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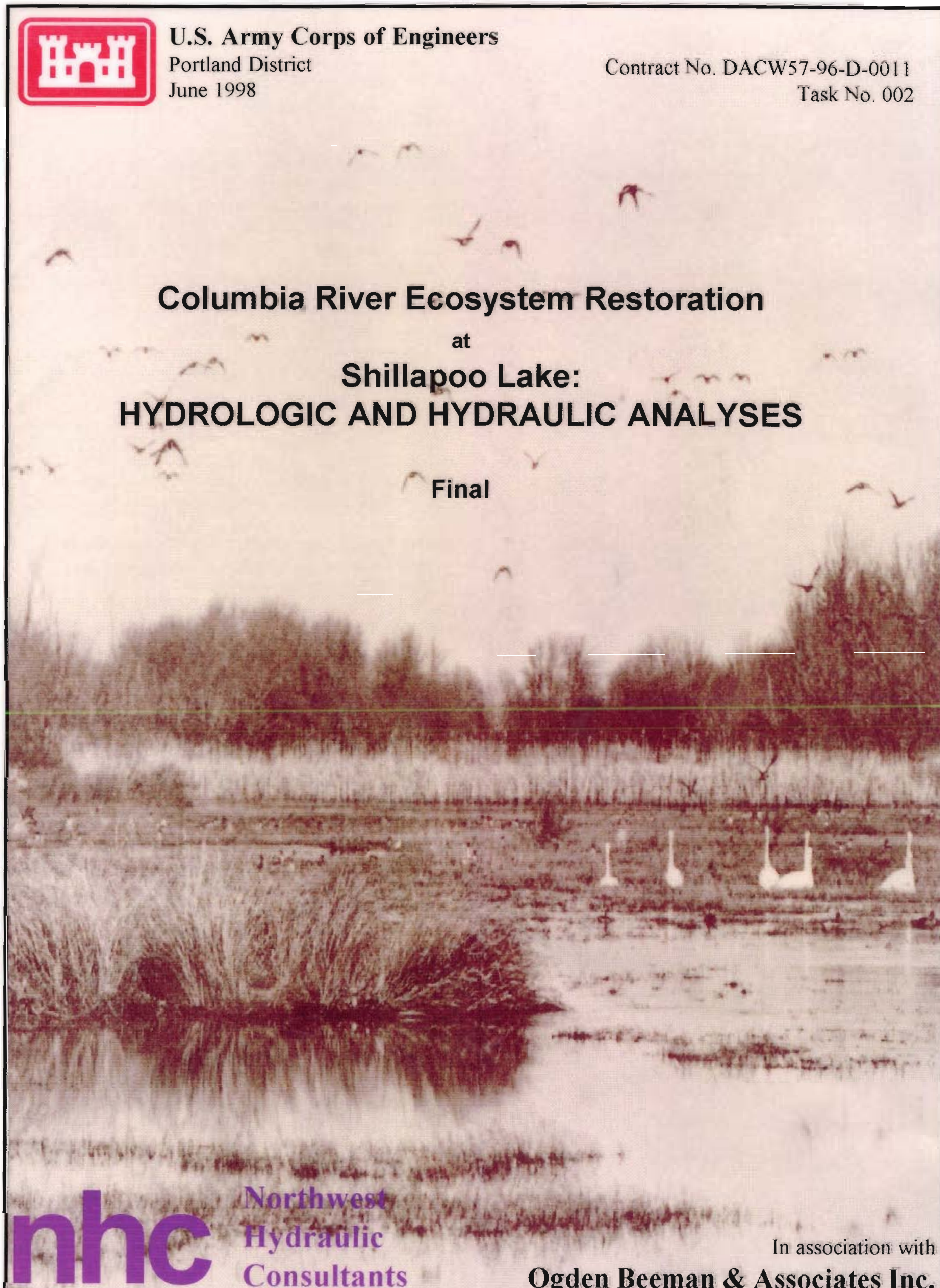
Columbia River Ecosystem Restoration
at
Shillapoo Lake:
HYDROLOGIC AND HYDRAULIC ANALYSES

Final



Northwest Hydraulic Consultants

In association with
Ogden Beeman & Associates Inc.

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Prepared for:

U.S. ARMY CORPS OF ENGINEERS, PORTLAND DISTRICT

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CREDITS AND ACKNOWLEDGMENTS

The work described in this report was undertaken by Northwest Hydraulic Consultants, Inc. (NHC) under contract to the Portland District, U.S. Army Corps of Engineers (COE). Ogden Beeman & Associates, Inc. (OBAI) assisted in performance of the work as subconsultants to NHC. The project was managed for NHC by Malcolm Leytham and for the COE by Mike Knutson. Guidance on design criteria to meet ecosystem restoration goals was provided by Brian Calkins, Washington State Department of Fish and Wildlife (WDFW).

The principal contributors to the report were:

Malcolm Leytham (NHC) – conceptual design and principal author
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Cynthia Lowe (OBAI) – hydrology
Eric VanderMeer (NHC) - graphics

A number of individuals and agencies provided hydrometric data and other information necessary for successful completion of this project. In particular, we wish to acknowledge the assistance of Mr. Glen Dorsey, Clark County Public Works, for providing streamflow data on Burnt Bridge Creek and Salmon Creek.

1.0 INTRODUCTION

1.1 Authority

The work described in this report was authorized by the Portland District, U.S. Army Corps of Engineers under Delivery Order No. 0002 of Contract DACW57-96-D-0011 with Northwest Hydraulic Consultants, Inc.

1.2 Study Objectives

Shillapoo Lake is located in southern Washington State along the flood plain of the lower Columbia River just north of Vancouver Lake, at approximately River Mile (RM) 98. The Shillapoo Lake area is bounded on the north and the east by Lake River, on the west by the Columbia River, and on the south by Vancouver Lake. A location map, which also shows the principal water courses and water bodies of interest in this work, is provided in Figure 1.1.

Shillapoo Lake was historically interconnected with the Columbia River, Lake River, and Vancouver Lake, and fluctuated seasonally with the rise and fall of the Columbia. It was drained and cut off from daily interaction with the Columbia River system by a network of levees around 1950. Draining of the lake was accomplished through a network of ditches and a pump. For the past 50 years, the historic lakebed, which is relatively flat and covers an area of approximately 900 acres, has been used for agriculture.

The Washington State Department of Fish and Wildlife (WDFW) intends to restore ephemeral wetland conditions at Shillapoo Lake to enhance wildlife habitat in the area. Most of the area is managed for wintering waterfowl and it is WDFW's objective to enhance the existing habitat for these species. WDFW currently owns approximately one-third of the historic lakebed and hopes to acquire the entire area from private landowners in the near future.

The basis for ecosystem restoration investigated in this work is the re-establishment of hydraulic and hydrologic connections between and amongst the Columbia River, Lake River, and Shillapoo Lake. In addition to hydraulic connections to the Columbia River system, it is envisaged that restoration of Shillapoo Lake will include splitting the lake area into a number of wetland cells, with water control facilities to allow separate management of the water level regime within each of the individual cells to the extent possible.

The objective of the work described in this report was to provide hydrologic and hydraulic analyses and feasibility level hydraulic design for re-establishing hydraulic connections to Shillapoo Lake and for managing the water level regime within Shillapoo Lake to provide favorable conditions for wetland restoration.

1.3 Study Approach and Organization of Report

The approach adopted for this study was to estimate Shillapoo Lake levels for alternative hydraulic connections and alternative water management scenarios based on water levels in the Columbia River, Vancouver Lake, Lake River, and the connecting channel network. Identification of a preferred alternative for wetland restoration was accomplished through the following steps.

1. The general hydraulic and hydrologic setting of Shillapoo Lake was established, including an approximate characterization of the water level regime external to Shillapoo Lake as determined through analysis of long term stage records for the Columbia River. (Chapter 2)

2. A one-dimensional, unsteady-flow (UNET) model of the Columbia River/Lake River/Shillapoo Lake system was developed using, as a basis, a pre-existing UNET model of the Lower Columbia River. (Chapter 3)

3. Several base level water management alternatives for Shillapoo Lake were defined and investigated to a reconnaissance level using the UNET model. The base level alternatives varied depending on the nature and extent of their connections to the Columbia River system. Analysis of the base level alternatives concentrated on characterizing the ability to fill and drain Shillapoo Lake given the prevailing water level regimes in the surrounding water bodies (the Columbia River and Lake River). Minimal consideration was given to water management internal to Shillapoo Lake (i.e. management of the water level regime in individual wetland cells within Shillapoo Lake) in examination of the base level alternatives. (Chapter 4)

4. A single preferred or recommended alternative was developed from evaluation of the base level alternatives. The preferred alternative was designed to a feasibility level, including: channel design and layout; interior cell design and layout; hydraulic connections to the Columbia River system; operational descriptions of hydraulic controls; and characterization of the water level regime within Shillapoo Lake. (Chapter 5)

2.0 HYDRAULIC AND HYDROLOGIC SETTING

This Chapter describes the general hydraulic and hydrologic setting for the restoration of wetland conditions at Shillapoo Lake. Existing hydraulic and hydrologic conditions at Shillapoo Lake are first described and then the target water level regime for restoration, local meteorological conditions, and the water level regime on the Columbia River external to Shillapoo Lake are discussed.

2.1 Existing Conditions at Shillapoo Lake

The Shillapoo Lake study area is bounded on the west by the Columbia River, on the south by Buckmire Slough and Vancouver Lake, and on the east by Lake River. The northern boundary of the study area is delimited by a cross dike which runs east-west just south of Round Lake and which connects the west bank Lake River levee to the Columbia River levee. A topographic map of the study area is provided as Plate 1. Topographic mapping was provided by the Portland District COE on the basis of surveys performed by the Portland District COE during the period of December 1997 through March 1998 supplemented with a survey of WDFW land in the southern portion of the lake bed performed by Forsight Survey in October 1995 for Ducks Unlimited. Plate 1 was prepared by overlaying the topographic mapping provided by the COE on digital versions of the US Geological Survey 1:24,000 quadrangle maps for Vancouver and Sauvie Island to show the locations of the surrounding major hydrographic features (Columbia River, Lake River, Vancouver Lake, etc).

Although there are a number of areas of natural high ground in the study area, much of Shillapoo Lake is surrounded by levees that protect the area from flooding from the Columbia River and Lake River. Elevations within the study area range from a low of about +1 ft NGVD[1] at the lowest point in the historic lake bed to high points which range from about +23 ft to +28 ft along the surrounding levee system. Elevations below +1 ft to as low as about -6 ft are found along drainage ditches and drainage sumps within the lakebed. The total area of Shillapoo Lake to elevation +25 ft is approximately 1,700 acres, with the historic lakebed itself (to about elevation +7 ft) occupying approximately 950 acres. Stage/storage data for the lake were determined from the topographic mapping provided by the COE and are shown in Figures 2.1 and 2.2

Under present day conditions, Shillapoo Lake is cut off from interaction with the Columbia River system by the network of levees. Interior runoff from the area bounded by the levees is generated by rainfall and by springs and other groundwater inflow. Two significant sources of groundwater inflow have been identified at Shillapoo Lake; a perennial spring at the southern end of the study area, and an artesian well (Brian Calkins, WDFW, personal communication, April 1998). In addition to these, additional groundwater inflows to the lake area can be expected, especially when Columbia River water levels are high. No data are available on interior runoff rates or volumes. Estimation of interior runoff rates for modeling purposes is discussed in Section 3.2.6.

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[1] Unless otherwise stated, all elevations provided in this report are to the National Geodetic Vertical Datum of 1929 (NGVD).

The Shillapoo Lake area is currently drained by a network of ditches, as shown on Plate 1, a pump (referred to here as the "expulsion pump") and two tide gated culverts (culverts fitted with flap gates).

The expulsion pump has a reported capacity of approximately 10,000 gpm (22.3 cfs) (Calkins, WDFW, personal communication, April 1998) and discharges into Lake River at about RM 10.5. Once crops have been harvested in the fall, the expulsion pump is shut down for the winter. As a result, interior runoff accumulates in the lake bottom, reportedly reaching depths of 4 ft or more in a typical winter. More specific data on interior water levels are not available. The pump is reactivated in late winter (mid-January in 1998) to allow time to drain the lake area and dry out the soil sufficiently for farming operations to start in the spring. A 42-inch culvert and tide gate are located adjacent to the pump station. The culvert has an approximate invert elevation of +1.7 ft and provides for gravity discharge from Shillapoo Lake as water levels in Lake River allow.

The second existing outlet from Shillapoo Lake is via a 36-inch culvert and tide gate under the northern cross dike, the east-west levee which forms the northern boundary of the study area. The culvert, with an approximate invert elevation of +5.6 ft discharges to a vestigial historic channel connecting the north end of Shillapoo Lake to Lake River at about RM 8.7. The channel connection to Lake River is heavily silted and no channel survey information is available.

2.2 Target Water Level Regime and Approximate Water Supply Needs

Central to evaluation of project alternatives is the ability to maintain water level regimes favorable to the restoration of targeted wetland plant communities. Discussions were held with the COE, WDFW, and Ducks Unlimited staff to identify a target water level regime that would meet the ecosystem restoration goals for the project. These goals include enhancing habitat for ducks and other wintering waterfowl, and re-introducing ephemeral wetland plants that will encourage the development of these waterfowl populations by providing forage (food) and protection. The principal characteristics of the target water level regime are as follow:

- *Late fall/winter/spring (November through May).* A maximum water depth of about 2.5 ft should be maintained with the flexibility to increase depths to 5 or 6 ft at the start of the growing season (about mid-March) for Reed Canarygrass control. Winter water depths over most of the restored area should ideally not exceed about 1 ft to maximize overwintering habitat for waterfowl. The flexibility to increase water depths to 5 ft or so in mid-March is intended as one of several possible management options to control Reed Canarygrass (in this case by drowning out at the start of the growing season).

- *Late spring/early summer (Late May/early June through July).* Water levels should be drawn down starting in late May or early June to expose mud flats. Drawdown to allow native moist-soil plant communities (including smart weed and millets) to mature and produce seed could be as late as 1 July. However some of the target wetland species, such as wapato (know also as arrowhead [*Sagitaria latifolia*]), are perennial plants and would not need to produce seed each year to maintain their viability. Wapato are also capable of growing under both day mud flats or wet conditions.

- *Summer/early fall (August through October).* Wetland cells should be allowed to mostly dry up by August and should be maintained dry through to the start of fall rains in October or November when re-flooding of the lake would commence. The project should have the capability to dry areas out more or less completely for Carp control.

The target water level regime is depicted in the rule curve of Figure 2.3. Water control facilities for individual cells should be such that different water management practices can be applied to or rotated through different cells. For example, increases in water levels in mid-March for Reed Canarygrass control should be implementable on a cell-by-cell basis.

Achievement of the target water level regime requires the ability to supply certain volumes of water to and drain similar volumes of water from the lake. The volumes involved depend on the total area to be restored to wetland conditions. For an initial estimate of water supply requirements, it was assumed that all 1,180 acres of land below elevation +10 ft (see Figure 2.1) would be restored to wetland conditions and that normal water depths would average 1.5 ft. (Note that these assumptions imply that the higher northern part of the lake area, which has a minimum elevation of about +7 ft would be restored as wetlands [see Plate 1]). Under this scenario, approximately 1,800 acre ft of water would be required between October 1 and November 1, if the rule curve of Figure 2.3 is to be strictly followed, or between say October 1 and December 31 if more flexibility is allowed.

If one also assumes that in any one year flooding of about one quarter (300 acres) of the restored area is required for Reed Canarygrass control, then an additional 1,050 acre ft of water would be required between about mid-March and April 1, for a total annual requirement of 2,850 acre ft. Assuming that Reed Canarygrass control is required each year, then a total of 2,850 acre ft of water, plus an incremental volume of interior runoff, would have to be drained from the lake between May 1 and July 31 to match the target water level regime. With no Reed Canarygrass control, approximately 1,800 acre ft of water plus an incremental volume of interior runoff, would have to be drained between June 1 and July 31.

2.3 Climatological Conditions

Climatological conditions at Shillapoo Lake are typical of the maritime Pacific Northwest with mild wet winters and warm relatively dry summers. The closest long-term weather station to the project is Vancouver 4 NNE (NWS station 458773), which lies approximately six miles east of Shillapoo Lake. Published station normals for the period 1961 – 1990 (Owenby and

Ezell, 1992a) give mean monthly temperatures at Vancouver 4 NNE ranging from 38.1°F in January to 64.8°F in August, and mean annual precipitation of 41.3 inches.
Observed monthly precipitation amounts from Vancouver 4 NNE for water years 1973 through 1997 are given in Table 2.1.

Monthly precipitation amounts for Portland WSFO AP (NWS station 356751) are shown in Table 2.2 for water years 1994 through 1997. Portland WSFO AP (approximately 11 miles southeast of Shillapoo Lake) is the closest weather station to Shillapoo Lake reporting hourly precipitation data. Hourly precipitation for water year 1994 through 1997 were used for simulation of interior runoff as discussed in Section 3.2.6. The mean annual precipitation at Portland WSFO AP for the period 1961-1990 (Owenby and Ezell, 1992b) is 36.3 inches.

2.4 Analysis of Historic Water Level Data

The ability to fill and drain Shillapoo Lake by gravity to achieve the target water level regime is largely dependent on water levels in the Columbia River. Columbia River water levels obviously determine the ability to fill from the Columbia by gravity. As will be seen in Chapters 3 and 4, water levels in Lake River, one of the potential receiving water bodies for draining Shillapoo Lake, are also controlled to a significant degree by water levels in the Columbia. Hence the ability to drain Shillapoo Lake by gravity is also dictated by water levels in the Columbia. The reliability with which Shillapoo Lake could be filled or drained by gravity was assessed in an approximate manner through analysis of mean daily stage data for the Columbia River at Vancouver (RM 106.5) for the period water years 1973-1990, and 1993-1997, for a total of 23 water years. Missing data precluded analysis of water years 1991 and 1992.

The mean daily stage data used in this analysis are shown as time series plots in Appendix A. To evaluate the ability to fill Shillapoo Lake, the mean daily stage data were analyzed to determine, by water year and month, the number of days in each month that the stage at Vancouver exceeded specified thresholds, for thresholds of +6, +8, +10, +12, and +14 ft NGVD. The results of these analyses are shown in Table 2.3. The data in Table 2.3 are further summarized to show the number of days in seasons October-December, October-March, October-May, and March-April that the stage at Vancouver exceeded the specified thresholds. These latter data are plotted on frequency paper in Figures 2.4 through 2.7.

Similar analyses were performed for the period June-September to assess the ability to drain Shillapoo Lake by gravity. In this case, the data for the months of June through September were analyzed to determine the number of days in each month that the stage at Vancouver was below the specified threshold for thresholds of +10, +8, +6, +4, and +2 ft NGVD. The results, along with a summary for the season June through September, are shown in Table 2.4. The seasonal data are again plotted on frequency paper in Figure 2.8.

To use the data presented in Tables 2.3 and 2.4 and in Figures 2.4 through 2.8 to assess the ability to fill or drain Shillapoo Lake requires transposition from the Vancouver stage gage at RM 106.5 to potential intake or discharge points for Shillapoo Lake. Potential gravity intake points from the Columbia River are between RM 97 and RM 98. Potential discharge locations for draining the lake are along the Columbia River between RM 97 and RM 98 and along Lake River between about RM 9 and RM 11. **UNET modeling described in Chapters 3 and 4, indicates an approximate difference in water levels on the Columbia River between RM 106.5 and RM 98 in the range of 0.5 to 1 ft when the stage at Vancouver is between about +8 and +10 ft. For similar conditions, water levels on Lake River at about RM 9 would be between 1 and 2 ft lower than water levels on the Columbia at Vancouver.** Water level differences are smaller at lower mainstem Columbia River stages. For water levels at Vancouver between +4 ft and +6 ft, water level differences on the Columbia between Vancouver (RM 106.5) and RM 98 and between Vancouver and Lake River at RM 9 can both be expected to be less than about 0.5 ft.

The data presented in Tables 2.3 through 2.4 and Figures 2.4 through 2.8 reflect the effects of both hydrologic conditions and reservoir operations on the Columbia River system over the past 20 years or so. It is possible that future Columbia River reservoir operations will produce periods of higher flows to benefit fisheries production and may result in somewhat different stage/duration characteristics than seen in the recent record.

3.0 BASELINE HYDRAULIC MODEL DEVELOPMENT

Detailed analysis of the water level regime of Shillapoo Lake and surrounding water courses was accomplished by developing a UNET model (a one-dimensional, unsteady-flow model) of the system. The basis for the modeling effort described here was a pre-existing UNET model of the Lower Columbia River developed by the Portland District COE. The development and calibration of this original model is documented in an internal report prepared by the Portland District COE and reproduced in Appendix B. The original model was modified for this project to incorporate Shillapoo Lake and interconnections between the Columbia River, Vancouver Lake, and Lake River as described in this chapter.

3.1 Existing Condition Hydraulic Model Configuration

The original UNET model of the Lower Columbia River extends along the mainstem Columbia from Astoria upstream to Bonneville Dam. This original model was modified for this project by incorporating additional detail in the Shillapoo Lake area, including Shillapoo Lake and interconnections between the Columbia River, Vancouver Lake, and Lake River.

A schematic of the branch network for the original UNET model is provided in Figure 3.1, and a simplified branch network schematic of the baseline existing condition model as modified for this project is shown in Figure 3.2, with an enlarged schematic of the portion of the model covering the Shillapoo Lake and Vancouver Lake connections provided as an inset. Note in Figure 3.2 that the branch numbering system is different from that used in the original model; the numbering system was modified for this project so as to maintain acceptable computational efficiency after the incorporation of additional model features.

The principal changes to the original model to develop the baseline existing condition model are described below:

Shillapoo Lake

The principal physical features of Shillapoo Lake were described in Section 2.1 and a topographic map of the project area was provided as Plate 1.

The original UNET model was modified as follows:

- For the purposes of reconnaissance level simulations, Shillapoo Lake was included as a single storage element. As discussed in Section 2.1, stage/storage data for the lake were developed from surveys and mapping data provided by the Portland District COE and are shown in Figures 2.1 and 2.2.

- The "expulsion" pump which currently serves to drain Shillapoo Lake and which discharges to Lake River at about RM 10.5 was added to the model. The expulsion pump was assumed to have a constant capacity of 22.3 cfs. For the purpose of model calibration (see Section 3.3) the pump was assumed to operate from January 15 through to the end of the water year whenever water levels interior to Shillapoo Lake exceeded 0 ft, corresponding approximately to the highest level that could be maintained in the pump station forebay and ditch system without flooding farmland.

- The 42-inch culvert and tide gate located adjacent to the expulsion pump were added. The culvert and tide gate, which have an invert elevation of about +1.7 ft, provide for gravity discharge from Shillapoo Lake to Lake River as water levels in Lake River allow. A rating for this culvert as a function of head difference between Shillapoo Lake and Lake River is shown in Figure 3.3.

- The 36-inch culvert and tide gate which discharges to a historic channel connecting the north end of Shillapoo Lake to Lake River at about RM 8.7 was added. The culvert and tide gate, which has an invert elevation of about +5.6 ft, also provide for gravity discharge from Shillapoo Lake to Lake River as water levels in Lake River allow.

Vancouver Lake Flushing Channel

The Vancouver Lake Flushing Channel, which connects the Columbia River to Vancouver Lake, has two important components: the channel itself, and the control structure where the channel enters Vancouver Lake. The original UNET model was modified to include the Flushing Channel and control structure as follows

- The Flushing Channel was represented in the model (branch 15 in Figure 3.2) as having a constant trapezoidal section with a bottom width of about 100 ft, 3H:1V side slopes, and an invert elevation of -8 ft. The channel geometry and invert elevations were taken from 1981 design drawings by BE&C Engineers Inc.

- The Flushing Channel control structure comprises twin 84-inch diameter concrete pipes with invert elevations of –5.5 ft, fitted with flap gates to prevent backflow from Vancouver Lake into the Columbia River. A rating for the structure established as a function of head difference between the Flushing Channel and Vancouver Lake is provide in Figure 3.4. The control structure is also fitted with slide gates which are used to prevent flows from the Columbia River to Vancouver Lake when Columbia River levels are high, so as to control flooding around Vancouver Lake. Under current operations, the slide gates are closed when the Columbia River reaches +16 ft at the Port of Vancouver stage gage at about RM 104 (Walter Morey, Port of Vancouver, personal communication, April 1998). Information on the Flushing Channel control structure was obtained from 1981 design drawings and augmented by conversations with staff from the Port of Vancouver, which is responsible for operating the facility

Vancouver Lake

Vancouver Lake was added as a storage element. No detailed surveys or bathymetry were available for Vancouver Lake or its immediate surrounding area. Approximate stage/storage data were therefore computed based on the lake surface area as determined from the 1990 US Geological Survey 1:24,000 quadrangle map for Vancouver. The approximate stage/storage data are shown in Figure 3.5.

Lake River

Lake River (shown as UNET branches 19, 20 and 24 in Figure 3.2) and Bachelor Island Slough (UNET branch 23) were added to complete the existing connections between the Columbia River and Vancouver Lake, based upon in-channel cross-section data provided by the Portland District USACE. Twenty-one channel cross-section of Lake River and Bachelor Island Slough were obtained during the period march 10-16, 1998. Cross-sections were sounded from a boat-mounted depth finder and cross-sectional distances measured by laser with a target located on the shore. Horizontal datum of the cross-section was established at a single point on the shore with a hand-held GPS locator for each cross-section. Vertical datum was established from calculated water surface profiles at the time of each cross-sectional survey utilizing the Columbia River at St. Helens gage, the Columbia River at Vancouver gage and Lake River at Felida Moorage gage. The in-channel sections were extended to cover out-of-bank areas at selected locations using the 1990 USGS 1:24,00 quadrangle maps of Vancouver and Ridgefield.

3.2 Time Series Boundary Condition Data and System Hydrology

The original (March 1998) Lower Columbia River UNET model is driven by the following time series boundary condition data (see Figures 3.1 and 3.2):

- Columbia River at Bonneville Dam inflow
- Washougal River lateral inflow
- Sandy River lateral inflow
- Willamette River at Portland inflow
- Lewis River lateral inflow
- Kalama River lateral inflow
- Cowlitz River at Castle Rock inflow
- Columbia River at Astoria stage

The modification of the baseline existing condition UNET model for the present study incorporated the following additional time series boundary condition data and input:

- Inflows to Vancouver Lake (primarily from Burnt Bridge Creek)
- Salmon Creek inflows to Lake River

- Other local inflows to Lake River (including Whipple, Flume, and Gee Creeks)
- Shillapoo Lake precipitation and interior runoff

Time series boundary condition data for UNET modeling were developed at an hourly time step for the above locations for the following periods of historic record:

- 1 October 1994 through 30 September 1995 (water year 1995)
- 1 February 1996 through 31 August 1996 (water year 1996, February 1996 flood through summer)
- 1 April 1997 through 31 August 1997 (water year 1997, spring freshet through summer)
- 1 February 1998 - 31 March 1998 (model calibration period)

Hourly data were already available for February through June 1996 and for April through June 1997 at the boundary condition locations in the original model. With the exception of data from the Lewis and Kalama Rivers, these data were used directly in the present study. Lewis River data from the original model, believed to have been based on reported releases from Merwin Dam, were determined to be unreliable. Lewis River hourly flows for these months were therefore determined from the mean daily flows at the USGS streamgage on the Lewis River at Ariel (just downstream from Merwin Dam). Kalama River hourly flows were estimated as a percentage of Lewis River flows, as in the original model, using an adjustment factor provided by the Portland District COE (see Section 3.2.4).

In addition to the periods of historic data, time series boundary condition data were also synthesized for a hypothetical low water year (dry year), nominally referred to here as water year 1999. The data for water years 1995, 1996, 1997, and the hypothetical dry year were developed to provide a basis for detailed UNET simulations of project alternatives, as described in Chapter 4. The data for February and March 1998 were developed primarily for calibration of the modified existing condition UNET model, as described in Section 3.3.

Derivation of the boundary condition data for the historic periods of record are discussed in more detail in the following report Sections 3.2.1 through 3.2.6. Synthesis of the hypothetical low water year data is discussed in Section 3.2.7. Except where noted, all gage data were provided by the Portland District Corps of Engineers and were taken from the Columbia River Operational Hydromet System (CROHMS) database.

3.2.1 Columbia River at Bonneville Dam

Hourly flows for the Columbia River at Bonneville Dam were obtained from Bonneville Dam gage records. The available record included several short periods of missing data (no more than a few hours at a time) which were filled by linear interpolation

3.2.2 Willamette River at Portland

Hourly Willamette River flows at Portland were computed, using a methodology provided by the Portland District COE, by combining flows from: the Willamette River at Oregon City, Clackamas River at its mouth, Johnson Creek at its mouth, and local inflows to the Willamette River. Hourly flows for the Willamette River at Oregon City were taken from the gage at Oregon City. Hourly flows for the Clackamas River at its mouth were estimated as hourly flows recorded at the Clackamas River at Estacada gage multiplied by a scaling factor of 1.5. Hourly flows for Johnson Creek at its mouth were assumed to equal mean daily flows recorded by the Johnson Creek at Milwaukie gage. Other ungaged local runoff in Portland was represented as a fraction of the Johnson Creek flows (0.35 x Johnson Creek at Milwaukie).

For the time periods of interest, the Willamette River gage at Oregon City recorded flows only once a day from May through September 1995. In addition, data were missing from the Willamette River gage for April 12-30, 1995 and August 31, 1997, and from the Clackamas River gage for December 8-16, 1994 and June 15-26, 1995. There were no significant gaps in the Johnson Creek record. All missing flow data were estimated by linear interpolation.

3.2.3 Columbia River at Astoria

The stage records for the Columbia River at Astoria served as the downstream boundary condition for the UNET model, however, gage malfunctions that extended for several hours affected nearly every month during the time periods of interest. To create a record that reflected the influence of the tidal cycles, periods of missing data were estimated using the predicted times and elevations of the high and low tides from Nautical Software's "Tides & Currents" computer program (Nautical Software Inc., 1995) which bases its values on harmonic data from the National Oceanic and Atmospheric Administration (NOAA). Astoria stage records were shifted as necessary from the Columbia River Datum (CRD) to the National Geodetic Vertical Datum (NGVD). The adjustment, provided by the Portland District COE, is –3.07 ft at Astoria.

3.2.4 Columbia River Tributaries

Cowlitz River

Hourly inflows from the Cowlitz River were obtained from the Cowlitz River at Castle Rock gage. Hourly data were missing for the periods: July 23 to August 26, 1996; February 2, 3, 10-18, 1998; and March 10-11, 1998. Since no significant hydrologic events occurred in the local area during these periods, missing data were estimated by linear interpolation.

Lewis River

The gage on the Lewis River at Ariel (immediately below Merwin Dam) provided the Lewis River flows in either an hourly or daily format for different times within the periods of interest. The recorded flows were scaled by a factor of 1.271 to represent flows at the river's mouth, using information provided by the Portland District COE. There were no significant gaps in the Ariel data record for the time periods of interest.

Kalama River

As no gage directly records flows on the Kalama River, flows at the mouth of the Kalama River were estimated as a fraction (0.276) of the Lewis River flows recorded at the Ariel gage, with the scaling factor provided by the Portland District COE.

Sandy River

Hourly flows for the Sandy River were obtained from the Sandy River below Bull Run River gage. The recorded flows were scaled by a factor of 1.128 to represent the flows at the river's mouth, with the scaling factor provided by the Portland District COE. Gage malfunctions occured for the periods November 7-14, 17-21, and 26, 1994. As other gages in the local area showed no significant hydrologic events during these periods, missing data were again estimated by linear interpolation.

Washougal River

As no gage directly records the flows on the Washougal River, the flows at the mouth of the Washougal River were estimated as a fraction (0.248) of the Sandy River flows recorded at the Bull Run gage, with the scaling factor provided by the Portland District COE.

3.2.5 Lake River Local Tributary Inflows

Local inflows to Vancouver Lake and Lake River were determined by prorating observed flows on Burnt Bridge Creek at Alki Point (drainage area 29 sq. mi.) and Salmon Creek at Kleinline (drainage area 92 sq. mi.) as follows:

Vancouver Lake Inflows

Hourly inflows to Vancouver Lake were estimated by scaling up hourly flows recorded at the Burnt Bridge Creek at Alki Point gage by a factor of 1.297, determined on the basis of difference in tributary area to account for all local inflow to Vancouver Lake. Discharge records from the Burnt Bridge Creek gage were provided by Clark County Department of Public Works, the owner and operator of the gage. Data missing for the period August 7-8, 1995 were estimated by linear interpolation. Rainfall directly on the surface of

Vancouver Lake was based on precipitation records from the Portland Airport multiplied by a factor of 1.14 (see Section 3.2.6).

Salmon Creek Inflows to Lake River

Hourly discharge data for Salmon Creek were provided for the Kleinline gage by Clark County Department of Public Works, owner and operator of the gage. Data were not available from Salmon Creek for the calibration period (1 February - 31 March 1998) but were estimated by regression against data from Burnt Bridge Creek. Data missing for the periods April 1-2 and 25-30, 1996 were estimated by correlation against records from an upstream gage (Salmon Creek at NE 156th Street) and linear interpolation. The gage data were scaled up by a factor of 1.102, on the basis of difference in tributary area, to represent combined Lake River inflows from Salmon Creek and the local tributary area between Vancouver Lake and Salmon Creek.

Other Lake River Local Inflows

The following scaling factors, determined on the basis of drainage area, were applied to the Burnt Bridge Creek hourly flow record, to account for the remaining tributaries and local inflows to Lake River:

- Whipple Creek (including some direct local inflows to Lake River): 0.434
- Flume Creek (including some direct local inflows to Lake River): 0.114
- Local inflows between Flume Creek and confluence of Bachelor Island Slough: 0.069
- Gee Creek (flows directly into Ridgefield NWR storage area): 0.479

3.2.6 Shillapoo Lake Interior Runoff

Interior runoff currently results in winter flooding of the Shillapoo Lake bed. Under current conditions, water depths are reported to reach a maximum of four or five ft before the expulsion pump is reactivated in mid to late January to evacuate storage (Brian Calkins, WDFW, personal communication, April 1998). There are, however, no quantitative data on interior runoff rates, runoff volumes, water levels, or groundwater interactions with the Columbia River system. Despite this lack of data, interior runoff is a potentially important source to maintain water levels in the restored wetlands.

Interior runoff is generated both as groundwater inflows and as a result of precipitation over the lakebed and surrounding area. Two specific point sources of groundwater inflows have been identified; a perennial spring near the south end of the project, and an artesian well. No data are available on flow rates from either the spring or artesian well. There are likely other unidentified seeps and groundwater inputs to Shillapoo Lake. Groundwater inputs are likely a function of Columbia River water levels. Note that the median winter water level in the Columbia River at RM 98 is approximately +8 ft as determined from available long-term data from the stage gage at Vancouver (RM 106.5). The Shillapoo Lake bed by contrast has a minimum elevation of about +1 ft.

Various alternatives were considered for estimating interior runoff to Shillapoo Lake, including transposition of streamflow data from Burnt Bridge Creek or Salmon Creek and synthesis of runoff using observed rainfall data. There are obvious difficulties irrespective of the approach adopted. While the Burnt Bridge and Salmon Creek watersheds are subject to similar meteorological influences to those at Shillapoo Lake, their hydrologic response as upland catchments may have little similarity to the response of Shillapoo Lake in the Columbia River flood plain. Synthesis of runoff using rainfall data would similarly be quite uncertain because of the lack of data for model validation. The situation is further complicated by the fact that the hydrologic response of the interior area would be affected by water management practices.

The approach adopted to generating interior runoff data for Shillapoo Lake for UNET modeling was to simulate runoff on a continuous basis for the period October 1994 through September 1997 using the USEPA's HSPF model (Bicknell et al, 1993). HSPF is a continuous conceptual deterministic rainfall runoff model which has been widely used in the Puget Sound lowlands, primarily for land use planning and the design of urban stormwater control facilities. The model has also been used in the Portland area but to a lesser extent.

The U.S. Geological Survey has established generalized model parameter estimates for HSPF for several counties in the Puget Sound area based on soil type and land use (Dinicola, 1990). In particular, the USGS has established generalized parameters for what are referred to as "saturated" soils. This definition encompasses wetland soils and valley alluvium. In the absence of other suitable approaches and the need to generate runoff on a continuous basis for an extended period of time, HSPF was applied using the USGS "saturated" soil parameters for the Puget Sound region. The HSPF model parameters are provided in Table 3.1, and the HSPF User Control Input (UCI) is provided in Appendix C.

HSPF requires continuous rainfall data and evaporation data as input. For this application, the HSPF model was run at an hourly time step using, as input, hourly rainfall data from Portland WSFO AP (NWS station 356751) increased by 14% to account for long term differences in rainfall totals between Portland Airport and Vancouver, and daily pan evaporation data from North Willamette Experimental Station (NWS station 356151).

The HSPF model produces, as output, simulated runoff in terms a depth of runoff per computational time step (inches per unit area per hour). This is then scaled up on the basis of drainage area to produce total runoff amounts for the catchment or area of interest. The simulated runoff has three components; surface runoff, interflow and groundwater. The selected model parameters are such that in wet months when rainfall has saturated the soils, infiltration rates fall to near zero and large quantities of surface runoff are generated. After an extended dry period has lowered soil moisture levels, the selected parameters allow soils to absorb significant quantities of rainfall such that, in this climatic regime, little runoff is generated in the summer months. This behavior is what might be expected qualitatively for soils similar to those in the Shillapoo Lake area. However, the model ignores the effect of the Columbia River on groundwater and soil moisture levels and it does not reflect proposed water management scenarios, which maintain saturated soils or flooded conditions through the winter months and early part of the summer. In other words, the hydrologic response simulated by HSPF is solely based on meteorological and soil conditions absent other "external" influences.

Use of HSPF generated runoff for the entire Shillapoo Lake area would likely understate total interior runoff amounts for two reasons:

1. The HSPF model ignores the effects of the Columbia River and Shillapoo Lake water management practices on soil saturation and interior flooding and therefore overstates effective infiltration rates, and,
2. The HSPF model ignores groundwater inflows

To offset these effects to some extent, interior runoff for UNET modeling was generated as follows:

- For the months October through December, runoff was determined from the HSPF simulations for the entire interior area.

- For the months January through May, it was assumed that all ground below elevation +5 ft was saturated or flooded. Runoff from this area was computed directly from rainfall amounts assuming 100% runoff. Runoff from the area above elevation +5 ft was determined from HSPF simulations. Rainfall on the lake area was taken from records from Portland WSO AP (NWS station 356751) increased by 14% to account for long term differences in rainfall totals between Portland Airport and Vancouver, consistent with assumptions used for HSPF simulations.

- For the months June through September, runoff was determined from the HSPF simulations for the entire interior area.

Runoff was simulated at an hourly time step but was aggregated to a daily time step for input to the UNET hydraulic model.

Since this approach still ignores groundwater inflows, estimates of interior runoff are still likely to be conservative in that they will lead to overestimation of the amount of water that has to be imported (e.g. withdrawn from the Columbia River) to maintain target water levels needed for wetland restoration.

3.2.7 Synthesis of Hypothetical Low Water Year Data

As can be seen in Appendix A, the three years of data available to perform detailed UNET simulations of the system (water years 1995, 1996, and 1997) have average or higher than average water levels on the Columbia River at Vancouver and would not be suitable for evaluating the ability to fill Shillapoo Lake by gravity in a dry year. In order to examine the performance of the project alternatives under a drier than average year, Time series boundary condition data for a hypothetical low water year were constructed for the purposes of UNET simulations by concatenating data from low water months from water year 1995. The hypothetical low water year (designated 1999 for convenience) was constructed to approximately match the stage duration characteristics of the water level regime at Vancouver in the low water years of 1977 and 1994 by concatenating data for each time series input from the following months:

Month of Hypothetical Dry Year	Month Used
October	October 1994
November	August 1995
December	November 1994
January	November 1994
February	July 1995
March	August 1995
April	November 1994
May	July 1995
June	August 1995
July	August 1995
August	August 1995
September	September 1995

A plot of mean daily stage at Vancouver for the hypothetical dry year is shown in Figure 3.6 for comparison with plots of observed water level data provided in Appendix A.

It is recognized that the months selected to construct the dry year are probably not representative of local conditions on the lower Columbia and in particular may not provide a good representation of interior runoff to Shillapoo Lake. However, the selection is appropriate for overall system simulation given that it is Columbia River water levels that are the greatest controlling factor in the ability to fill and drain Shillapoo Lake using a gravity system.

3.3 Baseline Hydraulic Model Calibration

The original UNET model was calibrated by the Portland District COE against water level and discharge data at various locations along the Lower Columbia River for various periods of time in water years 1996 and 1997. Calibrated or partly calibrated models were available from the COE for three separate periods: for February 1996 (an extreme high flow period), and for the spring freshets of 1996 and 1997. Documentation available for the COE calibration is provided in Appendix B.

The UNET model with the Lake River/Vancouver Lake/Shillapoo Lake additions was calibrated against stage data at various locations for water years 1995 and 1998 and the COE calibrations for water years 1996 and 1997 were checked and refined as necessary to further improve simulation results for those years.

3.3.1 Water Year 1998 Calibration

The UNET model with the Lake River/Vancouver Lake/Shillapoo Lake additions was calibrated against hourly stage data for February and/or March 1998 from four sites as follows:

Gage Site	Period of Record Available or Used for Calibration
Columbia River at Vancouver (RM 106.5)	1 February 1998 - 31 March 1998
Columbia River at St. Helens (RM 86.5)	1 February 1998 - 31 March 1998
Lake River approximately 0.5 miles downstream from the outlet of Vancouver Lake (Felida Moorage)	1 February 1998 - 31 March 1998
Vancouver Lake Flushing Channel	4 March 1998 – 31 March 1998

Stage recording gages were installed by the Portland District USACE along Lake River at Felida Moorage (RM 10.8) and along the Vancouver Lake Flushing Channel (near Columbia River RM 101). The Felida Moorage gage was installed and operational on February 4, 1998. The gage collects Lake River stage every 15 minutes and is currently budgeted for data collection through September 1998. The Vancouver Lake Flushing Channel gage was installed and operational on March 4, 1998. This gage records Columbia River stages every 15 minutes and is currently budgeted for data collection through September 1998. **The Lake River (Felida Moorage) gage site** closely **reflects water levels in Vancouver Lake**, while the Flushing Channel site essentially provides water levels in the Columbia River at the confluence with the Flushing Channel (about RM 101). Both gages utilize Druck pressure transducers linked to Sutron data collectors with a solar power supply. Gage data through April 29, 1998, are presented for both sites on Figure 3.7 (NGVD datum).

Boundary condition data for calibration were obtained as described in Sections 3.2.1 through 3.2.5. In the absence of data on water levels in Shillapoo Lake, it was assumed that the expulsion pump was discharging from Shillapoo Lake into Lake River at its rated capacity (22.3 cfs) for the entire duration of the calibration period. Shillapoo Lake interior runoff data were not actually used in simulations for the 1998 calibration period.

Initial roughness values for the mainstem Columbia River for the 1998 model were taken from the COE model for the 1996 spring freshet. Calibration was accomplished by modifying the assumed roughness of Lake River and the Columbia River between RM

57.8 (about 8 miles downstream from Longview) and RM 111.5 (upstream from Vancouver). Plots of simulated against observed water levels are shown for the calibration period in Figures 3.8 through 3.14 for the Vancouver, St. Helens, Lake River, and Flushing Channel data. Simulation results generally agree with observed data within ±1 ft. at Felida Moorage and within ±0.5 ft for the Vancouver Flushing Channel and mainstem Columbia River gages. Note that the rather poorer results at Felida Moorage may be partly the result of poor estimates of inflow data on Salmon Creek. As noted in Section 3.2.5 above, data from Salmon Creek were not available for the calibration period but were estimated from the record on Burnt Bridge Creek.

It was found that water levels at Felida Moorage are relatively insensitive to roughness in **Lake River. Water levels are controlled to a noticeable degree by water levels on the Columbia River at the confluence with Lake River**, approximately 11 miles downstream.

Note that the 1998 calibration period is very short (limited by the data available) and conditions on the Columbia River were relatively uneventful. There were however two moderately large local runoff events which are reflected in the flow record for Burnt Bridge Creek. The calibration should be regarded as approximate and the calibrated model should be used with circumspection.

3.3.2 Water Year 1995 Calibration

The UNET model for water year 1995 simulations was based directly on model parameters and roughness values for the mainstem Columbia River and Lake River obtained from the water year 1998 calibrations. A comparison of simulated and recorded stages for the Columbia River at Vancouver is provided in Figures 3.15 through 3.20.

3.3.3 Water Year 1996 Calibration

Minor adjustments were made to the mainstem Columbia River roughness values contained in the original COE UNET model for the 1996 spring freshet to improve simulations for moderate to low mainstem Columbia River stages. The roughness values obtained, together with Lake River model parameters obtained from the 1998 calibration, were used throughout the water year 1996 simulations. This produced somewhat poorer simulation of stages for the extreme flood of February 1996. However, this period is expected to have little or no influence on the design of the restoration project. Simulated and recorded stages for the Columbia River at Vancouver are shown in Figures 3.21 through 3.23.

3.3.4 Water Year 1997 Calibration

Roughness values for the water year 1997 simulations were taken directly, without adjustment, from the original COE model for the 1997 spring freshet for the mainstem Columbia and from the 1998 calibration (Section 3.3.1) for Lake River. Simulated and recorded stages for the Columbia River at Vancouver are shown for selected months in Figures 3.24 and 3.25.

4.0 BASE LEVEL ALTERNATIVE ANALYSES

Four base level alternatives were investigated at a reconnaissance level to explore alternative means for achieving target water level regimes in Shillapoo Lake.

4.1 Existing Condition Alternative

The existing condition alternative assumes that existing connections to the Columbia River and Lake River would remain unchanged. A conceptual layout of the existing condition is shown in Figure 4.1. As described in Chapter 3, under the existing conditions there are no connections to the Columbia River while connections to Lake River consist of the following:

- A 22.3-cfs pump station discharging into Lake River near RM 10.5.
- A 42-inch culvert with an invert elevation of about +1.7 ft and tide gate (on the Lake River end of the culvert) on the east side of Shillapoo Lake discharging into Lake River near RM 10.5.
- A 36 inch culvert with an invert elevation of about +5.6 ft and tide gate (on the Lake River end of the culvert) at the north end of Shillapoo Lake discharging into Lake River near RM 9.0.

For the existing condition base level simulations, the basin line UNET model described in Chapter 3 was modified slightly to reflect a pump station operation more appropriate to the wetland restoration goals. Specifically, it was assumed that the pump station was shut down from October through May of each year and reactivated on 1 June to drain the lake.

Simulations were conducted for the following periods:

- 1 October 1994 through 30 September 1995 (water year 1995)
- 1 February 1996 through 15 August 1996 (water year 1996)
- 1 April 1997 through 15 August 1997 (water year 1997)

Initial conditions assumed for Shillapoo Lake were:

- Elevation -0.5 ft on 1 October 1994 (water year 1995)
- Elevation -0.5 ft on 1 February 1996 (water year 1996). The major flood of early February 1996 subsequently overtopped the levees surrounding Shillapoo Lake and completely filled the lake, thus the starting elevation is of no real significance.
- Elevation +6 ft on 1 April 1997 (water year 1997). This starting elevation was based on estimates of interior runoff amounts from October 1996 through March 1997 and consideration of the stage hydrograph for the Columbia River at Vancouver provided in Appendix A.

Comparisons of UNET simulation results for the following key points:

- stage on Columbia River near Langsdorf Landing (RM 98)
- stage on Lake River at Felida Moorage (RM 10.8)
- stage in Shillapoo Lake

are provided in:

- Figures 4.2 through 4.7 and Table 4.1 for water year 1995,
- Figures 4.8 through 4.11 and Table 4.2 for water year 1996, and,
- Figures 4.12 through 4.14 and Table 4.3 for water year 1997.

The following observations can be made from the existing condition simulation results:

Water Year 1995

- Despite the lack of a connection to fill Shillapoo Lake from either the Columbia River or Lake River, simulated interior runoff fills the lake steadily from late October on, reaching about +3 ft by January 1, 1995 and +5 ft by April 1, 1995. Note however that water year 1995 is wetter than average (see Table 2.1).
- Simulated Lake River stages in the desired drawdown window (June and July) have a minimum of about +5 ft. It is clear in this and simulations for other years that the lake cannot be drained by gravity. For 1995, the expulsion pump, which was assumed to be activated on June 1, provides complete drawdown by late July, consistent with the target water level regime.
- Columbia River stages in the fall and early winter (October through December 1994) barely reach +8 ft. Thus although the main body of Shillapoo Lake could be filled by gravity from the Columbia (or Lake River), water year 1995 stages are such that a gravity supply would not be possible for the northern part of the study area which has minimum elevations of about +7 ft.

Water Year 1996

- Water year 1996 simulations are dominated by the extreme flood of early February, which overtops the levee system and completely fills the lake. The lake drains by gravity via the existing tide gates but remains above +9 ft because of high Lake River elevations until June 1 when the expulsion pump is turn on.
- With the pump operating at capacity (23.3 cfs) from June 1 on, it is not possible to completely drain the lake. By the end of August, a combination of gravity drainage and pumping has only drawn the lake down to +4 ft. With the large volume of water

retained in the lake from the February flood, pumping would have had to start as early as April 1 to achieve the target draw down.

Water Year 1997

- **Water year 1997 simulation results are dominated by the very high spring** freshet. It is again obvious that draining the lake by gravity is infeasible. With interior runoff producing a June 1 interior stage of +7 ft, the expulsion pump is not able to achieve complete drawdown before late August. Again the pump would have to be activated before June 1 to match the target date for drawdown.

Hypothetical Dry Year

Simulations were not performed for the hypothetical dry year, which was analyzed only for the gravity supply alternative (see Section 4.3).

4.2 Natural Condition Alternative

The intent of the "natural condition" simulations was to investigate an alternative in which historical connections to the Columbia River are restored to the extent possible.

Inspection of aerial photographs from 1943 indicates that prior to construction of the current system of levees and drainage ditches in the 1950's there were at least two natural channels that served to fill and drain Shillapoo Lake. The principal connection to the Columbia River was a channel that joined the Columbia at about RM 96.3, ran south to about the north end of Caterpillar Island and then headed east toward the center and lowest part of Shillapoo Lake. A channel connection also existed to Lake River at the north end of Shillapoo Lake. Vestiges of both these channels remain today though actual connections have been blocked.

From recent COE topographic mapping, the historical connection to the Columbia River appears to have had an invert elevation of about +12 ft at which water from the Columbia would have started spilling into Shillapoo Lake. Similarly, water levels in Lake River would have had to exceed about +9 ft for water to spill into Shillapoo Lake from Lake River. Prior to construction of major dams on the Columbia, the Columbia River connection probably carried water into Shillapoo Lake in most years. However under the current water level regime on the Columbia (see Appendix A and Figures 2.4 through 2.7), with effective regulation of flood peaks, there would be a significant proportion of years in which there would be no or minimal spills into Shillapoo Lake unless the connections were lowered. It is possible that even in dry years, interior runoff within Shillapoo Lake would be sufficient to meet water needs for the proposed restoration project. There are however no interior runoff data and the reliability of interior runoff as a source of water is unknown.

With the historical connection to Lake River apparently having a minimum controlling elevation of about +9 ft, it is likely that Shillapoo Lake historically retained water all year, as is evident from aerial photographs taken in August 1943. It is clear that restoration of the historical connections would not allow Shillapoo Lake to be drained as desired. Without additional low elevation outlets, it is unlikely that water levels in Shillapoo Lake would drop much below about +7 ft in the summer months.

It is clear that a "natural condition" alternative could not meet several features of the target water level regime:

- With the historical connection to the Columbia River having an invert elevation of +12 ft, Shillapoo Lake would not receive water from the Columbia in a relatively high proportion of years. Interior runoff would still provide a water source, however the volume of interior runoff available (particularly due to groundwater inflows) is uncertain.

- With the historical connection to Lake River having an invert elevation of +9 ft, Shillapoo Lake would retain water all summer and it would not be possible to drain the lake without pumping.

With these points in mind, a modified "natural" alternative was defined, as shown in the conceptual layout in Figure 4.15, having the following elements:

- A gravity connection to the Columbia via a constructed ditch with a trapezoidal section and a controlling invert elevation of +10 ft. The ditch is assumed to have a 50 ft. bottom width and 3H:1V side slopes, similar to the cross-sectional geometry of the historic connection.

- A gravity connection to Lake River via a ditch with a trapezoidal section and a controlling invert elevation of +8 ft. The ditch is assumed to have a 40 ft. bottom width and 3H:1V side slopes, again similar to the cross-sectional geometry of the historic connection. Note that much of the extreme northern part of Shillapoo Lake has a low elevation of between +8 and +9 ft.

- A gravity drain to Lake River at the location of the existing expulsion pump. This would consist of the existing 42-inch culvert and tide gate which has an invert elevation of approximately +1.7 ft, supplemented by a slide gate to allow water to be retained to desired depths in the winter months. No pumping will be assumed under this alternative. Simulations would assume that the slide gate remains closed from October through May. The slide gate is assumed to be opened on June 1 to allow drainage of the lake via the tide gate.

Note in Figure 4.15 that the connection to the Columbia River is not shown along its original alignment. It may not be possible to restore this connection along its original alignment or the alignment actually shown in the figure because of property ownership issues. However, the exact alignment of a restored channel is not considered important for this base level analysis.

Simulations were conducted for the following periods:

- 1 October 1994 through 30 September 1995 (water year 1995)
- 1 February 1996 through 15 August 1996 (water year 1996)
- 1 April 1997 through 15 August 1997 (water year 1997)

Initial conditions assumed for Shillapoo Lake were:

- Elevation -0.5 ft on 1 October 1994 (water year 1995)
- Elevation -0.5 ft on 1 February 1996 (water year 1996). The major flood of early February 1996 subsequently overtopped the levees surrounding Shillapoo Lake and completely filled the lake, thus the starting elevation is of no real significance.
- Elevation +6 ft on 1 April 1997 (water year 1997). This starting elevation was based on estimates of interior runoff amounts from October 1996 through March 1997 and consideration of the stage hydrograph for the Columbia River at Vancouver provided in Appendix A.

Comparisons of UNET simulation results for the following key points:

- stage on Columbia River near Langsdorf Landing (RM 98)
- stage on Lake River at Felida Moorage (RM 10.8)
- stage in Shillapoo Lake

are provided in:

- Figures 4.16 through 4.21 and Table 4.1 for water year 1995,
- Figures 4.22 through 4.25 and Table 4.2 for water year 1996, and,
- Figures 4.26 through 4.28 and Table 4.3 for water year 1997.

The following observations can be made from the natural condition simulation results:

Water Year 1995

- Shillapoo Lake fills from 1 October 1994 through 1 February 1995 due to interior runoff in essentially the same manner as the existing condition simulations. The sudden drop in Shillapoo Lake water levels in early October to elevation -2 ft is a result of model instability. However, since storage in Shillapoo Lake is minimal below elevation zero, this has no effect on simulation results beyond November 1.
- Further filling of Shillapoo Lake occurs in two periods in February 1995 when Columbia River and Lake River levels rise above +10 ft resulting in short periods of gravity inflow via both the Columbia and Lake River connections.

- Draining of the lake in the summer months is constrained by water levels in Lake River. As a result Shillapoo Lake never completely drains, reaching a minimum elevation of only +3.3 ft at the end of September 1995.

Water Year 1996

- Simulation results at the start of the water year 1996 simulations are similar to the existing alternative. The Shillapoo Lake levees are overtopped at the beginning of February 1996 completely filling the lake. Immediately after the February event, the lake drains more rapidly than in the existing condition through the open channel hydraulic connections to the Columbia and Lake River.

- Following the February flood, Shillapoo Lake levels track the Columbia and Lake River levels quite closely. Note that for Columbia River and Lake River elevations above +10 ft, there are periods of significant flow through Shillapoo Lake with a head loss of up to about 2 ft between the Columbia River and Lake River. Draining of the lake in the summer months is constrained by consistently high levels in Lake River with Shillapoo Lake falling to a minimum of only +5 ft by the end of August 1996.

Water Year 1997

- Water year 1997 is similar in many respects to 1996 simulations. The lake is filled by gravity from the Columbia and Lake River during the spring freshet when Columbia and Lake River water levels exceed +10 ft for an extended period of time.
- Draining of Shillapoo Lake is again constrained by consistently high water levels in Lake River with Shillapoo Lake falling to a minimum of only about +6 ft by the end of August 1997.

Hypothetical Dry Year

Simulations were not performed for the hypothetical dry year, which was only analyzed for the gravity supply alternative only (see Section 4.3).

4.3 Gravity Supply Alternative

It is clear that neither the Existing Condition nor Natural Condition Alternative would be able to achieve target water level regimes with any degree of reliability. In the case of the Existing Condition Alternative, the lack of a connection to the Columbia means interior runoff must be relied on to provide the water supply for the project. There is no control over interior runoff rates and insufficient information to reliably estimate the amount and seasonality of the water supply. In the case of the Natural Condition Alternative, it is not possible to supply water by gravity in low water years on the Columbia. Furthermore, the project cannot be drained as desired.

The "Gravity Supply" Alternative was designed to improve the reliability with which water level targets could be reached without pumping. The alternative, for which a conceptual layout is provided in Figure 4.29, has the following components:

- A low water year gravity connection to Lake River is provided. The connection is assumed to consist of a 42-inch culvert fitted with flap gates (or equivalent) and hoists at either end and a slide gate. The culvert is assumed to have an invert elevation of -2 ft. This structure would be operated to allow inflow from Lake River in the winter and spring to the extent possible or desired. Gate settings would be modified in June to allow the lake to drain by gravity back to Lake River as head in Shillapoo Lake and tidal conditions permitted.

- A moderate/high water year gravity connection to the Columbia River is provided. This is assumed to consist of a trapezoidal channel with a bottom width of 15 ft, 3H:1V side slopes, and an invert elevation of +8 ft. A control and distribution structure would be provided at the Shillapoo Lake end of the channel to distribute water among different wetland cells and to shut off water supply when desired levels have been reached. The effect of this control structure was ignored for the purposes of reconnaissance level simulations.

- The existing 36-inch culvert and tide gate at the northern end of Shillapoo Lake is assumed to be abandoned and plugged and was not included in simulations for this alternative.

The "gravity supply" alternative would allow shallow flooding of Shillapoo Lake in low water years on the Columbia River. In such years it would not, however, be possible to flood the elevated northern part of the area (which has minimum elevations of about +8 ft) or have the flexibility to raise water levels in the spring for Reed Canarygrass control. Use of the higher elevation gravity inlet from the Columbia would allow greater flexibility in terms of depth and spatial variation of flooding in moderate to high water years on the Columbia.

Simulations were conducted for the following periods:

- 1 October 1994 through 30 September 1995 (water year 1995)
- 1 February 1996 through 15 August 1996 (water year 1996)
- 1 April 1997 through 15 August 1997 (water year 1997)
- hypothetical dry water year (water year "1999")

Initial conditions assumed for Shillapoo Lake were:

- Elevation 0 ft on 1 October 1994 (water year 1995)
- Elevation 0 ft on 1 February 1996 (water year 1996). The major flood of early February 1996 subsequently overtopped the levees surrounding Shillapoo Lake and completely filled the lake, thus the starting elevation is of no real significance.

- Elevation +6 ft on 1 April 1997 (water year 1997). This starting elevation was based on estimates of interior runoff amounts from October 1996 through March 1997 and consideration of the stage hydrograph for the Columbia River at Vancouver provided in Appendix A.
- Elevation 0 ft on 1 October at the start of the hypothetical dry year (water year "1999")

Comparisons of UNET simulation results for the following key points:

- stage on Columbia River near Langsdorf Landing (RM 98)
- stage on Lake River at Felida Moorage (RM 10.8)
- stage in Shillapoo Lake

are provided in:

- Figures 4.30 through 4.35 and Table 4.1 for water year 1995,
- Figures 4.36 through 4.39 and Table 4.2 for water year 1996,
- Figures 4.40 through 4.42 and Table 4.3 for water year 1997, and,
- Figures 4.43 through 4.48 and Table 4.4 for the hypothetical low water year.

The following observations can be made from the existing condition simulation results:

Water Year 1995

- The combination of interior runoff and a low-level gravity inlet allows water levels in Shillapoo Lake to be raised to elevation +5 ft by about December 31. This indicates, as one might expect, that shallow flooding of the lake bed to target elevations could be readily achieved by provision of a gravity connection to Lake River.
- The simulations demonstrate again the infeasibilty of draining the lake by gravity. In the absence of a pump, summer levels in Shillapoo Lake are dictated by levels in Lake River which only allow lake levels to be drawn down to a low of about +3 ft by the end of September 1995.

Water Years 1996

- Simulation results are similar to the natural condition alternative except that elimination of the open channel connection at the north end of Lake River results in slower draining of the lake. Elimination of the northern connection also prevents flow through from the Columbia River to Lake River except at the height of the February flood when levees surrounding Shillapoo Lake are overtopped. End of summer lake levels, controlled by the level of Lake River, are identical to those under the natural alternative at about elevation +5 ft.

Water Year 1997

- The principal point of interest is again the draw down period. Because of sustained high Lake River levels, the minimum water level achieved at the end of August 1997 is about +7 ft.

Hypothetical Low Water Year

- The low level gravity connection to Lake River allows Shillapoo Lake to be filled rapidly to elevation +4 ft (the stage in Lake River) early in October. The lake continues to fill as Lake River elevations allow. Inflows from Lake River plus interior runoff (amounts of which are highly uncertain for this period) increase lake levels to +8 ft by 1 June, when the lake is allowed to drain.

4.4 Combined Pump/Gravity Supply Alternative

The pump/gravity alternative is intended to improve the flexibility of water management in dry years and to permit goals for draining Shillapoo Lake in the late summer to be more readily achieved. A conceptual layout of the alternative is provided in Figure 4.49. This alternative would have the following features:

- A low-level connection to the Columbia would be provided. This would consist of a trapezoidal channel with a 10 ft bottom width, 2H:1V side slopes, and an invert elevation at 0 ft. This would lead to a 20-cfs pump station close to Lower River Road which would lift water in low water years to a higher elevation water distribution system at about elevation +12 ft. This would produce sufficient head to provide water by gravity to all parts of Shillapoo Lake including the relatively high ground (elevations +7 to +9 ft) at the north end of the lake bed. The pump station could also be used to augment supply as needed to rapidly raise water levels in the months of March and April for Reed Canarygrass control. The higher elevation water distribution system would be fed by gravity from the Columbia River during moderate and high water levels on the Columbia.

- A low water year gravity connection to Lake River is provided as in the "gravity supply" alternative. The connection is assumed to consist of a 42-inch culvert fitted with flap gates (or equivalent) and hoists at either end and a slide gate. The culvert is assumed to have an invert elevation of -2 ft. This structure would be operated to allow inflow from Lake River in the winter and spring to the extent possible or desired. Gate settings would be modified in June to allow the lake to drain by gravity back to Lake River as head in Shillapoo Lake and tidal conditions permitted.

- The existing 22.3-cfs pump station on the east side of Shillapoo Lake would be retained to augment gravity drainage to Lake River.

- The existing 36-inch culvert and tide gate at the northern end of Shillapoo Lake is shown as being retained under this alternative. However its exact function and potential modifications would be determined by the interior wetland cell configuration.

A number of variants on this alternative are possible. These could include, for example, provision of a low water year gravity supply from the Columbia via a pipe or siphon at about elevation 0 ft.

Detailed UNET simulations were not performed for the pump/gravity supply alternative. Since the natural dynamics of the Columbia system has little influence on the ability to fill and drain Shillapoo Lake by pump, it was felt that UNET simulations would provide only minimal new information for evaluation of the base level alternatives.

The pump station size needed to fill Shillapoo Lake under this alternative was determined from estimated water supply needs in 1977 (the lowest water year in the 23-year period of record analyzed at Vancouver) and consideration of the potential need for raising water levels rapidly in the early growing season for Reed Canarygrass control. A 20-cfs pump could provide approximately 1000 acre ft over a 25-day period. This would be sufficient to raise the water level in a 300-acre wetland cell by an incremental amount of about 3.5 ft, ignoring interior runoff contributions.

5.0 RECOMMENDED ALTERNATIVE

5.1 Selection of Recommended Alternative

The base level alternatives analyses presented in Chapter 4, together with the stage-duration analyses presented in Section 2.3, produced the following principal findings:

- Given the water level regime in both the Columbia River and Lake River it will not be possible to drain Shillapoo Lake as desired without pumping. It was estimated in Section 2.2 that the maximum volume of water to be contained in Shillapoo Lake on May 1 might be as great as 2,850 acre ft depending on the total area restored to wetland conditions and the area which might be deeply flooded in any one year for Reed Canarygrass control. The existing expulsion pump with a nominal rated capacity of 22.3 cfs could drain this volume into Lake River in about 65 days. The total time to drain the lake would actually be greater than this after allowing for incremental interior runoff volumes, including seepage into Shillapoo Lake. Nevertheless, this draw down time is consistent with the target draw down period shown in Figure 2.3.

- It appears that shallow flooding of the Shillapoo Lake bed (to elevations of about +4 ft) could be readily achieved by December 1 even in low water years through provision of a gravity inlet from Lake River. A gravity inlet from Lake River could be provided at considerably less expense than a comparable low elevation gravity inlet from the Columbia River, however a gravity inlet from Lake River may have limited flexibility in terms of its ability to distribute water amongst individual wetland cells.

- While interior runoff (including seepage and groundwater inflows) may be a useful source of water for producing shallow flooding of the Shillapoo Lake bed, there is at present insufficient data to assess the reliability, quantity and timing of such runoff.

- Early winter water levels higher than about +8 ft cannot be reliably achieved by gravity (see Figure 2.4). Consistent annual flooding of the northern portion of the Shillapoo Lake area, which has minimum elevations of about +7 ft, would require provision of a pumped supply from either Lake River or the Columbia River. A similar problem would be faced in supplying water to any wetland cells at this or higher elevations.

- The reliability with which wetland cells could be deeply flooded by gravity for Reed Canarygrass control is severely limited by March/April water levels in the Columbia River. The ability to consistently flood cells in March or April to elevations above about +8 ft (see Figure 2.7) would require provision of a pumped supply.

- With the above points in mind, a Recommended Alternative was selected in consultation with WDFW and COE staff. The Recommended Alternative is similar to the Pump/Gravity Supply Alternative discussed in Section 4.4 and depicted in Figure 4.49. A conceptual layout of the Recommended Alternative, including the proposed internal wetland cell configuration, is shown in Figure 5.1. It would have the following principal features:

- The existing 22.3-cfs expulsion pump on the east side of Shillapoo Lake would be retained to provide drainage to Lake River.

- The existing 42-inch culvert (invert elevation +1.7 ft) adjacent to the expulsion pump would be lowered to an invert elevation of -2.0 ft. A slide gate and flap gate and hoist (to raise the flap gate) would be provided to allow gravity filling and draining of the main body of Shillapoo Lake (wetland cells 1 through 7) as water levels in Lake River permit.

- The existing 36-inch culvert and flap gate at the northern end of Shillapoo Lake would be retained but modified by provision of a hoist (to raise the flap gate) and slide gate to allow gravity filling and draining of the northern cell (cell 8) as water levels in the cell and Lake River permit.

- The reliability of low water year operations would be improved by provision of conveyance facilities to take advantage of excess capacity in a pumped water supply from the Columbia River being planned by WDFW to serve existing parts of the Shillapoo Wildlife Area to the west and south of Shillapoo Lake. WDFW has applied for a water right to pump 11.2 cfs from the Columbia River at Langsdorf Landing. Conveyance and control facilities would be provided to supply pumped water to the main body of Shillapoo Lake for Reed Canarygrass control and to the northern cell (cell 8) for both Reed Canarygrass control and normal operations.

It should be noted that accurate characterization of the Lake River water level regime, upon which the recommendation for a gravity inlet from Lake River is based, relies heavily on the reliability of the UNET model described in Chapter 3. As discussed in Section 3.4, calibration of the Lake River/Vancouver Lake additions to the original UNET model relied on only two months of Lake River water level data. Simulated Lake River water levels will inevitably be subject to error. For low water conditions, this error is expected to be less than ±1 ft, and more likely in the range ±0.5 ft. Any oversimulation of low water levels in Lake River will translate directly into an overstatement of the ability to supply water to Shillapoo Lake by gravity.

Further details of the recommended alternative, including conceptual level design of water control structures and conveyance facilities and an evaluation of the project's performance are provided in the following report sections.

5.2 Hydraulic Performance Standards

No specific quantitative hydraulic performance standards have been established for the Recommended Alternative other than maintenance of the target water level regime discussed in Section 2.2.

Following discussion with WDFW and COE biologists, the following guidelines were developed for evaluation of pump station performance and the design of the water control facilities necessary for maintenance of the target water level regime:

1. Fill of wetland cells to their normal maximum operating levels should be achievable before 31 December in the lowest water year on the Columbia River in the period water years 1973 - 1997.

2. Once normal maximum operating levels have been achieved, control structures should be such that in an "average" year, water levels can be maintained within ±0.5 ft of those levels until either summer drawdown begins (about June 1) or flooding for Reed Canarygrass control begins (about mid-March). No restrictions on deviations from the target levels would be imposed in unusually wet years, except that the integrity of facilities should be guaranteed for conditions with a 1% chance of exceedance.

3. Complete draining of the project from normal maximum operating levels should be achievable between June 1 and July 31 for the wettest June and July in the period 1973 - 1997.

5.3 Configuration of the Recommended Alternative

A conceptual layout of the Recommended Alternative was shown in Figure 5.1. Additional details including the configuration and location of all proposed water control structures, embankments, and conveyance improvements is provided in Plate 2. Analysis of the performance of the Recommended Alternative is discussed in Section 5.4.

5.3.1 Cell Configuration

The wetland cell configuration for the Recommended Alternative was determined in consultation with WDFW and COE as shown in Figure 5.1 and Plate 2. The cell configuration was determined considering: flexibility of project operation; cell size; and likely range of construction, operation, and maintenance costs.

Cells 1 through 7 in the main body of Shillapoo Lake have minimum lake bed elevations in the range +1.5 to +2.5 ft. Invert elevations along ditches may be as low as -4 ft or lower, however detailed surveys of ditch inverts have not been made. Normal maximum water levels are anticipated to be in the range +4 to +5 ft and water levels for Reed Canarygrass control are anticipated to be in the range +6.5 to +8.0 ft.

Cell 8 in the northern part of the project area is at a higher elevation than the main body of Shillapoo Lake. Minimum ground elevations are about +7 ft, which translates into normal maximum water levels of the order of +9 to +10 ft and water levels for Reed Canarygrass control in the range +12 to +13 ft.

Elevation/surface area/volume data for cells 1 through 8 are provided for the range of interest in Table 5.1 along with the total area estimated to be tributary to each cell.

5.3.2 Embankments

Embankments separating wetland cells 1 through 7 should be constructed to elevation +10 ft. The embankment will have a top width of 10 ft, 3H:1V side slopes down to elevation +6 ft, and 6H:1V side slopes below elevation +6 ft. The break in side slope is intended to increase the overall embankment width at the normal maximum water level to reduce the potential damage from nutria burrows and dens. The maximum embankment height will be approximately 10 ft. The total length of embankment required for completion of cells 1 through 7 is approximately 23,800 ft.

A containment embankment will be required along the high ground forming the easterly and southerly boundary of wetland cell 8. This should be constructed to elevation +15 ft with a top width of 10 ft, 3H:1V side slopes down to elevation +11 ft, and 6H:1V side slopes below elevation +11 ft. The embankment will mostly be less than 5 ft in height, but will locally be up to 10 ft in height. The required length of embankment will be approximately 6,500 ft.

It is anticipated that embankments will be constructed from soil excavated from the lake bed. However, no soils or other geotechnical investigation has been performed for this work and the suitability of lake bed material for embankment construction is not known. The appropriate cross-sectional geometry of the embankments should be confirmed after geotechnical investigations have been completed.

The embankments should be surfaced with gravel where required to provide wet weather access to water control structures. Typical embankment sections are shown in Figure 5.2.

5.3.3 Water Control Structures

Locations of water control structures proposed for Shillapoo Lake are shown on Plate 2. The proposed control structures take advantage of existing facilities to the extent possible. For the most part the structures have been designed for manual operation. The conceptual designs which follow are for a relatively large number of simple structures. The number of structures required could be reduced by combining their functions through somewhat more complex designs. Optimization of structure designs will be considered at a later date by the Portland District COE as part of the project's detailed engineering design and costing.

Main Fill/Drain Structures (WCS Type A)

Provision should be made for filling and completely draining each cell via the existing or an extension of the existing channel system. Controls should be provided where existing or proposed drainage channels cross each embankment. The controls should consist of either a 2 ft or 2.5 ft diameter corrugated metal culvert and slide gate. In cells 1 through 7, culverts should be installed on the ditch line and set at invert elevations ranging from -3 ft to -4 ft depending on location. Provision should be provided for draining the northern cell (cell 8) either to cell 7 via control structure A-9, for subsequent pumped discharge to Lake River by the expulsion pump, or directly to Lake River by gravity via control structure A-10 and the 36-inch northern connection to Lake River (WCS G-1), as water levels in Lake River permit. Dimensions and invert elevations for Type A water control structures are provided in Table 5.2, and a sketch of a proposed structure is provided in Figure 5.3. Typical ratings for these structures are shown in Figures 5.4 and 5.5.

Low Level Head Control Structures (WCS Type B)

The low level head control structures provide water level control up to the normal maximum operating level of +4 to +5 ft in cells 1 through 7 and +9 to +10 ft in cell 8. The low level head controls consist of a riser with adjustable stop logs discharging via a 2 ft diameter corrugated metal culvert to an adjacent cell or to the expulsion pump forebay for evacuation to Lake River. Stop logs should be 3 ft wide and should allow operating levels to vary by up to 2 ft. The culvert entrance from the riser should be fitted with a manually operated slide gate. During normal operations, the culvert's slide gate would remain open. The gate would be closed to maintain high water levels for Reed Canarygrass control. While the primary purpose of these structures is to drain excess water during normal operations, they could, with the exception of control structure B-9, also be used as a secondary means of filling cells for Reed Canarygrass control. Cell 1 should be provided with duplicate structures B-1 and B-2 to allow drainage to either cell 2 or 3 in the event that one of those two cells is flooded for Reed Canarygrass control. Cell 8 should similarly be provided with duplicate structures B-9 and B-10 to allow drainage either to cell 7 or to the 36-inch northern culvert (WCS G-1) for gravity discharge to Lake River if water levels in Lake River allow. Dimensions and invert elevations for Type B water control structures are provided in Table 5.3, and a sketch of a proposed structure is provided in Figure 5.6. Typical ratings for this structure are provided in Figure 5.7.

High Level Head Control Structures (WCS Type C)

The high level head control structures provide water level control for Reed Canarygrass suppression to elevations in the range +6.5 to +8 ft in cells 1 through 7 and +12 to +13 ft in cell 8. The high level head controls consist of a riser with adjustable stop logs discharging via a 2 ft diameter ungated corrugated metal culvert to an adjacent cell or to the expulsion pump forebay for evacuation to Lake River. Stop logs should be 3 ft wide and should allow operating levels to vary by up to 2 ft. The structure is essentially the same as the Type B structure shown in Figure 5.5, but set at a higher elevation and without a slide gate. Erosion protection should be provided at the culvert outlet. This could be done by placing riprap down to the toe of the embankment or by extending the culvert to discharge at the toe of the embankment. The purpose of the Type C structures is to drain excess water during Reed Canarygrass control operations. Cell 1 should be provided with duplicate structures C-1 and C-2 to allow drainage to either cell 2 or 3 in the event that one of those two cells is also flooded for Reed Canarygrass control. Cell 8 should similarly be provided with duplicate structures C-9 and C-10 to allow drainage either to cell 7 or to the 36-inch northern culvert (WCS G-1) for gravity discharge to Lake River if water levels in Lake River allow. Dimensions and invert elevations for Type C water control structures are provided in Table 5.3 and a sketch rating for a typical structure are provided in Figures 5.8 and 5.9, respectively.

There are several possible options for combining the functions of the Type A, B and C structures into one or two somewhat larger or more complex structures. Optimization of structure design will be done by the Portland District COE as part of detailed design and costing.

Emergency Overflow Spillways (WCS Type D)

Each cell embankment should be provided with an emergency overflow spillway to protect the embankment during flood conditions or operational failures. Spillways in embankments for cells 1 through 7 should be 12 ft wide with a crest elevation of +9 ft and 3H:1V end slopes. The spillway for cell 8 (structure D-8) should be similar but should have a crest elevation of +14 ft. The spillway sections should be armored with rip-rap extending to the toe of each face of the embankment. A sketch of a proposed structure is provided in Figure 5.10.

Gravity Connection to Lake River for Cells 1-7 (WCS E-1)

Gravity supply to and drainage from cells 1 through 7 should be via a 42-inch culvert adjacent to the existing expulsion pump. The existing 42-inch culvert and flap gate at this location has an invert elevation of +1.7 ft and is currently used for gravity drainage from Shillapoo Lake to Lake River. The proposed replacement culvert should have an invert elevation of -2 ft and controls should be provided in the form of a slide gate and a flap gate and flap gate hoist. The slide gate and flap gate hoist should be manually operated. The modifications are intended to allow controlled gravity supply of water from Lake River to Shillapoo Lake, and gravity drainage from Shillapoo Lake to Lake River as wetland and river levels permit. A sketch of the proposed control structure is provided in Figure 5.11. Ratings for the culvert assuming a 0.5 ft head loss through the flap gate (draining condition) and slide gate fully open are provided in Figure 5.12.

The recommendation to replace the existing culvert with one of the same diameter at a lower elevation is intended to improve the hydraulic performance of the structure and to account to some degree for uncertainty in simulated Lake River water levels in the analyses presented previously and in Section 5.4.

Expulsion Pump

The existing expulsion pump discharging from the east side of Shillapoo Lake to Lake River should be retained. The reported nominal pump capacity of 22.3 cfs should be confirmed. The exact capabilities of this pump and its control equipment are not known and a detailed evaluation of the pump station and its equipment should be performed to identify any necessary modifications.

Gravity Connection to Lake River for Cell 8 (WCS G-1)

Gravity supply and drainage of cell 8 will take advantage of the existing 36-inch culvert at the northern end of Shillapoo Lake. This is currently used to drain the northern end of the project area to Lake River. Available survey data show pipe invert elevations of +5.6 ft at the Shillapoo Lake end and +5.3 ft at the Lake River end of the culvert. The existing flap gate should be removed and replaced by a slide gate and a new (or modified) flap gate and hoist mechanism. The slide gate and flap gate hoist should be manually operated. The modifications are intended to allow controlled gravity supply of water from Lake River to cell 8 and gravity drainage from cell 8 to Lake River as wetland and river levels permit. An inspection and complete survey of the existing structure should be conducted to evaluate its suitability for the proposed use. The channel connection from this structure to Lake River appeared to be heavily silted during field inspection in April 1998. However no channel survey information is available and no determination has been made as to whether this channel must be dredged to allow effective use of this structure. The proposed controls for this structure are sketched in Figure 5.13 and is similar to those proposed for structure E-1 as shown in Figure 5.11. Ratings for the culvert assuming a 0.5 ft head loss through the flap gate (draining condition) and slide gate fully open are provided in Figure 5.14.

Fish Screen (WCS Type F)

The gravity supply to cells 1 through 7 (via WCS E-1) and cell 8 (via WCS G-1) should both be screened to prevent juvenile and adult fish from entering Shillapoo Lake. The most recent criteria from the National Marine Fisheries Service call for a screen opening of 1 mm when screening for juvenile salmonids. Conventional screens for the two gravity inlets to Shillapoo Lake would impose a significant maintenance burden on the project. Alternative non-conventional screening techniques (such as use of a gravel berm for screening) will be investigated by the Portland District COE.

Langsdorf Landing Pump

WDFW intends to install a pump to supply water from the Columbia River at Langsdorf Landing to existing parts of the Shillapoo Wildlife Area to the west and south of Shillapoo Lake. WDFW has applied for a water right for this purpose of 11.2 cfs and it is assumed that the water right will be granted and that the pump station will be designed to this capacity. Water in excess of demand will be available to supply the Shillapoo Lake project. It is likely that water would be needed mostly to supply cell 8 for both normal operations and for Reed Canarygrass control. The pumped supply from Langsdorf Landing would be capable of filling cell 8 to a normal maximum operating level of +10 ft in less than seven days. Pumping for about a further 20 days would be required to raise cell 8 water levels from +10 ft to +13 ft for Reed Canarygrass control. Water may also be needed for Reed Canarygrass control in cells 1 through 7 in the main body of the wetland. It is assumed that the Langsdorf Landing Pump will be capable of supplying water to the ditch system on the east side of Lower River Road at a minimum head of +16 ft. Design of this facility is being performed independently of the Shillapoo Lake project. Delivery of water from the Langsdorf Landing pump station to Shillapoo Lake will require construction of water control structures H-1, I-1, I-2, J-1, and J-2.

Buckmire Slough Control Structure (WCS H-1)

A water level control structure is required at the head of the channel leading to Buckmire Slough to direct pumped water north for delivery to Shillapoo Lake (see Plate 2). Control structure H-1 should consist of a 4 ft wide adjustable overflow gate, capable of passing the pumped supply of 11.2 cfs at a head of less than 1 ft., with an 8 ft wide check wall set at elevation +14 ft. The overflow gate should be adjustable between elevations +12 ft and +14 ft. The structure should be tied in to the banks of the channel by means of an earthen embankment. A sketch of the proposed structure is provided in Figure 5.15. WDFW is apparently also designing a control structure for the channel leading to the head of Buckmire Slough. Design of control structure H-1 should be coordinated with the WDFW work. It is possible that H-1 may be redundant, depending on the design features of the WDFW structure.

Shillapoo Lake Distribution Structures (WCS Type I)

Water control structures I-1 and I-2 are required to direct pumped water from Langsdorf Landing either north for delivery to cell 8 or east for delivery to the main body of Shillapoo Lake via cells 2 and 6 (see Plate 2). Control structures I-1 and I-2 will consist of a 4 ft wide adjustable overflow gate, capable of passing the pumped supply of 11.2 cfs at a head of less than 1 ft. The overflow gate will be adjustable between elevations +13 ft and +15 ft. A sketch of the proposed structures is provided in Figure 5.16.

Turn Out Structures (WCS Type J)

Structures J-1 and J-2 are simple turnout structures consisting of a gated 1.5-ft diameter corrugated metal pipe and headwall discharging to cells 2 or 6. A sketch of a proposed structure is provided in Figure 5.17.

Table 5.4 summarizes the different types of water control structures and the total number of each in the recommended alternative.

5.3.4 Conveyance Systems

Gravity Supply/Drainage Channels (CHN A – H, J)

The gravity supply/drainage channels for the wetland cells in the main body of Shillapoo Lake are shown on Plate 2 as channels (CHN) A through H. The channels take advantage of the existing drainage system, however the drainage system has been modified and extended as necessary to allow independent filling and draining of wetland cells to the extent possible. In the main body of Shillapoo Lake, cells 2 through 6 can all be managed independently. Cell 1 would have to be filled or drained through cells 2 or 3. Similarly, cell 8 would be drained through cell 7 to the expulsion pump when Lake River levels prevent gravity drainage via the 36-inch northern culvert WCS G-1. The principal concern regarding gravity supply of cell 8 is the condition of the historic connection from the north end of Shillapoo Lake to Lake River. This channel is known to be heavily silted but no survey information is available.

The hydraulic performance of the channels serving the main body of Shillapoo Lake is expected to be heavily influenced by the type and density of aquatic vegetation that will become established. The cross-sectional geometry of channels A through H was estimated assuming a Manning's roughness of 0.12, based on recommendations by Hydraulics Research Limited (1988), and an assumed energy slope of 0.00025 (0.25 ft per 1000 ft). It was assumed that channel E-1 leading to the expulsion pump would be required to convey at least 22.3 cfs (the expulsion pump capacity) at a flow depth of about 4 ft. Similarly, it was assumed that channels internal to the wetland cells dicharging either to the pump forebay or to channel E-1, would be required to convey at least 10 cfs at a flow depth of about 4 ft.

With the above assumptions, all channels A through H except for A-2 and E-1 should have a uniform invert of -4 ft, a bottom width of 6 ft and side slopes of 1.5H:1V to the approximate limits shown on Plate 2. There is little information on the cross-sectional geometry of the existing ditches. In particular there is only limited information on ditch inverts and it is possible that the existing ditches already have inverts at -4 ft or lower. The cross-sectional geometry proposed above is the minimum that would provide acceptable hydraulic performance. Larger sections could be excavated to provide material needed for embankment construction and anticipated siltation.

Channel A-2 is provided to intercept discharge from a spring at the south end of cell 1. Flows from the spring are currently conveyed north in an excavated channel that follows the east side of Shillapoo Lake. Channel A-2 should have approximately the same dimension as the existing channel.

Channel E-1 is an embanked channel functioning as an extension of the expulsion pump forebay and intended primarily to allow independent management of water levels in individual wetland cells. For a part of its length, the channel follows an existing minor ditch. Channel E-1 will be approximately 3,600 ft in length. It should have an invert elevation of -5 ft, a bottom width of 10 ft, and 2H:1V side slopes. It will be embanked on both sides by standard embankments with top widths of 10 ft at elevation +10 ft, 3H:1V side slopes down to elevation +6 ft, and 6H:1V side slopes below elevation +6 ft. A 12 ft wide bench will be provided at elevation +2 ft between the toe of the embankment and the top of channel bank on both sides of the channel to provide maintenance access. A typical cross-section is provided in Figure 5.18.

The historic channel connection from the northern end of Shillapoo Lake to Lake River, identified as channel J-1, should be dredged to a minimum depth of 0 ft.

Conveyance Channels for Langsdorf Landing Pumped Supply (CHN I-1, I-2, I-3)

Pump supply channels I-1 through I-3 (see Plate 2) follow the alignment of the historical connection to the Columbia River on the west side of Shillapoo Lake. These channels currently have existing invert elevations ranging from about +10 to +12 ft, bottom widths of from about 15 ft (channel I-1) to 40 ft (channel I-3), approximately 2H:1V side slopes, and minimum top of bank elevations of about +15 ft. The channels have significantly greater capacity than needed for supply purposes for this project. The channel reaches identified on Plate 2 should be cleared of any significant local obstructions. Local high spots in the inverts should be lowered to +10 ft, and channel banks should be raised locally as necessary to +16 ft.

HDPE Supply Pipe to Cell 8

A 24-inch diameter HDPE pipe should be provided from the northern end of channel I-2 to the southwest corner of cell 8. The pipe should be approximately 1,200 ft in length with invert elevations of +11 ft at the upstream end and +10 ft at the downstream. Under a differential head of 2 ft the pipe will be capable of supplying approximately 9 cfs to cell 8 enabling the cell to be filled for Reed Canarygrass control from elevation +10 ft to +12 ft in approximately 15 days and from elevation +10 ft to +13 ft in about 24 days.

5.4 Analysis of Recommended Alternative

Two different analytical tools were used to evaluate the hydraulic performance of the Recommended Alternative: a simple UNET model, and a more detailed ROUTE model. The UNET model was similar to that used for the base level evaluations described in Chapter 4 in that it modeled the dynamics of the Columbia River/Vancouver Lake/Lake River interactions and incorporated the gravity connection to Lake River, but treated Shillapoo Lake as a single storage element (i.e. it did not incorporate the proposed internal cell configuration). The ROUTE model in contrast took Lake River levels as a boundary condition, and incorporated the 42-inch gravity connection to Lake River (WCS E-1), the internal wetland cell configuration, and all significant interior water control structures to allow detailed modeling of cell water levels within Shillapoo Lake.

ROUTE is a library of routines developed by NHC to support continuous water balance modeling and simple hydraulic routing using hourly time series data. The internal time step at which ROUTE operates can be reduced to 1 minute or less. Computations at each time step assume steady state conditions, but use of a small time step allows quasi unsteady state simulations. ROUTE is patterned after the Canadian Inland Waters Directorate SIMPAK model (Environment Canada, 1979) which performs a similar function but operates strictly at a daily time step.

ROUTE provides considerably greater flexibility than UNET in the simulation of interior runoff for Shillapoo Lake. It was configured to determine interior runoff for each wetland cell from the summation of rainfall falling on the flooded area of the cell and HSPF generated runoff for the remaining tributary area of the cell. The program is capable of tracking and accounting for the hour to hour variation in flooded and non-flooded area as wetland water levels change throughout the period of simulation. ROUTE also allows interior runoff to be augmented by an additional constant inflow to each cell (which could be a different constant value for each cell) to represent seepage inflows, which as stressed earlier are not simulated by HSPF.

As pointed out earlier, no data are available on seepage rates into Shillapoo Lake. The total perimeter length of the lake at elevation +15ft (a representative elevation for the Columbia River under high flow conditions - see Appendix A) is approximately 6.25 miles. Assuming seepage inflows ranging from 0.5 to 2 cfs per mile, results in total seepage inflows to Shillapoo Lake from about 3 cfs to 12.5 cfs. Given the lack of information on seepage inflows, selected simulations were performed for total seepage inflows of 1, 5, and 10 cfs.

The following simulations were carried out to evaluate system performance:

5.4.1 Dry Year Filling

The consistently lowest fall (October - December) water levels on the Columbia River at Vancouver in the period 1973 - 1997 were in water year 1977 (see Appendix A). UNET simulations were performed to evaluate the ability to fill the main body of Shillapoo Lake (cells 1 through 7) by gravity to a target elevation of +4 ft before 31 December for conditions similar to water year 1977. Since detailed boundary condition data were not available to run UNET for water year 1977, UNET simulations were run for October 1994 under the following assumptions:

- no interior runoff and no seepage inflows to Shillapoo Lake, and,
- a gravity connection consisting of the 42-inch culvert at invert elevation +1.7 ft with flap gate and slide gate both fully open.

October 1994 water levels for the Columbia River at Vancouver are actually lower than those for October 1976. It was judged that if simulated Shillapoo Lake water levels approached +4 ft by late October 1994 with gravity inflow from Lake River under the above assumptions, then target water levels could be readily achieved by December 31 in a repeat of water year 1977.

UNET simulation results showing water levels for the Columbia River at Vancouver, Lake River at Felida Moorage, and Shillapoo Lake are shown in Figure 5.14. Shillapoo Lake water levels reach +3.5 ft by October 26 before the end-of-month rise in Lake River water levels. Although the UNET simulations do not include hydraulic losses due to interior water control structures, rates of inflow to Shillapoo Lake by late October are small enough that such losses become negligible. Overall, losses are expected to delay refill by a matter of only a few days.

Note that the final recommendation, as provided in Section 5.3.3 above, calls for the 42-inch culvert WCS E-1 to be lowered to elevation –2 ft to improve its hydraulic performance at low Lake River water levels.

5.4.2 Maintenance of Normal Maximum Water Levels

The ability of the proposed water control structures and pump station to maintain normal maximum water levels was investigated using the ROUTE model by simulating the interior runoff and routing through the network of cells for the periods November 1994 through May 1995, November 1995 through May 1996, and November 1996 through May 1997. Simulations for each period assumed that the wetland cells had been filled to a normal maximum water level of +4 ft for cells 1 through 7 and +9 ft for cell 8 by October 31. It was further assumed that cell 8 was drained through cell 7 to the expulsion pump and that Lake River levels throughout the simulations were too high to permit gravity drainage.

The simulations include the following periods of particular interest:

- Recalling (Section 3.2.6) that HSPF-generated runoff and rainfall used in the simulations are based on data from Portland WSFO AP (NWS station 356751) scaled up by a factor of 1.14, the wettest months of water year 1995 (November 1994 through January 1995) represent close to average conditions.
- Water year 1996 simulations include the February 1996 storm that produced approximately 100-year 3-day and 5-day rainfall amounts at Portland.
- Water year 1996 simulations included a total rainfall input for the 4-month period November 1995 through February 1996 of 38.2 inches (Portland adjusted), which compares closely with the greatest 4-month rainfall amount experienced at the Vancouver 4NNE gage in the period 1973 - 1997 of 37.6 inches.
- Water year 1997 simulations included a total rainfall input for the 3-month period November 1996 through January 1997 of 34.2 inches (Portland adjusted), which compares closely with the greatest 3-month rainfall amount experienced at the Vancouver 4NNE gage in the period 1973 - 1997 of 32.4 inches.

Results in Figures 5.15 through 5.26 show simulated water levels in the expulsion pump forebay and in wetland cells 1, 2 and 4. Cell 1 is the single largest cell; cell 2, through which cell 1 was assumed to drain, has the largest total tributary area; and cell 4 is the smallest cell. Simulations were performed using three seepage rates as discussed above: 1 cfs, 5 cfs, and 10 cfs.

For the average year (water year 1995, Figures 5.15 through 5.18) water levels in all cells are maintained within 0.5 ft of the target for the entire simulation period, and simulation results are relatively insensitive to seepage rates.

For wet years (water years 1996 and 1997, Figures 5.19 through 5.22 and 5.23 through 5.26), water levels are quite sensitive to seepage rates. Water levels exceed the target levels by more than 0.5 ft for significant periods for all assumed seepage rates. With the 10 cfs seepage rate, the pump is activated in mid-November and is operated continuously until the end of the simulation periods on May 31. Simulated water levels reach a high of about +5.5 ft in water year 1996 and +6.5 ft in water year 1997, and remain significantly higher than the target level of +4 ft at the end of the simulations. The deviation from the targets levels are much less with lower the seepage rates of 1 cfs and 5 cfs and water levels return to close to the target level by the end of the simulation periods.

With the expulsion pump at its existing capacity and the proposed water control structures, water level fluctuations meet the performance standards discussed in Section 5.2 in an average year. Deviations from the performance standards in wet years were discussed with COE and WDFW biologists and determined to be acceptable.

5.4.3 Wet Year Draining

The ability of the proposed water control structures and pump station to drain the lake from normal maximum water levels was investigated using the ROUTE model by simulating the interior runoff and routing through the wetland cells for conditions similar to the period June-July 1983, the wettest June and July at Vancouver in the period 1973 through 1997. Since HSPF-generated runoff data were not available for 1983, simulations of the ability to drain the project were actually done for March through mid-May 1995. The rainfall total for March-April 1995 used in the simulation was 8.33 inches as opposed to a June-July 1983 total at Vancouver of 7.40 inches. Evaporation from open water surfaces was ignored. To meet the performance standards presented in Section 5.2, it should be possible to completely drain the project by July 31 (simulated April 30), starting with the project at its normal maximum operating level on June 1 (simulated March 1).

The ROUTE simulations were done assuming wetland water levels in cells 1 through 7 starting at +4.5 ft and cell 8 starting at +9.5 ft on June 1 (simulated March 1), the main fill\drain structures (A-1 through A-9) open with cell 8 draining to cell 7, and the expulsion pump operating at its nominal capacity of 22.3 cfs. Simulated pump forebay elevations for seepage rates of 1, 5, and 10 cfs are shown in Figure 5.27. Simulated cell water levels (not shown) track the simulated forebay elevations quite closely until cells are essentially empty at about 0 ft. It can be seen that the results are again sensitive to the assumed seepage rate. With a seepage rate of 1 cfs, the project is emptied by round July 24 (simulated April 24), and meets the target two-month period for project drawdown. With a seepage rate of 5 cfs, the target drawdown period is not quite met, with the project not being emptied until about August 11 (simulated May 11). With a 10 cfs seepage rate, the expulsion pump is barely able to keep up with interior runoff and seepage inflows and the project ends the simulation period close to full at elevation + 3.3 ft. Given the history of farming the Shillapoo Lake bed, the 10 cfs seepage rate is believed to be unreasonably high. The performance for lower seepage rates was reviewed by COE and WDFW biologists and again determined to be acceptable.

5.4.4 Impact on Flooding

Since the Recommended Alternative provides no direct or indirect gravity connection from the Columbia River to Lake River, the recommended project will have no impact on flooding levels, external to Shillapoo Lake; i.e., around Vancouver Lake or along Lake River.

Under the Recommended Alternative, water levels in the main body of Shillapoo Lake could be as high as +8 to +9 ft and in the northern cell (cell 8) +12 to +14 ft. It is assumed that property acquisition will include all land below elevation +10 ft in the main body of Shillapoo Lake and below +15 ft in the northern part of project area.

5.5 Project Operation

Operation of the project can be split into four distinct tasks by season:

- filling of wetland cells to normal maximum water levels (October - December)
- maintenance of normal maximum water levels (October - May)
- flooding of cells for Reed Canarygrass control (March - April)
- draining of cells (May - July)

5.5.1 Filling of Wetland Cells to Normal Operating Levels

Filling Cells 1 through 7

Filling of cells 1 through 7 to normal operating levels can be achieved by gravity even in unusually dry years. Filling will normally be started about October 1 with the maximum normal operating level of about +4 ft to be achieved between about October 31 and December 31. The rate of filling will depend on the stage in Lake River and interior runoff amounts. Assuming the project is starting empty on October 1, filling to normal maximum water levels will be done as follows:

Prepare Cells 1 - 7 for Filling

1. Open slide gates on the main drains for cells to be filled (WCS A-1 through A-8) if not already open.
2. Close slide gates on the main drains for cells not to be filled.
3. Set stop logs in the low level head-control structures (WCS B-1 through B-8) to the desired initial filling level for each cell.
4. Open slide gates on the low level head-control structures for each cell.
5. Shut down the expulsion pump.
6. Close the slide gate on the 42-inch pipe connection to Lake River (WCS E-1).
7. Open the fill valve to relieve seating pressure on the flap gate.
8. Hoist the 42-inch flap gate at WCS E-1 open and secure.

Fill Cells 1-7

9. Partially open the slide gate on the 42-inch connection to Lake River (WCS E-1) and modulate to allow filling of the project at the desired rate. The maximum filling rate should be no greater than 0.2 ft/day for the project as a whole.
10. Monitor water levels in the individual wetland cells and the adjust the opening of the slide gate on the 42-inch connection to Lake River and the main drain slide gates as necessary to match desired filling rates.

Maintain Water Levels in Cells 1 - 7

11. When water levels in individual cells have reached the desired level, close main drain slide gates (WCS A-1 through A-8).
12. When all cells have reached desired levels, close the slide gate on the 42-inch Lake River connection (WCS E-1) and any main drain slide gates still open. Leave slide gates on the low level head-control structure for each cell open.
13. Lower the 42-inch flap gate and leave in the normal operational position.
14. Open the slide gate on the 42-inch Lake River connection.
15. Turn on the expulsion pump with pump-on setting of +2 ft and pump-off setting of -2 ft, assuming all cells filled. The pump-on setting should be -2 ft and the pump-off setting -4 ft if any cells are to be maintained dry.

Filling Cell 8 by Gravity from Lake River

Filling of cell 8 to normal operating levels will be by gravity during periods when water levels in Lake River are high enough. Otherwise this cell will be filled by pumping from the WDFW Langsdorf Landing pump station as excess pumped water is available. Filling will take advantage of high water conditions and excess pumped water as opportunities arise. Filling to elevation +10 ft would normally be achieved between about October 31 and December 31. The rate of filling by gravity will depend on the stage in Lake River and interior runoff amounts. Assuming the project is starting empty on October 1, filling by gravity to normal maximum water levels will be done as follows:

Prepare Cell 8 for Filling

1. Open the slide gate on the northern drain (WCS A-10) if not already open and close the slide gate on the southern drain (WCS A-9).
2. Set stop logs in the low level head-control structures WCS B-9 and B-10 to the desired initial filling level for the cell.
3. Close the slide gate on the 36-inch pipe connection to Lake River (WCS G-1).
4. Open the fill valve for relieving seating pressure on the flap gate.
5. Hoist the 36-inch flap gate open and secure it in the open position.

Fill Cell 8

6. Partially open the slide gate on the 36-inch connection to Lake River and modulate to allow filling of the project at the desired rate. The maximum filling rate should be no greater than 0.2 ft/day.
7. Monitor water levels in the cell and the adjust the opening of the slide gate on the 36-inch connection to Lake River as necessary to match desired filling rate.

Maintain Water Level in Cell 8

8. When the cell has reached the desired level, close the slide gate on the 36-inch Lake River connection and close the main drain slide gate A-10. Leave slide gates on the low level head-control structures open.
9. Lower the 36-inch flap gate and leave in the normal operational position.
10. Open the slide gate on the 36-inch Lake River connection.

Filling Cell 8 by Pumped Supply from Langsdorf Landing

Filling from the Langsdorf Landing pump will be done in those years when low Lake River water levels preclude filling by gravity. Assuming the project is starting empty on October 1, filling of cell 8 from the Langsdorf Landing pump station to normal maximum water levels will be done as follows:

Prepare Cell 8 for Filling

1. Close the slide gate on the main cell drains (WCS A-9 and A-10).
2. Set stop logs in the low level head-control structures WCS B-9 and B-10 to the desired initial filling level for the cell.
3. Open slide gates on the low level head-control structures.
4. Open the slide gate on the 36-inch pipe connection to Lake River (WCS G-1) and the lower the 36-inch flap gate to normal operational position.
5. Fully raise the overflow gate on structure WCS I-2 (to elevation +15 ft).
6. Set the overflow gate on structure WCS I-1 at +13 ft.

Fill Cell 8

7. Activate the Langsdorf Landing pump station and adjust the overflow gate on structure WCS H-1 as necessary to control the rate of filling of cell 8, and to distribute flows between cell 8 to the north and the channel leading to Buckmire Slough (south from WCS H-1) as may be required by WDFW.

Maintain Water Level in Cell 8

8. When the cell has reached the desired level, shut down the pump and fully raise overflow gate I-1 to elevation +15 ft.

5.5.2 Maintenance of Normal Maximum Operating Levels

Once the project is filled, normal maximum operating levels will be maintained through June 1 if no special operations for Reed Canarygrass control are needed or through March 15 if flooding for Reed Canarygrass control is needed. Maximum normal water levels are maintained as follows:

1. Set stop logs in the low level head-control structures at the desired levels for each cell. Stop logs for B-10 should be maintained 0.5 ft lower than B-9 to provide for gravity drainage from cell 8 to Lake River via WCS G-1, as Lake River water levels permit.
2. Operate the expulsion pump throughout with a pump-on setting of +2 ft and a pump-off setting of -2 ft.

5.5.3 Flooding for Reed Canarygrass Control

Flooding for Reed Canarygrass control may take place between 15 March and May 1. Flooding of cells 1 through 7 is likely to take place by gravity from Lake River. In rare instances, it may be necessary to supplement the gravity supply for cells 1 through 7 by means of water pumped from Langsdorf Landing. The target water level for Reed Canarygrass control in cells 1 through 7 is expected to be between +6.5 ft and +8 ft. Flooding of cell 8 for Reed Canarygrass control is assumed to use the pumped supply from Langsdorf Landing. The target water level for Reed Canarygrass control in cells 8 is expected to be between +12 ft and +13 ft

Flooding Cells 1 through 7 by Gravity

Flooding for Reed Canarygrass control may take place between 15 March and May 1. Flooding of cells 1 through 7 by gravity presumes that Lake River water levels exceed the Reed Canarygrass control level which is expected to be in the range +6.5 to +8 ft. The procedure for flooding an individual cell will be as follows:

Prepare Cells 1 - 7 for Flooding

1. Set stop logs in the high level head control structure for the cell in question (WCS C-1 through C-8) to the desired level.
2. Close the slide gate on the low level head control structure for the cell in question (WCS B-1 through B-8).
3. Shut down the expulsion pump.
4. Close the slide gate on the 42-inch pipe connection to Lake River (WCS E-1).
5. Open the fill valve for relieving seating pressure on the flap gate.
6. Hoist the 42-inch flap gate open and secure in the open position

Flood Cells 1-7

7. Partially open the slide gate on the 42-inch connection to Lake River and fill the pump forebay to the cell's current water level.
8. Fully open the slide gate on the main drain (A-1 through A-8) to the cell to be flooded.
9. Monitor water levels in the wetland cell and the pump forebay and modulate the slide gate on the 42-inch connection to Lake River to allow filling of the cell for Reed Canarygrass control at the desired rate subject to a maximum forebay water level of +8.5 ft.

Maintain Water Levels in Cells 1 - 7

10. When the water level in the cell has reached the desired level, close the slide gate on the 42-inch Lake River connection and close the cell's main drain slide gate.
11. Turn on the expulsion pump with pump-on setting of +2 ft and pump-off setting of -2 ft, assuming all cells filled. The pump-on setting should be -2 ft and the pump-off setting -4 ft if any cells are to be maintained dry.

12. Lower the 42-inch flap gate and leave in the normal operational position.
13. Open the slide gate on the 42-inch Lake River connection.

Note that cell 1 cannot be flooded independently of other cells. Water for cell 1 must be supplied via cells 2 or 3, and these cells then drained back to normal operating levels.

Flooding Cell 8 by Pumped Supply from Langsdorf Landing

Flooding for Reed Canarygrass control may take place between 15 March and May 1. It is assumed that flooding of cell 8 by means of the pumped supply from Langsdorf Landing would be done when Lake River levels are below the target Reed Canarygrass control level for cell 8 which is expected to be in the range +12 to +13 ft. The procedure for flooding cell 8 will be as follows:

<u>Prepare Cell 8 for Flooding</u>

1. Set stop logs in the high level head control structures for the cell (WCS C-9 and C-10) to the desired level. Maintain stop logs for C-10 0.5 ft lower than C-9 to provide for gravity drainage from cell 8 to Lake River via WCS G-1, as Lake River water levels permit.
2. Close the slide gates on the low-level head control structures for cell 8 (WCS B-9 and B-10).
3. Close the slide gate on the 36-inch pipe connection to Lake River (WCS G-1).
4. Fully raise the overflow gate on structure WCS I-2 (to elevation +15 ft).
5. Set the overflow gate on structure WCS I-1 at +13 ft.

<u>Flood Cell 8</u>

6. Activate the Langsdorf Landing pump station and adjust the overflow gate on structure WCS H-1 as necessary to control the rate of filling of cell 8 and to distribute flows between cell 8 and the channel leading south to Buckmire Slough as may be required by WDFW.

<u>Maintain Water Level in Cell 8</u>

7. When cell 8 has reached the desired level, shut down the pump and fully raise the overflow gate I-1 to elevation +15 ft.

5.5.4 Draining the Project

Draining the project will begin about May 1 if flooding has been used for Reed Canary grass control or about June 1 if normal maximum water levels have been maintained throughout the spring. Operations for two scenarios are described below: draining of the entire project (from normal maximum operating levels) by the expulsion pump for pumped discharge to Lake River for high Lake River levels; and gravity drainage of cell 8 to Lake River for low Lake River water levels.

Discharge via the Expulsion Pump

Draining the project to Lake River via the expulsion pump would be accomplished as follows:

1. Adjust the set points for the expulsion pump to pump-on at elevation -2 ft and pump-off at -4 ft.
2. Partially or fully open the main drain structures slide gates (WCS A-1 through A-9) to control draw down to desired rates.
3. Once the project is fully evacuated, leave the main drain slide gates open and operate the pump until project refill in the fall.

Note that under this scenario, cell 8 will be drained to cell 7 via WCS A-9 and thence to the expulsion pump. Similarly, cell 1 would be drained through cell 2 or 3.

Gravity Drainage of Cell 8 to Lake River

Gravity Drainage of Cell 8 to Lake River would be via the 36-inch northern culvert WCS G-1, as follows:

1. Lower the flap gate on the 36-inch northern drainage structure WCS G-1 and fully open the slide gate if not already open.
2. Partially or fully open the main drain structure slide gate on WCS A-10 to control draw down to desired rates.
3. Once the cell is fully evacuated, leave the main drain slide gate and the slide gate on structure WCS G-1 open until project refill in the fall.

5.5.5 Monitoring Stages

Water level staff gages should be placed in each wetland cell to allow monitoring of stages during filling, flooding, or draining operations and to ensure target water levels are maintained over extended periods. It is suggested that the staff gages be placed near to (i.e., visible from) the main fill/drain structures to each cell (WCS Type A). The staff gages can be attached to the culvert head wall (at the slide gate), if they can remain on the wetland cell side of the head wall (i.e., the side facing away from the embankment) and still be read easily. At most locations, the gage will probably need to be attached to a solitary rod or board and driven into the lake bed a few feet away from the fill/drain structure, with the numbers facing the embankment for easy viewing. Staff gages should also be placed on both sides of the pump at Langsdorf Landing, the expulsion pump to Lake River, and the existing outlet culvert at the north end of Shillapoo Lake.

6.0 CONCLUSION & RECOMMENDATIONS

The Recommended Alternative in Chapter 5 presents a project layout and conceptual level hydraulic design of water control structures which will allow Shillapoo Lake to be managed to meet target water level regimes established by WDFW for wetland restoration, as described in Chapter 2.

The following additional work should be performed prior to or as part of final hydraulic design of the project:

1. Collection of stage data on Lake River at Felida Moorage should continue at least through the fall of 1998. Simulations of project performance presented in this report rely on only two months of Lake River stage data during conditions of moderately high stage. Simulated stages during low water conditions should be validated against observed data.

2. The assumed 22.3 cfs capacity of the existing "expulsion" pump should be verified and the pump station and its control equipment inspected to identify necessary modifications.

3. It is proposed that the 36-inch culvert at the north end of Shillapoo Lake be incorporated into the design of the Recommended Alternative. The culvert should be inspected to determine its suitability for use in this project.

4. The design of structures for the control of pumped discharges from Langsdorf Landing should be coordinated with similar design work being conducted by WDFW.

5. The final design and layout of the channel system interior to Shillapoo Lake should be coordinated with material requirements for embankment construction.

6. Options for optimizing the conceptual design of level control and drainage structures to reduce the currently proposed number of structures, should be pursued during final design and costing.

7. An attempt should be made to estimate seepage inflows to Shillapoo Lake from pump station records. The information obtained should be used to determine whether seepage rates assumed in this report are reasonable.

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Table 2.1
Monthly Precipitation (inches)
Vancouver 4 NNE

Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Water Year Total
1973	0.78	5.02	9.45	4.51	2.21	2.84	1.28	2.31	1.90	0.11	0.72	3.28	34.41
1974	3.39	12.92	10.44	9.02	5.18	5.84	3.06	2.40	0.88	2.91	0.18	0.22	56.44
1975	2.25	7.24	6.94	8.55	5.74	4.04	2.13	1.64	1.13	0.54	2.68	0.00	42.88
1976	6.35	3.97	7.37	5.50	5.08	3.06	2.58	2.78	0.98	1.12	3.66	1.02	43.47
1977	1.87	1.13	1.64	1.35	2.33	3.57	0.97	4.16	1.45	0.40	3.05	3.80	25.72
1978	2.45	5.64	8.87	5.15	4.68	1.83	4.16	3.94	1.22	1.35	2.90	M	M
1979	1.09	3.54	3.60	2.92	7.19	2.73	3.78	2.15	0.71	0.41	1.62	2.34	32.08
1980	5.79	3.80	6.75	8.85	5.04	3.38	3.55	1.74	1.91	0.26	0.48	1.70	43.25
1981	1.64	6.78	10.83	1.93	4.58	2.91	2.66	3.90	3.45	0.49	0.09	2.33	41.59
1982	4.06	4.40	8.91	7.53	6.82	3.12	3.58	0.44	1.24	1.08	1.42	4.33	46.93
1983	4.98	4.16	M	7.13	8.71	6.20	2.81	1.86	3.65 E	3.75	2.38	1.06	M
1984	2.29	10.58	5.86	3.05	M	4.81	3.76	4.28	4.02	0.00	0.13	1.81	M
1985	4.63	11.74	3.12	0.29	2.92	3.81	1.40	1.89	2.88	0.31	0.76	3.21	36.96
1986	3.25	4.77	2.39	6.66 E	6.46	3.28	2.34	3.23	0.69	1.28	0.10	4.44	38.89
1987	2.04	7.44	4.76	6.84	5.00	6.99	2.22	1.97	0.46	1.72	0.60	0.52	40.56
1988	0.45	2.40 E	8.76	5.73	2.11	4.47	3.75	3.24	2.88	0.44	0.20	1.41	35.84
1989	0.25	9.50	3.00	4.01 E	2.88	7.16	2.32	2.18	0.75	1.11	1.71	0.92	35.79
1990	2.60	3.64	3.23	9.27	3.92	2.96	2.41	2.47	1.65	0.69	0.91	0.26	34.01
1991	5.71	4.31	3.58	3.23	3.95	4.70	4.59	3.38	M	0.12	1.18	0.04	M
1992	1.91	6.91	5.18	4.71 E	4.40	2.05	4.24	0.17	0.64	0.05 E	0.71	1.50	32.47
1993	2.15	6.04	6.12	3.94	0.81	4.59	7.72	4.41	1.78	1.53	0.29	0.00	39.38
1994	1.49	1.34	6.16	4.65	5.01	2.52	2.32	1.05	2.45	0.00	0.10	1.07	28.16
1995	5.72	7.97	7.58	7.69	3.41	4.25	4.19	1.98	2.05	1.32	0.92 M	1.98	49.06
1996	4.62	10.68	6.63	7.44	10.58	2.85	5.40	4.65	0.94	0.70	0.23	2.79	57.51
1997	6.17	9.42	13.26	9.27	2.55	6.88	3.62	2.00	3.07	0.65	1.77	3.00	61.66
Average (1973-1997)	3.12	6.21	6.43 E	5.57	4.65 E	4.03	3.23	2.57	1.78 E	0.89	1.15 E	1.79 E	40.81 E

E - Estimated (up to two days data missing in month)
M - Missing (entire month missing - where value is given it is estimated from Portland WSFO AP)

Table 2.2
Monthly Precipitation (inches)
Portland WSFO AP

Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Water Year Total
1994	1.59	1.50	5.01	3.56	4.92	1.84	1.91	0.56	1.67	0.07	0.13	1.13	23.89
1995	8.41	5.91	4.85	5.56	3.19	3.82	3.49	1.65	2.62	1.23	0.81	1.31	42.85
1996	3.15	10.64	5.90	7.04	9.95	3.22	5.12	4.76	0.65	0.77	0.21	2.82	54.17
1997	5.29	9.47	13.12	7.38	1.61	7.06	3.73	3.63	2.83	0.52	1.58	1.98	58.20
Average (1994-1997)	4.61	6.88	7.22	5.89	4.92	3.99	3.56	2.65	1.94	0.65	0.68	1.81	44.78

Table 2.3
Stage-Duration Analysis for Stage Above Threshold
Columbia River at Vancouver

Water Year	Number of Days Mean Daily Stage at Vancouver (RM 106.5) Exceeds +6 NGVD															
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Oct - Dec	Total Oct - Mar	Total Oct - May	Total Mar - Apr
1973	2	8	28	30	12	22	8	10	4	1	1	0	38	102	120	30
1974	0	22	31	31	28	31	30	31	30	31	18	3	53	143	204	61
1975	2	12	28	31	28	31	30	31	30	24	2	0	42	132	193	61
1976	8	30	31	31	29	31	30	31	30	31	31	20	69	160	221	61
1977	7	6	5	13	3	10	0	10	5	0	0	0	18	44	54	10
1978	1	21	31	31	28	26	30	31	30	25	1	14	53	138	199	56
1979	2	8	24	19	26	31	29	31	16	2	0	0	34	110	170	60
1980	6	17	31	31	26	21	21	31	30	13	0	0	54	132	184	42
1981	0	15	31	31	28	24	28	31	30	31	12	0	46	129	188	52
1982	7	19	30	31	28	31	30	31	30	31	13	3	56	146	207	61
1983	5	27	31	31	28	31	30	31	30	31	17	4	63	153	214	61
1984	0	28	31	31	29	31	30	31	30	24	6	0	59	150	211	61
1985	10	29	31	31	28	24	30	31	23	4	0	0	70	153	214	54
1986	4	23	19	29	28	31	30	31	27	10	0	1	46	134	195	61
1987	2	23	26	31	28	26	12	31	12	0	0	0	51	136	179	38
1988	1	2	27	29	11	9	19	31	17	0	0	0	30	79	129	28
1989	1	23	28	27	18	28	30	31	22	1	0	0	52	125	186	58
1990	0	12	12	27	28	31	30	31	30	22	1	0	24	110	171	61
1993	0	5	25	23	11	17	30	31	29	17	1	0	30	81	142	47
1994	0	0	8	17	10	18	18	31	26	4	0	0	8	53	102	36
1995	0	12	29	31	28	31	23	31	30	26	4	0	41	131	185	54
1996	5	23	31	31	29	31	30	31	30	31	20	0	59	150	211	61
1997	5	14	31	31	28	31	30	31	30	31	30	11	50	140	201	61
Average (1975-1997)	3.0	16.5	26.0	28.2	23.5	26.0	25.1	29.2	24.8	17.0	6.8	2.4	45.5	123.1	177.4	51.1

Table 2.3 (continued)
Stage-Duration Analysis for Stage Above Threshold
Columbia River at Vancouver

Water Year	Number of Days Mean Daily Stage at Vancouver (RM 106.5) Exceeds +8 NGVD															
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Oct - Dec	Total Oct - Mar	Total Oct - May	Total Mar - Apr
1973	0	0	15	12	0	0	0	0	0	0	0	0	15	27	27	0
1974	0	20	31	30	28	31	30	31	30	26	0	0	51	140	201	61
1975	0	0	12	27	21	31	15	30	30	11	0	0	12	91	136	46
1976	0	8	30	31	23	26	30	31	30	23	20	1	38	118	179	56
1977	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1978	0	6	28	25	12	7	27	31	22	0	0	0	34	78	136	34
1979	0	0	12	1	20	12	8	27	1	0	0	0	12	45	80	20
1980	0	0	9	18	5	8	4	28	29	0	0	0	9	40	72	12
1981	0	0	20	21	14	0	0	21	30	13	0	0	20	55	76	0
1982	0	1	29	27	28	31	30	31	30	23	0	0	30	116	177	61
1983	1	3	29	29	27	31	28	31	25	13	0	0	33	120	179	59
1984	0	14	26	29	24	24	30	31	30	10	0	0	40	117	178	54
1985	0	24	17	9	15	4	20	28	13	0	0	0	41	69	117	24
1986	0	5	7	11	26	31	28	31	19	0	0	0	12	80	139	59
1987	0	9	7	11	6	9	0	17	0	0	0	0	16	42	59	9
1988	0	0	9	12	0	0	2	12	8	0	0	0	9	21	35	2
1989	0	9	5	16	1	23	24	30	11	0	0	0	14	54	108	47
1990	0	0	1	13	25	17	11	19	30	3	0	0	1	56	86	28
1993	0	1	3	0	0	11	10	30	16	0	0	0	4	15	55	21
1994	0	0	0	0	3	0	0	1	0	0	0	0	0	3	4	0
1995	0	1	15	13	23	17	9	31	30	4	0	0	16	69	109	26
1996	0	18	31	31	29	31	29	31	30	18	0	0	49	140	200	60
1997	0	12	31	31	28	31	29	31	30	28	0	0	43	133	193	60
Average (1975-1997)	0.0	5.7	16.0	17.3	15.6	16.3	15.8	24.0	19.3	7.5	0.9	0.0	21.7	70.8	110.7	32.1

Table 2.3 (continued)
Stage-Duration Analysis for Stage Above Threshold
Columbia River at Vancouver

	Number of Days Mean Daily Stage at Vancouver (RM 106.5) Exceeds +10 NGVD															
Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Oct - Dec	Total Oct - Mar	Total Oct - May	Total Mar - Apr
1973	0	0	9	6	0	0	0	0	0	0	0	0	9	15	15	0
1974	0	7	22	21	23	28	30	31	30	13	0	0	29	101	162	58
1975	0	0	4	15	4	16	5	24	30	4	0	0	4	39	68	21
1976	0	0	18	21	9	11	20	31	24	1	0	0	18	59	110	31
1977	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1978	0	4	22	7	3	0	2	13	3	0	0	0	26	36	51	2
1979	0	0	0	0	8	2	0	10	0	0	0	0	0	10	20	2
1980	0	0	0	9	0	0	0	6	25	0	0	0	0	9	15	0
1981	0	0	12	7	8	0	0	6	29	3	0	0	12	27	33	0
1982	0	0	24	12	14	31	27	29	30	13	0	0	24	81	137	58
1983	0	0	13	16	19	31	19	25	15	0	0	0	13	79	123	50
1984	0	8	11	6	13	17	23	31	30	4	0	0	19	55	109	40
1985	0	1	3	0	0	0	3	2	2	0	0	0	4	4	9	3
1986	0	0	0	0	11	26	11	7	10	0	0	0	0	37	55	37
1987	0	2	0	0	4	0	0	2	0	0	0	0	2	6	8	0
1988	0	0	2	6	0	0	0	0	1	0	0	0	2	8	8	0
1989	0	2	0	5	0	2	0	8	0	0	0	0	2	9	17	2
1990	0	0	0	7	6	0	5	2	24	0	0	0	0	13	20	5
1993	0	0	0	0	0	6	0	26	11	0	0	0	0	6	32	6
1994	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1995	0	0	2	8	11	1	0	10	6	0	0	0	2	22	32	1
1996	0	5	25	31	28	27	22	27	27	1	0	0	30	116	165	49
1997	0	7	22	31	28	29	23	31	30	11	0	0	29	117	171	52
Average (1975-1997)	0	1.6	8.2	9.0	8.2	9.9	8.3	14.0	14.2	2.2	0	0	9.8	36.9	59.1	18.1

Table 2.3 (continued)
Stage-Duration Analysis for Stage Above Threshold
Columbia River at Vancouver

	Number of Days Mean Daily Stage at Vancouver (RM 106.5) Exceeds +12 NGVD															
Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Oct - Dec	Total Oct - Mar	Total Oct - May	Total Mar - Apr
1973	0	0	5	0	0	0	0	0	0	0	0	0	5	5	5	0
1974	0	2	15	17	10	9	26	30	30	11	0	0	17	53	109	35
1975	0	0	0	5	0	3	0	18	19	0	0	0	0	8	26	3
1976	0	0	10	13	3	4	6	25	4	0	0	0	10	30	61	10
1977	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1978	0	0	14	0	0	0	0	0	0	0	0	0	14	14	14	0
1979	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1980	0	0	0	5	0	0	0	0	0	0	0	0	0	5	5	0
1981	0	0	6	4	5	0	0	2	26	0	0	0	6	15	17	0
1982	0	0	12	6	12	20	11	20	18	8	0	0	12	50	81	31
1983	0	0	4	9	15	31	12	12	14	0	0	0	4	59	83	43
1984	0	0	0	0	4	3	12	22	30	3	0	0	0	7	41	15
1985	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1986	0	0	0	0	7	15	0	2	8	0	0	0	0	22	24	15
1987	0	0	0	0	2	0	0	0	0	0	0	0	0	2	2	0
1988	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1989	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1990	0	0	0	3	0	0	0	0	12	0	0	0	0	3	3	0
1993	0	0	0	0	0	2	0	17	3	0	0	0	0	2	19	2
1994	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1995	0	0	0	0	9	0	0	0	0	0	0	0	0	9	9	0
1996	0	3	24	20	23	20	19	22	21	0	0	0	27	90	131	39
1997	0	3	15	23	12	21	14	31	30	0	0	0	18	74	119	35
Average (1975-1997)	0	0.3	4.6	4.6	4.4	5.6	4.3	8.7	9.3	1.0	0.0	0.0	4.9	19.5	32.6	9.9

Table 2.3 (continued)
Stage-Duration Analysis for Stage Above Threshold
Columbia River at Vancouver

	Number of Days Mean Daily Stage at Vancouver (RM 106.5) Exceeds +14 NGVD															
Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Final Oct - Dec	Final Oct - Mar	Final Oct - May	Total Mar - Apr
1973	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1974	0	0	7	16	6	0	8	15	30	9	0	0	7	29	52	8
1975	0	0	0	0	0	0	0	4	1	0	0	0	0	0	4	0
1976	0	0	8	7	0	0	0	10	0	0	0	0	8	15	25	0
1977	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1978	0	0	9	0	0	0	0	0	0	0	0	0	9	9	9	0
1979	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1980	0	0	0	2	0	0	0	0	0	0	0	0	0	2	2	0
1981	0	0	6	1	2	0	0	0	20	0	0	0	6	9	9	0
1982	0	0	0	0	11	15	4	11	14	4	0	0	0	26	41	19
1983	0	0	0	5	10	23	5	4	4	0	0	0	0	38	47	28
1984	0	0	0	0	1	0	0	6	19	1	0	0	0	1	7	0
1985	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1986	0	0	0	0	6	10	0	0	2	0	0	0	0	16	16	10
1987	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1988	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1989	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1990	0	0	0	0	0	0	0	0	2	0	0	0	0	0	0	0
1993	0	0	0	0	0	0	0	8	0	0	0	0	0	0	8	0
1994	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1995	0	0	0	0	2	0	0	0	0	0	0	0	0	2	2	0
1996	0	2	14	8	23	3	10	16	20	0	0	0	16	50	76	13
1997	0	0	9	14	6	10	9	31	26	0	0	0	9	39	79	19
Average (1975-1997)	0	0.1	2.3	2.3	2.9	2.7	1.6	4.6	6.0	0.6	0	0	2.4	10.3	16.4	4.2

Table 2.4
Stage-Duration Analysis for Stage Below Threshold
Columbia River at Vancouver

	Number of Days Mean Daily Water Level at Vancouver (RM 106.5) is below +3 NGVD												
Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Jun - Sep
1973	0	0	0	0	0	0	0	0	0	0	1	3	4
1974	0	0	0	0	0	0	0	0	0	0	0	0	0
1975	0	0	0	0	0	0	0	0	0	0	0	0	0
1976	0	0	0	0	0	0	0	0	0	0	0	0	0
1977	0	0	0	0	0	0	0	0	0	0	0	0	0
1978	0	0	0	0	0	0	0	0	0	0	0	0	0
1979	0	0	0	0	0	0	0	0	0	0	0	0	0
1980	0	0	0	0	0	0	0	0	0	0	0	0	0
1981	0	0	0	0	0	0	0	0	0	0	0	0	0
1982	0	0	0	0	0	0	0	0	0	0	0	0	0
1983	0	0	0	0	0	0	0	0	0	0	0	0	0
1984	0	0	0	0	0	0	0	0	0	0	0	0	0
1985	0	0	0	0	0	0	0	0	0	0	0	0	0
1986	0	0	0	0	0	0	0	0	0	0	0	0	0
1987	0	0	0	0	0	0	0	0	0	0	0	0	0
1988	0	0	0	0	0	0	0	0	0	0	0	0	0
1989	0	0	0	0	0	0	0	0	0	0	0	0	0
1990	0	0	0	0	0	0	0	0	0	0	0	0	0
1993	0	0	0	0	0	0	0	0	0	0	0	0	0
1994	0	0	0	0	0	0	0	0	0	0	1	1	2
1995	0	0	0	0	0	0	0	0	0	0	0	0	0
1996	0	0	0	0	0	0	0	0	0	0	0	0	0
1997	0	0	0	0	0	0	0	0	0	0	0	0	0
Average (1973-1997)	0	0	0	0	0	0	0	0	0	0	0.1	0.2	0.3

Table 2.4 (continued)
Stage-Duration Analysis for Stage Below Threshold
Columbia River at Vancouver

	Number of Days Mean Daily Stage at Vancouver (RM 106.5) is below +4 NGVD												
Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Jun - Sep
1973	2	0	0	0	0	0	2	0	1	1	2	10	14
1974	7	0	0	0	0	0	0	0	0	0	0	1	1
1975	1	0	0	0	0	0	0	0	0	1	0	2	3
1976	4	0	0	0	0	0	0	0	0	0	0	0	0
1977	0	0	0	0	0	0	5	0	2	17	6	8	33
1978	5	0	0	0	0	0	0	0	0	0	0	0	0
1979	0	0	0	1	0	0	0	0	0	2	5	5	12
1980	5	0	0	0	0	0	0	0	0	0	4	2	6
1981	3	0	0	0	0	0	0	0	0	0	0	2	2
1982	1	1	0	0	0	0	0	0	0	0	3	2	5
1983	0	0	0	0	0	0	0	0	0	0	0	0	0
1984	4	0	0	0	0	0	0	0	0	0	1	1	2
1985	1	0	0	0	0	0	0	0	0	7	18	11	36
1986	1	0	0	0	0	0	0	0	0	0	3	2	5
1987	1	0	0	0	0	0	0	0	0	1	7	6	14
1988	2	3	0	0	0	0	0	0	0	6	14	10	30
1989	10	0	0	0	0	0	0	0	0	4	9	4	17
1990	4	2	0	0	0	0	0	0	0	0	0	4	4
1993	9	2	0	0	0	0	0	0	0	0	3	12	15
1994	10	8	3	0	0	0	0	0	0	1	14	25	40
1995	17	0	0	0	0	0	0	0	0	0	0	7	7
1996	0	0	0	0	0	0	0	0	0	0	0	2	2
1997	2	2	0	0	0	0	0	0	0	0	0	1	1
Average (1973-1997)	3.9	0.8	0.1	0	0	0	0.3	0	0.1	1.7	3.9	5.1	10.8

Table 2.4 (continued)
Stage-Duration Analysis for Stage Below Threshold
Columbia River at Vancouver

Water Year	Number of Days Mean Daily Stage at Vancouver (RM 106.5) is below +5 NGVD												Total
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Jun - Sep
1973	16	2	0	0	0	2	13	3	7	14	22	29	72
1974	22	2	0	0	0	0	0	0	0	0	4	15	19
1975	17	2	0	0	0	0	0	0	0	5	15	21	41
1976	11	0	0	0	0	0	0	0	0	0	0	3	3
1977	5	7	5	2	10	6	24	2	14	30	29	16	89
1978	24	3	0	0	0	0	0	0	0	3	13	9	25
1979	13	10	2	5	1	0	0	0	1	16	25	23	65
1980	14	4	0	0	0	3	0	0	0	4	19	25	48
1981	23	8	0	0	0	0	0	0	0	0	1	20	21
1982	10	9	0	0	0	0	0	0	0	0	6	11	17
1983	10	0	0	0	0	0	0	0	0	0	1	10	11
1984	14	0	0	0	0	0	0	0	0	1	6	13	20
1985	9	0	0	0	0	0	0	0	0	22	29	21	72
1986	11	3	3	0	0	0	0	0	0	10	10	16	36
1987	17	1	0	0	0	0	3	0	6	23	24	23	76
1988	20	18	0	0	5	5	0	0	2	23	28	26	79
1989	23	5	0	1	1	0	0	0	3	22	29	23	77
1990	18	8	5	1	0	0	0	0	0	4	5	24	33
1993	26	17	0	1	6	4	0	0	0	3	21	30	54
1994	30	28	15	2	2	2	5	0	1	9	31	30	71
1995	29	4	0	0	0	0	0	0	0	0	5	25	30
1996	9	4	0	0	0	0	0	0	0	0	0	21	21
1997	10	8	0	0	0	0	0	0	0	0	0	5	5
Average (1973-1997)	16.6	6.2	1.3	0.5	1.1	1.0	2.0	0.2	1.5	8.2	14.0	19.1	42.8

Table 2.4 (continued)
Stage-Duration Analysis for Stage Below Threshold
Columbia River at Vancouver

	Number of Days Mean Daily Stage at Vancouver (RM 106.5) is below +6 NGVD												
Water Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total Jun - Sep
1973	29	22	3	1	16	9	22	21	26	30	30	30	116
1974	31	8	0	0	0	0	0	0	0	0	13	27	40
1975	29	18	3	0	0	0	0	0	0	7	29	30	66
1976	23	0	0	0	0	0	0	0	0	0	0	10	10
1977	24	24	26	18	25	21	30	21	25	31	31	30	117
1978	30	9	0	0	0	5	0	0	0	6	30	16	52
1979	29	22	7	12	2	0	1	0	14	29	31	30	104
1980	25	13	0	0	3	10	9	0	0	18	31	30	79
1981	31	15	0	0	0	7	2	0	0	0	19	30	49
1982	24	11	1	0	0	0	0	0	0	0	18	27	45
1983	26	3	0	0	0	0	0	0	0	0	14	26	40
1984	31	2	0	0	0	0	0	0	0	7	25	30	62
1985	21	1	0	0	0	7	0	0	7	27	31	30	95
1986	27	7	12	2	0	0	0	0	3	21	31	29	84
1987	29	6	5	0	0	5	16	0	18	31	31	30	110
1988	30	28	4	2	18	21	10	0	13	31	31	30	105
1989	30	7	3	4	10	3	0	0	7	30	31	30	98
1990	31	18	19	4	0	0	0	0	0	9	29	30	68
1993	31	25	6	8	17	14	0	0	1	14	30	30	75
1994	31	30	23	14	18	13	12	0	4	27	31	30	92
1995	31	18	2	0	0	0	7	0	0	5	27	30	62
1996	25	7	0	0	0	0	0	0	0	0	11	30	41
1997	25	16	0	0	0	0	0	0	0	0	1	19	20
Average (1973-1997)	28.0	13.5	5.0	2.8	4.7	5.0	4.7	1.8	5.1	14.0	24.1	27.6	70.9

Table 2.4 (continued)
Stage-Duration Analysis for Stage Below Threshold
Columbia River at Vancouver

Water Year	Number of Days Mean Daily Stage at Vancouver (RM 106.5) is below +8 NGVD												Total
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Jun - Sep
1973	31	30	16	19	28	31	30	31	30	31	31	30	122
1974	31	10	0	1	0	0	0	0	0	5	31	30	66
1975	31	30	19	4	7	0	15	1	0	20	31	30	81
1976	31	22	1	0	6	5	0	0	0	8	11	29	48
1977	31	30	31	31	28	31	30	31	30	31	31	30	122
1978	31	24	3	6	16	24	3	0	8	31	31	30	100
1979	31	30	19	30	8	19	22	4	29	31	31	30	121
1980	31	30	22	13	24	23	26	3	1	31	31	30	93
1981	31	30	11	10	14	31	30	10	0	18	31	30	79
1982	31	29	2	4	0	0	0	0	0	8	31	30	69
1983	30	27	2	2	1	0	2	0	5	18	31	30	84
1984	31	16	5	2	5	7	0	0	0	21	31	30	82
1985	31	6	14	22	13	27	10	3	17	31	31	30	109
1986	31	25	24	20	2	0	2	0	11	31	31	30	103
1987	31	21	24	20	22	22	30	14	30	31	31	30	122
1988	31	30	22	19	29	31	28	19	22	31	31	30	114
1989	31	21	26	14	27	8	6	1	19	31	31	30	111
1990	31	30	30	18	3	14	19	12	0	28	31	30	89
1993	31	29	28	31	28	20	20	1	14	31	31	30	106
1994	31	30	31	31	25	31	30	30	30	31	31	30	122
1995	31	29	16	18	5	14	21	0	0	27	31	30	88
1996	31	12	0	0	0	0	0	0	0	13	31	30	74
1997	31	18	0	0	0	0	1	0	0	3	31	30	64
Average (1973-1997)	31.0	24.3	15.0	13.7	12.7	14.7	14.1	7.0	10.7	23.5	30.1	30.0	94.3

Table 2.4 (continued)
Stage-Duration Analysis for Stage Below Threshold
Columbia River at Vancouver

Water Year	Number of Days Mean Daily Stage at Vancouver (RM 106.5) is below +10 NGVD												Total
	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Jun - Sep
1973	31	30	22	25	28	31	30	31	30	31	31	30	122
1974	31	23	9	10	5	3	0	0	0	18	31	30	79
1975	31	30	27	16	24	15	25	7	0	27	31	30	88
1976	31	30	13	9	20	20	10	0	6	30	31	30	97
1977	31	30	31	31	28	31	30	31	30	31	31	30	122
1978	31	26	9	24	25	31	28	18	27	31	31	30	119
1979	31	30	31	31	20	29	30	21	30	31	31	30	122
1980	31	30	31	22	29	31	30	25	5	31	31	30	97
1981	31	30	19	24	20	31	30	25	1	28	31	30	90
1982	31	30	7	19	14	0	3	2	0	18	31	30	79
1983	31	30	18	15	9	0	11	6	15	31	31	30	107
1984	31	22	20	25	16	14	7	0	0	27	31	30	88
1985	31	29	28	31	28	31	27	29	28	31	31	30	120
1986	31	30	31	31	17	5	19	24	20	31	31	30	112
1987	31	28	31	31	24	31	30	29	30	31	31	30	122
1988	31	30	29	25	29	31	30	31	29	31	31	30	121
1989	31	27	31	26	28	29	30	23	30	31	31	30	122
1990	31	30	31	24	22	31	25	29	6	31	31	30	98
1993	31	30	31	31	28	25	30	5	19	31	31	30	111
1994	31	30	31	31	28	31	30	31	30	31	31	30	122
1995	31	30	29	23	17	30	30	21	23	31	31	30	115
1996	31	25	6	0	1	4	8	4	3	30	31	30	94
1997	31	23	9	0	0	1	7	0	0	20	31	30	81
Average (1973-1997)	31	28.4	22.8	21.9	20.0	21.1	21.7	17.0	15.7	28.8	31	30	105.6

Table 3.1
Generalized HSPF Model Parameters for Saturated Soils

Land Segment	Model Parameter															
	LZSN (in.)	INFILT (in./hr)	LSUR (ft.)	SLSUR	KVARY (1/in.)	AGWRC (1/day)	INFEXP	INFILD	BASETP	AGWETP	CEPSC (in.)	UZSN (in.)	NSUR	INTFW	IRC (1/day)	LZETP
SA	4.0	2.0	100	0.001	0.5	0.996	10.0	2.0	0.0	0.7	0.1	3.0	0.5	1.0	0.7	0.8

Units are printed below parameter name; where units are not listed, the parameter has no units.

Land-segment definitions:

SA = saturated soils

Parameters:

LZSN = lower-zone nominal storage
INFILT = infiltration capacity
LSUR = average length of the overland flow plane
SLSUR = average slope of the overland flow plane
KVARY = groundwater outflow modifier
AGWRC = groundwater recession parameter
INFEXP = infiltration equation exponent
INFILD = ratio of the maximum to mean infiltration rate of a pervious area
BASETP = fraction of available-PET demand that can be met with groundwater outflow
AGWETP = fraction of available-PET demand that can be met with stored groundwater
CEPSC = interception storage capacity of plants
UZSN = upper-zone nominal storage
NSUR = average roughness of the overland flow plane
INTFW = interflow index
IRC = interflow recession parameter;
LZETP = lower-zone ET

Units:

in. = inches
in./hr = inches per hour
ft = feet
1/in. = 1/inch

Table 4.1
Comparison of Simulated Peak Water Surface Elevations (ft NGVD)
Water Year 1995

	Existing Condition	Natural Condition	Gravity Supply Alternative
Vancouver Gage (RM 106.5)	13.8 (Feb 1, 9 PM)	13.8 (Feb 1, 9 PM)	13.8 (Feb 1, 9 PM)
Langsdorf Landing (RM 97.5)	13.0 (Feb 1, 8 PM)	13.0 (Feb 1, 8 PM)	13.0 (Feb 1, 8 PM)
Lake River near north end of Shillapoo Lake	11.7 (Feb 2, 9 AM)	11.6 (Feb 2, 8 PM)	11.7 (Feb 2, 9 AM)
Vancouver Lake	11.8 (Feb 2, 12 PM)	11.7 (Feb 2, 10 PM)	11.8 (Feb 2, 12 PM)
Shillapoo Lake	5.5 (June 1, 12 AM)	7.0 (Mar 5, 12 PM)	9.8 (Feb 23, 5 PM)

Table 4.2
Comparison of Simulated Peak Water Surface Elevations (ft NGVD)
1996 (February - August)

	Existing Condition	**Natural Condition**	**Gravity Supply Alternative**
Vancouver Gage (RM 106.5)	27.1 (Feb 9, 5 PM)	27.1 (Feb 9, 5 PM)	27.1 (Feb 9, 5 PM)
Langsdorf Landing (RM 97.5)	26.1 (Feb 9, 5 PM)	26.1 (Feb 9, 5 PM)	26.1 (Feb 9, 6 PM)
Lake River near north end of Shillapoo Lake	24.3 (Feb 9, 8 PM)	24.3 (Feb 10, 12 AM)	24.5 (Feb 9, 9 PM)
Vancouver Lake	24.3 (Feb 9, 8 PM)	24.4 (Feb 10, 1 AM)	24.5 (Feb 9, 9 PM)
Shillapoo Lake	21.8 (Feb 11, 7 AM)	24.5 (Feb 10, 1 AM)	24.9 (Feb 9, 9 PM)

Table 4.3
Comparison of Simulated Peak Water Surface Elevations (ft NGVD)
1997 (April - August)

	Existing Condition	**Natural Condition**	**Gravity Supply Alternative**
Vancouver Gage (RM 106.5)	20.6 (June 16, 8 AM)	20.6 (June 16, 8 AM)	20.6 (June 16, 8 AM)
Langsdorf Landing (RM 97.5)	19.3 (June 5, 7 AM)	19.3 (June 5, 7 AM)	19.3 (June 5, 7 AM)
Lake River near north end of Shillapoo Lake	17.5 (June 5, 8 AM)	17.5 (June 5, 8 AM)	17.5 (June 5, 8 AM)
Vancouver Lake	17.6 (June 5, 9 AM)	17.6 (June 5, 9 AM)	17.6 (June 5, 9 AM)
Shillapoo Lake	7.0 (June 1, 12 AM)	17.9 (June 5, 12 PM)	19.0 (June 17, 9 AM)

Table 4.4
Comparison of Simulated Peak Water Surface Elevations (ft NGVD)
Low Water Year (1999)

	Existing Condition	Natural Condition	Gravity Supply Alternative
Vancouver Gage (RM 106.5)	N/A	N/A	9.6 (Jan 4, 6 PM)
Langsdorf Landing (RM 97.5)	N/A	N/A	9.2 (Jan 4, 5 PM)
Lake River near north end of Shillapoo Lake	N/A	N/A	8.2 (Jan 1, 3 PM)
Vancouver Lake	N/A	N/A	8.2 (Jan 1, 6 PM)
Shillapoo Lake	N/A	N/A	8.3 (Apr 12, 7 PM)

Table 5.1
Wetland Cell Elevations, Surface Area, Volume Data

	CELL 1		CELL 2		CELL 3		CELL 4		CELL 5		CELL 6		CELL 7		Total Volume CELLS 1-7	CELL 8	
Elevation (ft NGVD)	Surface Area (acres)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)	Volume (acre ft)	Surface Area (acres)	Volume (acre ft)
0	0.1	0.0	2.2	0.7	1.5	0.5	1.5	0.5	0.5	0.2	1.2	0.4	1.2	0.4	2.7		
1	0.4	0.2	8.8	5.9	1.7	2.1	2.1	2.3	0.6	0.8	2.0	1.9	2.4	2.2	15.4		
2	36.5	13.8	52.1	33.3	38.6	18.2	2.8	4.8	32.1	13.2	12.9	8.6	5.9	6.2	98.1		
3	85.4	73.1	79.7	98.7	77.0	74.9	56.1	28.6	77.5	66.3	51.9	38.8	50.7	30.9	411.2		
4	112.9	171.9	102.4	189.5	95.1	160.8	67.1	90.0	85.3	147.7	96.3	111.7	96.8	103.4	975.0		
5	143.3	299.7	122.2	301.7	102.1	259.3	73.1	160.1	87.8	234.2	116.8	218.1	115.7	209.5	1682.6		
6 E	239.6	616.8	197.1	588.4	157.7	495.7	116.2	319.7	131.7	439.1	193.1	461.5	187.7	443.8	3365.1	0.0	0.0
7	192.5	634.3	149.9	573.3	111.4	472.7	86.3	319.3	87.8	409.8	152.8	486.8	144.0	468.7	3364.9	7.0	2.3
8 E	199.6	837.4	158.6	736.1	112.1	585.2	89.3	410.0	87.8	497.6	166.1	659.2	153.8	627.2	4352.6	25.9	17.8
9 E	206.6	1036.4	167.2	895.7	112.8	695.4	92.2	498.9	87.8	583.6	179.1	828.1	163.5	782.5	5320.6	62.9	60.9
10	213.9	1243.5	176.1	1061.7	113.6	810.1	95.2	591.4	87.8	673.1	192.7	1003.9	173.5	944.2	6328.1	106.6	144.7
11																131.6	263.6
12																151.5	405.0
13																170.7	566.0
14 E																182.8	748.7
15																194.8	931.3
Total Trib. Area	302.5		267.0		130.2		112.9		87.9		273.8		241.7			241.6	

E = estimated by interpolation

Table 5.2
Control Structure Type A
Dimensions

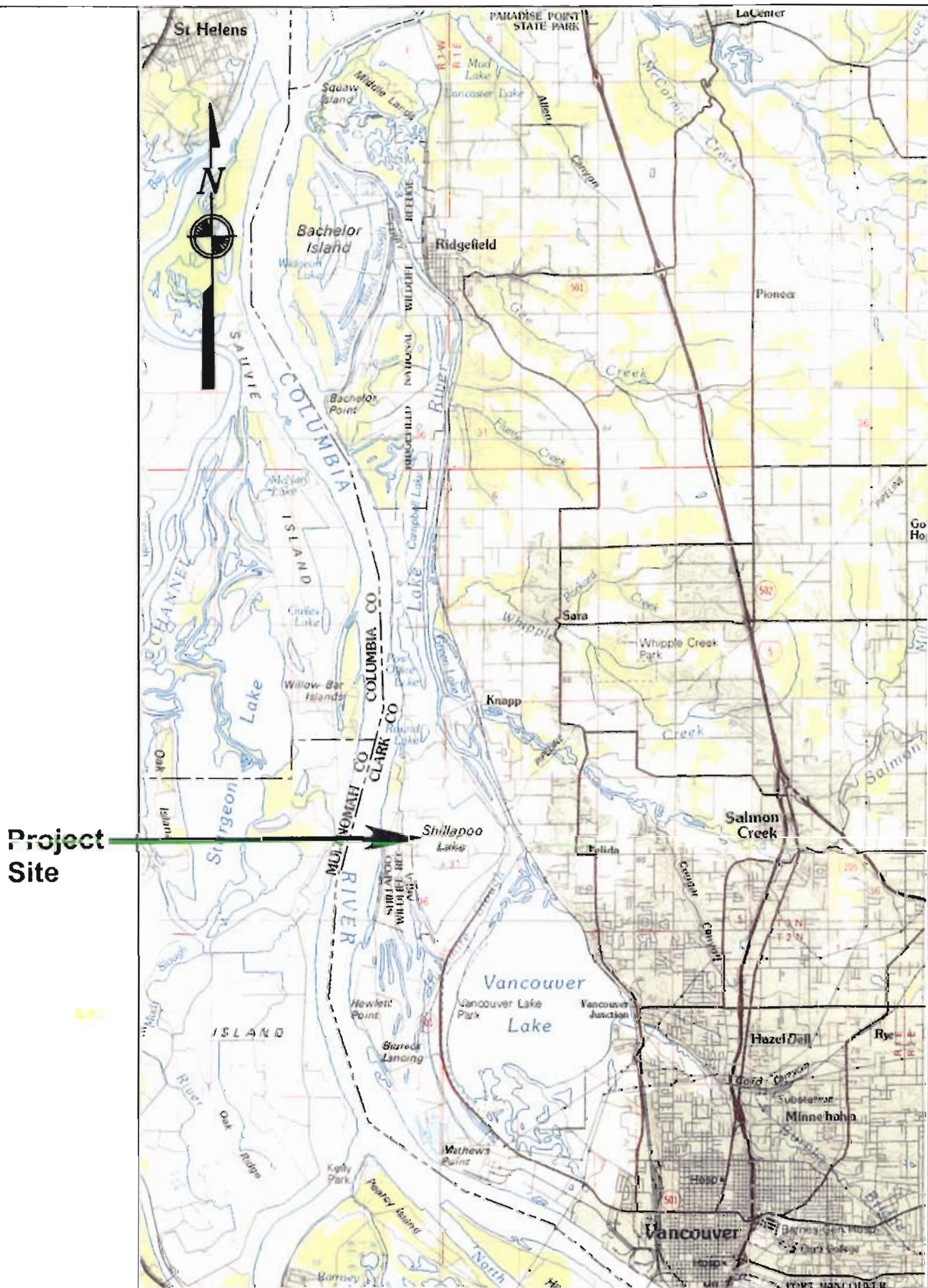
Control Structure Number	Culvert Diameter (ft)	Culvert Invert Elevation (ft)
A-1	2.0	-3.0
A-2	2.0	-3.0
A-3	2.5	-4.0
A-4	2.5	-4.0
A-5	2.0	-3.0
A-6	2.0	-3.0
A-7	2.0	-3.0
A-8	2.0	-3.0
A-9	2.0	5.0
A-10	2.0	6.0

Table 5.3
Control Structure Type B and C Dimensions

Control Structure Number	Culvert		Riser		
	Diameter (ft)	Invert Elevation (ft)	Fixed Crest Elevation	Maximum Crest Elevation	Rim Elevation
B-1 to B-8	2.0	2.0	3.0	5.0	7.0
B-9 to B-10	2.0	8.0	9.0	11.0	13.0
C-1 to C-8	2.0	5.0	6.0	8.0	10.0
C-9 to C-10	2.0	10.0	11.0	13.0	15.0

Table 5.4
Type and Number of Control Structures

Water Control Structure	Description	Number of Units
Type A	2 to 2.5 ft dia culvert with slide gate	10
Type B	2 ft dia culvert with slide gate and stop logs	10
Type C	2 ft dia culvert with stop logs	10
Type D	emergency overflow spillway	8
Type E	3.5 ft dia culvert with slide gate and flap gate	1
Type F	fish screen	1
Type G	3 ft dia culvert with slide gate and flap gate	1
Type H	adjustable overflow gate	1
Type I	adjustable overflow gate	2
Type J	1.5 ft dia culvert with slide gate (turn out structu	2



SHILLAPOO LAKE RESTORATION

Location Map

Scale:

1 0 1 2 3 miles

5 000 0 10 000 ft.

Source: USGS 1:100 000

Topographic Map Vancouver, WA–OR.

File:	Date:
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 1.1

Shillapoo Lake - Elevation vs Area and Volume (Elevation 0 to 10 ft NGVD)

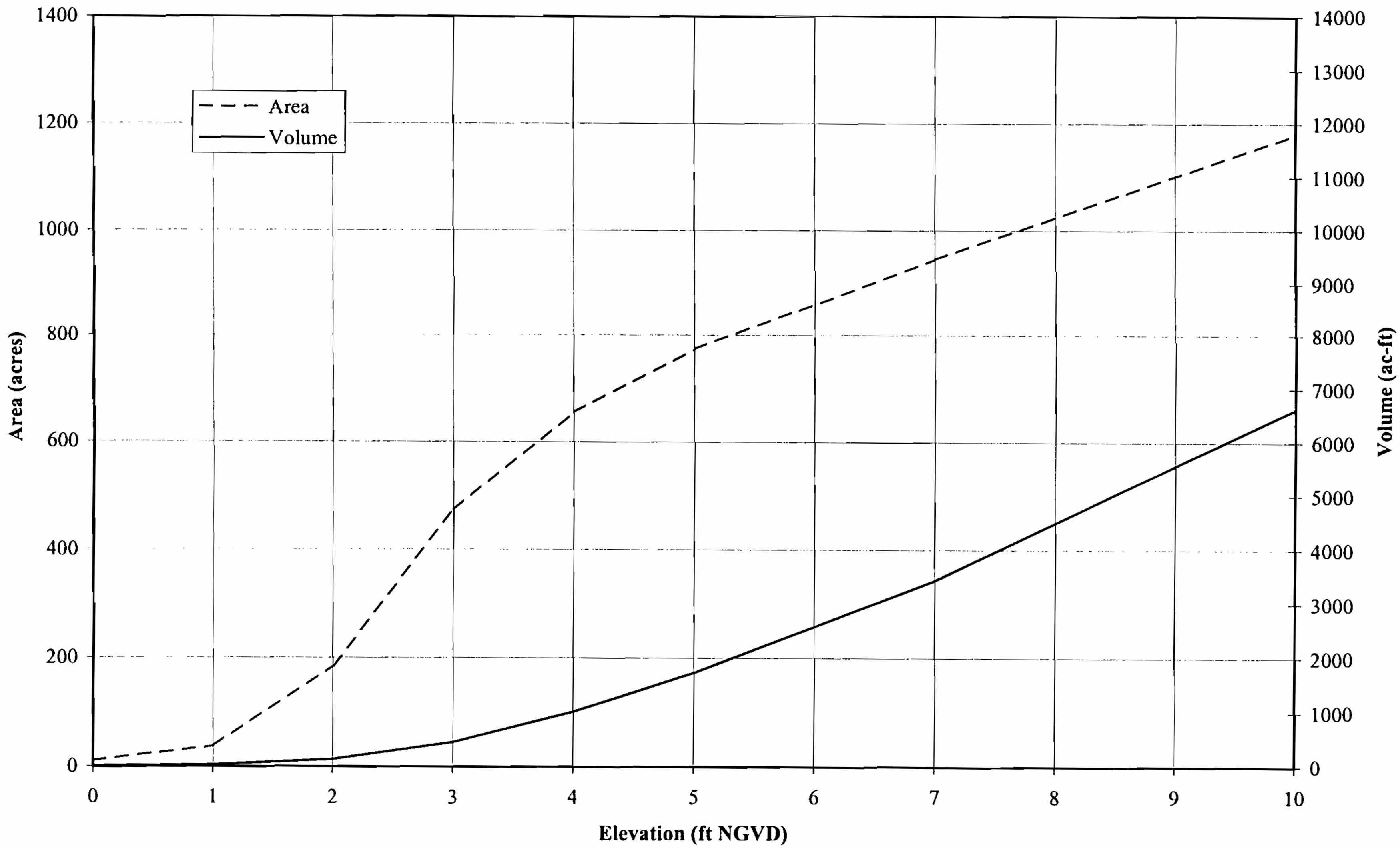


Figure 2.1

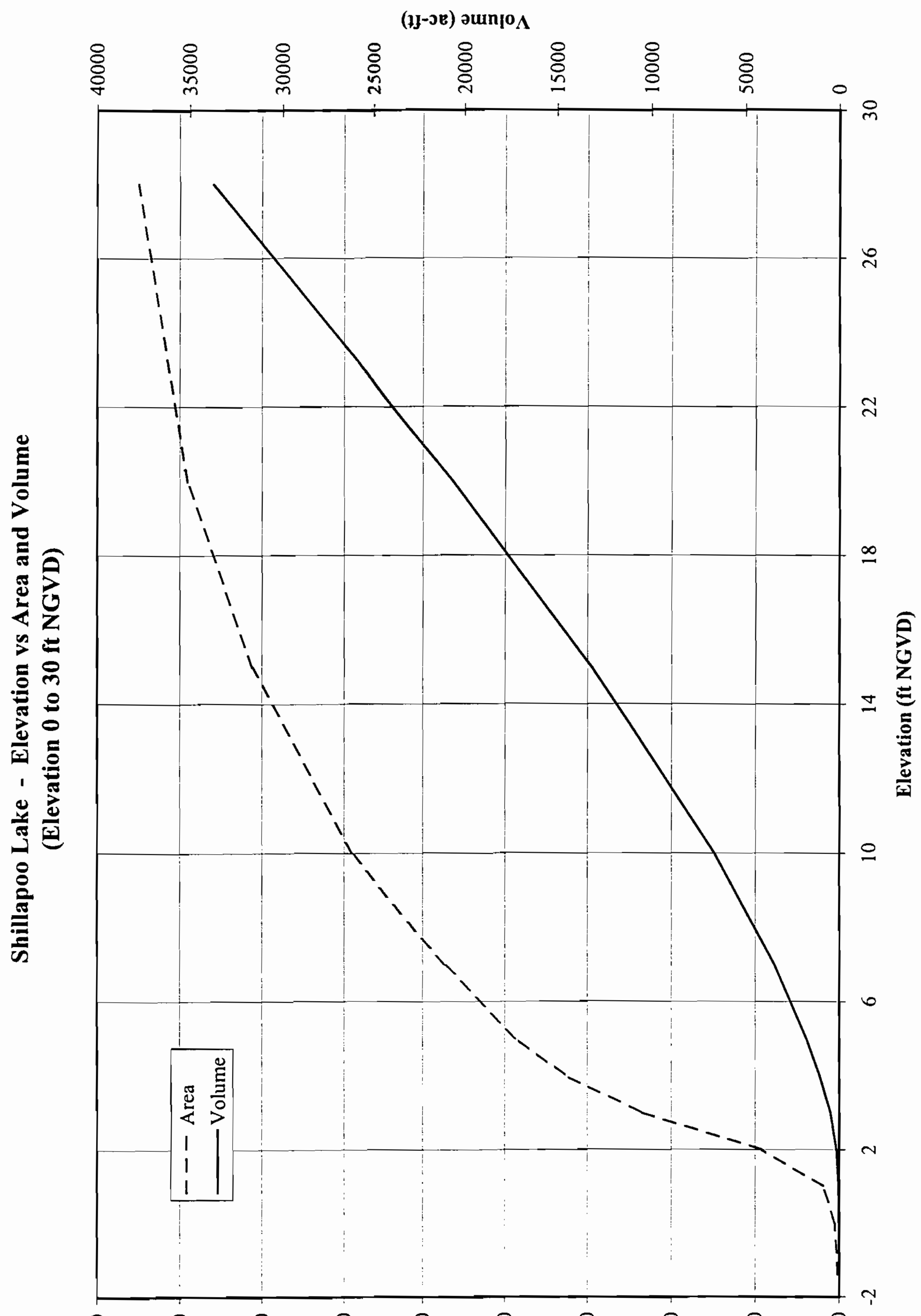
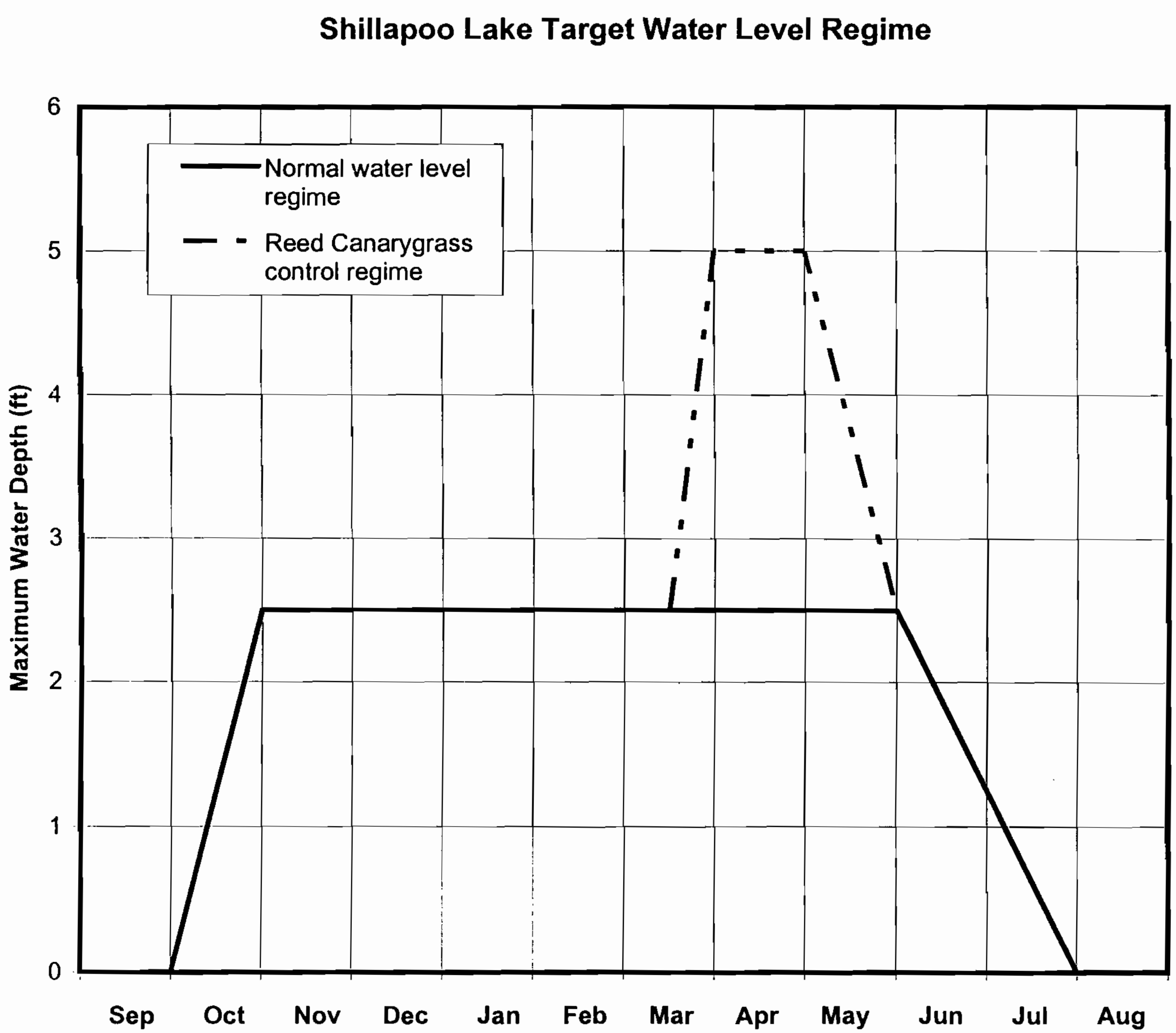

Shillapoo Lake - Elevation vs Area and Volume
(Elevation 0 to 30 ft NGVD)
Area
Volume
Area (acres)
Volume (ac-ft)
Elevation (ft NGVD)
0
200
400
600
800
1000
1200
1400
1600
1800
0
5000
10000
15000
20000
25000
30000
35000
40000
-2
2
6
10
14
18
22
26
30

Figure 2.2



Shillapoo Lake Target Water Level Regime
Normal water level regime
Reed Canarygrass control regime
Maximum Water Depth (ft)
0
1
2
3
4
5
6
Sep
Oct
Nov
Dec
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug

Figure 2.3

Stage Duration Frequency Analysis
Columbia River at Vancouver, WA
October thru December (92 days)
Water Years 1973-1990, 1993-1997 (23 years)

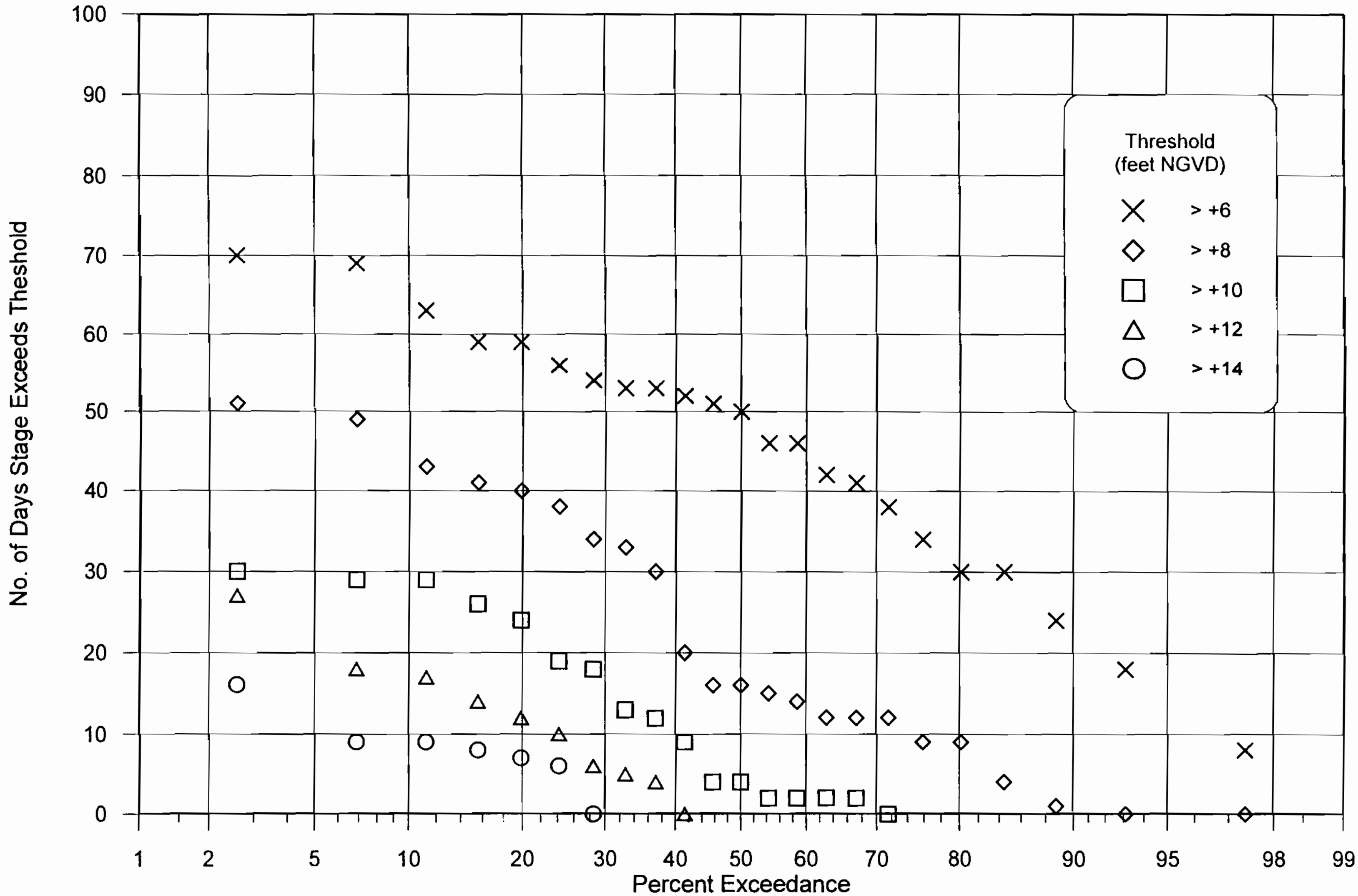


Figure 2.4

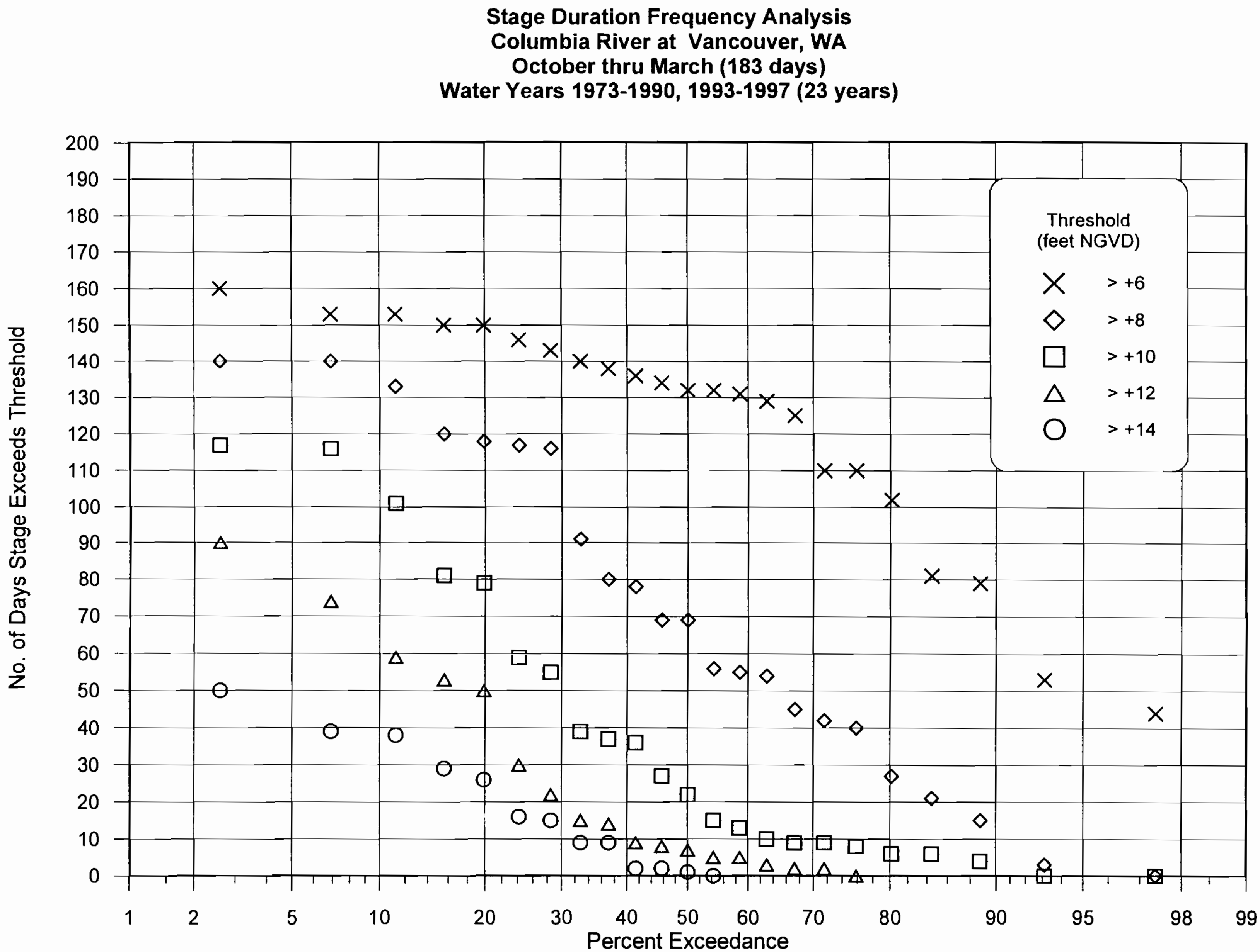

Stage Duration Frequency Analysis
Columbia River at Vancouver, WA
October thru March (183 days)
Water Years 1973-1990, 1993-1997 (23 years)
No. of Days Stage Exceeds Threshold
Percent Exceedance
Threshold
(feet NGVD)
> +6
> +8
> +10
> +12
> +14
200
190
180
170
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
0
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99

Figure 2.5

Stage Duration Frequency Analysis
Columbia River at Vancouver, WA
October thru May (244 days)
Water Years 1973-1990, 1993-1997 (23 years)

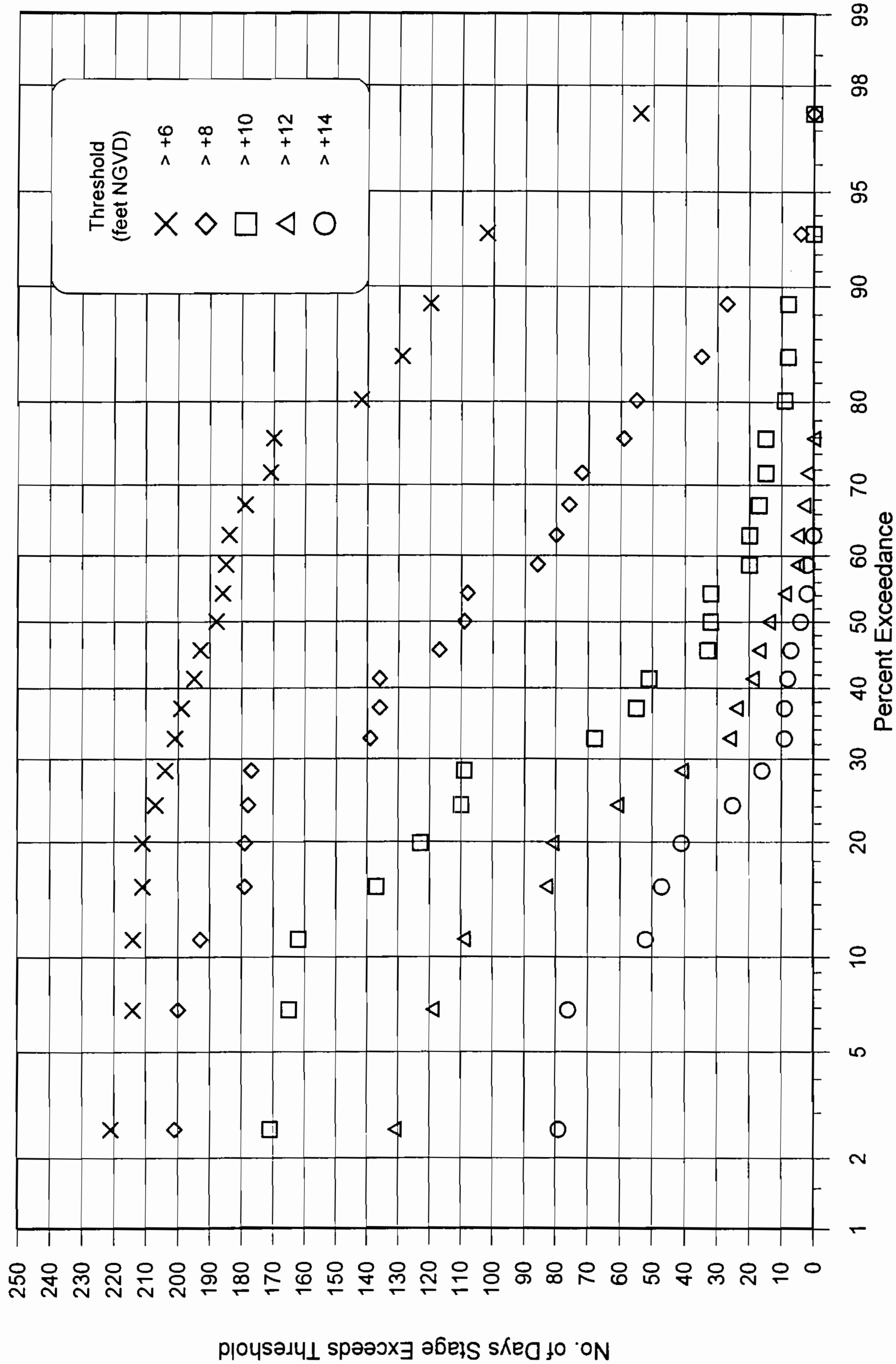


Figure 2.6

Stage Duration Frequency Analysis
Columbia River at Vancouver, WA
March and April (61 days)
Water Years 1973-1990, 1993-1997 (23 years)

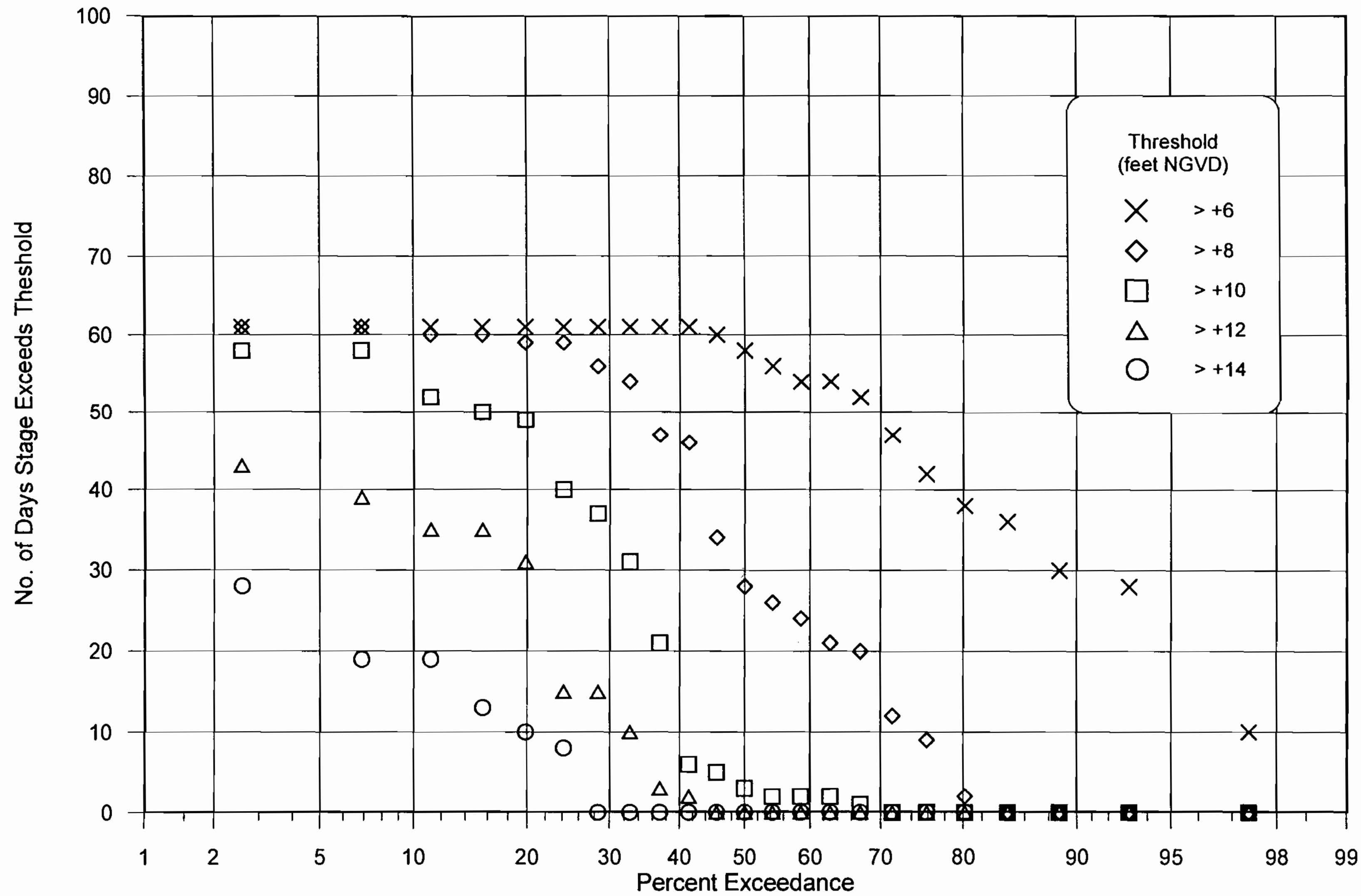
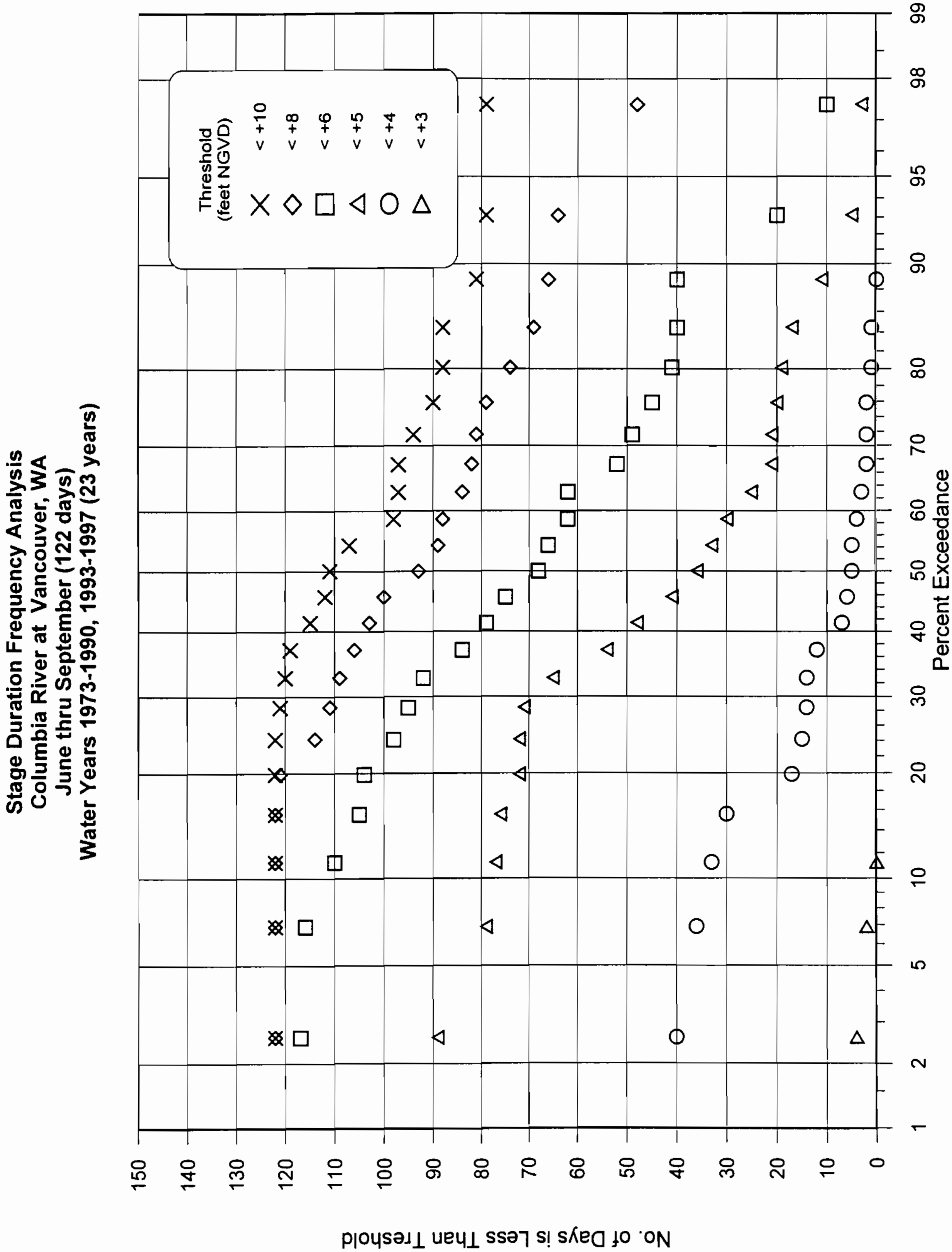
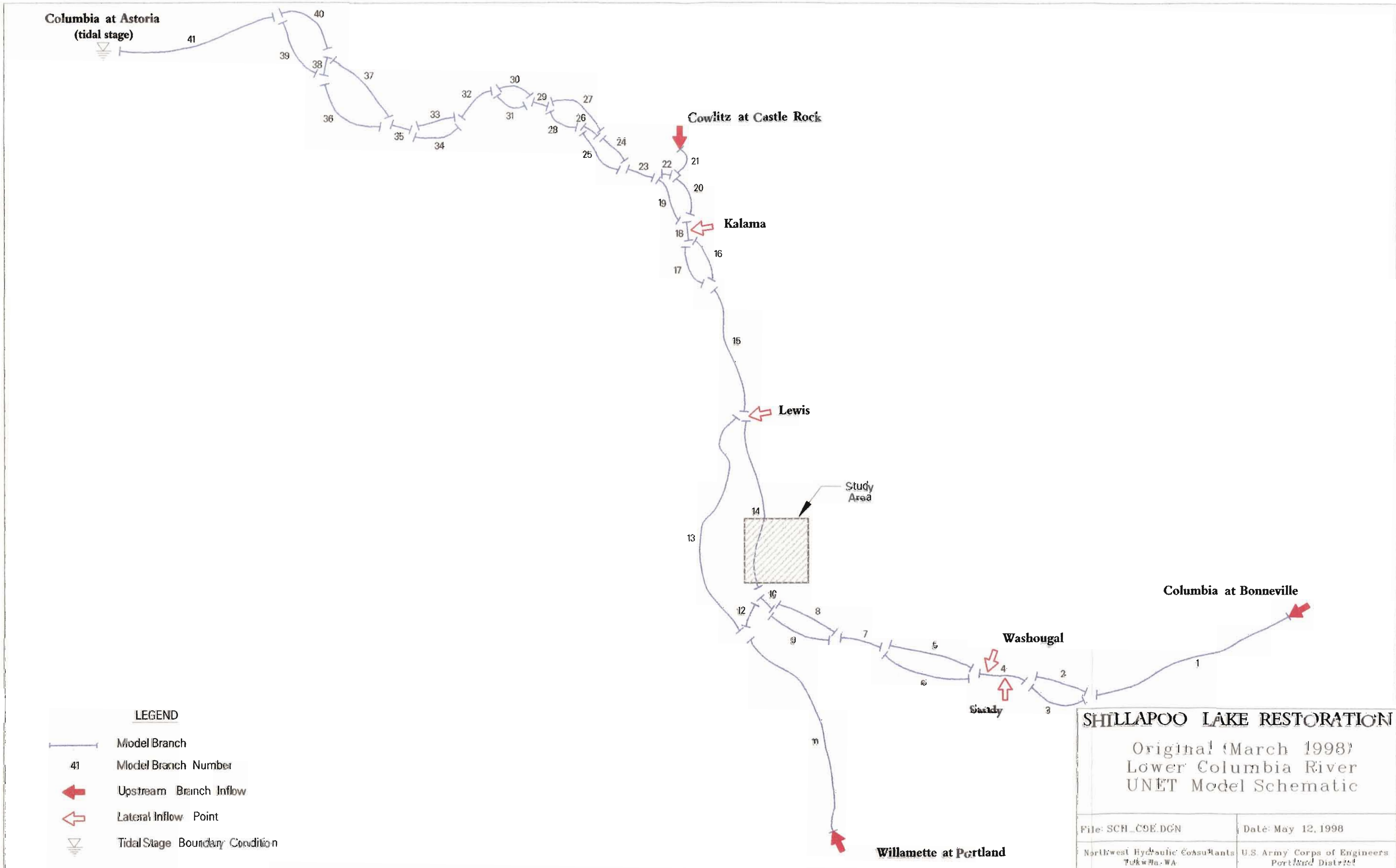


Figure 2.7



Stage Duration Frequency Analysis
Columbia River at Vancouver, WA
June thru September (122 days)
Water Years 1973-1990, 1993-1997 (23 years)
Threshold
(feet NGVD)
< +10
< +8
< +6
< +5
< +4
< +3
No. of Days is Less Than Treshold
Percent Exceedance
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
1
2
5
10
20
30
40
50
60
70
80
90
95
98
99

Figure 2.8



Columbia at Astoria
(tidal stage)
Cowlitz at Castle Rock
Kalama
Lewis
Study Area
Columbia at Bonneville
Washougal
Sandy
Willamette at Portland
LEGEND
Model Branch
Model Branch Number
Upstream Branch Inflow
Lateral Inflow Point
Tidal Stage Boundary Condition
SHILLAPOO LAKE RESTORATION
Original (March 1998)
Lower Columbia River
UNET Model Schematic
File: SCH_COE.DGN
Date: May 12, 1998
Northwest Hydraulic Consultants
Tukwila, WA
U.S. Army Corps of Engineers
Portland District

Figure 3.1

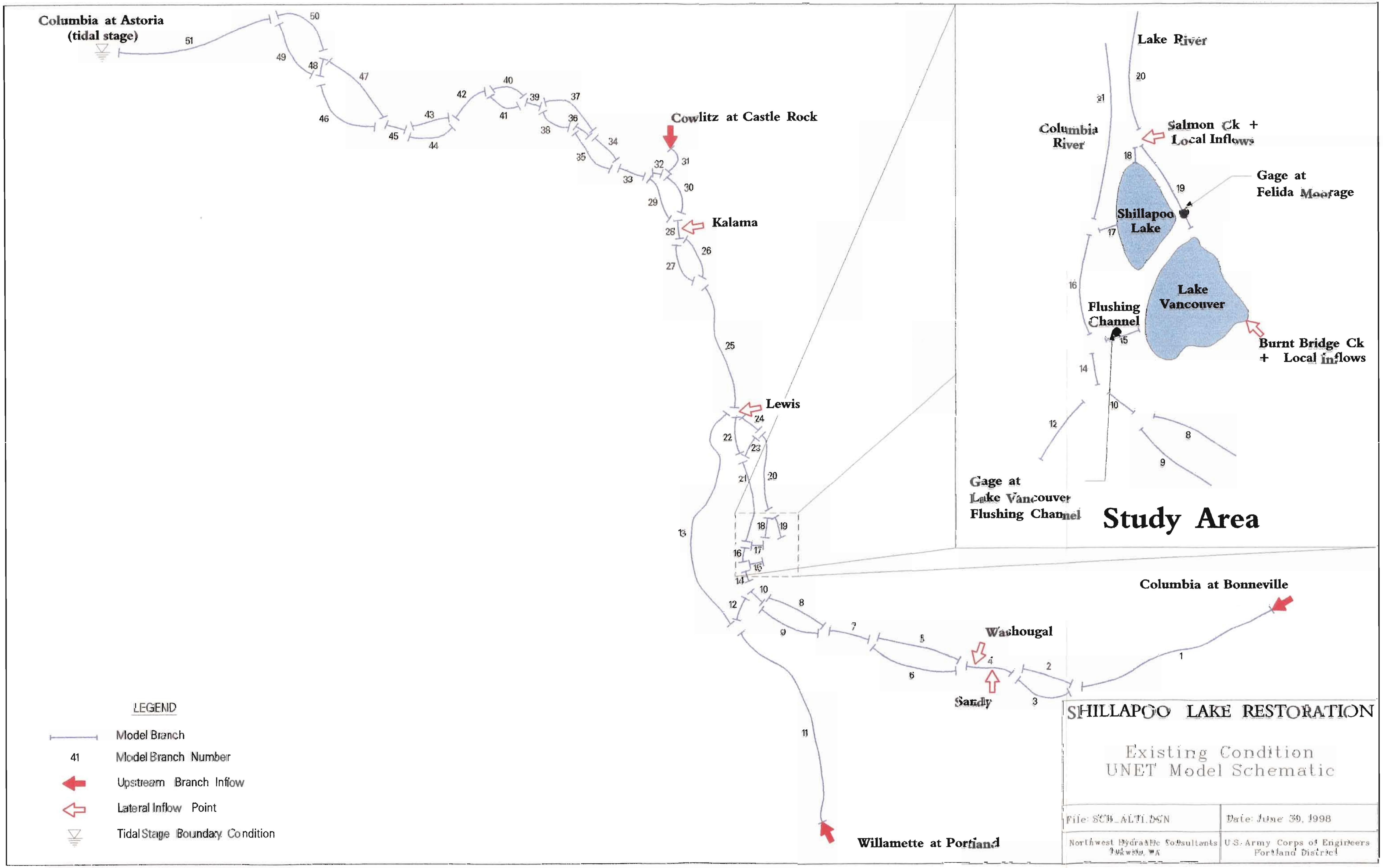

Columbia at Astoria (tidal stage)
Cowlitz at Castle Rock
Kalama
Lewis
Washougal
Sandy
Columbia at Bonneville
Willamette at Portland
Lake River
Columbia River
Salmon Ck + Local Inflows
Gage at Felida Moorage
Shillapoo Lake
Lake Vancouver
Flushing Channel
Burnt Bridge Ck + Local inflows
Gage at Lake Vancouver Flushing Channel
Study Area
LEGEND
Model Branch
41 Model Branch Number
Upstream Branch Inflow
Lateral Inflow Point
Tidal Stage Boundary Condition
SHILLAPOO LAKE RESTORATION
Existing Condition
UNET Model Schematic
File: SCH_ALTI.DGN
Date: June 30, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District

Figure 3.2

Approximate Rating Curve for 42" Culvert
at the Shillapoo Lake Expulsion Pump

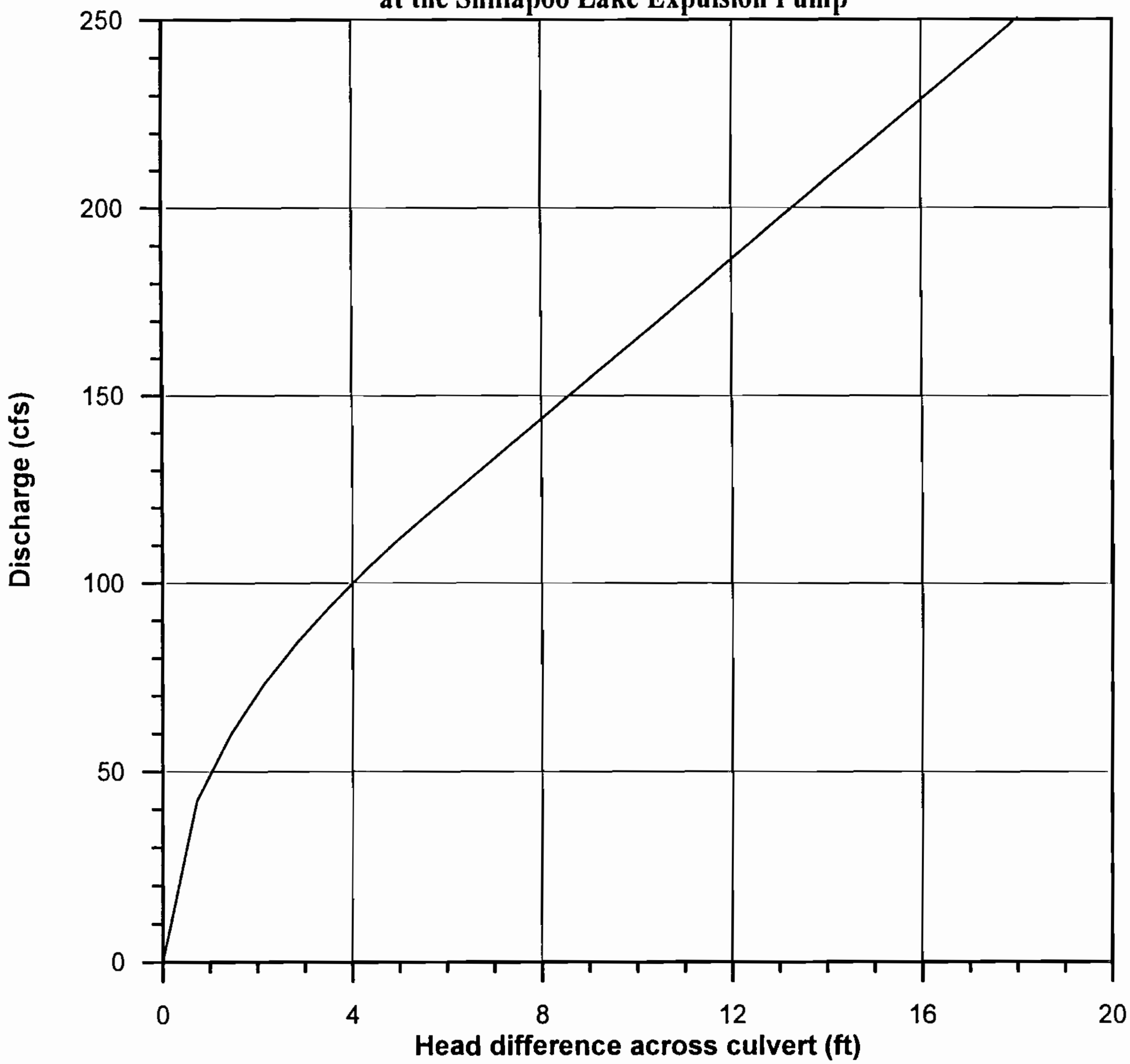


Figure 3.3

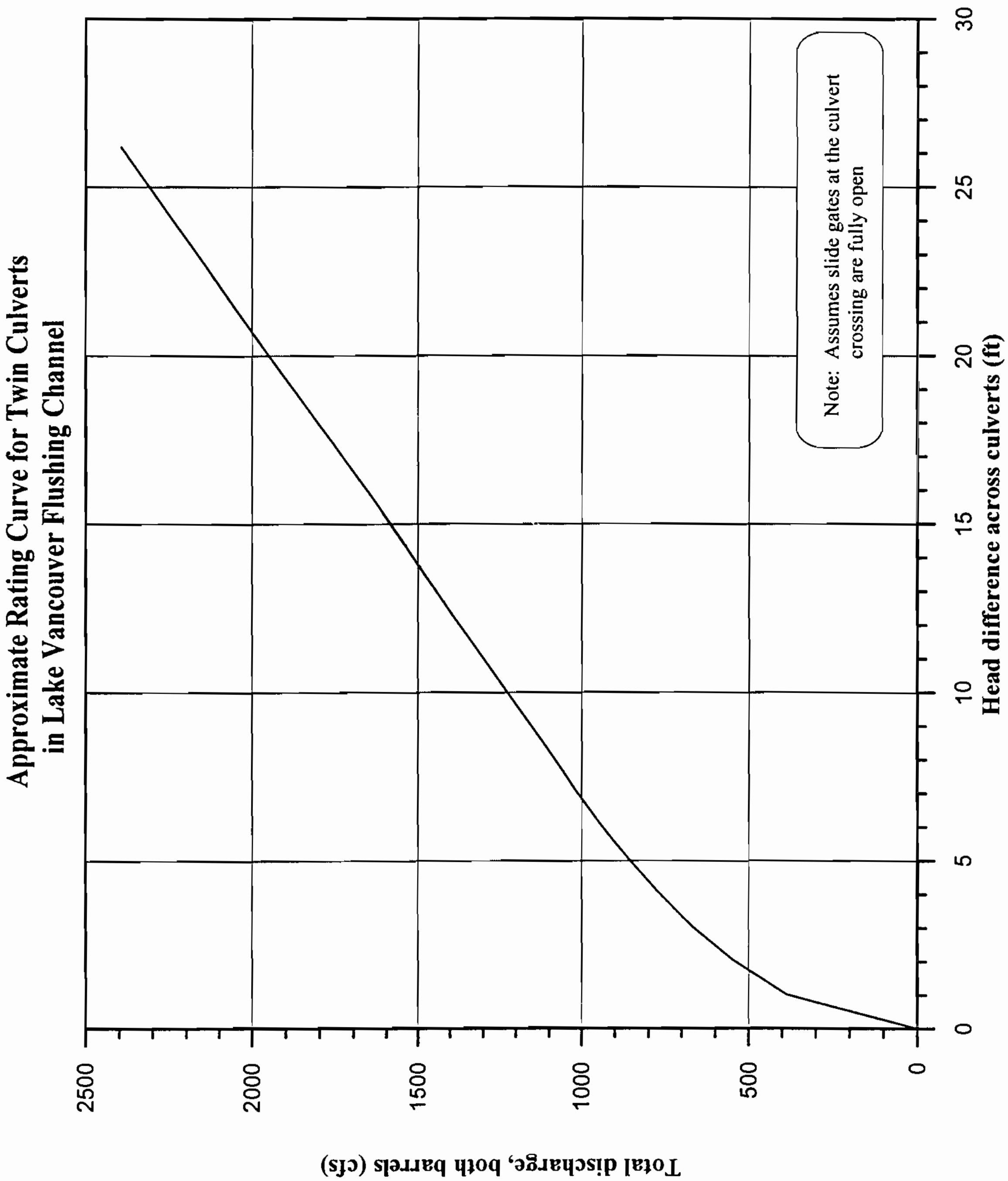

Approximate Rating Curve for Twin Culverts
in Lake Vancouver Flushing Channel
Total discharge, both barrels (cfs)
0
500
1000
1500
2000
2500
Head difference across culverts (ft)
0
5
10
15
20
25
30
Note: Assumes slide gates at the culvert crossing are fully open

Figure 3.4

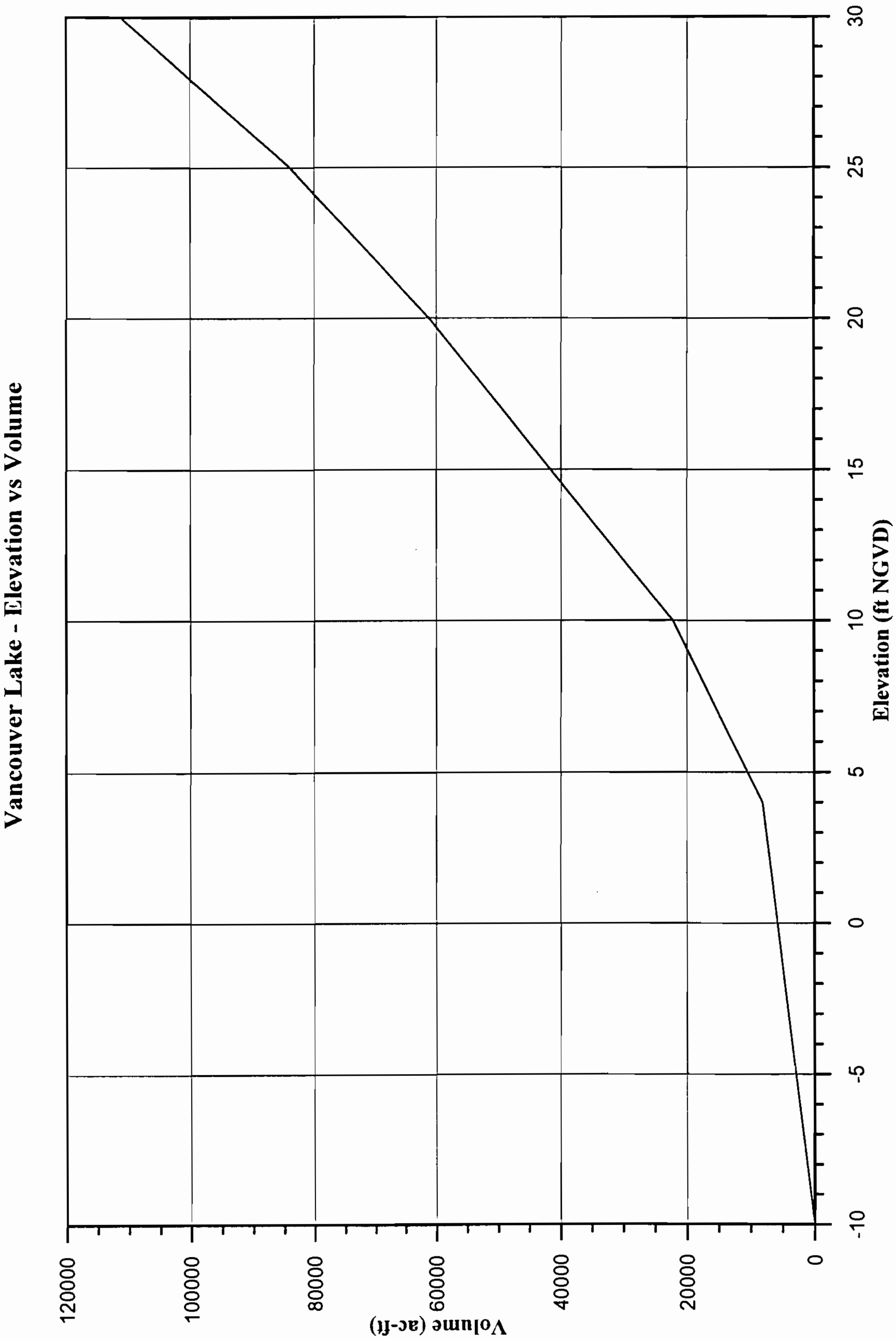
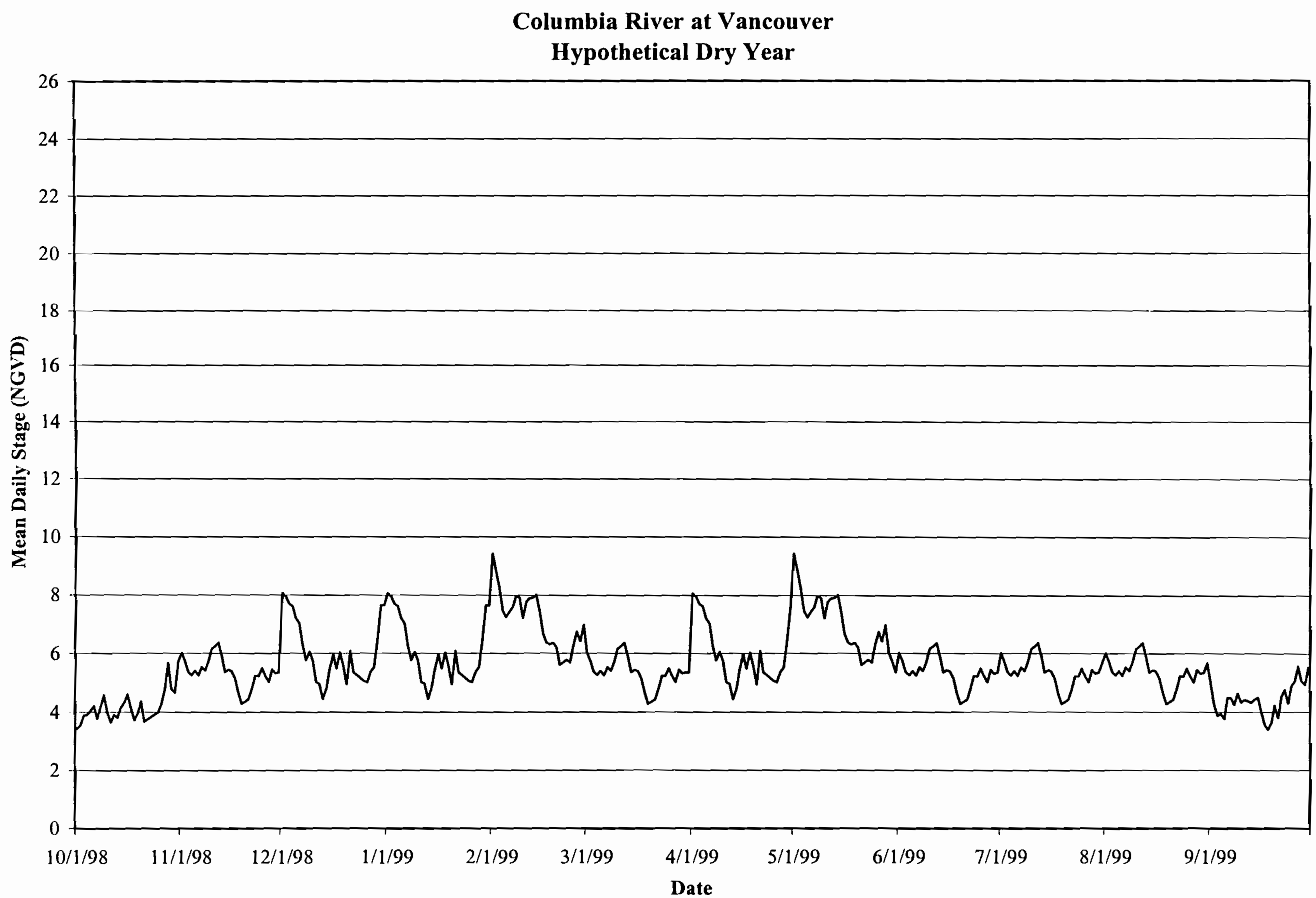
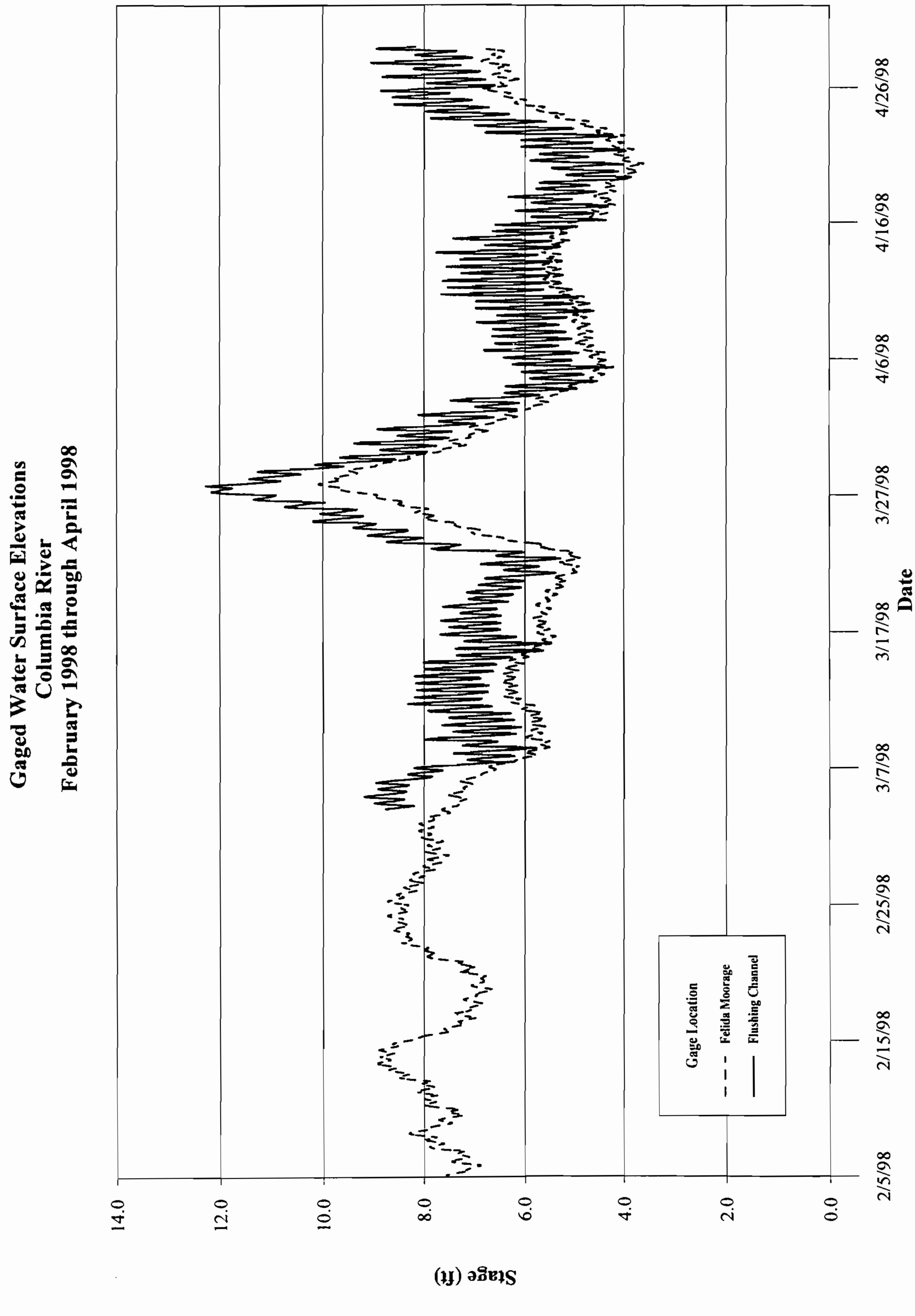

Vancouver Lake - Elevation vs Volume
Volume (ac-ft)
0
20000
40000
60000
80000
100000
120000
Elevation (ft NGVD)
-10
-5
0
5
10
15
20
25
30

Figure 3.5



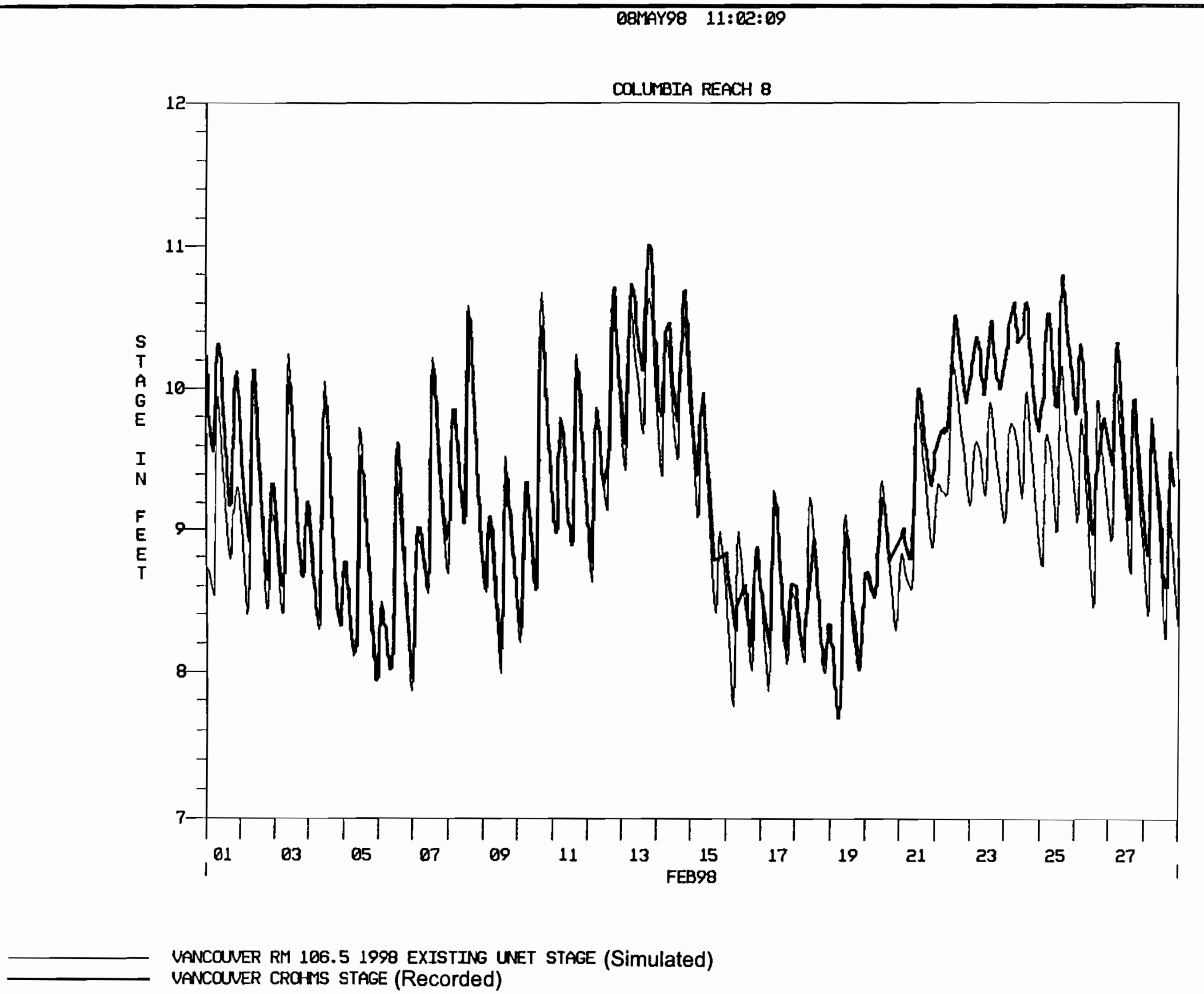
Columbia River at Vancouver
Hypothetical Dry Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/98
11/1/98
12/1/98
1/1/99
2/1/99
3/1/99
4/1/99
5/1/99
6/1/99
7/1/99
8/1/99
9/1/99
Date

Figure 3.6



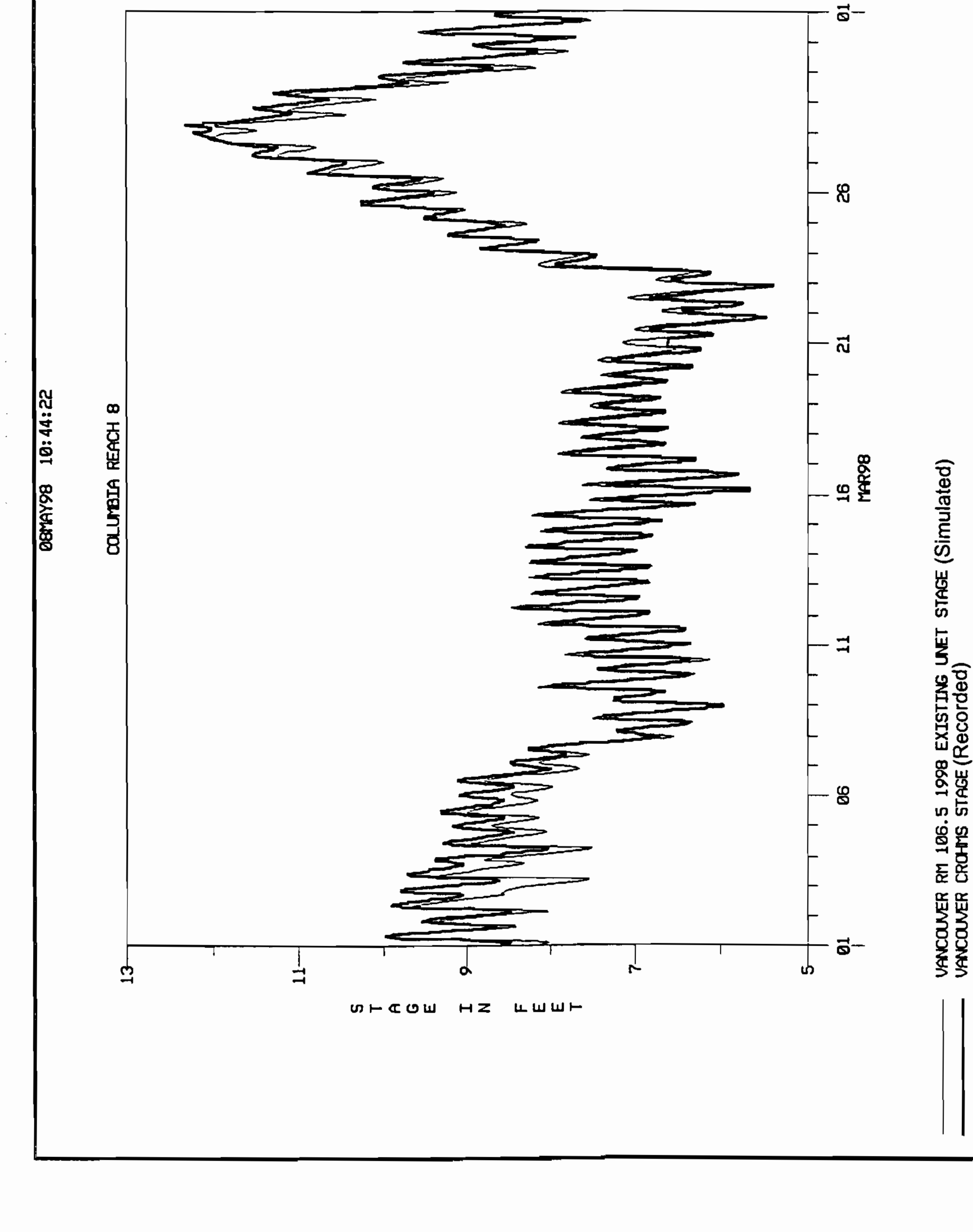
Gaged Water Surface Elevations
Columbia River
February 1998 through April 1998
Stage (ft)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
2/5/98
2/15/98
2/25/98
3/7/98
3/17/98
3/27/98
4/6/98
4/16/98
4/26/98
Date
Gage Location
Felida Moorage
Flushing Channel

Figure 3.7



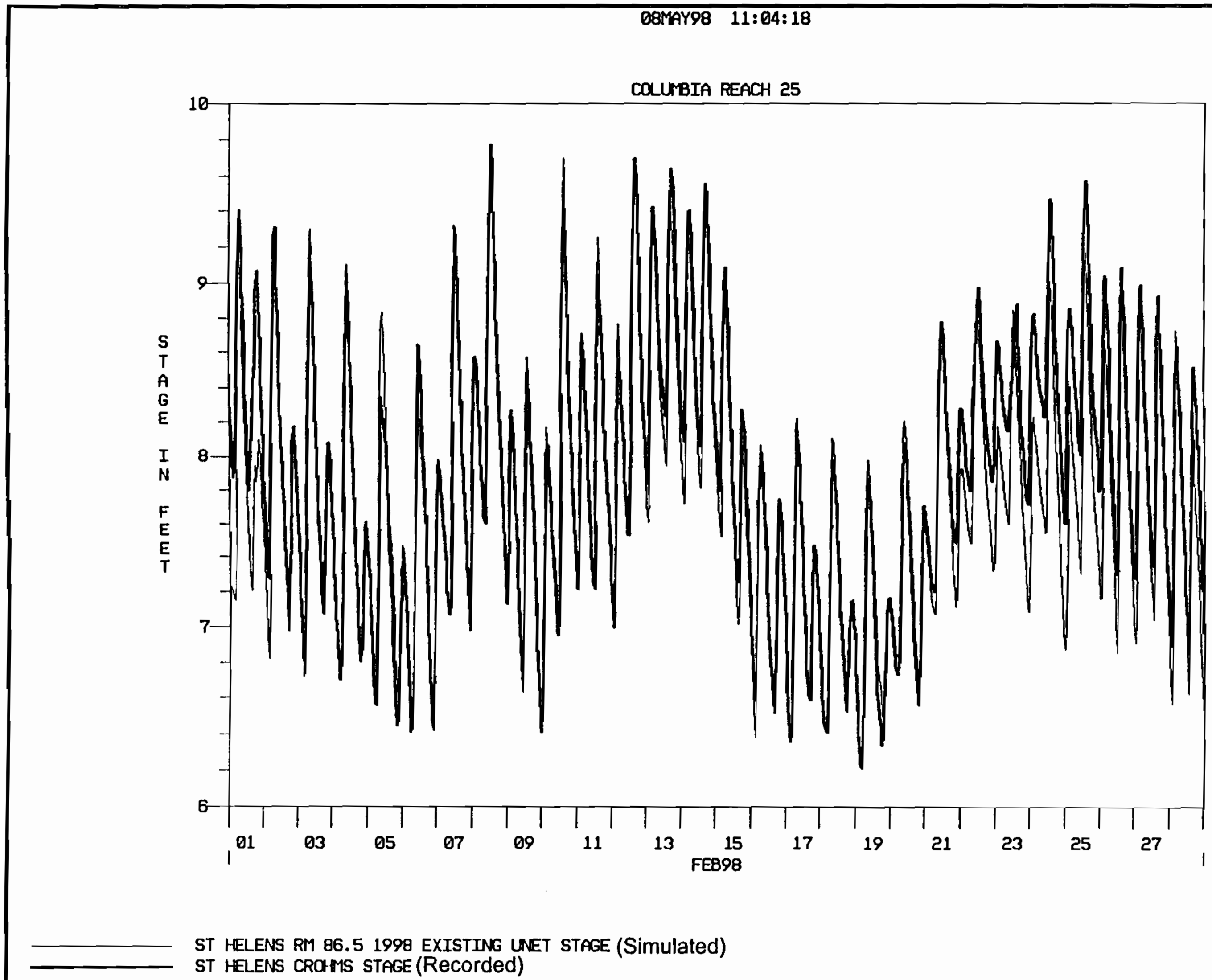
Columbia River at Vancouver. Simulated vs. Recorded Stage, February 1998

Figure 3.8



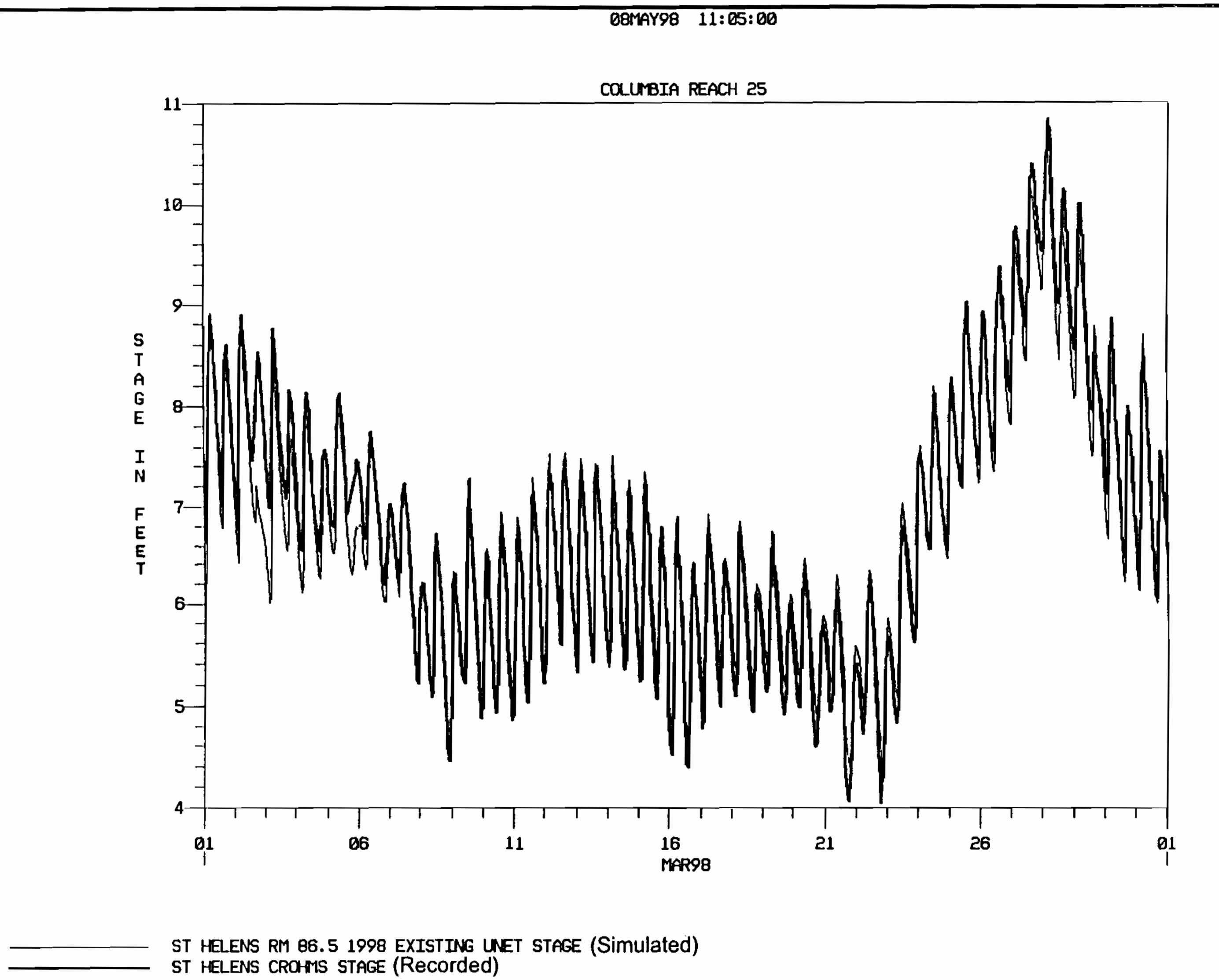
Columbia River at Vancouver. Simulated vs. Recorded Stage, March 1998

Figure 3.9



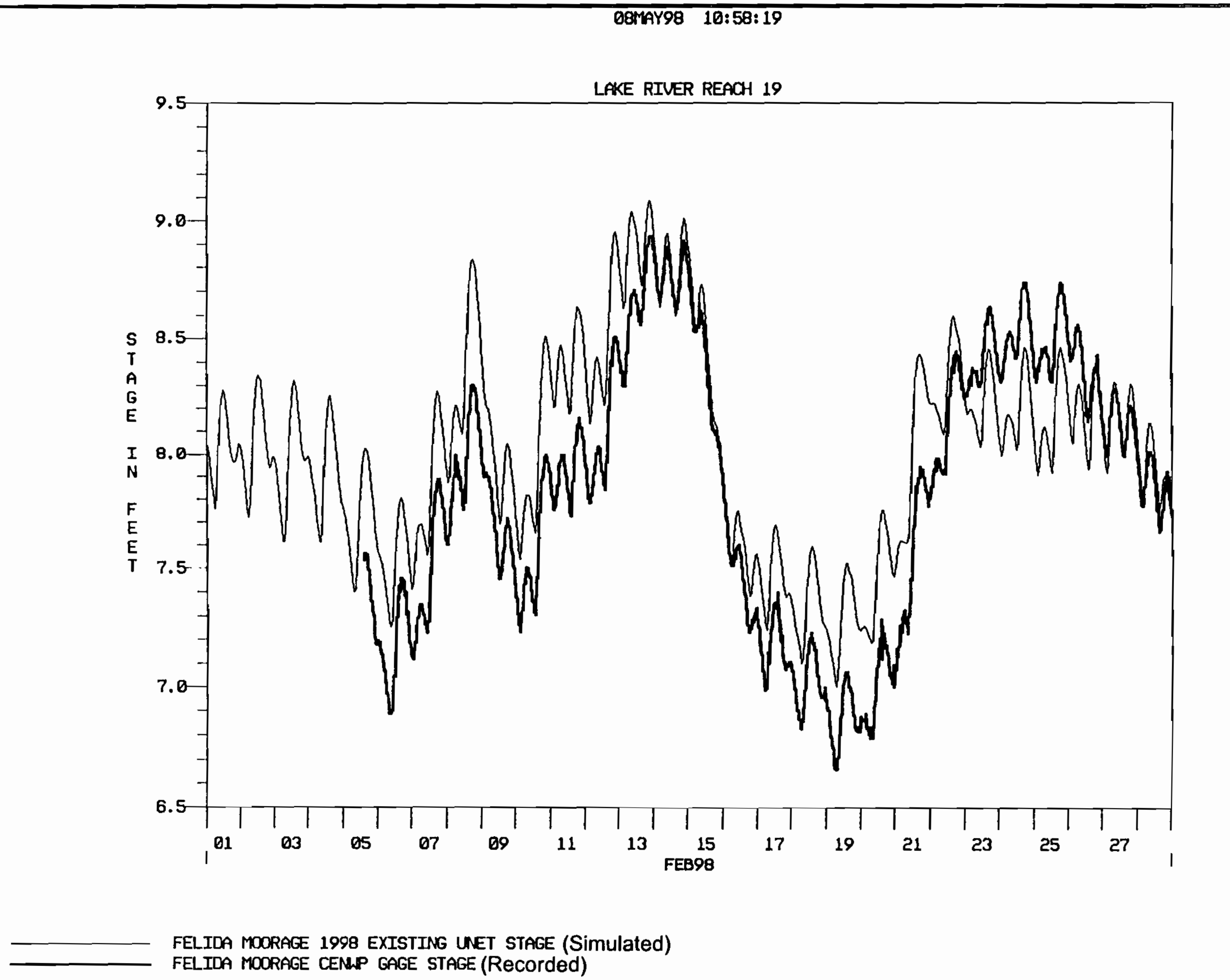
Columbia River at St. Helens. Simulated vs. Recorded Stage, February 1998

Figure 3.10



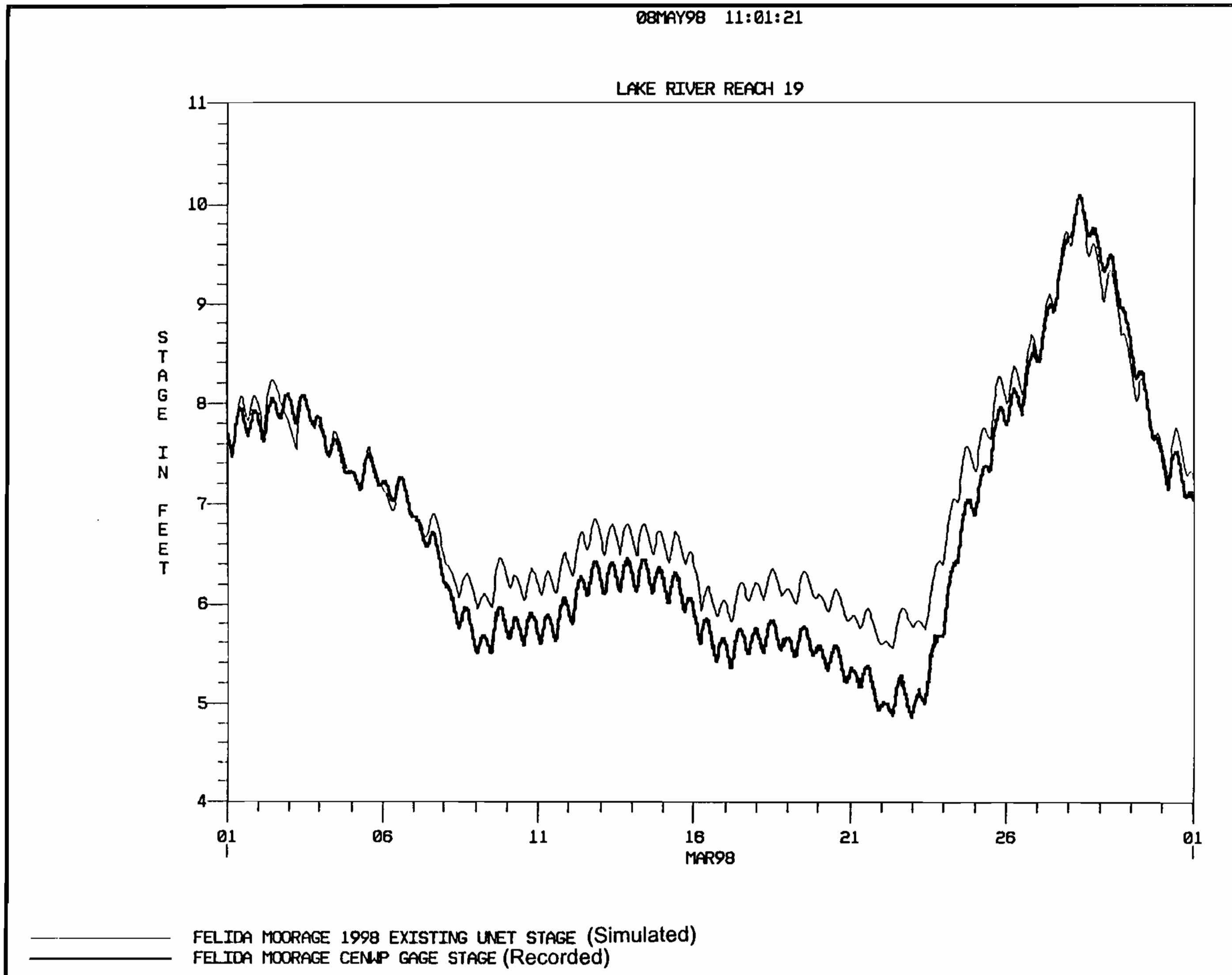
Columbia River at St. Helens. Simulated vs. Recorded Stage, March 1998

Figure 3.11



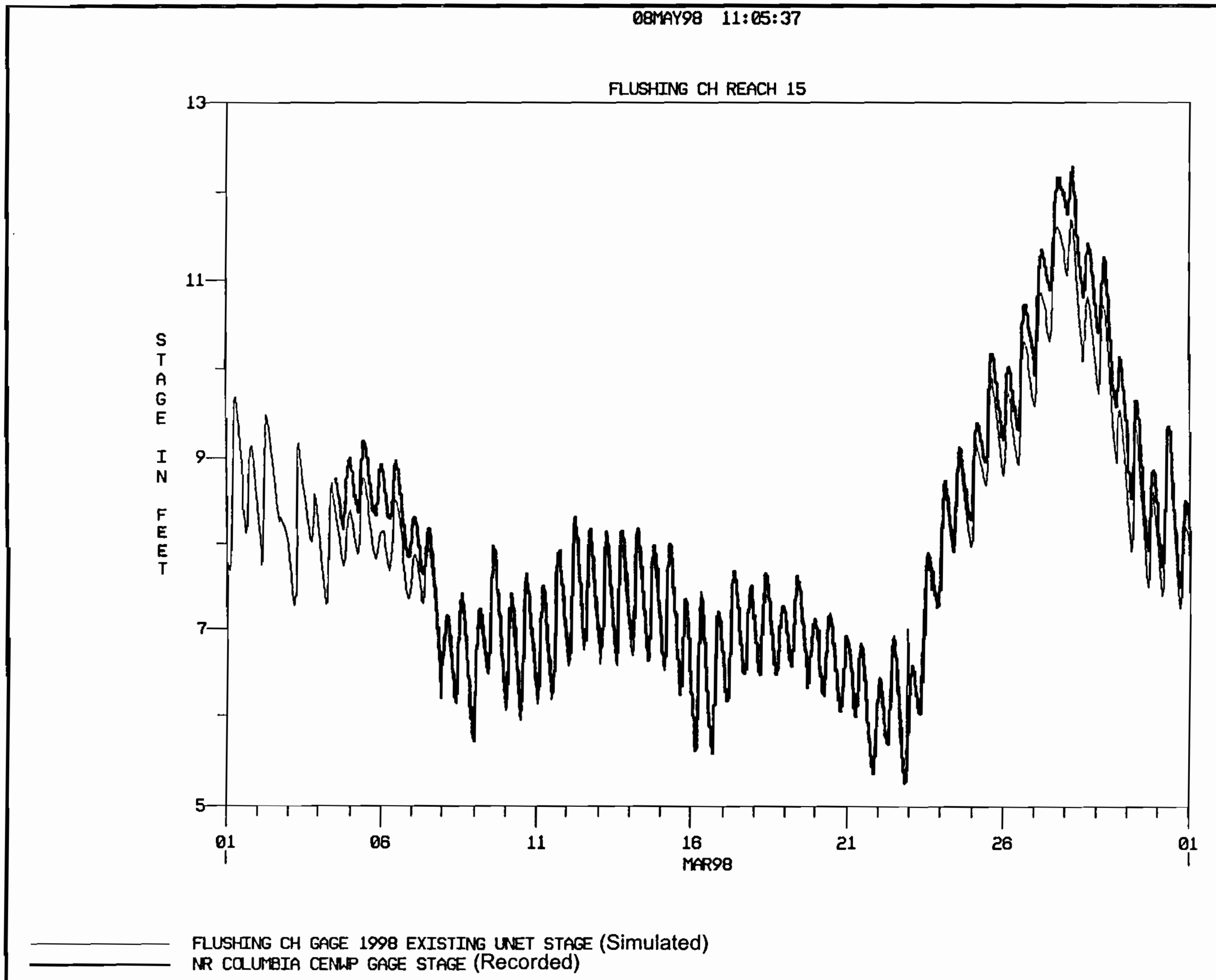
Lake River at Felida Moorage (RM 10.8). Simulated vs. Recorded Stage, February 1998

Figure 3.12



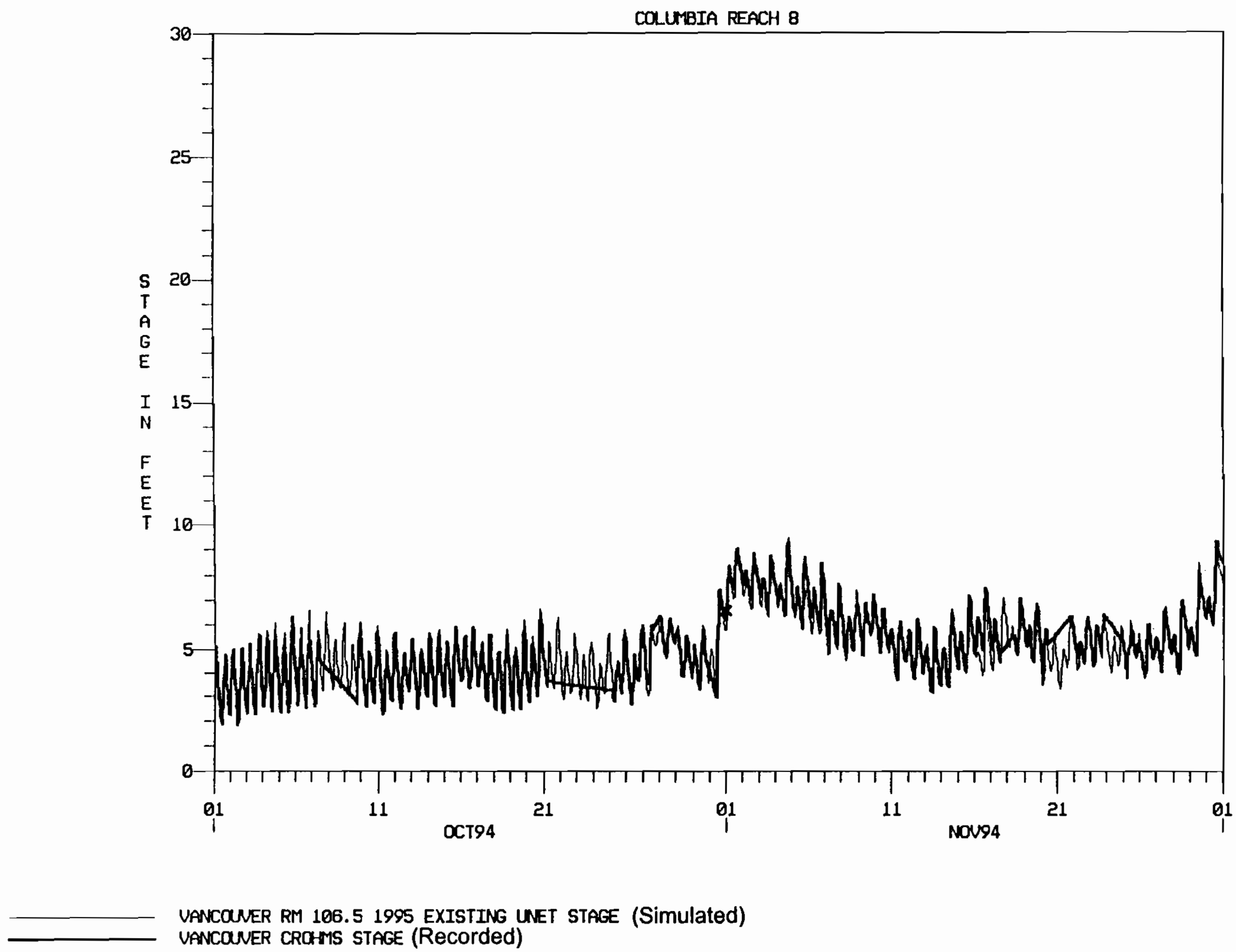
Lake River at Felida Moorage (RM 10.8). Simulated vs. Recorded Stage, March 1998

Figure 3.13



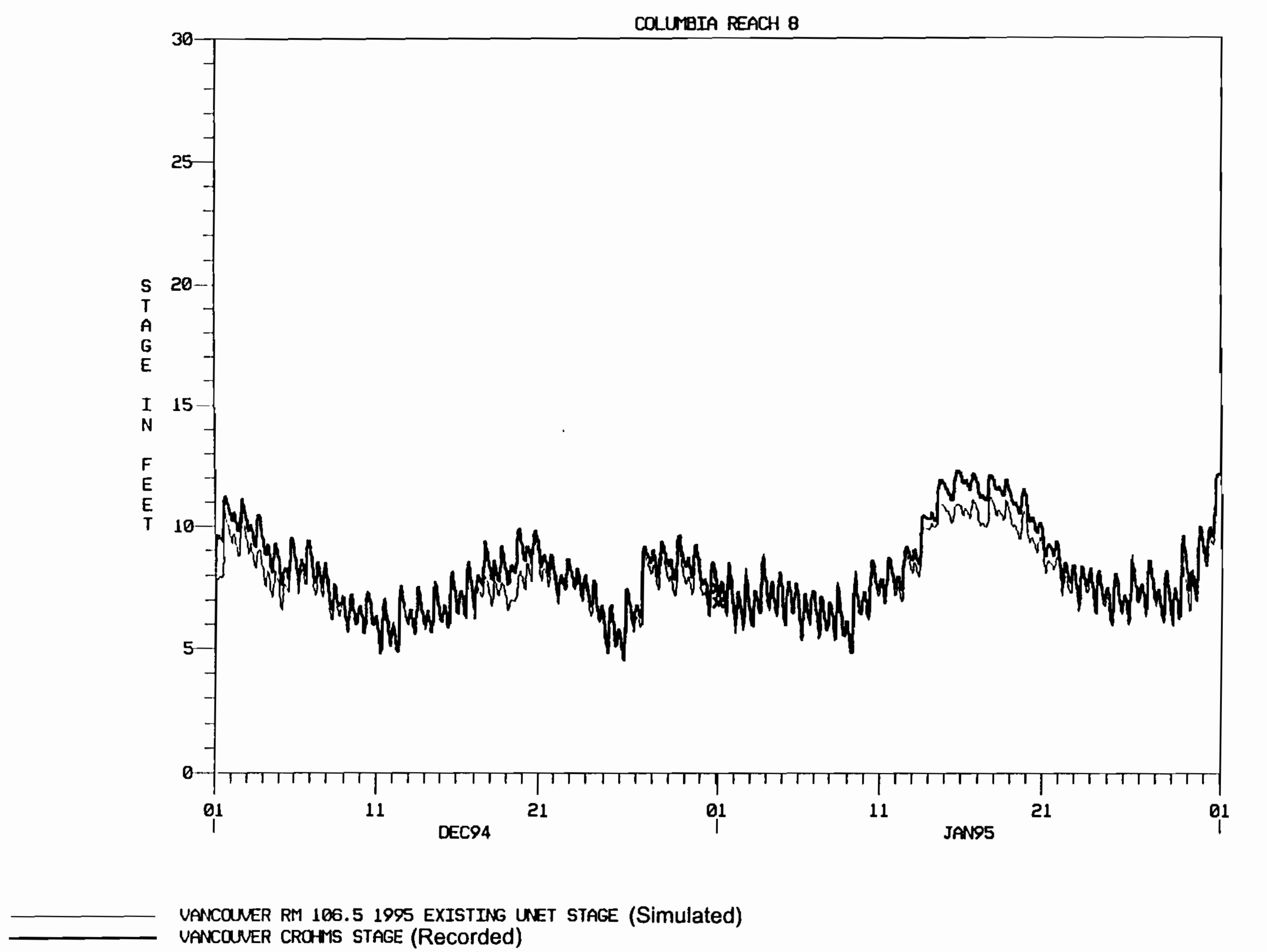
Vancouver Lake Flushing Channel. Simulated vs. Recorded Stage, March 1998

Figure 3.14



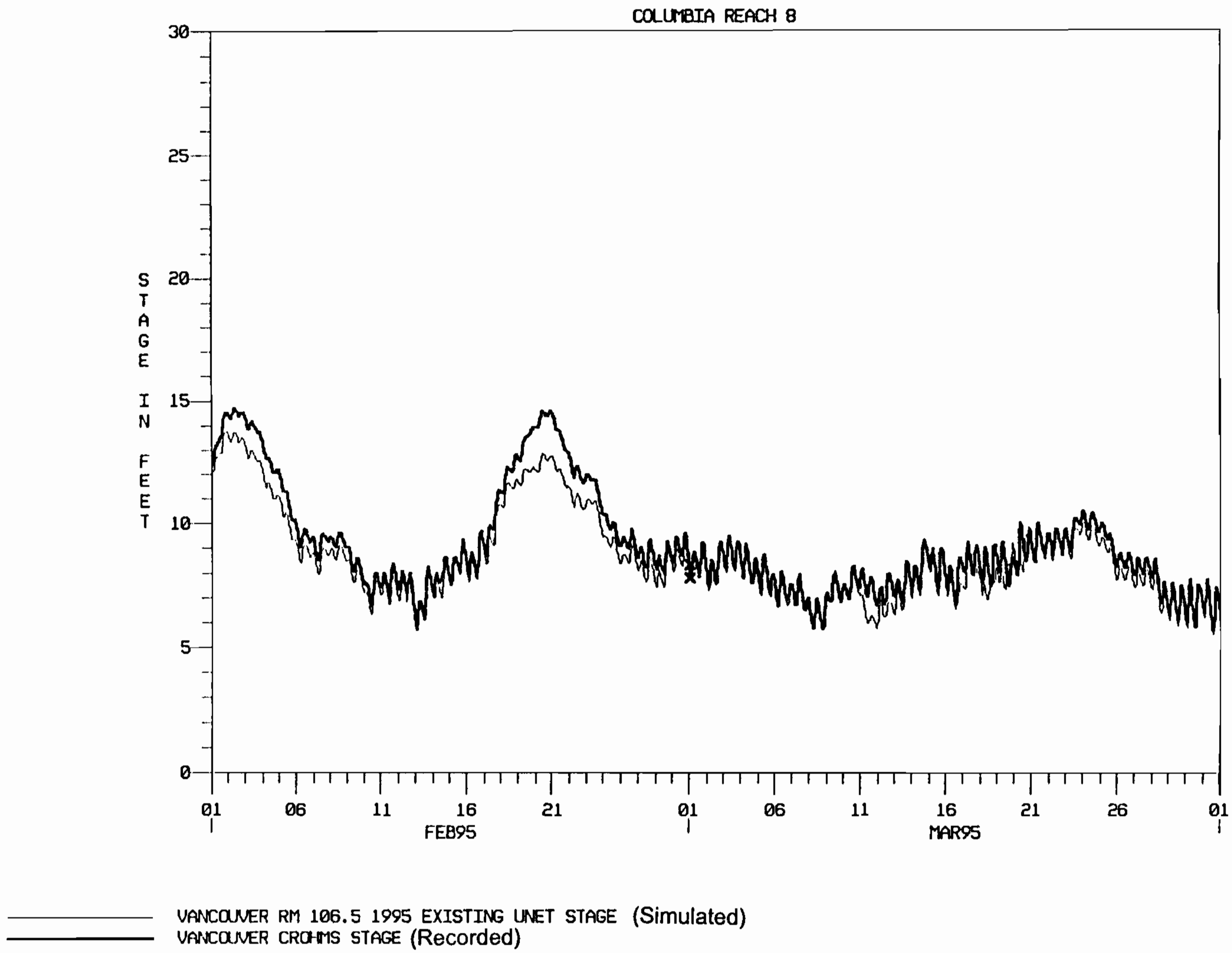
Columbia River at Vancouver. Simulated vs. Recorded Stage, October-November 1994

Figure 3.15



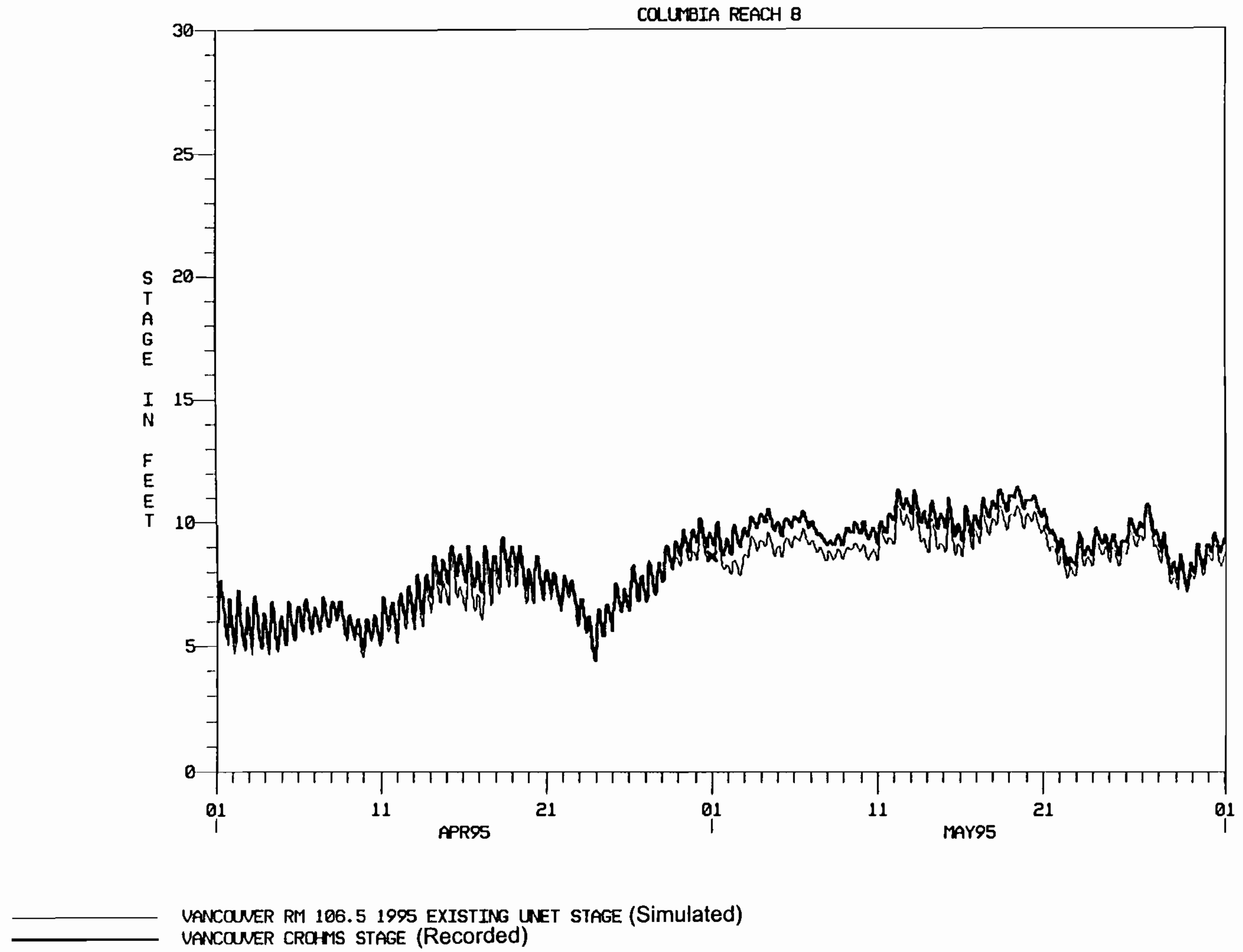
Columbia River at Vancouver. Simulated vs. Recorded Stage, December 1994-January 1995

Figure 3.16



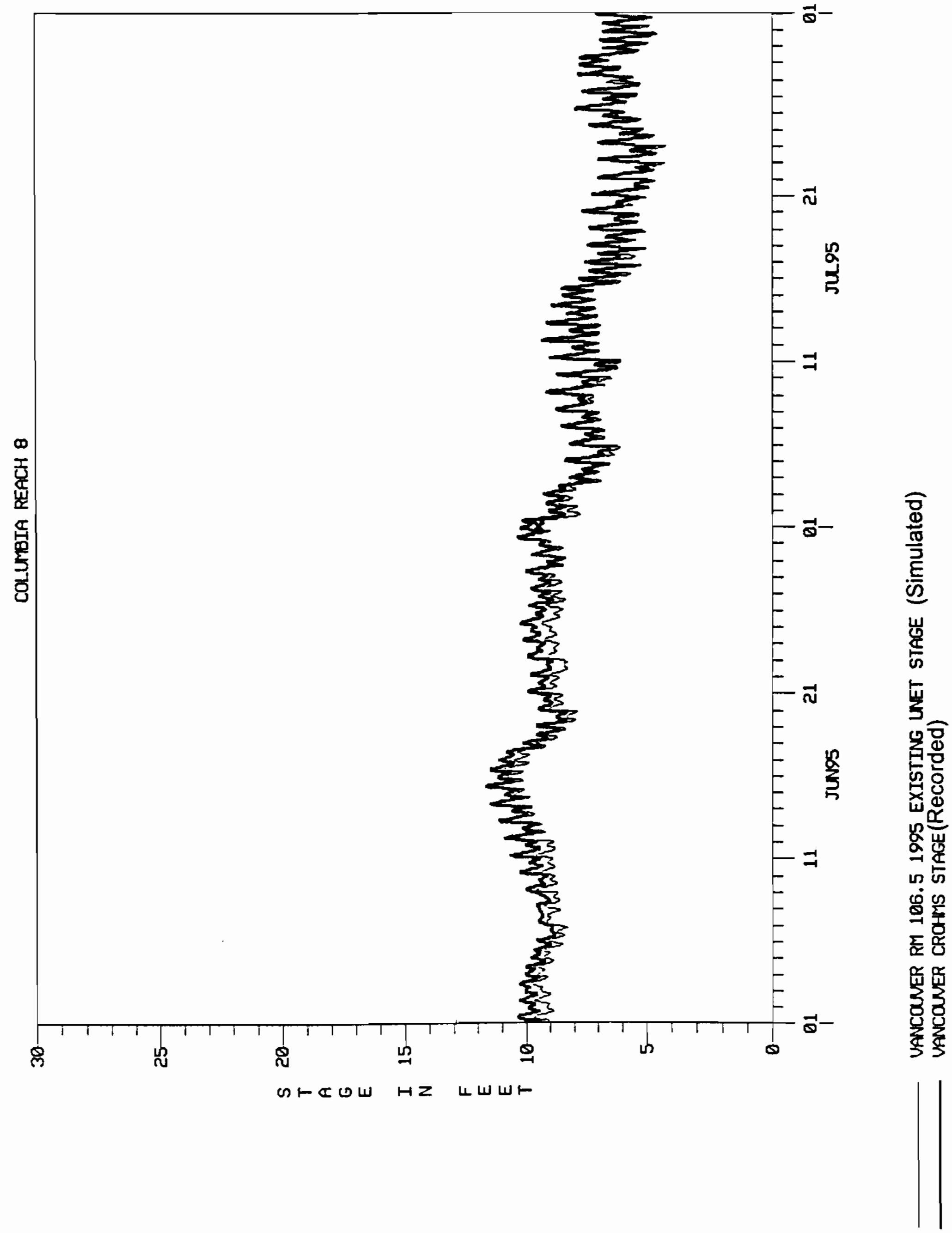
Columbia River at Vancouver. Simulated vs. Recorded Stage, February-March 1995

Figure 3.17



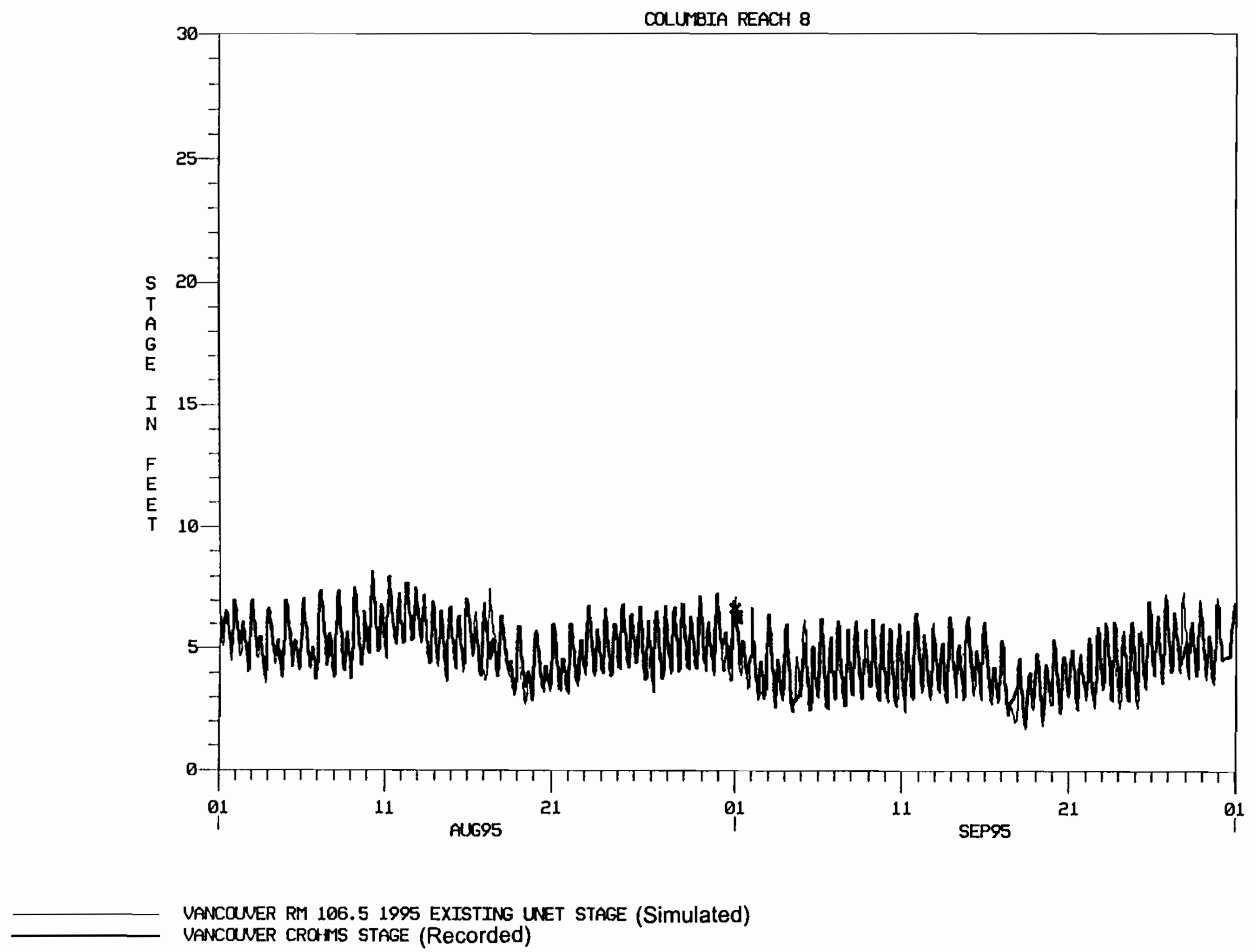
Columbia River at Vancouver. Simulated vs. Recorded Stage, April-May 1995

Figure 3.18



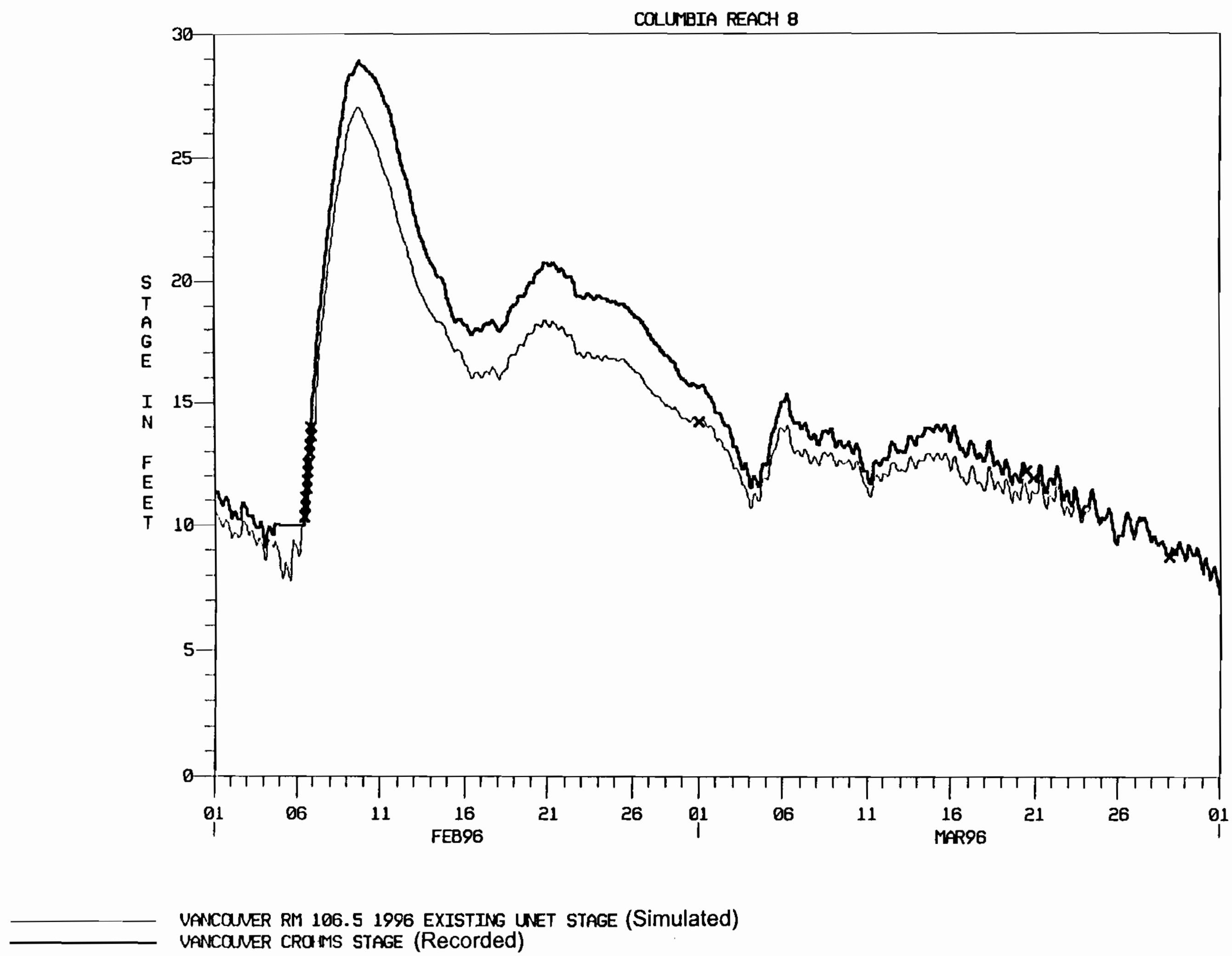
Columbia River at Vancouver. Simulated vs. Recorded Stage, June-July 1995

Figure 3.19



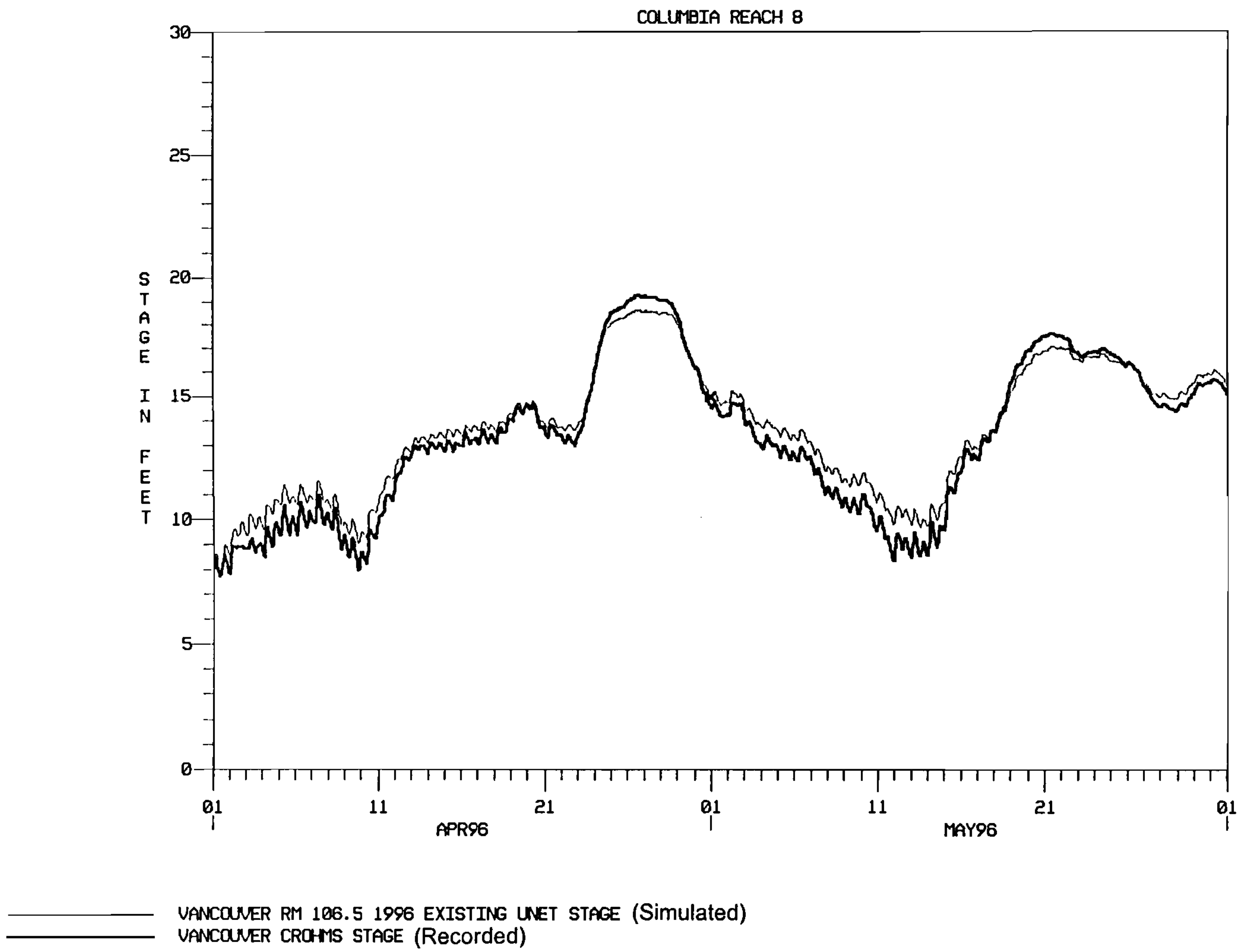
Columbia River at Vancouver. Simulated vs. Recorded Stage, August-September 1995

Figure 3.20



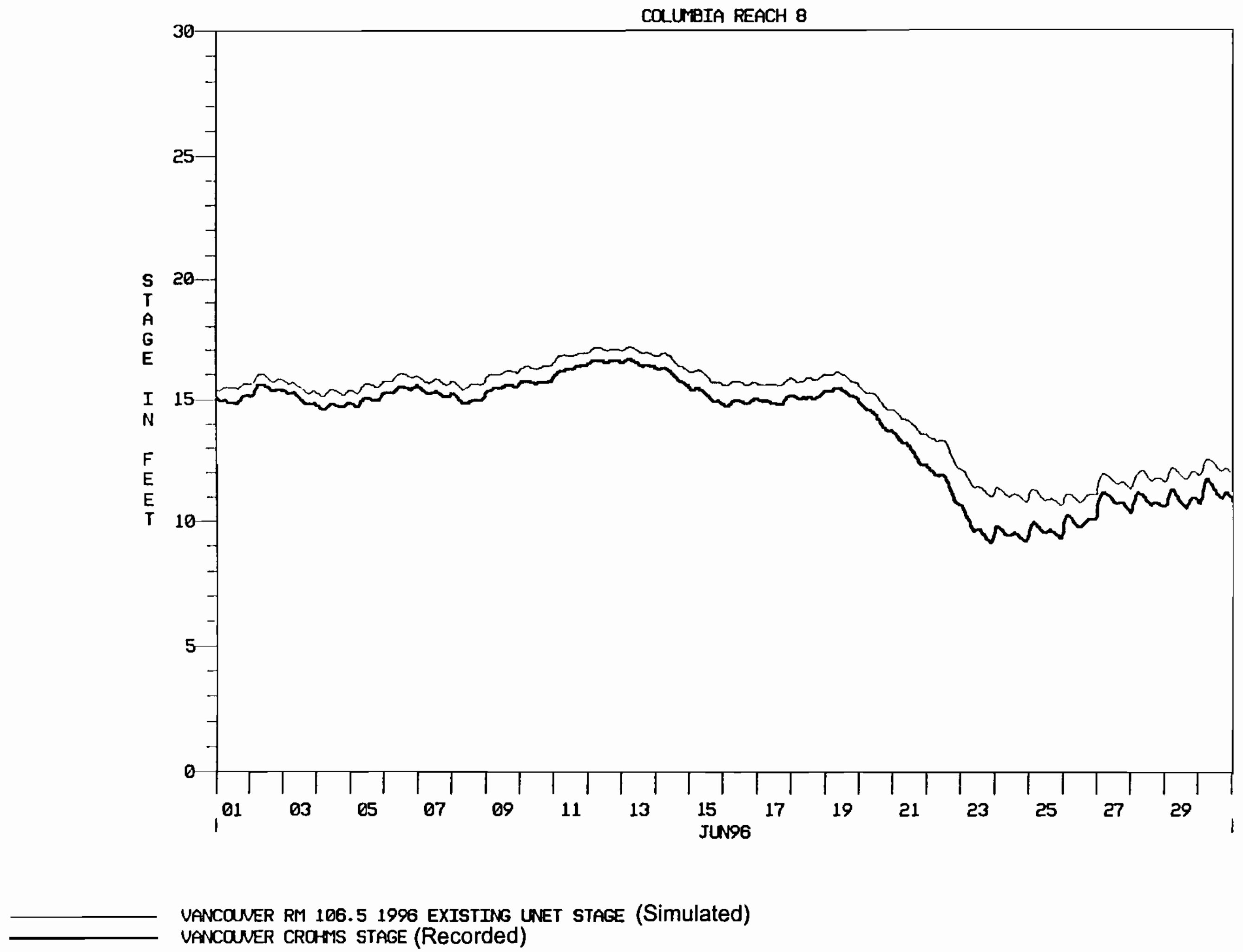
Columbia River at Vancouver. Simulated vs. Recorded Stage, February-March 1996

Figure 3.21



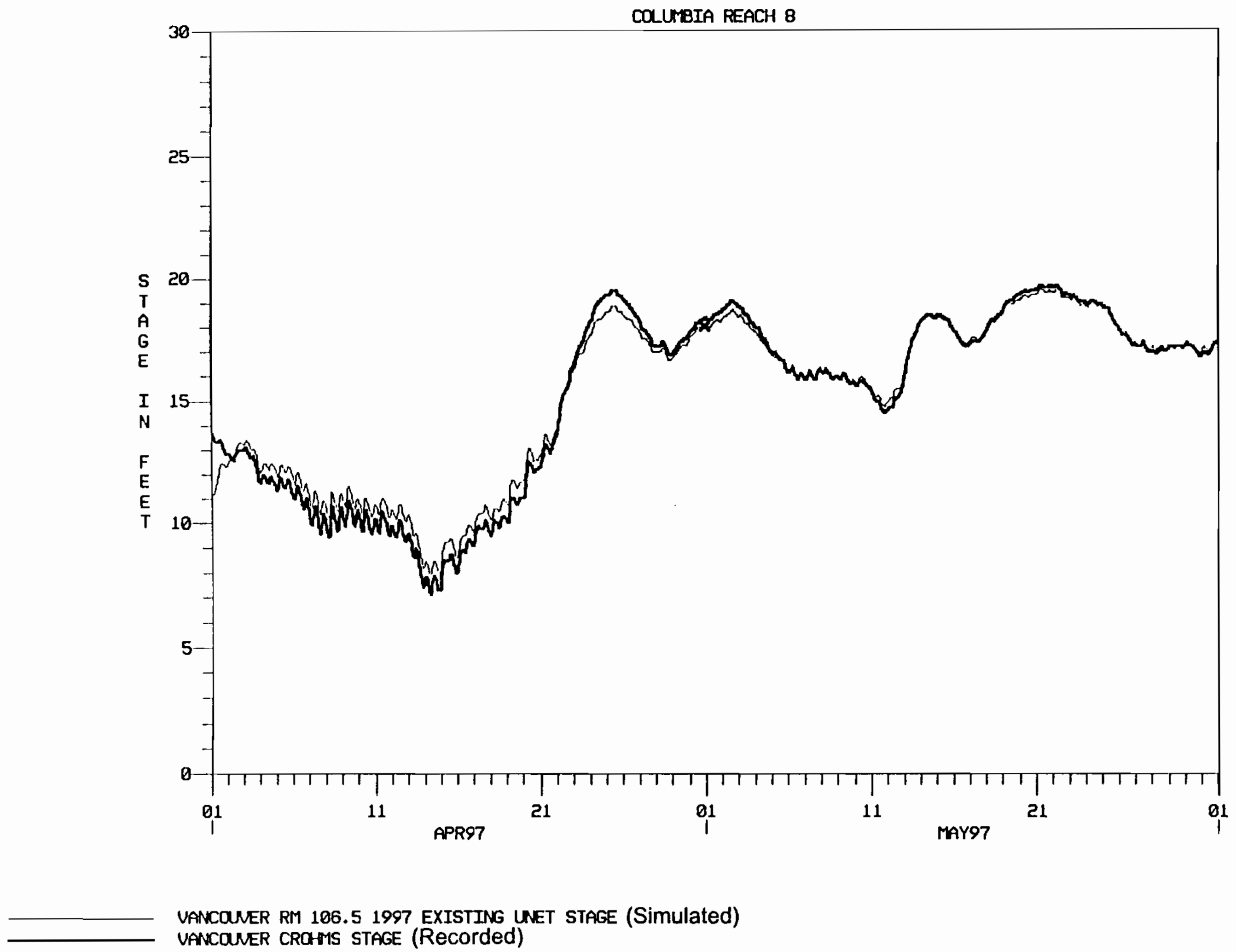
Columbia River at Vancouver. Simulated vs. Recorded Stage, April-May 1996

Figure 3.22



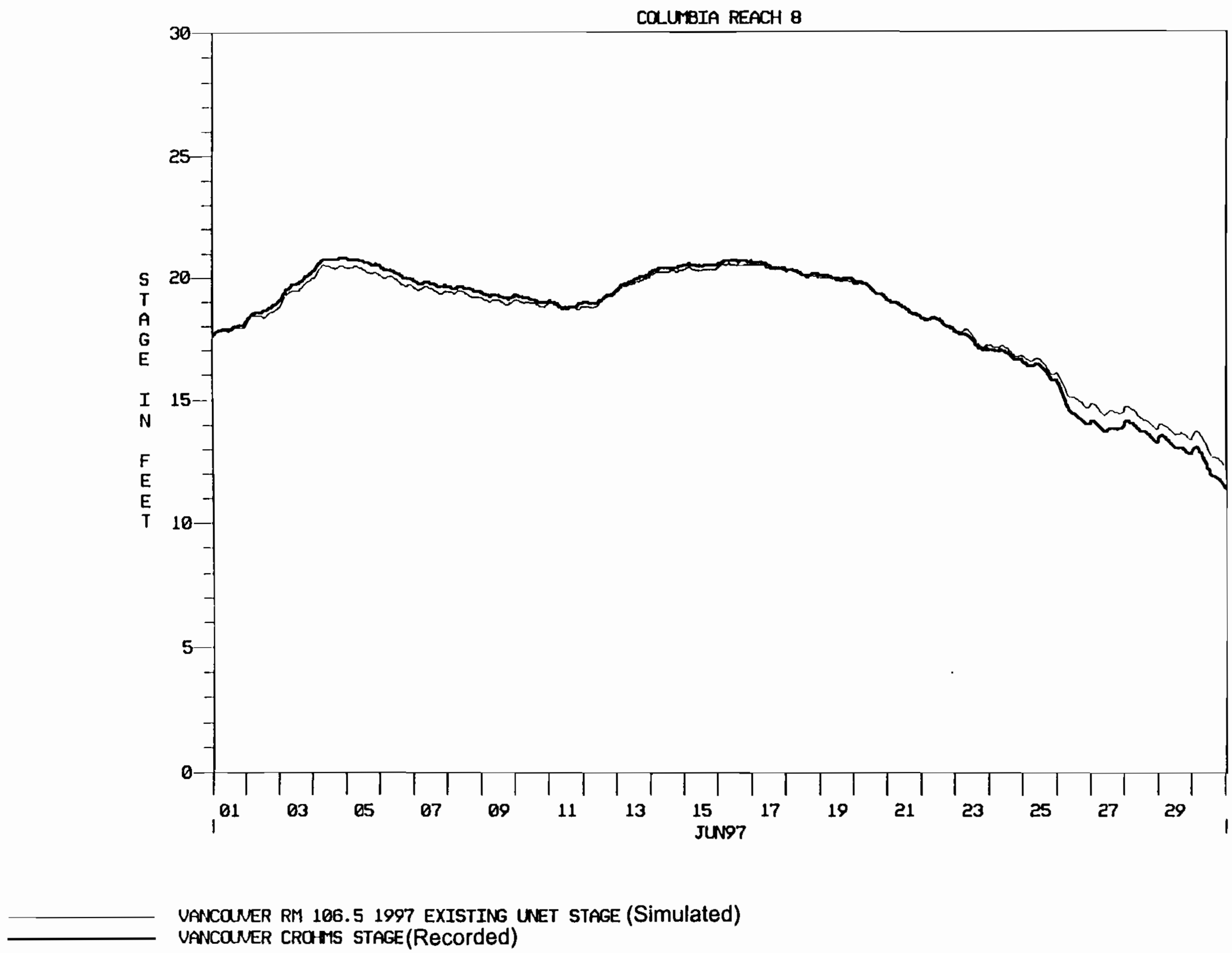
Columbia River at Vancouver. Simulated vs. Recorded Stage, June 1996

Figure 3.23



Columbia River at Vancouver. Simulated vs. Recorded Stage, April-May 1997

Figure 3.24



Columbia River at Vancouver. Simulated vs. Recorded Stage, June 1997

Figure 3.25

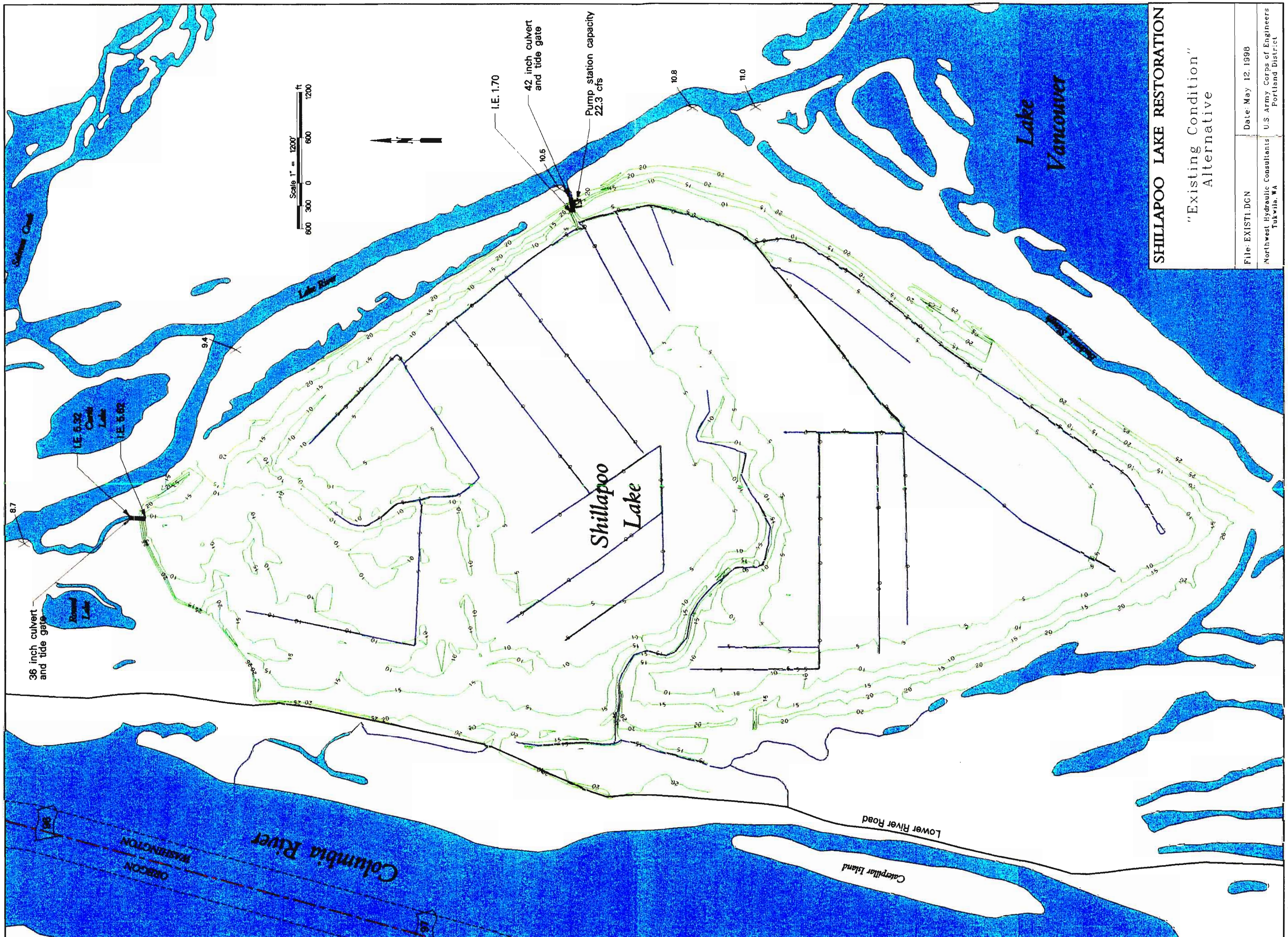
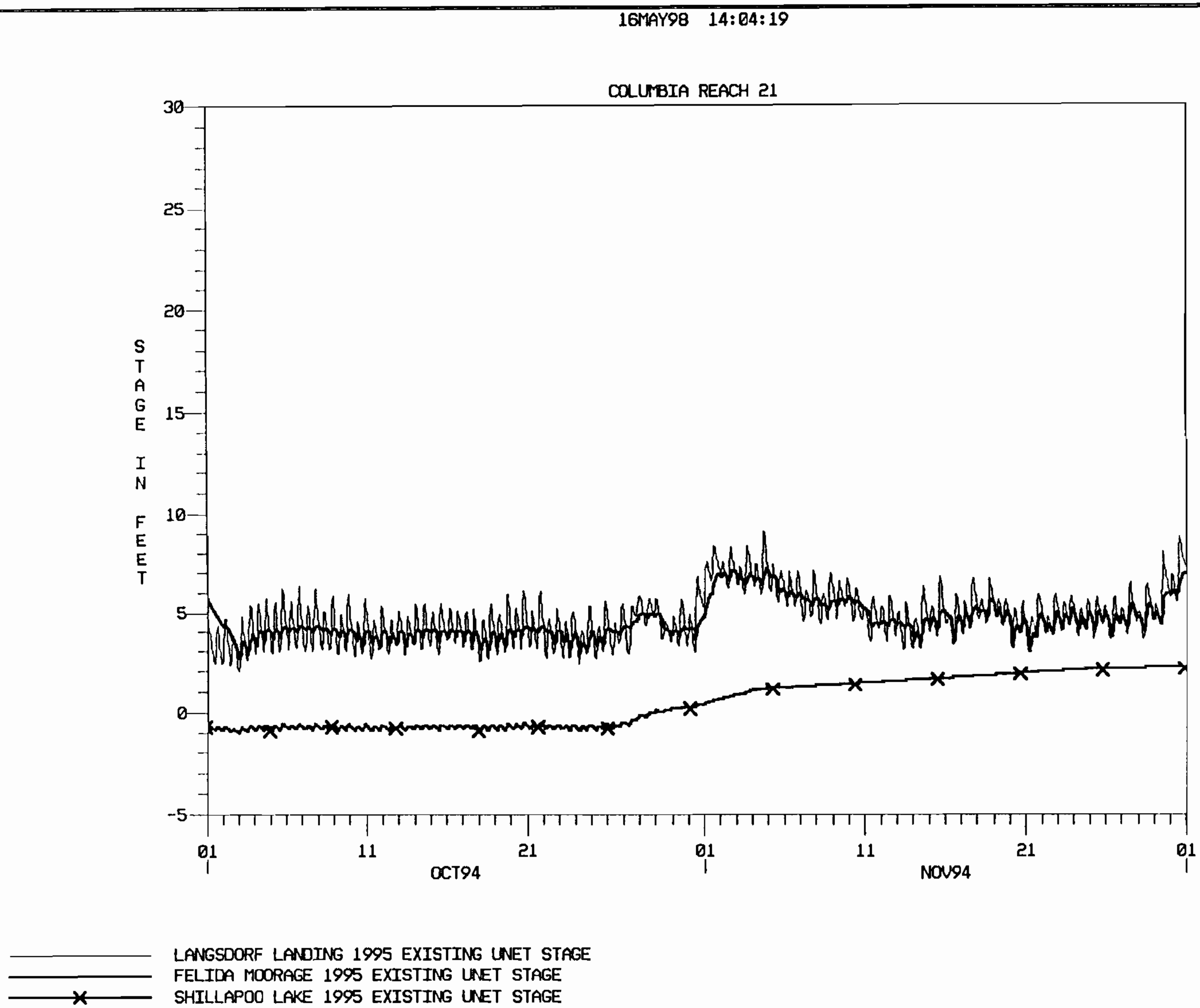
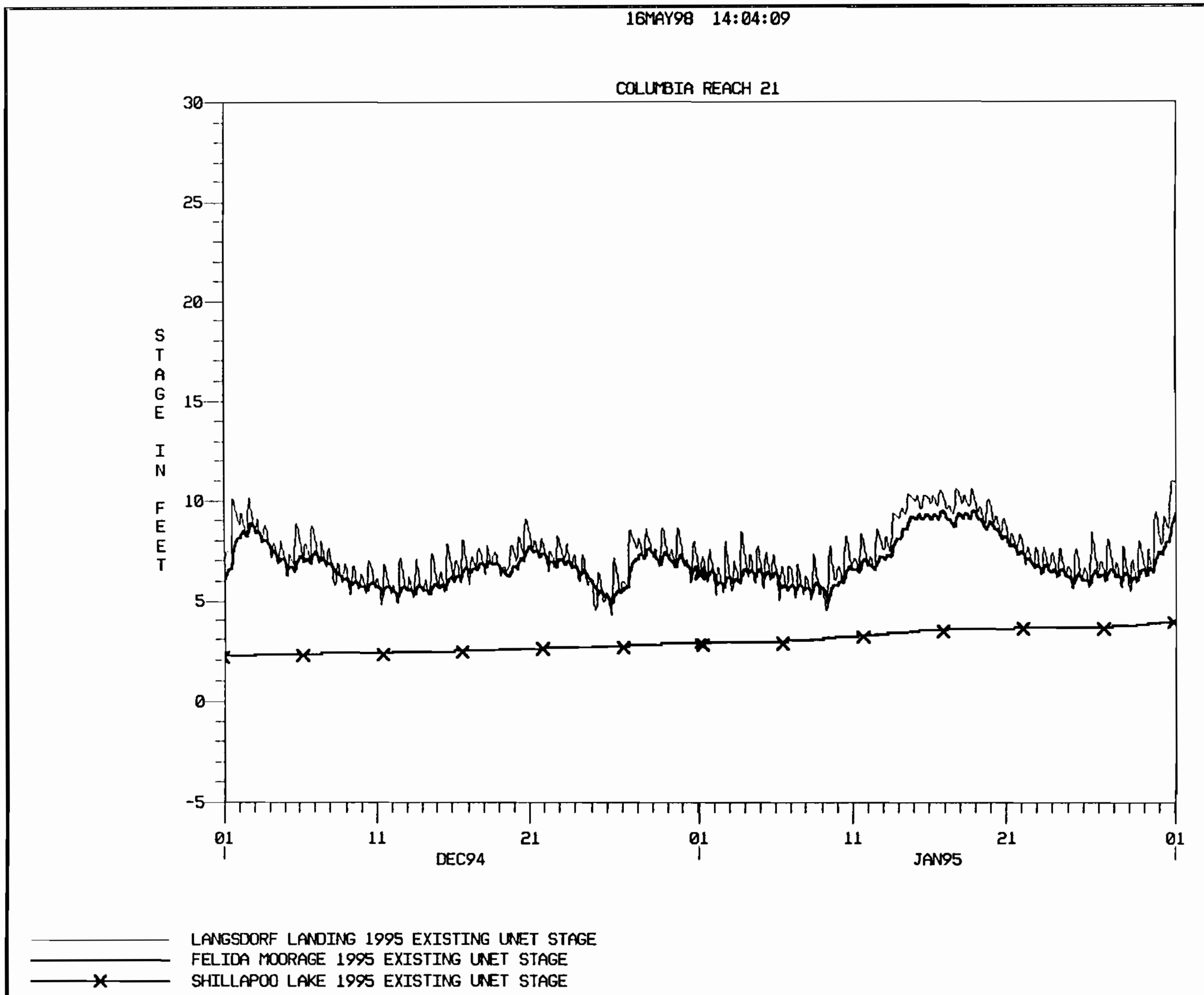

SHILLAPOO LAKE RESTORATION
"Existing Condition" Alternative
File: EXIST1.DGN
Date: May 12, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District
Lake Vancouver
Lake River
Shillapoo Lake
Columbia River
Lower River Road
Caterpillar Island
WASHINGTON
OREGON
I.E. 1.70
42 inch culvert and tide gate
Pump station capacity 22.3 cfs
36 inch culvert and tide gate
I.E. 5.52
I.E. 5.62
Scale 1" = 1200'
10.8
11.0
10.5
9.4
8.7

Figure 4.1



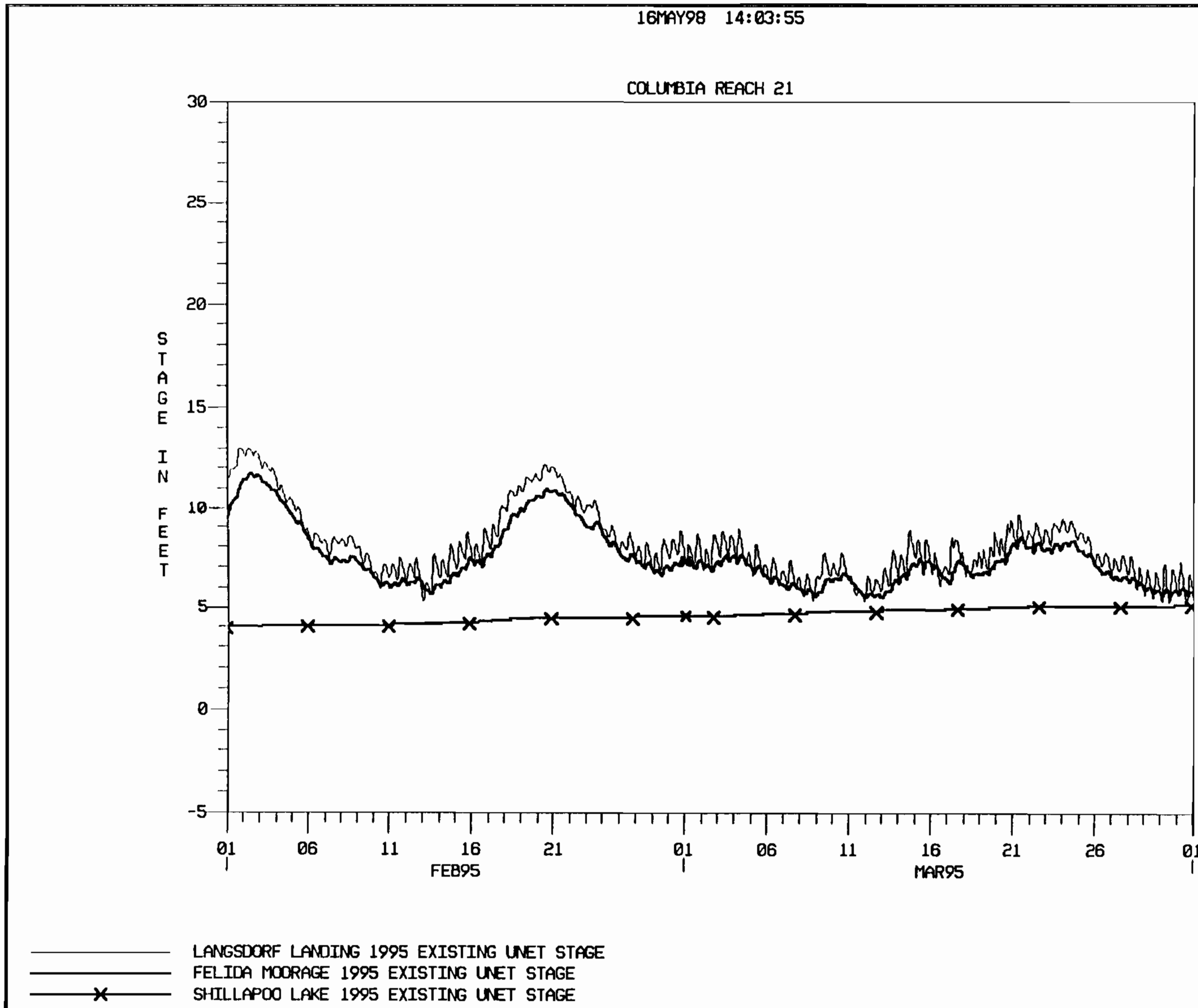
Existing Condition Alternative. Simulation Results October-November 1994

Figure 4.2



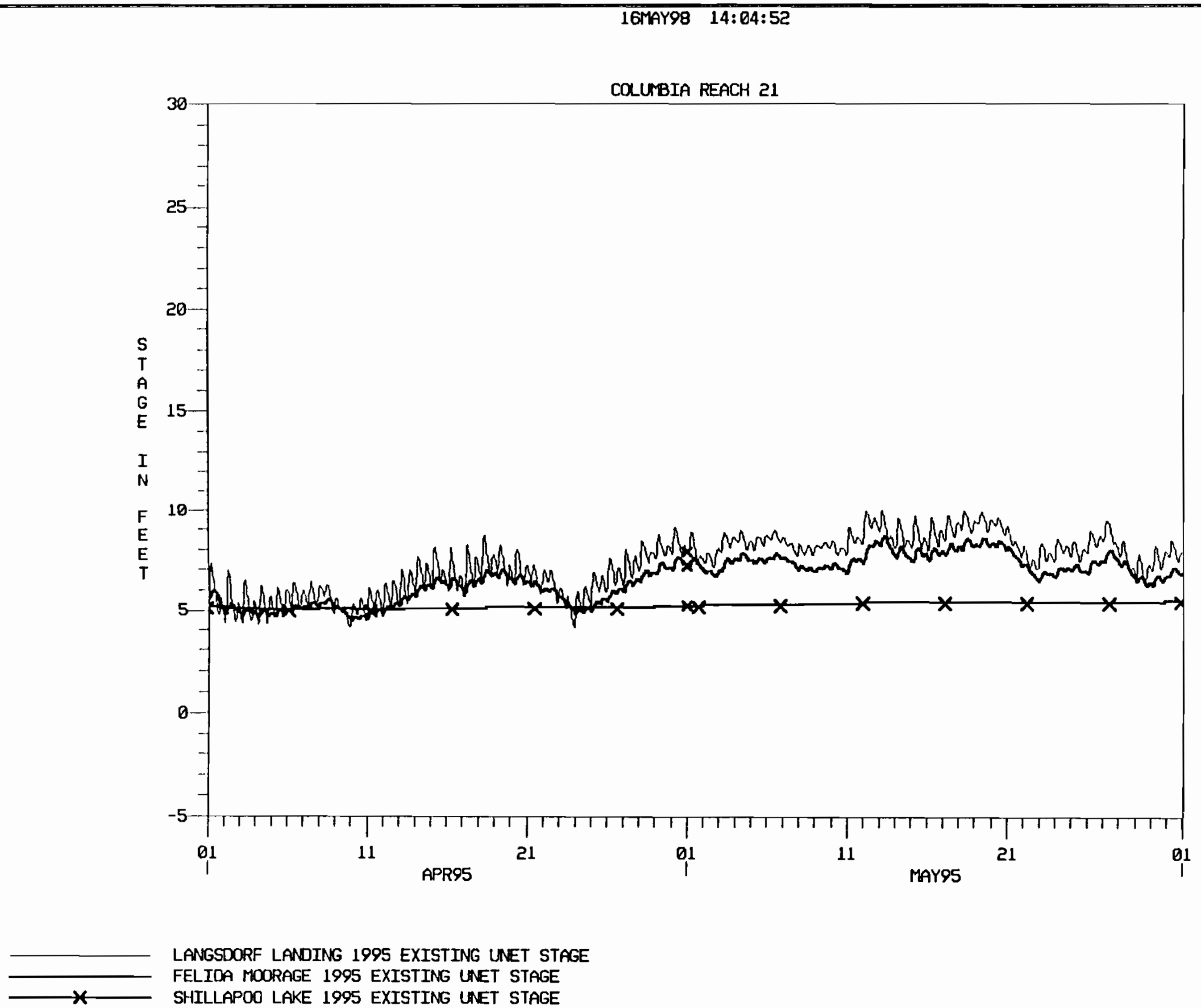
Existing Condition Alternative. Simulation Results December 1994-January 1995

Figure 4.3



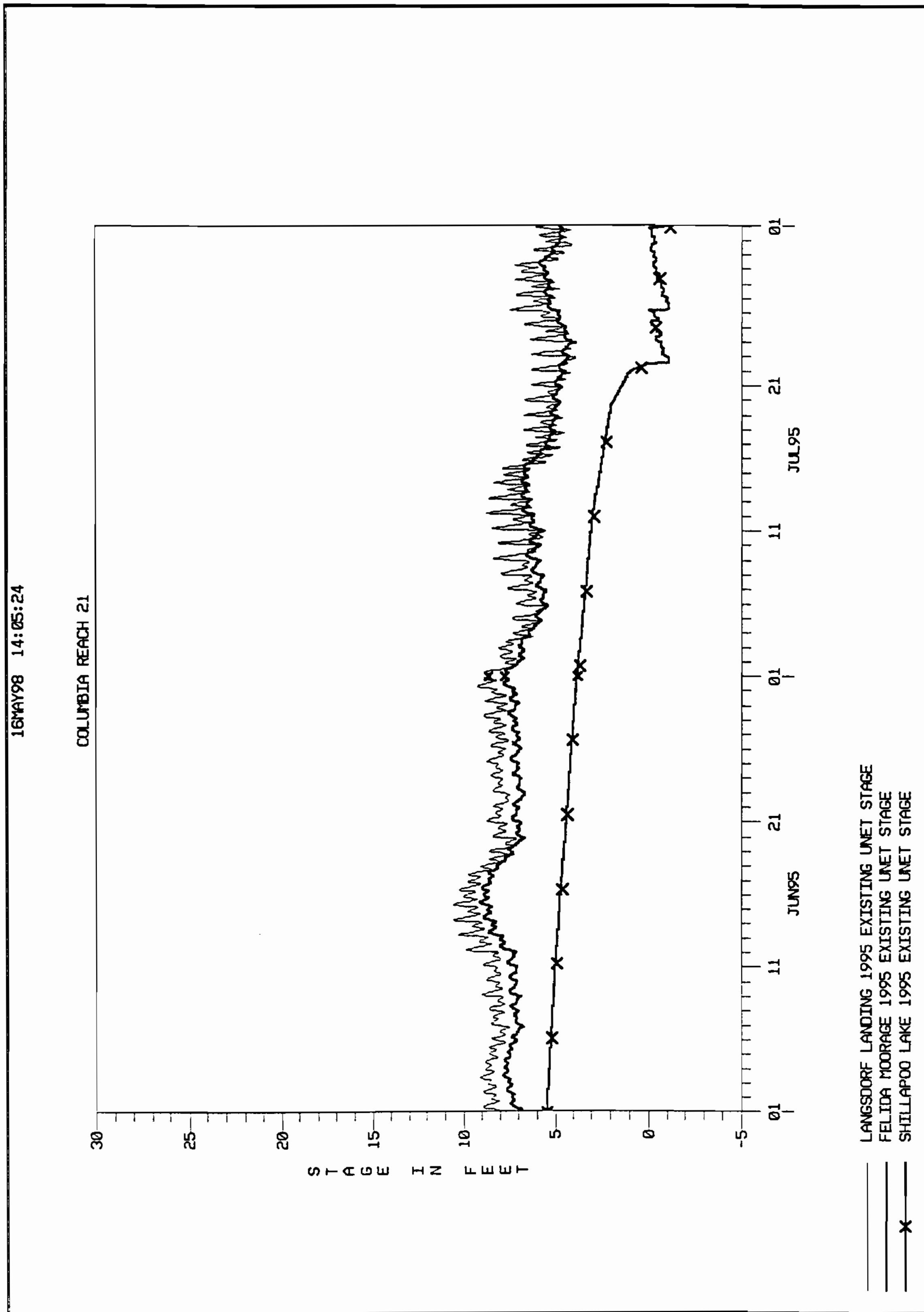
Existing Condition Alternative. Simulation Results February-March 1995

Figure 4.4



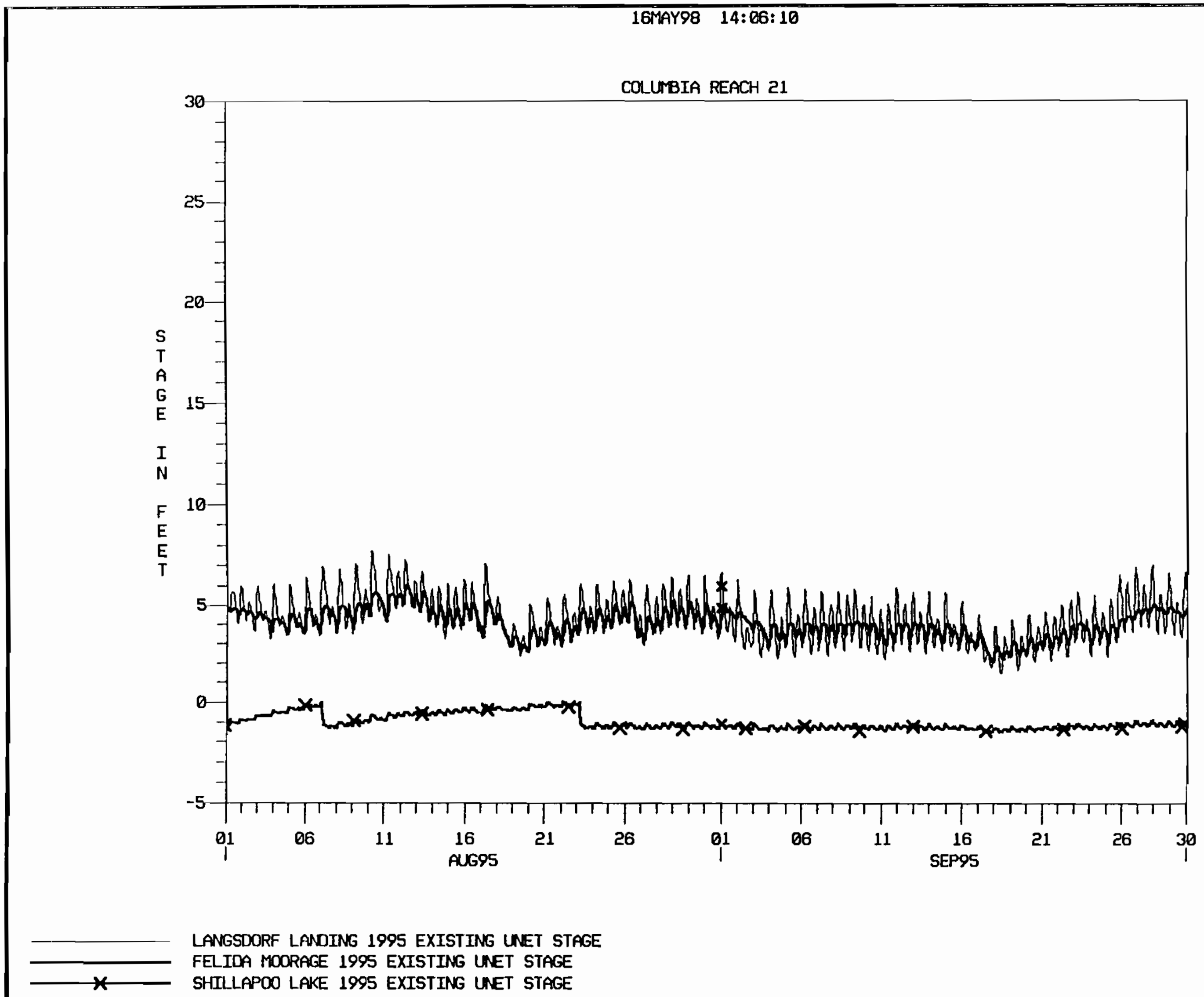
Existing Condition Alternative. Simulation Results April-May 1995

Figure 4.5



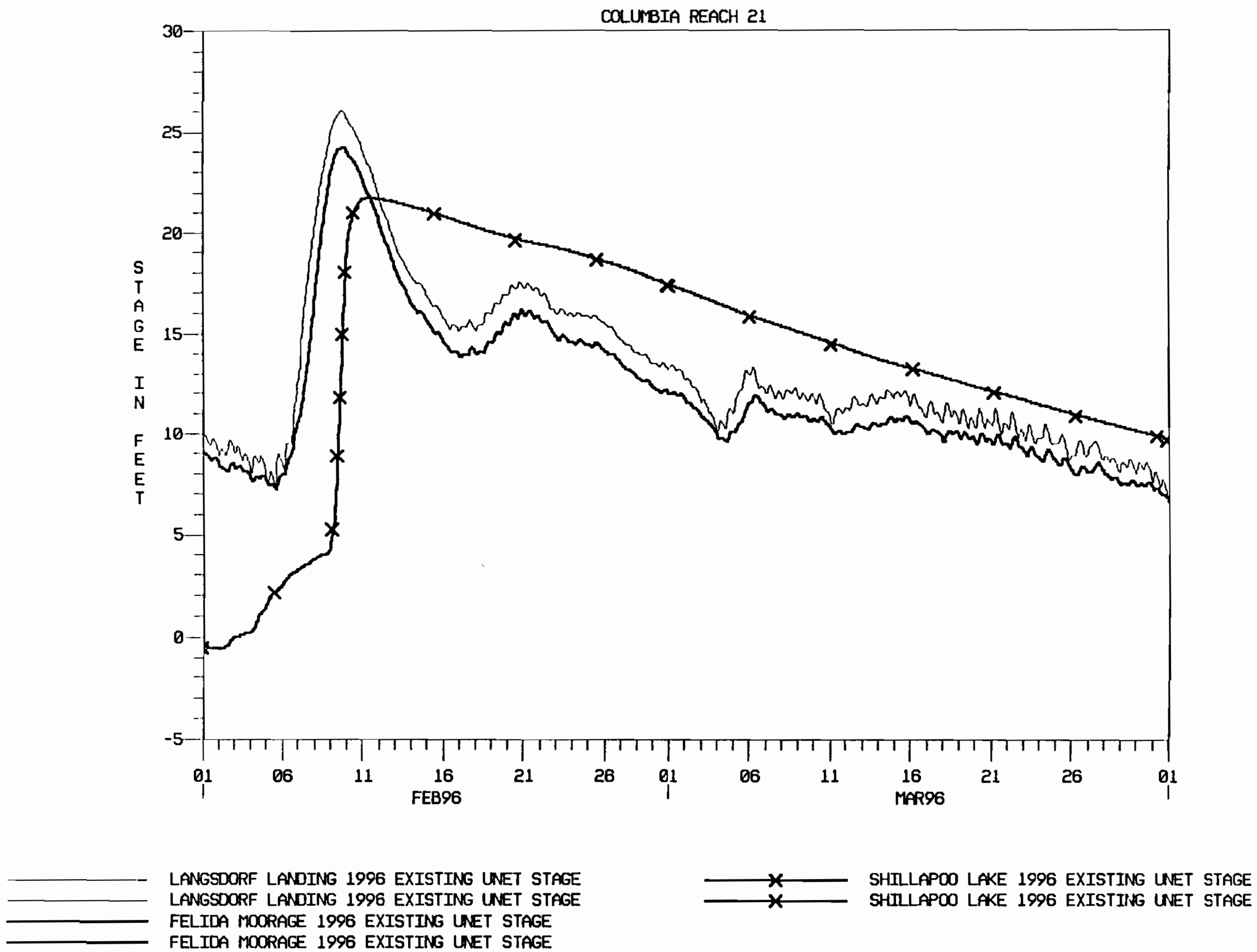
Existing Condition Alternative. Simulation Results June-July 1995

Figure 4.6



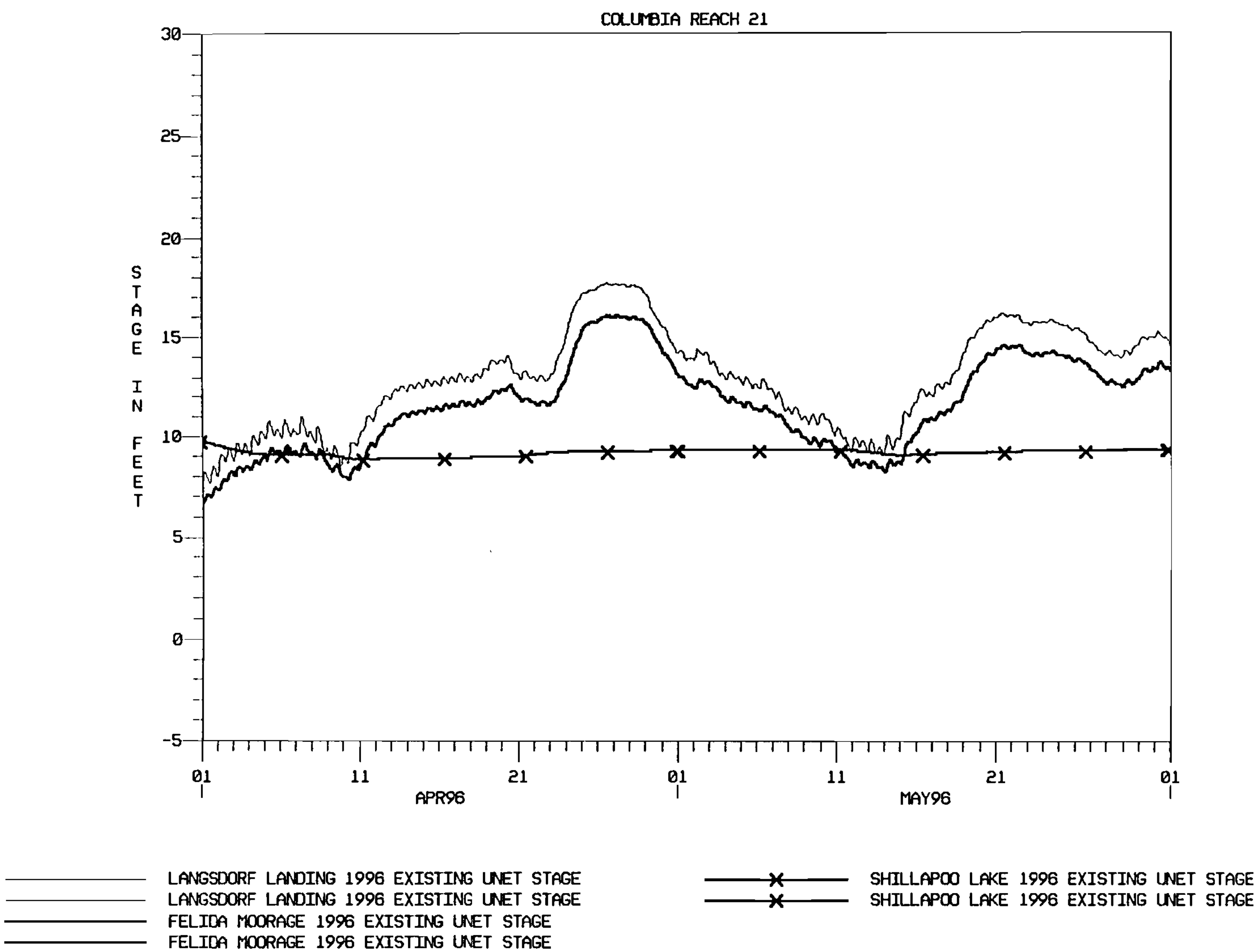
Existing Condition Alternative. Simulation Results August-September 1995

Figure 4.7



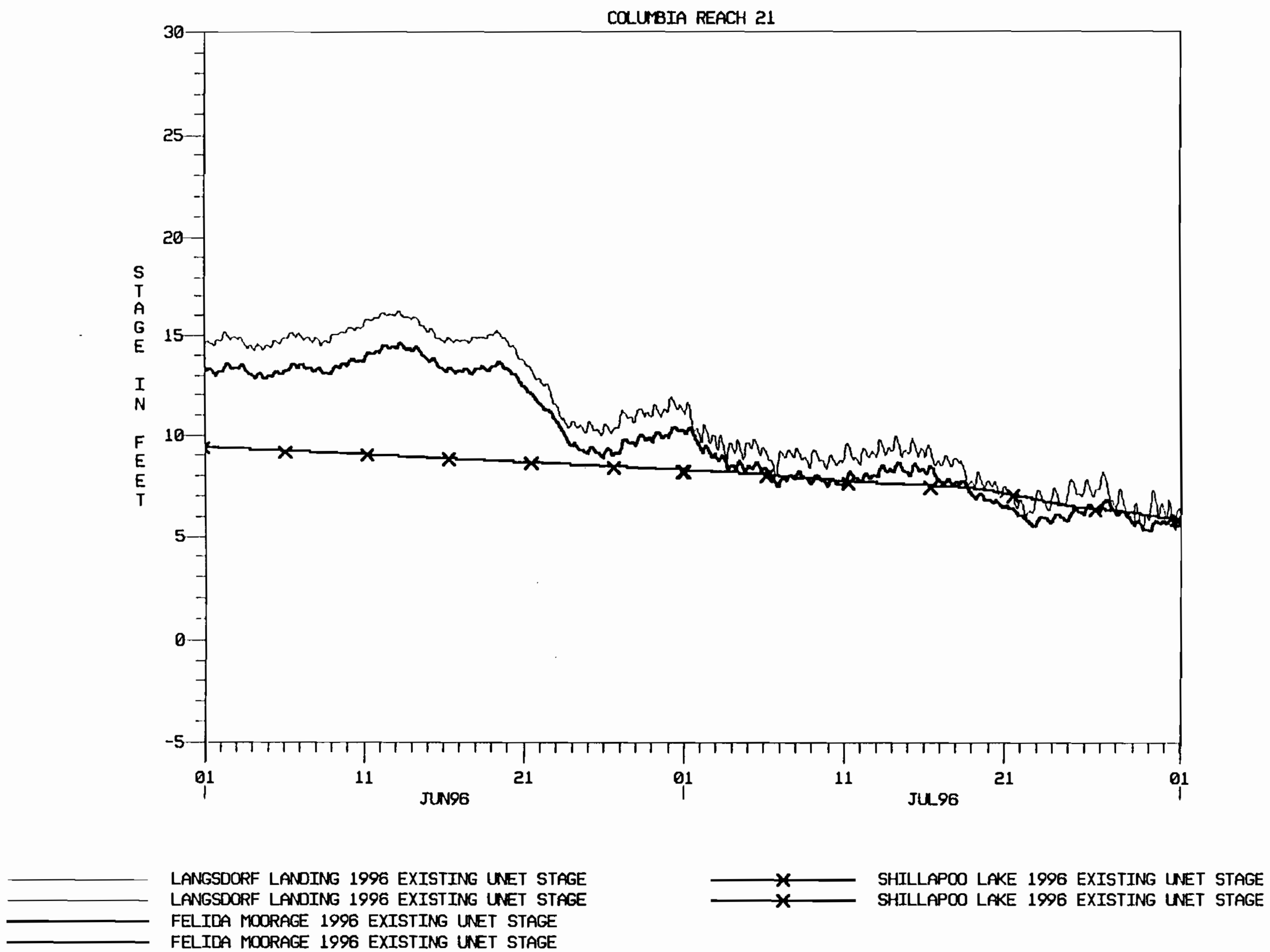
Existing Condition Alternative. Simulation Results February-March 1996

Figure 4.8



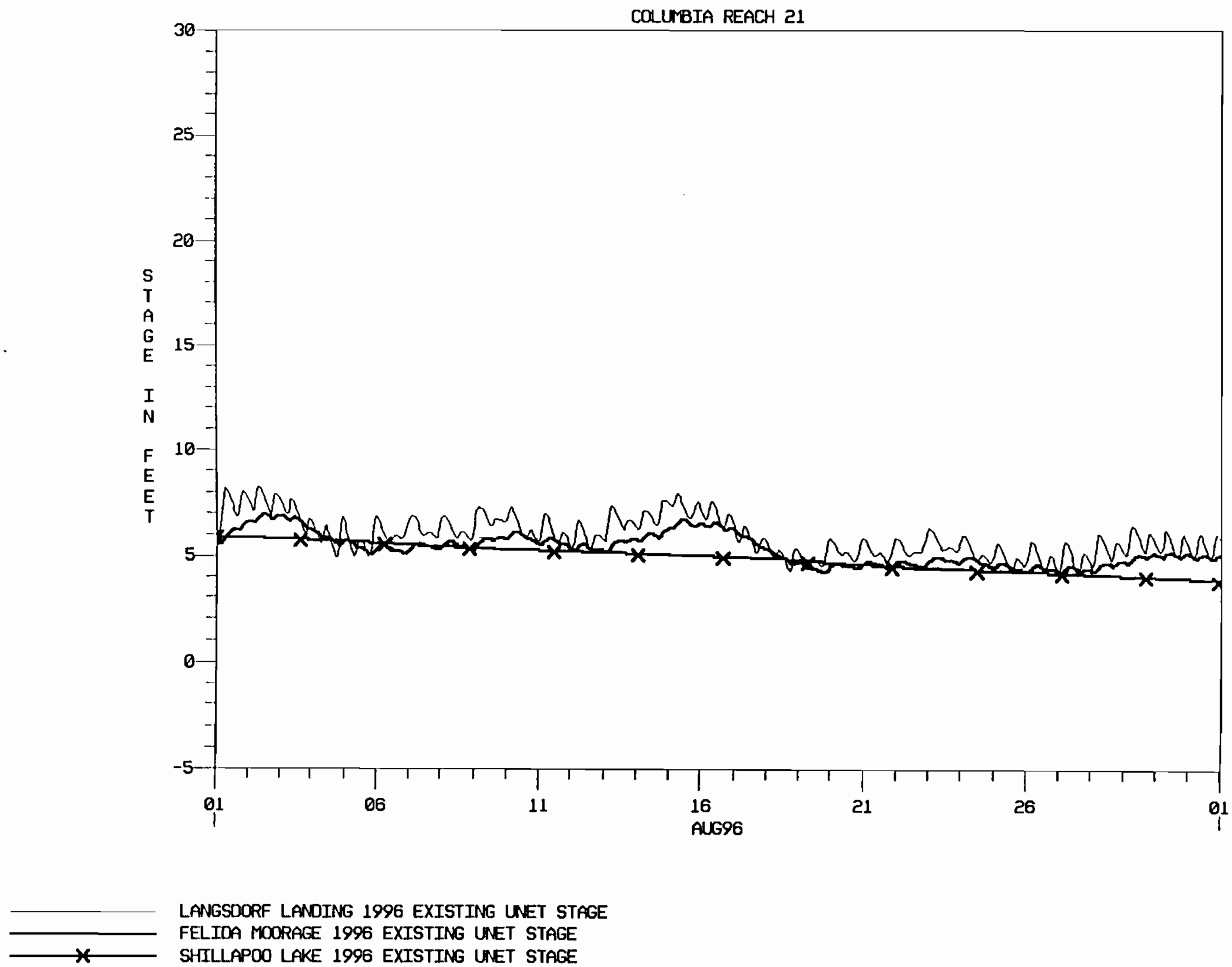
Existing Condition Alternative. Simulation Results April-May 1996

Figure 4.9



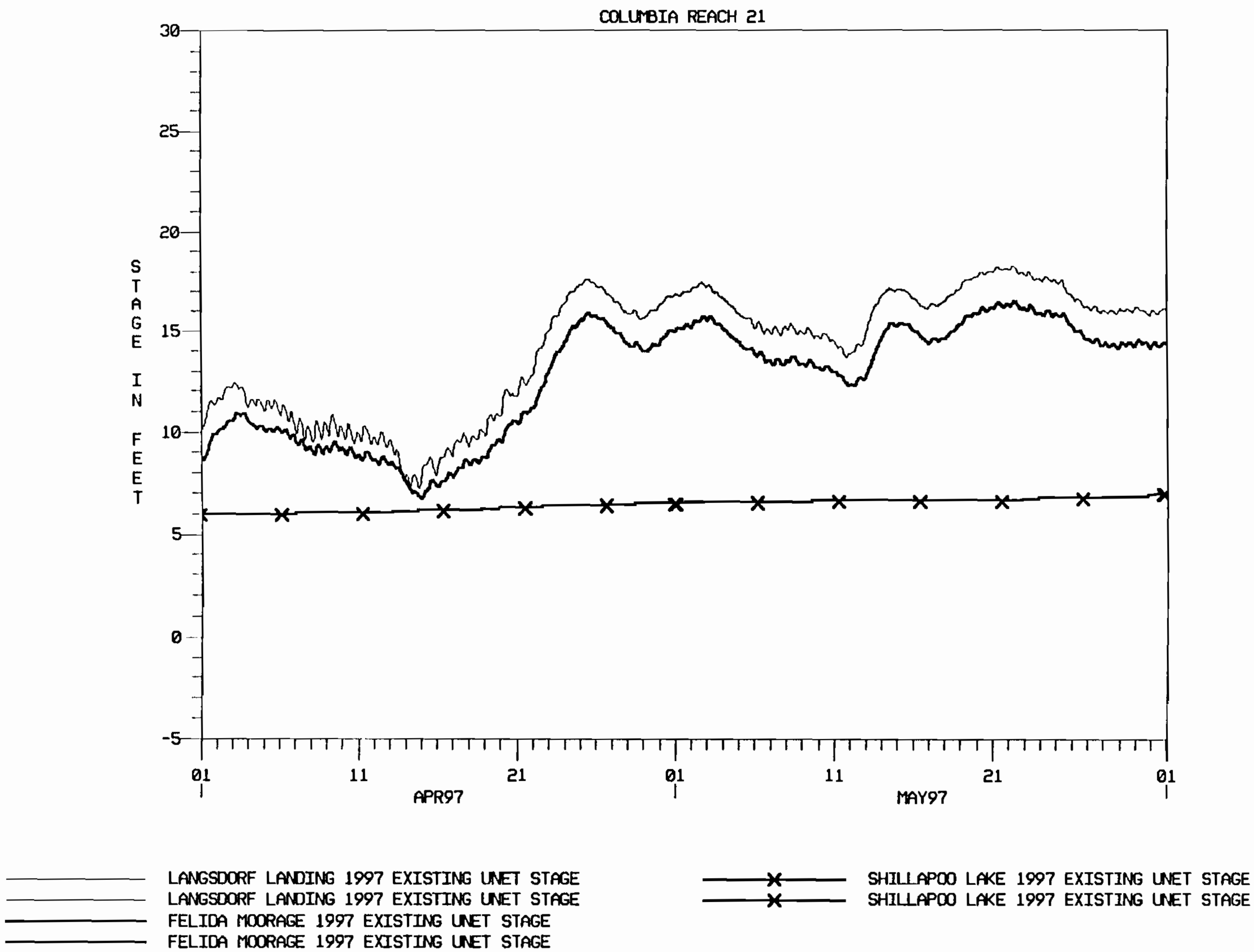
Existing Condition Alternative. Simulation Results June-July 1996

Figure 4.10



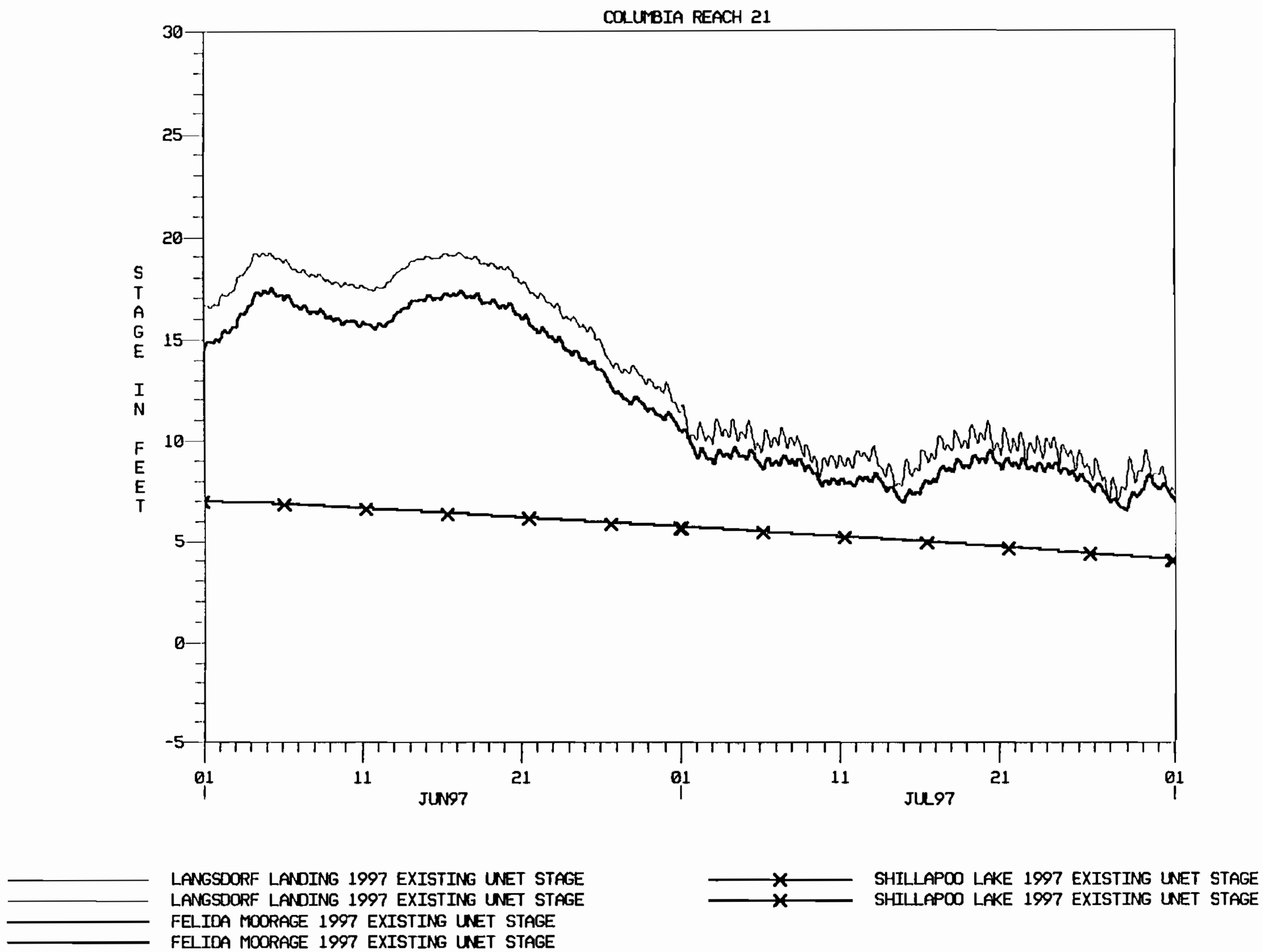
Existing Condition Alternative. Simulation Results August 1996

Figure 4.11



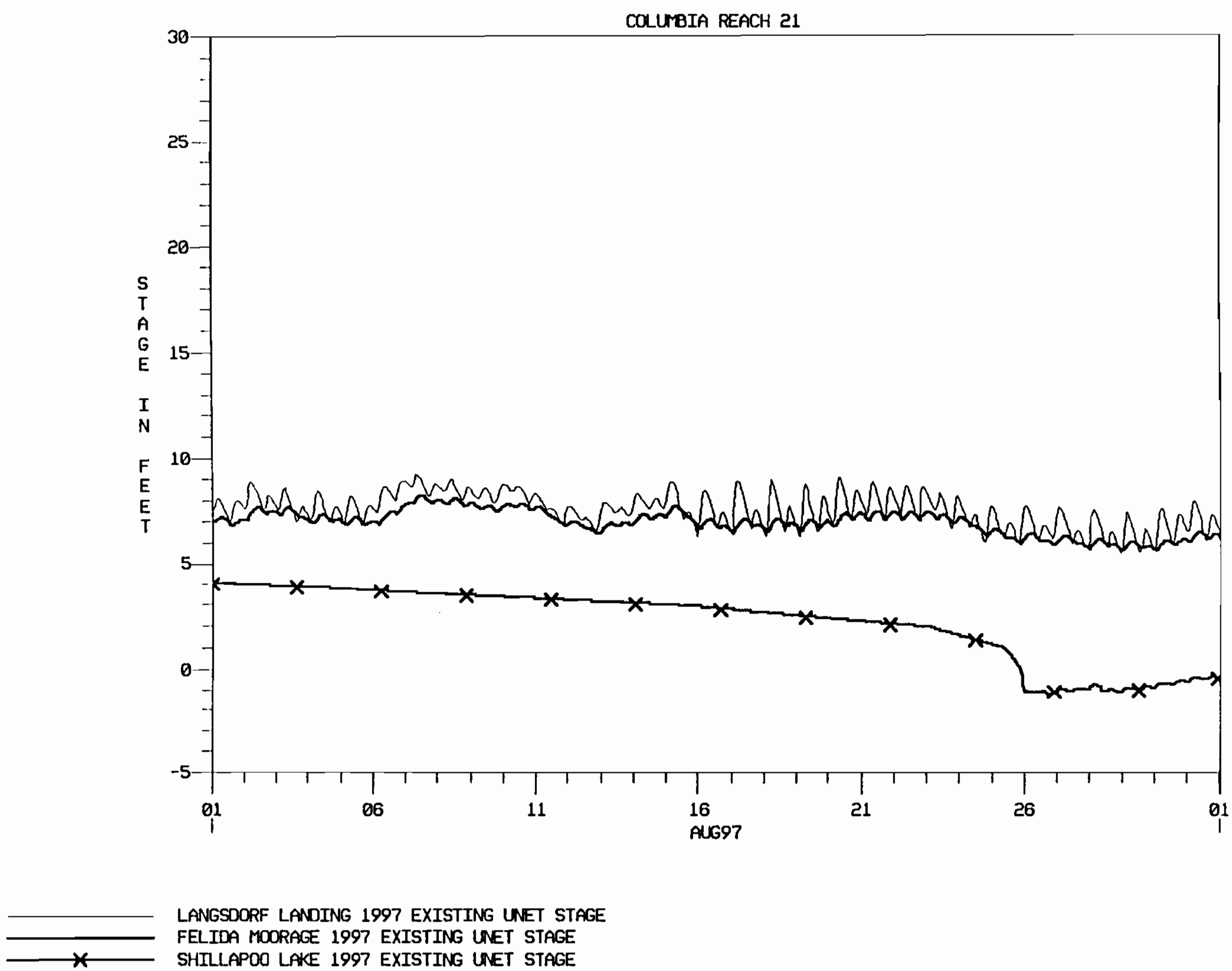
Existing Condition Alternative. Simulation Results April-May 1997

Figure 4.12



Existing Condition Alternative. Simulation Results June-July 1997

Figure 4.13



Existing Condition Alternative. Simulation Results August 1997

Figure 4.14

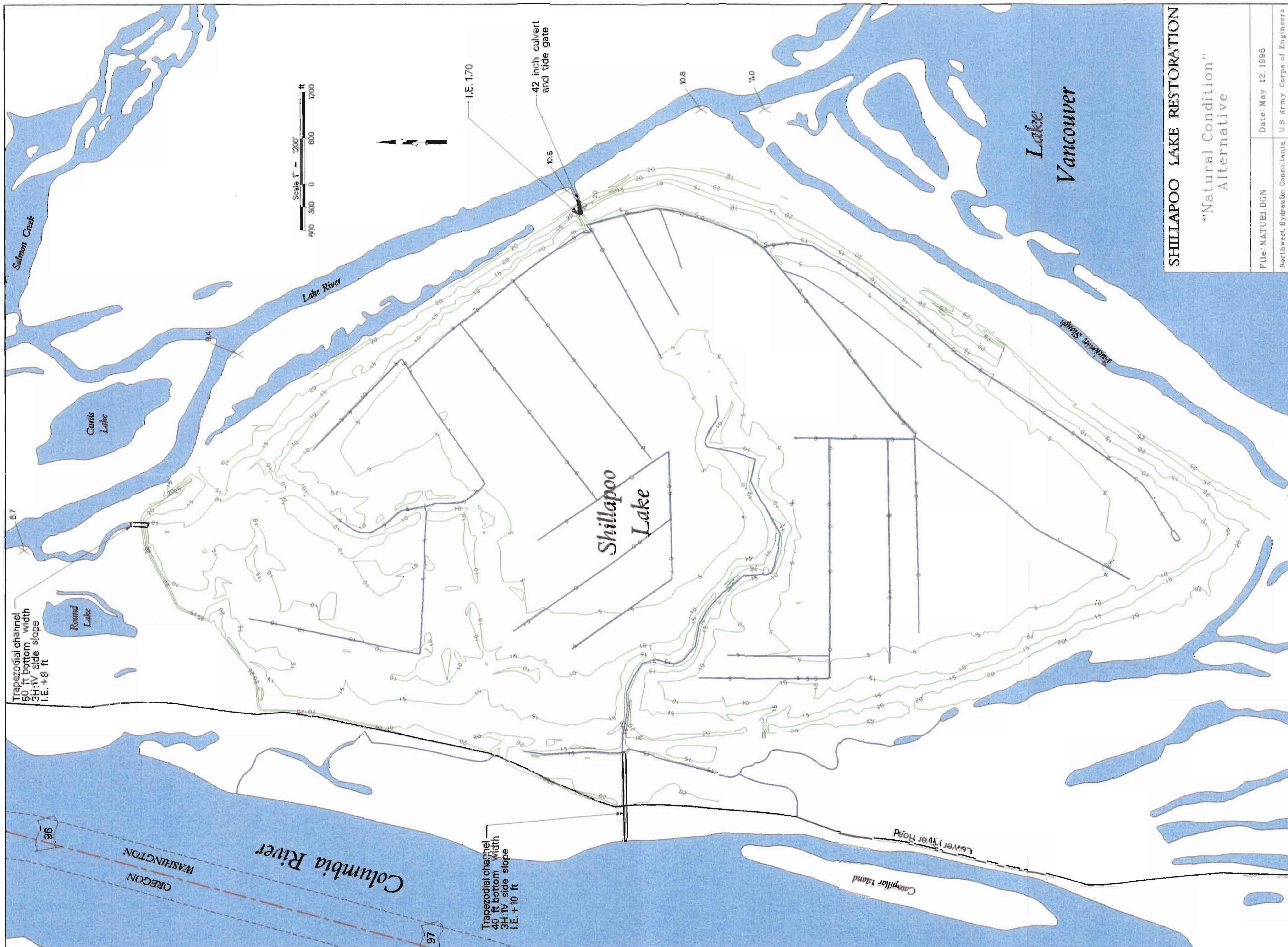
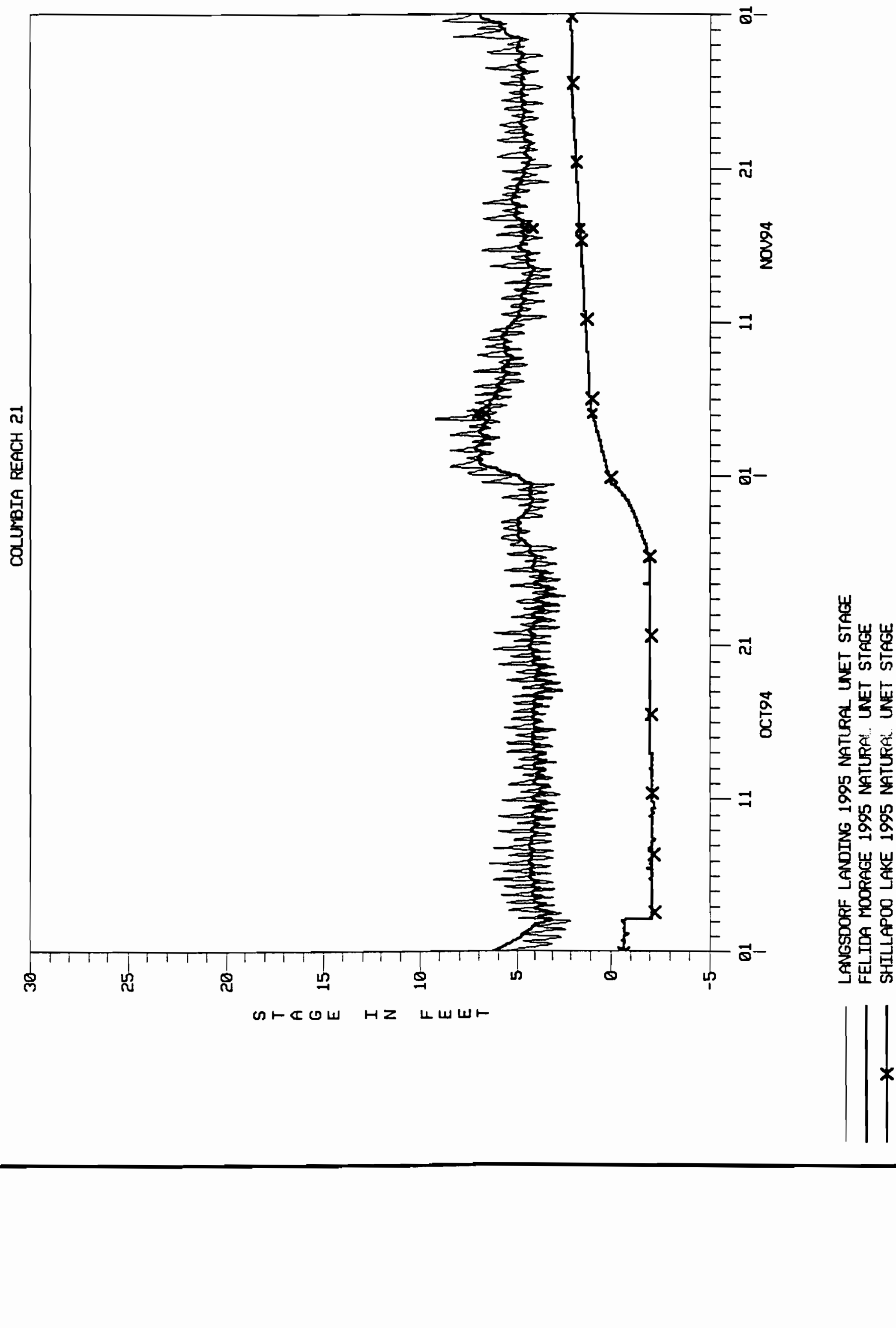
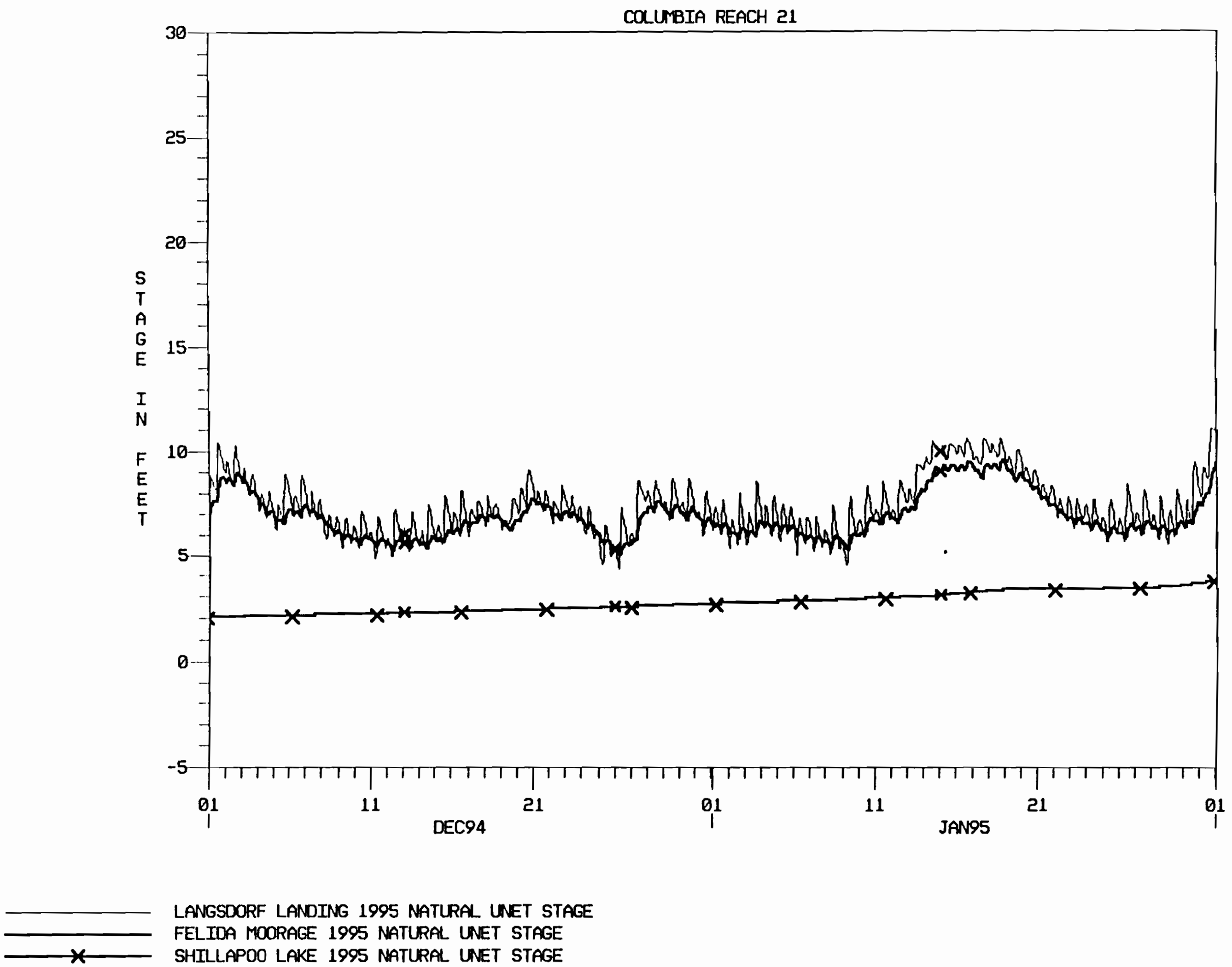

SHILLAPOO LAKE RESTORATION
"Natural Condition" Alternative
File: NATUR1.DGN
Date: May 12, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District
Scale 1" = 1200'
Lake Vancouver
Shillapoo Lake
Lake River
Salmon Creek
Curtis Lake
Round Lake
Columbia River
WASHINGTON
OREGON
Lower River Road
Caterpillar Island
42 inch culvert and tide gate
I.E. 1.70
Trapezoidal channel 50 ft bottom width 3H:1V side slope I.E. +8 ft
Trapezoidal channel 40 ft bottom width 3H:1V side slope I.E. +10 ft

Figure 4.15



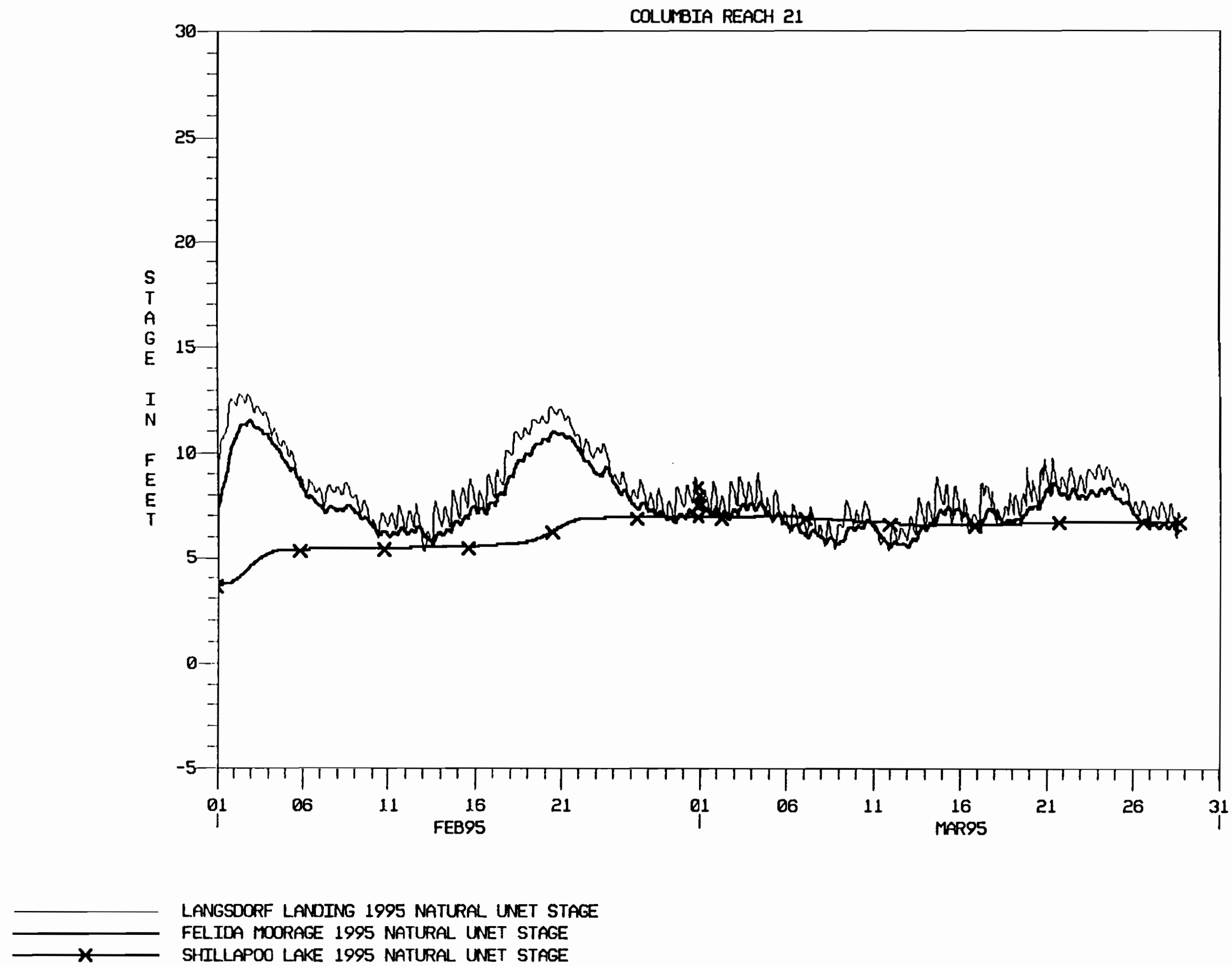
Natural Condition Alternative. Simulation Results October-November 1994

Figure 4.16



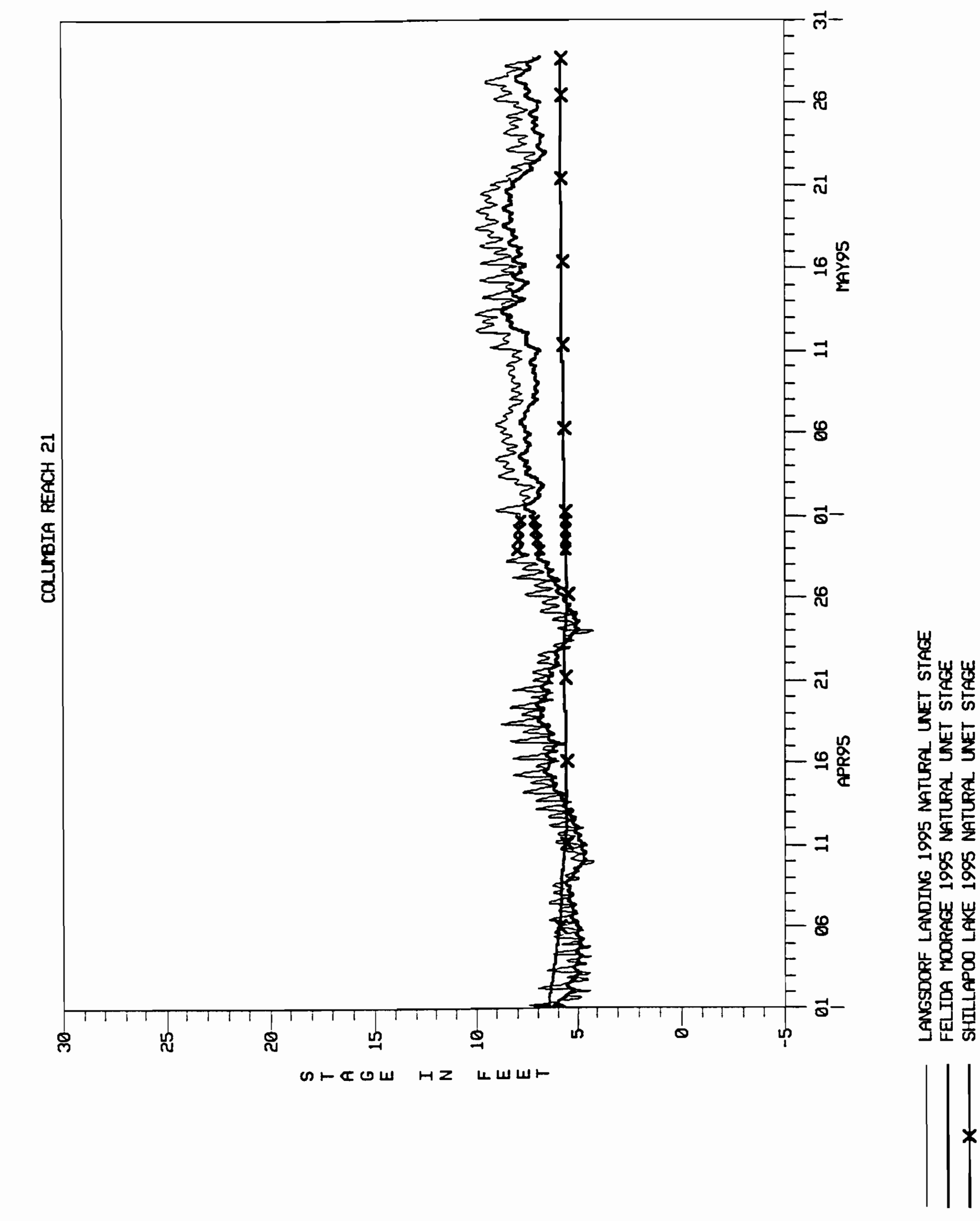
Natural Condition Alternative. Simulation Results December 1994-January 1995

Figure 4.17



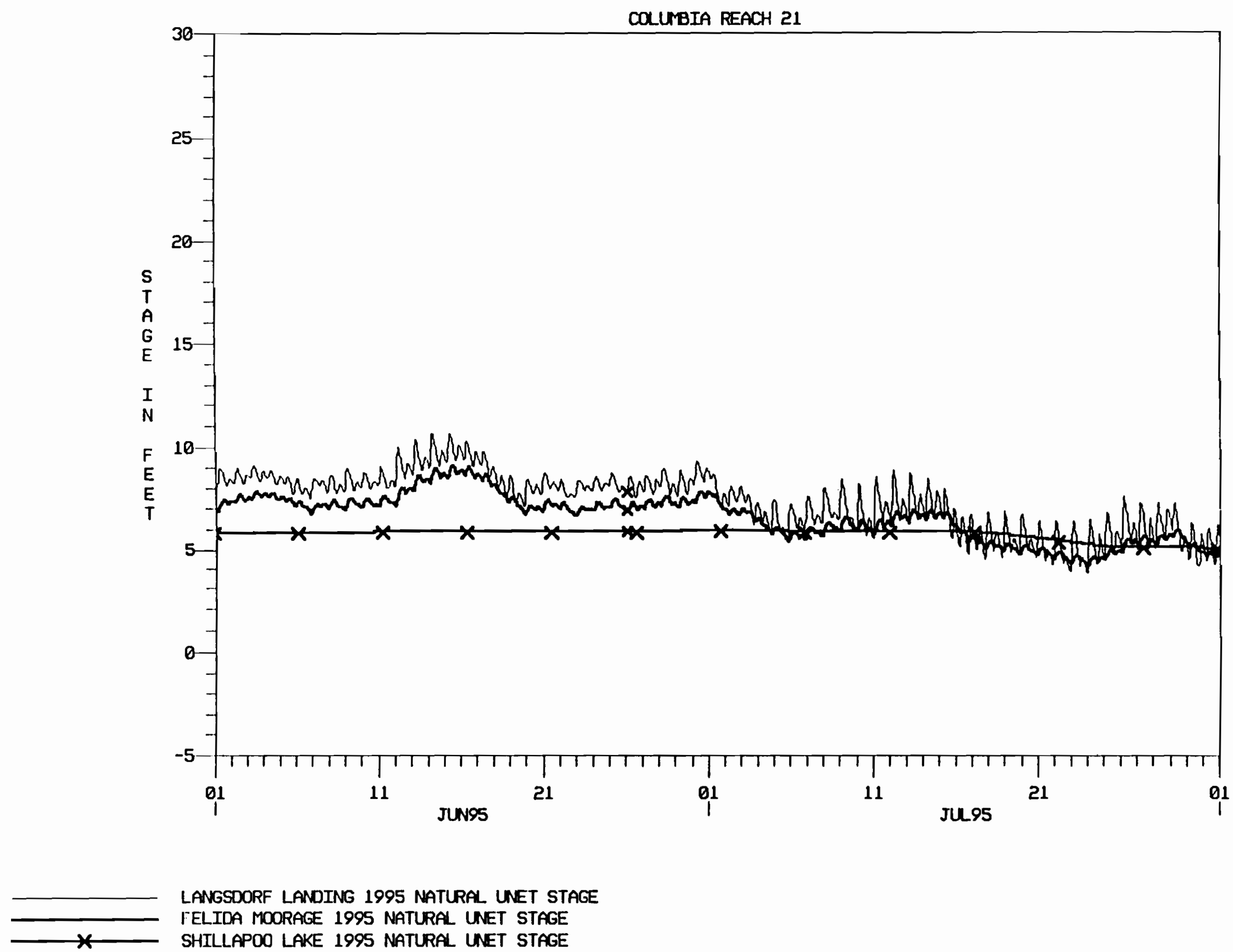
Natural Condition Alternative. Simulation Results February-March 1995

Figure 4.18



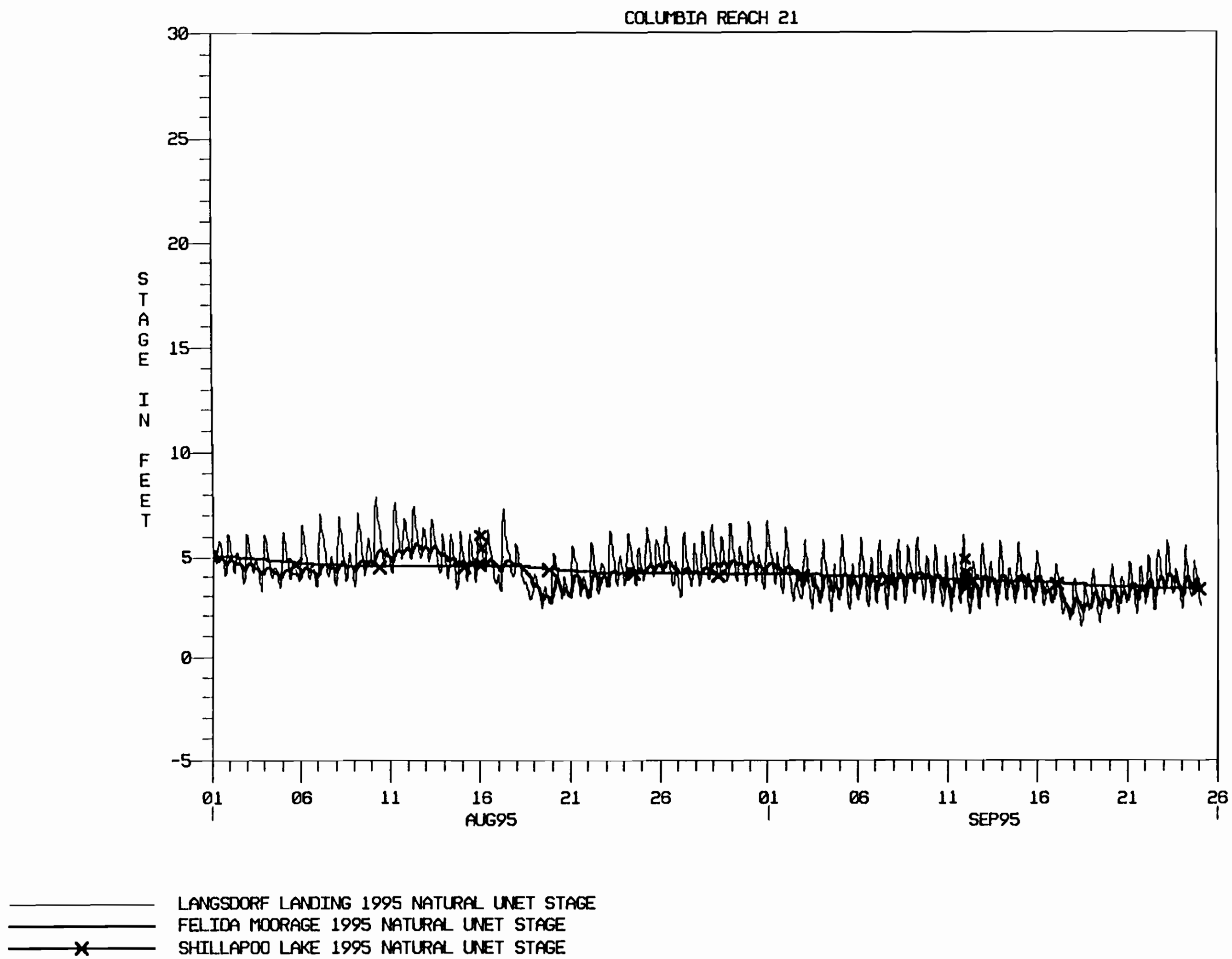
Natural Condition Alternative. Simulation Results April-May 1995

Figure 4.19



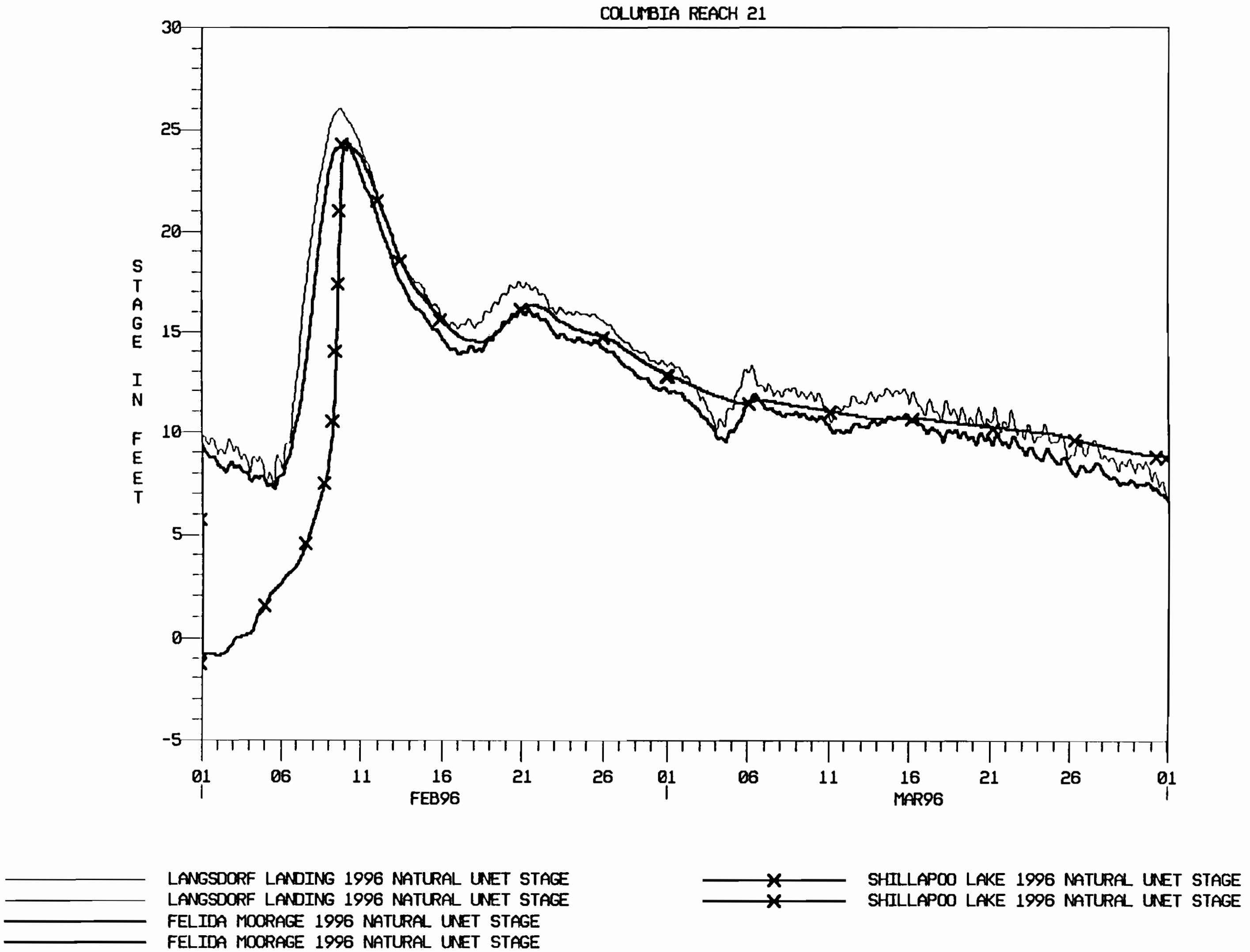
Natural Condition Alternative. Simulation Results June-July 1995

Figure 4.20



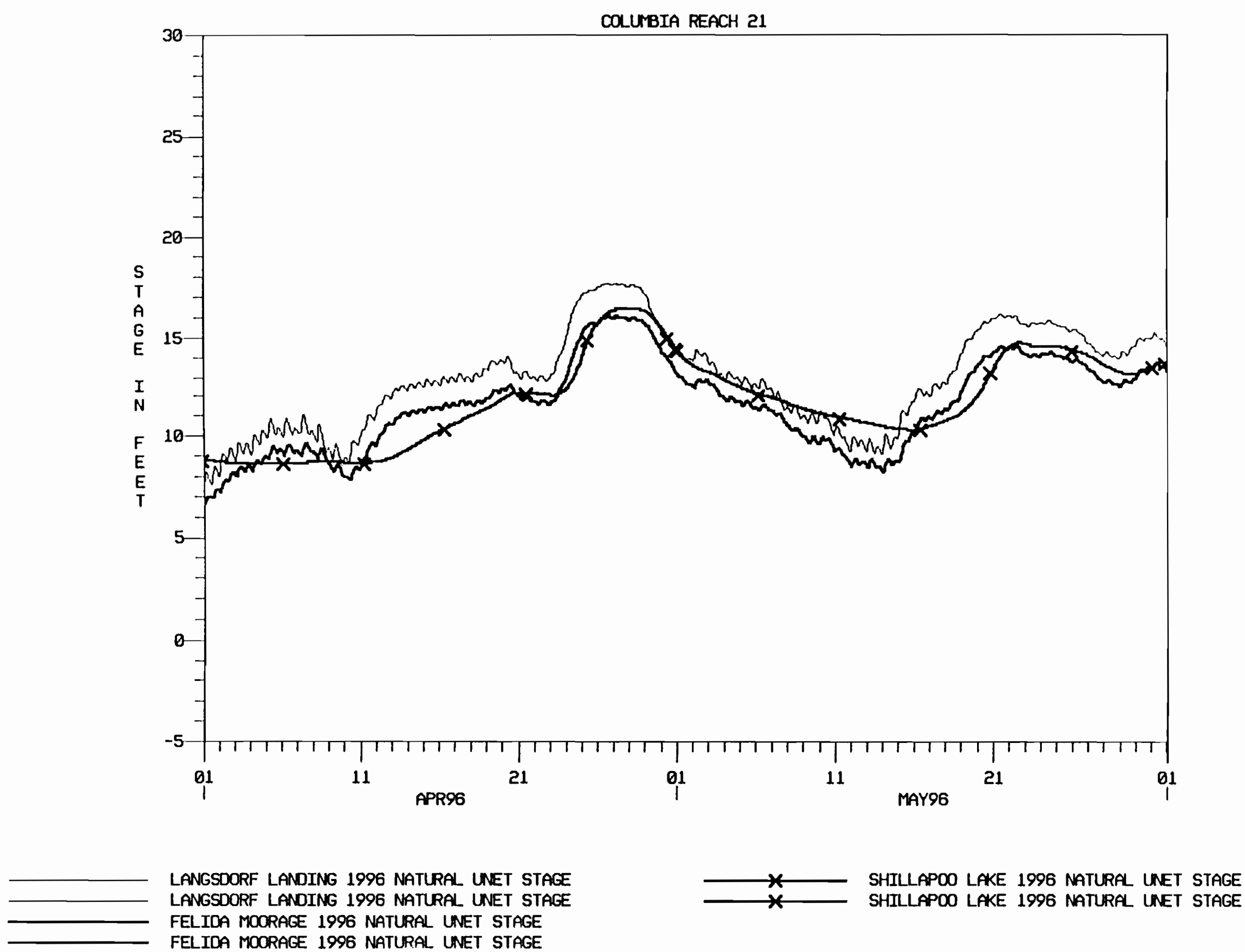
Natural Condition Alternative. Simulation Results August-September 1995

Figure 4.21



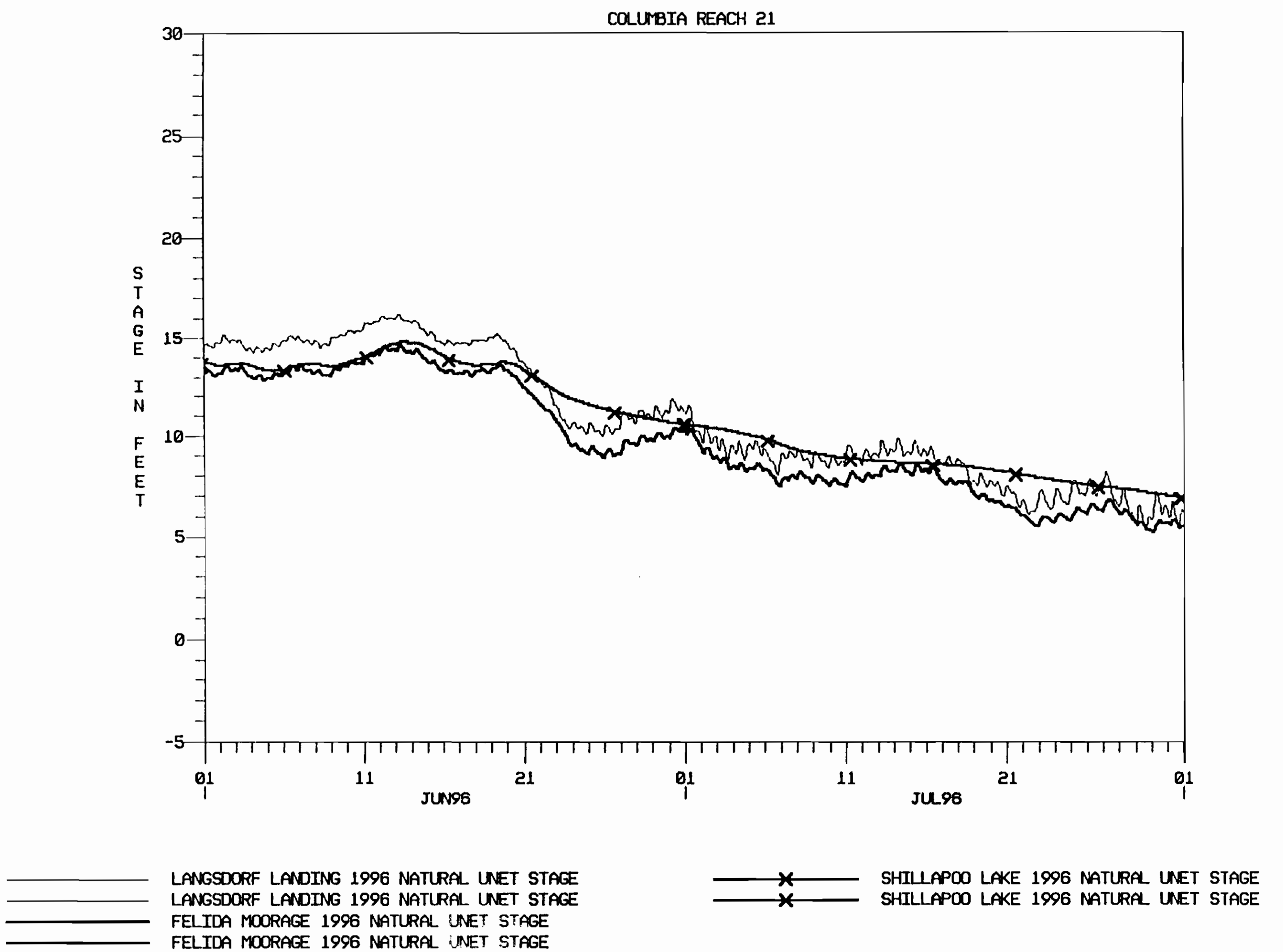
Natural Condition Alternative. Simulation Results February-March 1996

Figure 4.22



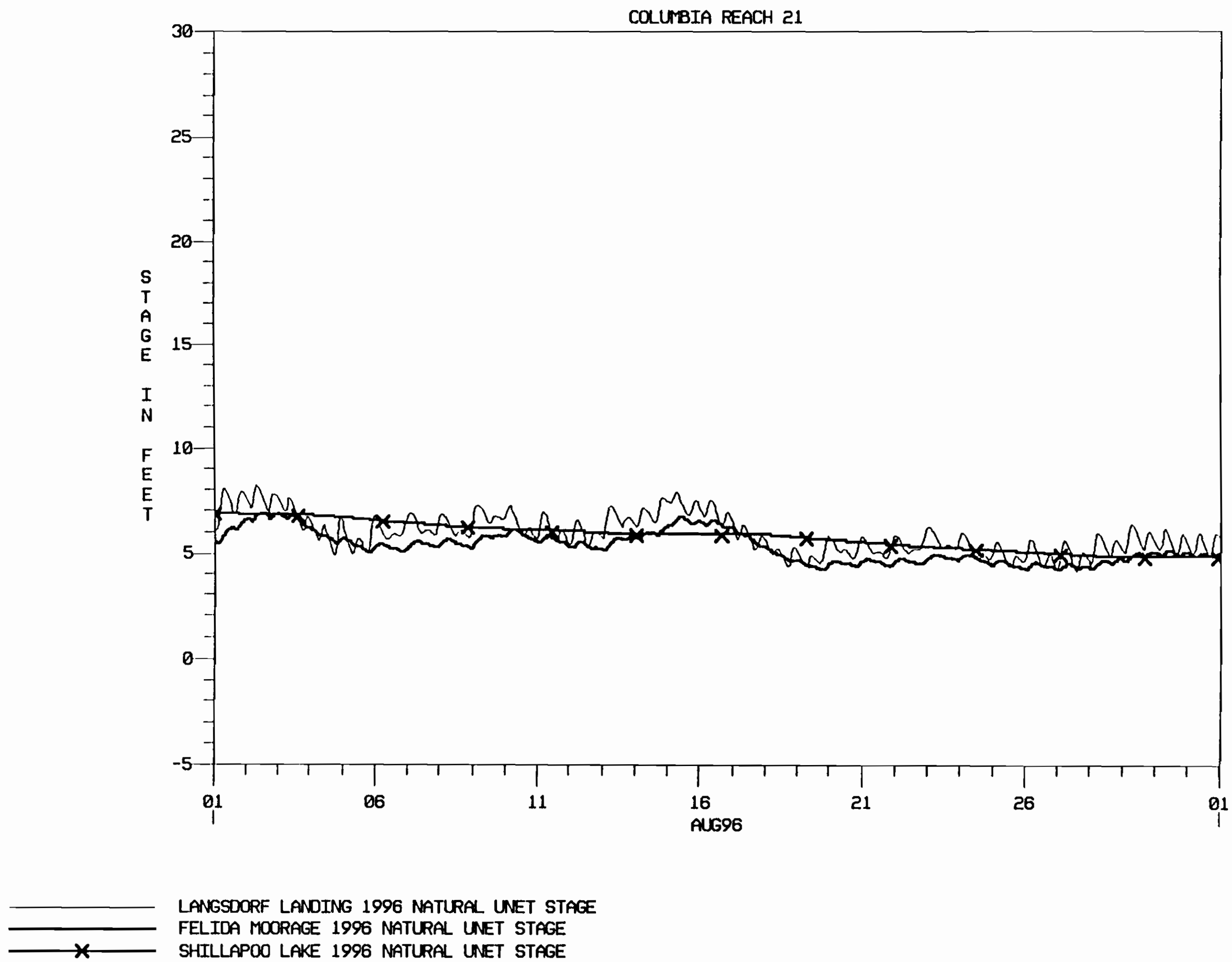
Natural Condition Alternative. Simulation Results April-May 1996

Figure 4.23



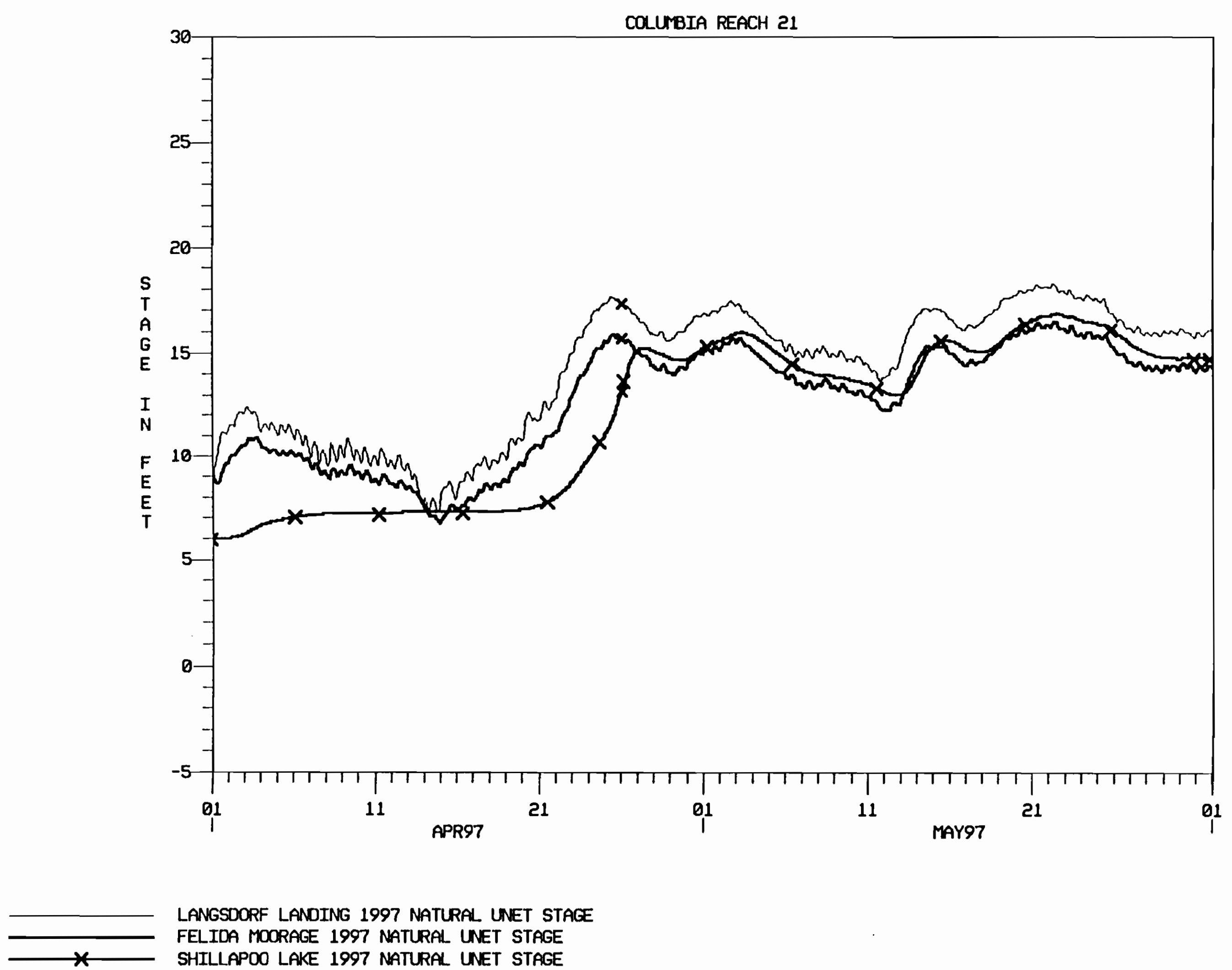
Natural Condition Alternative. Simulation Results June-July 1996

Figure 4.24



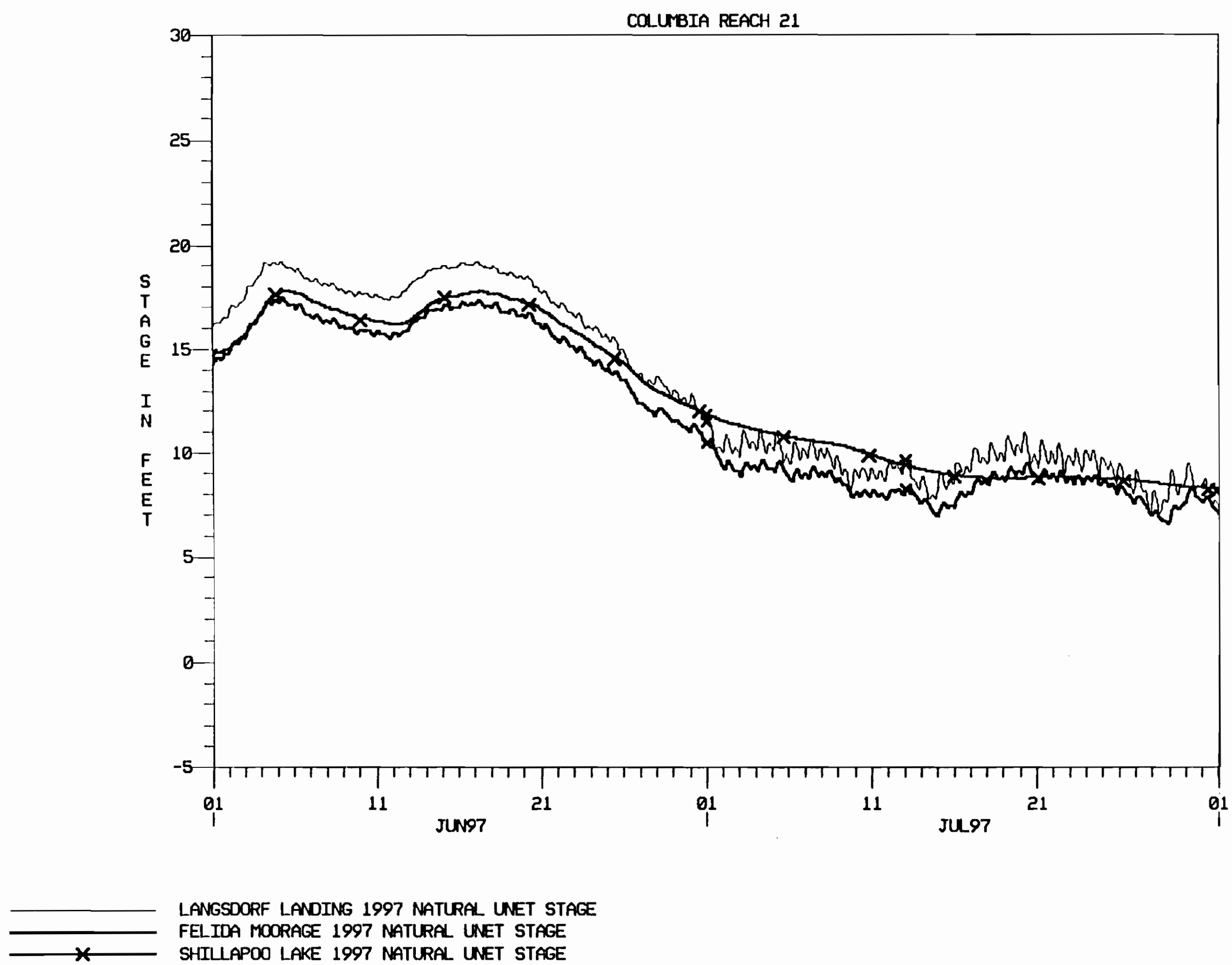
Natural Condition Alternative. Simulation Results August 1996

Figure 4.25



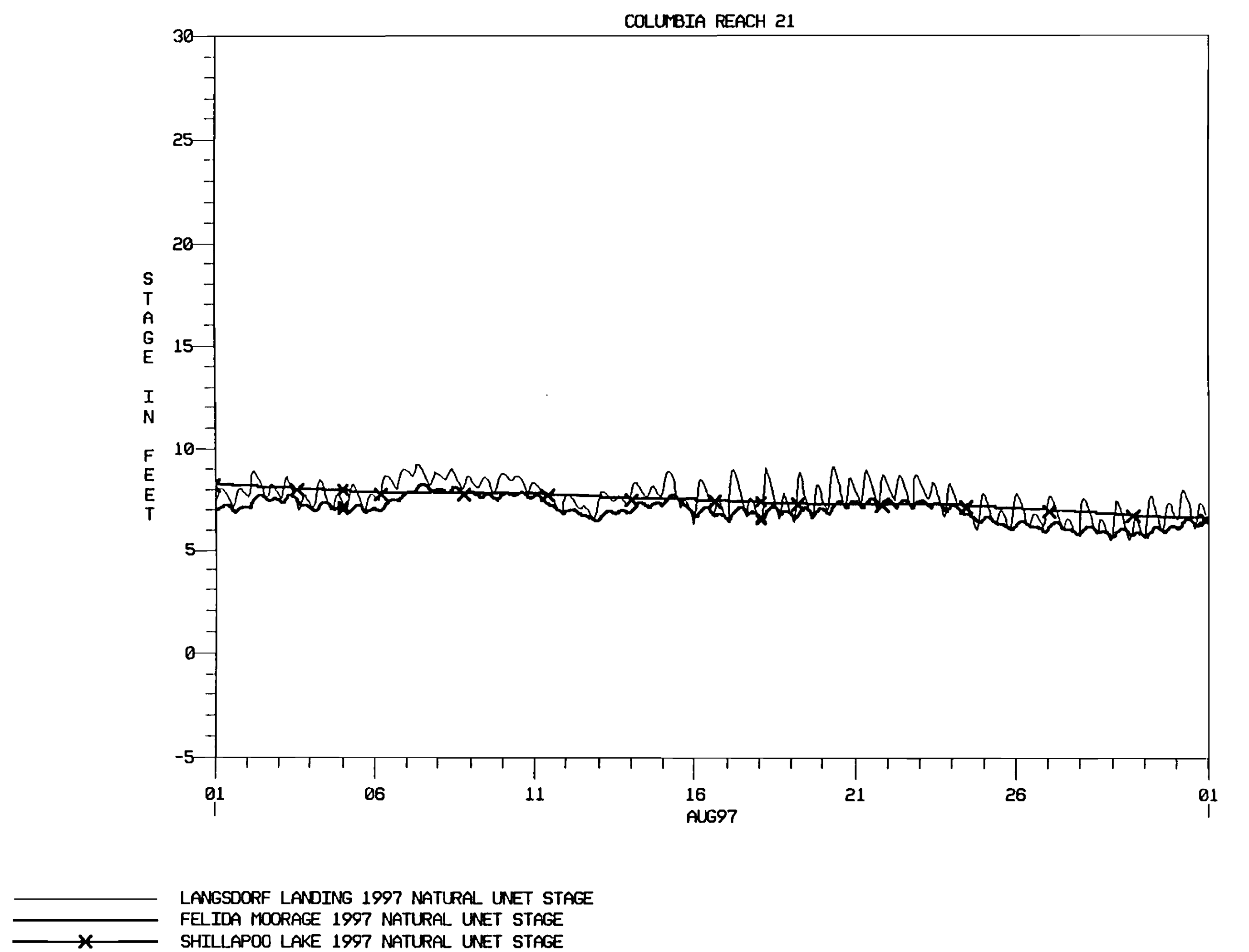
Natural Condition Alternative. Simulation Results April-May 1997

Figure 4.26



Natural Condition Alternative. Simulation Results June-July 1997

Figure 4.27



Natural Condition Alternative. Simulation Results August 1997

Figure 4.28

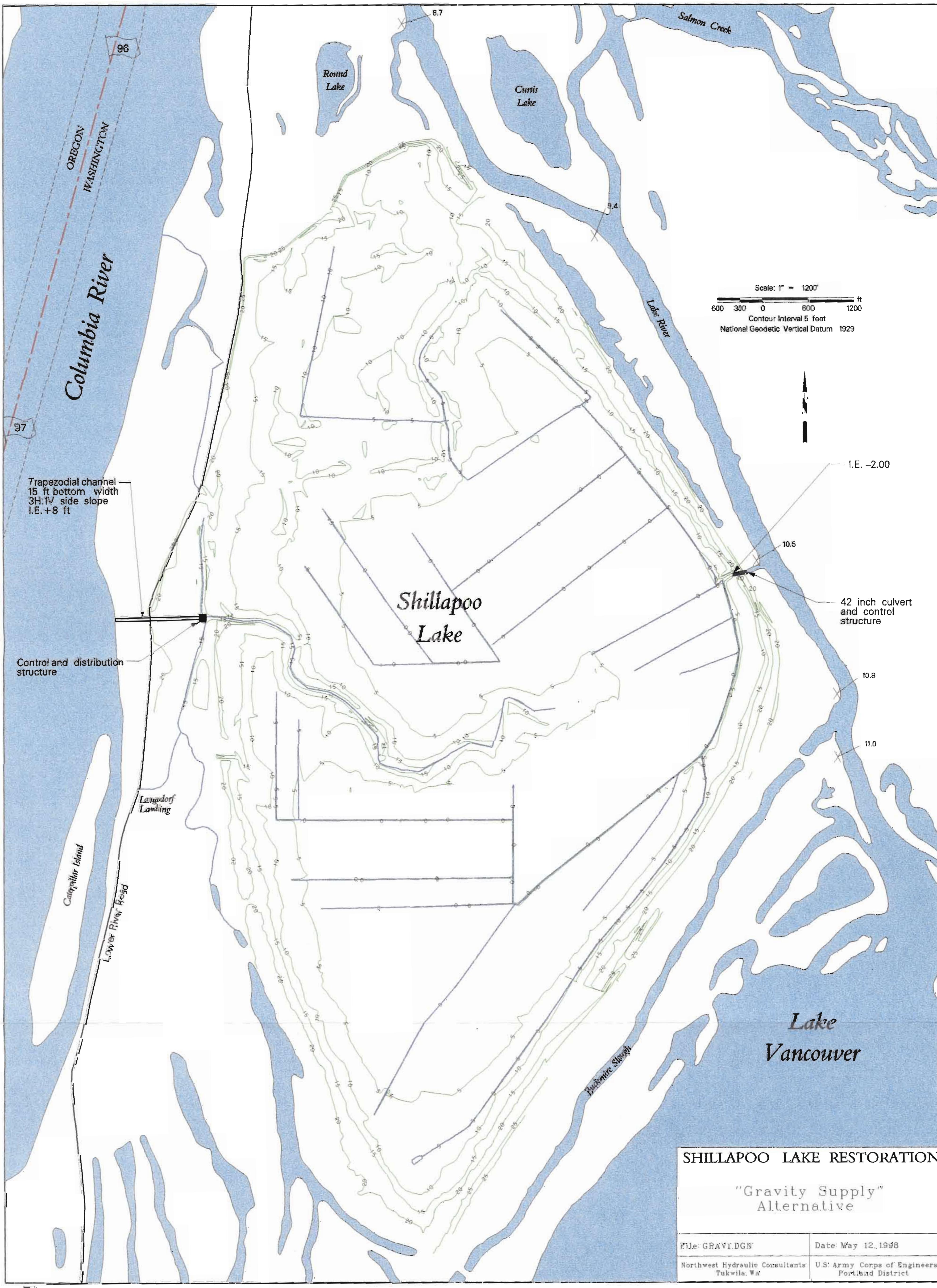
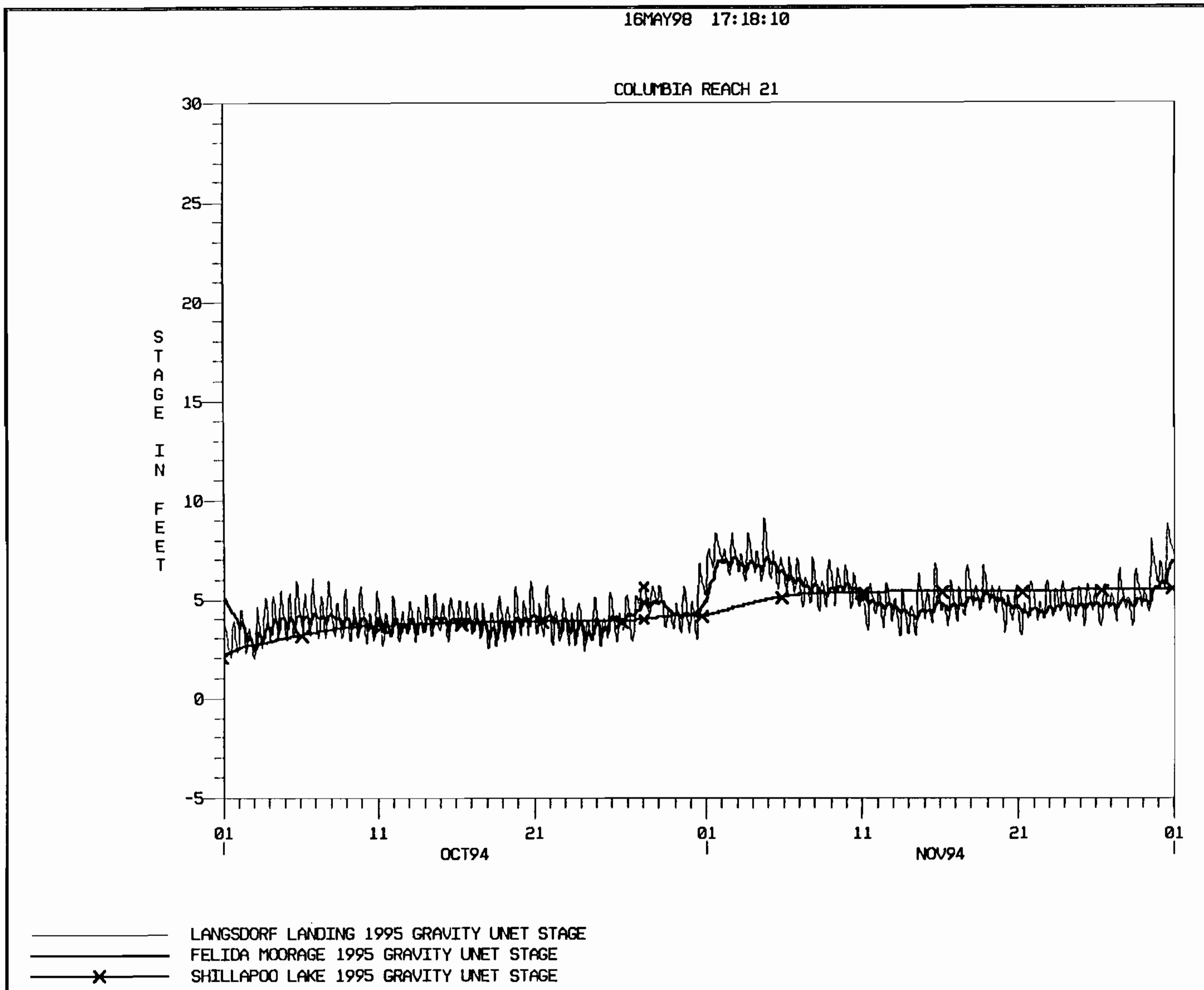
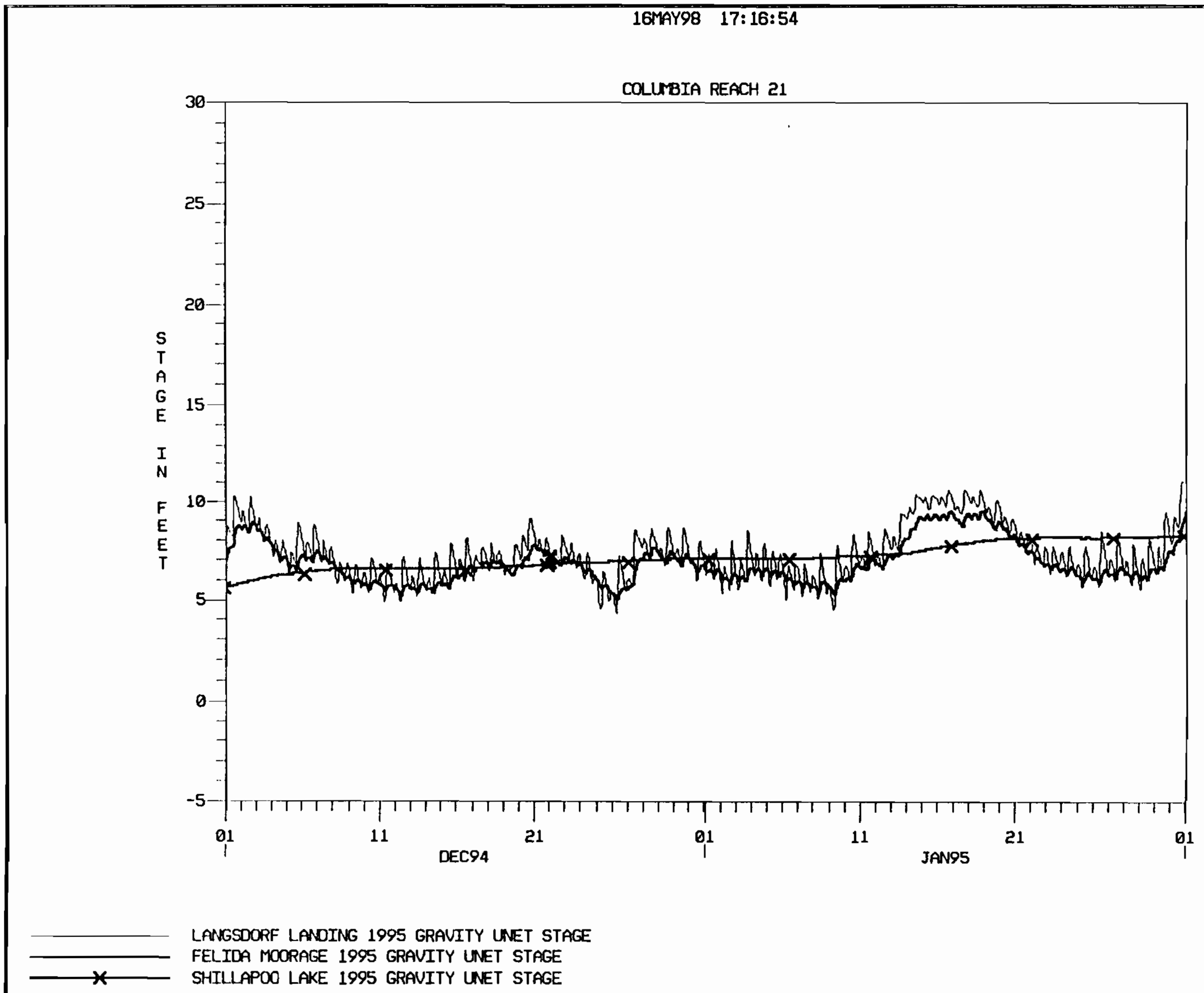

SHILLAPOO LAKE RESTORATION
"Gravity Supply" Alternative
File: GRAVI.DGN
Date: May 12, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District
Scale: 1" = 1200'
600 300 0 600 1200 ft
Contour Interval 5 feet
National Geodetic Vertical Datum 1929
Columbia River
OREGON
WASHINGTON
96
97
Round Lake
Curtis Lake
Salmon Creek
Lake River
Shillapoo Lake
Lake Vancouver
Buckmire Slough
Caterpillar Island
Lower River Road
Langsdorf Landing
8.7
9.4
10.5
10.8
11.0
I.E. -2.00
42 inch culvert and control structure
Trapezodial channel 15 ft bottom width 3H:1V side slope I.E. +8 ft
Control and distribution structure

Figure 4.29



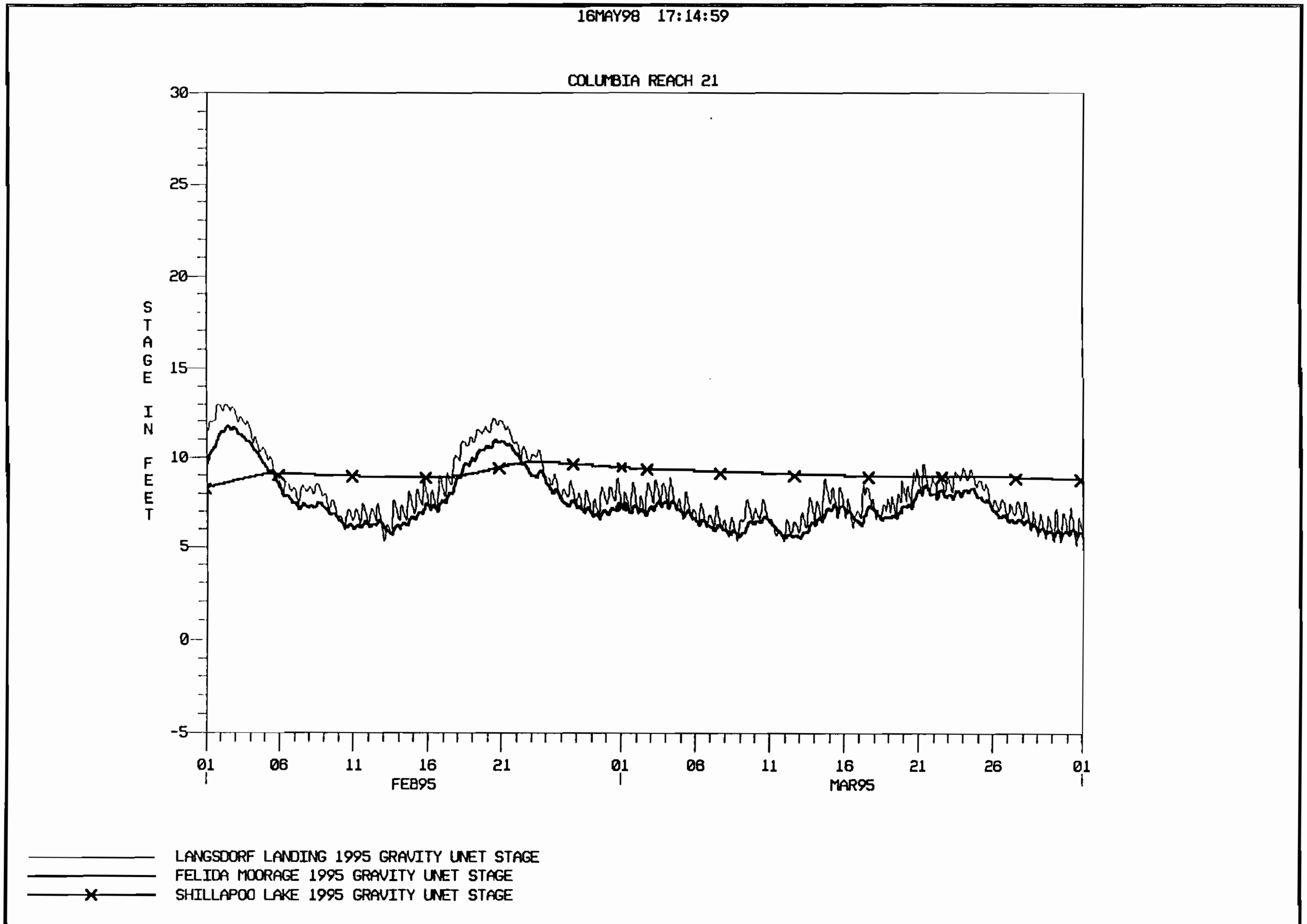
Gravity Supply Alternative. Simulation Results October-November 1994

Figure 4.30



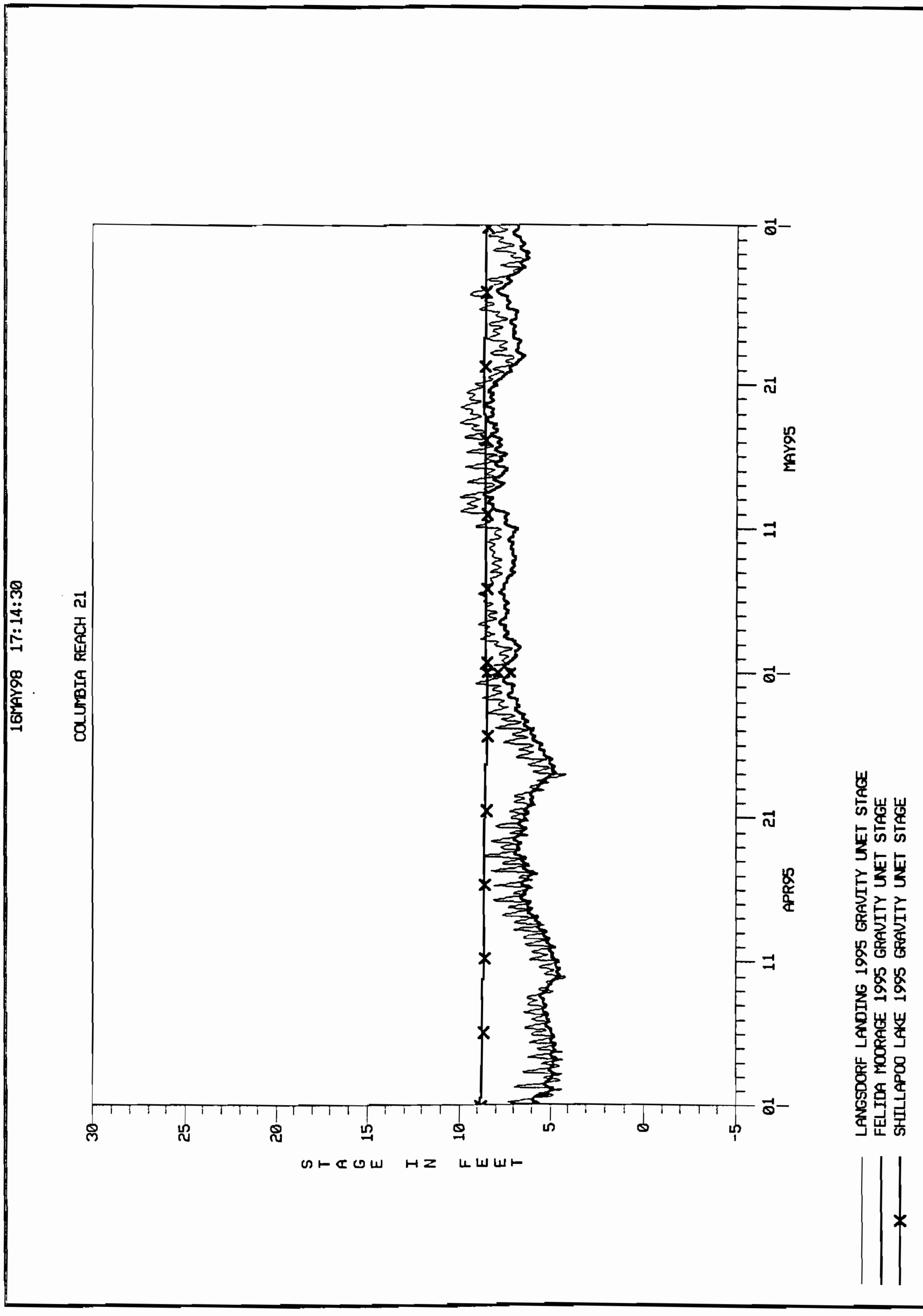
Gravity Supply Alternative. Simulation Results December 1994-January 1995

Figure 4.31



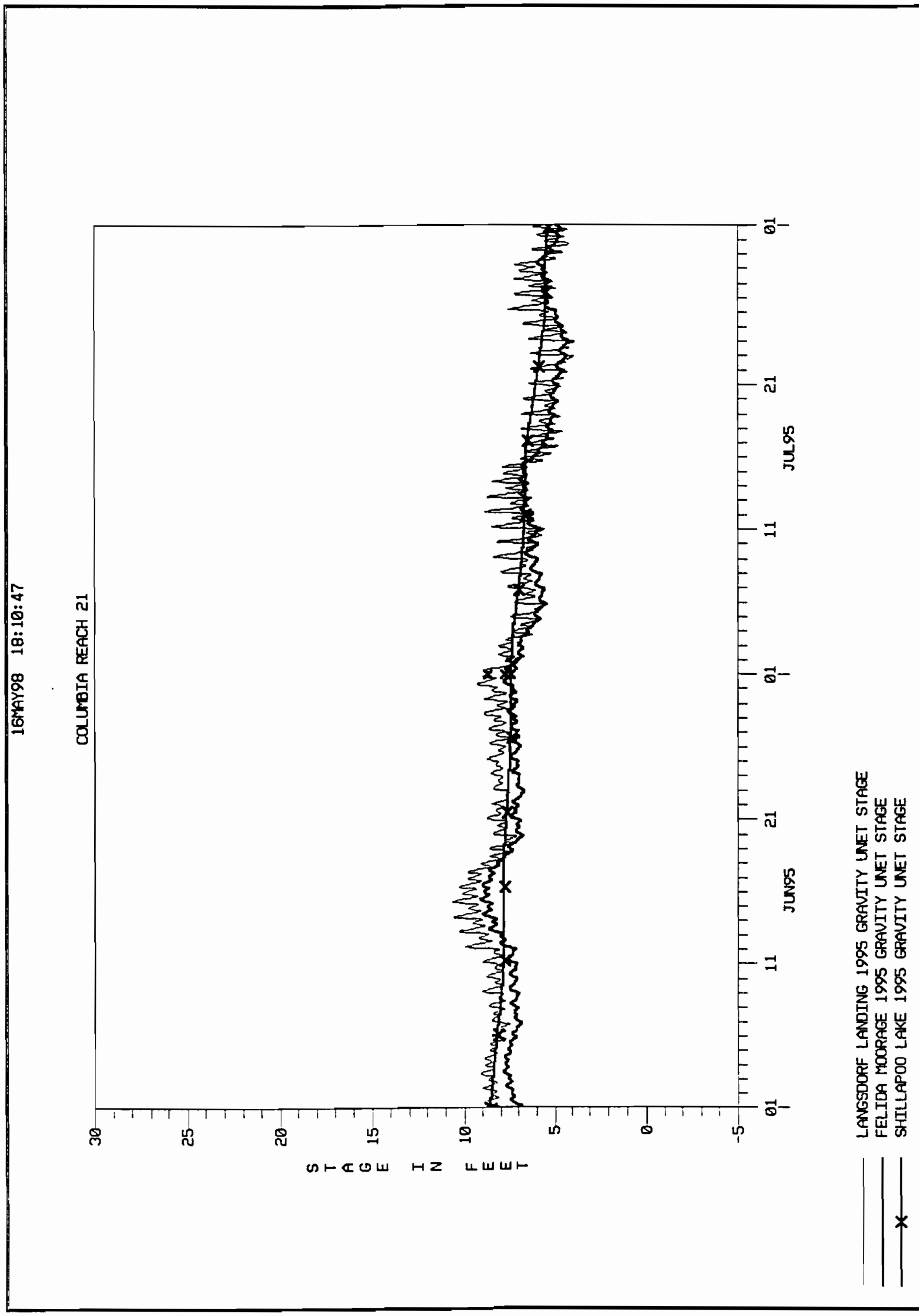
Gravity Supply Alternative. Simulation Results February-March 1995

Figure 4.32



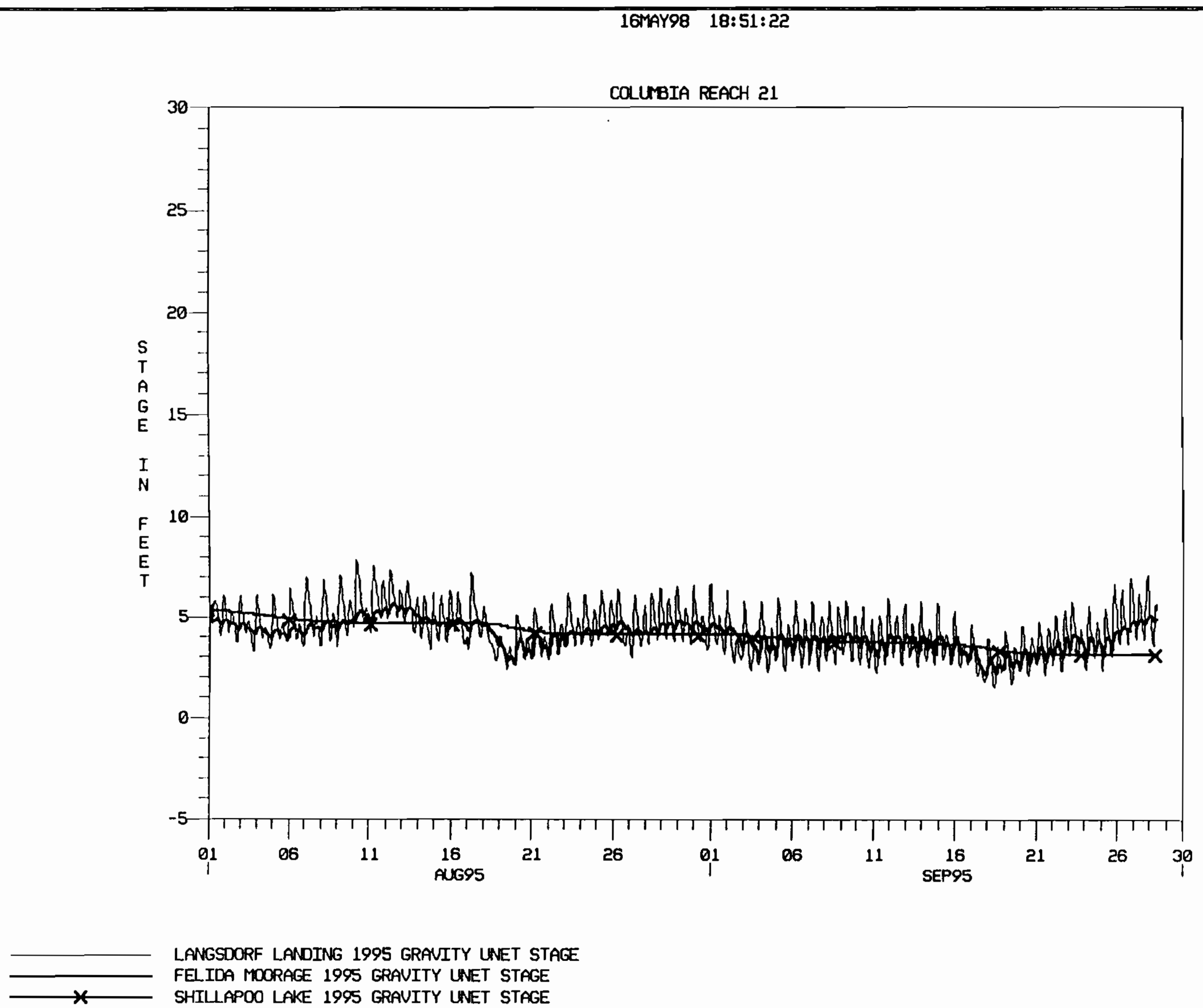
Gravity Supply Alternative. Simulation Results April-May 1995

Figure 4.33



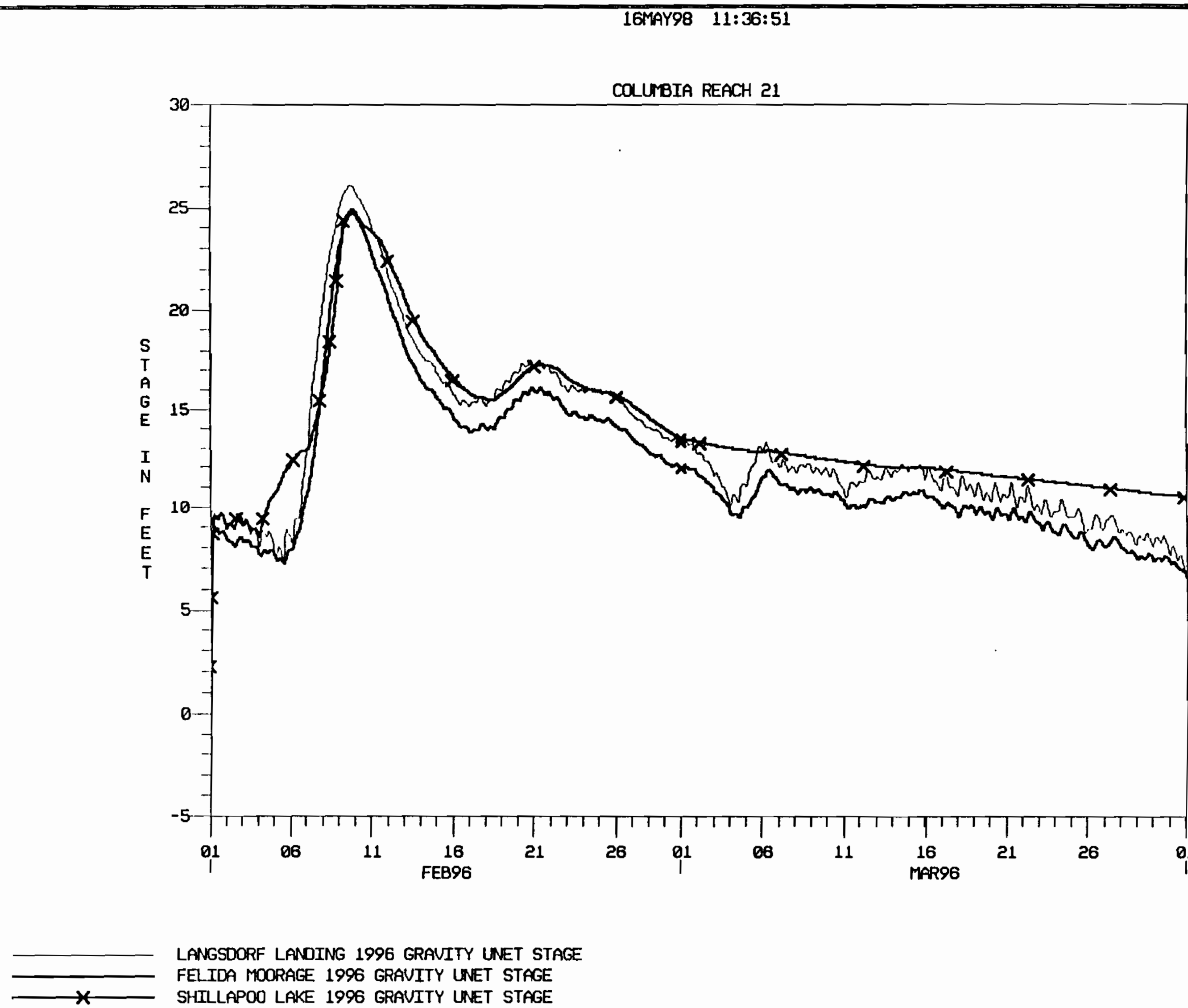
Gravity Supply Alternative. Simulation Results June-July 1995

Figure 4.34



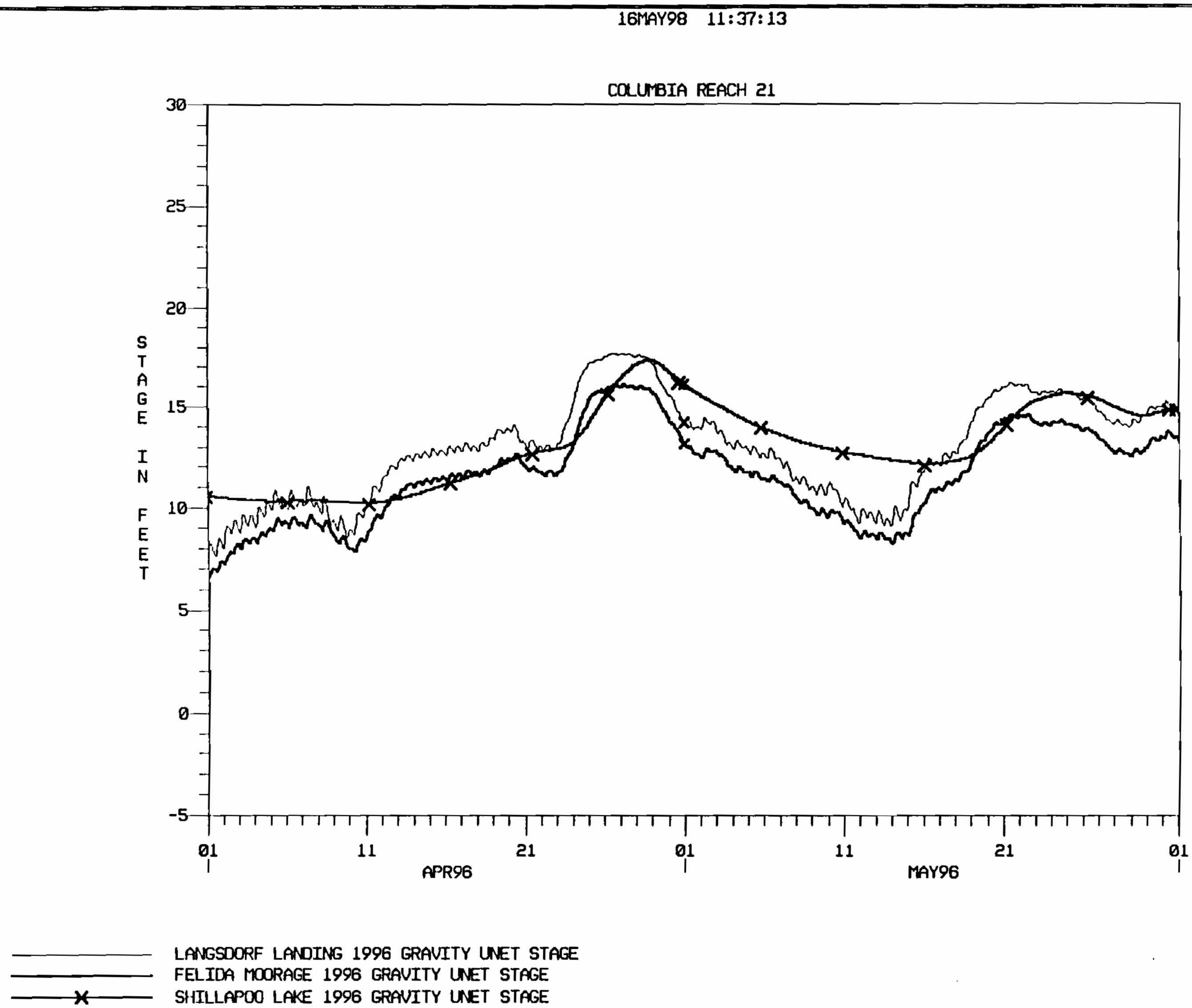
Gravity Supply Alternative. Simulation Results August-September 1995

Figure 4.35



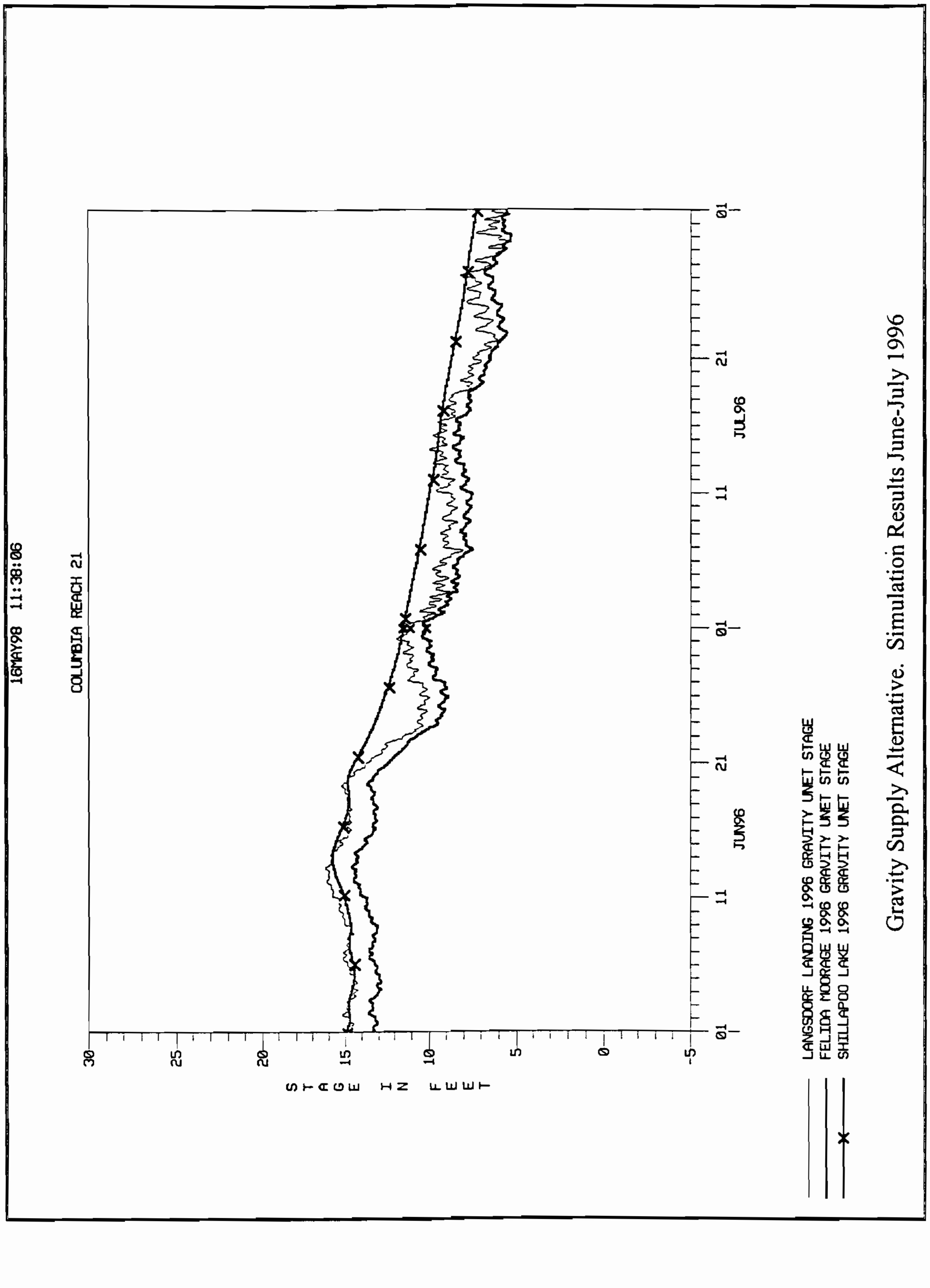
Gravity Supply Alternative. Simulation Results February-March 1996

Figure 4.36



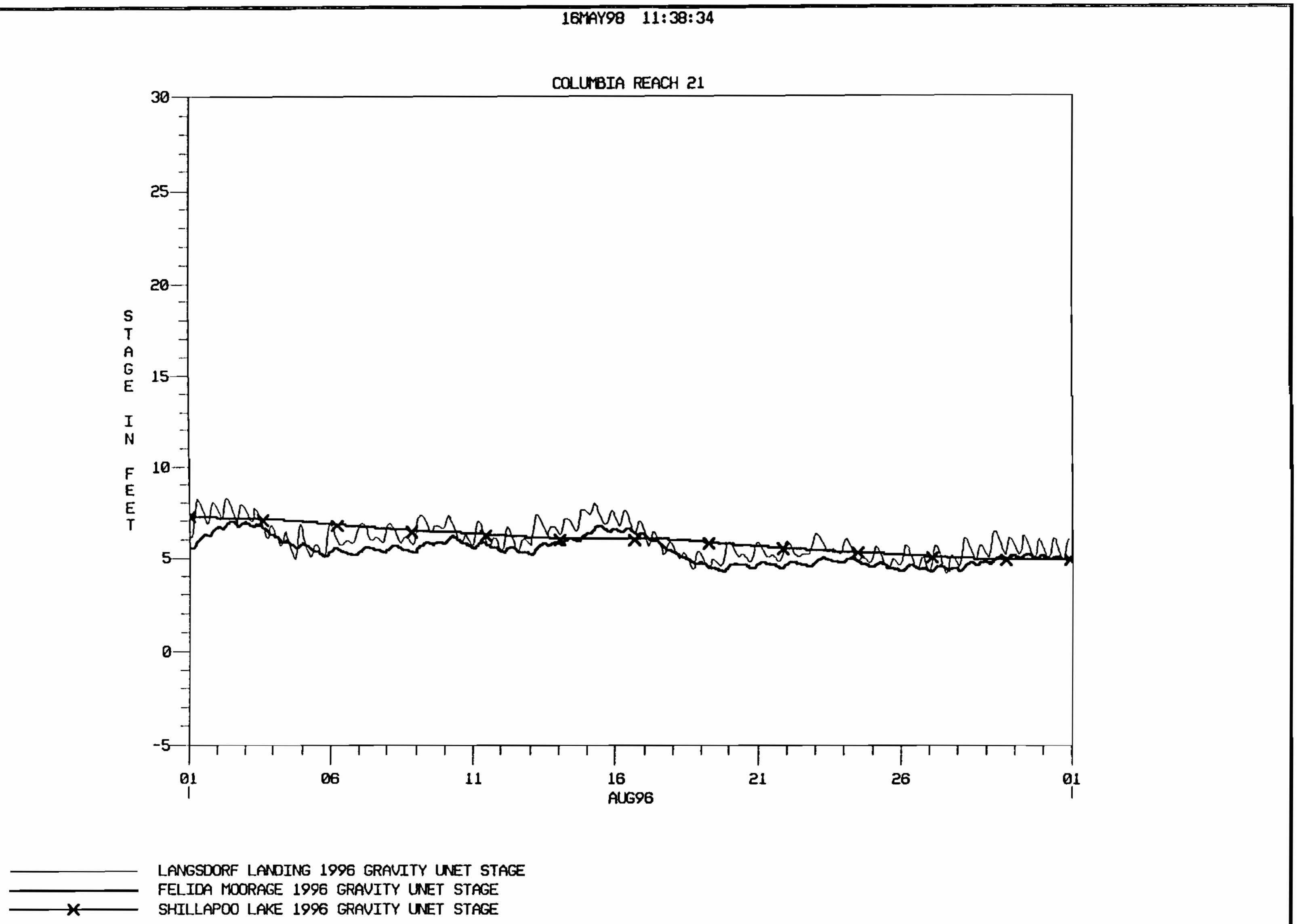
Gravity Supply Alternative. Simulation Results April-May 1996

Figure 4.37



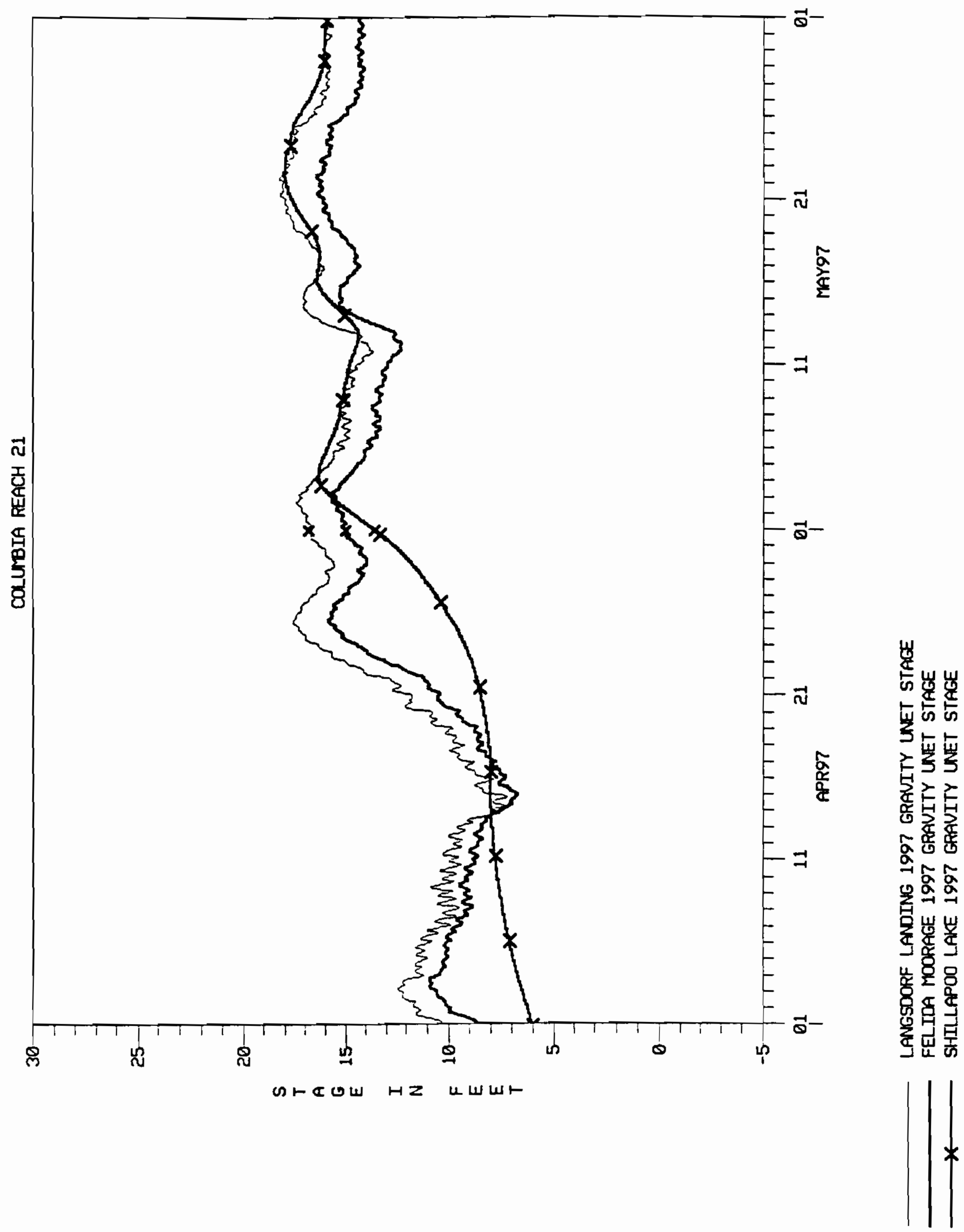
Gravity Supply Alternative. Simulation Results June-July 1996

Figure 4.38



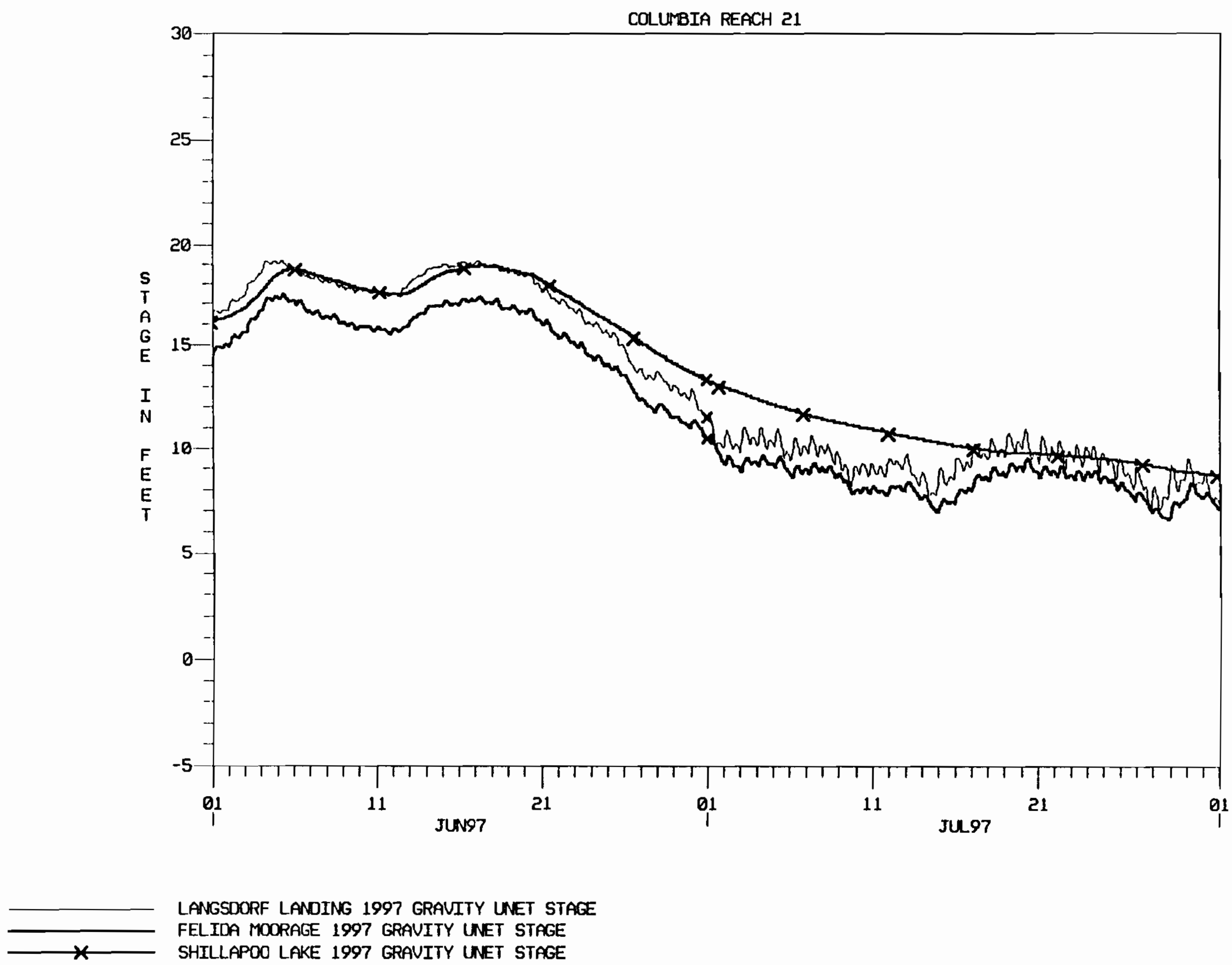
Gravity Supply Alternative. Simulation Results August 1996

Figure 4.39



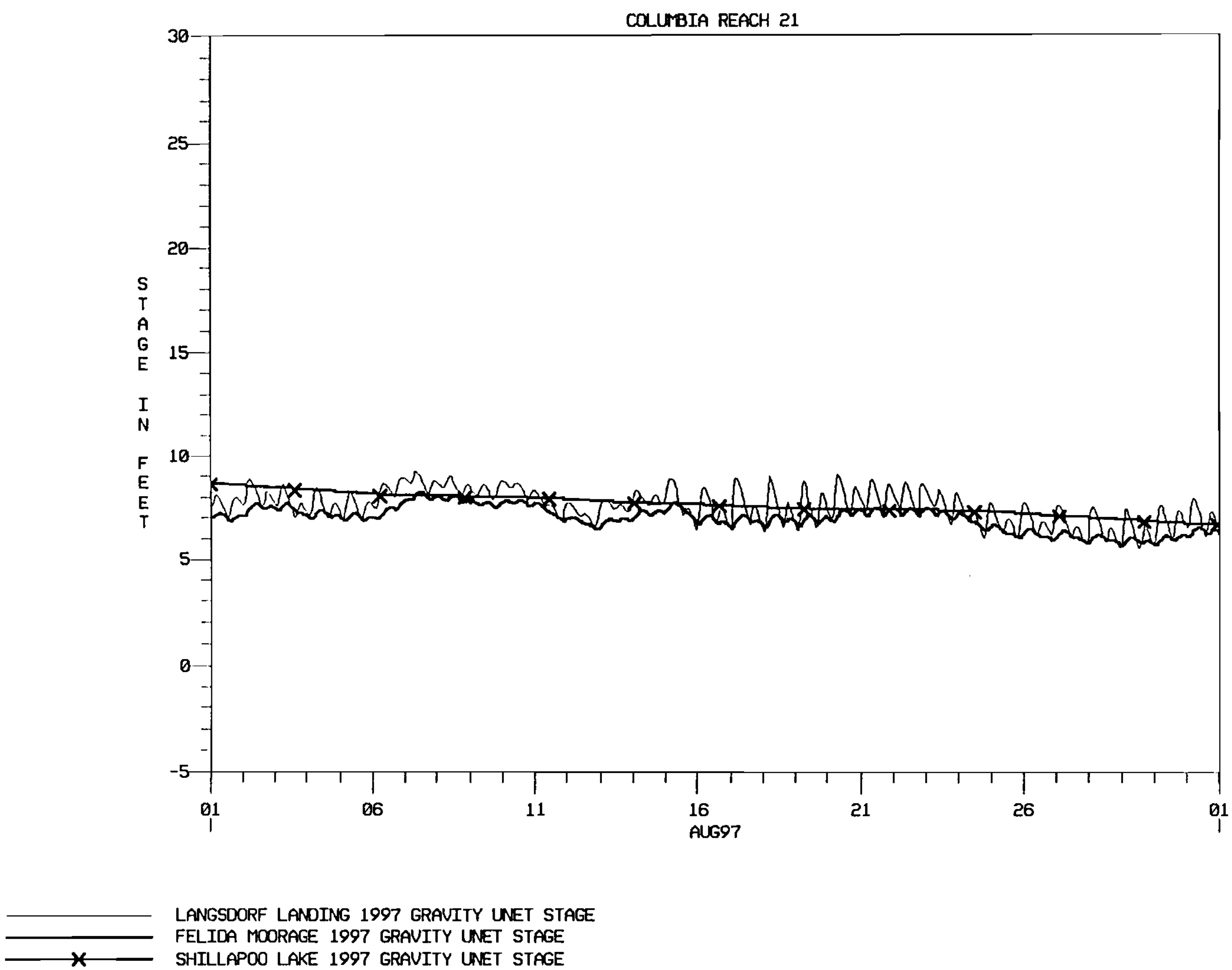
Gravity Supply Alternative. Simulation Results April-May 1997

Figure 4.40



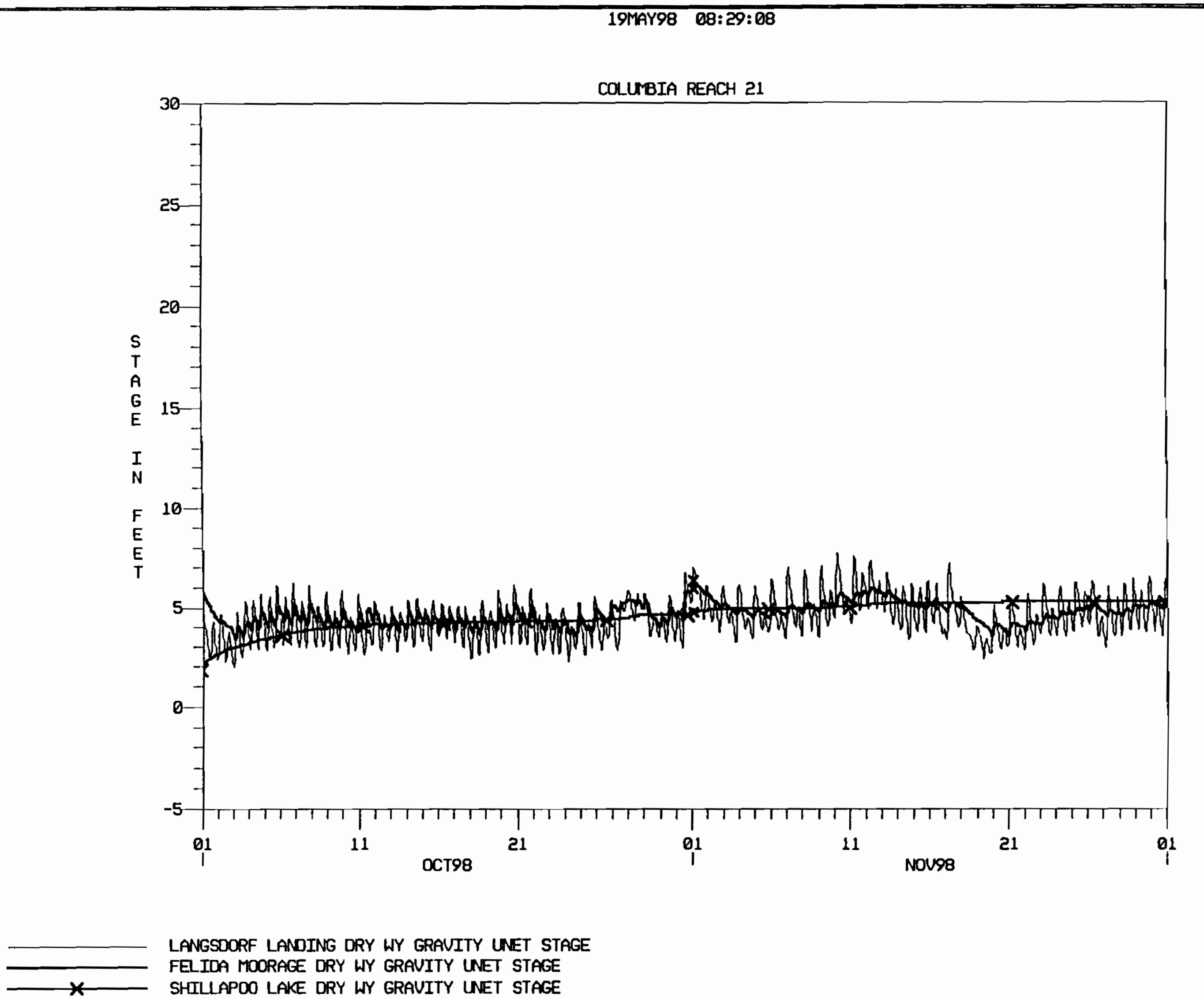
Gravity Supply Alternative. Simulation Results June-July 1997

Figure 4.41



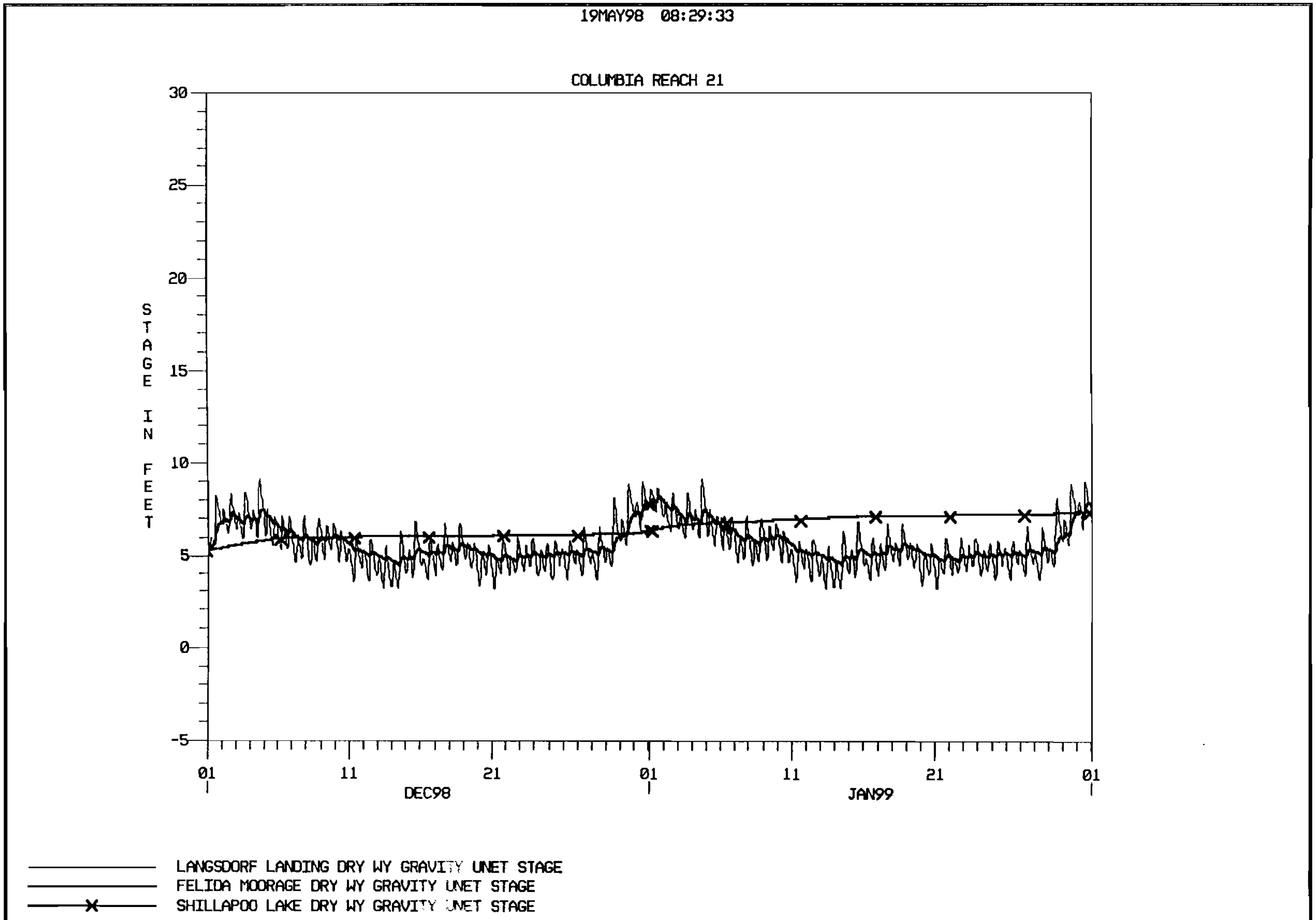
Gravity Supply Alternative. Simulation Results August 1997

Figure 4.42



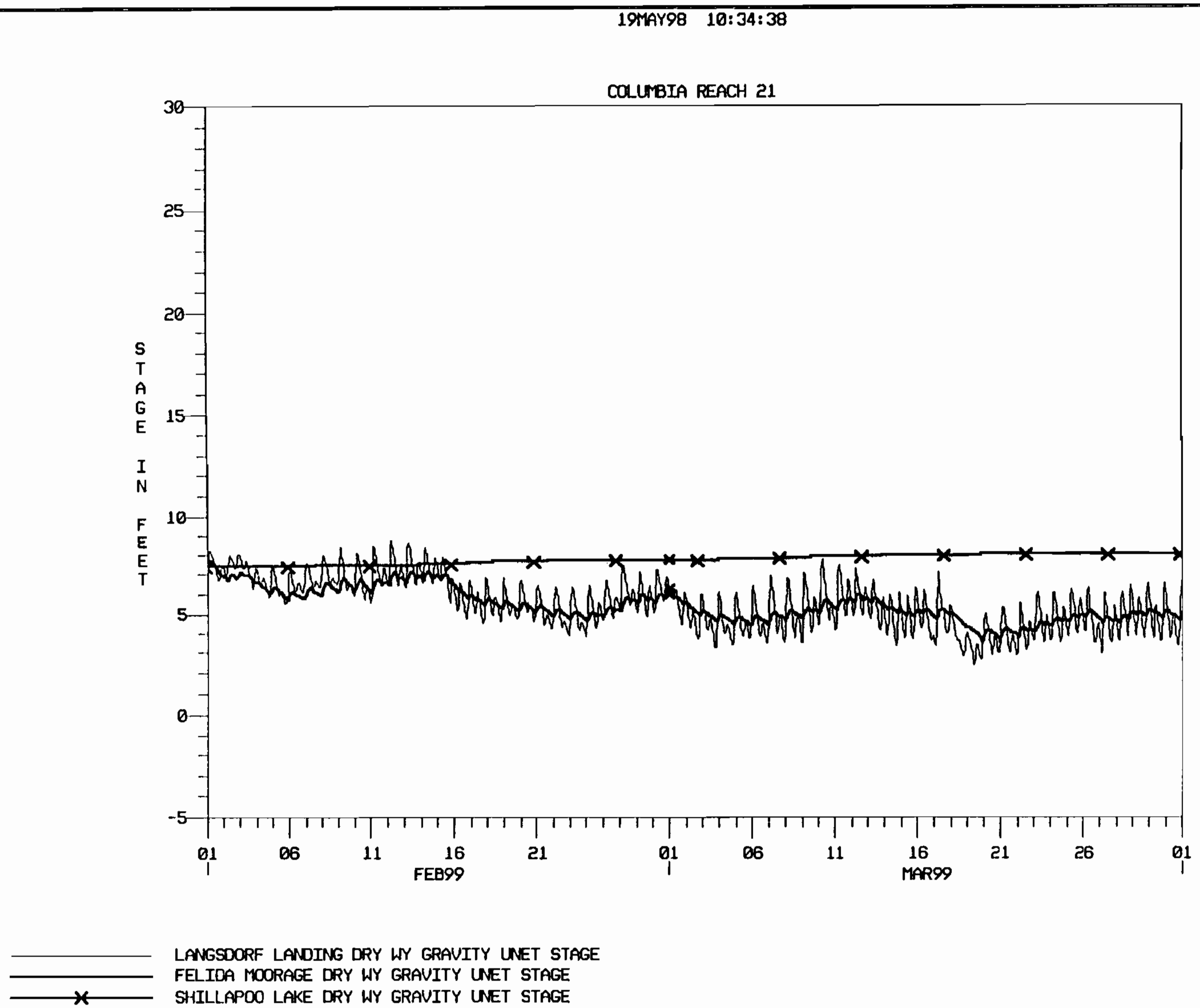
Gravity Supply Alternative. Simulation Results October-November 1998

Figure 4.43



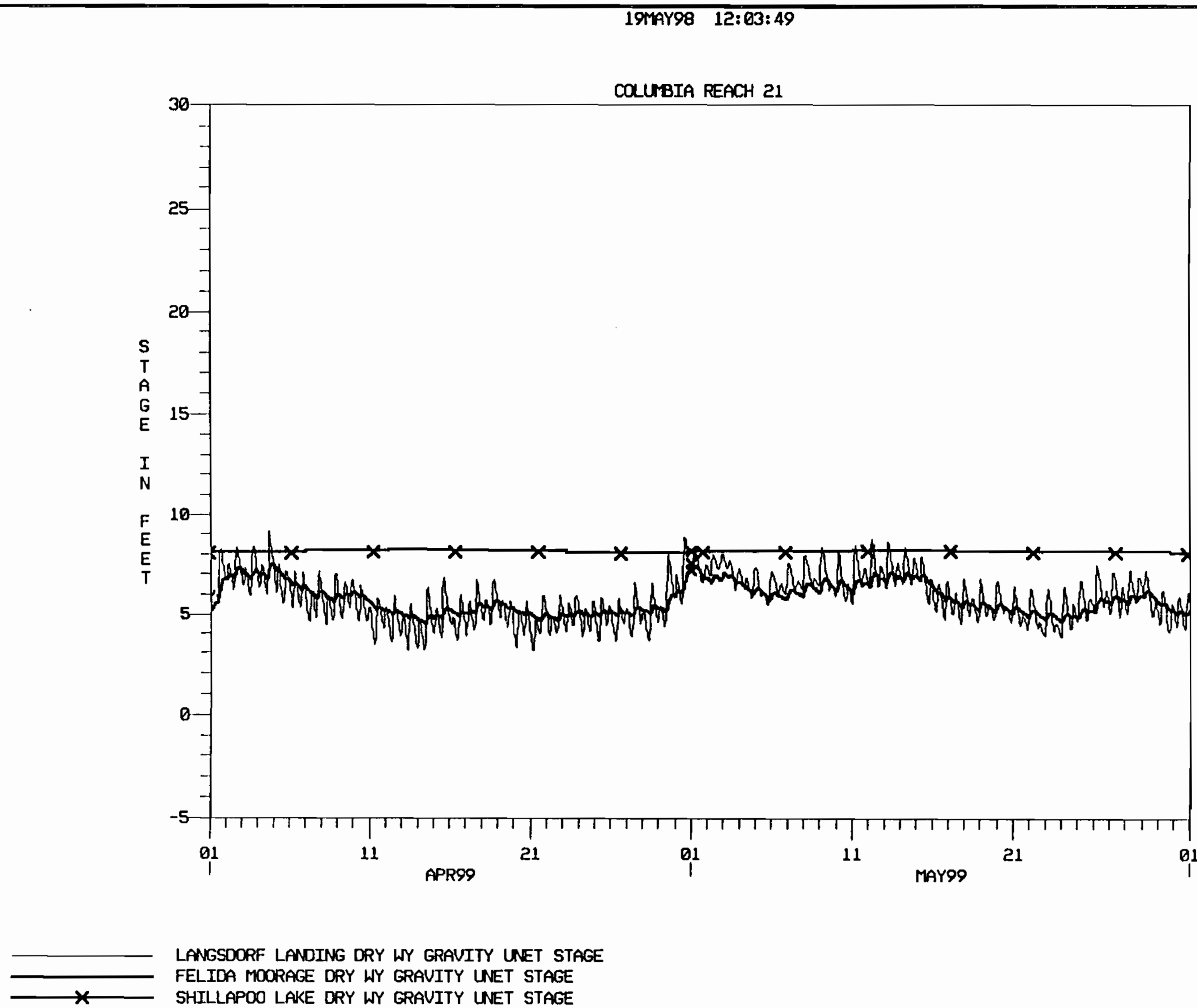
Gravity Supply Alternative. Simulation Results December 1998-January 1999

Figure 4.44



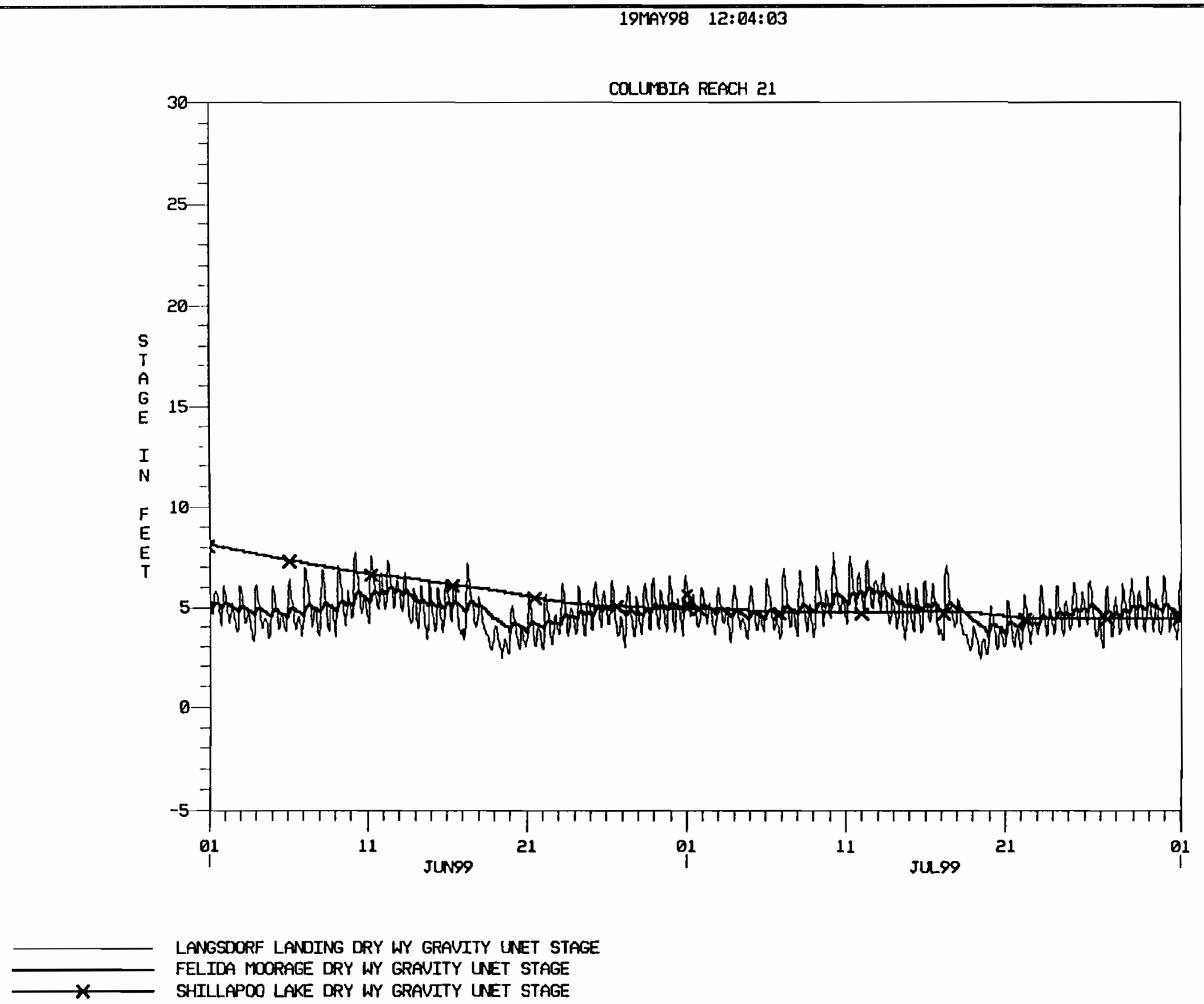
Gravity Supply Alternative. Simulation Results February-March 1999

Figure 4.45



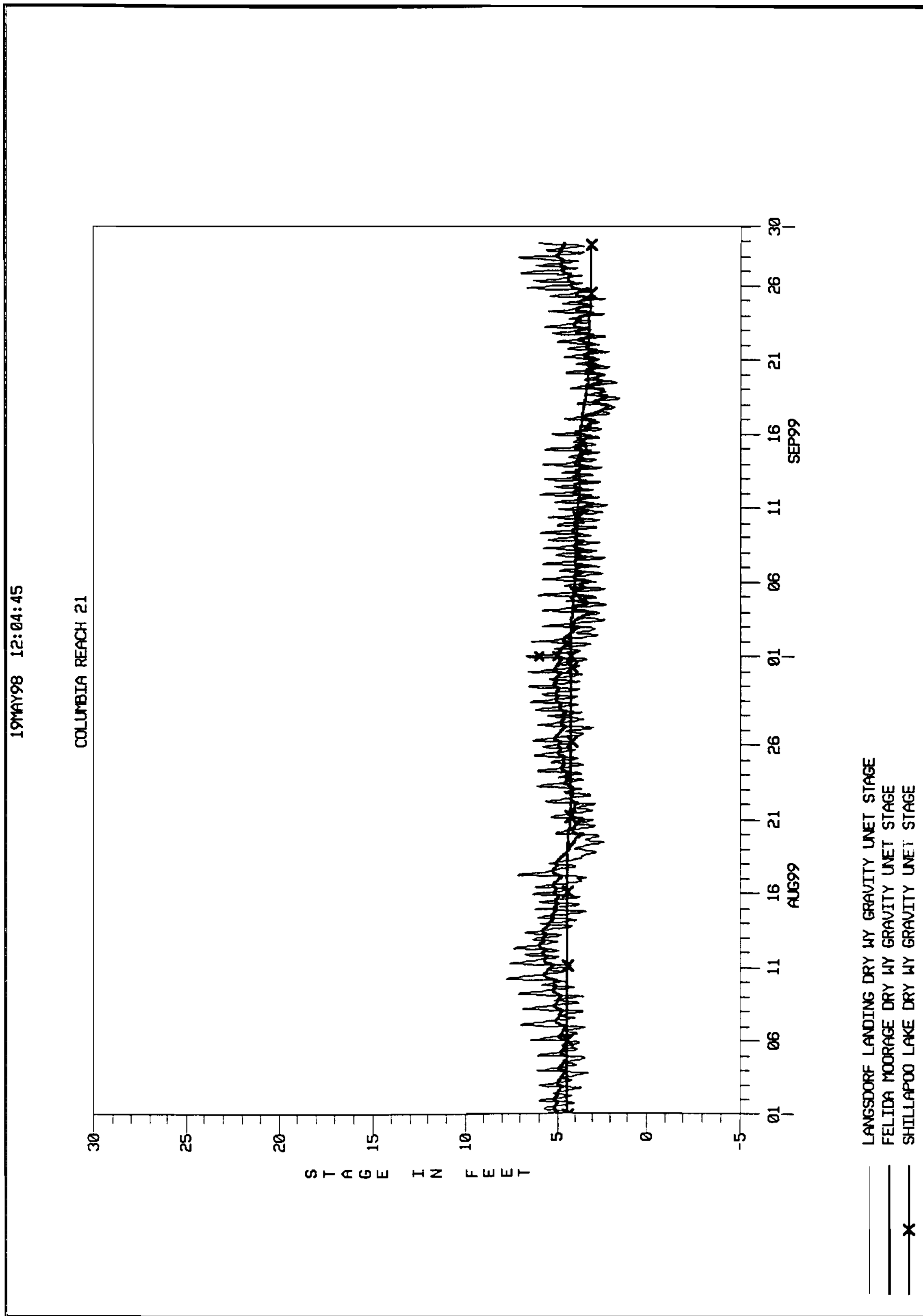
Gravity Supply Alternative. Simulation Results April-May 1999

Figure 4.46



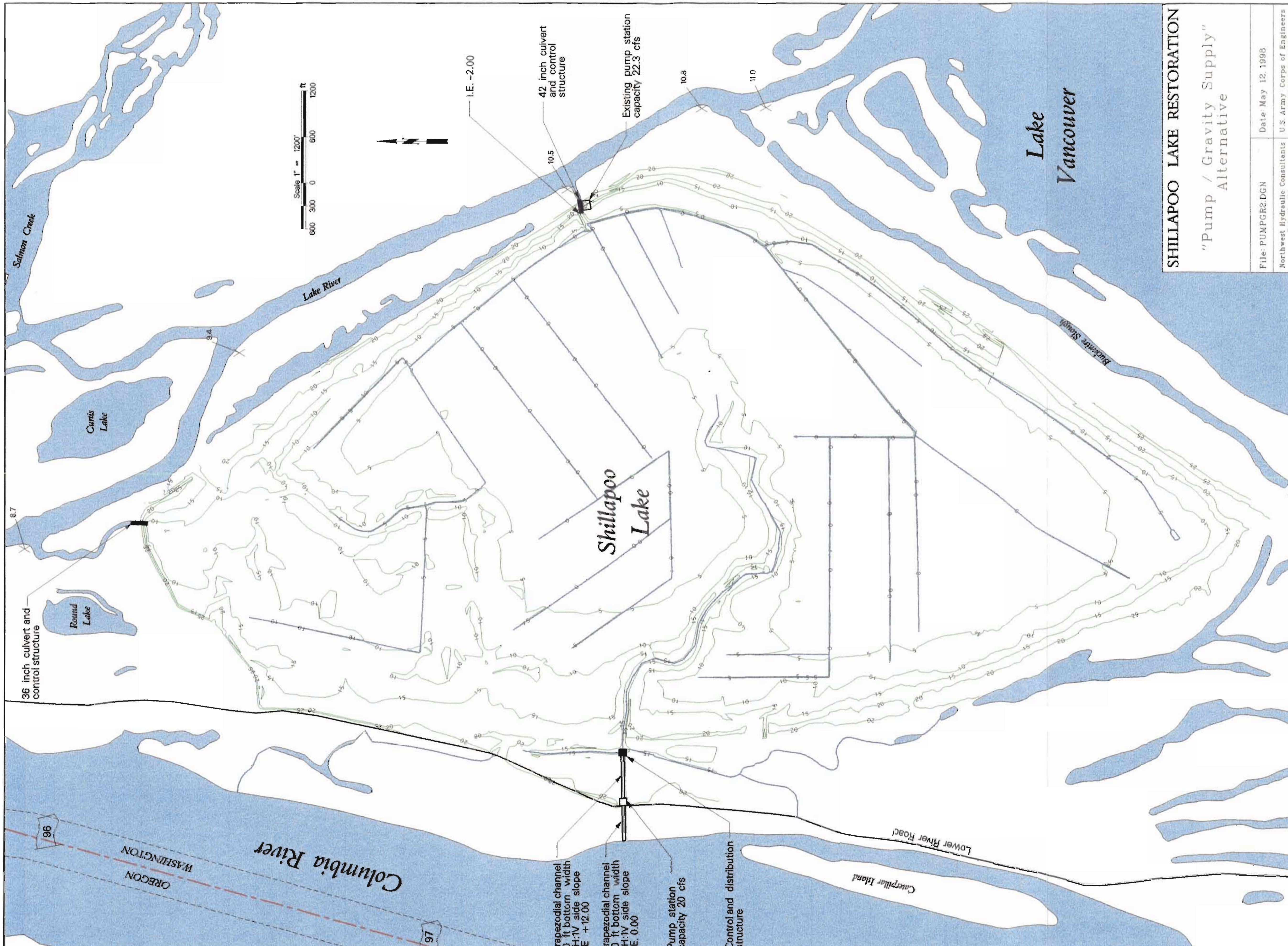
Gravity Supply Alternative. Simulation Results June-July 1999

Figure 4.47



Gravity Supply Alternative. Simulation Results August-September 1999

Figure 4.48



SHILLAPOO LAKE RESTORATION
"Pump / Gravity Supply" Alternative
File: PUMPGR2.DGN
Date May 12, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District
Scale 1" = 1200'
Lake Vancouver
Shillapoo Lake
Columbia River
Lake River
Salmon Creek
Curtis Lake
Round Lake
Buckmire Slough
Caterpillar Island
Lower River Road
WASHINGTON
OREGON
I.E. -2.00
42 inch culvert and control structure
Existing pump station capacity 22.3 cfs
36 inch culvert and control structure
Trapezoidal channel 10 ft bottom width 3H:1V side slope I.E +12.00
Trapezoidal channel 10 ft bottom width 2H:1V side slope I.E. 0.00
Pump station capacity 20 cfs
Control and distribution structure

Figure 4.49

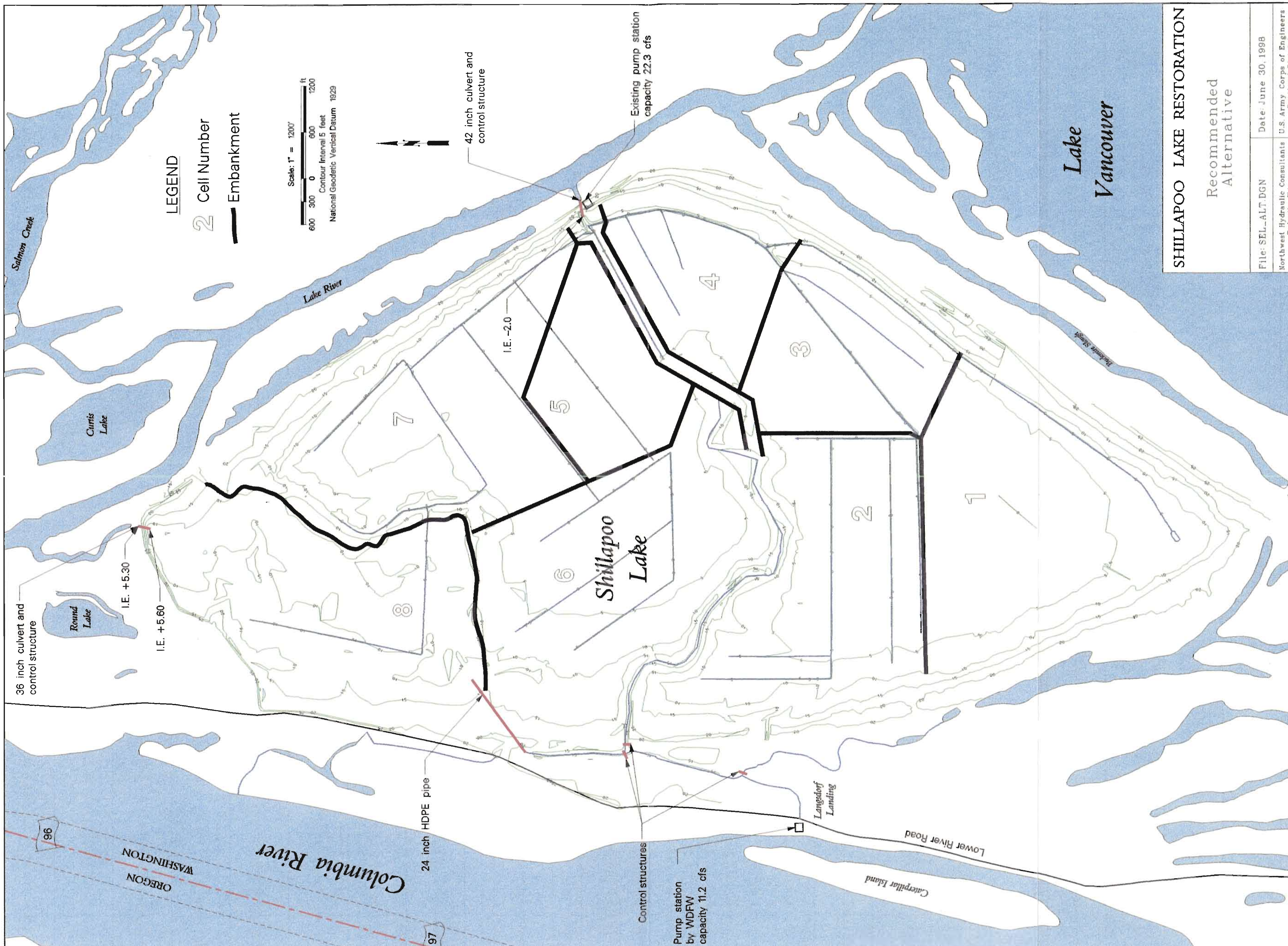
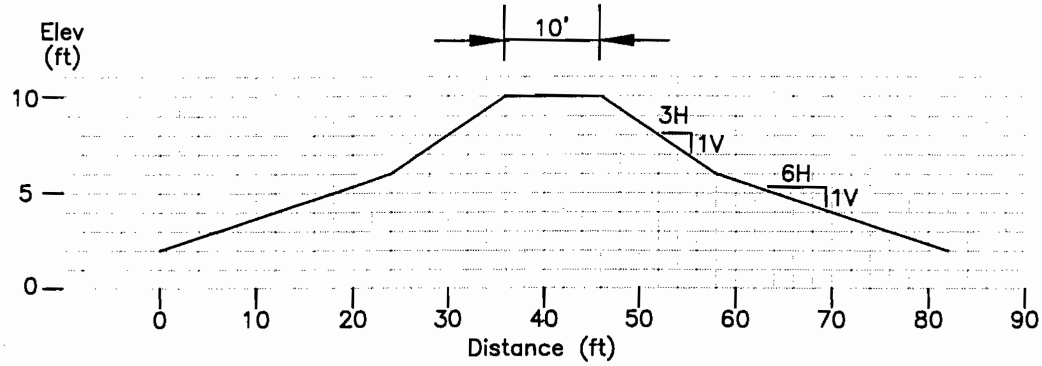
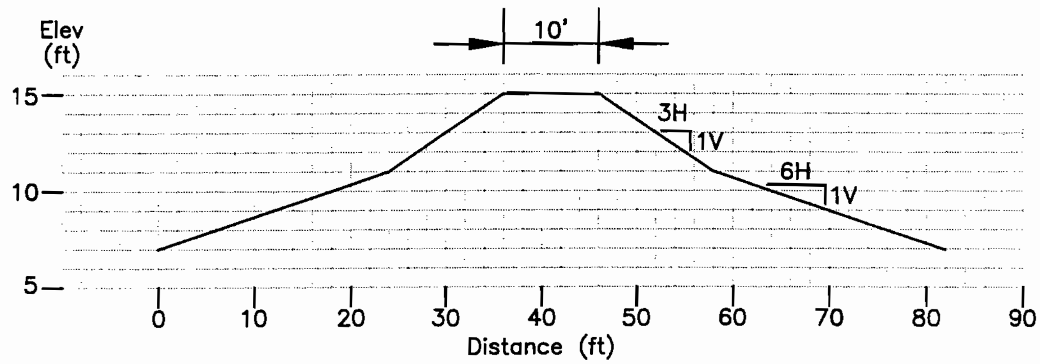

SHILLAPOO LAKE RESTORATION
Recommended Alternative
File: SEL_ALT.DGN
Date: June 30, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District
LEGEND
2 Cell Number
Embankment
Scale: 1" = 1200'
600 300 0 600 1200 ft
Contour Interval 5 feet
National Geodetic Vertical Datum 1929
42 inch culvert and control structure
Existing pump station capacity 22.3 cfs
I.E. -2.0
36 inch culvert and control structure
I.E. +5.30
I.E. +5.60
24 inch HDPE pipe
Control structures
Pump station by WDFW capacity 11.2 cfs
Lake Vancouver
Shillapoo Lake
Salmon Creek
Lake River
Curtis Lake
Round Lake
Columbia River
WASHINGTON
OREGON
Langsdorf Landing
Lower River Road
Caterpillar Island
96
97
1
2
3
4
5
6
7
8

Figure 5.1



Typical Embankment Section
Cell 1-7



Typical Embankment Section
Cell 8

Scale: Vertical 1"=10'
Horizontal 1"=20'

SHILLAPOO LAKE RESTORATION Typical Embankment Section	
File: TYP_EMBK.DWG	Date: JUNE 30, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.2

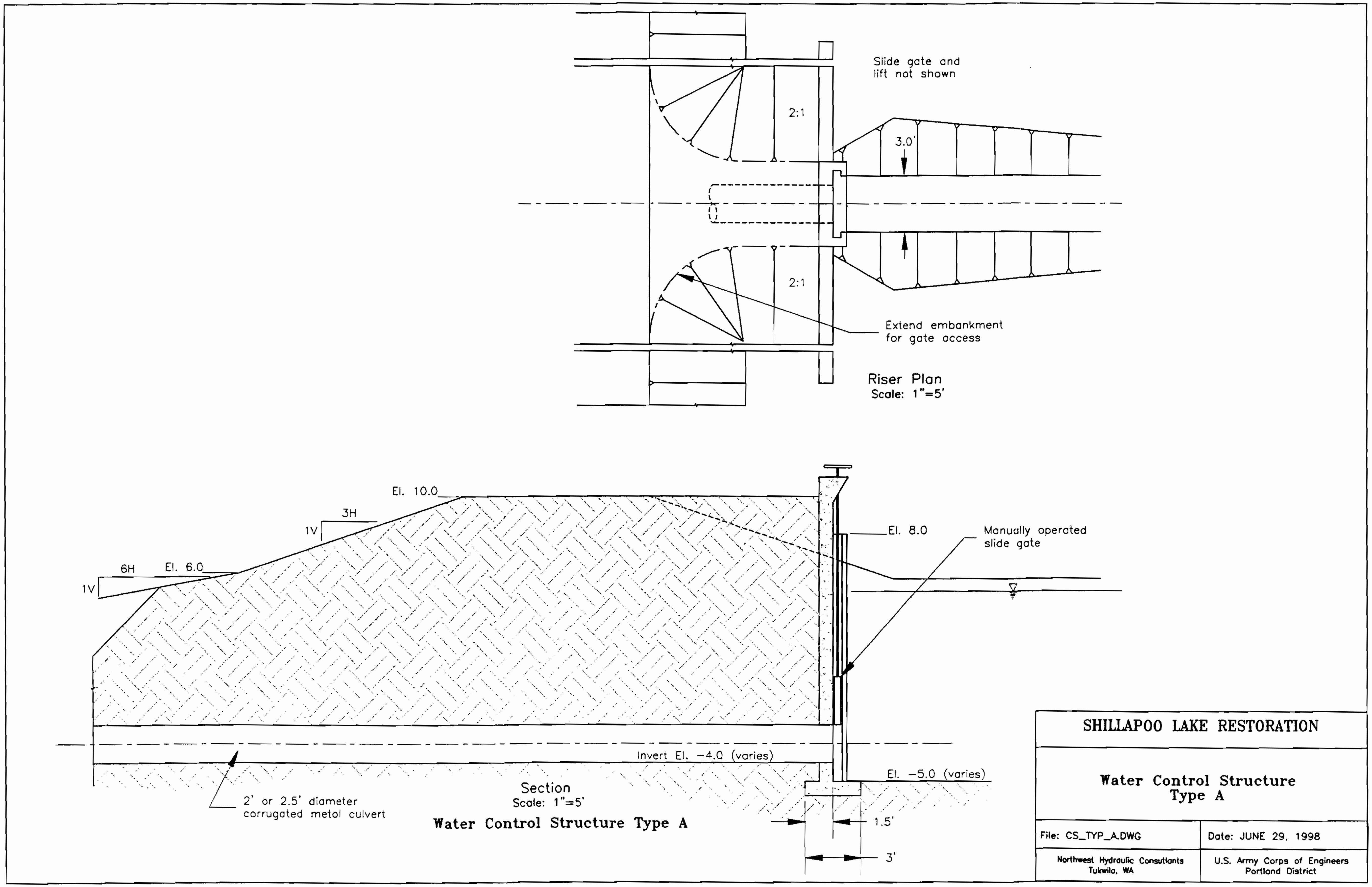

Slide gate and lift not shown
2:1
3.0'
2:1
Extend embankment for gate access
Riser Plan
Scale: 1"=5'
El. 10.0
3H
1V
6H
El. 6.0
1V
El. 8.0
Manually operated slide gate
Invert El. -4.0 (varies)
El. -5.0 (varies)
Section
Scale: 1"=5'
2' or 2.5' diameter corrugated metal culvert
1.5'
3'
Water Control Structure Type A
SHILLAPOO LAKE RESTORATION
Water Control Structure Type A
File: CS_TYP_A.DWG
Date: JUNE 29, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District

Figure 5.3

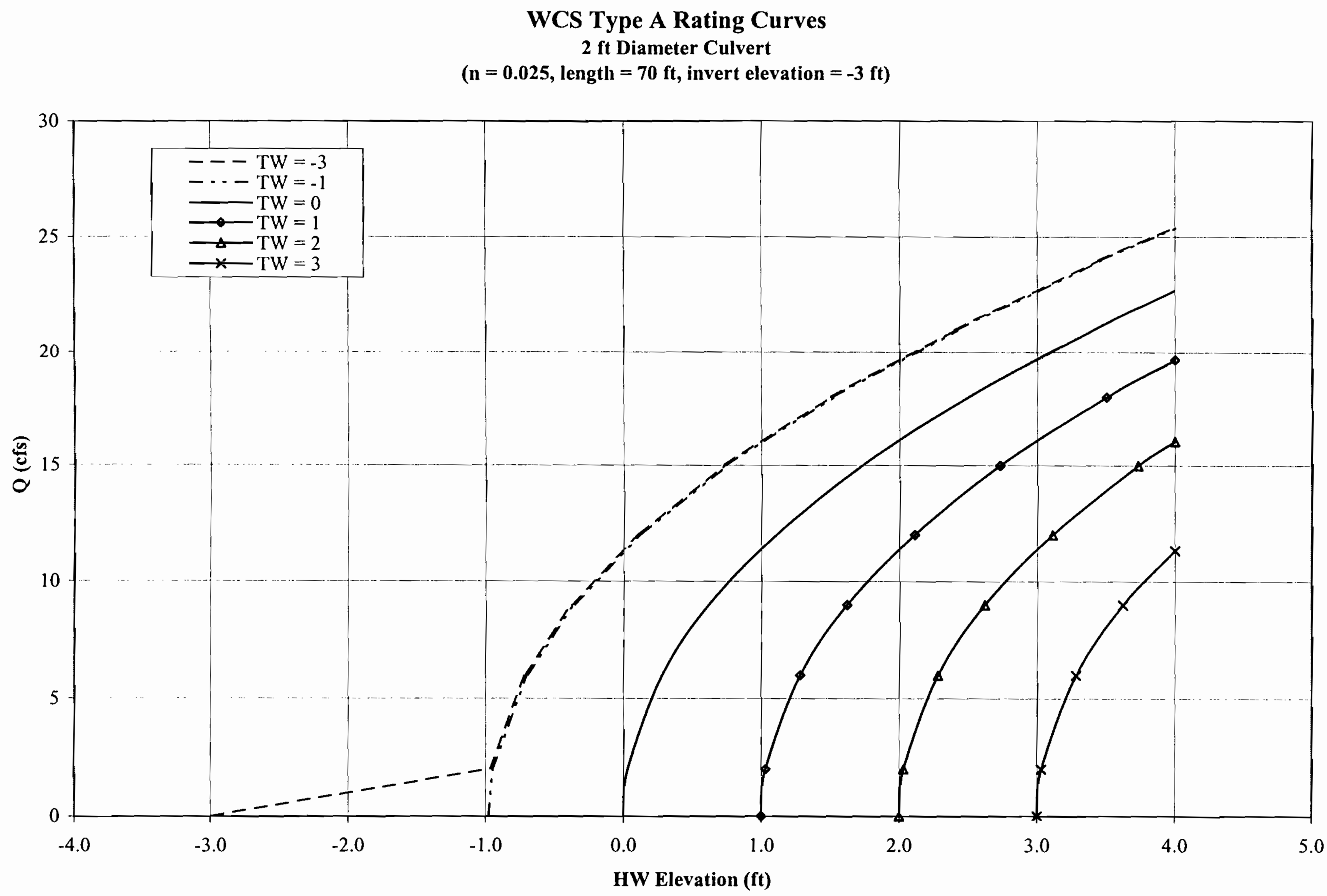
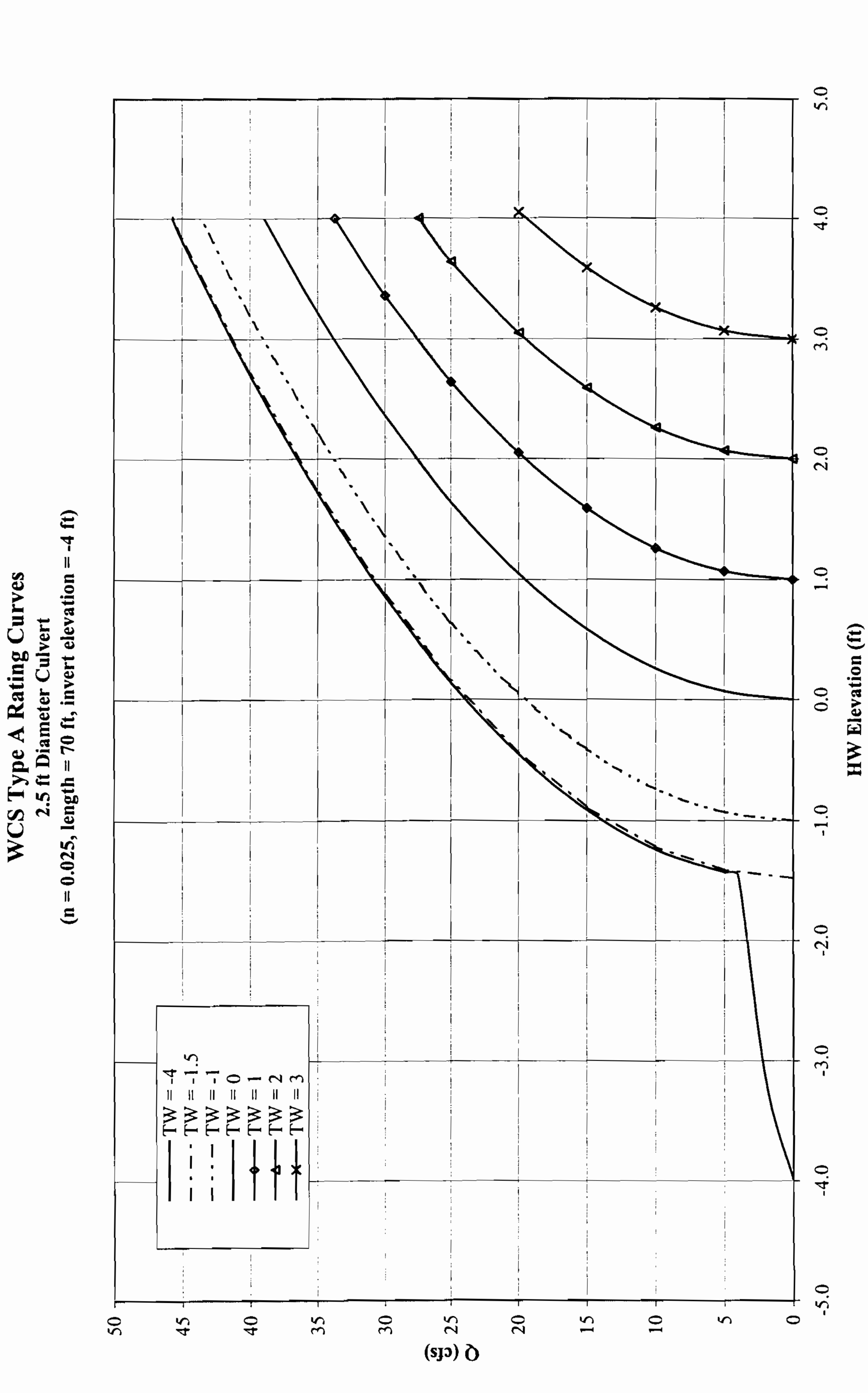

WCS Type A Rating Curves
2 ft Diameter Culvert
(n = 0.025, length = 70 ft, invert elevation = -3 ft)
TW = -3
TW = -1
TW = 0
TW = 1
TW = 2
TW = 3
Q (cfs)
30
25
20
15
10
5
0
-4.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0
4.0
5.0
HW Elevation (ft)

Figure 5.4



WCS Type A Rating Curves
2.5 ft Diameter Culvert
(n = 0.025, length = 70 ft, invert elevation = -4 ft)
Q (cfs)
HW Elevation (ft)
TW = -4
TW = -1.5
TW = -1
TW = 0
TW = 1
TW = 2
TW = 3
0
5
10
15
20
25
30
35
40
45
50
-5.0
-4.0
-3.0
-2.0
-1.0
0.0
1.0
2.0
3.0
4.0
5.0

Figure 5.5

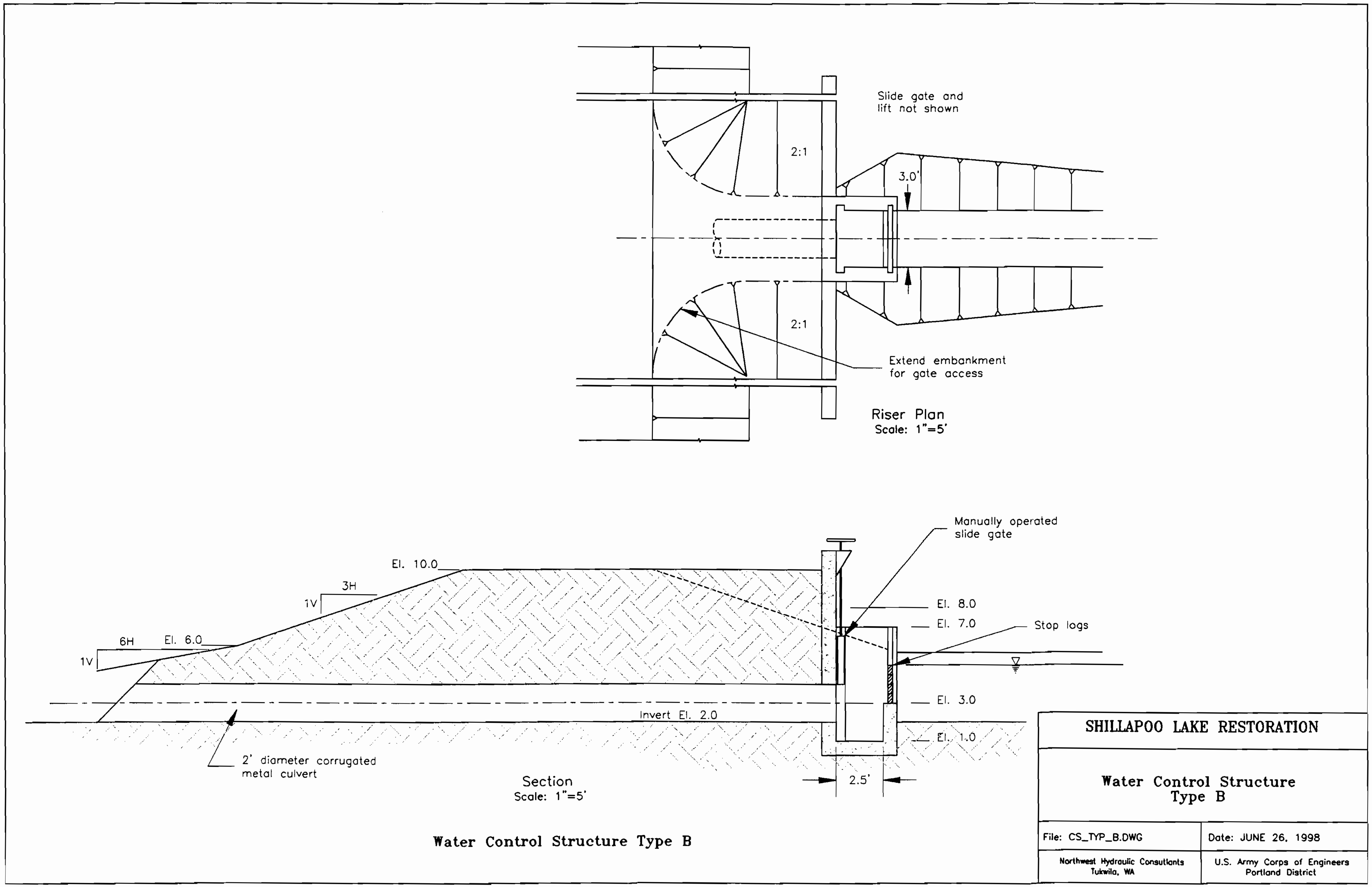

Slide gate and lift not shown
2:1
3.0'
2:1
Extend embankment for gate access
Riser Plan
Scale: 1"=5'
Manually operated slide gate
El. 10.0
3H
1V
6H
El. 6.0
1V
El. 8.0
El. 7.0
Stop logs
El. 3.0
Invert El. 2.0
El. 1.0
2' diameter corrugated metal culvert
Section
Scale: 1"=5'
2.5'
Water Control Structure Type B
SHILLAPOO LAKE RESTORATION
Water Control Structure Type B
File: CS_TYP_B.DWG
Date: JUNE 26, 1998
Northwest Hydraulic Consultants Tukwila, WA
U.S. Army Corps of Engineers Portland District

Figure 5.6

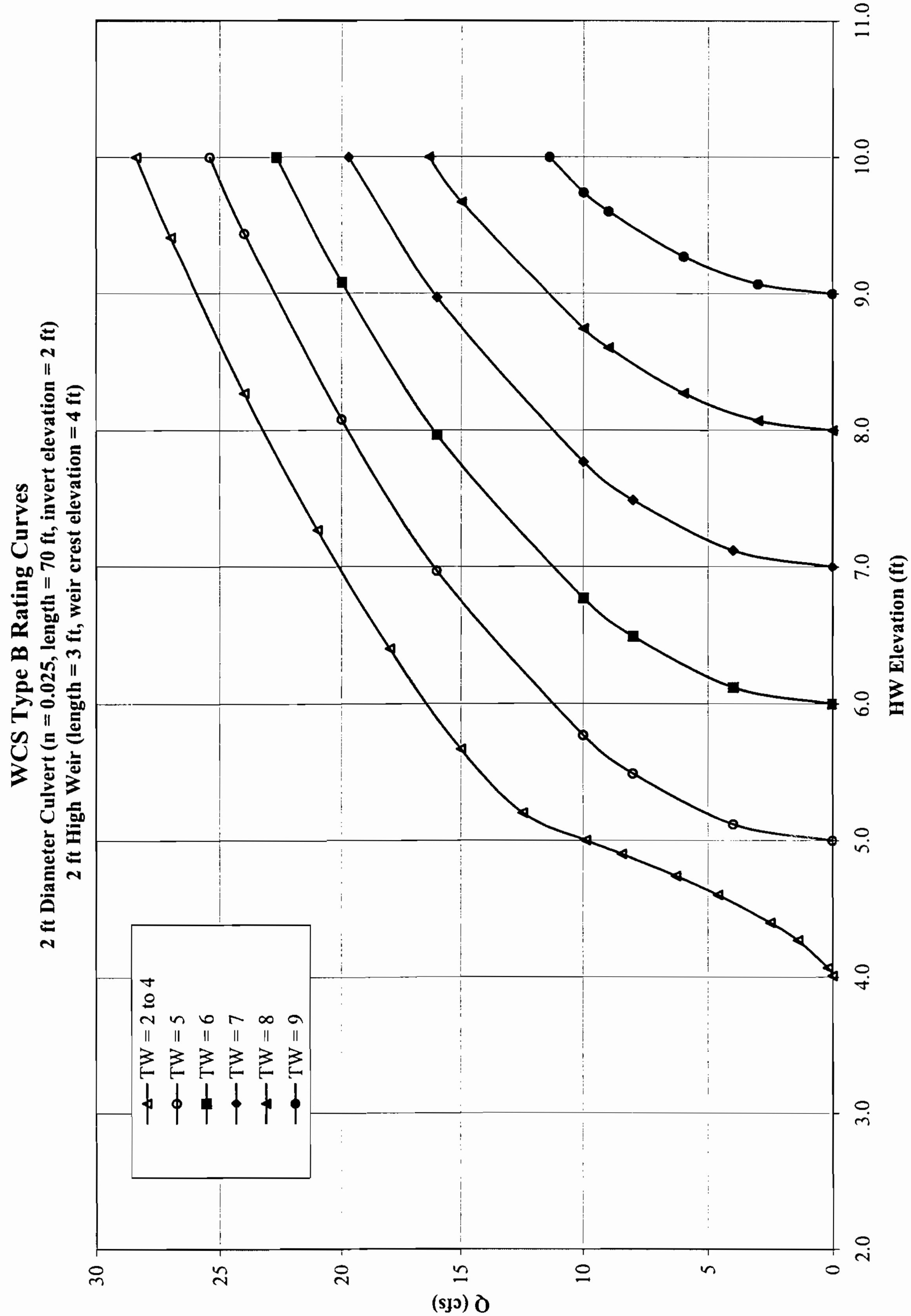
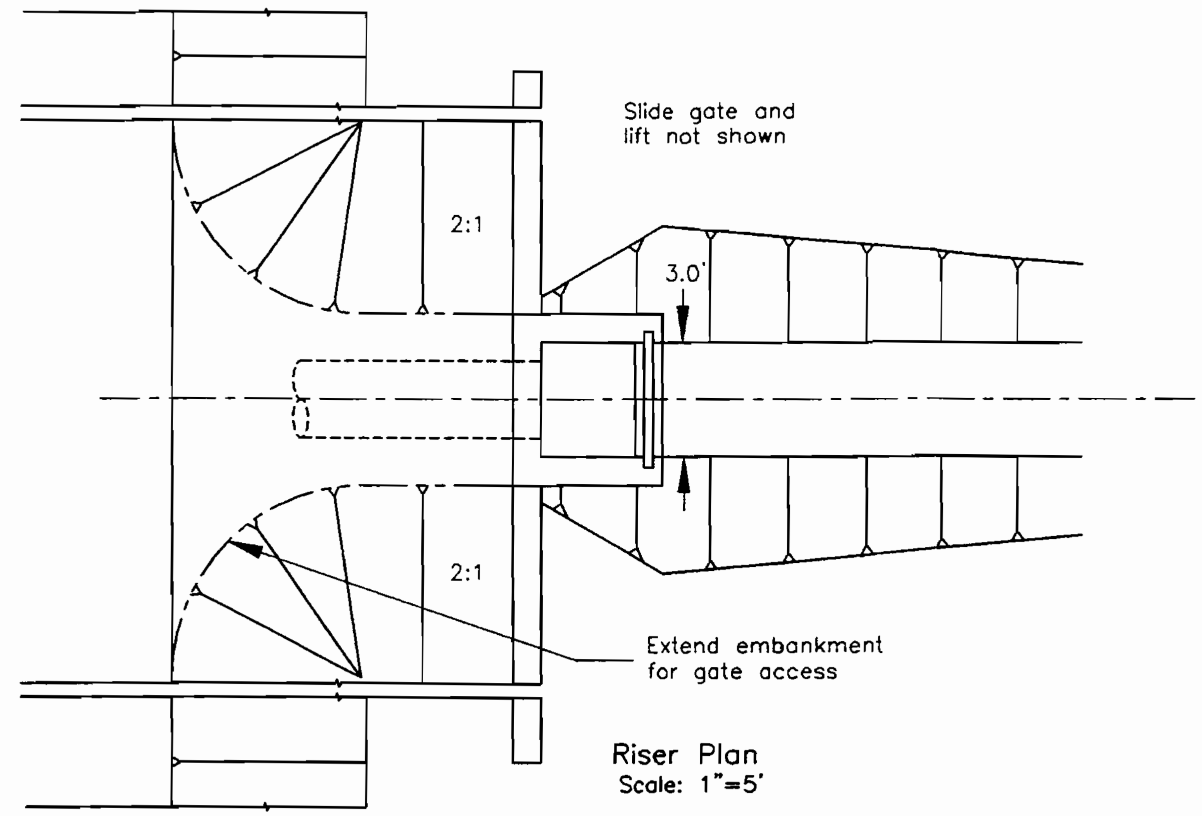
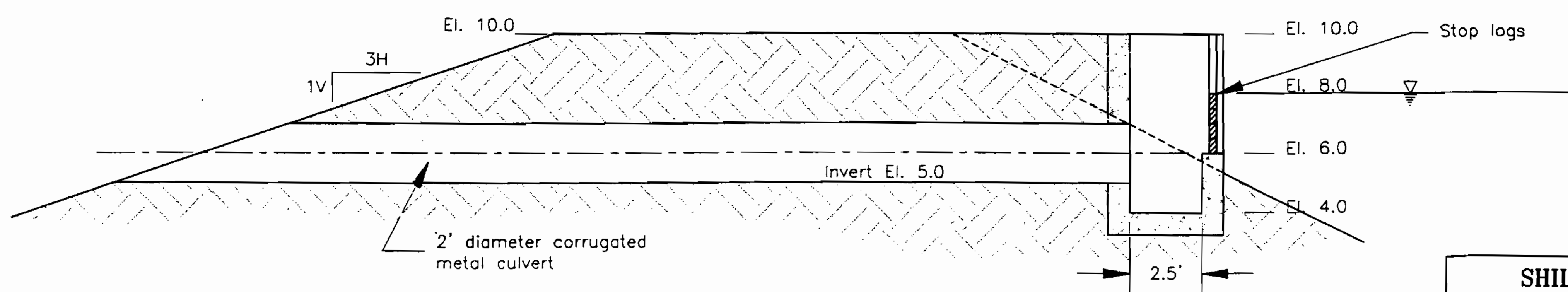

WCS Type B Rating Curves
2 ft Diameter Culvert (n = 0.025, length = 70 ft, invert elevation = 2 ft)
2 ft High Weir (length = 3 ft, weir crest elevation = 4 ft)
TW = 2 to 4
TW = 5
TW = 6
TW = 7
TW = 8
TW = 9
Q (cfs)
0
5
10
15
20
25
30
HW Elevation (ft)
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0

Figure 5.7



Riser Plan
Scale: 1"=5'



Section
Scale: 1"=5'

Water Control Structure Type C

SHILLAPOO LAKE RESTORATION	
Water Control Structure Type C	
File: CS_TYP_C.DWG	Date: JUNE 29, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.8

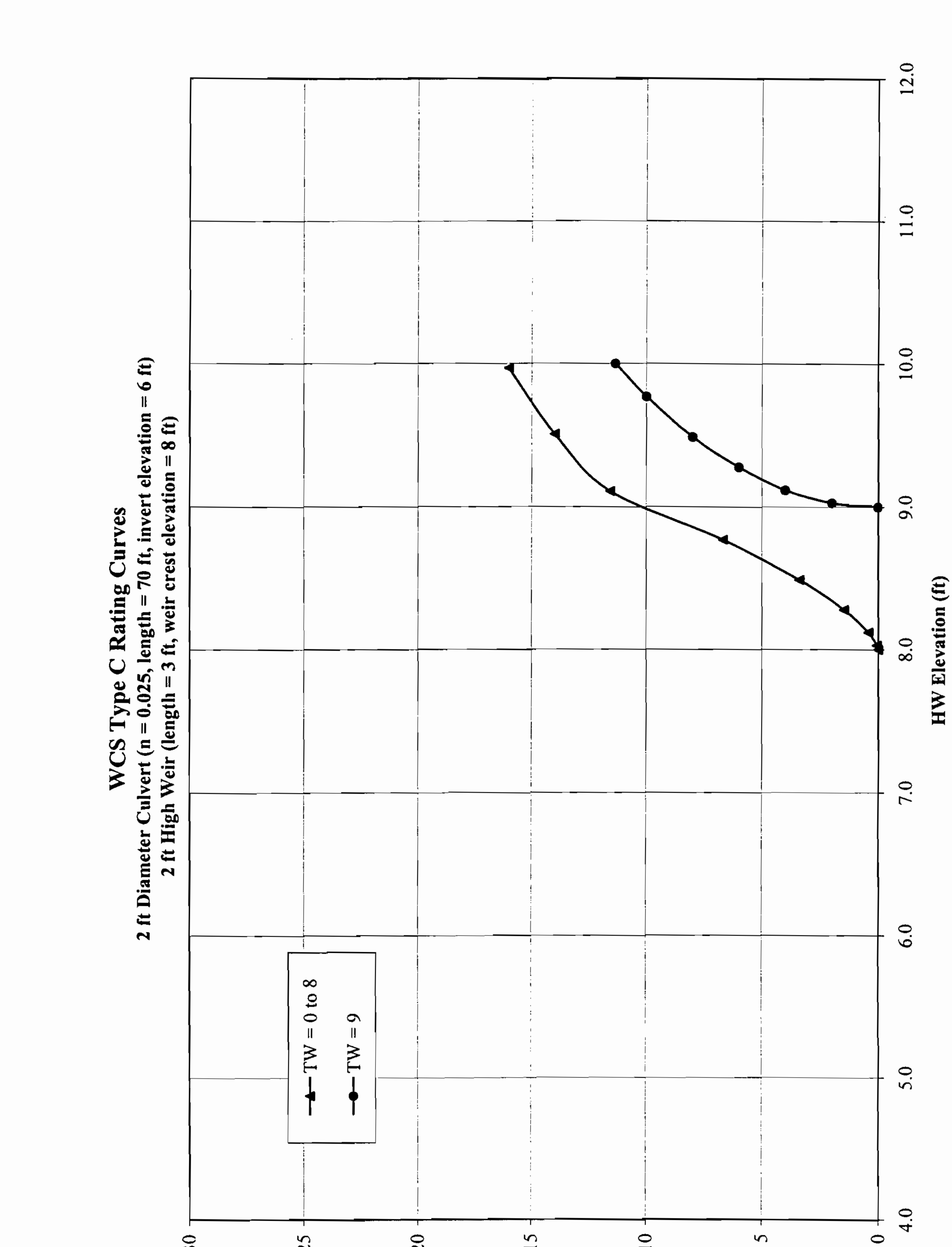
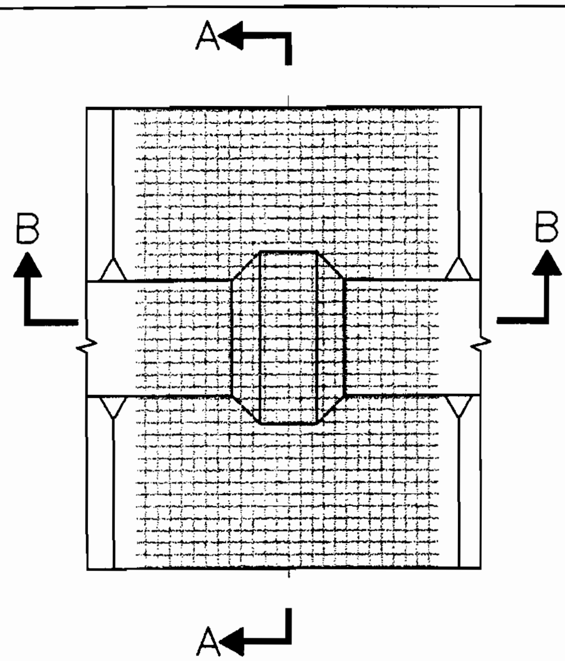
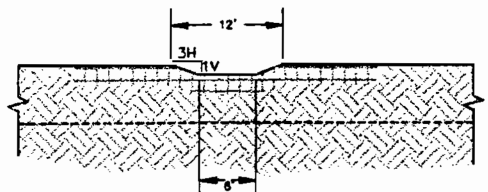

WCS Type C Rating Curves
2 ft Diameter Culvert (n = 0.025, length = 70 ft, invert elevation = 6 ft)
2 ft High Weir (length = 3 ft, weir crest elevation = 8 ft)
TW = 0 to 8
TW = 9
Q (cfs)
30
25
20
15
10
5
0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
HW Elevation (ft)

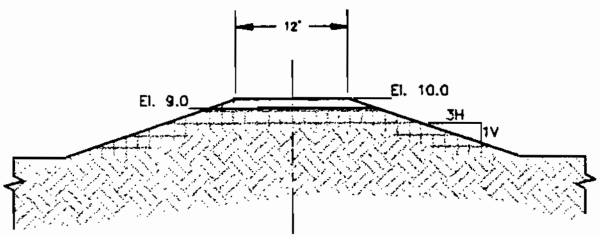
Figure 5.9



Plan
Water Control Structure D
Scale: 1"=20'



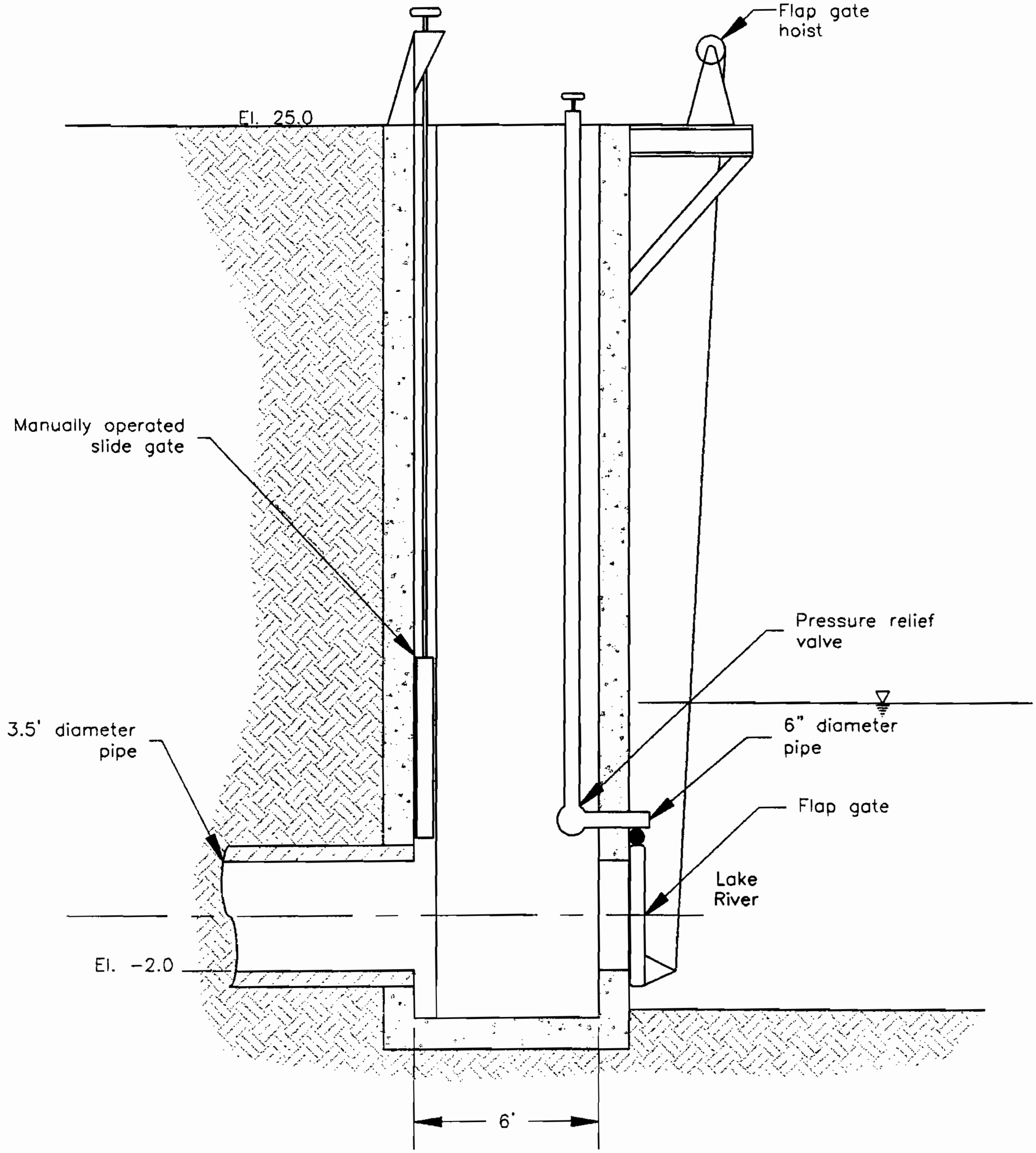
Section B – B
Water Control Structure D
Scale: 1"=20'



Section A – A
Water Control Structure D
Scale: 1"=20'

SHILLAPOO LAKE RESTORATION **Water Control Structure D**	
File: CS_TYP_D.DWG	Date: JUNE 29, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.10



Section
Water Control Structure E-1
Scale: 1"=5"

SHILLAPOO LAKE RESTORATION **Water Control Structure E-1**	
File: CS_TYP_E.DWG	Date: JUNE 29, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.11

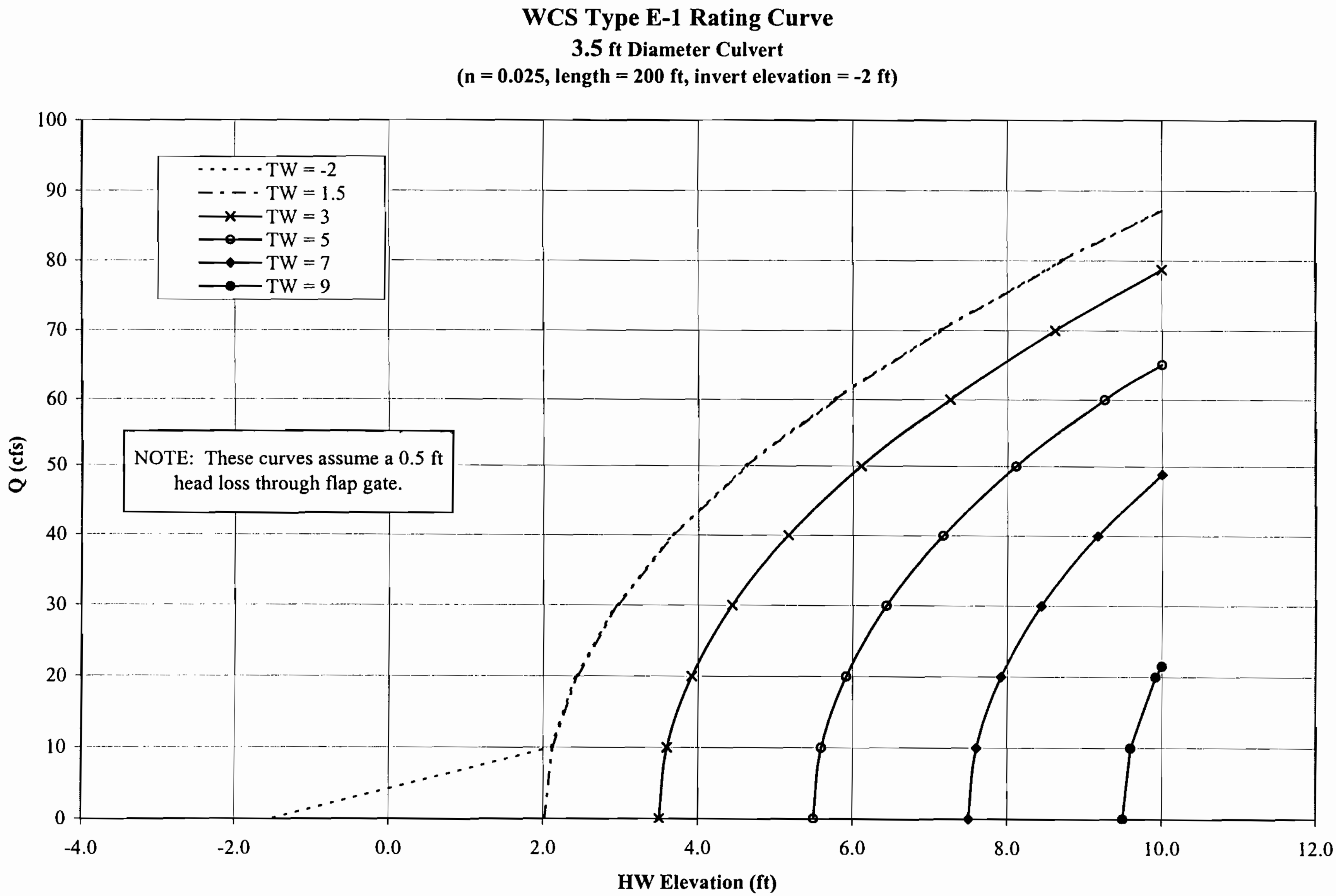
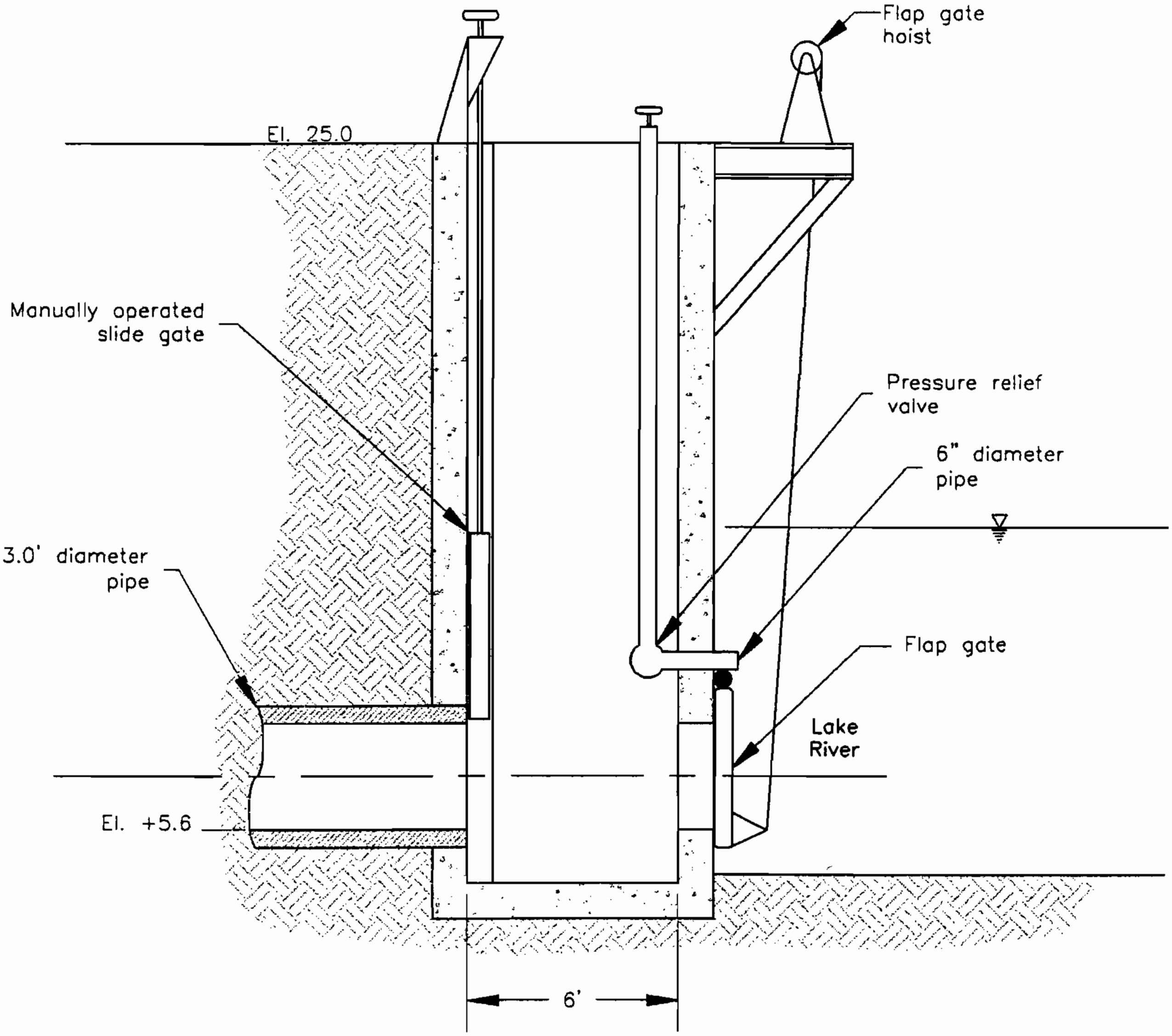

WCS Type E-1 Rating Curve
3.5 ft Diameter Culvert
(n = 0.025, length = 200 ft, invert elevation = -2 ft)
Q (cfs)
HW Elevation (ft)
TW = -2
TW = 1.5
TW = 3
TW = 5
TW = 7
TW = 9
NOTE: These curves assume a 0.5 ft head loss through flap gate.
100
90
80
70
60
50
40
30
20
10
0
-4.0
-2.0
0.0
2.0
4.0
6.0
8.0
10.0
12.0

Figure 5.12



Section
Water Control Structure G-1
Scale: 1"=5"

SHILLAPOO LAKE RESTORATION **Water Control Structure G-1**	
File: CS_TYP_G.DWG	Date: JUNE 29, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.13

WCS Type G-1 Rating Curve
3 ft Diameter Culvert
(n = 0.025, length = 100 ft, u/s invert elevation = 5.6 ft)

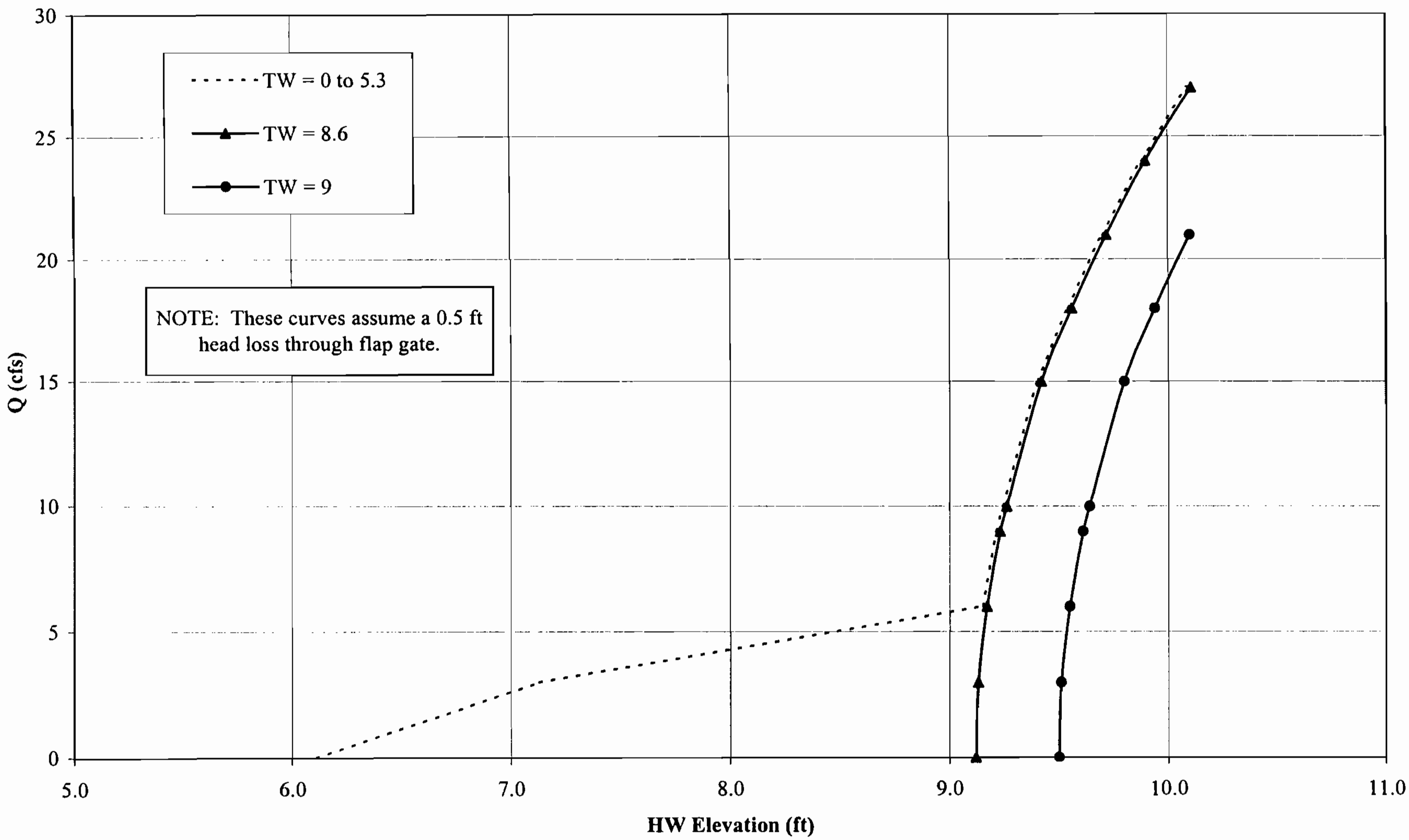
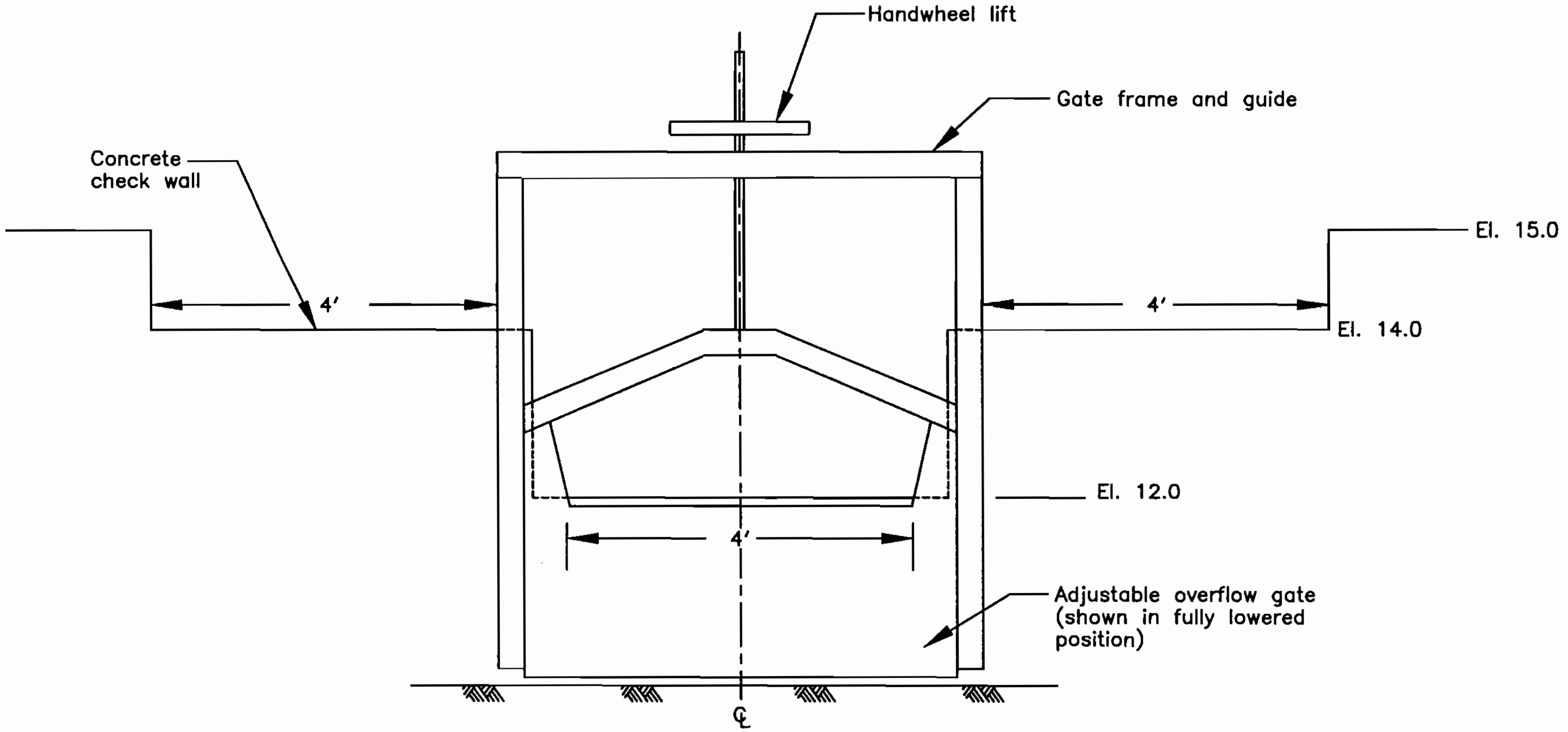


Figure 5.14



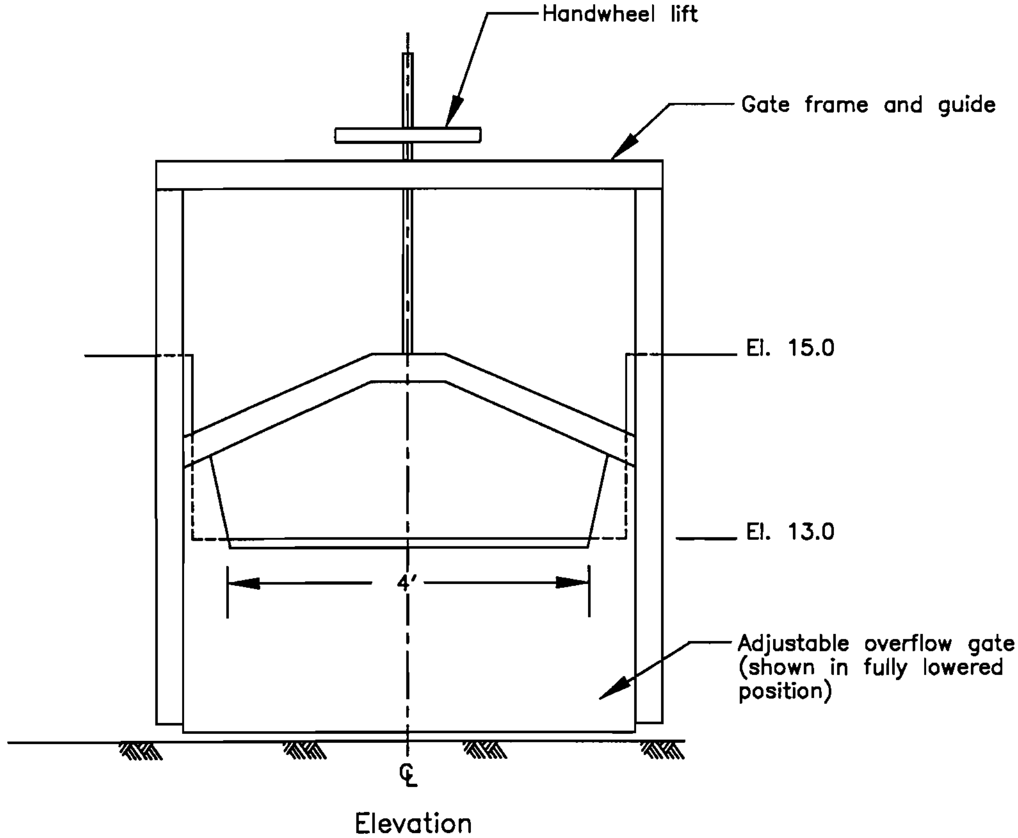
Elevation

Water Control Structure H-I

Scale: 1"=2'

SHILLAPOO LAKE RESTORATION **Water Control Structure H-I**	
File: E1-H1.DWG	Date: JUNE 30, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.15



Water Control Structure Type I

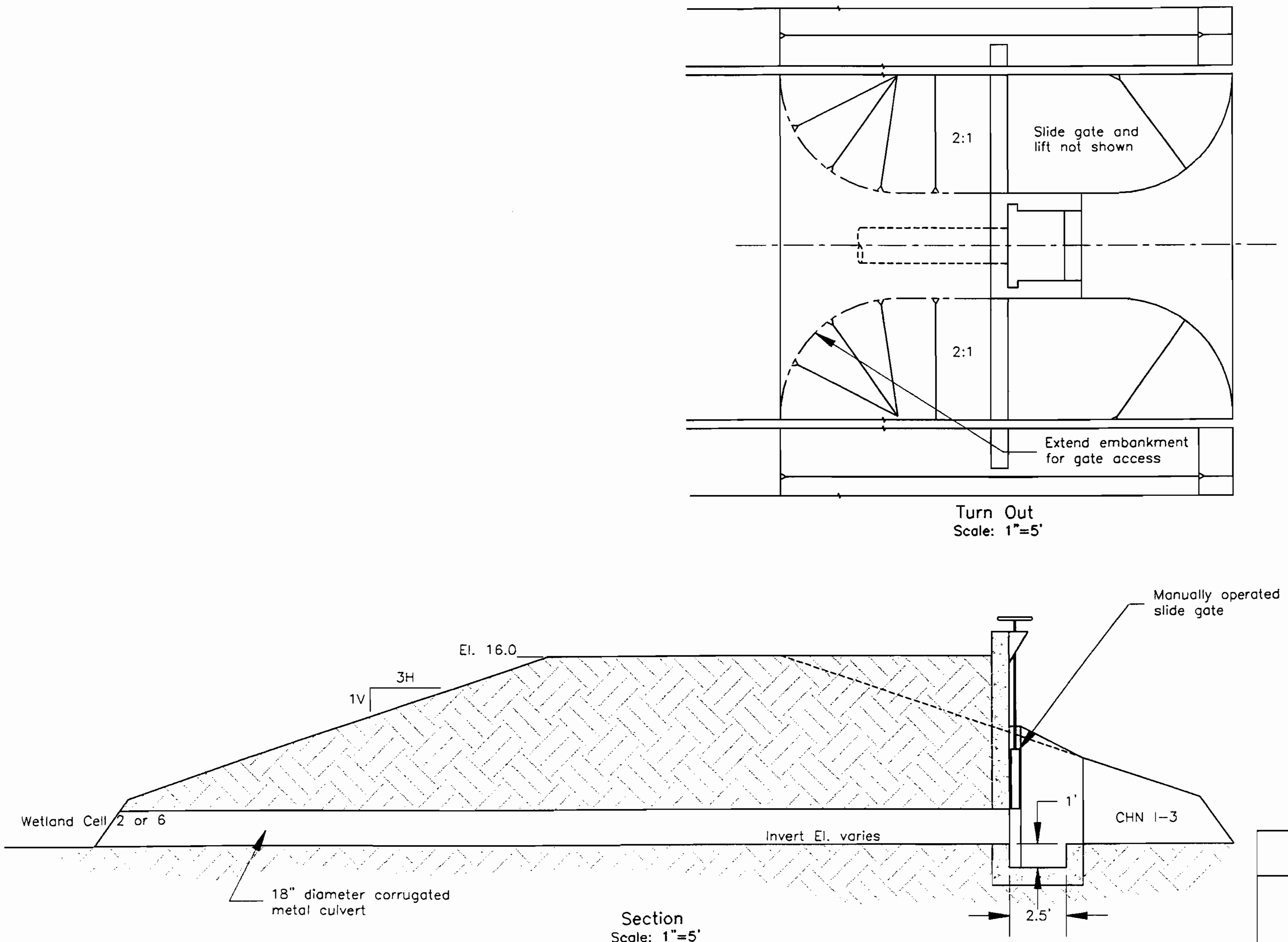
Scale: 1"=2'

SHILLAPOO LAKE RESTORATION

Water Control Structure Type I

File: E1-TYPEI.DWG	Date: JUNE 30, 1998
Northwest Hydraulic Consutlants Tukwila, WA	U.S. Army Corps of Engineers Portland District

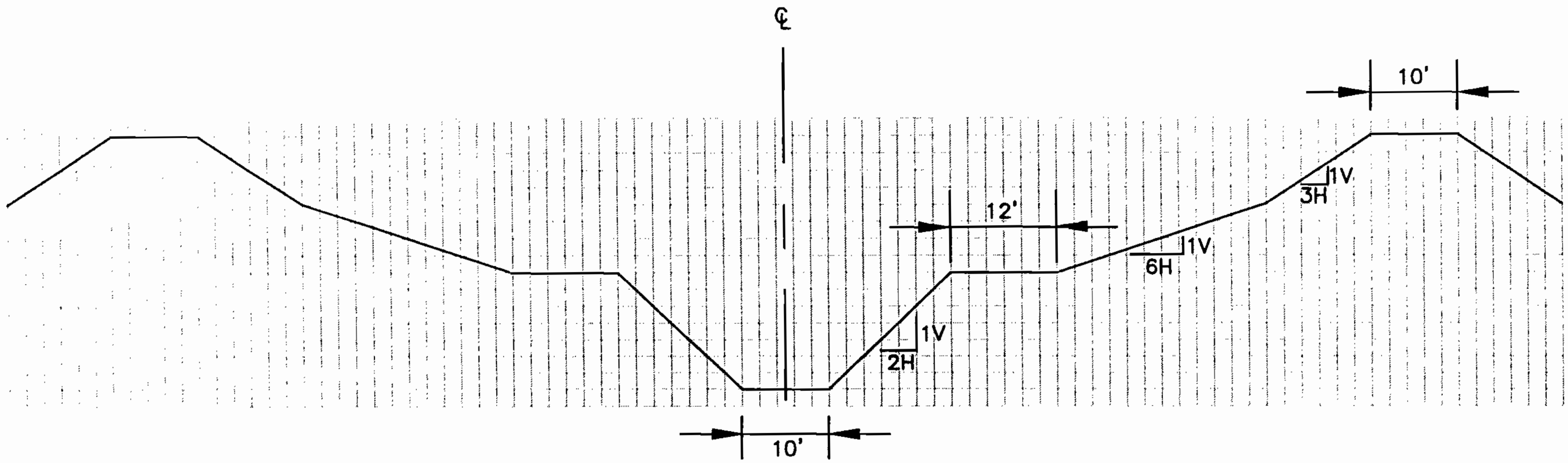
Figure 5.16



Water Control Structure Type J

SHILLAPOO LAKE RESTORATION	
Water Control Structure Type J	
File: CS_TYP_J.DWG	Date: JUNE 26, 1998
Northwest Hydraulic Consultants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.17

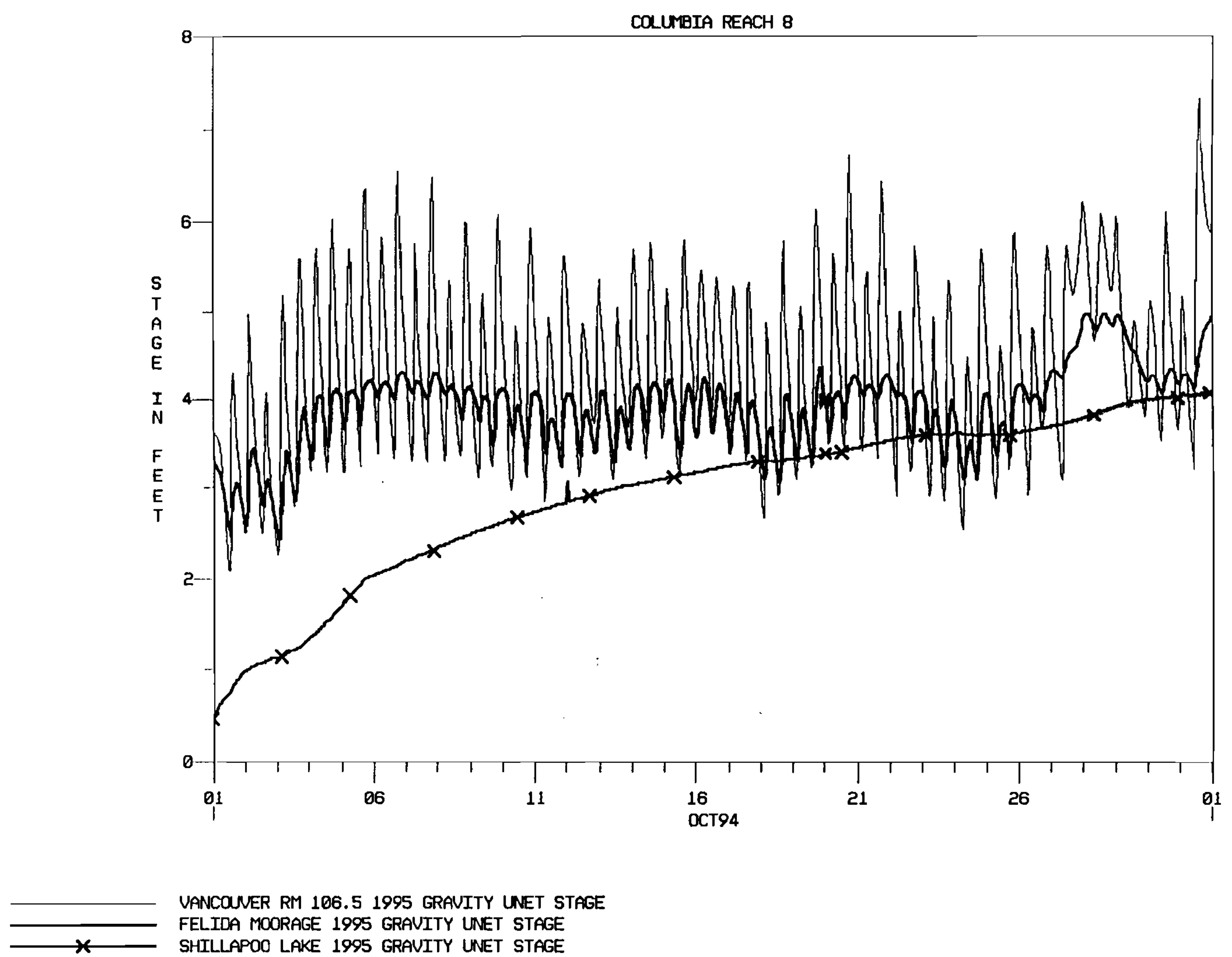


Channel E–I

Scale: Vertical 1"=10'
Horizontal 1"=20'

SHILLAPOO LAKE RESTORATION **Channel E–I**	
File: CHAN–E1.DWG	Date: JUNE 30, 1998
Northwest Hydraulic Consutlants Tukwila, WA	U.S. Army Corps of Engineers Portland District

Figure 5.18



Recommended Alternative. Simulation Results for Filling, October 1994

Figure 5.19

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1994 - May 1995
Forebay Stage

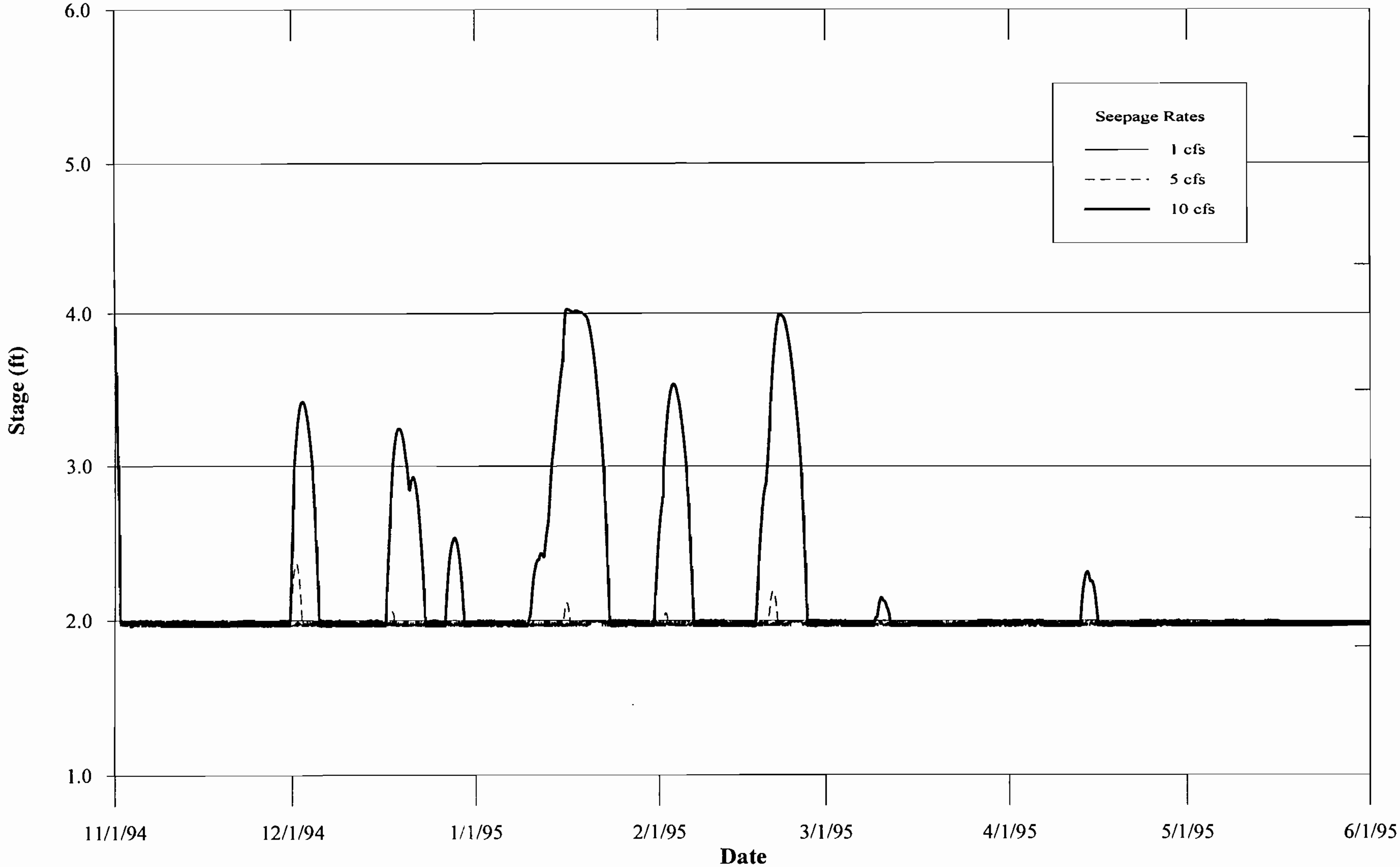


Figure 5.20

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1994 - May 1995
Cell 1 Stage

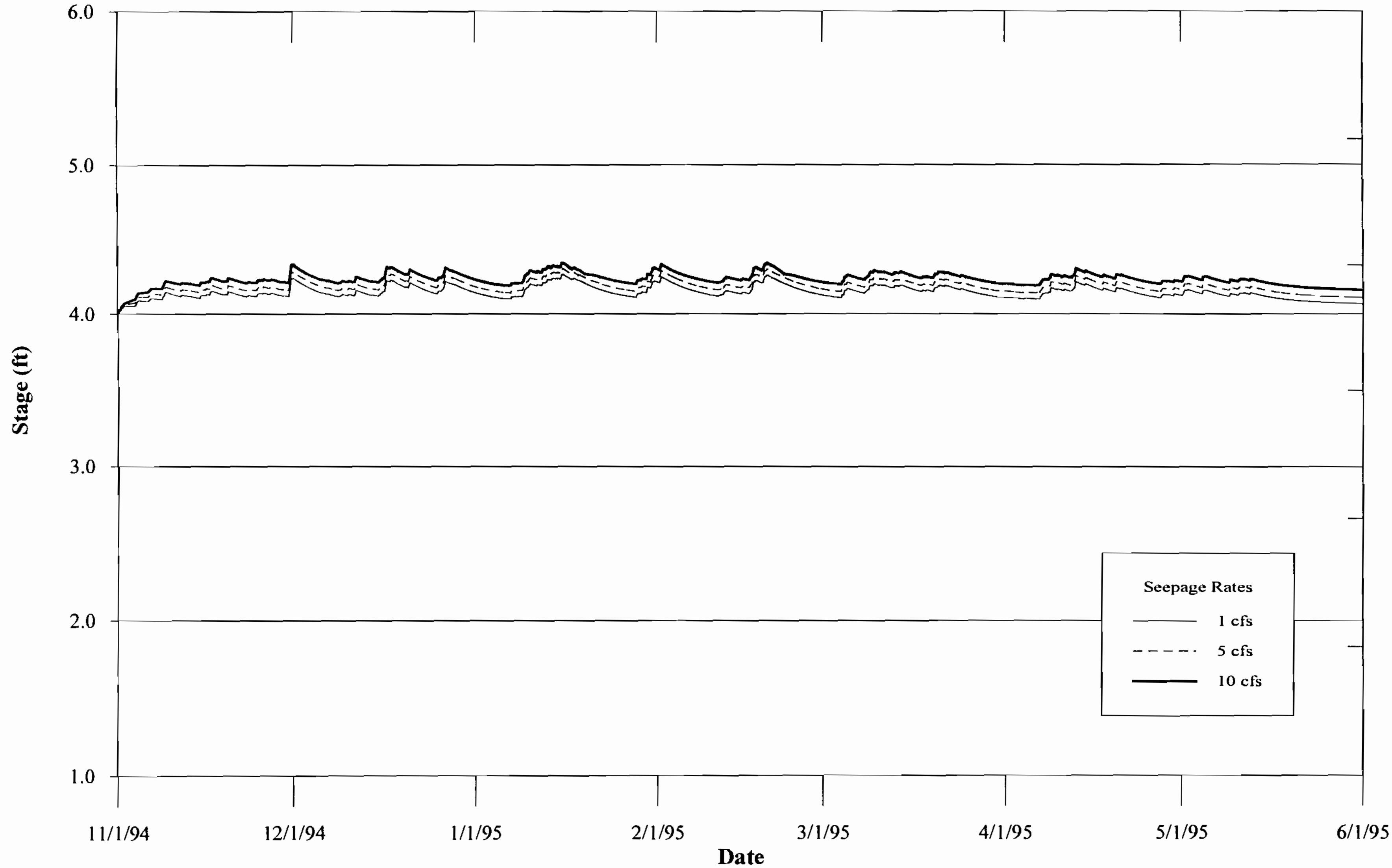


Figure 5.21

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1994 - May 1995
Cell 2 Stage

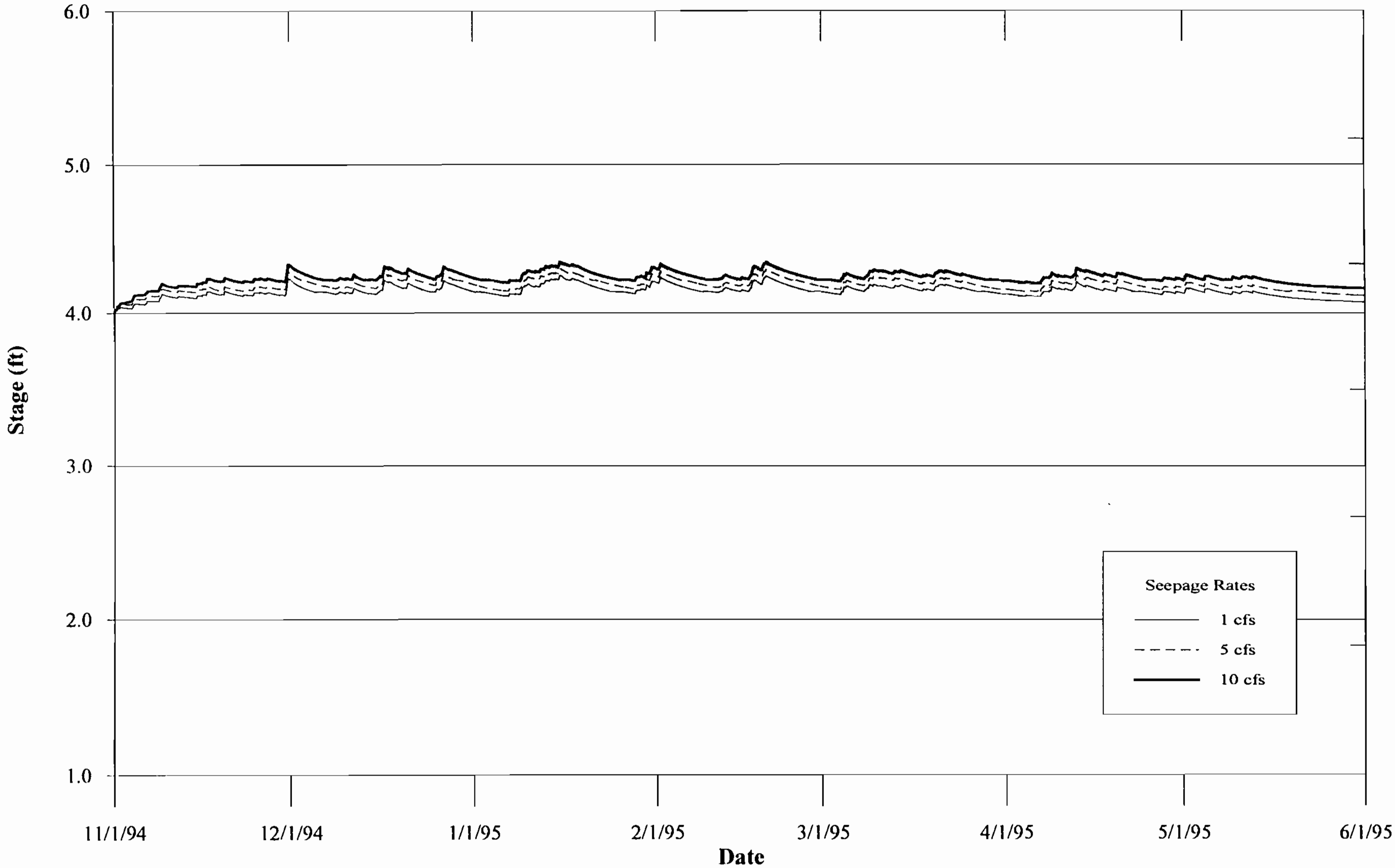


Figure 5.22

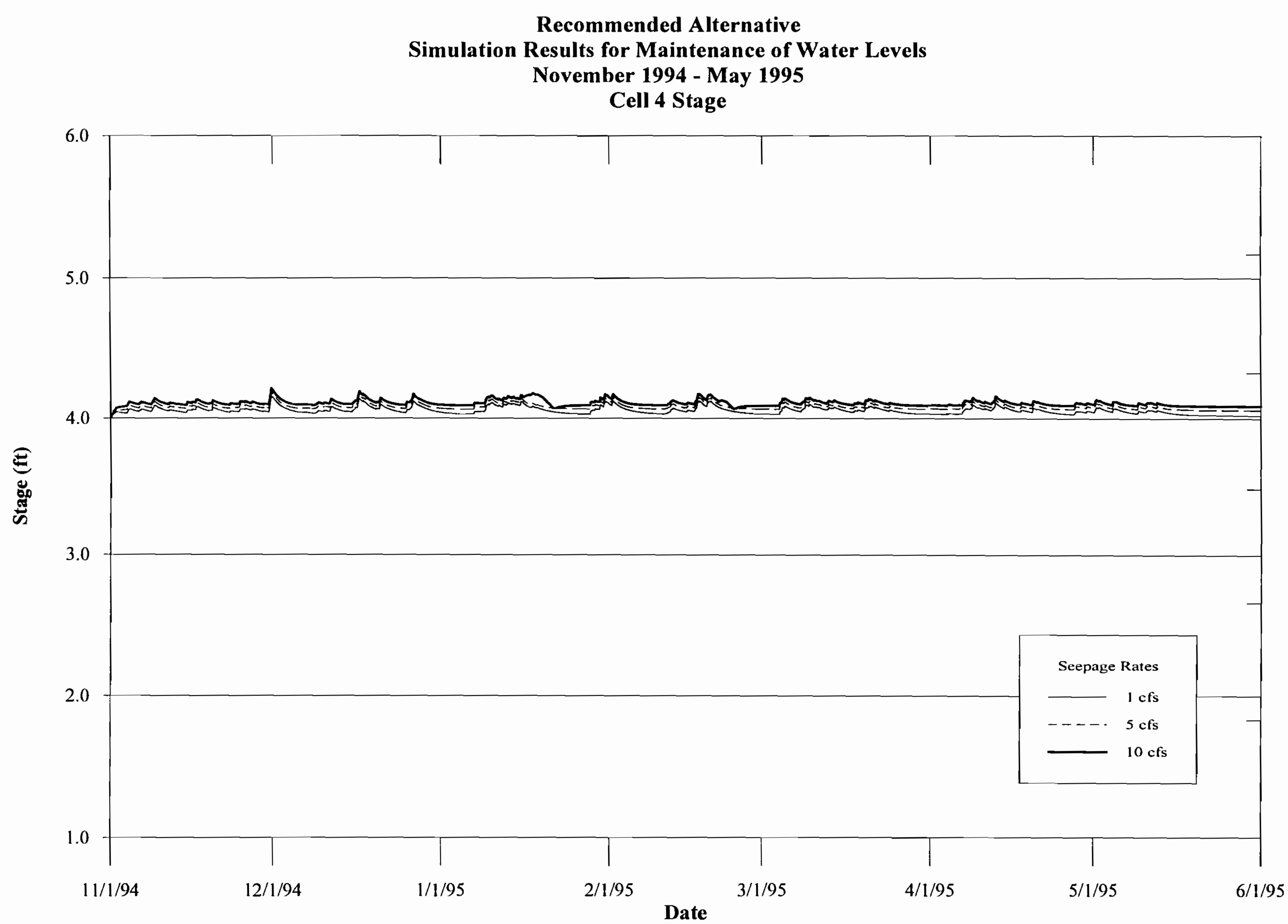

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1994 - May 1995
Cell 4 Stage
6.0
5.0
4.0
3.0
2.0
1.0
Stage (ft)
11/1/94
12/1/94
1/1/95
2/1/95
3/1/95
4/1/95
5/1/95
6/1/95
Date
Seepage Rates
1 cfs
5 cfs
10 cfs

Figure 5.23

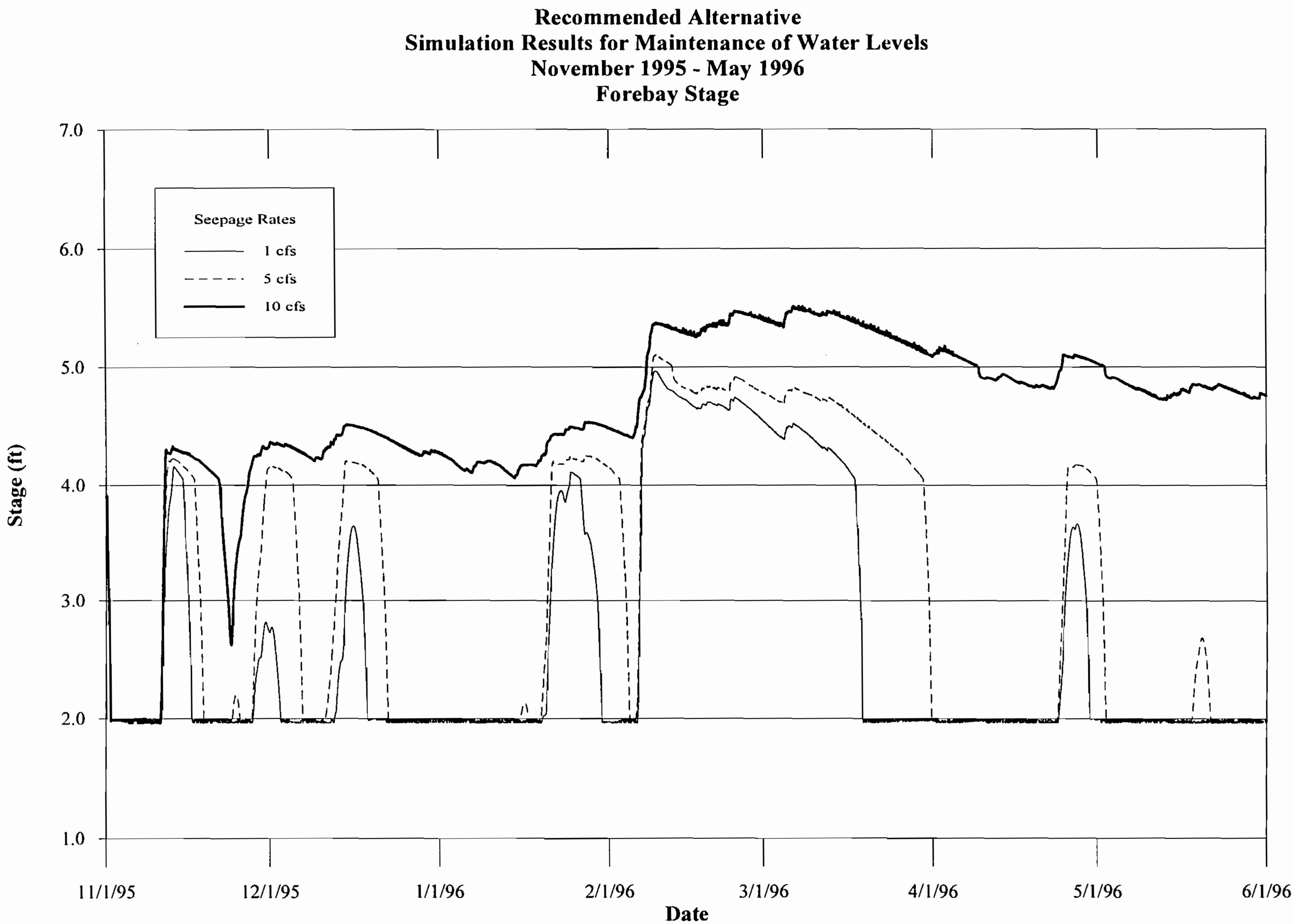

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1995 - May 1996
Forebay Stage
Seepage Rates
1 cfs
5 cfs
10 cfs
Stage (ft)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
11/1/95
12/1/95
1/1/96
2/1/96
3/1/96
4/1/96
5/1/96
6/1/96
Date

Figure 5.24

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1995 - May 1996
Cell 1 Stage

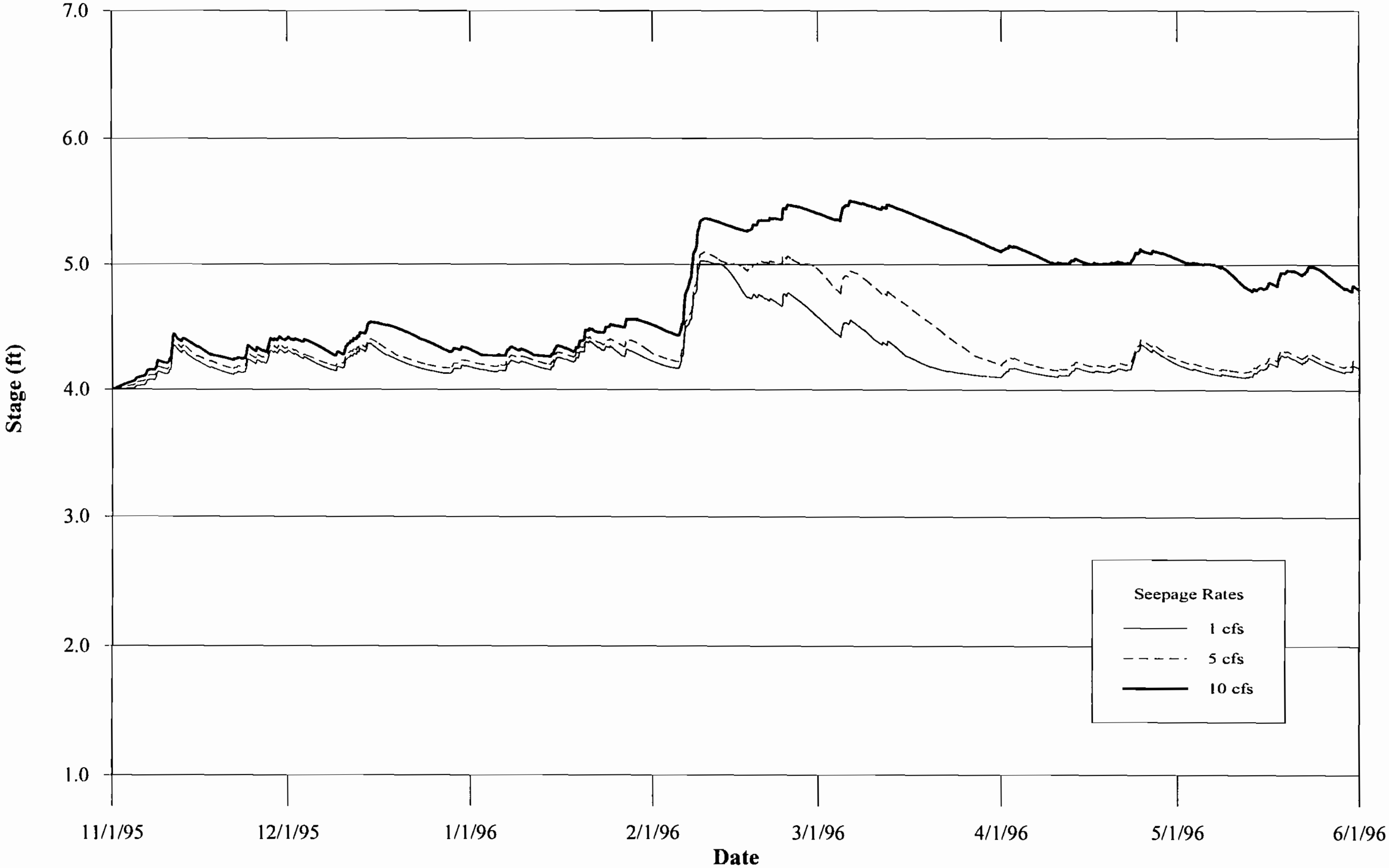
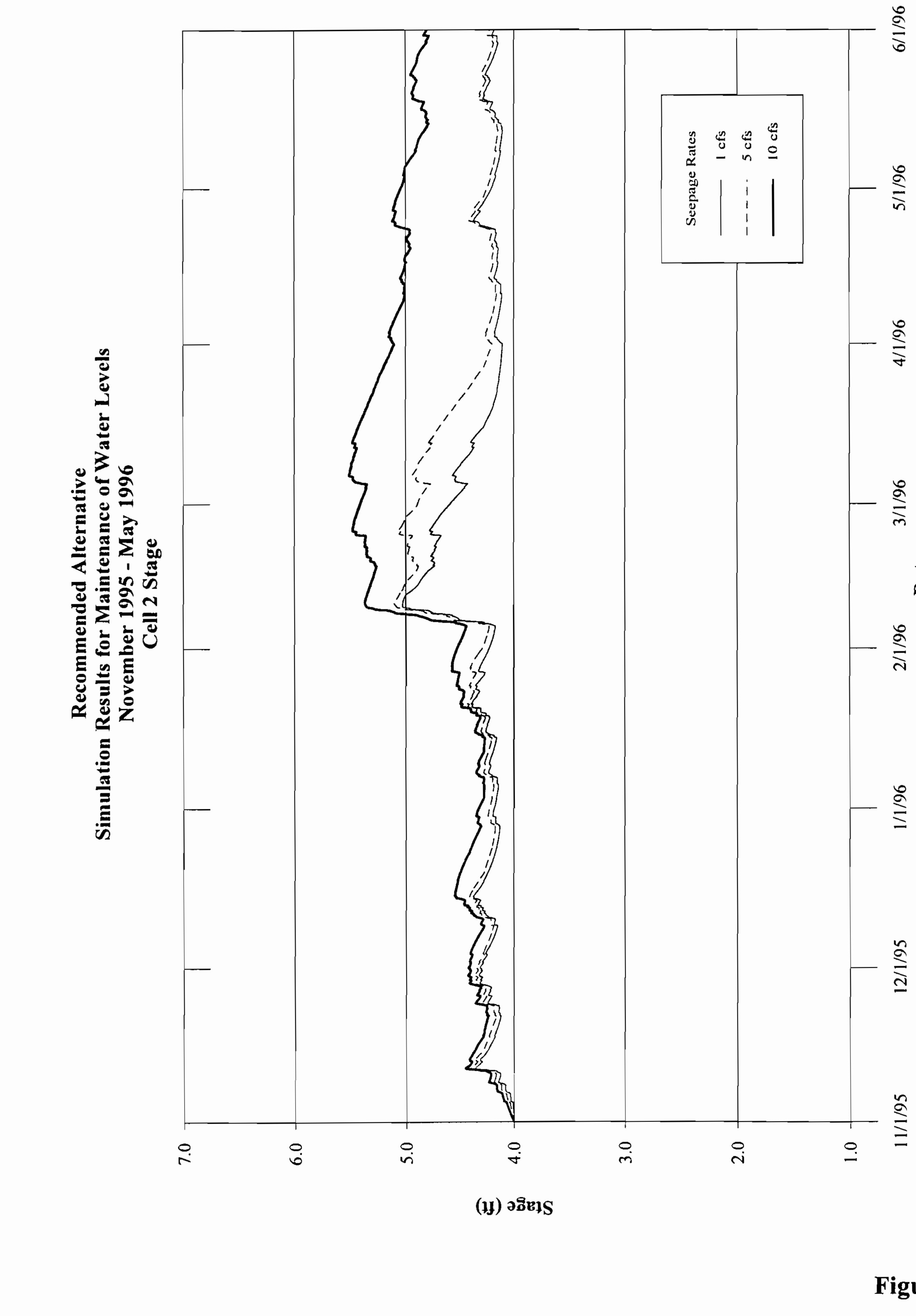


Figure 5.25



Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1995 - May 1996
Cell 2 Stage
Stage (ft)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
11/1/95
12/1/95
1/1/96
2/1/96
3/1/96
4/1/96
5/1/96
6/1/96
Date
Seepage Rates
1 cfs
5 cfs
10 cfs

Figure 5.26

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1995 - May 1996
Cell 4 Stage

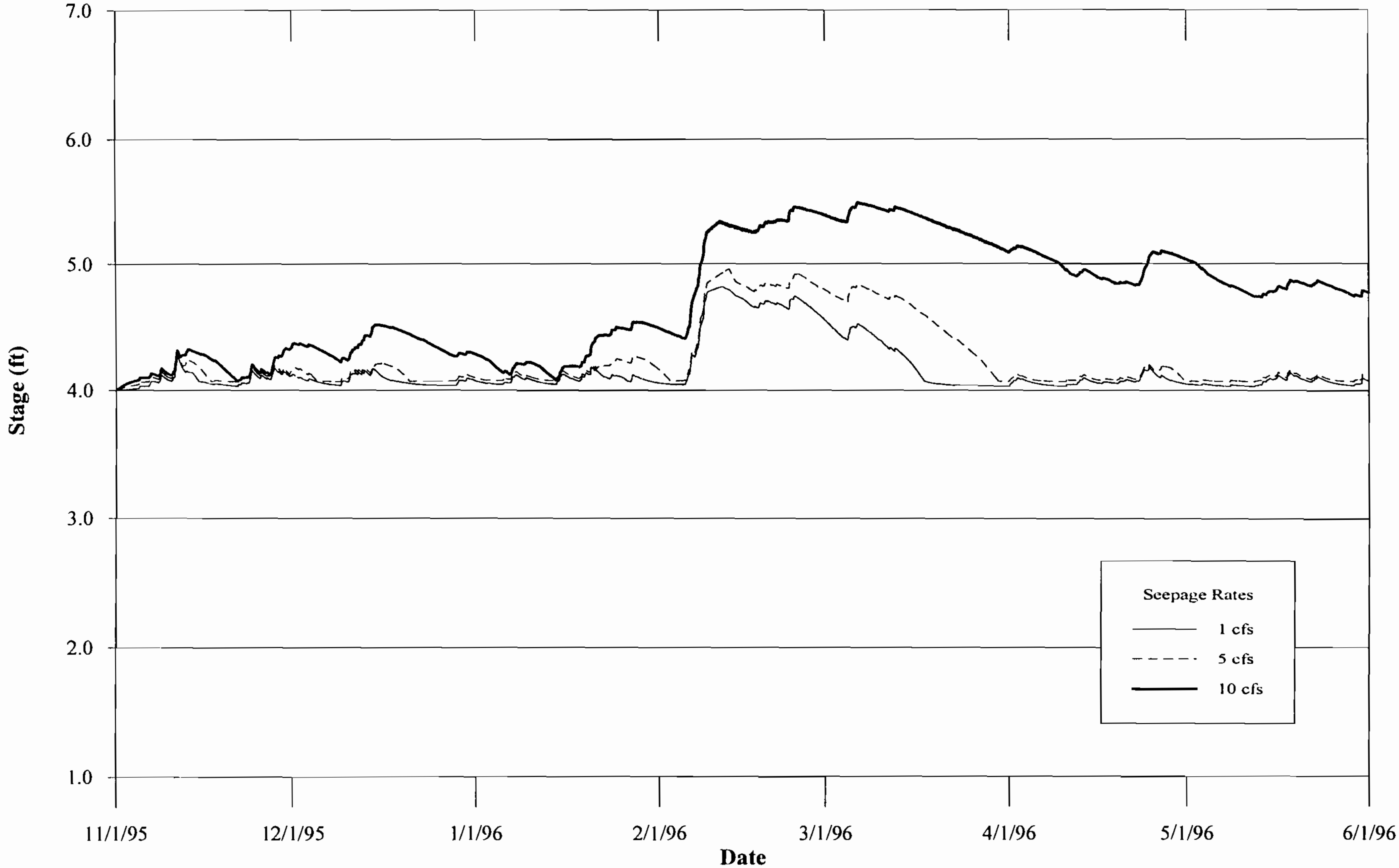
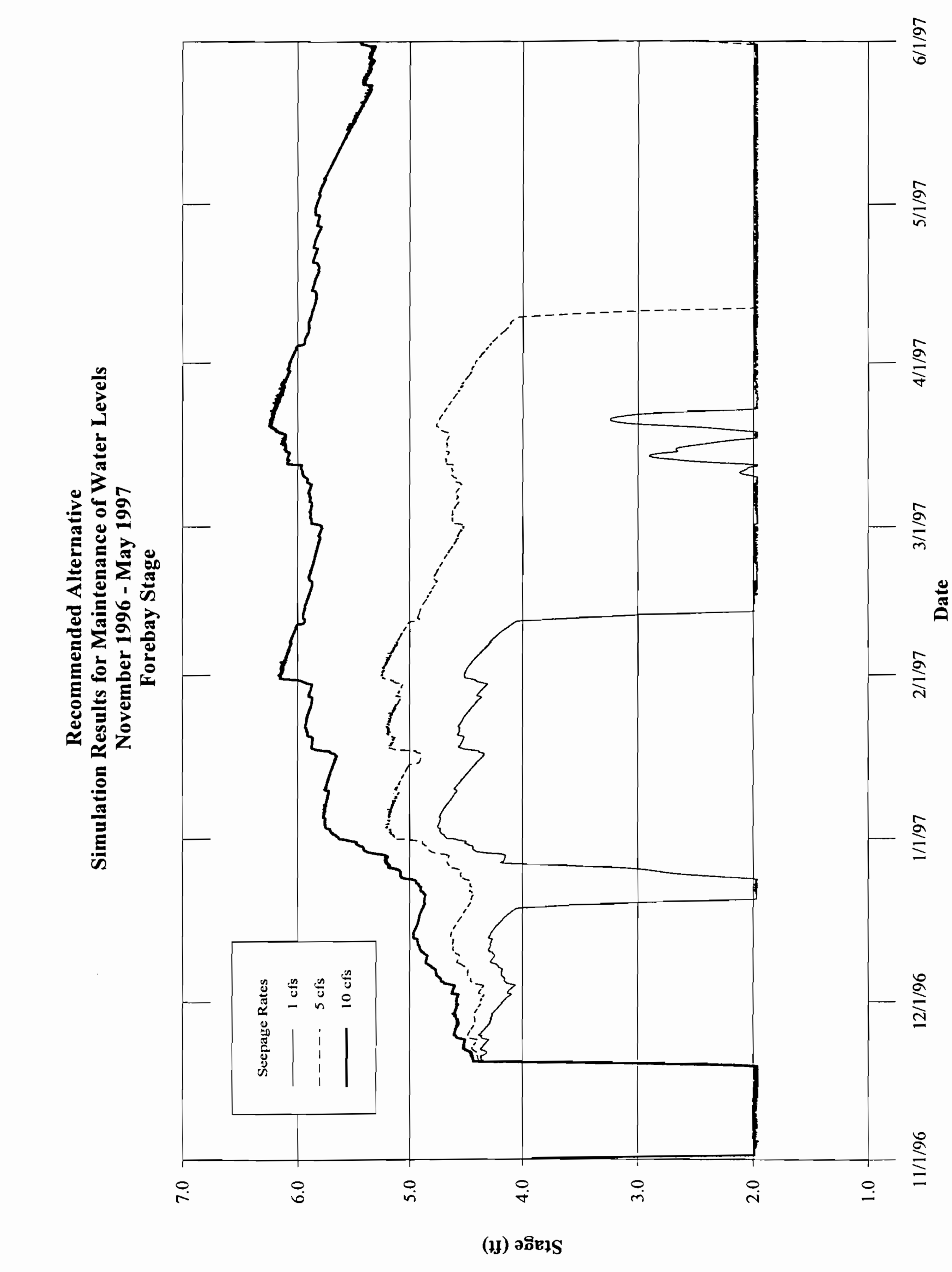


Figure 5.27



Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1996 - May 1997
Forebay Stage
Seepage Rates
1 cfs
5 cfs
10 cfs
Stage (ft)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
Date
11/1/96
12/1/96
1/1/97
2/1/97
3/1/97
4/1/97
5/1/97
6/1/97

Figure 5.28

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1996 - May 1997
Cell 1 Stage

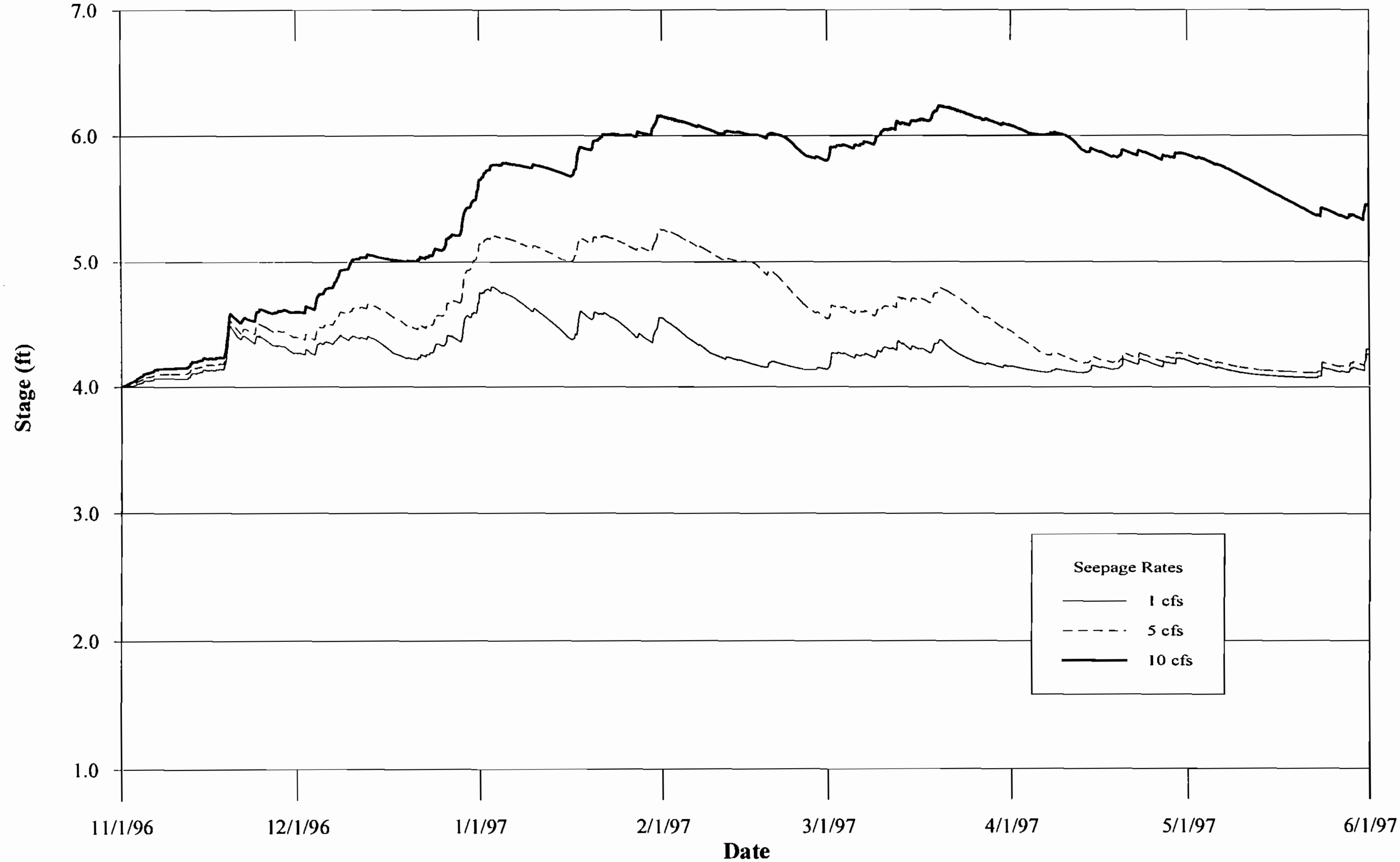
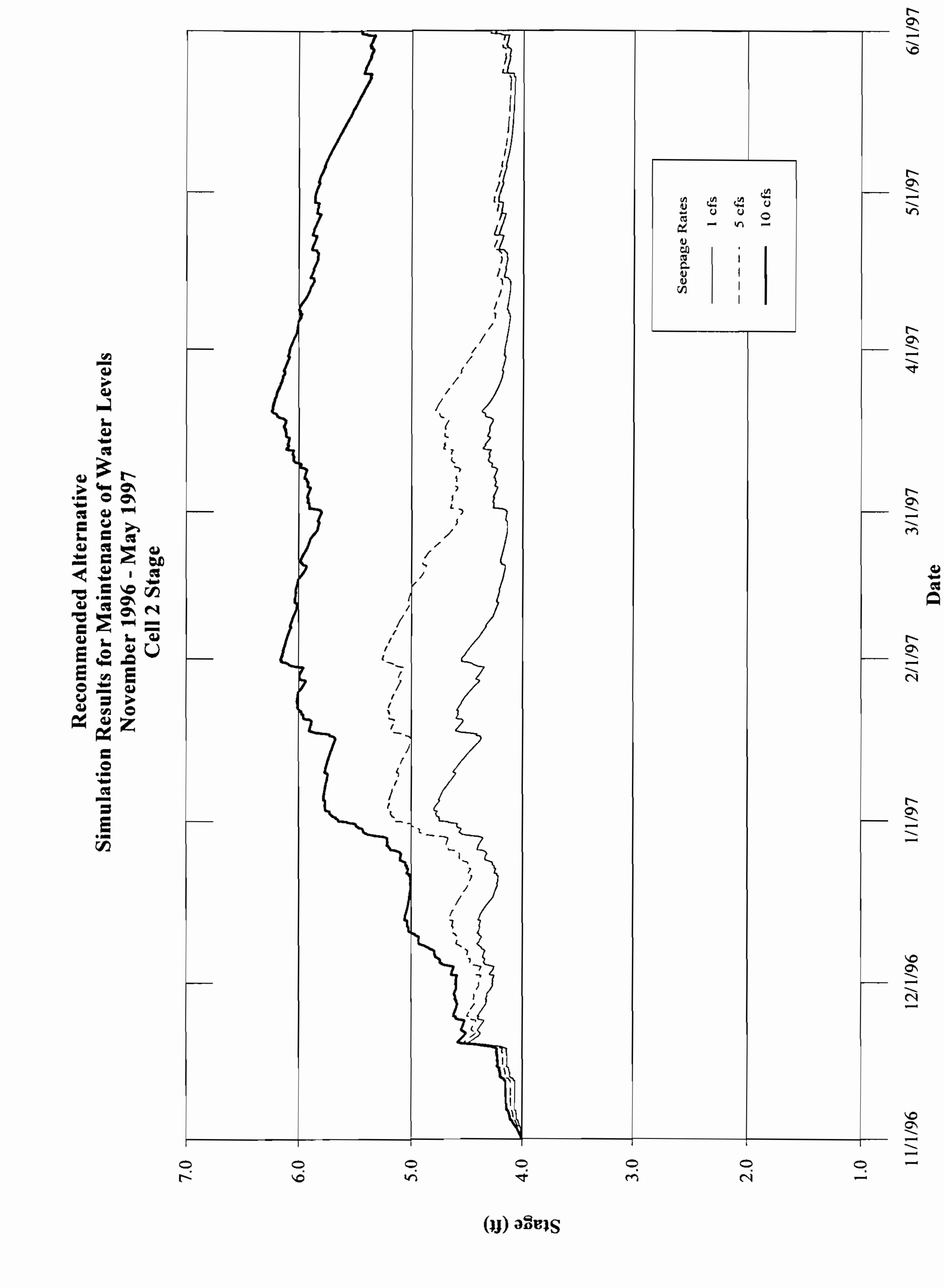


Figure 5.29



Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1996 - May 1997
Cell 2 Stage
Stage (ft)
7.0
6.0
5.0
4.0
3.0
2.0
1.0
11/1/96
12/1/96
1/1/97
2/1/97
3/1/97
4/1/97
5/1/97
6/1/97
Date
Seepage Rates
1 cfs
5 cfs
10 cfs

Figure 5.30

Recommended Alternative
Simulation Results for Draining
March through May 1995 (representing June through August 1983)
Forebay Stage

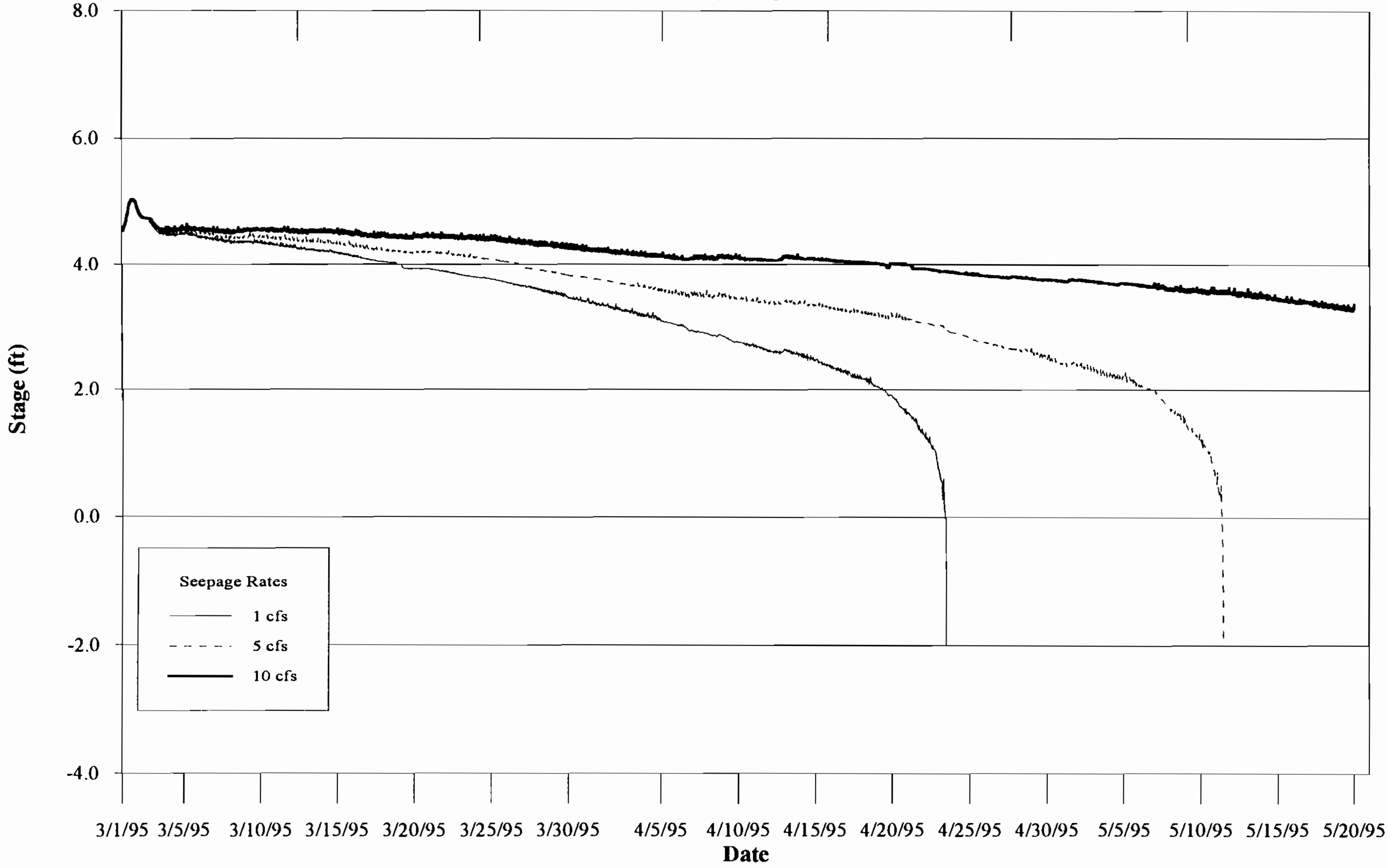


Figure 5.32

Recommended Alternative
Simulation Results for Maintenance of Water Levels
November 1996 - May 1997
Cell 4 Stage

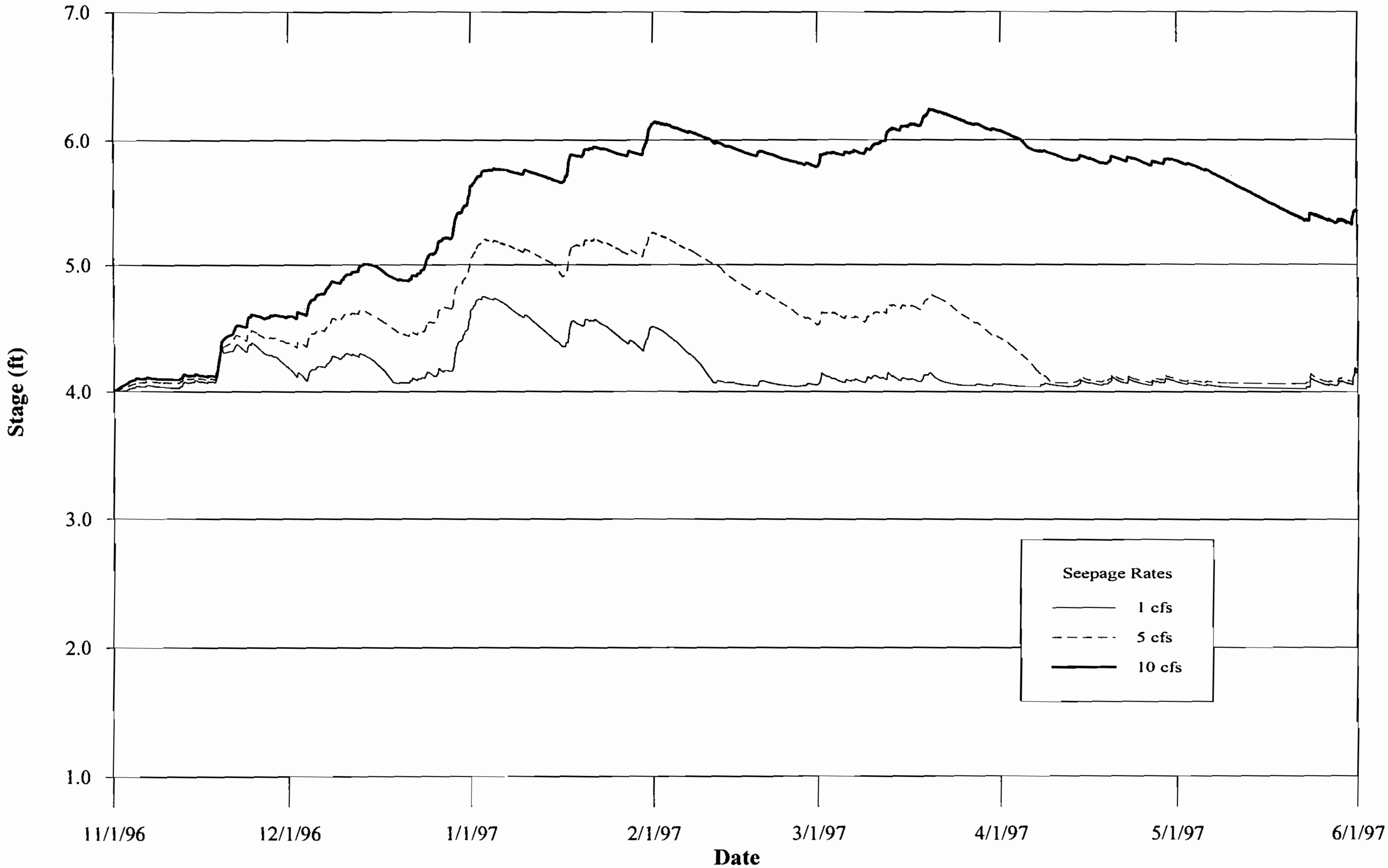
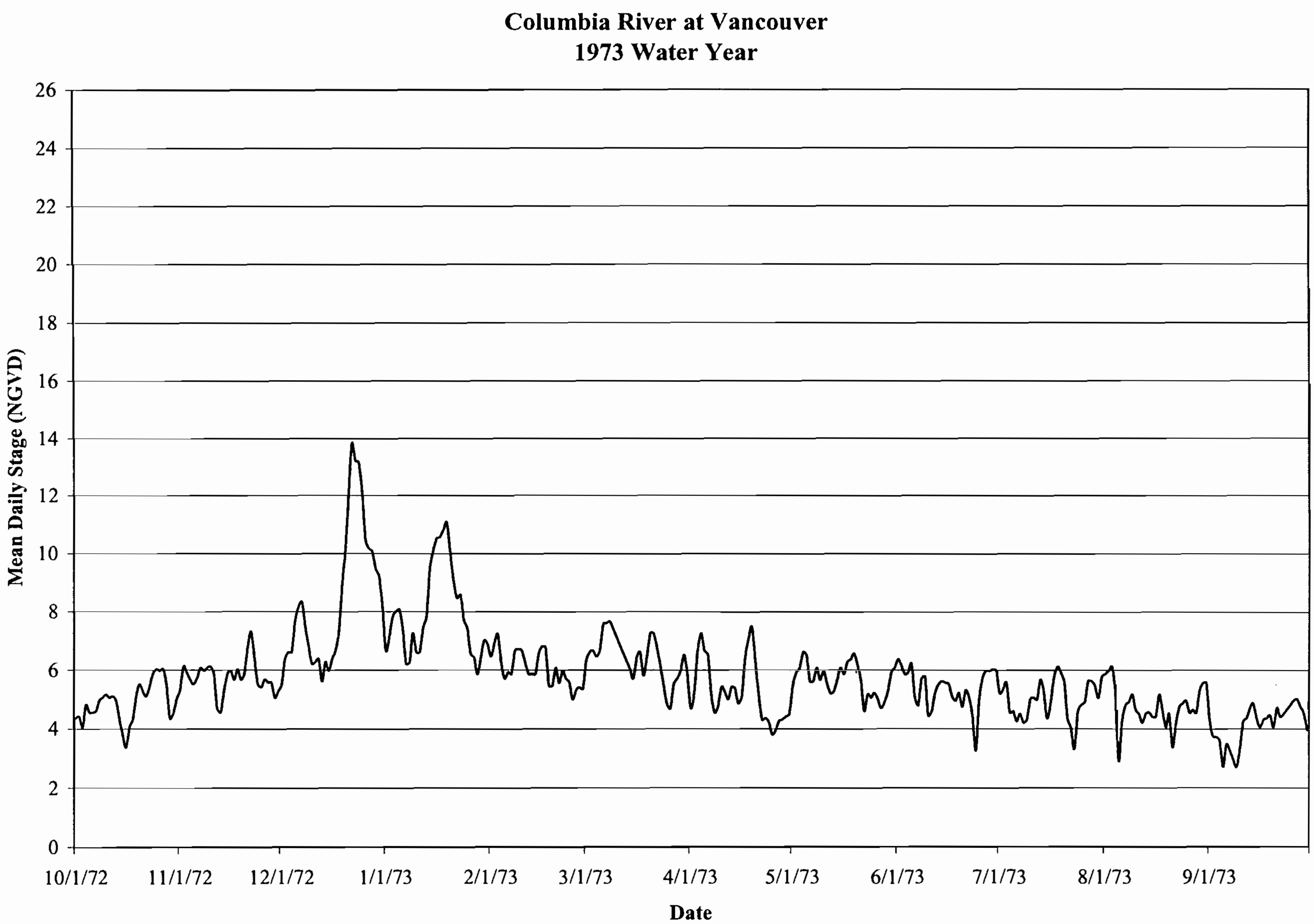


Figure 5.31

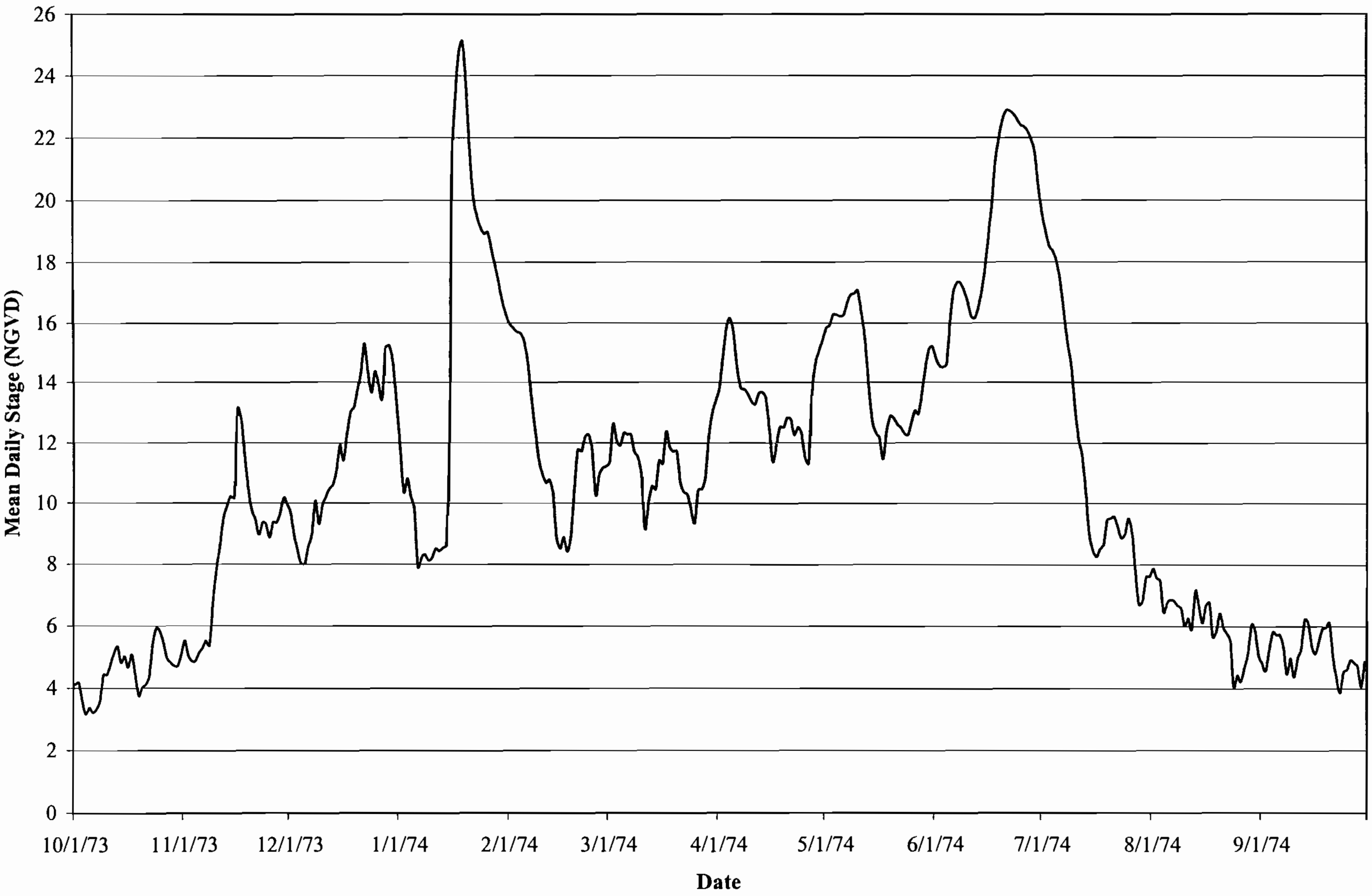
APPENDIX A

Mean Daily Stage Data
Columbia River at Vancouver
1973-1990, 1993-1997

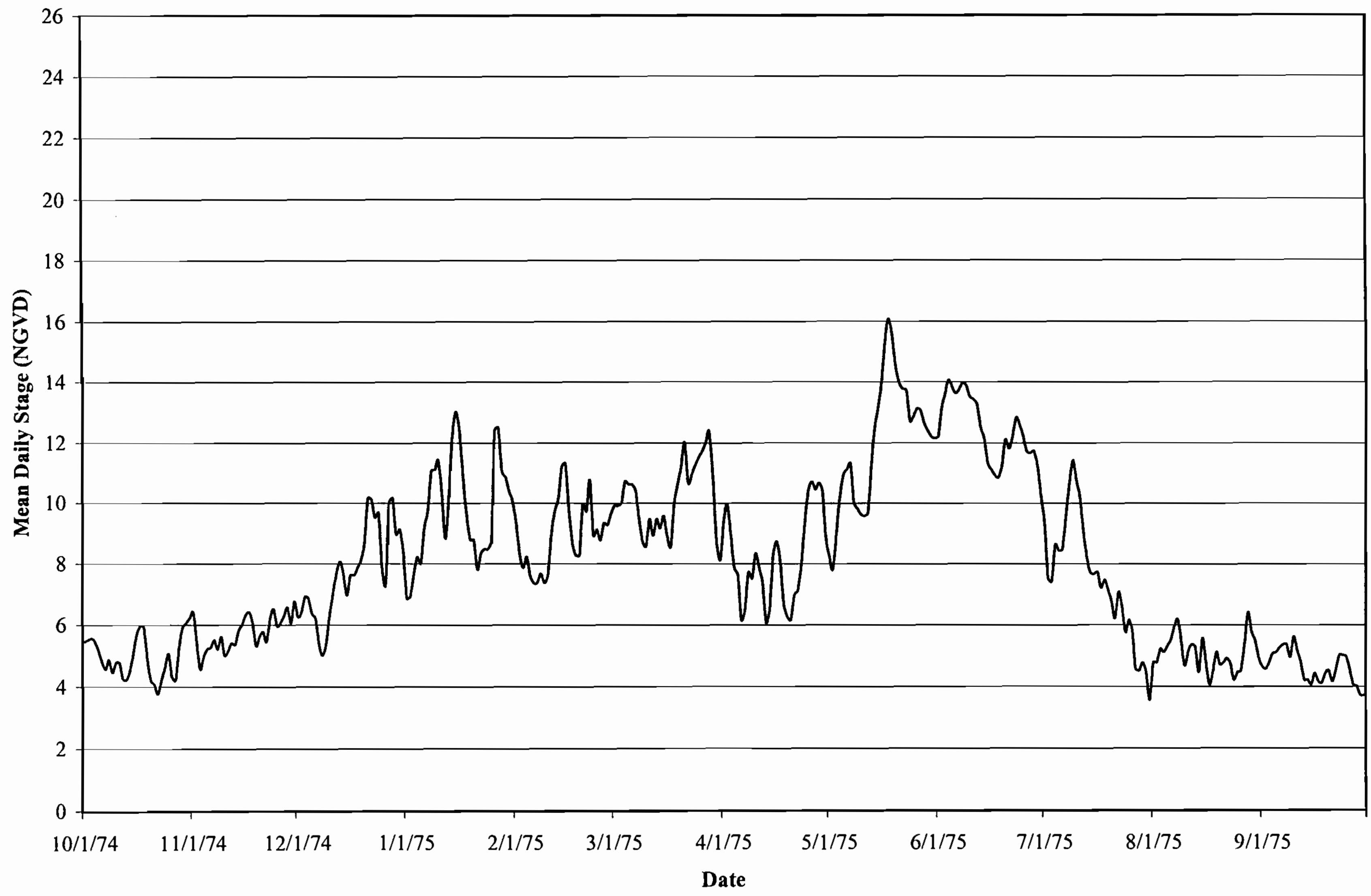


Columbia River at Vancouver
1973 Water Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/72
11/1/72
12/1/72
1/1/73
2/1/73
3/1/73
4/1/73
5/1/73
6/1/73
7/1/73
8/1/73
9/1/73
Date

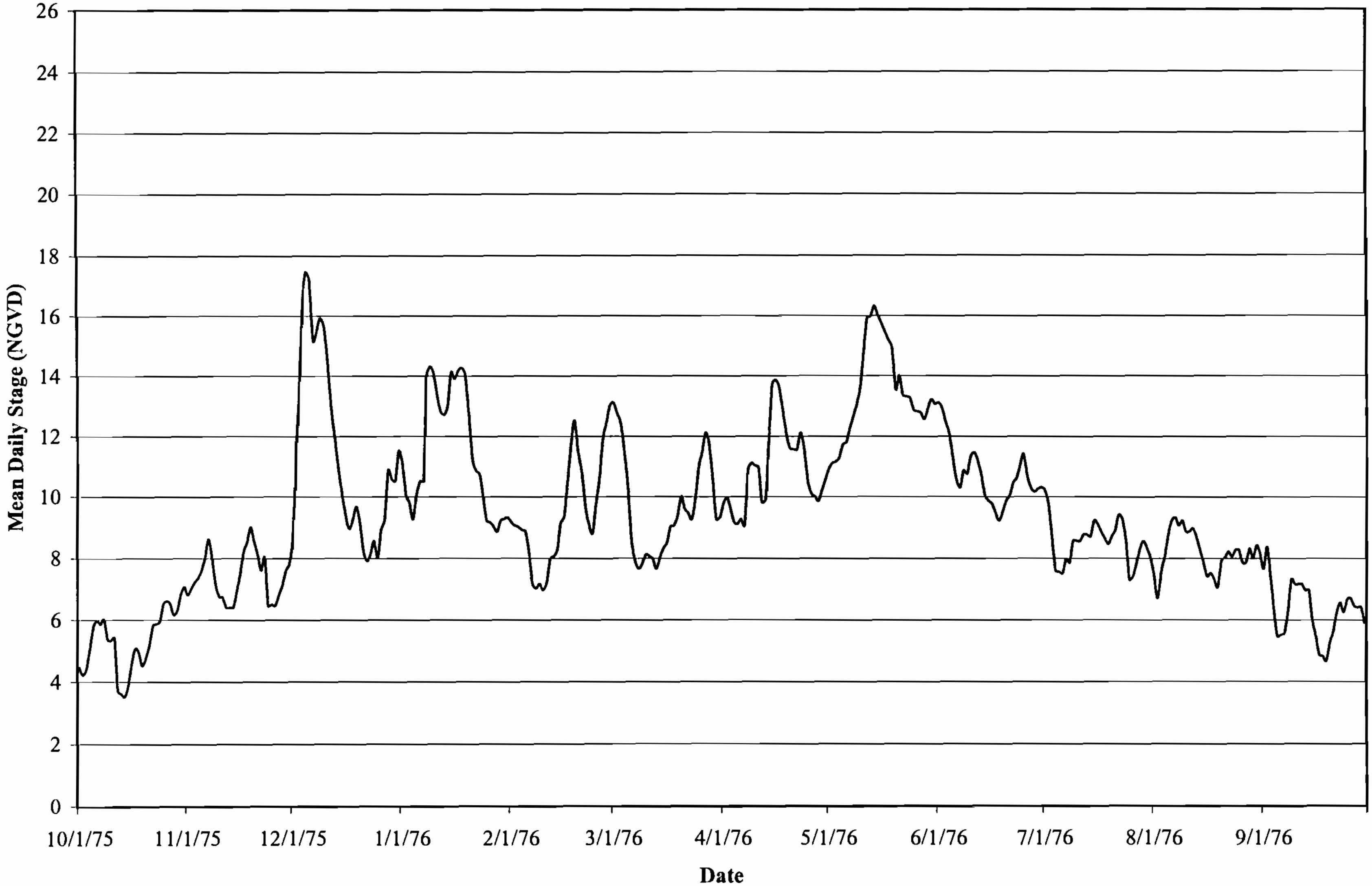
Columbia River at Vancouver
1974 Water Year

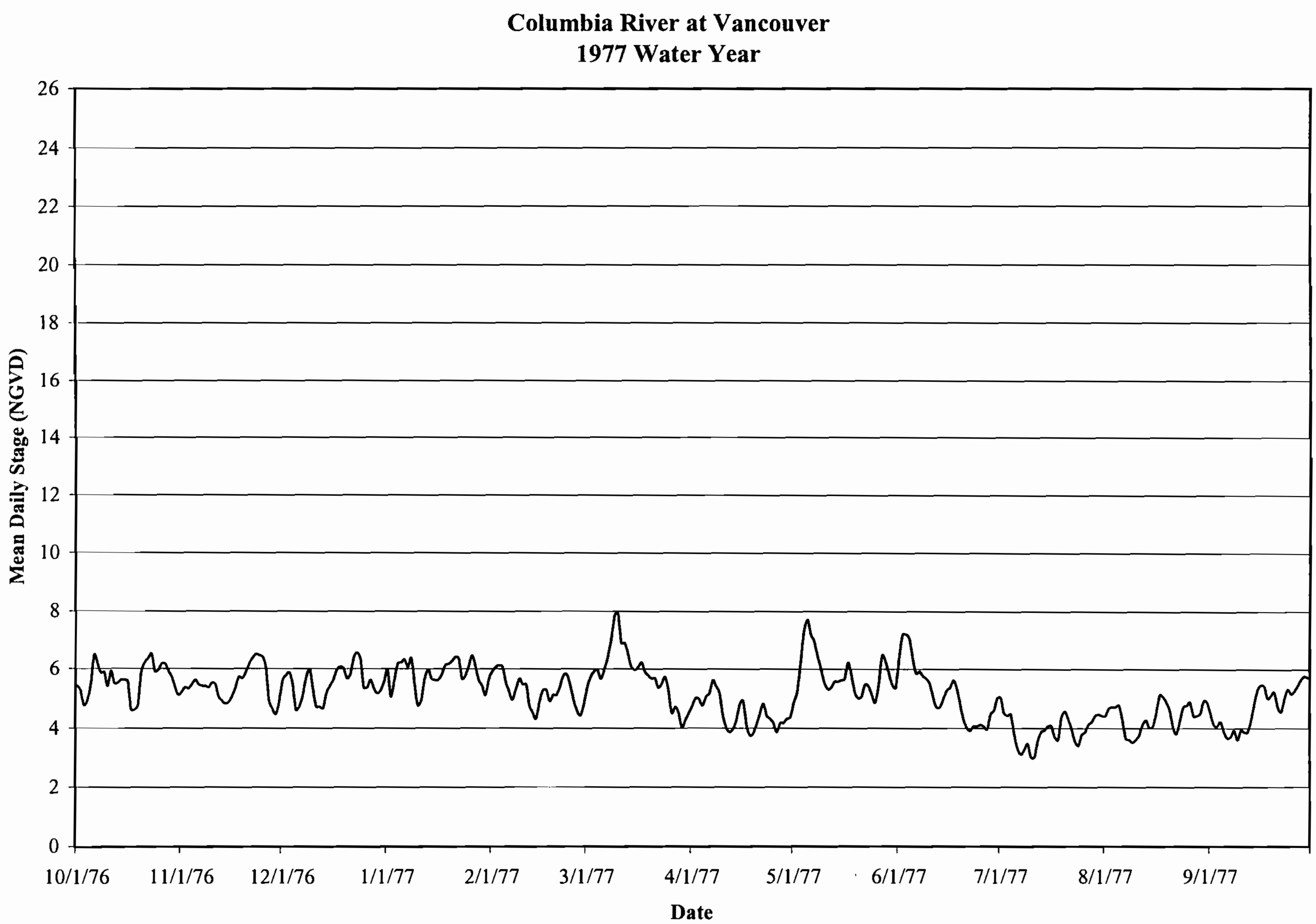


Columbia River at Vancouver
1975 Water Year



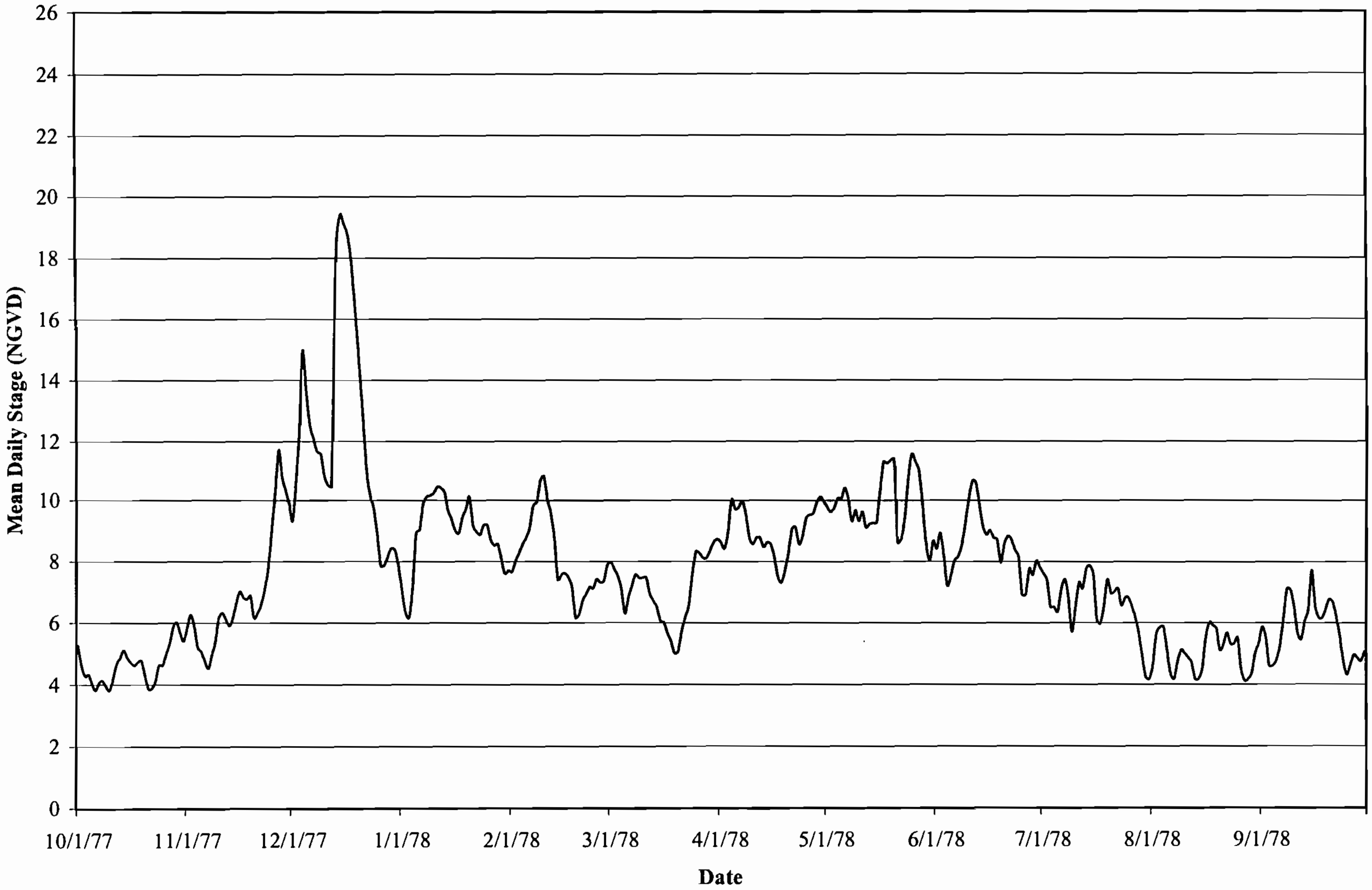
Columbia River at Vancouver
1976 Water Year

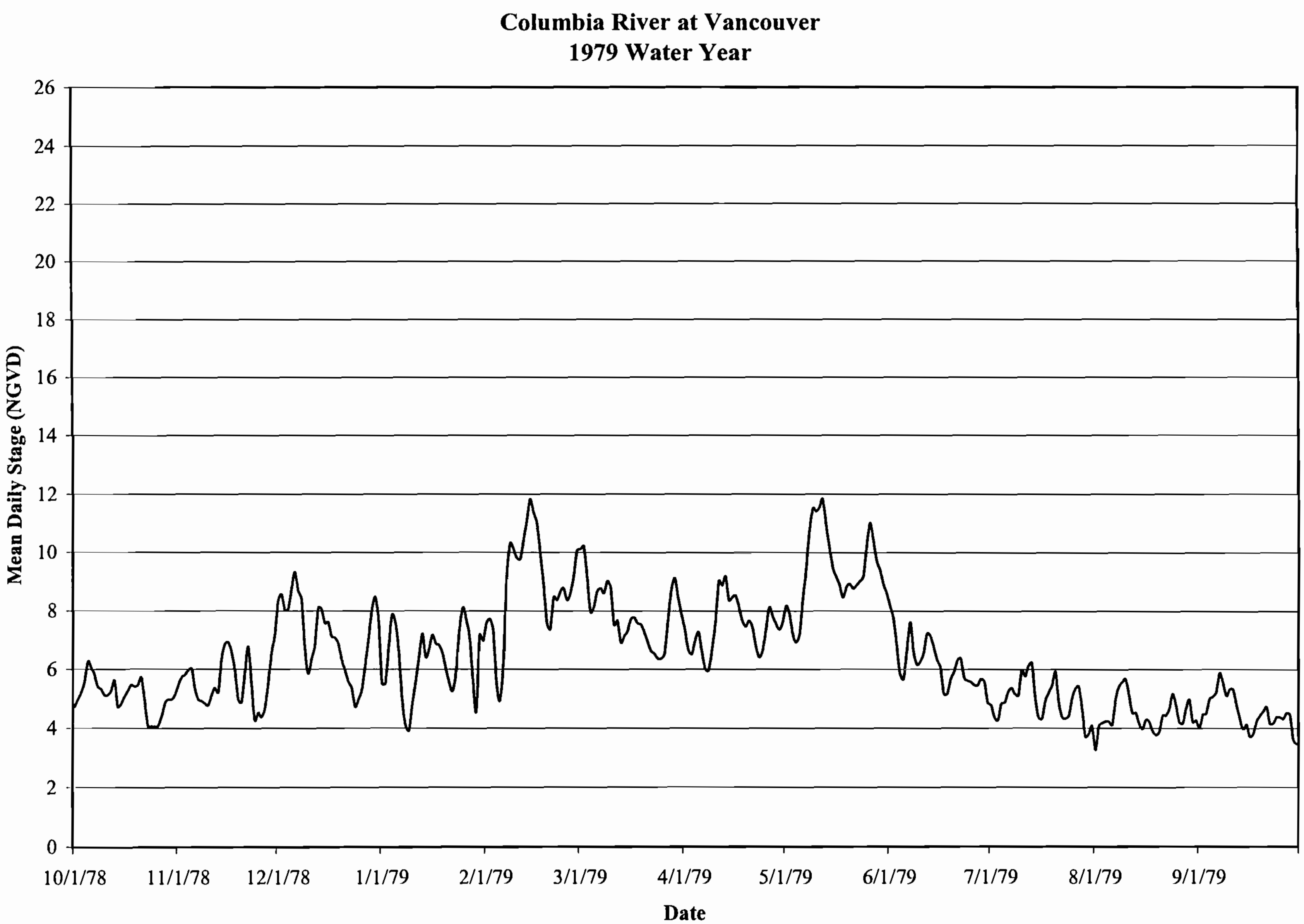




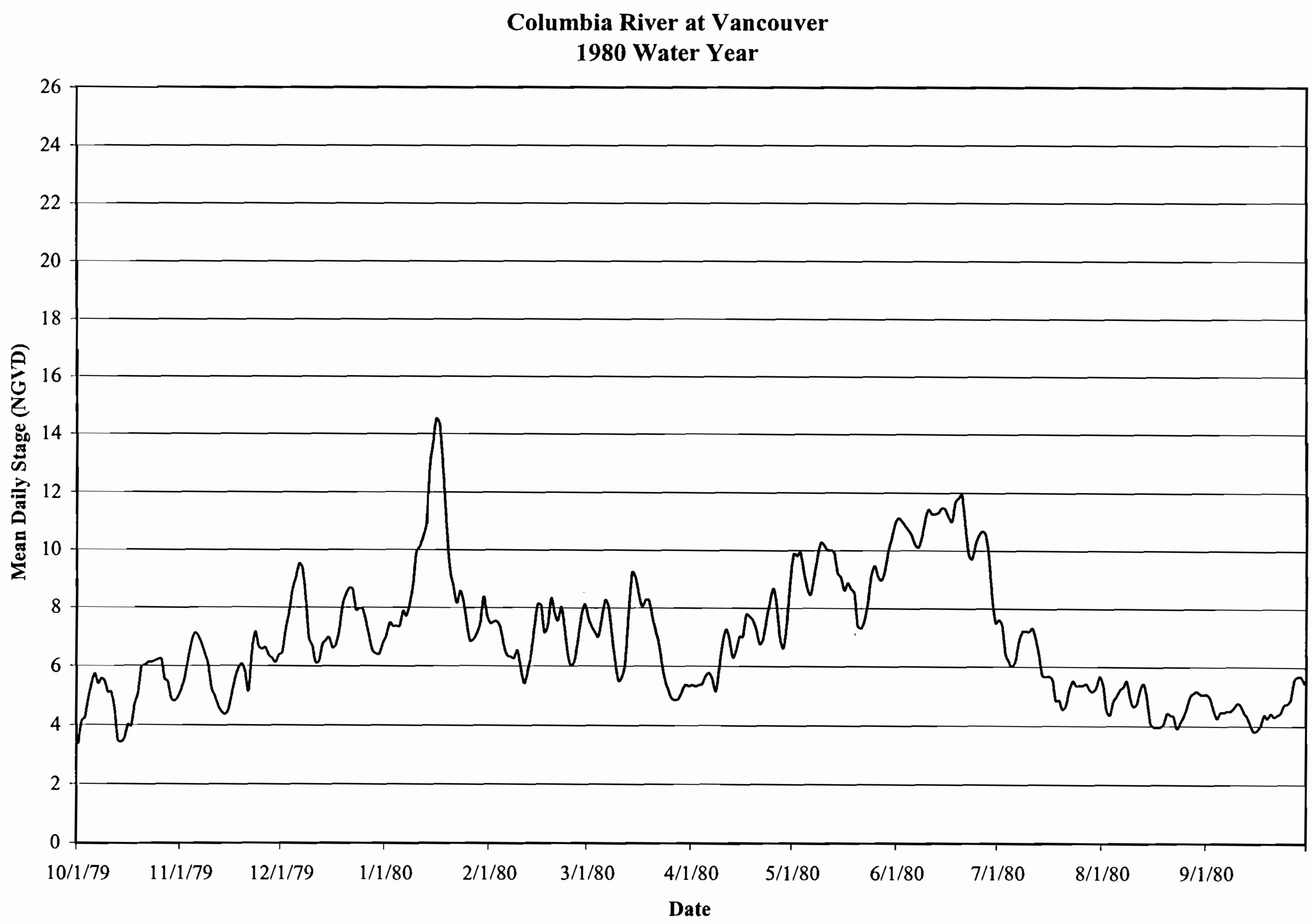
Columbia River at Vancouver
1977 Water Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/76
11/1/76
12/1/76
1/1/77
2/1/77
3/1/77
4/1/77
5/1/77
6/1/77
7/1/77
8/1/77
9/1/77
Date

Columbia River at Vancouver
1978 Water Year

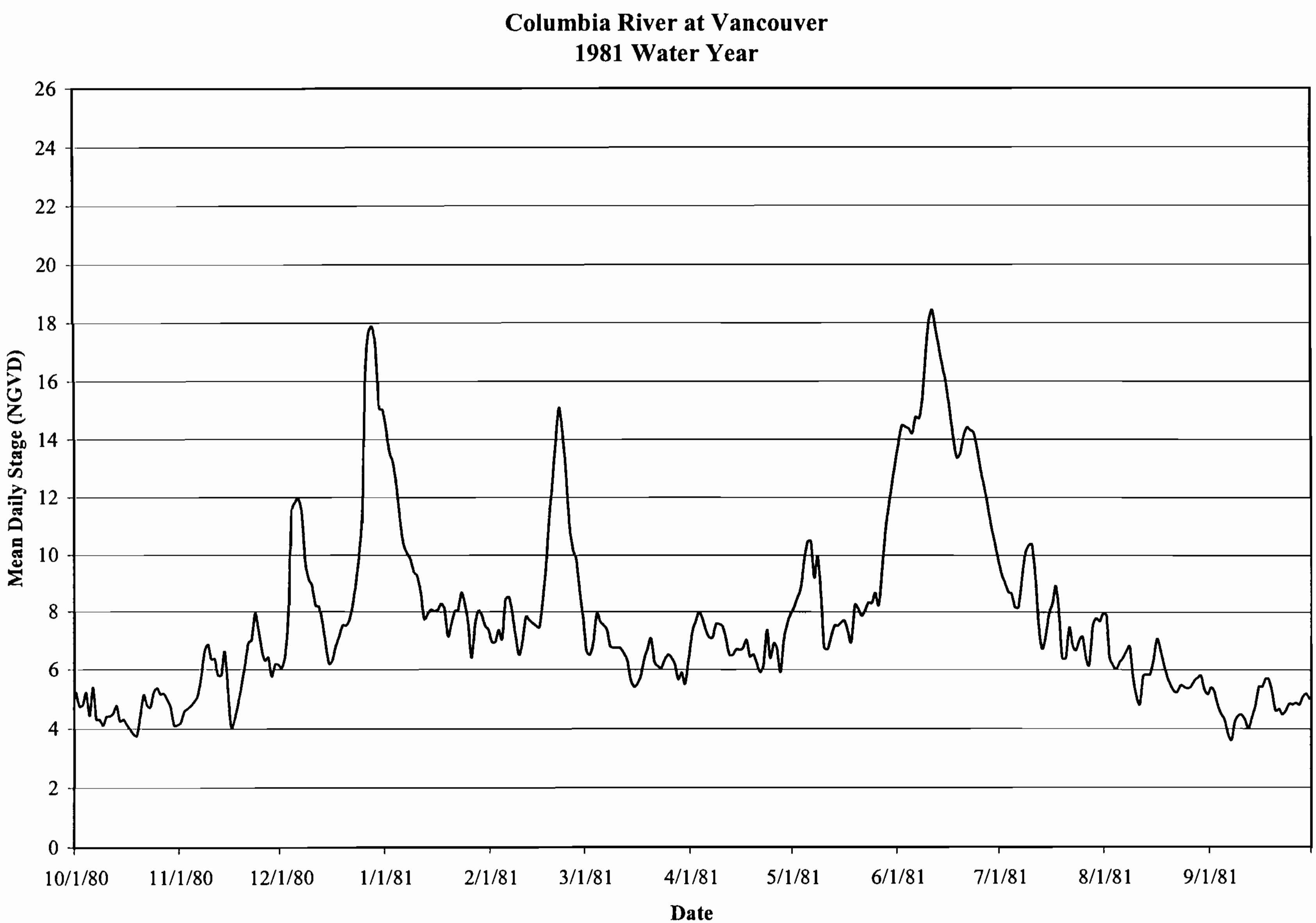




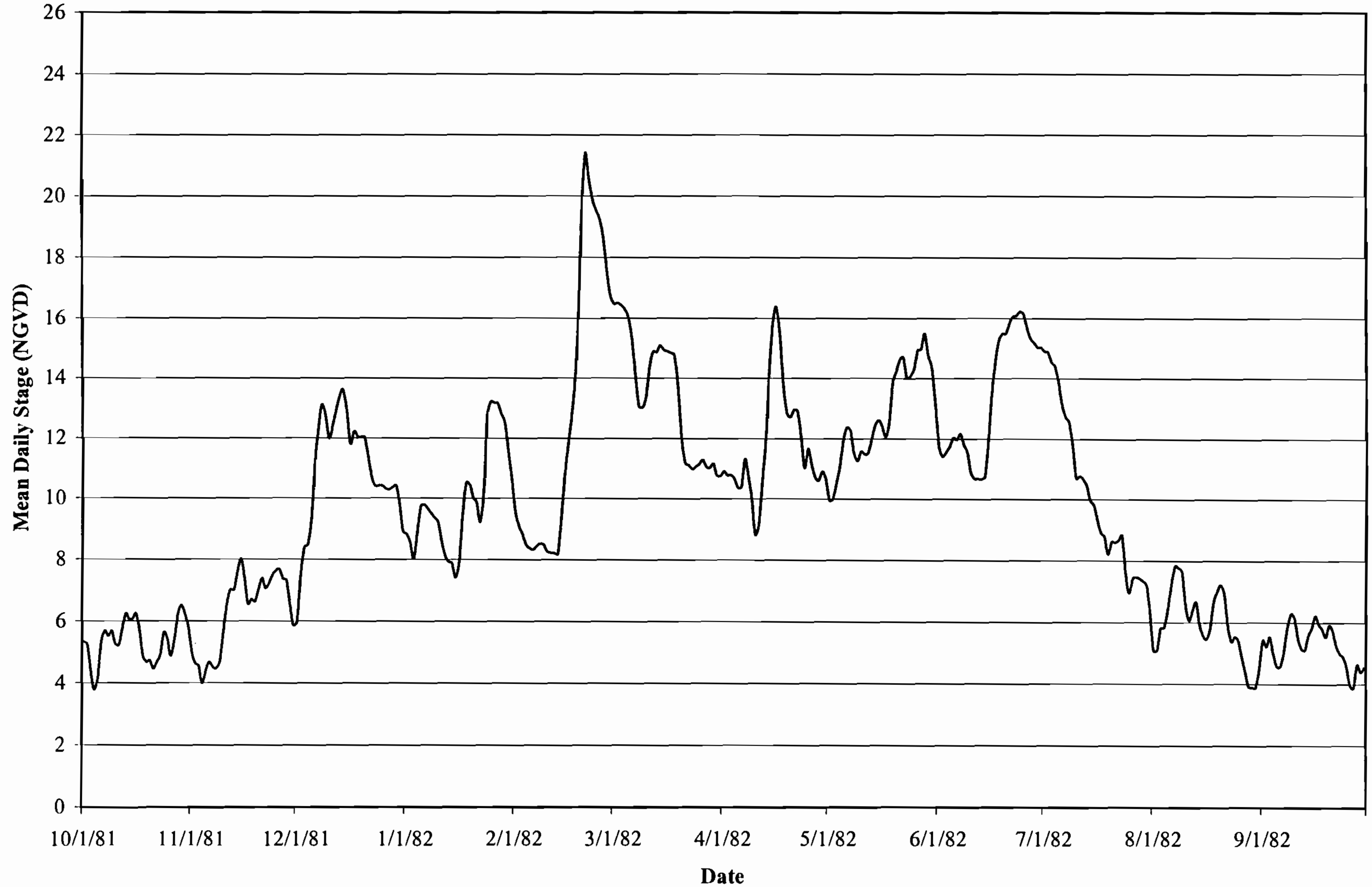
Columbia River at Vancouver
1979 Water Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/78
11/1/78
12/1/78
1/1/79
2/1/79
3/1/79
4/1/79
5/1/79
6/1/79
7/1/79
8/1/79
9/1/79
Date

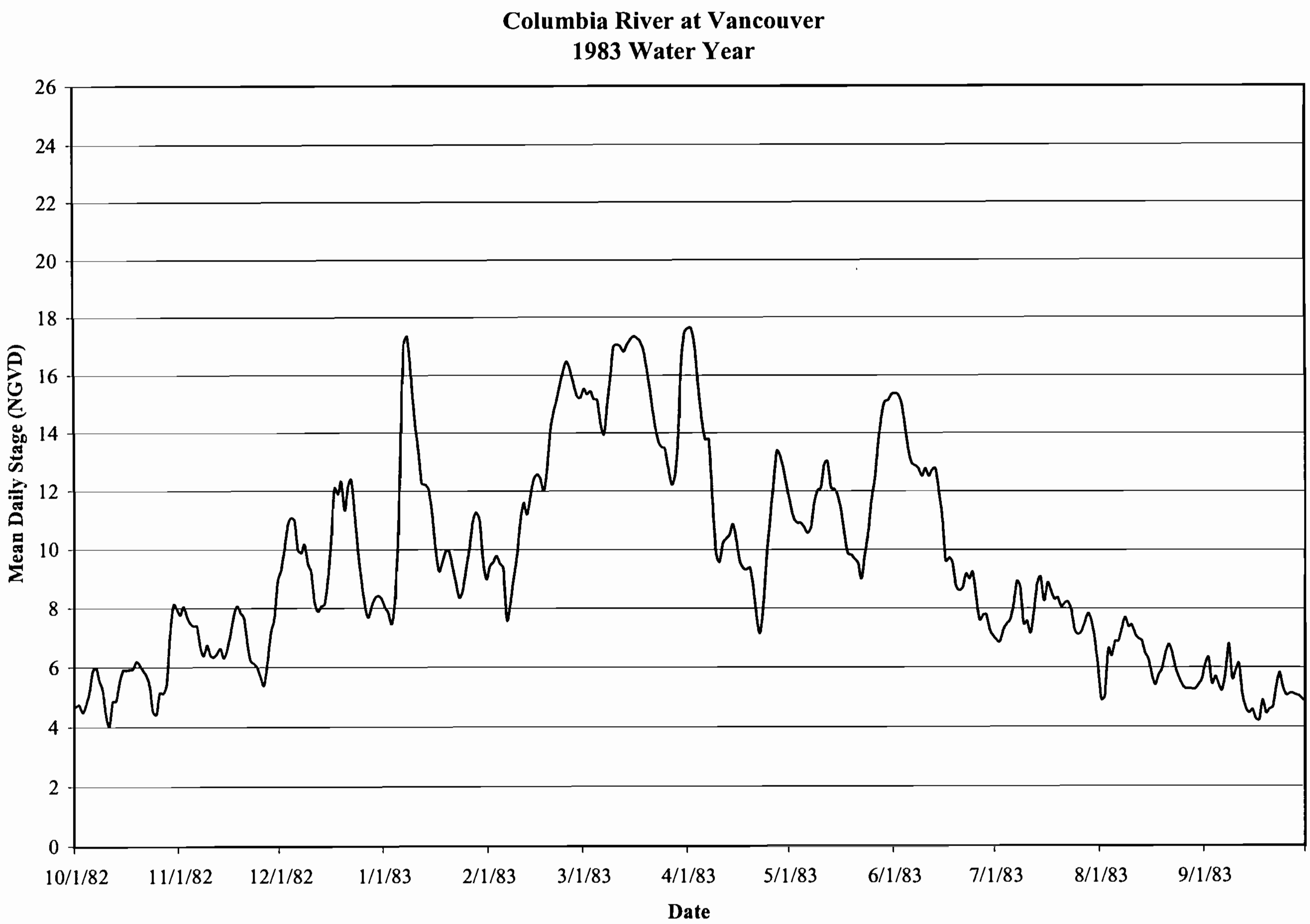


Columbia River at Vancouver
1980 Water Year
Mean Daily Stage (NGVD)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
10/1/79
11/1/79
12/1/79
1/1/80
2/1/80
3/1/80
4/1/80
5/1/80
6/1/80
7/1/80
8/1/80
9/1/80
Date

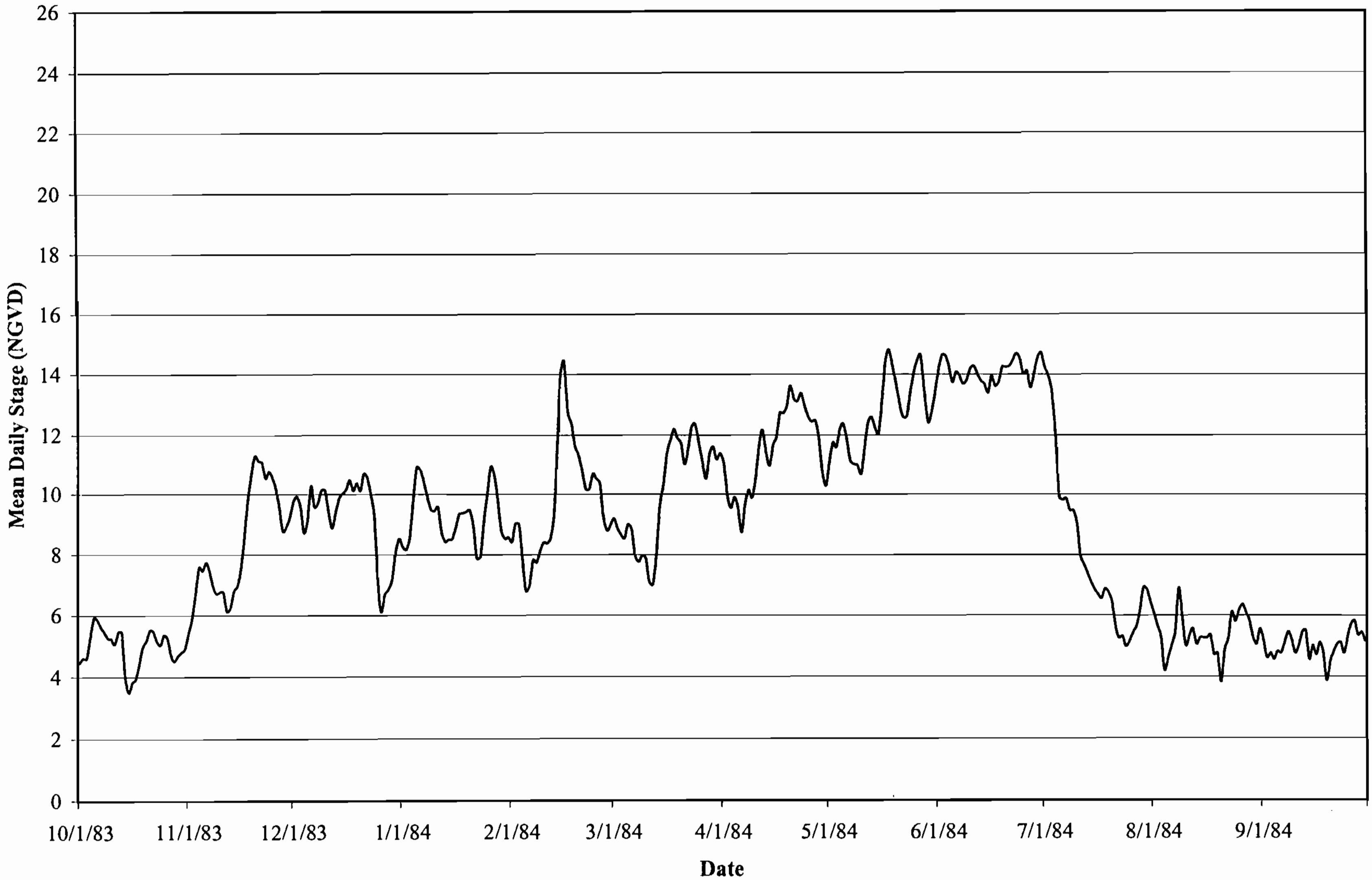

Columbia River at Vancouver
1981 Water Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/80
11/1/80
12/1/80
1/1/81
2/1/81
3/1/81
4/1/81
5/1/81
6/1/81
7/1/81
8/1/81
9/1/81
Date

Columbia River at Vancouver
1982 Water Year

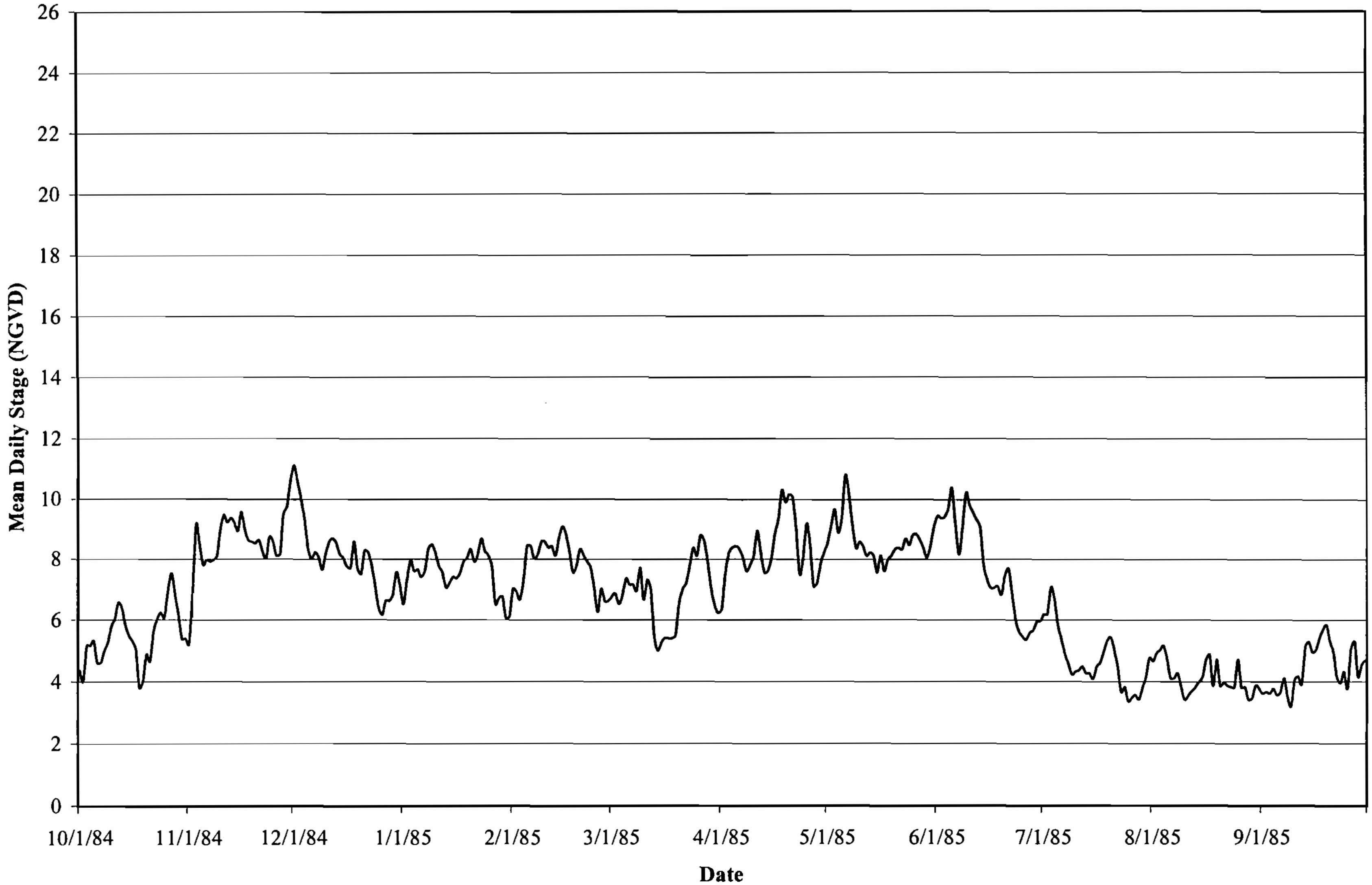


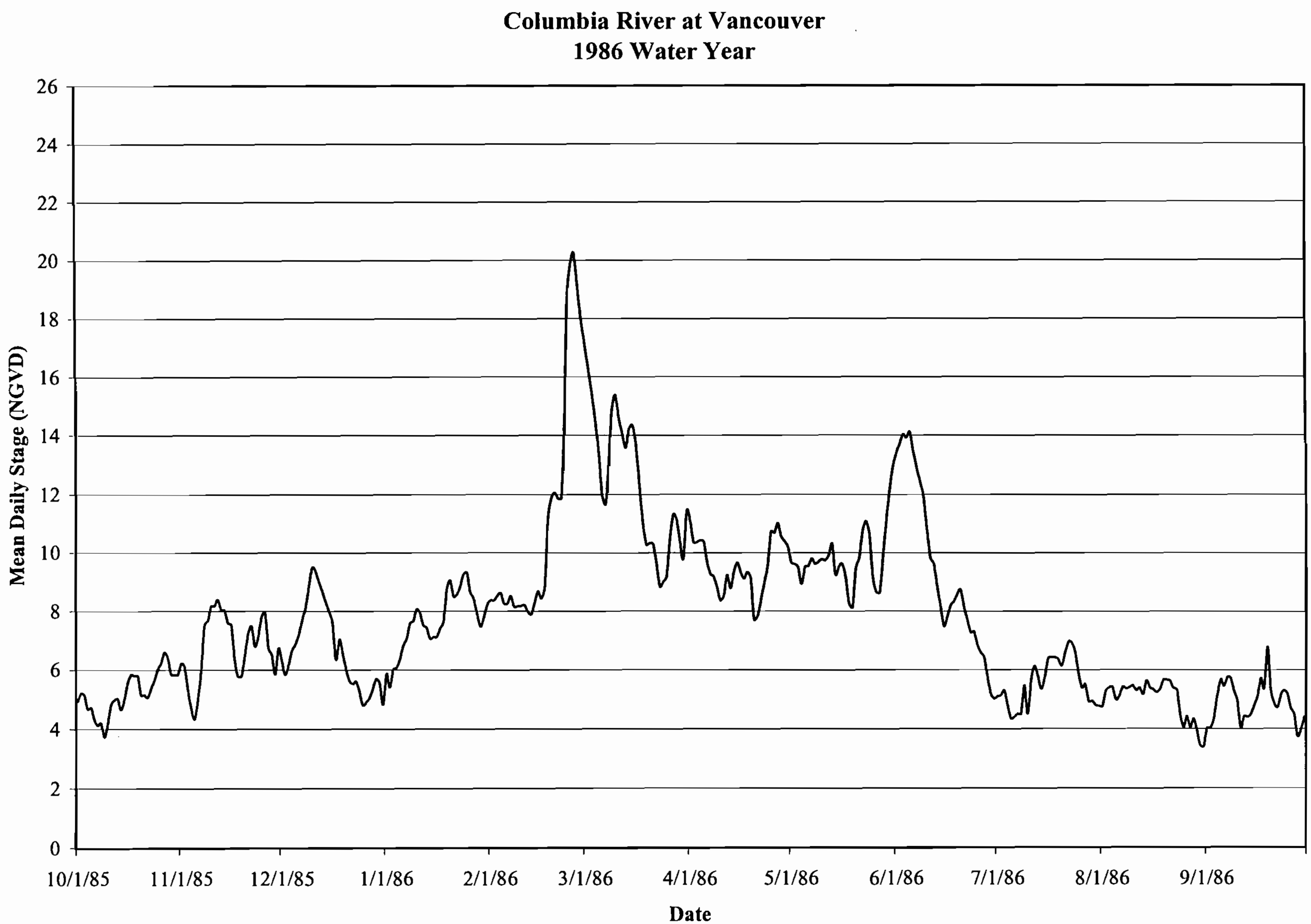

Columbia River at Vancouver
1983 Water Year
Mean Daily Stage (NGVD)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
10/1/82
11/1/82
12/1/82
1/1/83
2/1/83
3/1/83
4/1/83
5/1/83
6/1/83
7/1/83
8/1/83
9/1/83
Date

Columbia River at Vancouver
1984 Water Year

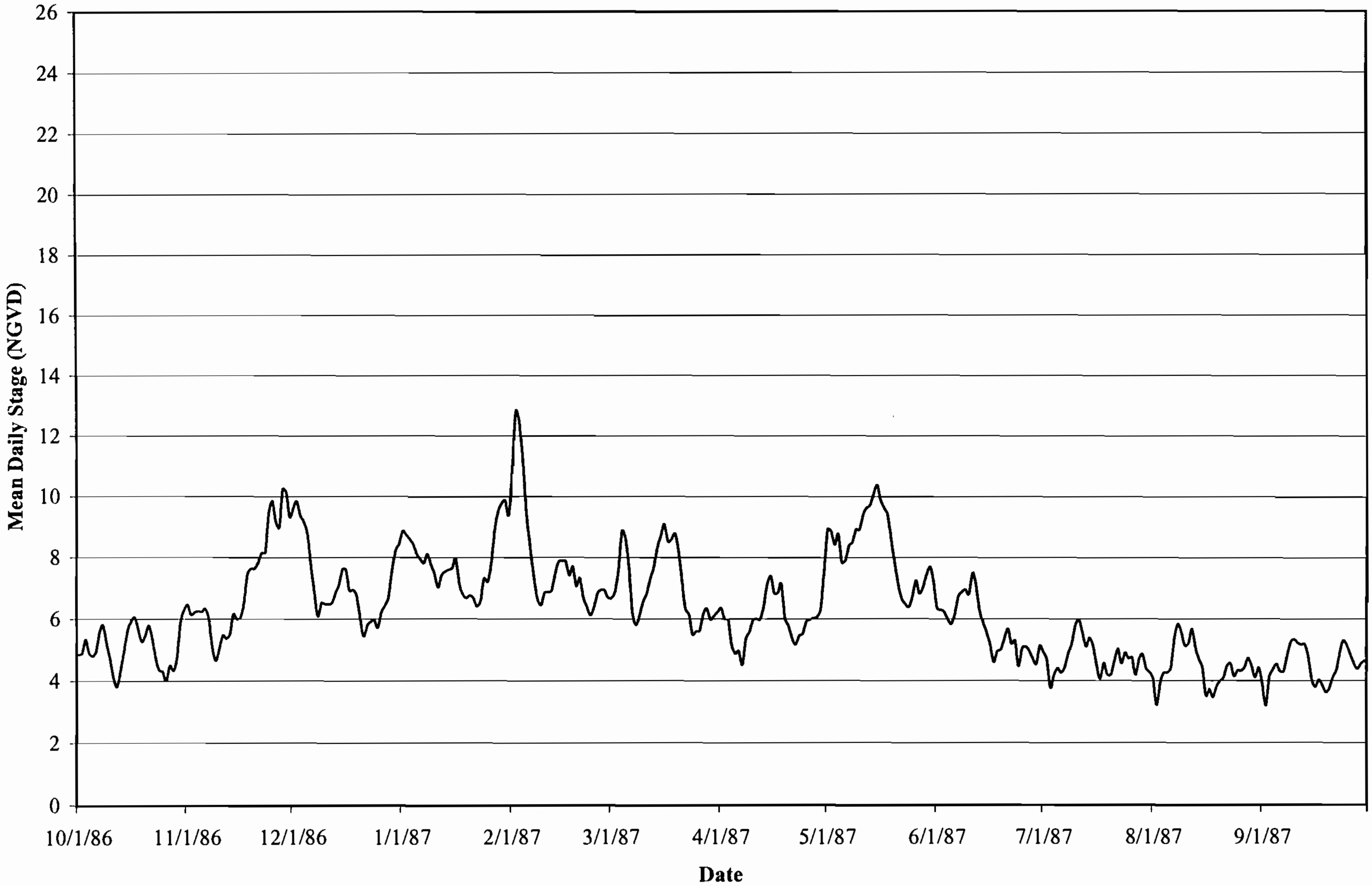


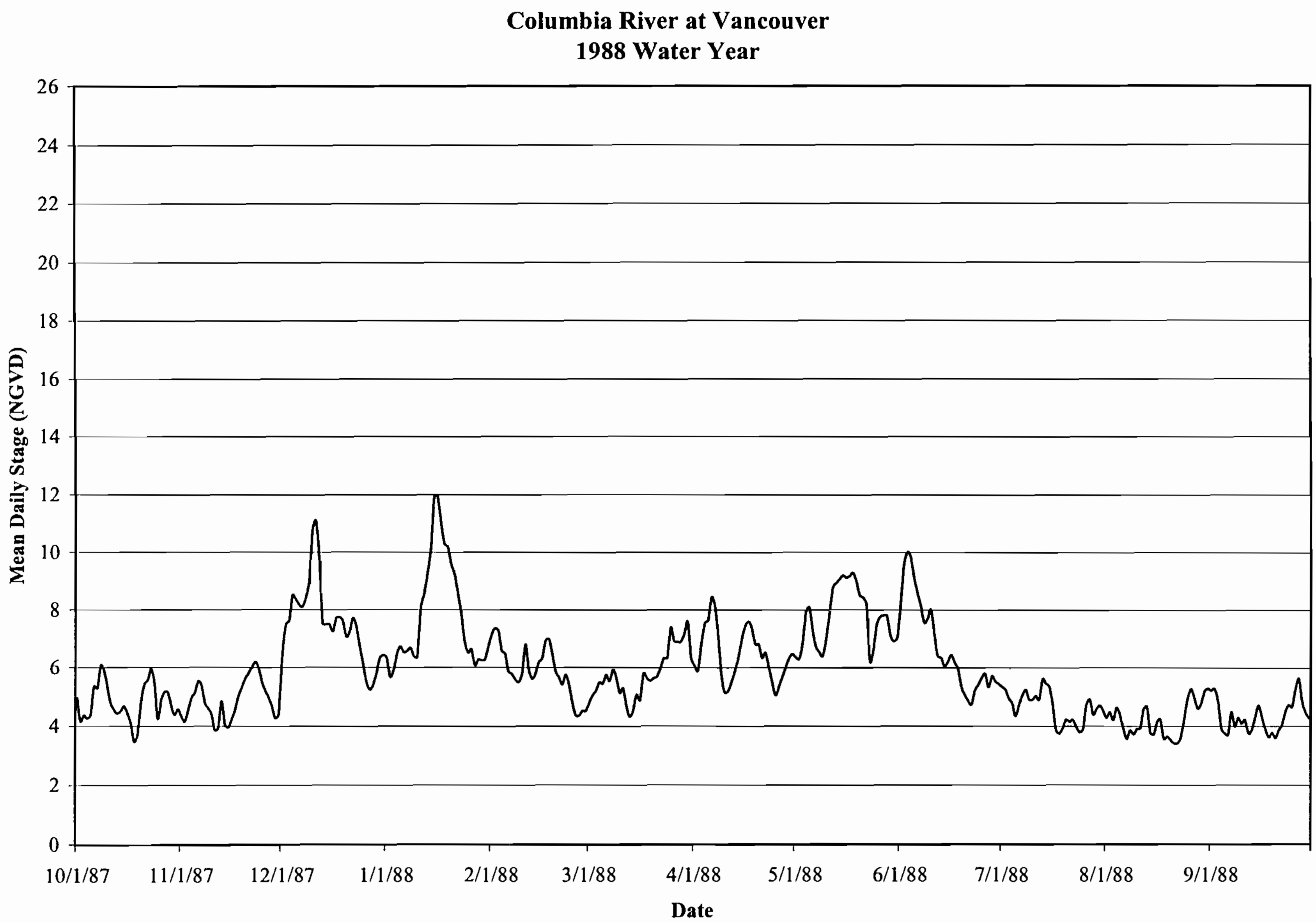
Columbia River at Vancouver
1985 Water Year




Columbia River at Vancouver
1986 Water Year
Mean Daily Stage (NGVD)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
10/1/85
11/1/85
12/1/85
1/1/86
2/1/86
3/1/86
4/1/86
5/1/86
6/1/86
7/1/86
8/1/86
9/1/86
Date

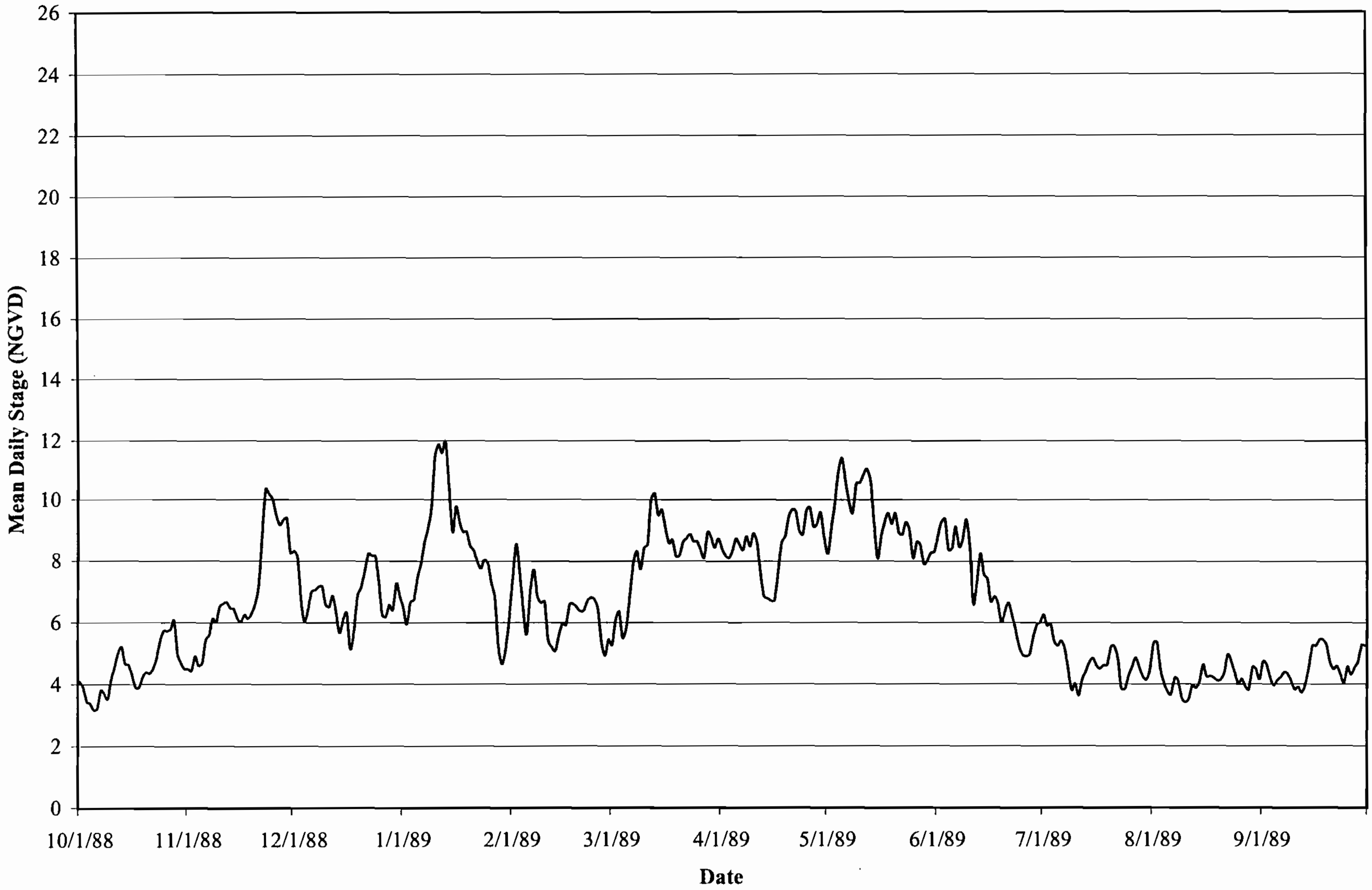
Columbia River at Vancouver
1987 Water Year

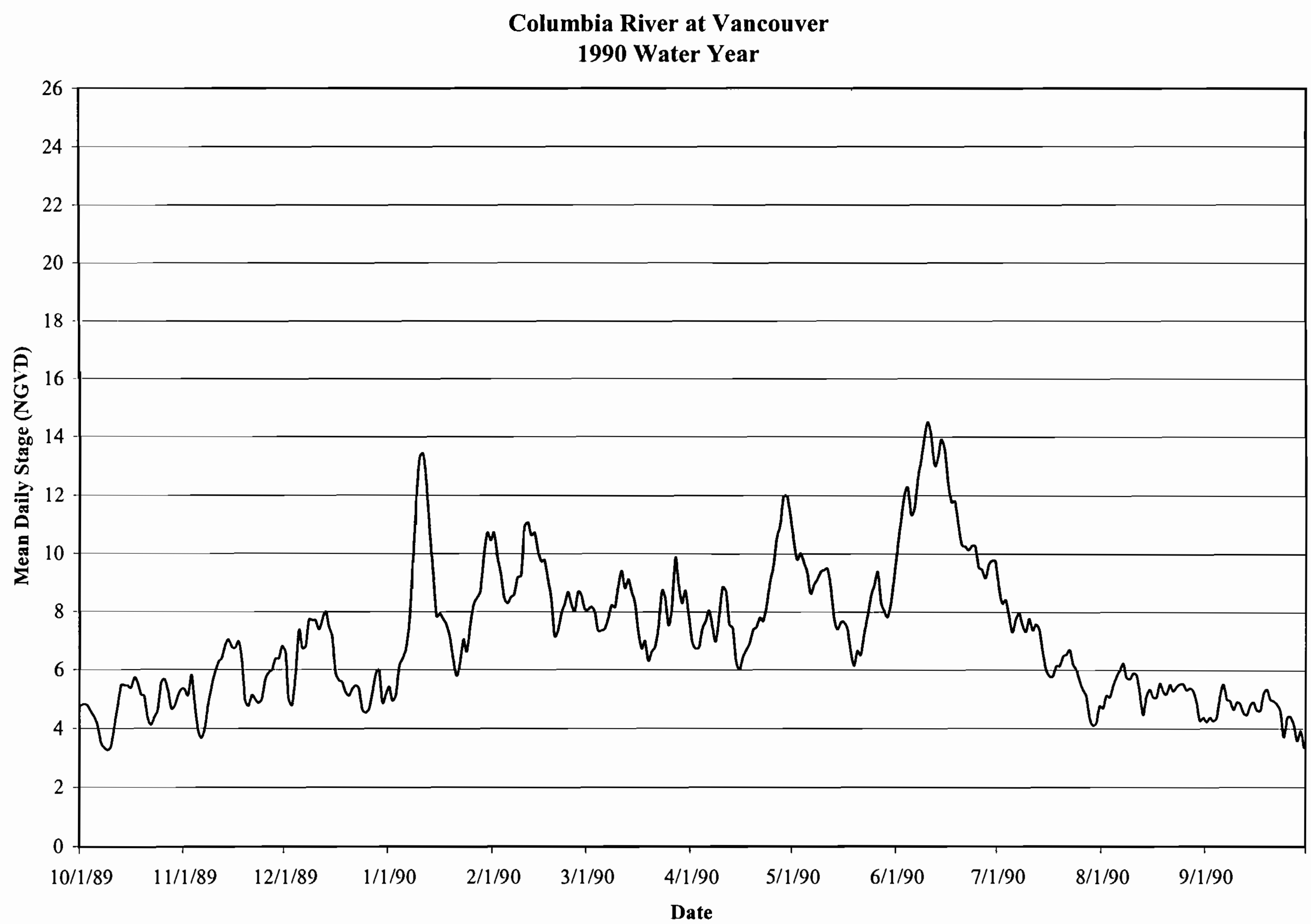




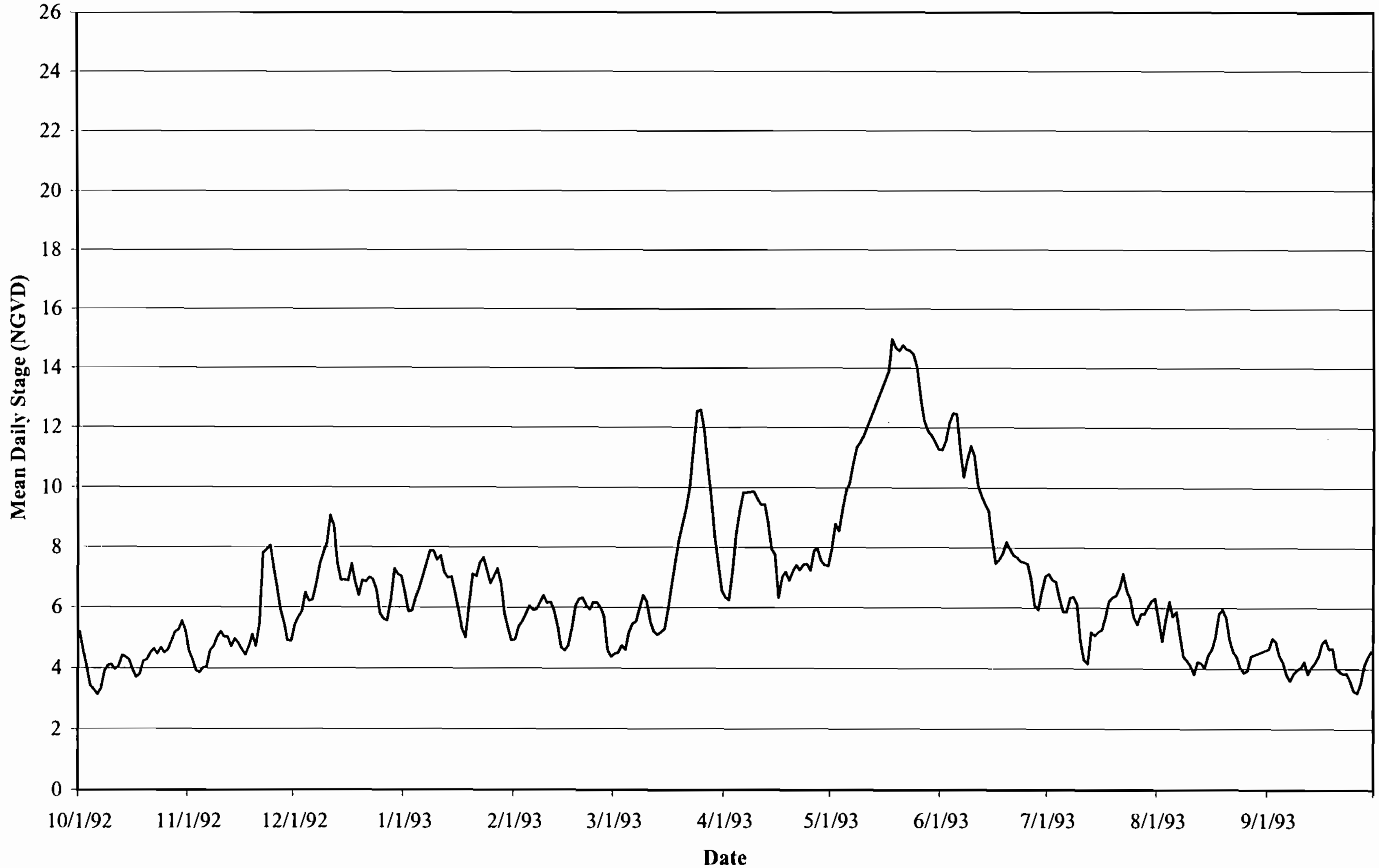
Columbia River at Vancouver
1988 Water Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/87
11/1/87
12/1/87
1/1/88
2/1/88
3/1/88
4/1/88
5/1/88
6/1/88
7/1/88
8/1/88
9/1/88
Date

Columbia River at Vancouver
1989 Water Year

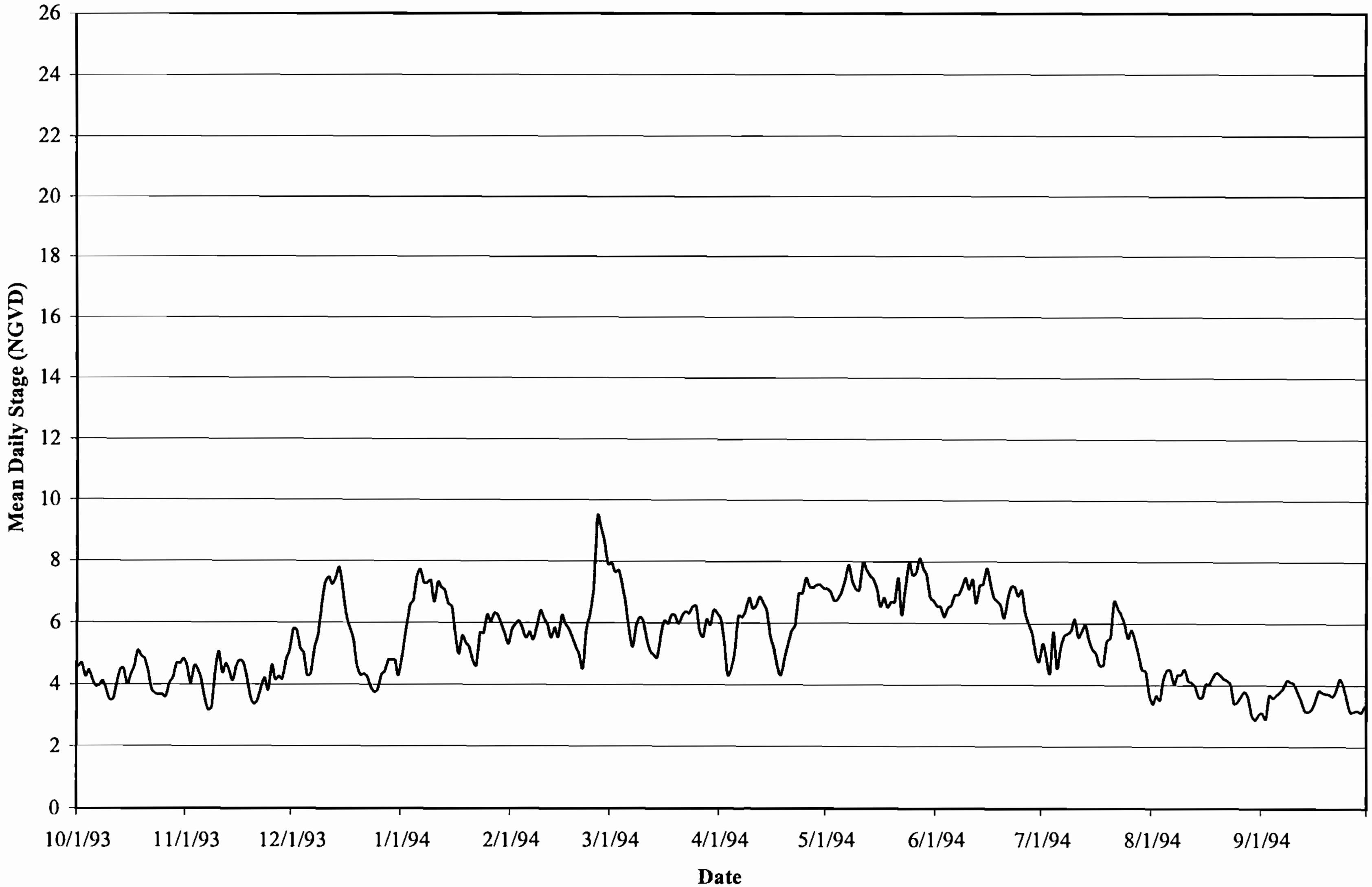



Columbia River at Vancouver
1990 Water Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/89
11/1/89
12/1/89
1/1/90
2/1/90
3/1/90
4/1/90
5/1/90
6/1/90
7/1/90
8/1/90
9/1/90
Date

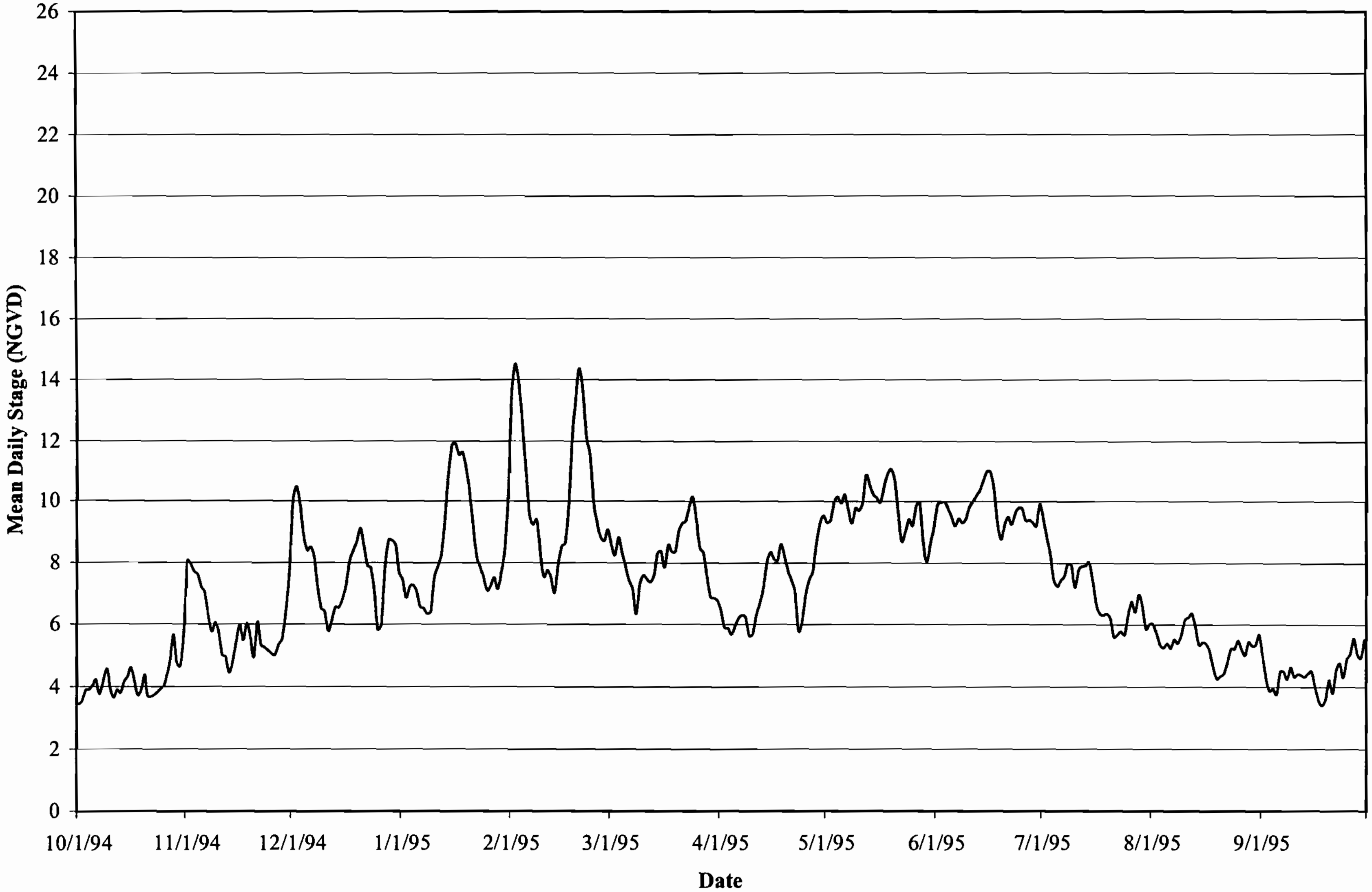
Columbia River at Vancouver
1993 Water Year



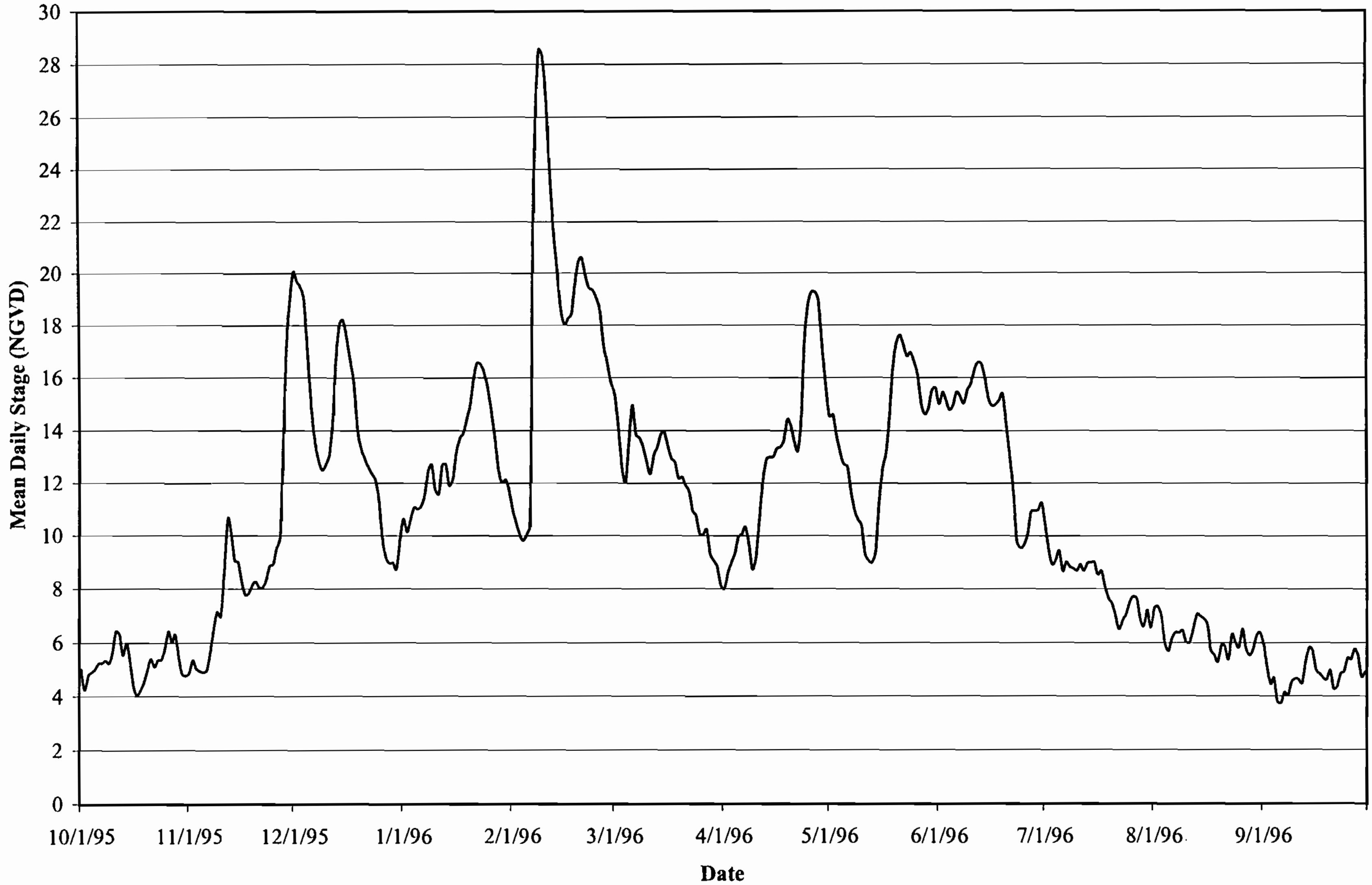
Columbia River at Vancouver
1994 Water Year



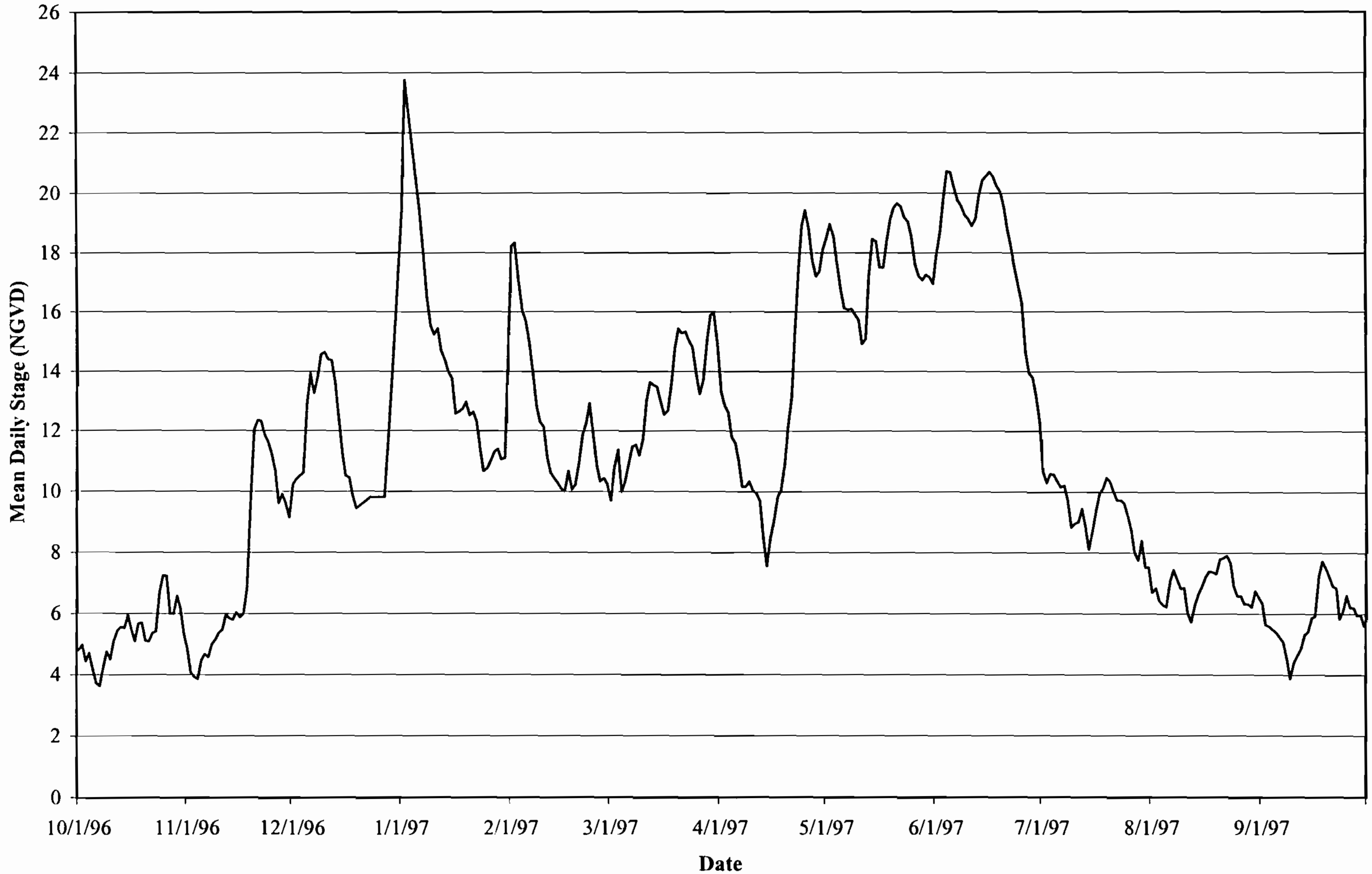
Columbia River at Vancouver
1995 Water Year



Columbia River at Vancouver
1996 Water Year



Columbia River at Vancouver
1997 Water Year



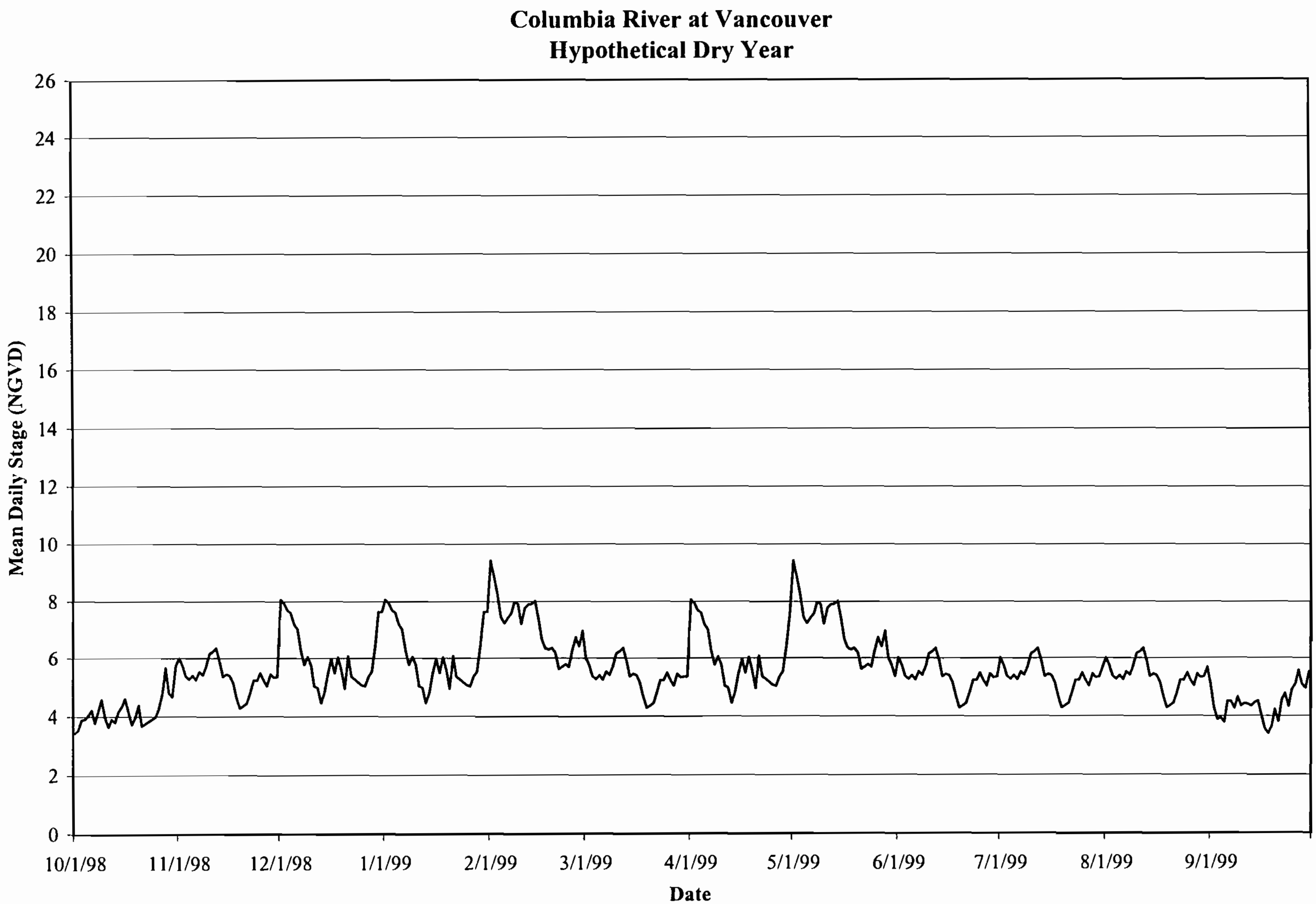

Columbia River at Vancouver
Hypothetical Dry Year
Mean Daily Stage (NGVD)
26
24
22
20
18
16
14
12
10
8
6
4
2
0
10/1/98
11/1/98
12/1/98
1/1/99
2/1/99
3/1/99
4/1/99
5/1/99
6/1/99
7/1/99
8/1/99
9/1/99
Date

Figure 3.6

APPENDIX B

Lower Columbia River UNET Model

1. Geometric Data:

Data for the Digital Terrain Models (DTM's) that were assembled using InRoads software was gathered from many sources and input as either ASCII raw data from the survey source or from digitization of existing maps and charts. Data was input to the models with the underlying assumption that newer data took precedence over older data whenever data overlapped. The resulting DTM's provided a 3-dimensional representation of the area from near the mouth of the Columbia River to Bonneville Dam and up the Willamette River from the Columbia River to Willamette Falls. This provided data up to the 100 foot M.S.L. contour and included all areas that would be hydraulically connected to the River. All data input to the DTM's was placed in 3-dimensional format, in other words, all points and lines had associated x,y, and z coordinates attached to them.

A. Data Sources:

1. Main Channel Data:

a. 1995 USACE hydrographic channel cross-line survey. This survey covered the entire shipping channel plus some overlap of the Columbia River from the mouth to near Bonneville Dam. This data was supplied as ASCII data points (soundings) that ran perpendicular to the flowpath with a spacing of approximately 500 feet. This data was the foundation for which all DTM's were based upon.

b. 1994 USACE hydrographic survey near Bonneville Dam. This survey was an extensive survey of all channels near outlets of Bonneville Dam. It was used to supplement data from the 1995 survey near the Dam.

c. 1995 USACE hydrographic channel cross-line survey for the Willamette River. This survey covered the majority of the main channel of the Willamette River and was used as the major data source for the instream Willamette River.

2. Off-Channel Areas (sloughs and secondary river channels):

a. 1981 - 1994 USACE hydrographic surveys. Various off-channel surveys have been conducted throughout the time period from 1981 - 1994. These surveys were used based upon research of all files and finding the most recent survey of a particular channel, slough or bay. From research it appeared that a survey had been conducted by the USACE for most off-channel areas, however, upon searching for the electronic files, it became apparent that many of the files could not be located or did not exist. Therefore, some surveys were digitized off copies of the survey plates while some surveys were never located.

b. 1984 Columbia River Estuary Data Development Program, cooperative study between Columbia River Estuary Study Taskforce, NOAA, and US Dept. of

Commerce. This study produced maps of the bathymetry of the Columbia River Estuary from approximately the mouth to river mile 30. These contours were digitized for all areas outside the shipping channel and were the primary geometric data source for the estuary area.

c. NOAA Navigation Charts 1984-1986. Data digitized from NOAA navigation charts was used as a "last resort," and was only used when there did not appear to be any other data available. This data was useful in off-channel slough areas where other data did not exist. It was also used in some secondary channels where data did not exist or to supplement sparse data in those areas. It appears that data found on charts has been collected from various government agencies, especially the USACE. This data may range in date from 1950 to 1984.

3. Flood Plain Data:

a. USGS Digital Elevation Maps (DEM's). USGS DEM's are basically USGS 7.5-Minute Topographic Quadrangles placed in digital format. However, they are not as accurate as the original maps since they are based on a grid system that has a 30 meter grid spacing between points. USGS DEM's were read into the DTM's for all points greater than zero but less than 100 feet M.S.L. In other words, this consisted of all USGS DEM points that fell within the band of shore from the water's edge up to 100 feet M.S.L.

b. USGS 7.5-Minute Topographic Quadrangles (USGS Quads). Four of the 40 Quadrangles for the project were not available in DEM format. Therefore, contours from the shoreline up to the 100 foot contour were digitized from these four USGS Quads. Also, all levees located along the lower Columbia River were represented as 3-dimensional lines digitized off of the USGS Quads. The Low-Low Water (LLW) line was digitized from USGS Quads and used as a bankline for the DTM's. This line was placed into the DTM's with a variable elevation attached to it based upon the location along the river. This elevation ranged from zero feet M.S.L. at the mouth to twenty feet M.S.L. near Bonneville Dam. The changes in elevation were "stair-stepped" along the river.

B. Data Coordinate Conversions:

Building the DTM models for the lower Columbia River required placing all input data into the same horizontal and vertical datum (State Plane-Oregon North and National Geodetic Vertical Datum of 1929 (NGVD29), respectively.) USACE hydrosurvey data was provided in separate ASCII files for each "range" (approximately four-mile reaches) of river. The data was in Columbia River Datum (CRD) coordinates which vary along the Columbia River. The CRD adjustment factor for each range was applied to each data file in order to place the data into NGVD29. Thus, adjustment from CRD to NGVD29 was approximated by stair-stepping up the river in four-mile increments. Data from the USGS DEM's was originally in Universal Transverse Mercator (UTM) coordinates. Due to the number of points within each of the 36 USGS DEM's, the conversion process from UTM coordinates to State Plane-Oregon North, NGVD29 coordinates was very time consuming. The data was converted by placing the DEM's into ASCII files, reformatting the files for use in CORPSCON (a USACE coordinate conversion program,) and converting the data using CORPSCON software. Some of the DEM's were also in metric units and these files were converted to English units.

Once all the data was placed into the same coordinate system, it was placed into Microstation as either point data or contour lines. Then all overlaps between data were "cleaned up." A DTM was then created using InRoads for a portion of the river (there were 10 DTM's covering the modeled area) from the combined data. The completed DTM was then viewed and all unnatural features were noted. In areas where irregularities were noticed, extra points were placed in "blank" areas using linear interpolation between data. The following figure is an image (snapshot) of the CADD file from Microstation that was used to create the geometry for the Lower Columbia UNET Model.

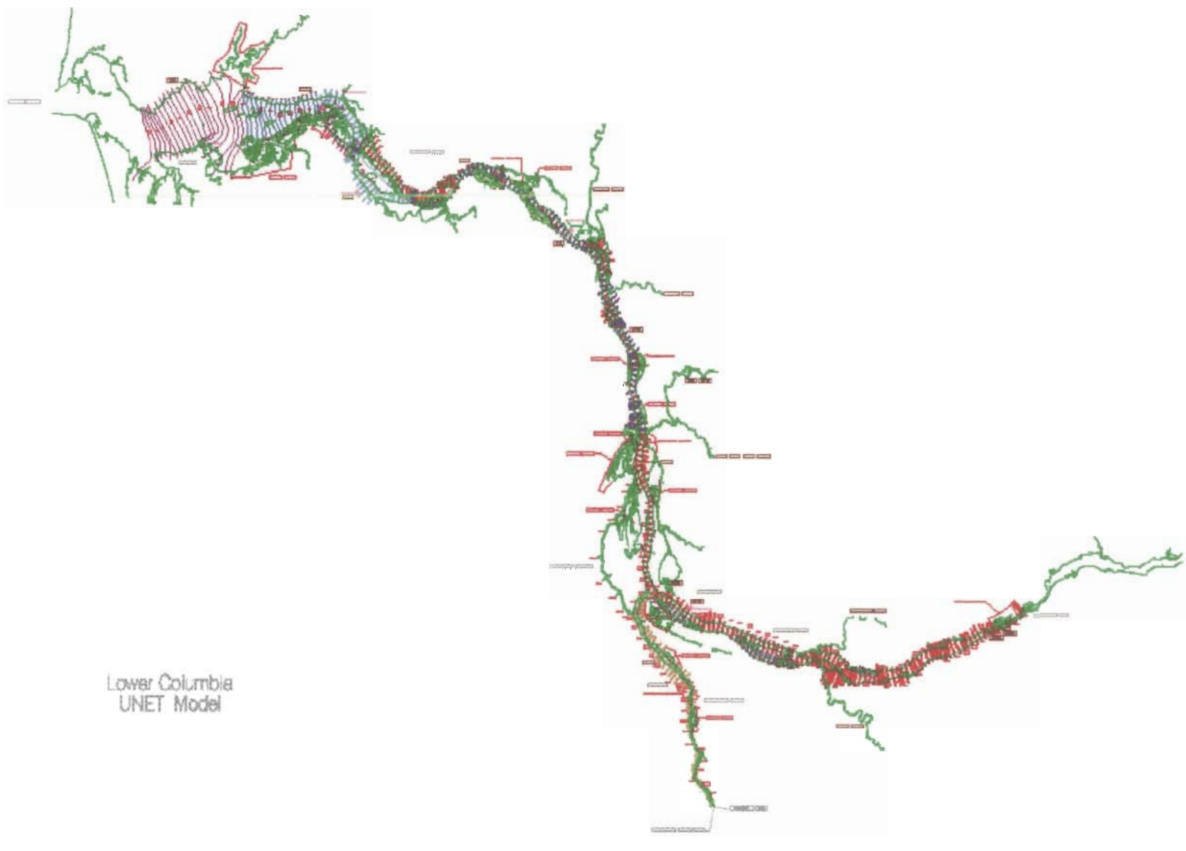


Figure IB-1. Lower Columbia UNET Model Cross-Section Layout

C. Model Hydrology:

Hourly inflows were developed for boundary conditions at the upstream ends of the upper reaches and input to the model in the form of time-series data utilizing HEC-DSS. An hourly stage hydrograph at Astoria was used for the downstream boundary condition. Six tributaries to the Columbia were included in the model: the Sandy, the Washougal, the Willamette, the Lewis, the Kalama, and the Cowlitz Rivers. This accounted for approximately 99% of the total drainage area in the Columbia River and 88% of the total tributary area to the Columbia River between Bonneville Dam (RM 145) and Beaver Army terminal (RM 55.0.) The following section briefly lists the methodology used for developing the Lower Columbia hydrology for model calibration, verification, and analysis of the flood of February 1996. Both regulated (actual) flood hydrographs and unregulated (without flood control projects on the system) flood hydrographs were developed.

1. Boundary Conditions:

a. Columbia at Bonneville Dam:

1. Regulated: Hourly flow for Bonneville was taken directly from the Columbia River Operational Hydro-met System (CHROMS) database.
2. Unregulated: A constant daily flowrate was input for Bonneville. This data was provided by Russ Morrow of the USACE at NPD.

b. Willamette at Willamette Falls:

1. Regulated: Flow for the Willamette was derived using HEC-5. Inflows and outflows were taken from CHROMS for all Willamette Valley Projects. Gages within the CHROMS database provided flows and stages for the majority of the Willamette system up to Salem. Between Salem and Oregon City ungaged local inflows to the Willamette were estimated from the South Yamhill at Whiteson Gage based on drainage area. Flows for the Tualatin Basin were estimated from gages at West Linn and at Farmington (incorporating Lake Oswego Diversion.) Flows for the Clackamas Basin were based on the gage at Estacada plus local inflows between Estacada and the Willamette. Local inflows for the Clackamas Basin between Estacada and the Willamette were estimated from the Johnson Creek gage at Milwaukee based on drainage area. Local Portland inflows were estimated from Johnson Creek based on drainage area (in addition to Johnson Creek itself.)
2. Unregulated: Unregulated flows were calculated by HEC-5 based on the inflow, outflow, and change-in-storage at each project. All flood control projects were unregulated. Otherwise, the same methods as were used to derive regulated flows were used.

c. Astoria Stage:
1. Regulated: The hourly stage at Astoria was taken directly from the CHROMS database and input as the downstream boundary condition.
2. Unregulated: Regulated Stage (actual) was used for the unregulated condition.

2. Tributaries:

a. Cowlitz River:
1. Regulated: Hourly flows for the Cowlitz were taken directly from the gage at Castle Rock.
2. Unregulated: It was assumed that projects on the Cowlitz provided little flood control, thus regulated flows were used.

b. Lewis River:
1. Regulated: Data for the Mainstem Lewis River was provided from the gage at Ariel. However, the gage malfunctioned for approximately 14 hours during the peak of the hydrograph. The early estimate of the peak was 101000 cfs provided by the USGS. This peak was used to fill in the missing data. The East Fork of the Lewis River was added by the drainage area ratio between it and the Ariel gage.
2. Unregulated: It was assumed that projects on the Lewis River provided little flood control, thus regulated flows were used.

c. Kalama River:
1. Regulated: Flows for the Kalama River were estimated from the Lewis River hydrograph based on the drainage area ratio between the Kalama and the Lewis Rivers.
2. Unregulated: Regulated flows were used.

d. Sandy River:
1. Regulated: Flows for the Sandy River were taken directly from the gage near Sandy, below the Bull Run River. No adjustments for added flow downstream were made.
2. Unregulated: It was assumed that projects on the Sandy River System provided little flood control, thus regulated flows were used.

e. Washougal River:
1. Regulated: Flows for the Washougal River were estimated from the Sandy River hydrograph based on the drainage area ratio.
2. Unregulated: Regulated Flows were used.

D. Model Calibration:

The model has been calibrated for a Winter flood event using the February 1996 flood data. The period modeled was from February 1, 1996 through February 13, 1996. The Flood of February, 1996 was one of the largest Winter flood events on the Lower Columbia and Willamette Rivers in recent history. The flood was estimated to have been approximately a 250-year to 300-year event at Vancouver and approximately a 200-year event at downtown Portland based on historic fall/winter stage frequency curves developed by the USACE, Portland District in 1976. Peak flowrates for the system were estimated as 457,000 cfs in the Willamette River at Willamette Falls on February 9th and approximately 440,000 cfs in the Columbia River at Bonneville on February 9th. This resulted in the following estimations for peak river flowrate on the Lower Columbia from the UNET model of 790,000 cfs at the Willamette River Confluence and 925,000 cfs at the Multnomah Channel Confluence with the Columbia River.

Calibration consisted of matching stages from model runs with actual data at six gages along the Columbia River System. These gages included Skamokawa (RM 33.8), Wauna (RM 42.0), Beaver Army Terminal (RM 55.0), Longview (RM 66.4), Vancouver (RM 106.5), and Morrison Bridge (Willamette RM 12.0). In order to fit the model output to the stage data, parameters such as Manning's roughness, effective flow area, and storage area were adjusted. The model matches the February 1996 flood event throughout the river system very well. As stated previously, the model is currently in a preliminary phase. Although model results for this event match observed data well, the model still needs to be tested against a variety of conditions and adjusted to match those conditions in order to be considered calibrated. The following Figures illustrate the calibration results at the six gaging locations along the system. As one can see, gages at Vancouver, Longview, Wauna and Skamokawa had some missing data values during the modeled period. Missing data values were replaced with the last data value read, therefore they appear as flat line segments.

LOWER COLUMBIA UNET MODEL

I. Model Development:

A. UNET Hydraulics:

The UNET model is a hydraulic, one-dimensional, unsteady flow model developed by the USACE Hydrologic Engineering Center (HEC.) The model was developed primarily for large river systems with an unsteady nature. Its capabilities include the modeling of split flow and flow combination, off-line storage, and levee breaches. It models hydrodynamics based upon the one-dimensional momentum equation and the continuity equation. A river network is broken down into reaches that are defined by cross-sections. Water is "routed" between pairs of cross-sections by solving the differential equations for fluid flow (momentum and continuity) using an implicit finite-difference scheme. Boundary conditions (i.e. stage, flow, or rating curve) are necessary at the extreme upstream and downstream locations for the river system. UNET also has the capability of modeling internal boundary conditions such as tributary point inflows and distributed flows.

The following sections give a very brief summary of the adaptation of UNET to the Lower Columbia River System from Bonneville Dam to Astoria, Oregon. The model developed for this analysis is still in a preliminary phase, however, initial model results have been very encouraging and the building blocks are in place for further model development.

B. Model Geometry:

For the Lower Columbia River Model, geometry data (x,y,z) was gathered from many sources, placed in electronic format and imported into Microstation. A series of triangulated Digital Terrain Models (DTM's) were created for the entire Lower Columbia River using InRoads software for Microstation. The River was broken down into reaches that split flow around islands and at channel junctions. Individual cross-sections were "cut" along lines drawn perpendicular to the normal flowpaths along the river. Cross-sections were placed at an interval no less than one-half mile apart. Many off-line flow areas (i.e. Smith & Bybee Lakes) were modeled as storage areas. These areas act as lakes connected via a conduit, weir, or channel to the main reach allowing for lateral inflow and outflow. The Columbia River was modeled in such a way from Astoria to Bonneville Dam and the Willamette River was modeled from its confluence with the Columbia to Willamette Falls.

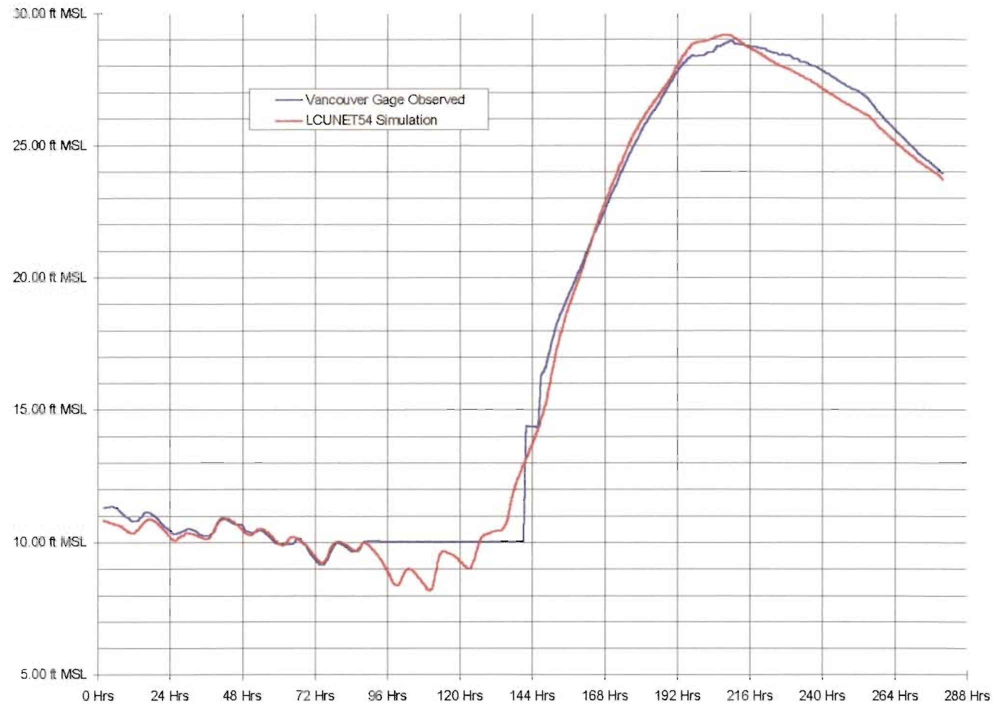


Figure ID-1. February 1996 flood at Vancouver Modeled Stage vs. Recorded Stage

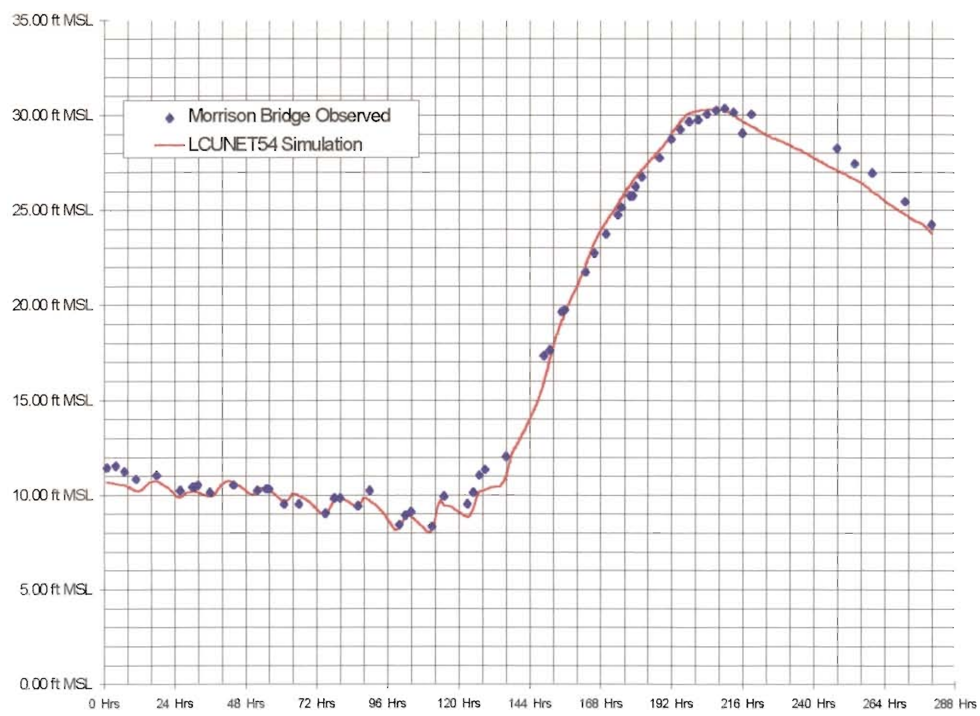


Figure ID-2. February 1996 flood at Portland Modeled Stage vs. Recorded Stage

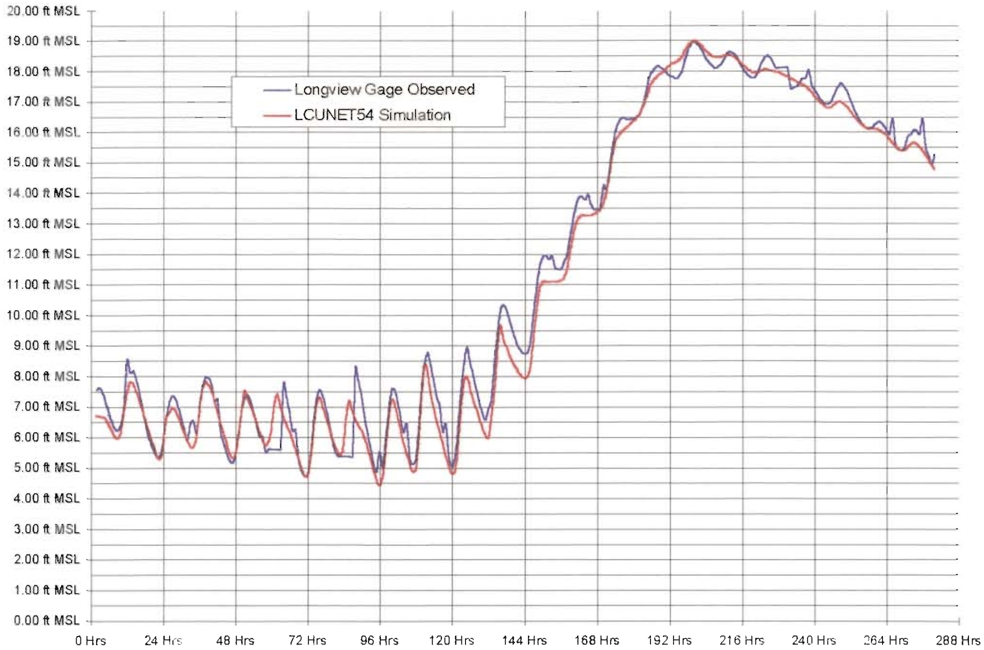


Figure ID-3. February 1996 flood at Longview Modeled Stage vs. Recorded Stage

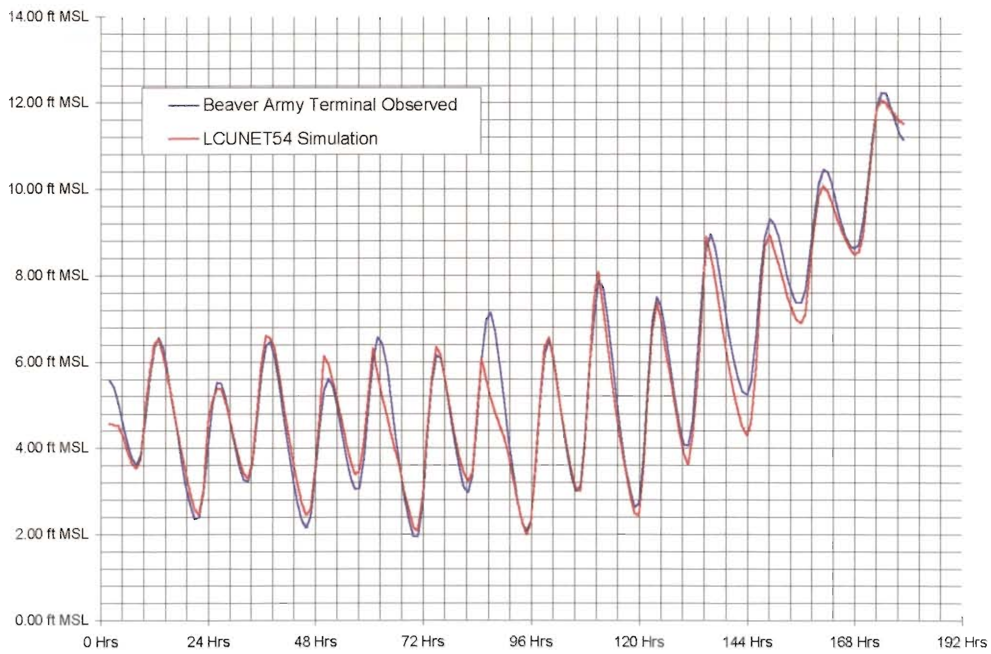


Figure ID-4. February 1996 flood at Beaver Modeled Stage vs. Recorded Stage

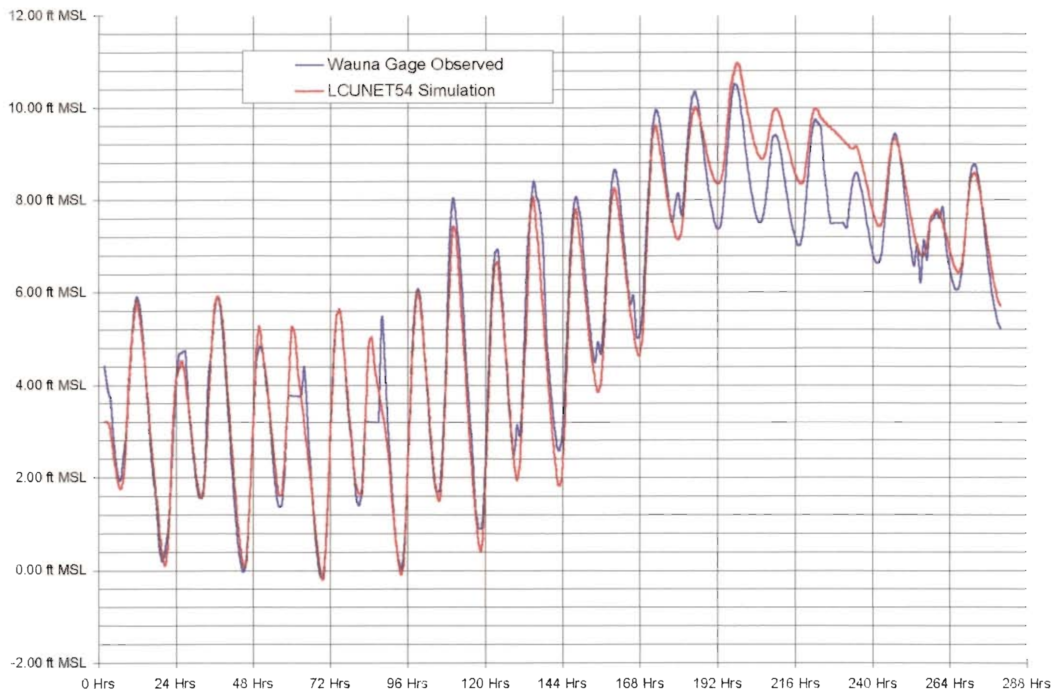


Figure ID-5. February 1996 flood at Wauna Modeled Stage vs. Recorded Stage

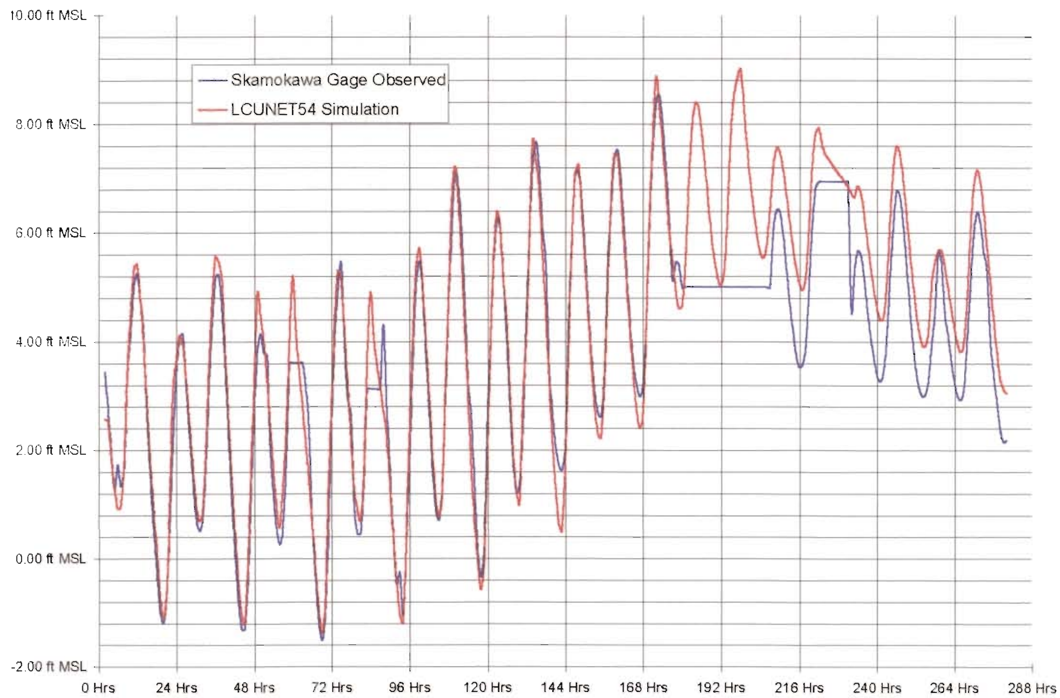


Figure ID-6. February 1996 flood at Skamokawa Modeled Stage vs. Recorded Stage

E. Model Verification:

Once the model had been reasonably calibrated with a single event, it was run with a longer series of events in order to check its reasonableness at matching variations in events. The modeled period was from November 1, 1995 through April 30, 1996 (6 months.) No parameter adjustments were made and the same assumptions were made for developing the input hydrographs (see "I.C. Model Hydrology" above) as during the calibration period. The results from this exercise were very promising. The model predicted peaks and the overall changes in stage throughout the river system well. However, model improvements need to be made for low flow conditions between events. The following five figures represent verification results at Vancouver, Portland, Longview, Wauna, and Skamokawa gages respectively.

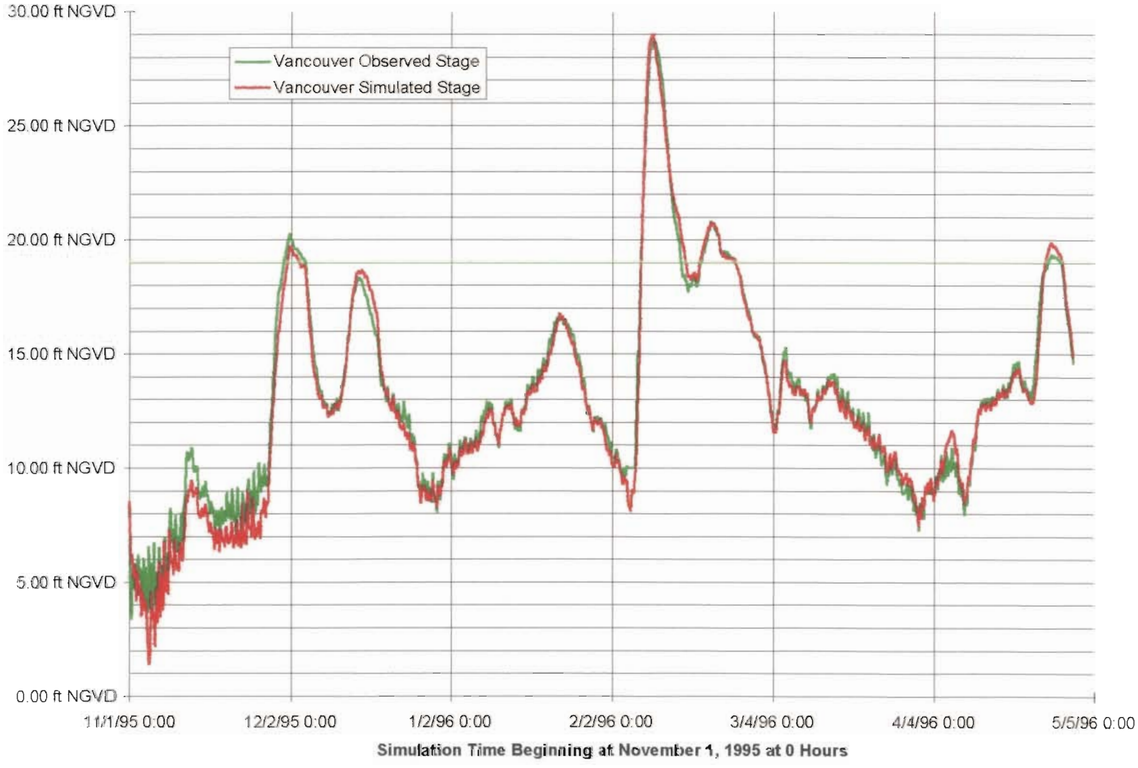


Figure IE-1. 1996 Winter Period at Vancouver Modeled Stage vs. Recorded Stage

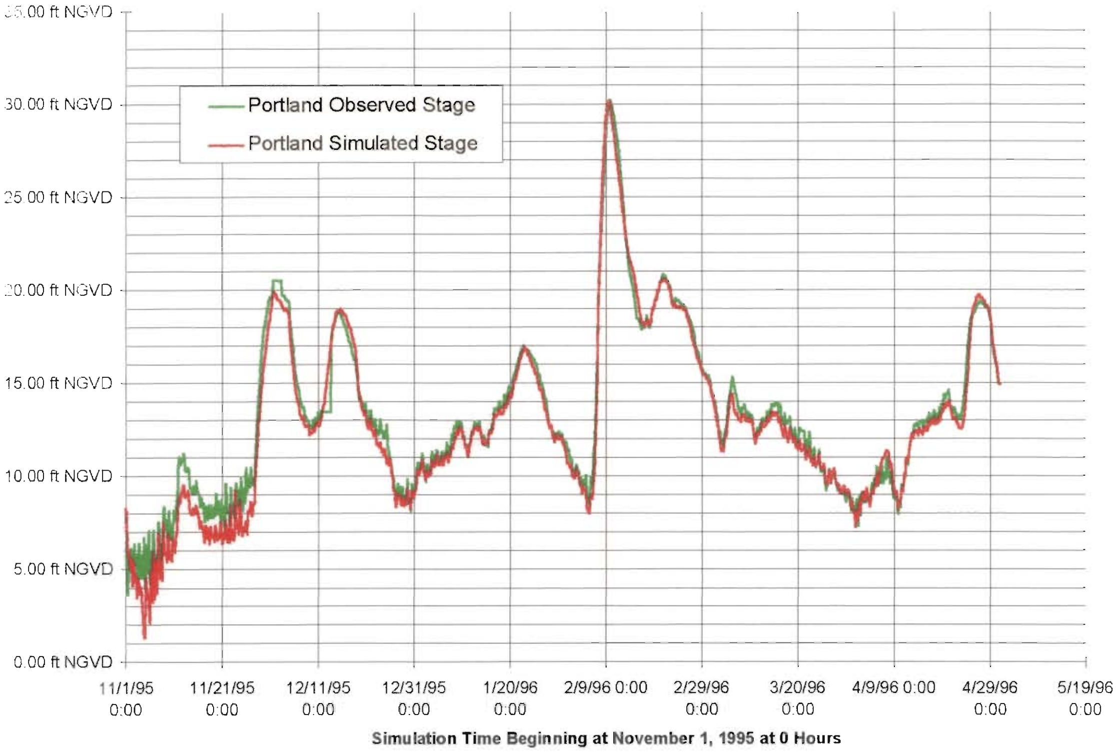


Figure IE-2. 1996 Winter Period at Portland Modeled Stage vs. Recorded Stage

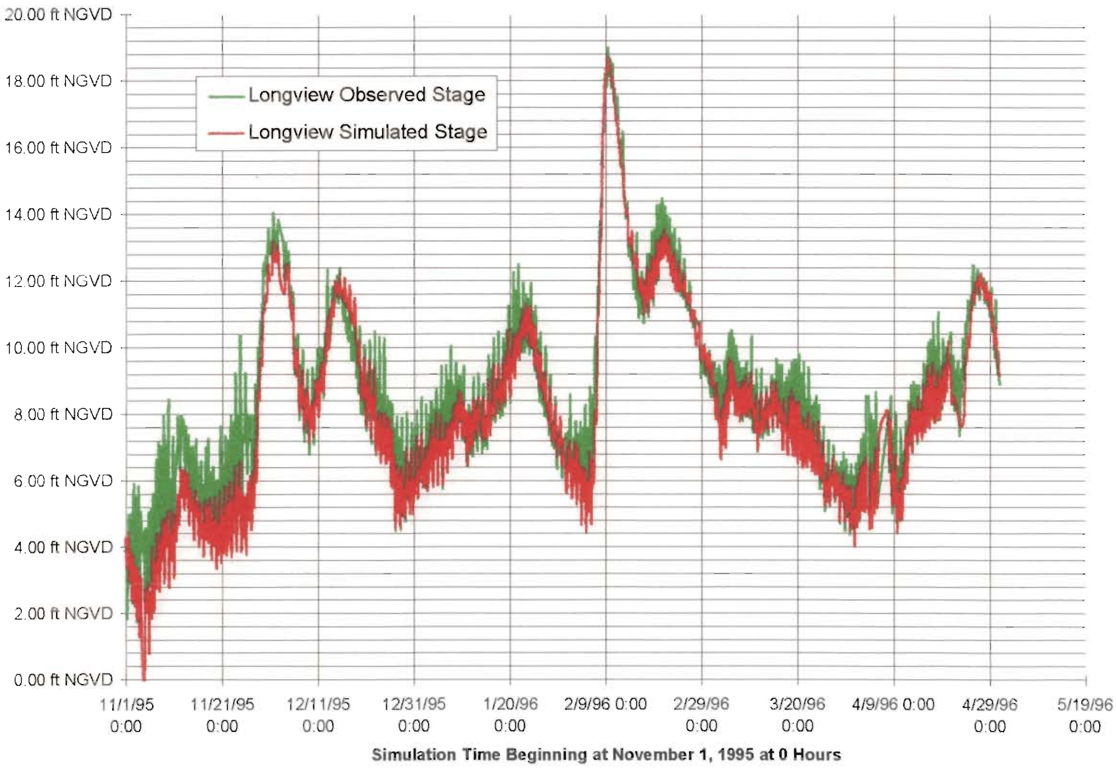


Figure IE-3. 1996 Winter Period at Longview Modeled Stage vs. Recorded Stage

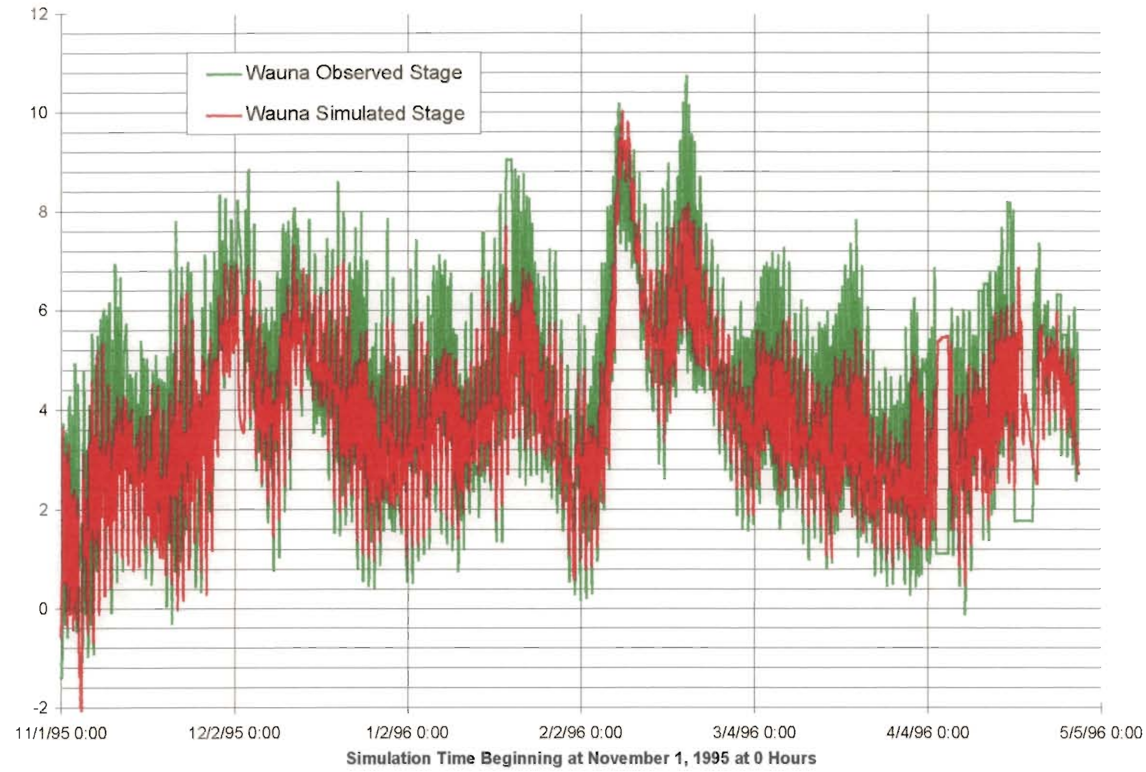


Figure IE-4. 1996 Winter Period at Wauna Modeled Stage vs. Recorded Stage

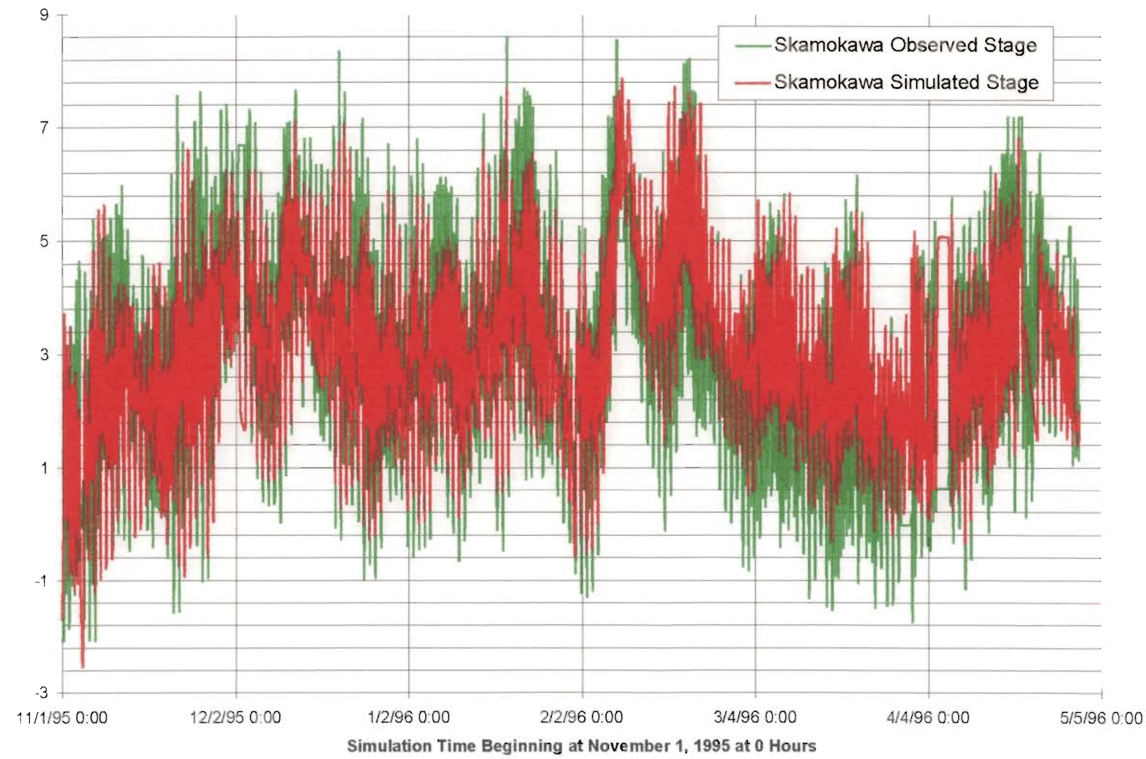


Figure IE-5. 1996 Winter Period at Skamokawa Modeled Stage vs. Recorded Stage

Background

During the winter of 1995-1996, a UNET (one-dimensional, unsteady-state flow) model was developed for the Lower Columbia River by the Hydrology, Coastal and River Engineering Section of the Portland District, U.S. Army Corps of Engineers (CENWP-PE-HY.) The UNET model was developed to a point where it showed promising results for hindcasting events on the lower river. The model was developed on the Columbia River from Astoria up to Bonneville Dam and on the Willamette River from the Columbia up to Willamette Falls. In its inception, the model was to be taken to a level where it could be reliably used to both hindcast and forecast stages and one-dimensional velocities along the Lower Columbia River. When funding levels were reduced, the model was set aside until further project funds became available. The model has been updated to accommodate specific projects on a piece-meal basis. At this time the model has the capability to hindcast events (downstream boundaries are set to gaged data) on the Columbia River for the hydrologic conditions similar to the calibrated events that follow.

Initially, the model was calibrated to the February, 1996 flood event. Calibration results for this event were excellent. In an attempt to verify the model, the modeled period was extended to November, 1995 through April, 1996. Results of the verification run indicated that the calibrated model could reproduce the correct shape of stage hydrographs along the river, but the magnitude varied considerably from observed data. The primary reason for deviations in the output from the observed data is from Manning's roughness being directly proportional to flowrate. The UNET model does not allow one to automatically adjust Manning's roughness to flowrate. Therefore, multiple UNET models have been calibrated to varying degrees spanning events of differing hydrology. The following section lists each UNET model currently available for the Lower Columbia River and the degree it has been calibrated:

Existing Lower Columbia River UNET Models

1. February 1996 UNET model

The first model was calibrated to the February 1996 flood (February 1, 1996 through February 13, 1996.) This flood was an extreme winter event on the Willamette and Lower Columbia Rivers. The regulated hydrograph peaked at 29.0 feet NGVD at Vancouver, Washington on February 8, 1996. It has been determined that this event was approximately a 200-year event at Vancouver. Calibration results for this event were quite promising (see attached – "The Lower Columbia UNET Model".) A UNET expert (Michael Gee of HEC) reviewed this model soon after calibration. He indicated that the model was applied correctly, had a good data foundation and appropriate model parameters. However, he also indicated that the hydraulic property tables could be refined to span only the active water surface layers. In order to change these tables, the entire model would need to be re-calibrated. Since that time the model has been updated. However, calibration has not been performed to the level that it had previously. Therefore, the February, 1996 model is in a preliminary state pending further calibration.

2. Spring 1997 UNET model

This model was calibrated to a heavily regulated Spring freshet on the Columbia River (April 1, 1997 through July 23, 1997.) The peak observed outflow for this event was 575,000 cfs at Bonneville. This peak was estimated to have approximately a 10-year regulated reoccurence interval. Calibration results for this event were excellent as the model reproduced correct shapes and magnitude of virtually all observed stage data on the Lower Columbia River.

3. Spring 1996 UNET model

This model was calibrated to a Spring freshet on the Columbia River with a frequent reoccurence interval (April 1, 1996 through June 30, 1996.) The peak observed outflow for this event was 440,000 cfs at Bonneville. This peak was estimated to have approximately a 2-year regulated reoccurence interval. Calibration results for this model are in a preliminary state. The model reproduces the shape of hydrographs on the lower river, but adjustment need to be made to capture the magnitude.

4. September, 1994 Low Flow Model

This model was developed for extreme low flow conditions in the Columbia River (September 3, 1994 through September 12, 1994.) The model was calibrated for flows ranging from 70,000 to 160,000 cfs out of Bonneville. Calibration results indicate that the model reproduces overall hydrographic shape, however, the tidal range (magnitude) was not captured. This model is in a preliminary state. Further calibration work needs to be performed to capture the dynamics of the low flow conditions.

APPENDIX C

HSPF User Control Input

```
RUN
GLOBAL
  SHILLAPOO HSPF LAND SEGMENTS
  START       1993/10/01 00:00  END    1997/09/30 24:00
  RUN INTERP OUTPUT LEVEL    0
  RESUME     0 RUN     1 TSSFL    15 WDMSFL   16
END GLOBAL

FILES
<FILE>  <UN#>***<----FILE NAME------------------------------------------------->
INFO      23    HSPINF.DA
ERROR     24    HSPERR.DA
WARN      25    HSPWRN.DA
WDM       26    D:\PROJS\SHILLAP\MET\SHILLAP.WDM
MESSU     60    landseg.out
           6    LANDS.OUT
END FILES

OPN SEQUENCE
    INGRP              INDELT  1:00
      PERLND       1
    END INGRP
END OPN SEQUENCE
                                                                            ***
PERLND
  GEN-INFO
                                                                            ***
    <PLS >      Name          NBLKS   Unit-systems   Printer                ***
    # -  #                          User  t-series Engl Metr                ***
                                             in  out                        ***
    1      USGS SATURATED        1    1    1    1    1    0
  END GEN-INFO
  ACTIVITY
    <PLS > ************* Active Sections *****************************
    # -  # ATMP SNOW PWAT  SED  PST  PWG PQAL MSTL PEST NITR PHOS TRAC       ***
    1    3    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 *********
    1    3    0    0    5    0    0    0    0    0    0    0    0    0    1    9
  END PRINT-INFO
  PWAT-PARM1
    <PLS > ***************** Flags ********************
    # -  # CSNO RTOP UZFG  VCS  VUZ  VNN VIFW VIRC  VLE                      ***
    1    3    0    0    0    0    0    0    0    0    0
  END PWAT-PARM1
  PWAT-PARM2
    <PLS > ***
    # -  # ***FOREST      LZSN    INFILT      LSUR     SLSUR     KVARY     AGWRC
    1           0.00    4.0000    2.0000    100.00    0.0010    0.5000    0.9960
  END PWAT-PARM2
  PWAT-PARM3
    <PLS >***
    # -  #*** PETMAX    PETMIN    INFEXP    INFILD    DEEPFR    BASETP    AGWETP
    1                           10.000    2.0000       .00        0.       0.7
  END PWAT-PARM3
  PWAT-PARM4
    <PLS >                                                           ***
    # -  #      CEPSC      UZSN      NSUR     INTFW       IRC     LZETP***
    1          0.1000    3.0000    0.5000    1.0000    0.7000    0.8000
  END PWAT-PARM4
  PWAT-STATE1
    <PLS >  PWATER state variables***
    # -  #***   CEPS      SURS       UZS      IFWS       LZS      AGWS      GWVS
    1             0.        0.    0.5000        0.     2.000     0.100      .100
  END PWAT-STATE1
END PERLND
                                                                            ***
EXT SOURCES
                                                                            ***
<-Volume-> <Member> SsysSgap<--Mult-->Tran <-Target vols> <-Grp> <-Member->  ***
<Name>   # <Name> # tem strg<-factor->strg <Name>   #   #        <Name> # #  ***
```

```
WDM     11 PREC     ENGL      1.14          PERLND   1    3 EXTNL  PREC
WDM     21 EVAP     ENGL      0.7      DIV  PERLND   1    3 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***
PERLND   1 PWATER PERO   1                 WDM     31 PERO     ENGL AGGR REPL
PERLND   1 PWATER SURO   1                 WDM     32 SURO     ENGL AGGR REPL
PERLND   1 PWATER IFWO   1                 WDM     33 IFWO     ENGL AGGR REPL
PERLND   1 PWATER AGWO   1                 WDM     34 AGWO     ENGL AGGR REPL
END EXT TARGETS
                                                                             ***
END RUN
```

APPENDIX D

Meeting Notes

- Calibration period data. Water levels gages for calibration were installed on Lake River on February 1 and the Vancouver Lake Flushing Channel on March 4. Mike Knutson will download data the week of April 6 for a calibration period running from February 1 through April 3. Mike will provide data to NHC and OBAI.
- Lake River cross-section data is available and will be provided in digital form the week of April 6. Mike will put data onto FTP site.
- If overbank data is needed this could be obtained from the USGS DEM which provides data on a 30 meter grid. NHC will advise Mike Knutson if they need to obtain a copy of the DEM.
- The survey of the Shillapoo Lake is not yet complete. It should be available within 1 week.
- The Corps can provide loan copies of aerials. (Suitable aerials were identified and either copied or ordered following the meeting)
- Mike Knutson is to provide two additional UNET models - the February 1996 model and the 1997 model (which had been provided earlier but files were not retrievable). There is no great interest in modeling the February 1996 flood since the entire area was inundated. The February 1996 event will be modeled using roughness parameters from the spring period.
- NHC will need to develop a 1995 UNET model with roughness parameters adjusted to allow reasonable reproduction of water levels as they affect the hydraulics of Shillapoo Lake.
- All boundary condition data should be available by April 10 and will be provided to OBAI by Mike Knutson. Mike expects that there will be periods of missing data in the CROHMS record. He will provide assistance to OBAI in estimating missing data and will provide HECLIB macros to convert from irregular to regular time series data. Mike will also provide appropriate conversions from Columbia River Datum to NGVD.
- Determination of Willamette River flows (one of the boundary conditions) is complicated and requires that HEC-5 be run. Mike Knutson will provide assistance.
- OBAI has made preliminary contact with Clark County and has advised them of the data we would like to obtain. Clark County has agreed to provide data but timeliness may be a problem. Bob Elliot will provide OBAI with preferred path names for storing data in the DSS.
- Hydrologic modeling of runoff from the interior of Shillapoo Lake will be necessary to model interior water level regime. This will be done in an approximate fashion using precipitation data form Portland Airport rather than scaling Salmon Creek or Burnt Bridge

Meeting Notes
Columbia River Ecosystem Restoration at Shillapoo Lake
Kickoff Meeting

Date:	3 April 1998
Venue:	Portland District Office, US Army Corps of Engineers
Participants:	Karl Eriksen (USACE)
	Mike Knutson (USACE)
	Malcolm Leytham (NHC)
	Bob Elliot (NHC)
	Cynthia Lowe (Ogden Beeman Associates)

Schedule

A tentative schedule for subsequent meetings was set as follows:

Checkpoint No. 1 (50% submittal): Friday, 8 May, 1998
Checkpoint No. 2 (75% submittal): Wednesday, 3 June, 1998

Since WDFW is expected to attend these meetings, Mike Knutson was to provide these dates to Brian Calkins (WDFW).

Scope of Work

Key aspects of the scope of work were discussed as follows:

Existing Condition Model & Data Needs

Knutson stressed that principal interest in existing condition model was to ensure that water level regime in water courses surrounding Shillapoo Lake were being modeled with acceptable accuracy. Information needed to configure and calibrate the existing condition model were discussed as follows:

- Capacity and operating policy for existing pump station. No information is currently available on pump station capacity. This will be obtained from Brian Calkins (WDFW). The pump is not operated in the winter, so allowing flooding of the Shillapoo Lake, but is operated from about February 1 on with the intent of drying the area out sufficiently for farm work to start in the spring.

Creek data. Malcolm Leytham is to investigate availability of Portland Airport data on CD and will advise Mike Knutson of any data periods to be obtained via CROHMS.

A discussion of problems running long periods of data with UNET followed. Mike Knutson indicated he would contact HEC to determine whether a copy of UNET could be obtained with larger array sizes to facilitate long simulations (the current model can run with a maximum of about 1 month of hourly data). It was noted that with the inclusion of new branches, the UNET model branches should be renumbered to optimize execution time. NHC will renumber the model branches.

Natural Condition Model

It is not clear that there is much sense in developing a "natural condition" model as described in the SOW (i.e. removal of a significant portion of the surrounding levees). With a natural condition model as in the SOW, connections between Shillapoo Lake and Lake River may result in unacceptable flooding around Vancouver Lake. A more useful "natural condition" alternative would be to assume a substantial opening to the Columbia at for example Langsdorf Landing possibly with controlled outlets at the existing pump station and at the existing tide gate at the north end of Shillapoo Lake.

A better assessment of an appropriate "natural condition" scenario will be made after the topographic mapping is available and perhaps after the existing condition model has been run.

Constructed Alternatives

The general intent of the two remaining base reconnaissance level alternatives would be to investigate options for wetland management with and without pumping. The reconnaissance level alternatives will not include interior cell configuration which is to be provided by WDFW. Eriksen pointed out that USACE would like to downplay the use of pumps because of the operational and maintenance costs and sustainability of a pump scenario. The pump alternative would be examined to show how much pumping would be involved to achieve water management goals.

WDFW has plans to install a pump at Langsdorf Landing to supply the Buckmire Slough area (to the south of Shillapoo Lake). Pumping from this site could also supply Shillapoo Lake.

The biggest issue with constructed alternatives will probably be how to get water out of Shillapoo Lake . This could in part be via existing pump station.

Constraints

Possible constraints on the alternatives were discussed:

Land ownership - it will be assumed for investigating alternatives that there are no land ownership constraints. It was noted that WDFW's preliminary cell configuration follows property boundaries. Eriksen stated that NHC should rely on WDFW cell configurations but point out potential problems. Cells will be configured to allow flexibility in water management (see notes for 3 April WDFW meeting).

Data Requirments

Data requirements were summarized as noted under Scope of Work above.

Meeting Notes
Columbia River Ecosystem Restoration at Shillapoo Lake
WDFW/Ducks Unlimited Meeting

Date: 3 April 1998
Venue: WDFW Vancouver Office
Participants: Karl Eriksen (USACE)
Mike Knutson (USACE)
Jeff Dorsey (USACE)
Malcolm Leytham (NHC)
Bob Elliot (NHC)
Brian Calkins (WDFW)
Steve Donovan (Ducks Unlimited)

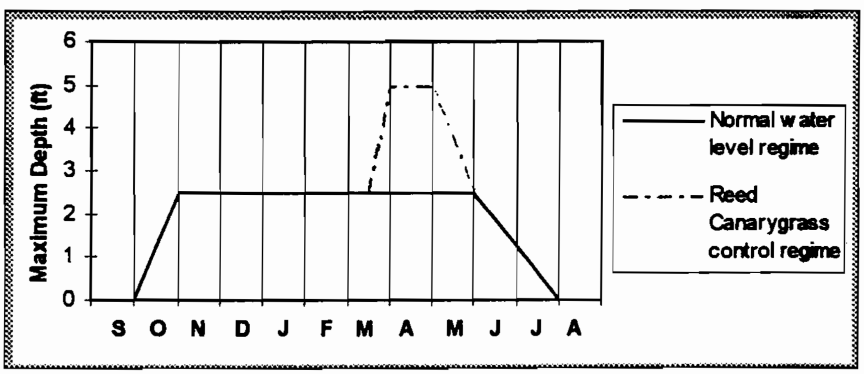
The principal purpose of the meeting was to obtain information on the environmental objectives of the restoration and the water management goals. Pertinent information as it might affect alternatives to be investigated in this study were provided primarily by Brian Calkins and Steve Donovan as follows:

Project Objectives

The principal project objective is restoration of wetland habitat at Shillapoo Lake.

Water Management Goals

The desired water level regime is approximately as shown in the "rule curve" below:



Principal considerations in arriving at this regime are as follows:

- Winter/spring - maintain maximum depth of about 2.5 ft with flexibility to increase depths to 5 or 6 ft for reed canarygrass control (see below).
- Late May/early June start drawing down to expose mud flats.
- Allow cells to dry up by August. (Need to have capability to dry areas out completely for Carp control).
- If Reed Canarygrass becomes a problem need the flexibility to increase water depths to 5 or 6 ft at the start of the growing season to drown plants out.
- Drawdown to allow native plants to mature and produce seed could be as late as 1 July.

Cell Configuration

NHC's modeling of base alternatives will not include interior cells. Cells will be included in modeling of the final or selected alternative. This gives WDFW about a month (until early May) to develop cell configuration concepts. Cells will be designed to allow flexibility in management or rotation of management strategies - e.g. reed canarygrass control in some areas but not in others. The existing expulsion pump will likely be maintained and could be used to drawdown water levels.

Constraints

Fish Ingress/Egress

This is currently a controversial area. It is likely that measures will be needed to keep fish out because of potential predation of juvenile salmonids. Dorsey mentioned use of a porous rock dike in lieu of screens. This has been suggested as an experimental method of screening but apparently none have yet been constructed.

Water Rights

WDFW has a water right for 1000 cu. ft./minute for the proposed pump station at Langsdorf Landing. However NHC will ignore potential water right constraints in its work.

Miscellaneous

Model Output

A brief discussion of model output needed by WDFW identified the following:

- rate of cell filling/emptying
- maximum cell water levels.

Expulsion Pump

Brian Calkins will determine the capacity of the existing pump station.

Successful Projects

NHC was interested in obtaining information on similar successful restoration efforts. Steve Donovan and Brian Calkins suggested talking to Jim Houck at the Finley Wildlife Refuge. Examples of successful projects include Basket Slough, and Tualatin Wildlife Refuge.

Meeting Notes

Columbia River Ecosystem Restoration at Shillapoo Lake

50% Checkpoint Meeting

Date:	22 May 1998
Venue:	Portland District Office, US Army Corps of Engineers
Participants:	Kark Eriksen (USACE)
	Mike Knutson (USACE)
	John Todd (USACE)
	Jeff Dorsey (USACE)
	Malcolm Leytham (NHC)
	Brian Calkins (WSFW)

The principal purpose of this meeting was to present the results of analysis of the base level alternatives and to establish concurrence on the basic approach to supplying water to and draining water from Shillapoo Lake. A tentative layout for the wetland cell configurations was also discussed. However, Malcolm Leytham presented the results of analysis of the base level alternatives, the principal conclusions from the work accomplished to date was as follows:

1. Minimum summertime water levels in Lake River are between +3 and +5 ft NGVD. This means that draining of Shillapoo Lake will always require pumping. It was noted that the existing pump station is reported to have a capacity of 22.3 cfs, is a reasonably good fit to the volumes and timing of water that would need to be evacuated from the lake.

2. Given the prevailing water level ratim(?) in the Columbia River, it would not be possible to reliably fill the northern area of Shillapoo Lake by gravity. The northern area has a minimum elevation of about +8, which would suggest a maximum normal water surface elevation in that area of about +10.

3. Given the prevailing water level ratim(?), a gravity supply of water for the canarygrass control cannot be guaranteed in all years. For the main body of the lake, water levels for reed canarygrass control would need to be at about +7 and for the northern area as much as +13.

4. A gravity supply of water to the main body of Shillapoo Lake for shallow flooding to an elevation of about +4 or +5 could be very readily achieved via Lake River, even in dry years.

There was an overall agreement on the conclusions reached from the base level alternatives analysis. In particular, it was accepted that a gravity supply from Lake River at the site of the existing expulsion pump would be the most logical means of supplying water to Shillapoo Lake and that a gravity supply from the Columbia was impractical because of the low water levels. A discussion followed to try to identify a preferred alternative and to develop tentative configurations for wetland cells for the final alternative. The following points were noted:

5. While the target water level ratim(?) discussed in the 3rd of April meeting showed Shillapoo Lake filling to a maximum depth of 2.5 ft in the month of October, Brian Calkins noted that slower filling would be okay and that filling to the target elevation by the end of December would be satisfactory.

6. Knutson noted that forecasting of flows on the Columbia River for the spring period might be useful for planning spring operations for reed canarygrass control in which Shillapoo Lake water levels would be raised to as high as +7 in the main body of the wetland and perhaps +13 in the northern part of the wetland.

7. Dorsey suggested that pipe systems could be used to distribute water from the approximate location of the expulsion pump to the various wetland cells. Calkins agreed that pipes would be okay from the point of view of WDFW.

8. It was noted that the Fish and Wildlife Service has recently constructed a stream pump on Lake River (presumably as part of the Richfield project) and that screening of the pump intake for Shillapoo Lake would be necessary. Dorsey noted that the screening might be achieved as disucussed in the 3rd of April meeting by use of a gravel filter.

9. It was recognized that because of the water level ratim, it would not be possible to supply or to reliably to the north area and that filling of the north area would have to rely on the combination of interior runoff and pumping. Pumping could occur either from Lake River via the existing historic channel connection, or by pumping from an adjacent wetland cell.

10. It was noted that rock overflow spillways would be needed in the berms that would be constructed to separate wetland cells.

11. A brief discussion was held of the area of the lake that would be purchased by WDFW. It was noted that Fazio only wanted to sell what he has called the lakebed portion of his property. It was decided to assume that Fazio would retain land at elevation 10 and above and that he would sell his property below elevation 10.

12. WDFW indicated that use of portable pumps to fill individual wetland cells would perhaps be satisfactory rather than having to construct pipe line or ditch supply systems.

13. The liability of interior runoff as a source of water was discussed. It was noted by Leytham that in most years there would likely be enough runoff in the months of October through December to meet target water level ratims. However, there is extreme uncertainty as to the interior enough amounts, especially when one considers the lack of information on groundwater in the first of Shillapoo Lake, including seepage through the levee system. Calkins noted that the spring south end of Shillapoo Lake has been observed to be dry in the summer months. There is no information on flow rates from the artesian well in the lakebed; however, Calkins indicated that the supply from that well is through a 4-inch diamond pipe.

14. A discussion was held of possible wetland cell configurations. Eriksen advocated a configuration in which any one cell could be managed to achieve desired water levels by feeding water through a maximum of one adjacent cell. If this approach were acceptable to WDFW then the project could be implemented without having to construct supply ditches or pipelines. WDFW indicated that this leap frog approach to filling cells would be acceptable.

15. A brief discussion was held of the potential water quality implications of a gravity supply from Lake River. Calkins noted that water from the water quality perspective, a supply from the Columbia would probably be better. However, this one consideration was out weighed by the advantages of a low-level gravity supply from Lake River. It was noted that because WDFW intends to drain the wetland area each year, it is unlikely that water quality would become a serious problem.

16. The potential for augmenting a supply from Lake River via a pump supply from the Columbia was discussed. This might be particularly useful for raising water levels to wetland cells for reed canarygrass control. Calkins noted that WDFW was proceeding with the construction of a pump station close to Landsdorf Landing. This had previously been reported as having a capacity of only 1,000 gallons per minute. However, there now seems to be some uncertainty as to the capacity of that pump station. Calkins will find out what the proposed pump station size is to be and will pass that information to Leythem and Knutson.

The main part of the meeting concluded with Leytham suggesting that NHC take the tentative wetland cell configuration prepared by Calkins and define it in the light of the discussions at the meeting and return the revised wetland cell configuration to Knutson and Calkins for their review. This was to be done early in the week of June 1. The meeting continued with Leytham and Knutson present only to discuss modifications to the technical approach to the project. The principal items of discussion was as follows:

17. Leytham noted that UNET model seems to be inappropriate for modeling at the level of detail needed for final design and that it is poorly suited to looking at operational issues. Leytham suggested that an alternative modeling approach be used. This would entail development of a ROUTE model by NHC. ROUTE is based on Canadian inland waterways directorate SIMPAK model, which is basically a water accounting model. Knutson agreed that this approach was acceptable.

18. Knutson expressed concern that the ability to drain Shillapoo Lake in a wet year be investigated. The concern here is that high spring freshet combined with high spring rainfall may result in significant amounts of interior runoff that cannot be handled by the existing expulsion pump. Knutson stressed that the final design, although it should recognize the existence of agency facilities, should not necessarily rely solely on those existing facilities. In other words, consideration should be given to using a larger pump as necessary. Leytham noted that there is no information on groundwater inflow rates from the Columbia but that a possible approach would be to look at the sensitivity of drain time to super?????.. rates. Knutson will research the availability of seepage inflow rates for Columbia River levees in general and will provide that information to Leytham.

19. The ability to fill by gravity in a dry year is clearly of importance. It was decided that assessment of the ability to fill by gravity in a dry year should be based on the period October through December 1993, this being the driest October through December in the period of record from 1973 through present. Knutson will supply hourly water level data from Vancouver for that period and NHC will establish a relationship between the Vancouver water level data and Lake River levels, based on units in relation for water year 1995, ??? depleted??

The date for the next meeting, the 75% Completion Checkpoint meeting was set for Wednesday, June 10.

Meeting Notes

Columbia River Ecosystem Restoration at Shillapoo Lake

75% Checkpoint Meeting

Date: 9 June 1998

Venue: Portland District Office, US Army Corps of Engineers

Participants: Laura Hicks (USACE)
Mike Knutson (USACE)
Karl Ericksen (USACE)
John Todd (USACE)
Jeff Dorsey (USACE)
Malcolm Leytham (NHC)
Brian Calkins (WDFW)

The principal purpose of this meeting was to present the work accomplished to date on definition and analysis of the recommended alternative. The conceptual design of water control structures, embankments, and conveyance systems was also discussed.

The meeting began with an overview of the final wetland cell configuration. A tentative layout for the wetland cells had been established at the 21 May checkpoint meeting. This had subsequently been refined by NHC in consultation with WDFW and USACE. It was agreed that the final cell configuration was acceptable.

Leytham presented three guidelines developed by NHC in consultation with WDFW for the design of the water control structures and assessment of the performance of the recommended alternative. These guidelines were as follows:

1. It should be possible to fill the project to elevation +4 ft by 31 December in the driest water year of the period 1973 to the present; that is in water year 1977.

2. Water control structures should be sized so that in an average year, target normal maximum operating levels of about +4 ft can be maintained to within ± 0.5 ft in the period November through May. No such restrictions would be placed on deviations from target levels in wet years providing water levels did not threaten the integrity of the facility.

3. It should be possible to drain the facility from elevation +4 ft in the wettest June and July in the period 1973 through present.

Leytham presented results of analyses from the UNET and ROUTE models of the project. The UNET model demonstrated that there was no difficulty in filling the main body of Shillapoo Lake to +4 ft in the driest year since 1973. However, Leytham pointed out that simulated Lake River water levels were subject to uncertainty and may be in error by ± 0.5 ft or more, implying some uncertainty as to the ability to achieve +4 ft in a dry year. This level of uncertainty was deemed to be acceptable by the participants.

Simulations of the ability to maintain target water levels were done using the ROUTE MODEL with a range of seepage rates into Shillapoo Lake from 1 cfs to 10 cfs. In an average year, the recommended alternative is able to maintain water levels within 0.5 ft of the target levels and meet the guidelines developed by NHC. However, in wet year simulations for 1996 and 1997, the model results show significant deviations from the target levels because of the inability of the expulsion pump at its current capacity of 22.3 cfs to handle the wet year inflow rates.

A discussion ensued about the possible need for upsizing the expulsion pump. Dorsey and Calkins both felt that if the project was able to meet targets in an average year, that would be acceptable and that deviations from those targets in a wet year was not of great concern from the point of view of wetland restoration. The possible need to develop firmer data on seepage inflow rates was discussed. One possible approach to this would be to estimate pumpage rates from pump electricity usage in water year 1997. After some discussion it was decided not to pursue the development of more reliable seepage data. It was also decided that the expulsion pump should be maintained at its existing capacity and to both accept the uncertainty of seepage rates and the fact that water levels would exceed target levels in wet years.

Leytham presented results from a somewhat conservative analysis of the ability to drain the project from elevation +4 ft in the wettest June and July since 1973. The performance of the project with the existing expulsion pump was again deemed by the group to be acceptable.

Leytham then presented NHC's conceptual layout and designs for water control structures, embankments, and conveyance facilities for the proposed project. The following points were noted by the participants:

1. With the layout currently being proposed by NHC, it would not be possible to completely drain cells independently of one another. NHC's current concept allows projects to be filled independently and controlled for Reed Canarygrass independently, but does not permit individual cells to be completely drained independently of one another. Minor revisions to the proposed channel system were suggested, which would improve the flexibility of project. These modifications will be incorporated into the conceptual layout by NHC.

2. Dorsey raised the point that nutria will likely be a problem at the restored project and could cause significant damage to levees and embankments. The levee design proposed by NHC, which was for an embankment with a 10 ft top width and a 3H:1V side slopes was modified so that 3H:1V side slopes were maintained down to an elevation of +6 ft and side slopes below +6 ft were flattened out to 6H:1V. The flatter side slope of the base of the levee is intended to increase the base width of the levee and reduce the susceptibility to damage by nutria.

3. The issues surrounding fish screening were discussed. It was noted that screens would be required on both the 42-inch gravity inlet and the 36-inch gravity inlet to Shillapoo Lake. Screening would be required for juveniles. Dorsey pointed out that conventional screens would present a significant maintenance problem. Screening by means of a gravel berm had

been suggested in previous meetings. This, and the potential for such filters to be plugged by sediment and algae, were again discussed. It was agreed that design of fish screens was outside NHC's scope. Dorsey agreed to pursue the development of design information for screening by means of a gravel berm.

4. The basic system of water control structures presented by NHC consists of three main types of structures to control water levels in individual wetland cells. NHC's philosophy here was to develop very simple structures at the expense of increasing the number of structures. It was acknowledged that one or more of these structures could be combined to reduce the total number of structures required. At Hicks' suggestions, NHC's report will continue to document the existing concept of three basic structures but will point out that the functions of one or more of these structures could be combined. Development of alternative structures will be investigated by the Corps in final design.

5. Calkins noted that WDFW was currently in the process of designing a control structure on the channel to Buckmire Slough. NHC was also proposing a structure on this channel to allow the diversion of water from the Langsdorf Landing pump station to Shillapoo Lake. The design of these structures should be coordinated so that the WDFW structure meets the performance standards required to deliver water from the Langsdorf Landing pump station to Shillapoo Lake.

6. Leytham pointed out that there were a certain number of unknowns in using existing facilities at Shillapoo Lake. These included:

   - Lack of specific information on the expulsion pump and its controls
   - Lack of information on the condition of the 42-inch culvert adjacent to the expulsion pump
   - Lack of information on the condition of the 36-inch culvert at the northern end of Shillapoo Lake.

   Hicks requested that NHC include these comments in its report.

1. Leytham reported that as requested at the 50% checkpoint meeting, NHC had looked into the use of portable pumps as a means of augmenting water supply to the project. Because of the amount of water needed to fill wetland cells, this idea has been found to be impractical and will not be pursued further.

A target date of Friday, 19 June, was set for completion of the draft final report.

Salmon Creek
Round Lake
Curtis Lake
8.7
96
97
OREGON
WASHINGTON
Columbia River
9.4
Lake River
Scale 1" = 600'
600 300 0 600 1200 ft
Contour Interval 1 foot
National Geodetic Vertical Datum 1929
Shillapoo Lake
10.5
10.8
11.0
Langsdorf Landing
Caterpillar Island
Lower River Road
Buckmire Slough
Lake Vancouver
SHILLAPOO LAKE RESTORATION
Topographic Map
Source:
USGS 1:24,000 quads
Vancouver, WA-OR and Suavie Island, OR-WA
Portland District COE Surveys 1998
File: PLATE1.DGN
Date: June 11, 1998
Northwest Hydraulic Consultants
Tukwila, WA
U.S. Army Corps of Engineers
Portland District