***
  END HYDR-PARM1

  HYDR-PARM2
    # -  #    FTABNO       LEN     DELTH     STCOR        KS      DB50       ***
  <------><--------><--------><--------><--------><--------><-------->       ***
  END HYDR-PARM2
  HYDR-INIT
    RCHRES  Initial conditions for each HYDR section                         ***
    # -  # ***   VOL     Initial  value  of COLIND     Initial  value  of OUTDGT
          *** ac-ft     for each possible exit       for each possible exit
  <------><-------->     <---><---><---><---><---> *** <---><---><---><---><--->
  END HYDR-INIT
END RCHRES

SPEC-ACTIONS
```

```
END SPEC-ACTIONS
FTABLES
END FTABLES

EXT SOURCES
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
WDM      2 PREC     ENGL    1.167          PERLND   1 999 EXTNL  PREC
WDM      2 PREC     ENGL    1.167          IMPLND   1 999 EXTNL  PREC
WDM      1 EVAP     ENGL    0.76           PERLND   1 999 EXTNL  PETINP
WDM      1 EVAP     ENGL    0.76           IMPLND   1 999 EXTNL  PETINP

END EXT SOURCES

EXT TARGETS
<-Volume-> <-Grp> <-Member-><--Mult-->Tran <-Volume-> <Member> Tsys Tgap Amd ***
<Name>   #        <Name> # #<-factor->strg <Name>   # <Name>    tem strg strg***
COPY     1 OUTPUT MEAN   1 1     48.4      WDM    701 FLOW     ENGL      REPL
COPY   501 OUTPUT MEAN   1 1     48.4      WDM    801 FLOW     ENGL      REPL
END EXT TARGETS

MASS-LINK
<Volume>   <-Grp> <-Member-><--Mult-->     <Target>       <-Grp> <-Member->***
<Name>            <Name> # #<-factor->     <Name>                <Name> # #***
  MASS-LINK        12
PERLND     PWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   12

  MASS-LINK        13
PERLND     PWATER IFWO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   13

  MASS-LINK        15
IMPLND     IWATER SURO       0.083333      COPY           INPUT  MEAN
  END MASS-LINK   15


END MASS-LINK

END RUN
```

Predeveloped HSPF Message File

Mitigated HSPF Message File

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LANDAU ASSOCIATES

JOB NO.	0992002.050.051		
JOB NAME	Queen City Farms - Phase III Refill		
SUBJECT	Channel sizing		
CALC BY	MDV	DATE	8/29/2019
CHK BY	KMS	DATE	8/29/2019

Objectives

1 Determine the design peak flow rate for site stormwater management
2 Determine if the V-ditch and rock lined ditches have the capacity for peak flows calculated in 1

1. Determine peak design flow rate to V-ditch channels.

Approach

Use the Rational Method, as explained in the 2016 King County Surface Water Design Manual, to determine required peak flow rate (Q,peak) for design of channels

Q,peak = C * I * A

where	C	curve number
	I	rainfall intensity, inches per hour
	A	drainage area, acres

Calculation

Total drainage area to each channel, A	**32.00**	**ac**	largest area draining to one ditch (after completion of refill)
Total Impervious	-	ac	
Total pervious	32.00	ac	

LANDAU ASSOCIATES

JOB NO.	0992002.050.051
JOB NAME	Queen City Farms - Phase III Refill
SUBJECT	Channel sizing
CALC BY	MDV
DATE	8/29/2019
CHK BY	KMS
DATE	8/29/2019

C value

Land Cover	C Value	Area (ac)	C*A
Pasture	0.2	32.00	6.40
	Total	32.00	6.40

Note: C values based on Table 3.2.1.A from the 2016 King County Surface Water Design Manual

Composite C, C_c **0.20** equivalent C value for 1 acre

Peak Rainfall Intensity, i

Time of Concentration, t_c
(based on KCSW Manual section 3.2.1, Tables 3.2.1.B and 3.2.1.C)

Segment	Distance (ft)	Slope of Flowpath, S_o (ft/ft)	Land Cover Category	Time of Concentration Velocity Factor, k_R	Average Velocity Across the Land Cover, V (ft/s)	Travel Time for Each Segment, T_t (min)
L1	100	0.25	Short grass pasture and lawns	7	3.50	28.6
					Time of Concentration, Tc (min)	28.6
					Within Equation Limitations ($6.3 \le t_c \le 100$)?	YES

Return Period, R	a_R	b_R	Unit Peak Rainfall Intensity Factor, i_R	Total Precip at Project Site for Given Return Period, P_R (in)	**Peak Rainfall Intensity, I_R**
25 yr	2.66	0.65	0.301	3.7	**1.114**

Solution Using the above values of C, i, and A, the peak flow is thus:

C	0.20
I, 25-year	1.114
A	32.00
Q,peak (25-year)	**7.13 cfs**

<u>2a. Determine flow capacity of V-ditch</u>

Approach Use the Manning Equation to determine the maximum channel flow capacity.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \qquad (4\text{-}2)$$

where
Q = discharge (cfs)
V = velocity (fps)
A = area (sf)
n = Manning's roughness coefficient; see Table 4.2.1.D below
R = hydraulic radius = area/wetted perimeter (ft)
S = slope of the energy grade line (ft/ft)

LANDAU ASSOCIATES

JOB NO.	0992002.050.051		
JOB NAME	Queen City Farms - Phase III Refill		
SUBJECT	Channel sizing		
CALC BY	MDV	DATE	8/29/2019
CHK BY	KMS	DATE	8/29/2019

Assumptions

Channel side slope	2	:1
Channel flow depth	1.5	ft
Slope	0.010	ft/ft
Manning's n	0.035	Table 4.4.1.B
Bottom width	2	ft
Top flow width	8.0	ft

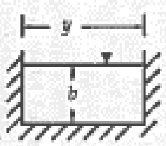
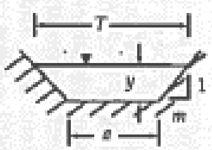
Conversions

1 ft =	12	in
π =	3.141592654	

Calculation

side slope, m	bottom width, a	Depth, y	Top width, T	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q
ft	ft	ft	ft	ft^2	ft	ft	cfs
2	2	1.5	8.0	7.50	8.71	0.86	54.07

TABLE 6.7. Geometric Relationships for Various Flow Sections.

Section	Area (A)	Wetted Perimeter (P)	Hydraulic Radius (R)	Top Width (T)
Rectangular 	by	$b + 2y$	$\frac{by}{b + 2y}$	T
Trapezoidal 	$(a + my)y$	$a + 2y\sqrt{1 + m^2}$	$\frac{(a + my)y}{a + 2y\sqrt{1 + m^2}}$	$a + 2my$

Solution

54.07 cfs (maximum channel capacity)
7.13 cfs (design peak flow rate, 25-year storm)
7.6 Calculated Factor of Safety

Conclusion The V-ditch capacity is greater than the peak flow rates for the the 25-year storm runoff flow rates.

2b. Determine flow capacity of rock-lined channel

Approach Use the Manning Equation to determine the maximum channel flow capacity.

Assumptions

Channel side slope	2	:1
Channel flow depth	1	ft, minimum
Slope	0.250	ft/ft, minimum
Manning's n	0.035	Table 4.4.1.B, Constructed Channels (rock lined - smooth and uniform)
Bottom width	1	ft

Conversions

1 ft =	12	in
π =	3.141592654	

Calculation

side slope, m	bottom width, a	Depth, y	Top width, T	Area, A	Wetted Perimeter, P	Hydraulic Radius, R	Q
ft	ft	ft	ft	ft^2	ft	ft	cfs
2	1	1	5	3.00	5.47	0.55	42.77

Solution

42.77	cfs (maximum channel capacity)
7.13	cfs (design peak flow rate, 25-year storm)
6.0	Calculated Factor of Safety

Conclusion The rock-lined channel capacity is greater than the peak flow rates for the the 25-year storm runoff flow rates.