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IMPACT ON THE COLUMBIA RIVER OF AN OUTBURST OF SPIRIT LAKE

By W. G. Sikonia

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ABSTRACT

A one-dimensional sediment-transport computer model was used to study the effects on the Columbia River of an outburst of Spirit Lake, near Mount St. Helens, Washington. According to the model, for an average flow of 233,000 cubic feet per second in the Columbia River, sediment from the Cowlitz River would block the Columbia River to a height of 44 feet above the current streambed, corresponding to a new streambed elevation of -3 feet with respect to the National Geodetic Vertical Datum of 1929 (NGVD), and would impound the waters of the Columbia River. Water-surface elevations upstream from the blockage would continue to increase for 16 days after the blockage formed. The river elevation at the Trojan Nuclear Power Plant, 5 miles upstream of the Cowlitz River, would rise to 32 feet; the critical elevation above which the plant would be flooded is 45 feet. The corresponding elevation without the blockage is 6 feet. High water-surface elevations would occur along the river to Bonneville Dam; for example, at Portland, Oregon, the elevation would be 32 feet with blockage and 10 feet without. If the outbreak were simultaneous with a 2-year flood of 410,000 cubic feet per second on the Columbia River, the Columbia would rise for 14 days to elevations of 38 feet at Trojan and 39 feet at Portland, compared to elevations of 11 and 16 feet, respectively, without the blockage. If the outbreak were simultaneous with a 100-year flood of 850,000 cubic feet per second on the Columbia River, the Columbia would rise for 10 days to elevations of 44 feet at Trojan and 45 feet at Portland, compared to 21 and 26 feet respectively, for such a flood without the blockage.

INTRODUCTION

A debris avalanche caused by the eruption of Mount St. Helens, Washington, on May 18, 1980, blocked the outlet of Spirit Lake and raised the lake level. Extreme concern exists over the stability of the dam left by the debris avalanche and the hazard presented by the possibility that the dam could be breached. Failure could cause devastating floods along the Toutle, Cowlitz, and Columbia Rivers (fig. 1). In early modeling efforts, Swift and Kresch (1983) predicted inundation along the Toutle and Cowlitz Rivers resulting from a hypothetical outburst of Spirit Lake, and Kresch and Laenen (1983) investigated the effect such an outburst would have on the Trojan Nuclear Power Plant on the Columbia River in Oregon. Bissel and Hutcheon (1983) studied both these reaches and the lower Columbia River, the subject of this report, but did not model the dynamics of the sediment transport.

The U.S. Geological Survey was requested by the Federal Emergency Management Agency (FEMA) in 1983 to study the distribution and timing of sedimentation and flooding along the lower part of the Columbia River, from Bonneville Dam to its mouth, in the event of a breakout of Spirit Lake. The study was designed to assess the impact of a breakout flood upon public safety and the regional economy and to aid FEMA in planning for the disruption such an event would cause.

A one-dimensional sediment transport model written by D. L. Fread of the National Weather Service was used to investigate the impact that an outburst flood would have on the lower reach of the Columbia River. The model was edited and modified to make it applicable to the study. The model's base is the Operational Dynamic Wave Model (DWOPER) (Fread, 1978; 1982) used by the National Weather Service for flood and day-to-day river forcasting. Sediment transport has been added to the model. The application is part of a longer-term project to develop a sediment transport model or set of models that will allow more comprehensive and accurate modeling than is now possible.

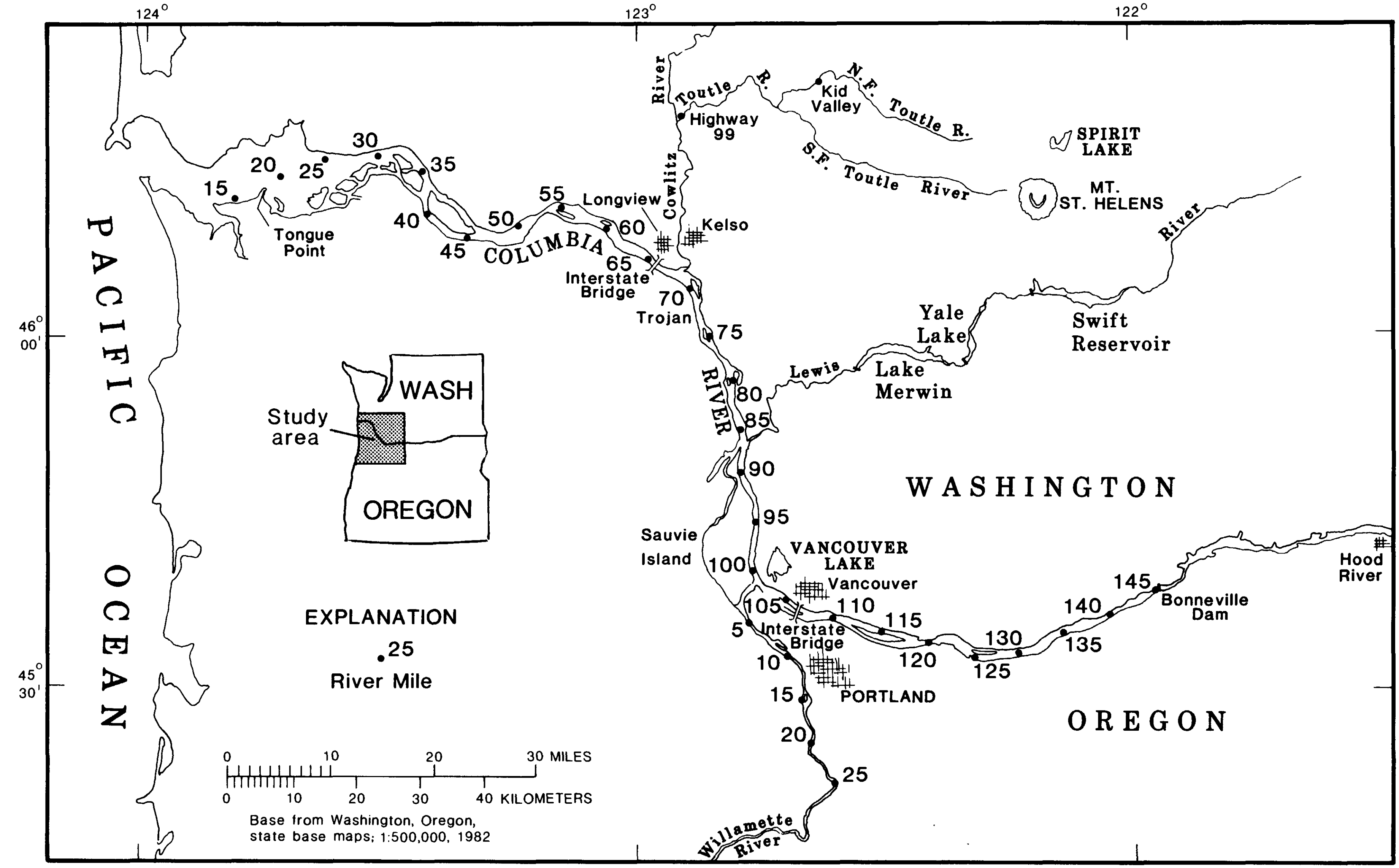

FIGURE 1.--Location of the study area showing river-mile coordinates.

MAGNITUDE AND COMPOSITION OF FLOOD INPUT TO THE COLUMBIA RIVER

The study described in this report was designed to arrive at a likely scenario for flooding and inundation levels along the Toutle and Cowlitz Rivers that was based on more probable conditions than those of Swift and Kresch (1983) and Kresch and Laenen (1983), who made assumptions that would produce some of the worst flooding and inundation levels. Swift and Kresch assumed that a bulk volume of 2.4 billion cubic yards (bcy) of debris material would be entrained by an outburst of Spirit Lake. This figure was obtained by adding enough debris material to 0.51 bcy of water from Spirit Lake to yield 65 percent sediment concentration by volume. On the basis of field measurements, the debris porosity and degree of saturation were assumed to be 32 and 50 percent, respectively, so the 2.4 bcy of bulk debris material added

$$2.4 \text{ bcy} \times (1.0 - 0.32) = 1.63 \text{ bcy} \quad (1)$$

of solids , and

$$2.4 \text{ bcy} \times 0.32 \times 0.50 = 0.38 \text{ bcy} \quad (2)$$

of pore water. The present (1984) degree of saturation is 90 percent (Meyer, written commun., 1984), rather than 50 percent, and it would be impossible to bulk the flow to 65 percent sediment concentration as in the earlier study: even inclusion of the entire 3 bcy of avalanche debris, which is not anticipated, would provide

$$3 \text{ bcy} \times (1.0 - 0.32) = 2.04 \text{ bcy} \quad (3)$$

of solids, but would add

$$3 \text{ bcy} \times 0.32 \times 0.90 = 0.86 \text{ bcy} \quad (4)$$

of water to the 0.51 bcy of water from Spirit Lake, for a solids concentration of

$$2.04/(2.04 + 0.86 + 0.51) = 60 \text{ percent} \quad (5)$$

by volume. For this study the author assumed that of the total 3 bcy of avalanche debris, 1.3 bcy of bulk material was a reasonable fraction to be scoured and entrained in the flow on its path downvalley from Spirit Lake, but included the water that would be contained in this material, namely

$$1.3 \times 0.32 \times 0.90 = 0.37 \text{ bcy} \quad (6)$$

in the total volume flowing downstream. To summarize (table 1), it was assumed that

$$1.3 \text{ bcy} \times (1.0 - 0.32) = 0.88 \text{ bcy} \quad (7)$$

of solids and

$$0.37 + 0.51 = 0.88 \text{ bcy} \quad (8)$$

TABLE 1.--Summary of flood magnitude and composition

Composition Just Below the Debris Dam

0.88 bcy	solids from debris = 1.3 bcy x (1.0 - 0.32)
0.37 bcy	water from debris = 1.3 bcy x 0.32 x 0.90
0.04 bcy	air from debris = 1.3 bcy x 0.32 x 0.10
0.51 bcy	water from Spirit Lake
1.76 bcy	total sediment plus water (50 percent sediment by volume)

Size Distribution Within the Debris Dam

40 percent greater than 5 millimeters
40 percent between 0.062 and 5 millimeters
20 percent less than 0.062 millimeters
(Porosity = 32 percent)

Composition at the Mouth of the Cowlitz

0.35 bcy	solids = 0.88 bcy - 0.53 bcy (in deposits)
0.48 bcy	water = 0.88 bcy - 0.40 bcy (in deposits)
0.83 bcy	total sediment plus water (42 percent sediment by volume)

Size Distribution at the Mouth of the Cowlitz

38 percent greater than 0.2 millimeters
38 percent between x and 0.2 millimeters
24 percent less than x millimeters

where x millimeters is an unspecified wash load delimiting size less than 0.062 millimeters
(Porosity of deposits = 43 percent)

Peak Discharge at the Mouth of the Cowlitz

124,500 ft^3/s	bed material sediment
38,900 ft^3/s	wash load sediment
245,600 ft^3/s	water
409,000 ft^3/s	total (30 percent bed material sediment by volume)

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of water would be incorporated in the flood at the debris dam, for a total volume of 1.76 bcy and a sediment concentration of 50 percent by volume.

In their study Swift and Kresch (1983), making largely conservative assumptions, considered no sediment deposition from a mudflow along the Toutle and Cowlitz valleys. However, considerable deposition, particularly of the larger particles, would be expected on the basis of previous mudflows (Dinehart, written commun., 1984). For the mudflow of March 19-20, 1982, for example, 12 percent of the fines (material less than 0.062 millimeters in diameter) and 46 percent of the larger material (36 percent of the total material) were deposited in the 20-mile reach of the North Fork Toutle and Toutle Rivers between Kid Valley and Highway 99. Tracking the small event of March 19-20, 1982, became difficult in the 21-mile reach between Highway 99 and the mouth of the Cowlitz River because of mixing with the flow of the Cowlitz River. A large mudflow due to the outburst of Spirit Lake could be expected to form additional deposits there. Preferential deposition of the larger sediment particles occurred during the March 19-20, 1982, mudflow: at Kid Valley, fines accounted for 29 percent of the sediment in transport, and larger material the remaining 71 percent. When the mudflow reached Highway 99, the fines accounted for 40 percent of the transported material.

The deposition of the larger particle sizes can also be deduced theoretically using Shield's criterion for incipient motion of sediment particles (Graf, 1971), given by

$$F = \tau/((\gamma_s - \gamma)\ D) \qquad (9)$$

where

$$\tau = \gamma\ R\ S \qquad (10)$$

is the shear stress on the bed, and

γ_s = specific weight of the sediment particles
γ = specific weight of water
D = particle diameter
R = hydraulic radius
S = slope
F = $fct(U_* d/\nu)$ = a dimensionless function
U_* = shear velocity
ν = kinemetic fluid viscosity

For application to the Toutle and Cowlitz Rivers, F can be taken as 0.047, and we also assumed that γ can be generalized to specific weight of the mixture for the hyperconcentrated flows under consideration here, rather than just the specific weight of water. The specific gravity of the sediment particles is approximately 2.65. For a sediment concentration of 50 percent by volume, the specific gravity of the mixture is 1.83, and for a sediment concentration of 42 percent, it is 1.69. Solving equation 9 for D provides estimates of the maximum particle sizes that one can expect to be transported (table 2). From table 2 it can be seen that, at least for hyperconcentrated flow as opposed to debris flow, particles larger than 33 millimeters would be deposited before the flood reaches the Columbia River. This is the maximum size to be transported; one would expect that some deposition of particles smaller than this would also occur. This picture must be modified somewhat, in that the front of the flood may have higher sediment concentration than average and be more in the nature of a non-Newtonian mudflow than the hyperconcentrated flow for which the analysis is valid, because such a rock-matrix-supported flow tends to produce deposits that are less sorted by size. The picture is, however, one to be expected for the flood-averaged sediment concentration.

TABLE 2.--Particle size for incipient motion from Shield's criterion

River	Specific gravity γ_s/γ_w	Slope S	Hydraulic radius R (meters)	Particle diameter D (millimeters)
N. F. Toutle	1.8	0.006 to 0.007	12	1900 to 2200
Toutle	1.8	0.0045	12	1400
Cowlitz	1.7	0.00006 to 0.0004	18	33 to 220

The composition of the avalanche debris (Lipman and Mullineaux, 1981) is 40 percent coarse particles (greater than 5 millimeters), 40 percent sand (0.062 to 5 millimeters), and 20 percent fine particles (less than 0.062 millimeters). On the basis of observations of previous mudflows and the argument using Shield's criterion, it was estimated that 60 percent of the material entrained near Spirit Lake, particularly the larger sized particles, would be deposited before the flood reached the Columbia River. This figure is subject to considerable uncertainty, and an estimated 15 percent standard error (68-percent confidence limits) would not be unreasonable. Thus, the deposits included

$$0.88 \text{ bcy} \times 0.60 = 0.53 \text{ bcy} \tag{11}$$

solids, and also retained

$$(0.43/(1.-0.43)) \times 0.53 \text{ bcy} = 0.40 \text{ bcy} \tag{12}$$

of water in the pore spaces, using a sediment-deposition porosity of 43 percent that was based on sediment samples taken May 20, 1980, (U.S. Army Corps of Engineers, 1981) and recent sediment studies on the Cowlitz River (Lombard, written commun., 1984). The remaining flood entering the Columbia River contained

$$0.88 - 0.53 = 0.35 \text{ bcy} \tag{13}$$

of sediment and

$$0.88 - 0.40 = 0.48 \text{ bcy} \tag{14}$$

of water, for a total volume of 0.83 bcy and a sediment concentration of 42 percent by volume.

Swift and Kresch assumed that 30 percent of the sediment reaching the Columbia River would be wash load of fine material in suspension, and this would be carried through to the lower Columbia. However, the context is somewhat different for our study than for theirs. Wash load usually refers to sediment with particle sizes smaller than those represented by the bed material and subject to uncertain introduction by upstream sources such as bank erosion. For our situation, the material introduced from upstream sources—that is, by the outburst flood—would have a complete size distribution, including fine-grained material. It would essentially become the bed material of question for this problem, and the pre-existing bed would be of little concern. Thus, for this study the wash load was not related to vagaries of source, but rather to possible inadequacies of the sediment transport relation to treat very fine material properly when giving the balance between material in the bed and sediment in transport.

A study of prior mudflows can provide some guide to the proportion of fine material transported and deposited. During the May 19, 1980, mudflow, 39 percent of the material in transport was fines (Dinehart, written commun., 1984). Analysis of sediments (U. S. Army Corps of Engineers, 1981) showed that at the mouth of the Cowlitz River, 28 percent of the deposited sediment was fines; in the Columbia River at the mouth of the Cowlitz River, virtually none of the deposited sediment was less than 0.062 millimeters. This difference in deposition of fines is presumably related to the differing sediment transport capacities of the Columbia River and the smaller Cowlitz River. For the situation modeled in our study, the sediment deposit itself would form a blockage that would substantially reduce, and even reverse, the discharge of the Columbia, so that we expect that the Columbia River deposits would contain more fines and would be more like the deposits in the Cowlitz River on May 19, 1980, than those in the Columbia River for that date. For this reason, and because of the connection between assumed wash load and the adequacy of the sediment transport relation to describe fines, it was assumed in this study that the finest 24 percent of the sediment would be wash load. The uncertainty in this value is quite high, estimated to be given by a standard error of plus or minus 10 percent.

The total discharge (water + sediment) hydrograph used in our study for the Cowlitz River at its junction with the Columbia River was obtained by adjusting the hydrograph developed by Swift and Kresch (1983). Their hydrograph was adjusted to have a reduced volume of 0.83 bcy, added to an assumed pre-existing Cowlitz River flow of 20,000 ft^3/s, during the 28-day period used in this study. Our total-discharge hydrograph is shown in figure 2. Zero on the time scale is when Spirit Lake begins to breach through the avalanche debris. The resulting flood would reach the Columbia River 9 hours after the breach, and would have a peak discharge of 409,000 ft^3/s at 16 hours. Recall that the assumed sediment concentration reaching the Columbia River due to the outburst was 42 percent by volume, and that 24 percent of this was assumed to be washload of fines. Thus,

$$(1.00 - 0.24) \times 42 \text{ percent} = 32 \text{ percent} \tag{15}$$

of that portion of the flood discharge attributable to the outburst (namely 409,000 - 20,000 = 389,000 ft^3/s) was the so-called bed material load, to be treated in the model by the sediment transport equation in the modeling, and 10 percent was wash load. The bed material sediment-discharge contribution from the pre-existing 20,000 ft^3/s was assumed to be 0.1 percent, or 20 ft^3/s; the corresponding wash load contribution was assumed to be 0.03 percent, or 6 ft^3/s. Thus, the portion of the 409,000-ft^3/s peak total discharge attributable to bed-material sediment discharge was

$$0.32 \times 389{,}000 + 0.001 \times 20{,}000 = 124{,}500 \text{ ft}^3/\text{s} \tag{16}$$

Similarly, the portion of the peak total attributable to wash-load sediment discharge was

$$0.10 \times 389{,}000 + 0.0003 \times 20{,}000 = 38{,}900 \text{ ft}^3/\text{s} \tag{17}$$

The sediment discharges for times other than at peak flow were calculated in a similar manner. A mean sediment diameter of 0.2 millimeter and porosity for sediment deposition in the Columbia River of 43 percent were assumed on the basis of the previously mentioned studies (U.S. Army Corps of Engineers, 1981; Lombard, written commun., 1984).

DESCRIPTION OF THE COMPUTER MODEL

The computer model used in the study (Fread, 1978; 1982) is based on a four-point implicit finite-difference scheme. It was chosen for this study because of its application in similar previous studies and because of its standard treatment of the relevant equations. The water-discharge modules have been used by the National Weather Service, the Corps of Engineers, and the Geological Survey for one-dimensional modeling for flood, dam-break, and day-to-day river forecasting. The water-discharge modules provide the core of the program known as the Operational Dynamic Wave Model, or DWOPER model; it contains the full non-linear development of the Saint Venant equations and can treat a limited river network involving first-order tributaries via an iterative scheme.

The Saint Venant equations consist of the conservation of total mass, that is, of water plus sediment,

$$\frac{\partial Q}{\partial t} + \frac{\partial (A+A_o)}{\partial t} - q = 0 \tag{18}$$

and the conservation of momentum equation,

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA(\frac{\partial h}{\partial x} + S_f + S_e) + L + W_f B = 0 \tag{19}$$

where

$$S_f = \frac{n^2 \; |Q| \; Q}{2.21 \; A^2 \; R^{3/4}} \tag{20}$$

$$S_e = \frac{K_e}{2g} \frac{\partial (Q/A)^2}{\partial x} \tag{21}$$

$$L = -q(V_\ell - Q/A) \tag{22}$$

$$W_f = -C \; |V_w \cos \omega - Q/A \; |(V_w \cos \omega - Q/A) \tag{23}$$

In these equations,

- x = distance along the longitudinal axis of the waterway
- t = time
- Q = total (water + sediment) discharge
- A = active cross-sectional area
- A_o = inactive (off-channel) storage area
- q = total (water + sediment) lateral inflow (positive) or outflow (negative)
- g = gravity acceleration constant
- h = water-surface elevation
- B = wetted top width of cross section
- L = momentum effect of lateral inflow
- S_f = friction slope computed from Manning's equation
- n = Manning's coefficient
- S_e = local loss slope due to sudden channel expansion or contraction
- W_f = wind term
- R = hydraulic radius
- K_e = expansion (negative) or contraction (positive) coefficient
- V_ℓ = component of lateral flow velocity in downstream direction
- C_w = dimensionless wind coefficient
- V_w = wind speed
- ω = angle between wind vector and downstream channel direction

The unknown variables for the model are thus total discharge, Q, and water-surface elevation, h. Channel geometry at a selection of cross sections is approximated by piecewise-linear functions as part of the input data. The active and off-channel areas and the wetted top width corresponding to h are determined at each Newton-Raphson iteration within each time step. Thus, irregular channel topography is taken into account in the equations, even though the model is referred to as a one-dimensional model (in longitudinal river coordinate x). Higher dimensional models would provide the details of the velocity distribution over the cross section, but at the expense of increased computer time.

Recently, modules for sediment transport and sediment conservation have been added to the model, including the approaches of Yang, Colby, Toffaleti, Meyer-Peter and Muller, DuBoys, and sediment transport ratings as functions of stage or discharge (Simons and Senturk, 1977). The sediment continuity equation is

$$\frac{\partial Q_s}{\partial x} + \frac{\partial}{\partial t}\left(C_s(A+A_o-A_s)\right) + \frac{\partial}{\partial t}\left((1-p)\ A_s - q_s = 0\right. \qquad (24)$$

where

Q_s = sediment discharge
C_s = sediment concentration by volume
A_s = sediment deposition (positive) or scour (negative) cross-sectional area
p = porosity of sediment deposit
q_s = lateral sediment inflow (positive) or outflow (negative)

A space-integrated form of this sediment continuity equation, similar to what would be used in a finite-element analysis, is used that provides a full n equations for the n values of cross-sectional deposition or scour.

In our study, the Yang sediment transport equation was employed; it is a simple, easily used equation for total bed material load. Explicitly, the equation is as follows:

$$\begin{aligned}\log C_t = {} & 5.435 - 0.286 \log (wD/\nu) - 0.457 \log (U_*/w) \\ & + (1.799 - 0.409 \log (wD/\nu) \\ & - 0.314 \log (U_*/w)) \log((US/w) - U_{cr} S/w))\end{aligned} \quad (25)$$

where

C_t = total sediment concentration in parts per million by weight
D = median sieve diameter
S = water-surface slope or energy slope
U_* = shear velocity
U = average water velocity
U_{cr} = critical average water velocity at incipient motion
ν = kinematic viscosity
w = terminal fall velocity

The term U_{cr}/w can be calculated as

$$U_{cr}/w = 2.5/(\log (U_* D/\nu)-0.06) + 0.66 \quad (26)$$

when

$$1.2 <(U_* D/\nu) <70 \quad (27)$$

and

$$U_{cr}/w = 2.05 \quad (28)$$

when

$$70 \leq (U_* D)/\nu \quad (29)$$

The sediment discharge is provided by the sediment transport relation even at the upstream and downstream cross sections, which in effect extrapolates conditions within the modeled reach to just above the first cross section and just below the last.

In the computer model, the sediment transport equations and hydrodynamic equations are solved sequentially rather than simultaneously during the linear approximation of the Newton-Raphson iteration, keeping one of the two sets of variables fixed during the solution of the linear system for the other. However, the Newton-Raphson loop is repeated within each time step until the full nonlinear set of equations, dependent on both sets of variables, is suitably approximated. Although such a scheme may not be quite as desirable as the simultaneous solution of the corresponding linear approximation for both sediment transport and hydrodynamic variables, implementation of such a high degree of coupling in the solution process would be difficult because of the complexity and variety of sediment transport equations. The momentum equation and continuity equations must balance at tributary junctions and in reaches with lateral inflows. Within each time step, the model sequentially solves each river (main river and its tributaries) until overall convergence of the water and sediment equations is obtained, subject to the restriction that continuity of the water-surface elevation at the junction of each tributary with the main river must be preserved.

In applying the computer model to this study, its code was edited to clarify the flow of logic. There were, in addition, some modifications that were necessary to make it applicable to this study. Examples of these modifications follow.

1. In the Yang sediment transport module, a calculation of concentration by weight was replaced with a calculation of sediment transport by volume, because that is how the concentration is used in the rest of the model.

2. In several places in the program, the possibility of negative (that is, upvalley) water-surface slopes needed special attention.

3. In the formation of the total (water plus sediment) continuity equation, the total cross-sectional area is needed, and this should not be reduced by the sediment deposition area. The active and inactive flow areas, and in particular the sediment deposition area, should be included in the total cross-sectional area because the equation specifies conservation of mass for the combined flow of water plus sediment.

4. In the momentum equation, the expression for the contribution due to lateral inflow should be $-q\ (V_{\ell} - U)$ instead of $-q\ V_{\ell}$, where q is the discharge of the lateral flow, V_{ℓ} the component of its velocity in the downstream direction, and U the average flow velocity in the river into which the lateral flow is taking place.

5. The initial estimates used in the Newton-Raphson scheme were prevented from resulting in spurious negative areas to avoid having the iterations stop, never to restart correctly, because of invalid numerical operations such as trying to find the logarithm of a negative number.

6. In the momentum equation, integral average values for the terms over a river element delta x and time element delta t are needed. In particular, for the friction slope term S_f, a (weighted) average of $S_f(Q,A)$ is desired in the four-point implicit formulation; because of the nonlinear way that discharge Q and cross-sectional area A enter the expression, the average of $S_f(Q,A)$ is not the same as $S_f(Q_{av}, A_{av})$, where Q_{av} and A_{av} are average values of Q and A (see equation 20). The expression $S_f(Q_{av}, Q_{av})$ that had appeared in the four-point evaluation of the friction slope was replaced by $(S_f(Q,A))_{av}$.

7. In the momentum equation, for the term due to sudden channel expansions or contractions, the reduced term

$$\partial/\partial x(Q^2/A) \qquad (30)$$

was replaced by

$$A\ \partial/\partial x\ ((Q/A)^2) \qquad (31)$$

averaged over the delta x-delta t interval.

8. The wind friction term was modified to

$$W_f = -C_w\ |V_w \cos\omega - Q/A\ |(V_w \cos\omega - Q/A) \qquad (32)$$

where C_W is a coefficient, ω the angle between the wind velocity vector and the downstream channel direction, Q the discharge, A the cross sectional area, and V_W the actual wind speed. This replaced the expression

$$W_f = C_w\ (V_w \cos\omega)^2 \qquad (33)$$

where V_W is, according to the documentation, the velocity of the wind relative to the velocity of the channel flow. What is needed for our study is that $V_W \cos\omega$ be relative to channel flow speed, and the requirement that it be such a relative velocity means that wind velocity cannot be specified independently of the (a priori) unknown water velocity. In addition, the way expression 33 is stated, the coefficient C_W must change sign depending on the direction of this relative velocity, and will be negative for the case of a downchannel relative velocity. The expression 32 corrects these difficulties.

9. Sediment deposition width was replaced in the program by a cross-sectional width calculated in a consistent manner, averaged between the current and forward time if necessary.

10. The update of the accumulated sediment deposition depth SDZ was relocated so that output would reflect the correct value.

MODELING OF COLUMBIA RIVER FLOODING

For this study, the Columbia River was taken to be the main river of the model, and the Willamette River a tributary. The modeled reach (fig. 1) extended from Bonneville Dam to Tongue Point near the mouth of the Columbia River. Upstream boundary conditions consisted of input discharge hydrographs to the modeled reaches of the Columbia and Willamette Rivers. The input discharge at Bonneville Dam was taken to be 200,000 ft^3/s, and an average discharge of 33,000 ft^3/s as input to the Willamette; these were assumed constant during the period modeled. The input discharge to Bonneville included average flow of 194,000 ft^3/s measured on the Columbia River at The Dalles, combined with an average 1,000 ft^3/s measured on the Hood River and an average 5,000 ft^3/s measured on the Lewis River. The Lewis River is actually downstream of both Bonneville Dam and the Willamette River, but its flow was added at Bonneville as a modeling simplification. These input discharges thus provided a combined average flow of 233,000 ft^3/s between the Willamette and Cowlitz Rivers. Total discharge (water plus sediment) and sediment discharge hydrographs, representing the flooding of the breach, specified the lateral inflow to the Columbia River from the Cowlitz River (figs 2 and 3). As a modeling simplification, the small tributary inputs to the Columbia River below the Cowlitz River were neglected; any effect from these flows at the mouth of the Cowlitz would be a great deal smaller than tidal influence. Downstream boundary conditions were tidal water-surface elevations, in time, from NOAA tide tables for Tongue Point (fig. 4). All elevations in this report are with respect to the National Geodetic Vertical Datum of 1929 (NGVD).

Manning's "n" values ranged from 0.0170 to 0.0410, based on a calibration of the DWOPER model to historical flood elevations done by the Corps of Engineers, Portland District (1983) during a study of flood elevations that would be produced by a failure of Bonneville Dam during a concurrent Columbia River flood. Fifty-one degrees Fahrenheit was used as water temperature, based on water temperature data at Vancouver, Wash. The computational time step delta t was 3 minutes for most of the run. However, the time step was decreased to 36 seconds during most of the time between 1.5 and 2.5 days after the breach. It was reduced still further, to 7.2 seconds, between 54 and 56 hours.

The results of the modeling using a Columbia River average flow of 233,000 ft^3/s are shown in the plots of figures 5 to 37. Figures 5 through 19 show a time sequence of longitudinal profiles along the Columbia River from Tongue Point to Bonneville Dam, as the sediment blockage is formed and the waters of the Columbia River subsequently are impounded behind it. Sediment deposition would take place during the flood at the blockage, with crest at river mile 66.1. Subsequent flow then would carry sediment from the blockage to downstream of the crest. The deposition depth downstream of the crest, and corresponding scour upstream, are averages over the channel width at the location. This width is much larger upstream of the blockage than below, and for this reason the scour upstream is barely perceptible on the plots; the corresponding sediment volumes match as required by continuity.

Figures 20 through 37 show predicted hydrographs at specific locations along the Columbia River. At Columbia River mile 53.4, 12.7 miles downstream of the crest of the blockage, the effect would be a reduction in total discharge as the Columbia River is impounded behind the blockage (figs. 20, 21, 22, and 23). This would be followed by a gradual return to 233,000 ft^3/s. Sediment deposition would take place gradually after the flood, as material is transported to this location from upstream.

At river mile 66.1, the effect during the flood from the Cowlitz River would be a rapid deposition of the sediment to produce the crest of the blockage of the Columbia River (figs. 24, 25, 26, and 27).

At river mile 69.1, 3.1 miles upstream of the crest, the discharge on the Columbia River would be reversed during the flood by the sediment blockage (figs. 28, 29, 30 and 31). That is, the flood from the Cowlitz River would be diverted upstream. The sediment deposition of the blockage would take place through this location, and a large amount of channel filling would occur during the flood. The water-surface elevation would continue to increase gradually after the flood as the Columbia River was impounded behind the blockage.

The sediment deposition of the blockage would not extend upstream as far as river mile 75.1, 9 miles upstream of the crest. The effect at that location would be one of gradual filling by the Columbia River in the impoundment area behind the blockage (figs. 32, 33, 34 and 35).

At the Trojan Power Plant, 5 miles upstream of the Cowlitz River, the water-surface elevation would reach a maximum elevation of 32 feet. Due to the large channel storage volume of the Columbia River between the crest of the blockage and Bonneville Dam, the river levels would continue to rise for 16 days after the blockage is formed. Water-surface elevation at the same location is 6 feet at an average Columbia River discharge of 233,000 ft^3/s without the blockage.

At the Interstate 5 highway bridge at Portland, Oreg., located at river mile 106.5, the effect again would be one of gradual water-surface rise due to filling behind the blockage (figs. 36 and 37). The water-surface elevation would rise to 32 feet at 16 days, compared to 10 feet without the blockage.

Water-surface elevations would generally be higher than levees (U.S. Army Corps of Engineers, 1978) from the Cowlitz River to just above the Willamette River (table 3). Flooding would occur in low-lying areas along the Columbia and Willamette Rivers and in the area around Vancouver Lake and Sauvie Island. Upstream of river mile 103.1, low areas would in general be protected by levees if it is assumed that they would not fail at the anticipated water-surface elevation of 32 feet. However, the Corps of Engineers gives a safe elevation for levees near the interstate bridge at river mile 106.5 as about 18 feet, even though levee crests are about 35 feet, and a safe elevation for levees near river mile 114.7, just upstream of Portland International Airport, as 33 feet, even though levee crests are about 41 feet. Downstream from the mouth of the Cowlitz River, flooding would be prevented by the blockage, even during peak flow from the Cowlitz into the Columbia River. Thus, maximum water-surface elevations at river mile 53.4, which is 12.6 miles downstream of the interstate bridge at Longview, Wash., were actually modeled some 14 days after the breach, due purely to a high tide at that time.

Water-surface elevations that would occur at higher Columbia River flows are also shown in table 3. In these cases, the input discharge to the Columbia River at Bonneville Dam was adjusted upward to produce the indicated flood discharge when combined with a flow of 35,000 ft^3/s from the Willamette River; otherwise, the input to the computer model was identical to the 233,000-ft^3/s average flow case.

All input discharges to the Columbia and Willamette Rivers were considered constant during the period modeled. Modeling results for these higher flows were similar to the average flow case, except that the waters from the concurrent Columbia River flood would stack to higher water-surface elevations behind the blockage, and would cause more extensive flooding as additional levees were overtopped. Thus, during a concurrent Columbia River flood of 410,000 ft^3/s, the levees near the Portland Interstate 5 bridge at river mile 106.5 would be overtopped, flooding, for example, Portland International Airport. For a concurrent flood of 610,000 ft^3/s, all the levees between the blockage at river mile 66.1 and Bonneville Dam at river mile 145.5 would be overtopped. (Again, safe elevations for the levees are in general less than crest elevations.) The sediment blockage as deposited during these higher flows would be similar to that of the average flow case. The crest would be almost as high (table 4). The time to the maximum crest elevation would be reduced for higher flows, because the impoundment behind the blockage would fill more quickly, to restore the Columbia River discharges and sediment transport and scour associated with these higher flows.

CONCLUSION

The results of using the one-dimensional sediment transport model in this study indicate that an outburst of Spirit Lake would cause a sediment deposit blockage of the Columbia River at the mouth of the Cowlitz River. This would result in impoundment of the river's flow behind the blockage, and would cause flood-level water-surface elevations upstream. There would be little, if any adverse effect downstream of the blockage. The predicted water-surface elevations are, to be sure, subject to considerable uncertainty because of uncertainty regarding the total volume of sediment which would first of all be entrained in such a breach of the avalanche debris, and which then actually would reach and be deposited in the Columbia River.

TABLE 3.--Predicted maximum water-surface elevations

[x, Columbia River mile; y_c, levee crest, in feet; y_s, safe water-surface elevation for levee; t, time in days after breach; y, water-surface elevation, in feet with respect to National Geodetic Vertical Datum (NGVD); y_0, water-surface elevation, in feet, at the same Columbia River discharge, but without the blockage; Columbia River discharges below the Willamette River are in cubic feet per second, shown together with recurrence interval.]

				Columbia River discharge														
				233,000 (average)			410,000 (2-year)			610,000 (10-year)			750,000 (50-year)			820,000 (100-year)		
x	Approximate location	y_c	y_s	t	y	y_0	t	y	y_0	t	y	y_0	t	y	y_0	t	y	y_0
17.50	Tongue Point, Oreg.			14	7	-1	14	7	-1	14	7	-1	14	7	-1	14	7	-1
23.36	Svensen, Oreg.			14	7	0	14	7	0	14	7	0	14	7	0	14	7	1
30.15	Three Tree Pt., Wash.			14	7	1	14	7	1	14	7	2	14	8	3	14	8	3
34.63	Skamokawa, Wash.	12	8	13	8	1	14	8	2	14	8	4	15	8	4	15	9	5
41.60	Wauna, Oreg.	12	8	14	8	2	14	8	3	14	9	6	14	10	7	14	10	8
53.40	Oak Point, Wash.	17	11	14	9	4	14	10	6	14	12	10	15	14	12	28	15	13
66.10	Longview Bridge	* 23/29	18/18	28	28	5	28	34	9	28	38	14	28	40	17	28	41	19
69.06	Cowlitz River			16	32	6	14	38	10	13	41	15	11	43	18	11	44	20
72.50	Trojan, Oreg.			16	32	6	14	38	11	12	41	16	10	43	19	10	44	21
75.05	Kalama, Wash.			16	32	7	14	38	12	12	41	17	10	43	20	9	44	22
84.00	Columbia City, Oreg.	28	25	16	32	8	14	38	13	12	42	19	10	44	22	9	45	23
92.50	Ridgefield, Wash.	30	23	16	32	9	14	38	14	12	42	20	10	44	23	9	45	25
100.00	Vancouver, Wash.	* 33/30	29/16	16	32	9	14	39	16	12	42	21	11	44	24	9	45	25
103.10	Willamette River	27	16	16	32	10	14	39	16	12	42	21	10	44	24	10	45	26
106.50	Portland I-5 Bridge	35	18	16	32	10	14	39	16	12	42	22	10	45	25	10	45	26
114.70	Portland Airport	41	33	16	32	11	14	39	18	12	43	25	10	45	28	9	46	30
122.90	Washoughal, Wash.	42	36	16	33	14	14	39	22	12	44	28	10	46	32	10	47	33
131.95	Bridal Veil, Oreg.			16	33	16	14	40	25	12	44	31	10	47	34	10	48	36
141.00	Warrendale, Oreg.			16	33	17	14	40	27	14	45	34	11	48	38	10	50	40
143.25	N. Bonneville, Wash.	41	36	16	33	18	14	41	28	12	47	37	11	50	43	9	52	44
145.50	Bonneville Dam			16	34	20	14	41	29	12	48	38	10	52	45	10	53	47

* left bank/right bank

TABLE 4.--Crest of sediment blockage

[At Columbia River Mile 66.1, or 0.1 mile upstream of the interstate bridge at Longview, Washington. The pre-blockage channel elevation is -47 feet. Elevations are with respect to National Geodetic Vertical Datum.]

Columbia River discharge (ft^3/s)	Recurrence interval (years)	Time at highest crest elevation (days after breach)	Crest elevation (feet)
233,000	Average	28	-3
410,000	2	5	-3
610,000	10	12	-3
750,000	50	6	-4
820,000	100	5	-5

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Figure 2
Cowlitz River at Mouth
Total Discharge Input to Columbia River Model

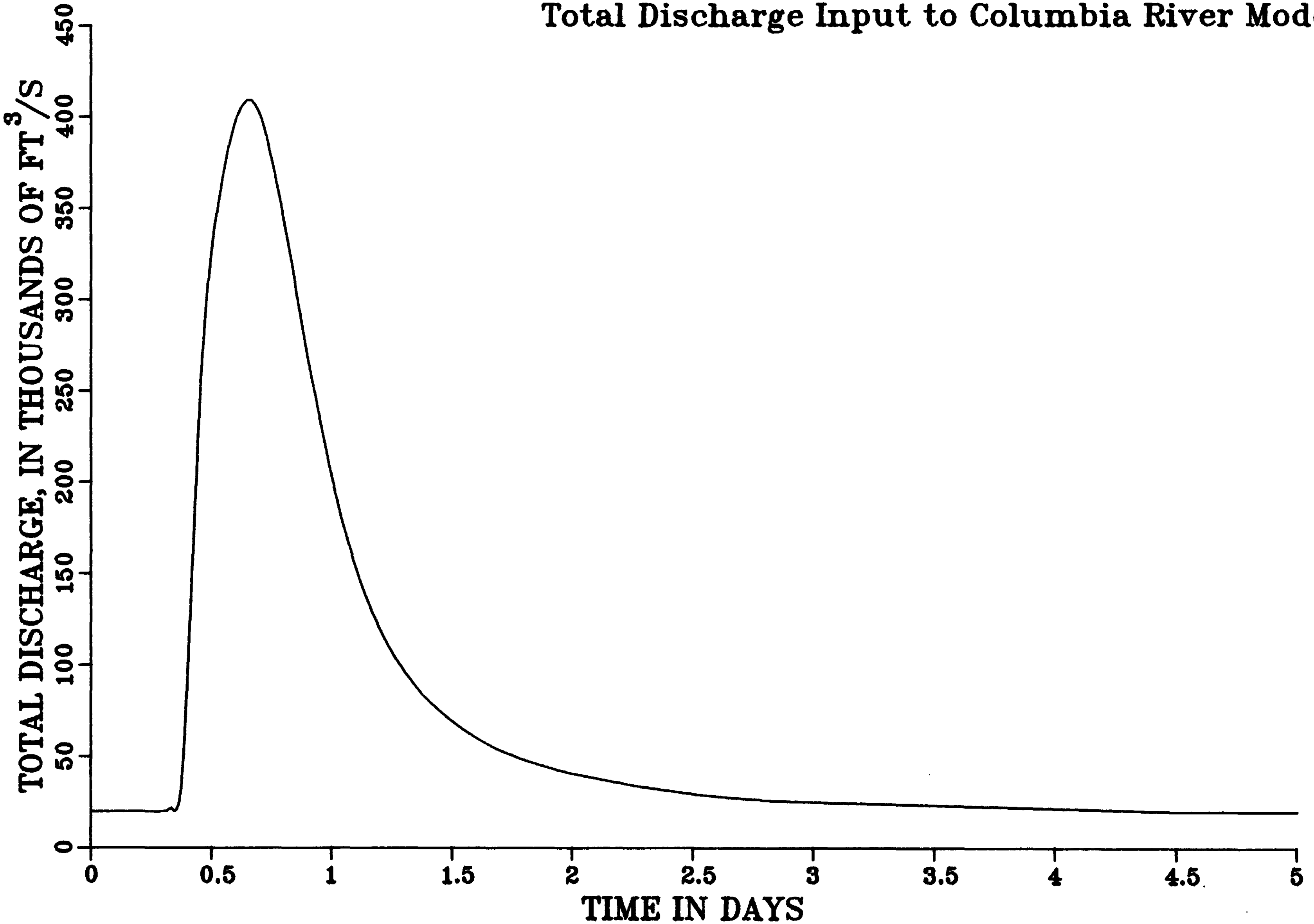


Figure 3
Cowlitz River at Mouth
Sediment Discharge Input to Columbia River Model

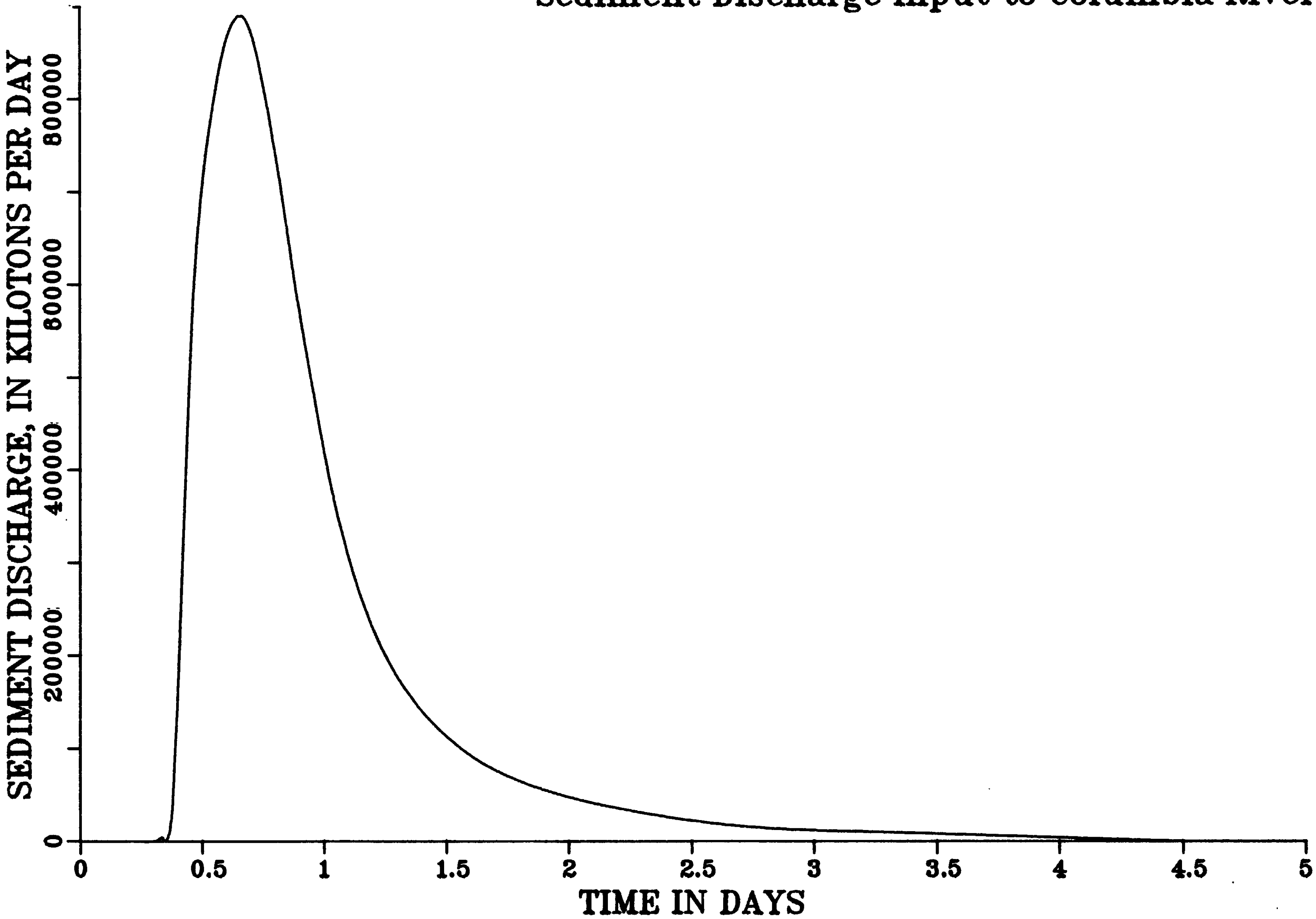


Figure 4
Columbia River at Mile 17.5
Tidal Water Surface Elevation
Tongue Point, Oregon

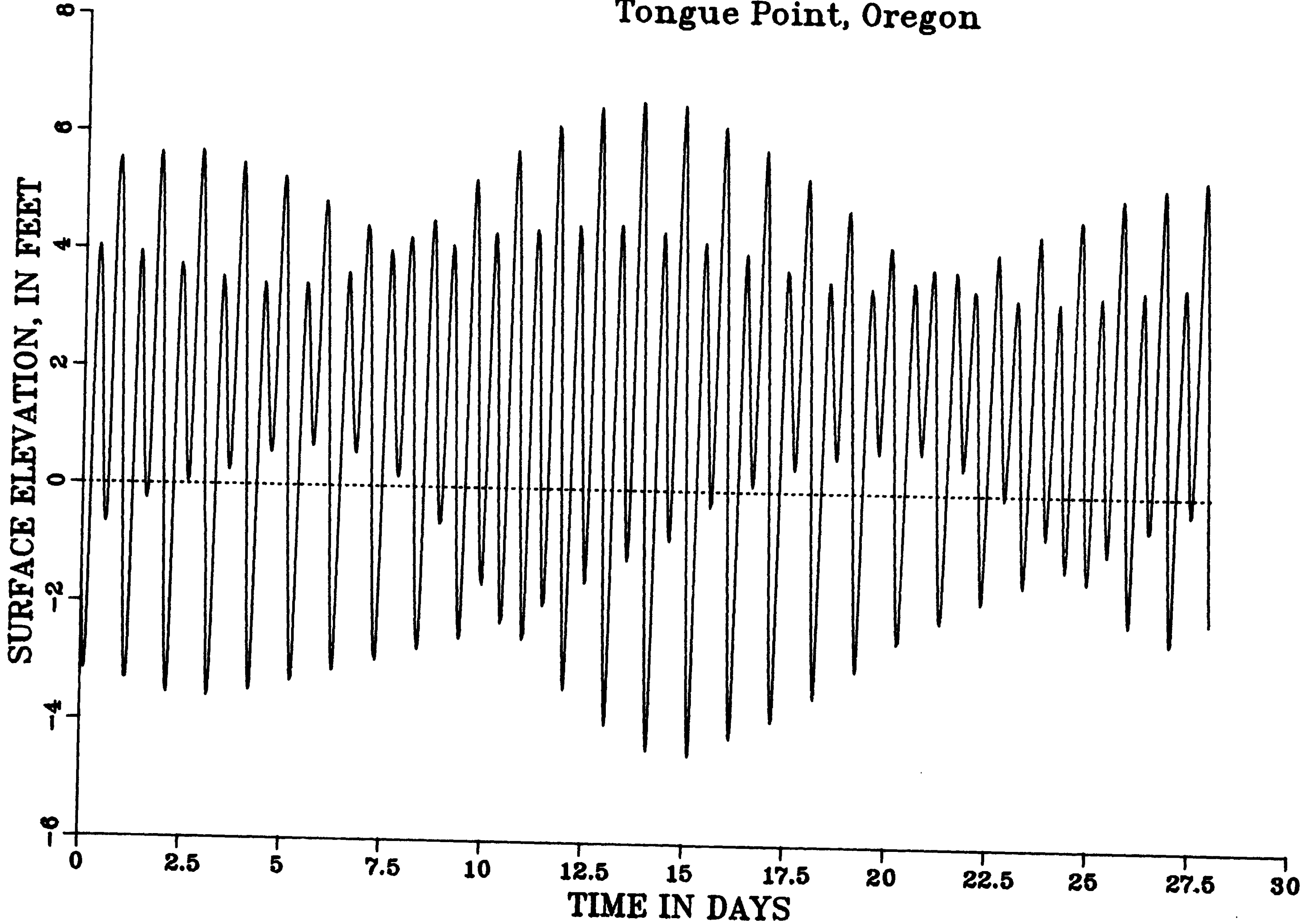
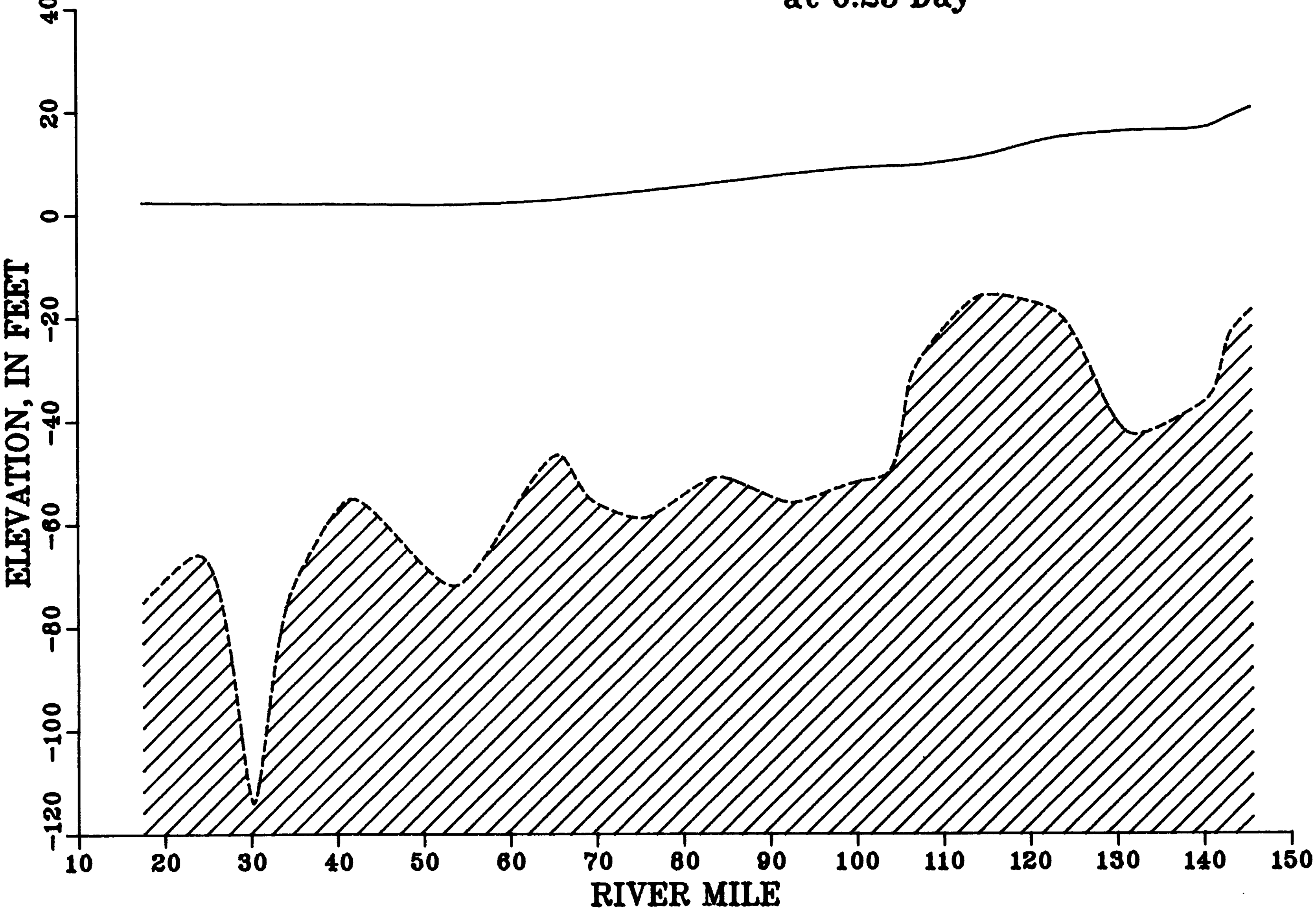


Figure 5
Columbia River
Predicted Water Surface and Bed Profiles
at 0.25 Day



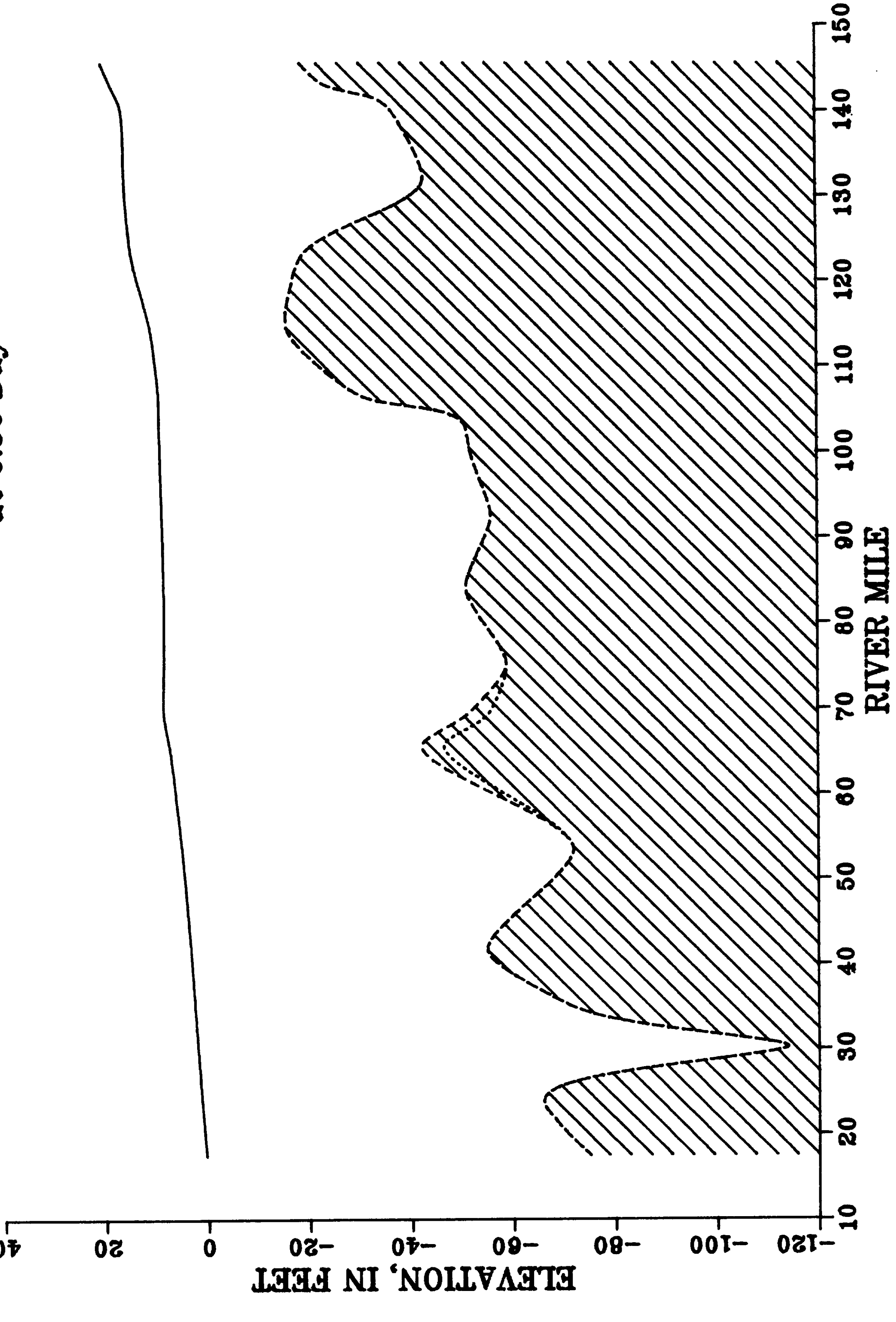


Figure 6
Columbia River
Predicted Water Surface and Bed Profiles
at 0.50 Day

Figure 7
Columbia River
Predicted Water Surface and Bed Profiles
at 0.75 Day

Figure 8
Columbia River
Predicted Water Surface and Bed Profiles
at 1 Day

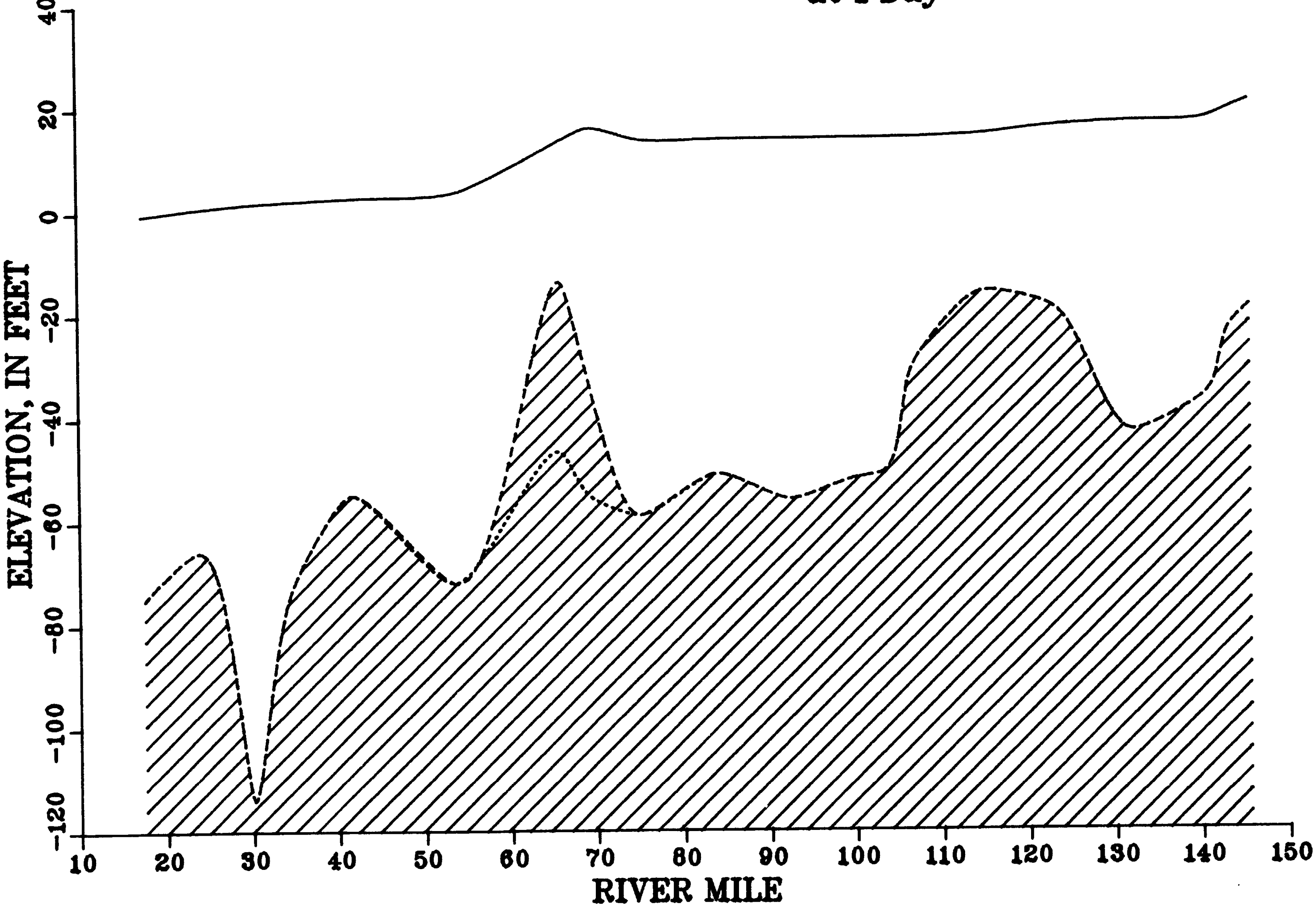


Figure 9
Columbia River
Predicted Water Surface and Bed Profiles
at 1.25 Days

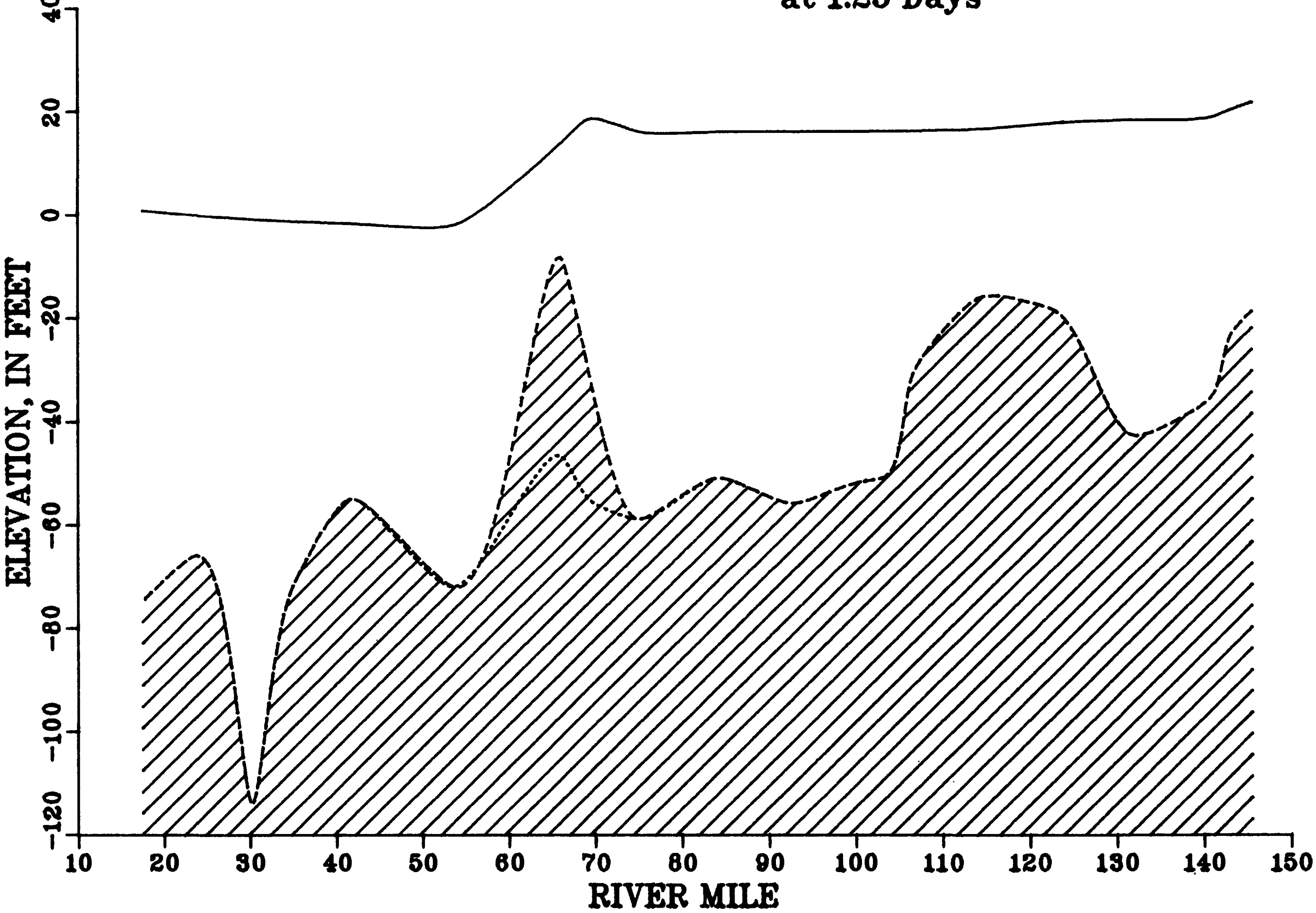


Figure 10
Columbia River
Predicted Water Surface and Bed Profiles
at 1.50 Days

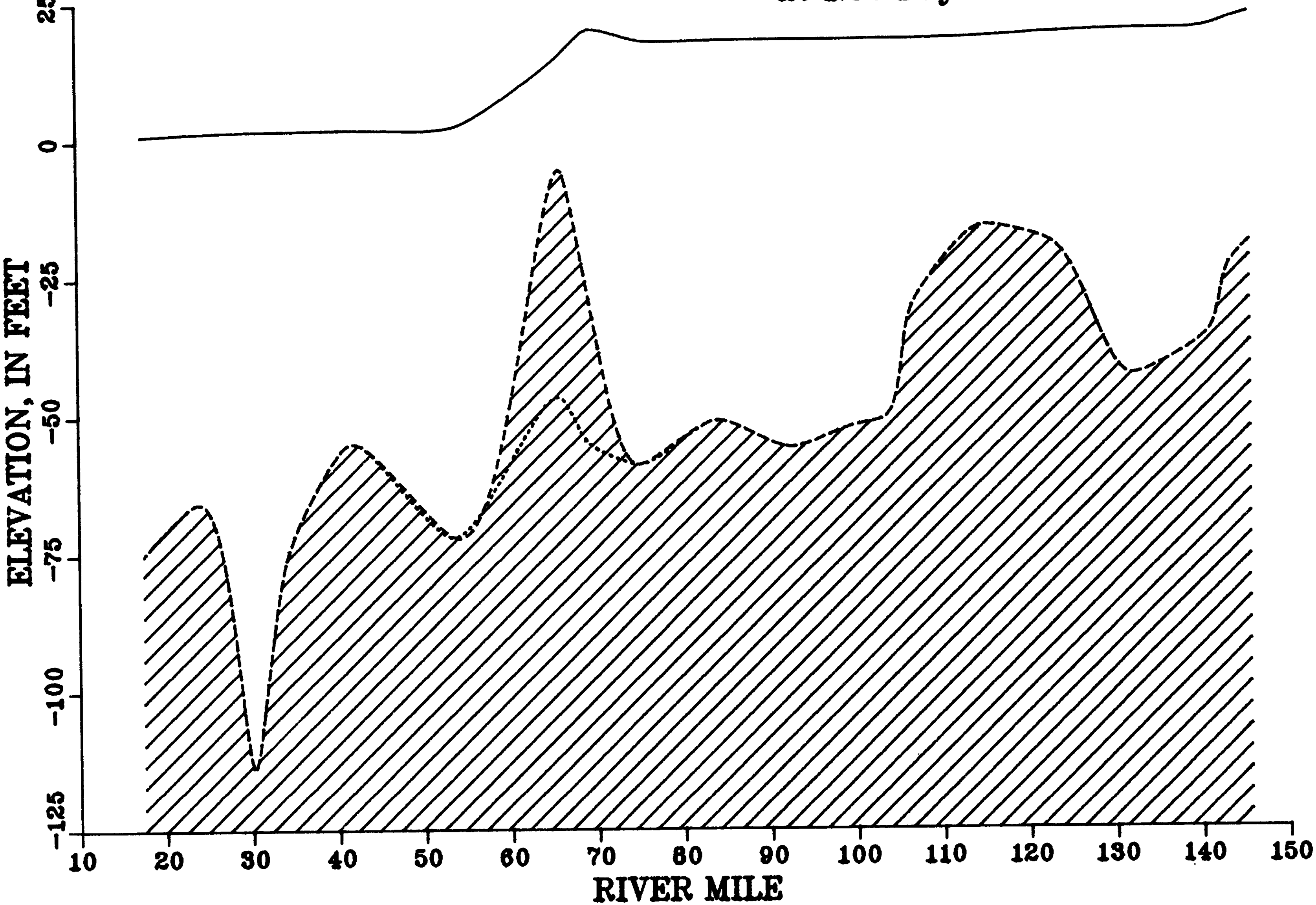


Figure 11
Columbia River
Predicted Water Surface and Bed Profiles
at 1.75 Days

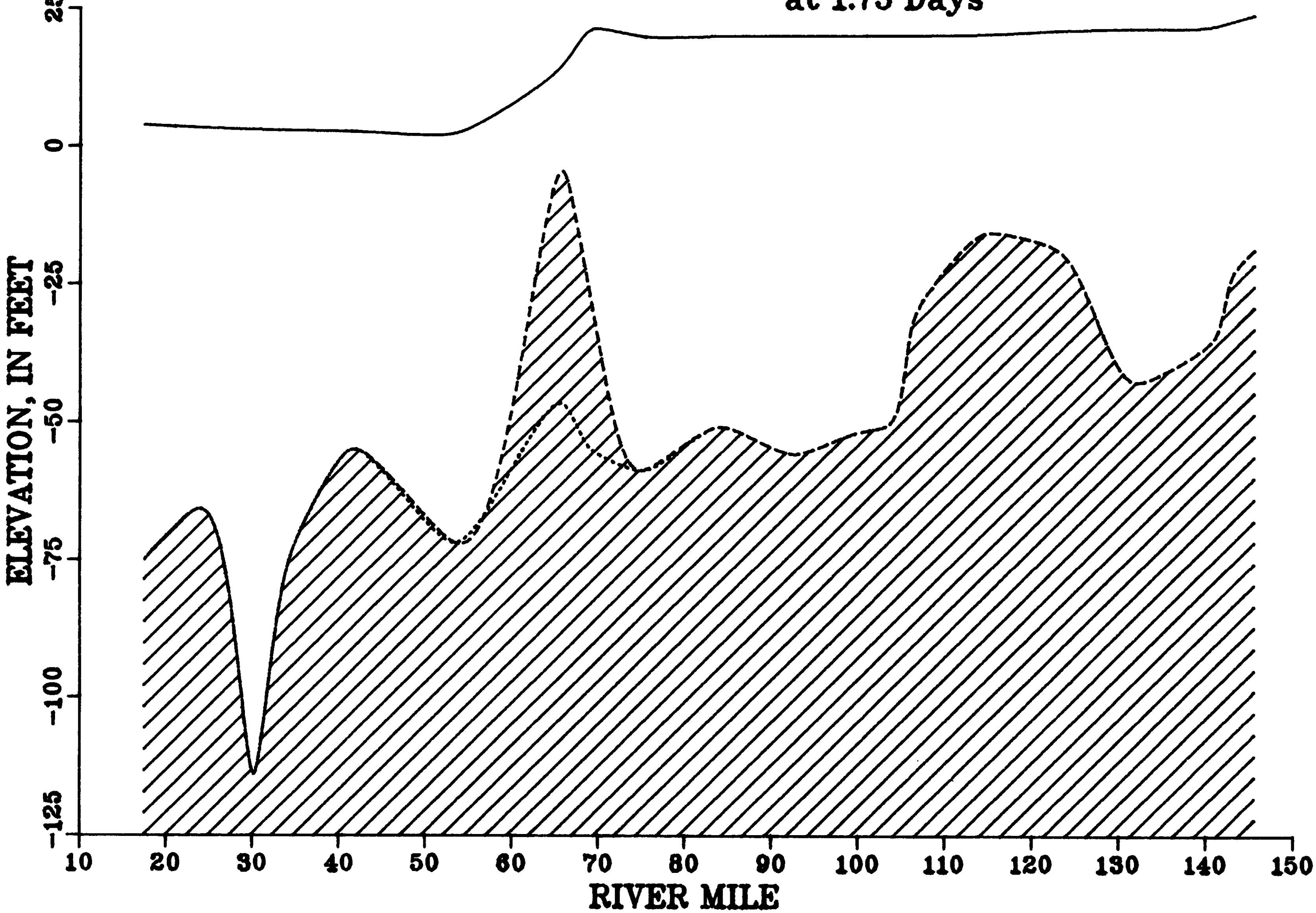


Figure 12
Columbia River
Predicted Water Surface and Bed Profiles
at 2 Days

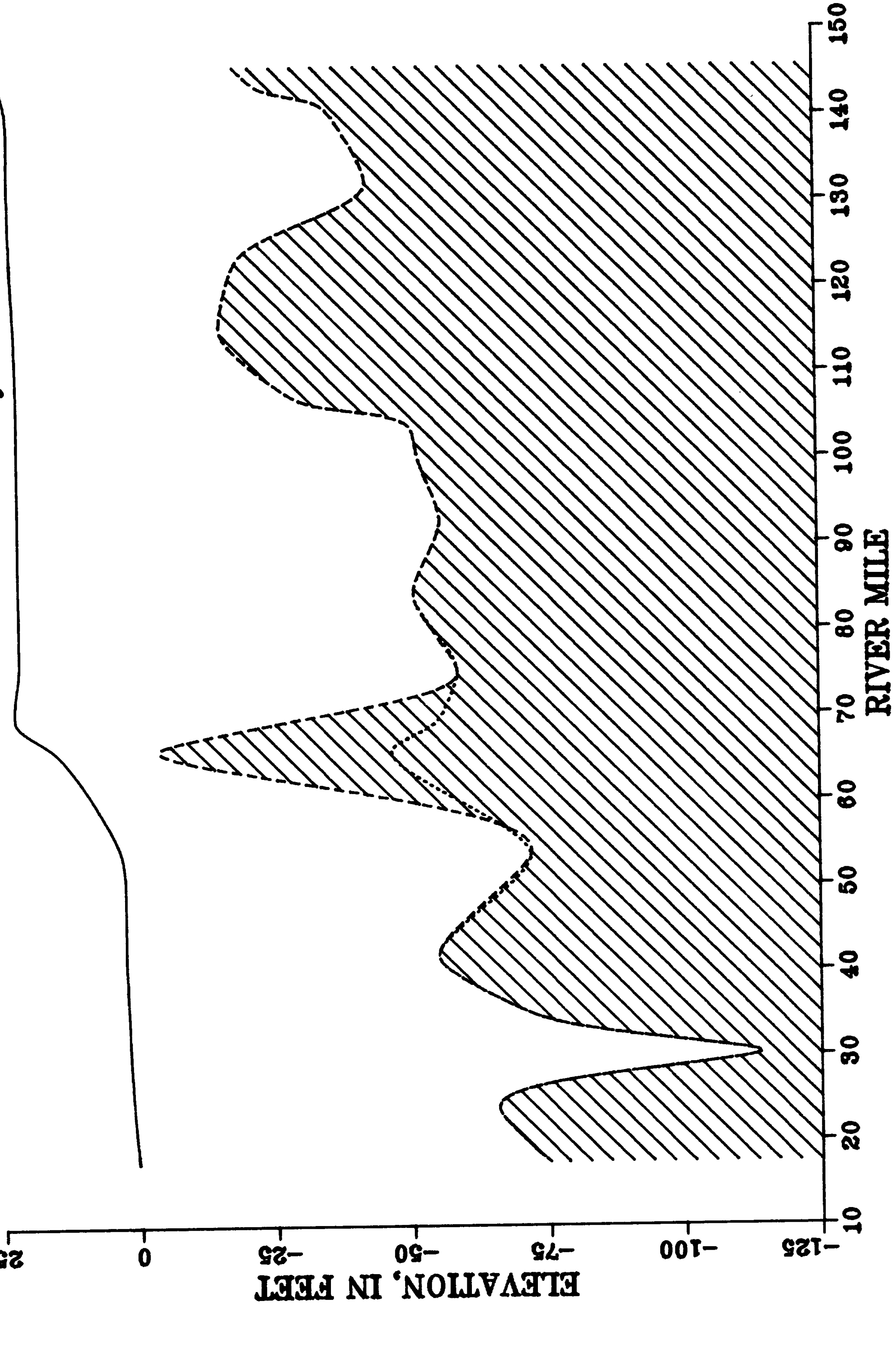


Figure 13
Columbia River
Predicted Water Surface and Bed Profiles
at 3 Days

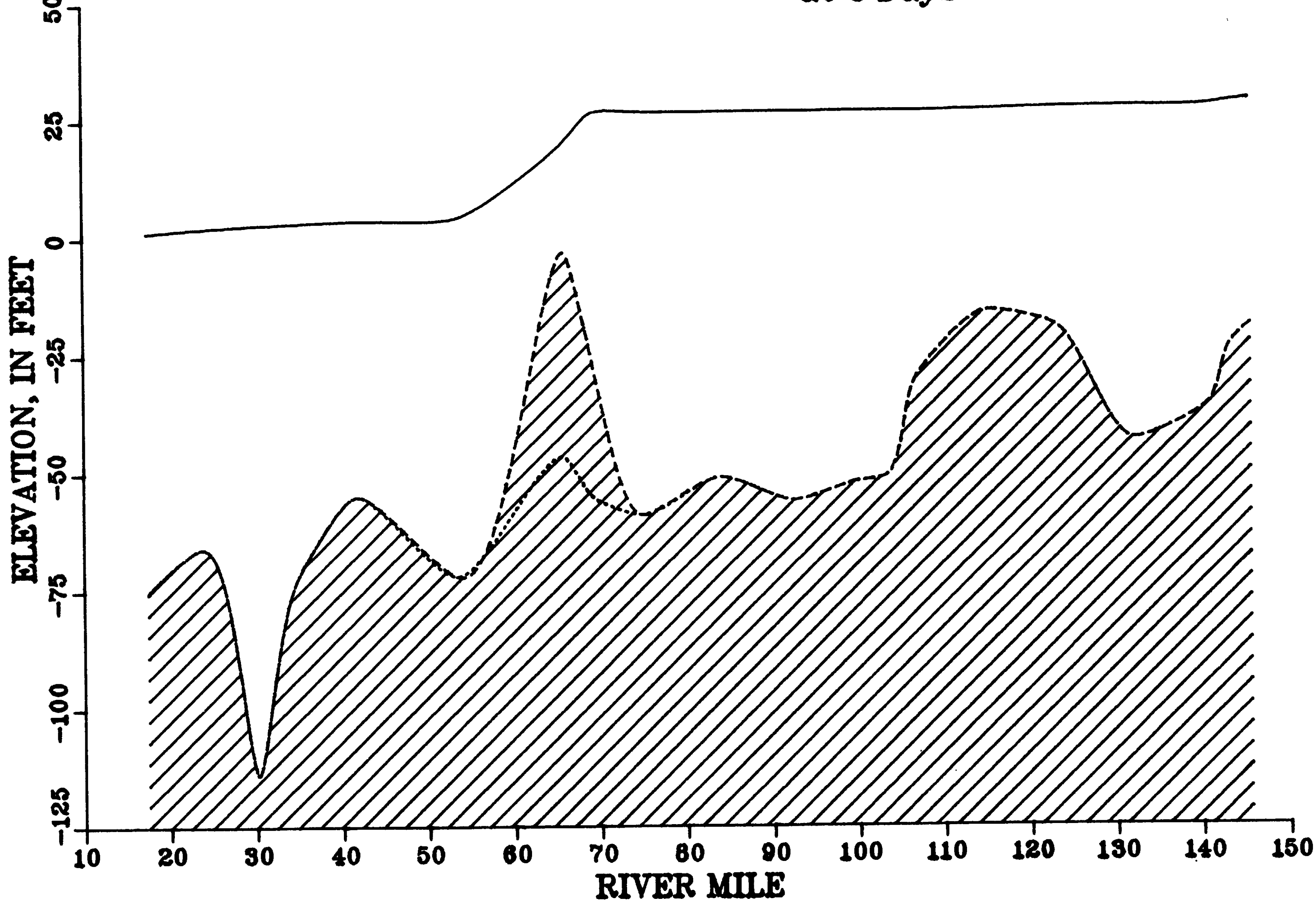


Figure 14
Columbia River
Predicted Water Surface and Bed Profiles
at 4 Days

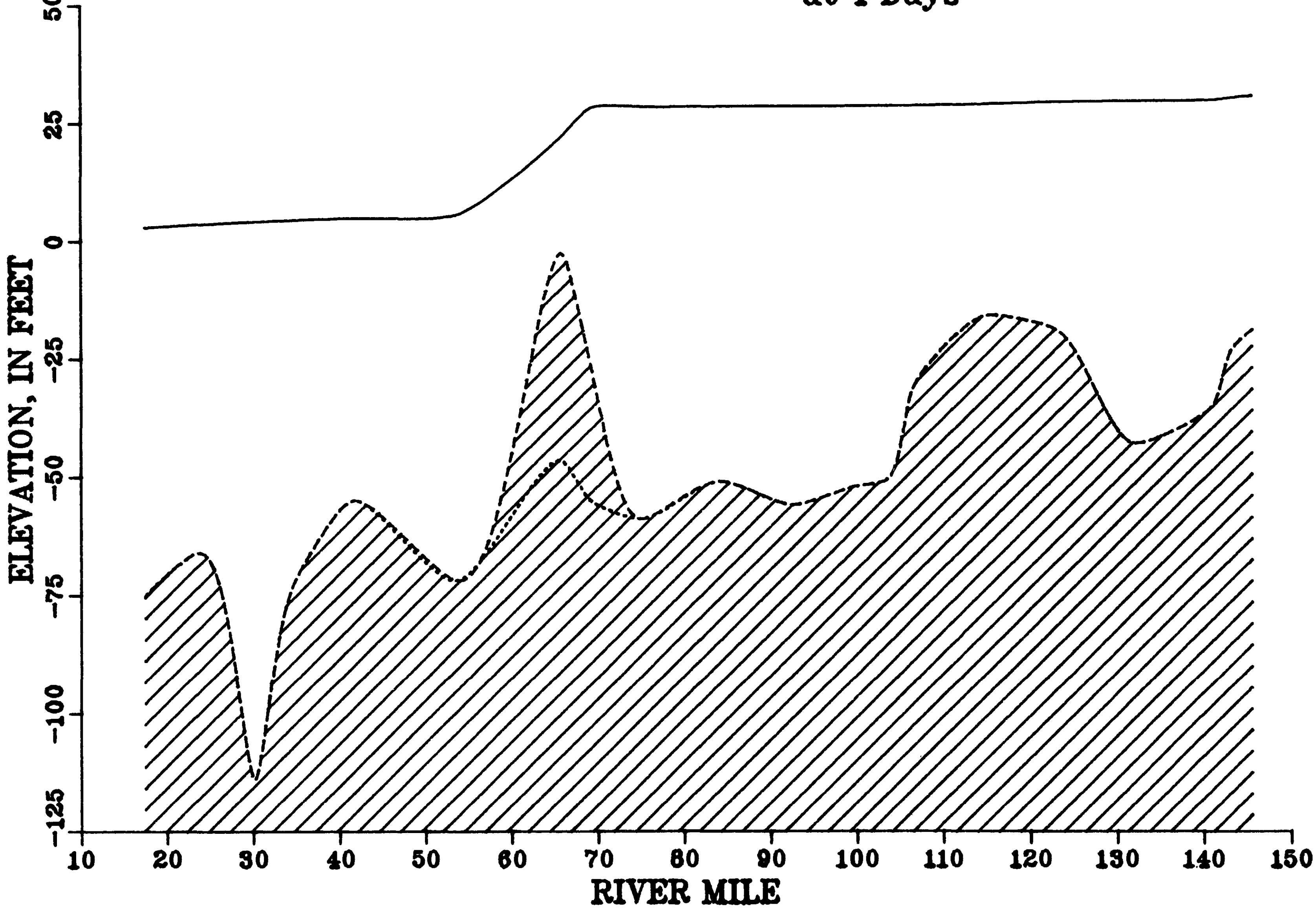


Figure 15
Columbia River
Predicted Water Surface and Bed Profiles
at 5 Days

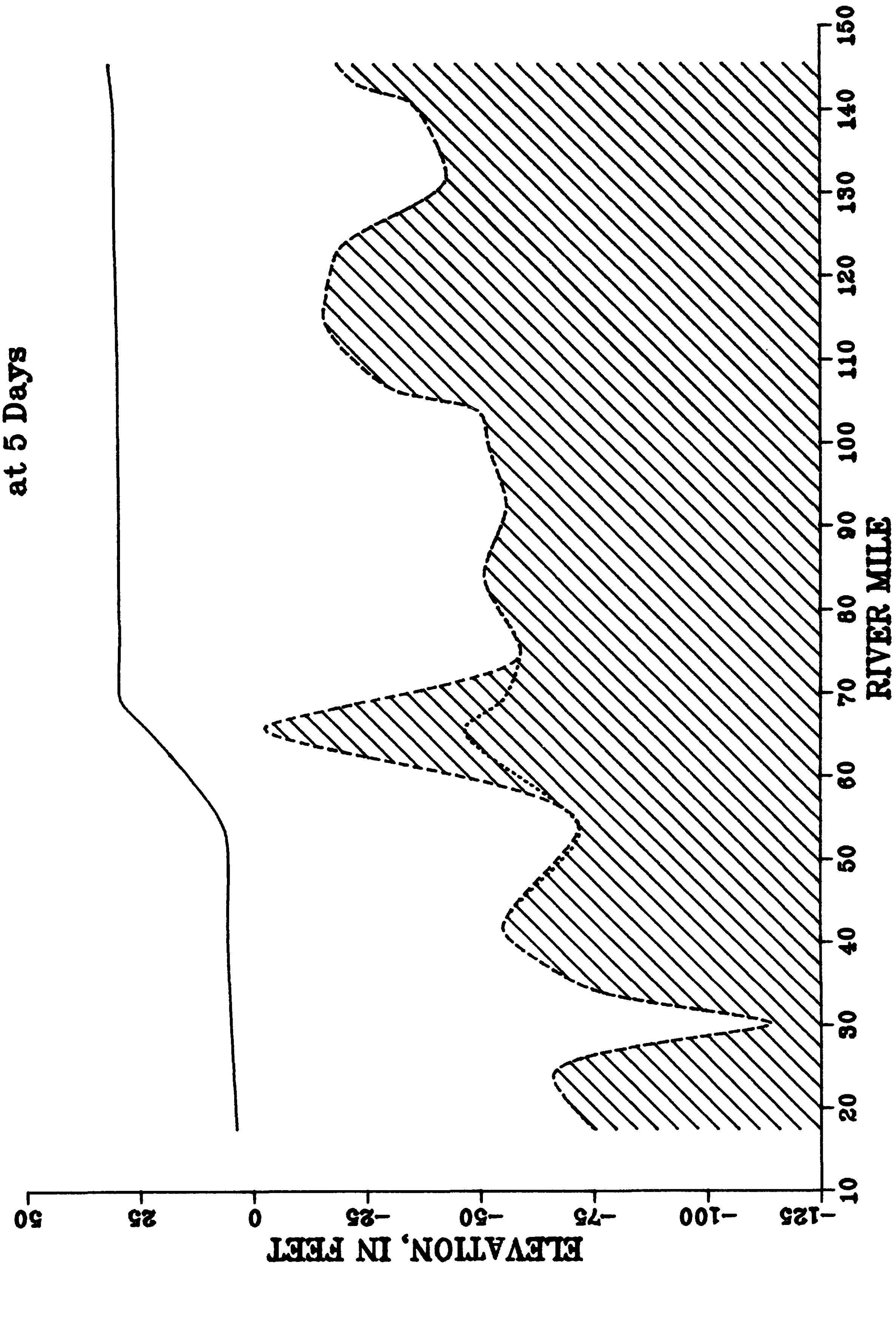


Figure 16
Columbia River
Predicted Water Surface and Bed Profiles
at 7 Days

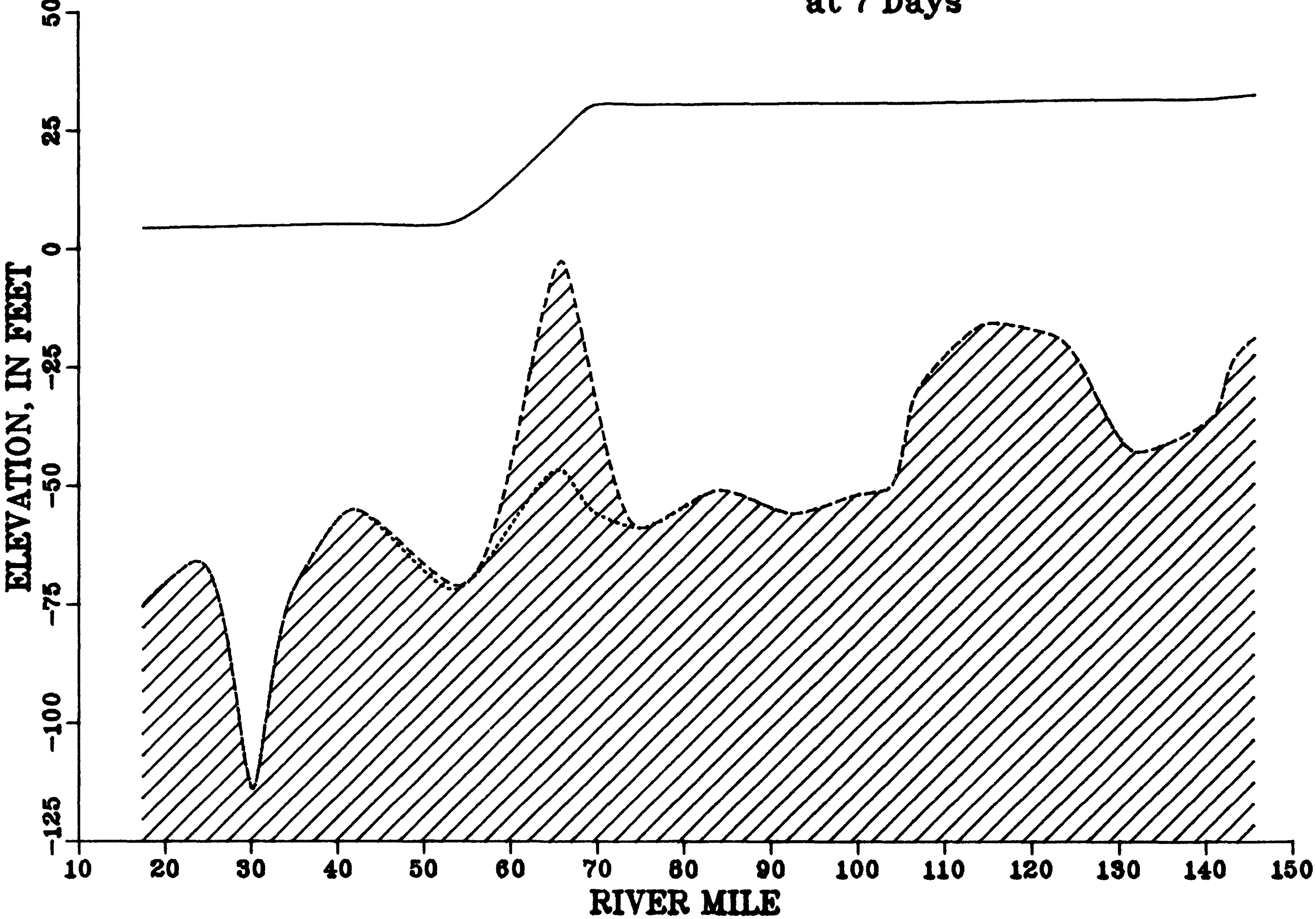


Figure 17
Columbia River
Predicted Water Surface and Bed Profiles
at 14 Days

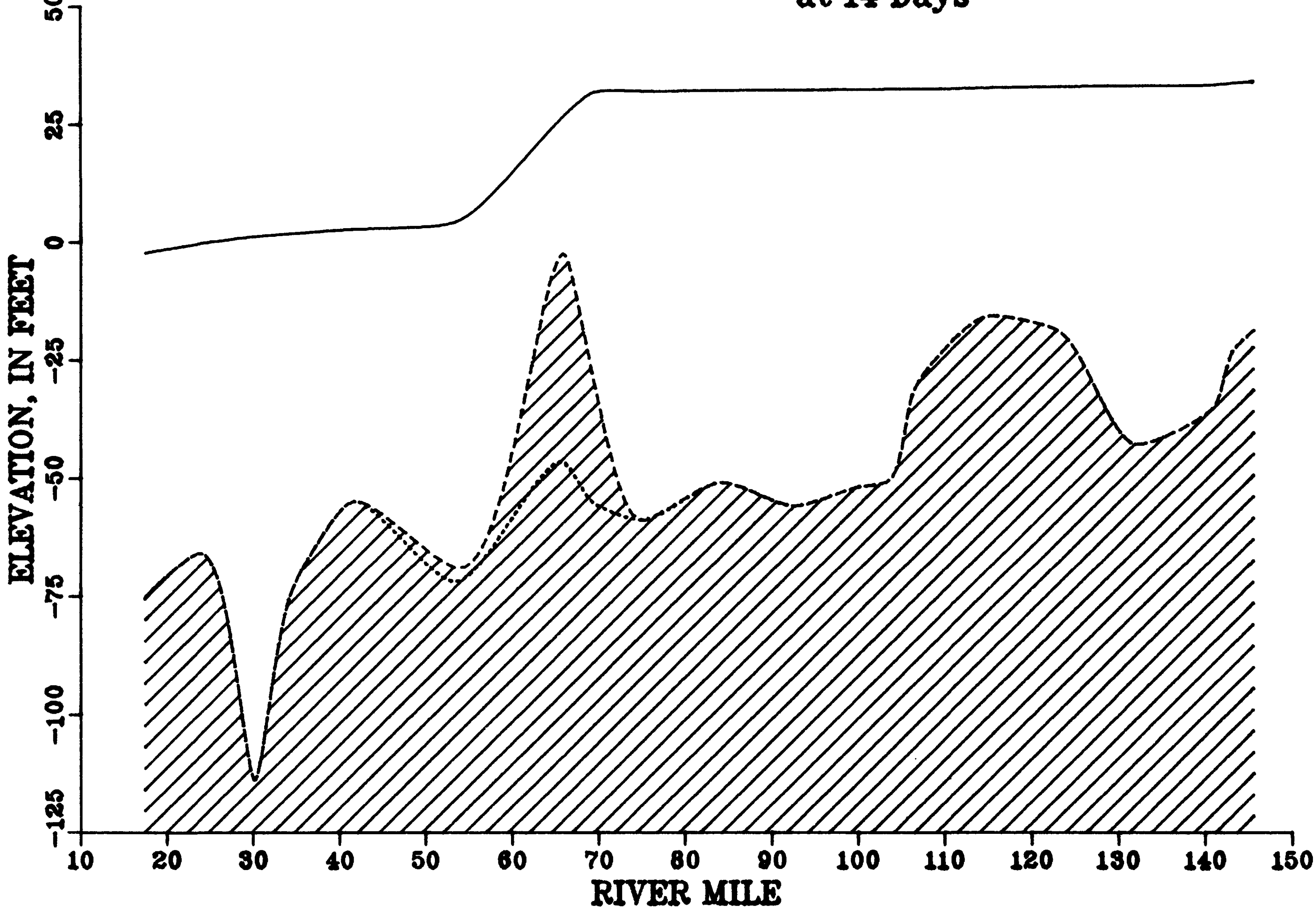


Figure 18
Columbia River
Predicted Water Surface and Bed Profiles
at 21 Days

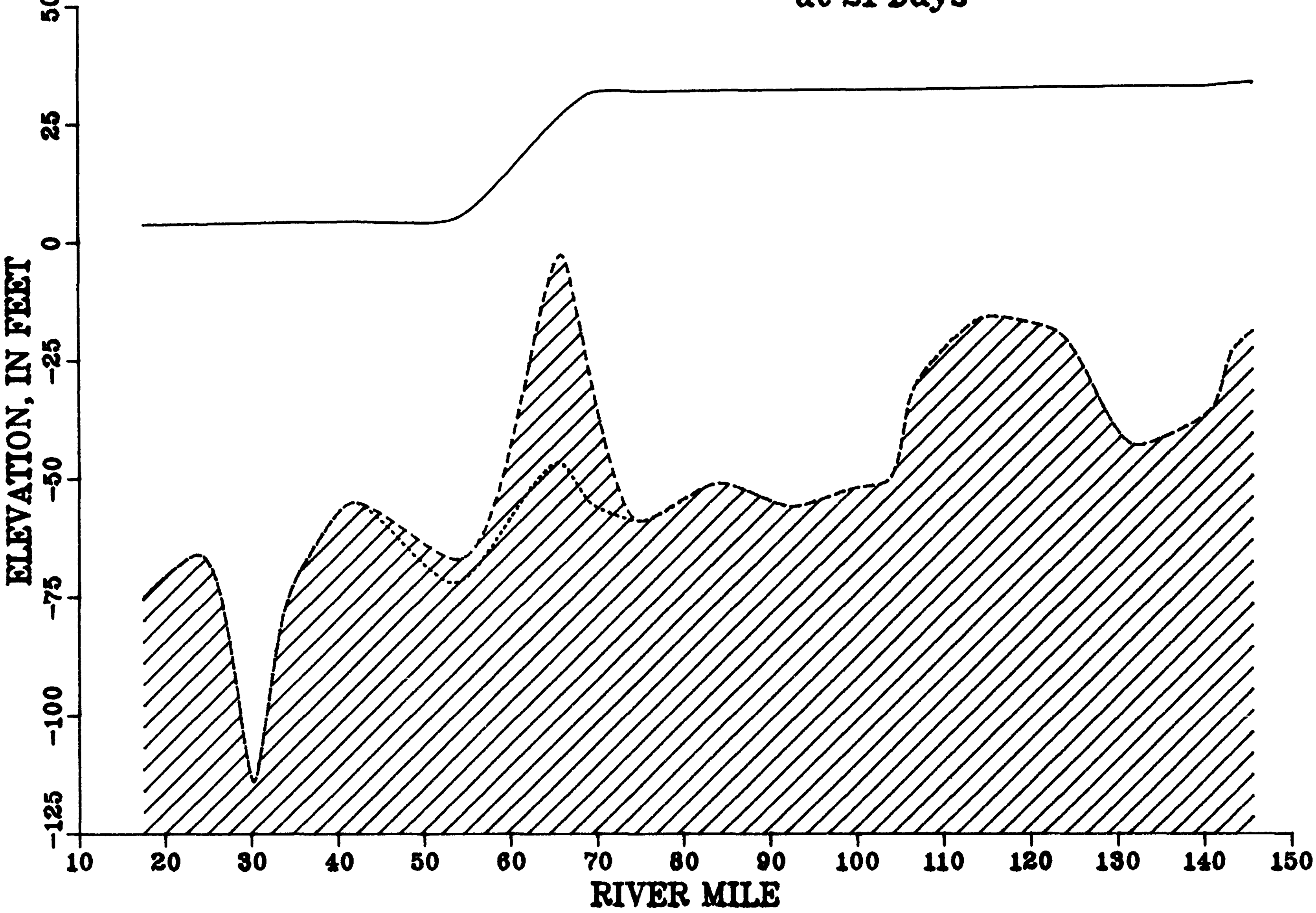


Figure 19
Columbia River
Predicted Water Surface and Bed Profiles
at 28 Days

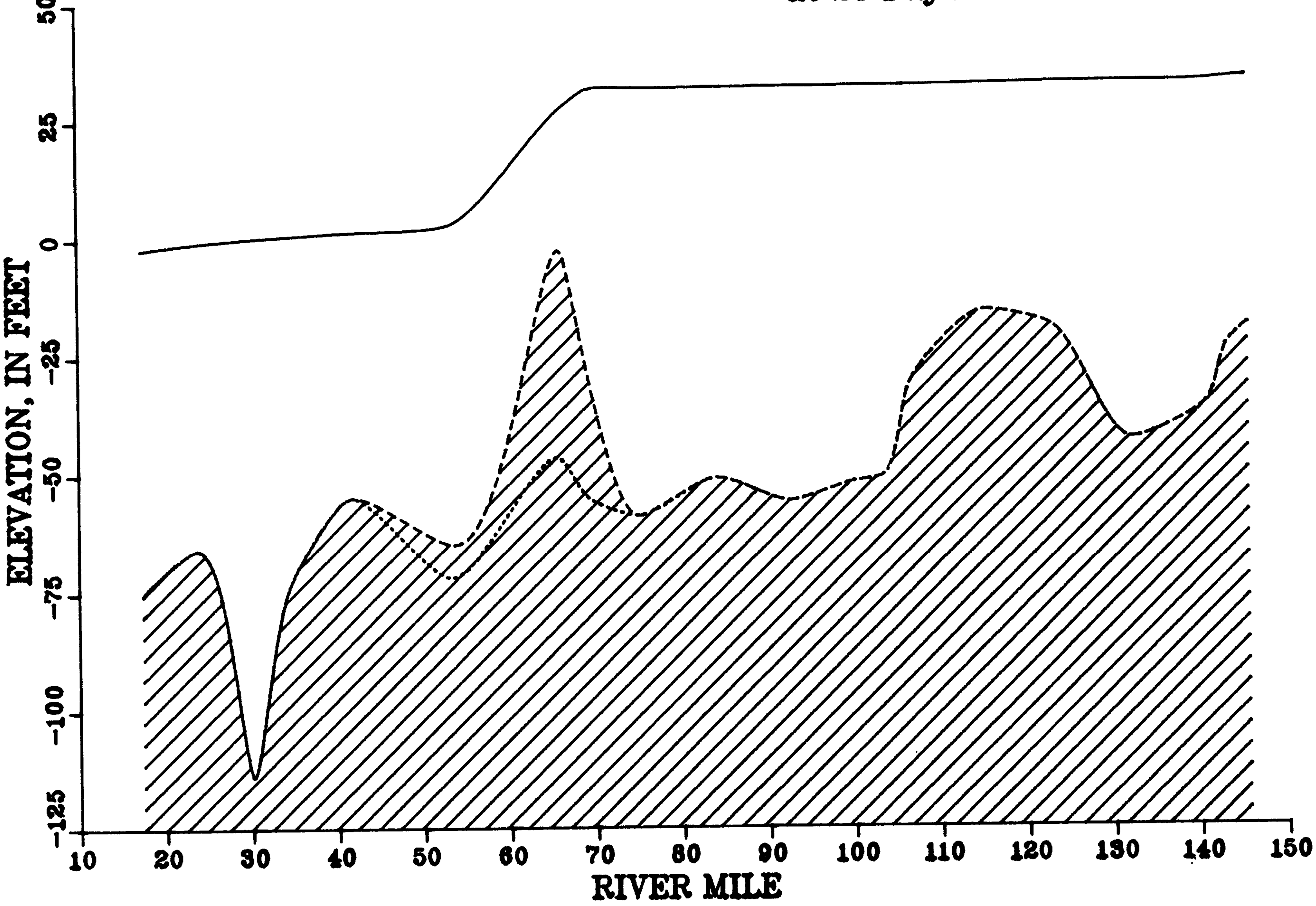


Figure 20
Columbia River at Mile 53.4
Predicted Water Surface Elevation
12.6 Miles Downstream of Interstate Bridge
at Longview, Washington

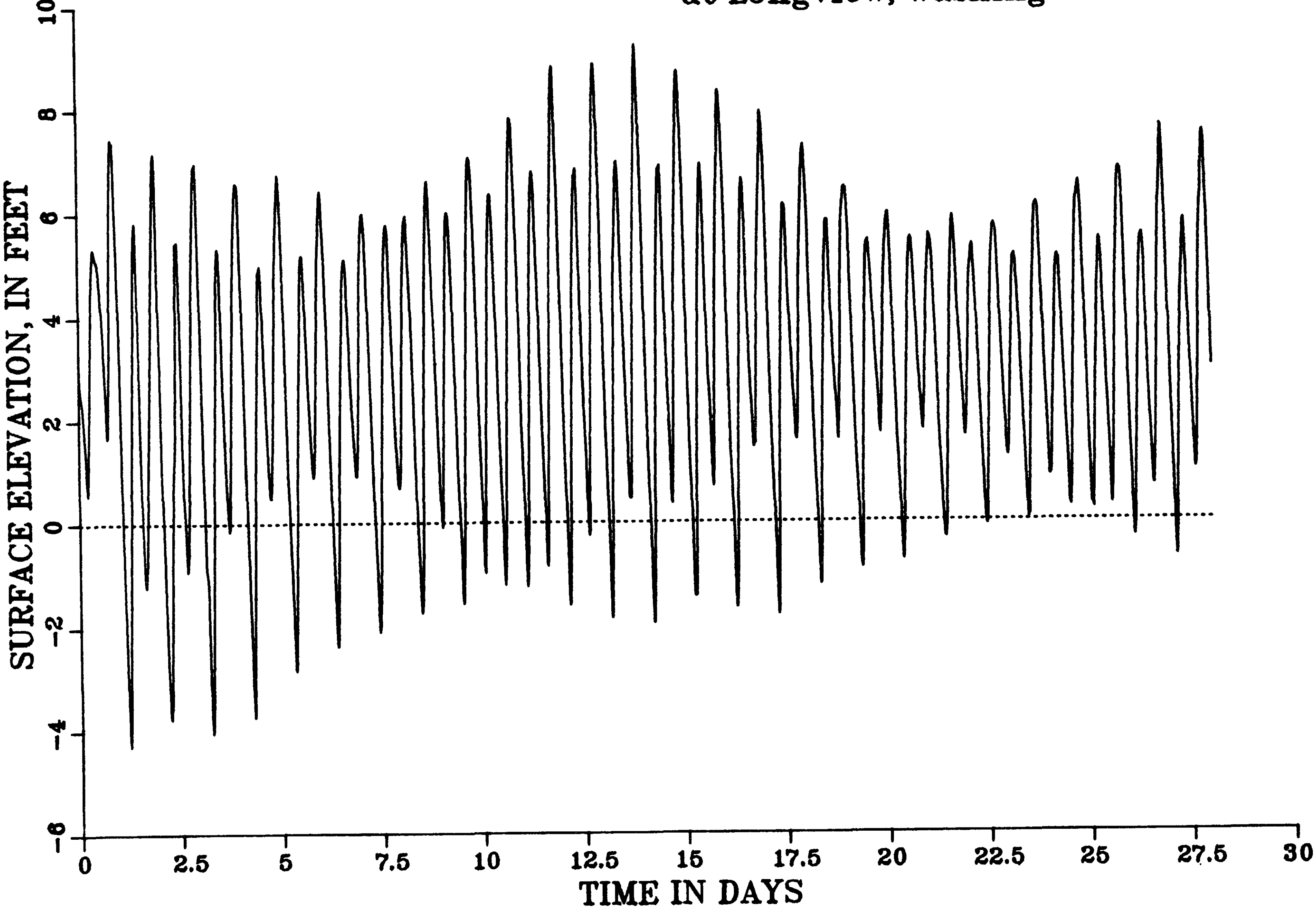


Figure 21
Columbia River at Mile 53.4
Predicted Total Discharge
12.6 Miles Downstream of Interstate Bridge
at Longview, Washington

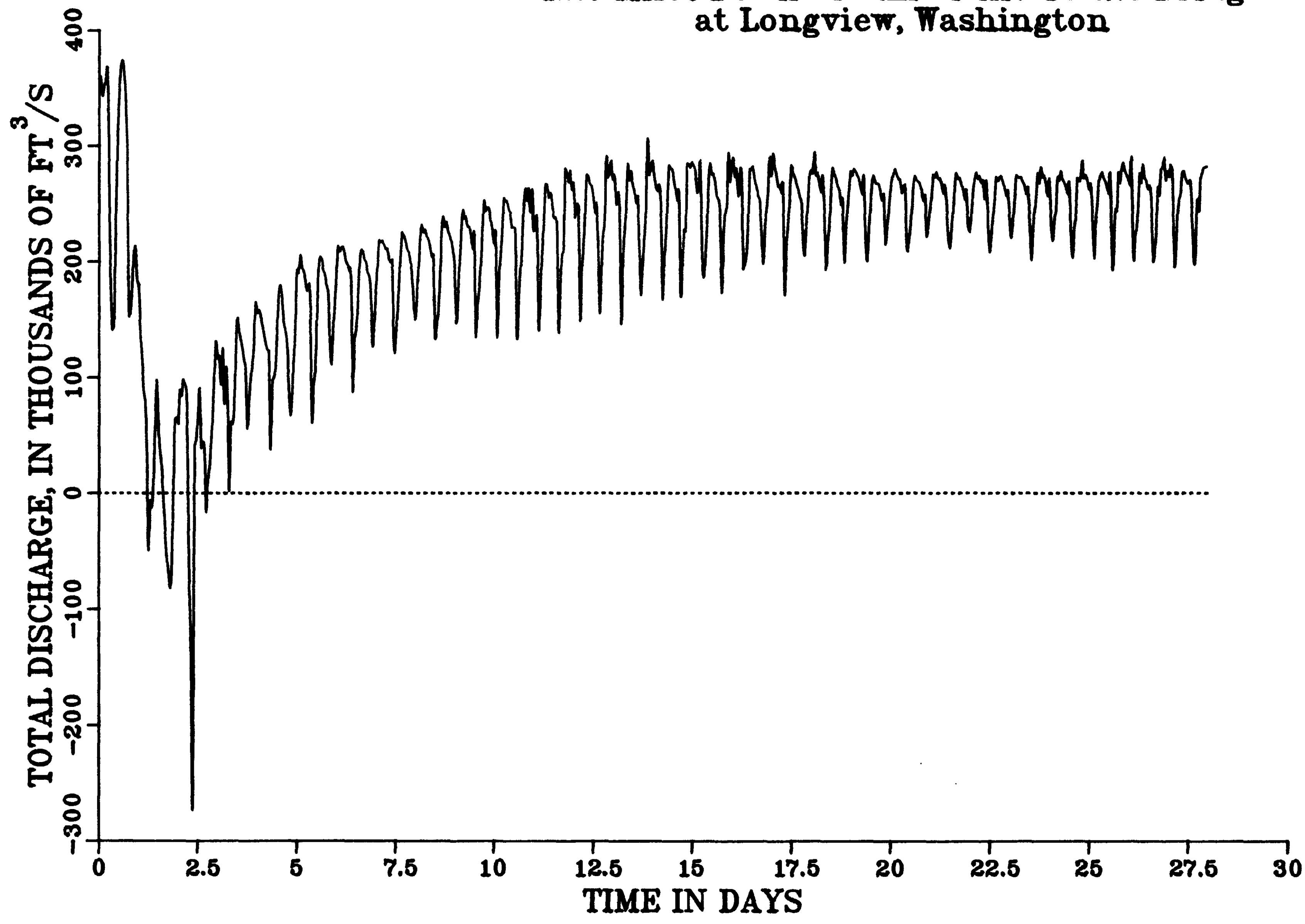


Figure 22
Columbia River at Mile 53.4
Predicated Bed Elevation
12.6 Miles Downstream of Interstate Bridge
at Longview, Washington

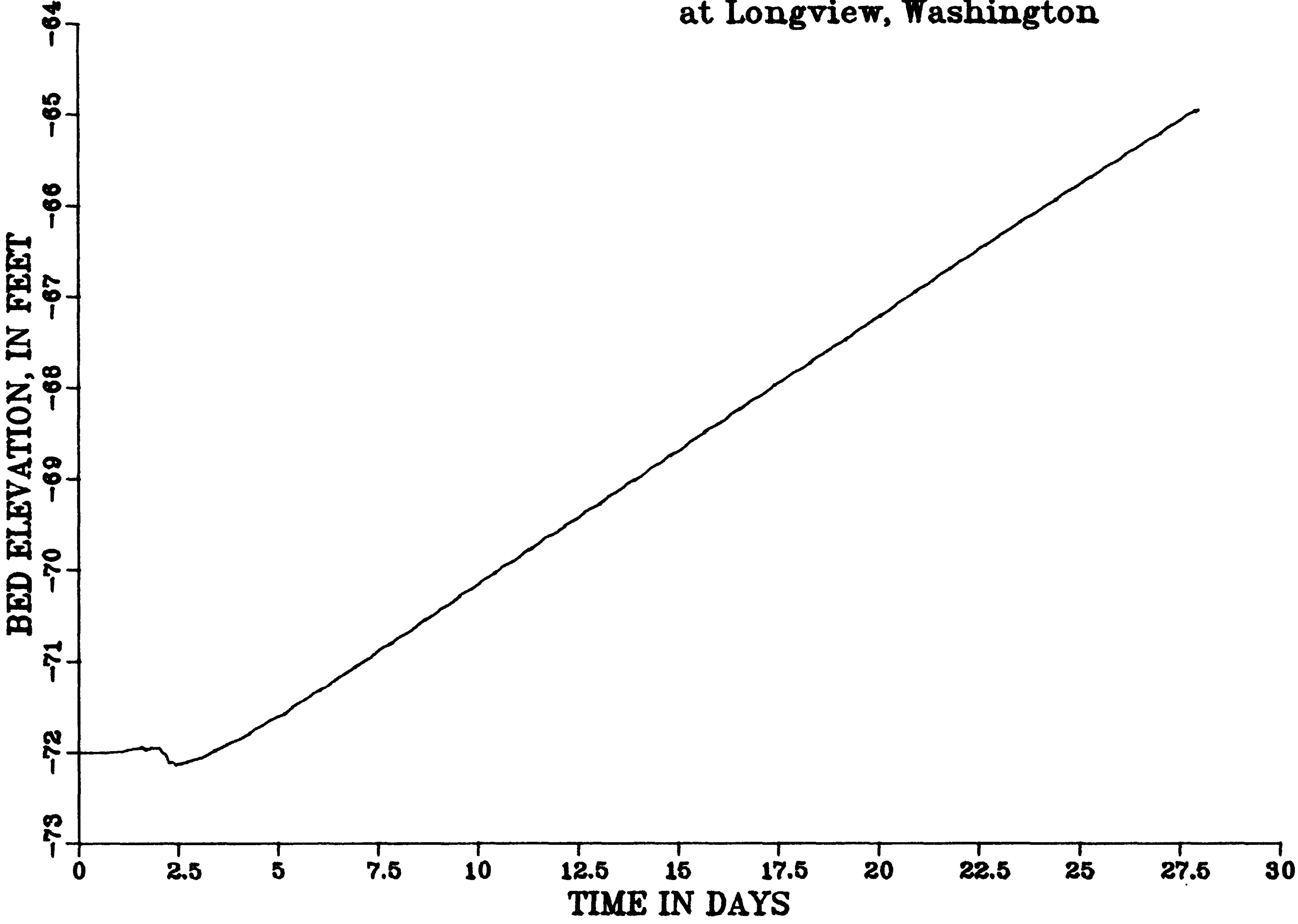


Figure 23
Columbia River at Mile 53.4
Predicted Sediment Discharge
12.6 Miles Downstream of Interstate Bridge
at Longview, Washington

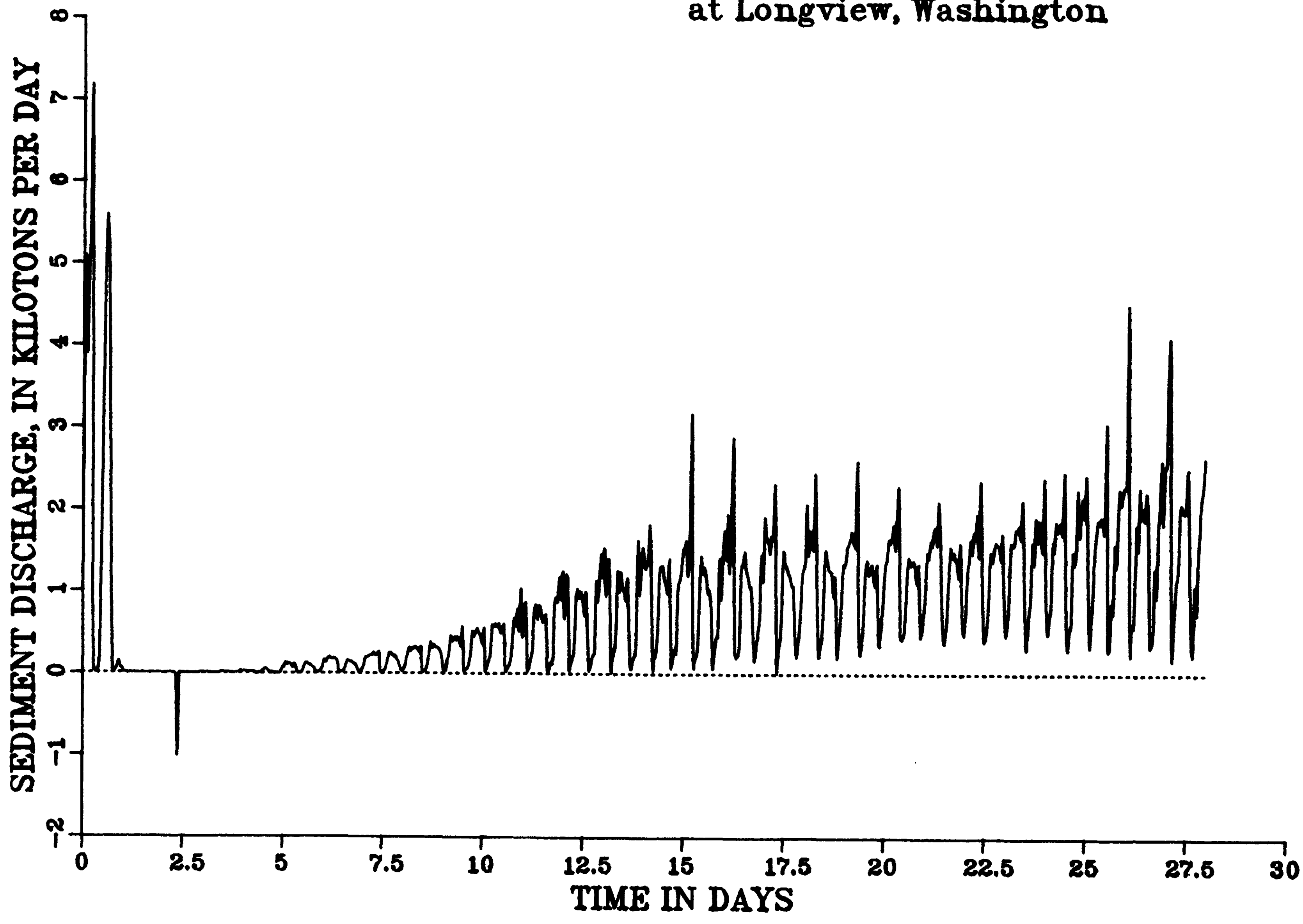


Figure 24
Columbia River at Mile 66.1
Predicted Water Surface Elevation
Longview, Washington
0.1 Mile Upstream of Interstate Bridge

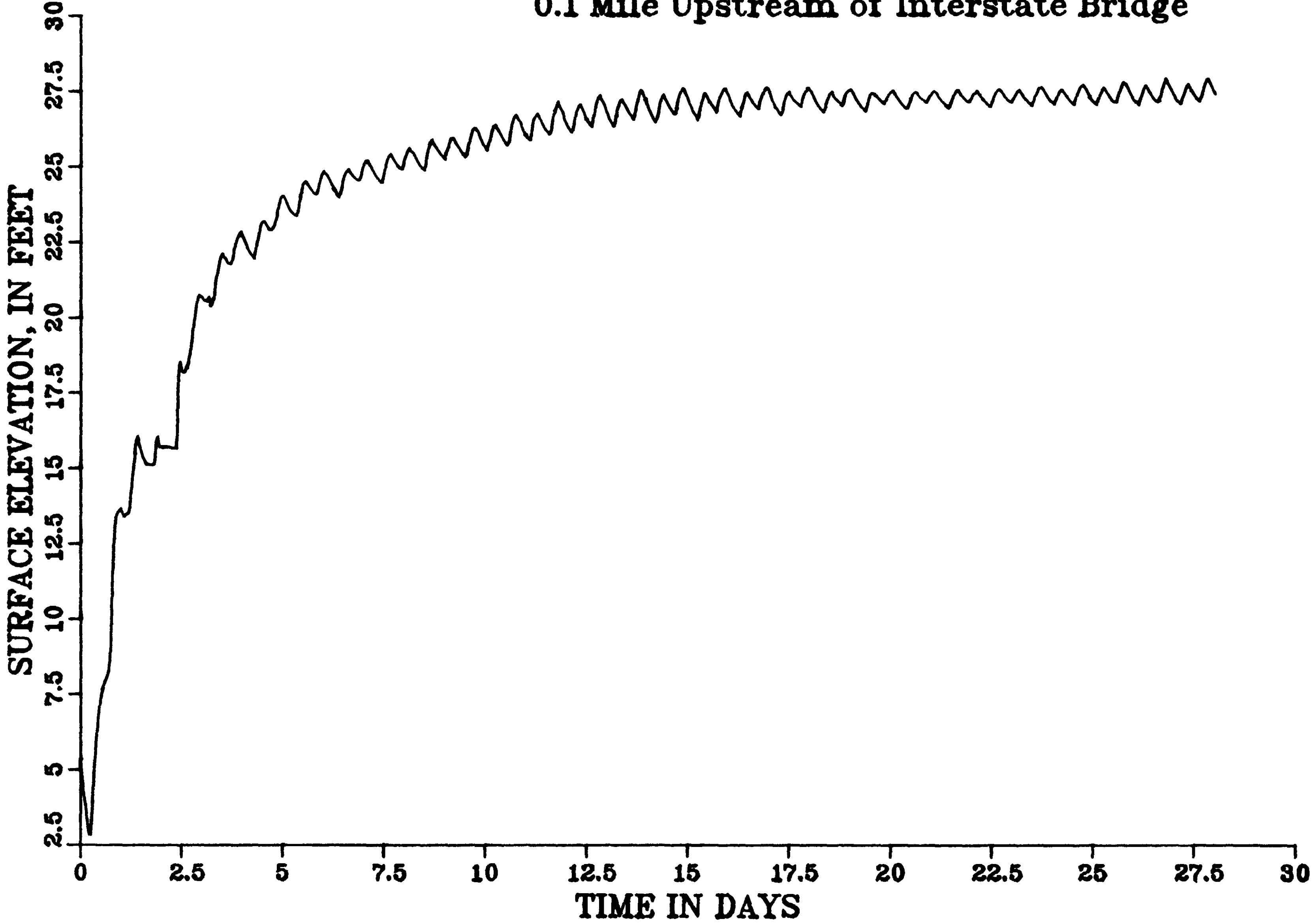


Figure 25
Columbia River at Mile 66.1
Predicted Total Discharge
Longview, Washington
0.1 Mile Upstream of Interstate Bridge

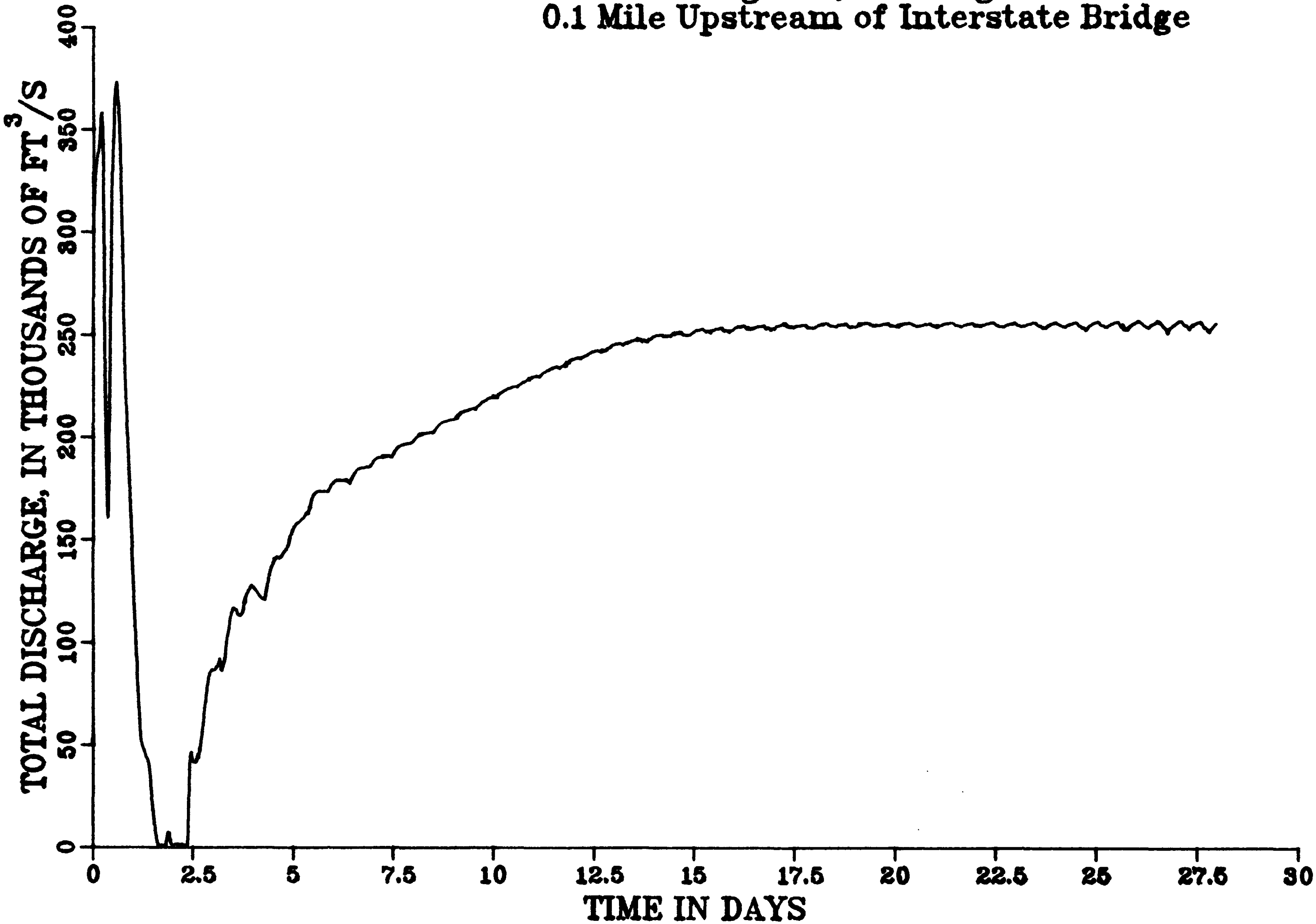


Figure 26
Columbia River at Mile 66.1
Predicated Bed Elevation
Longview, Washington
0.1 Mile Upstream of Interstate Bridge

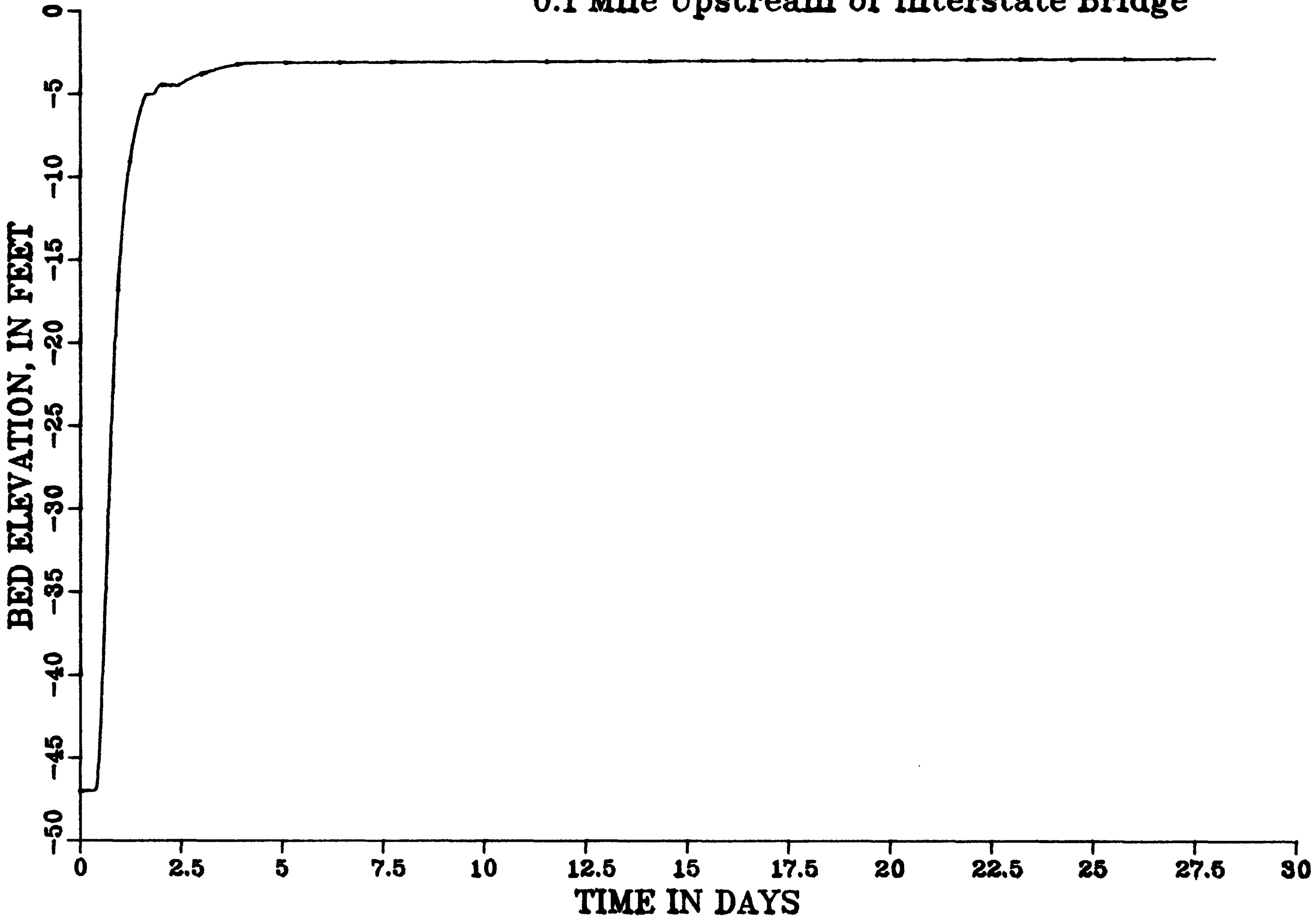


Figure 27
Columbia River at Mile 66.1
Predicted Sediment Discharge
Longview, Washington
0.1 Mile Upstream of Interstate Bridge

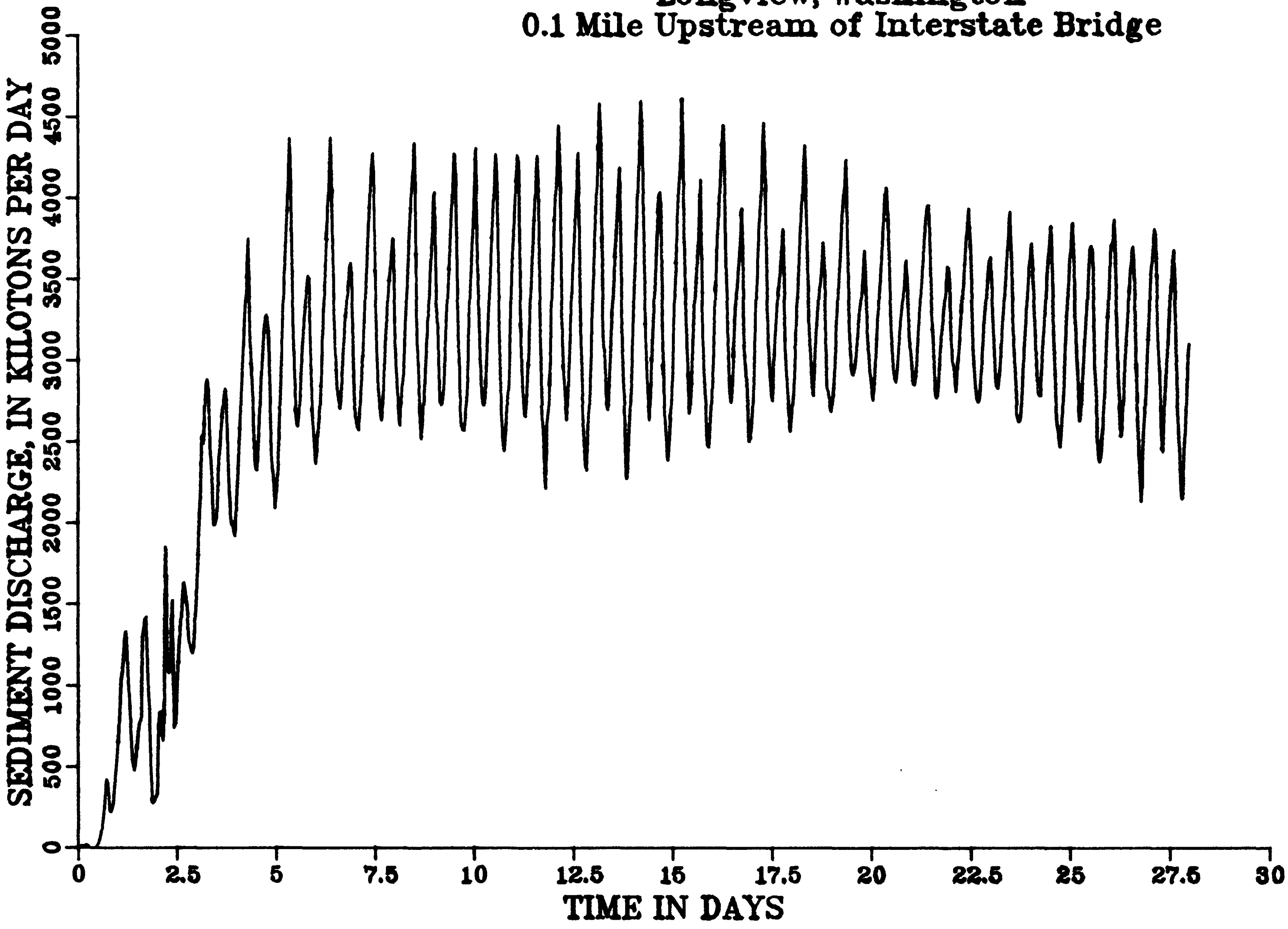


Figure 28
Columbia River at Mile 69.1
Predicted Water Surface Elevation
3.4 Miles Downstream of Trojan

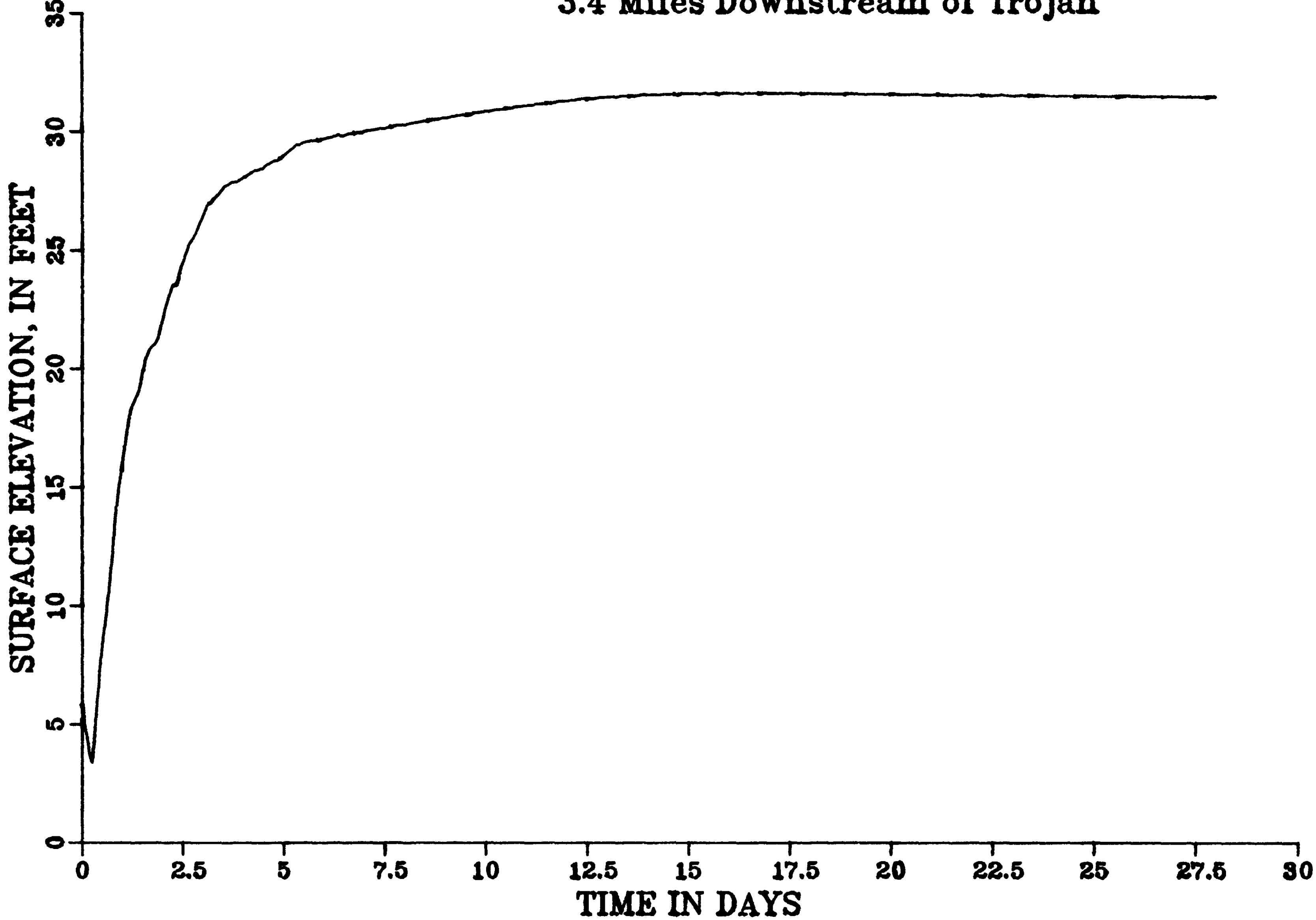


Figure 29
Columbia River at Mile 69.1
Predicted Total Discharge
3.4 Miles Downstream of Trojan

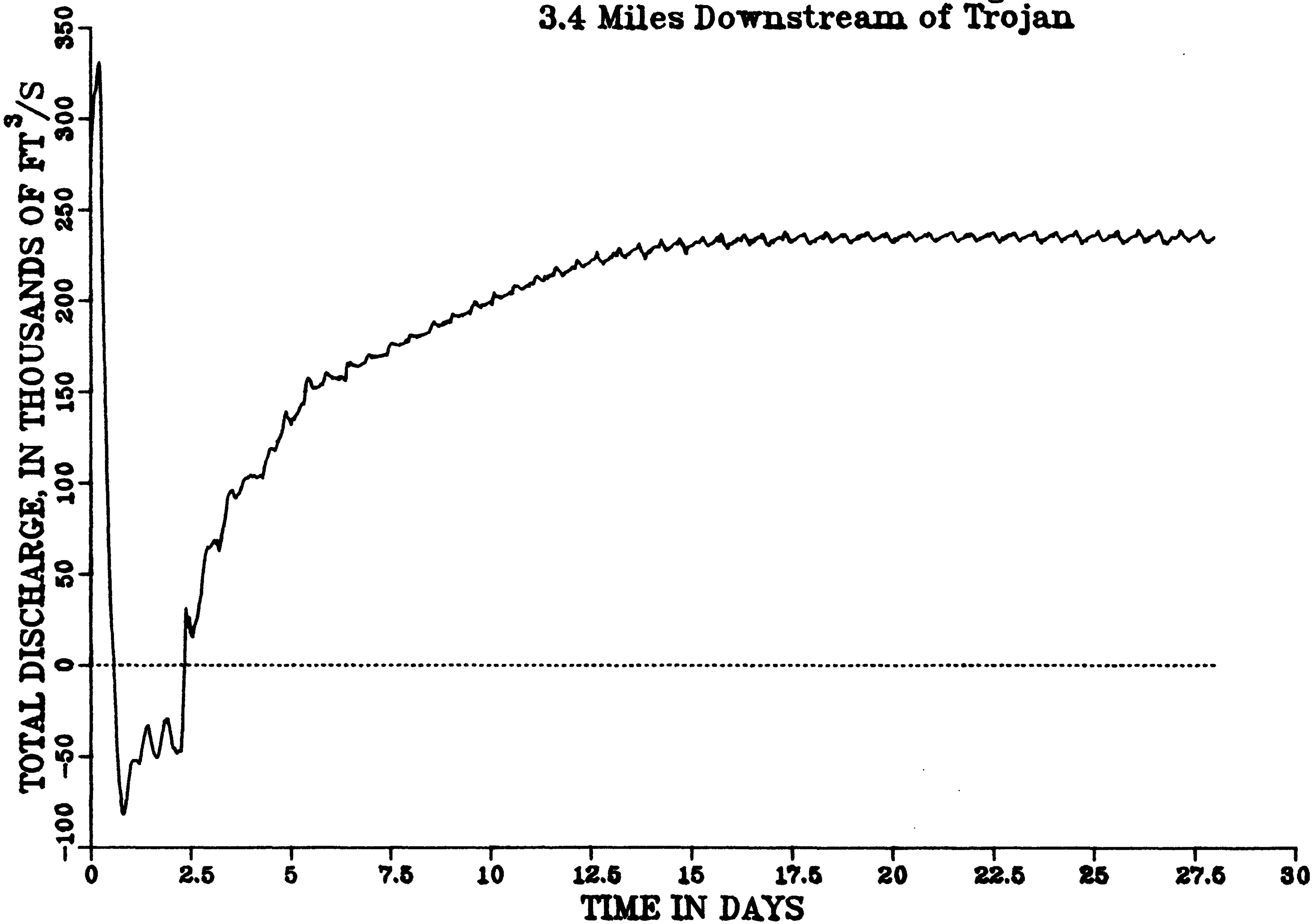


Figure 30
Columbia River at Mile 69.1
Predicated Bed Elevation
3.4 Miles Downstream of Trojan

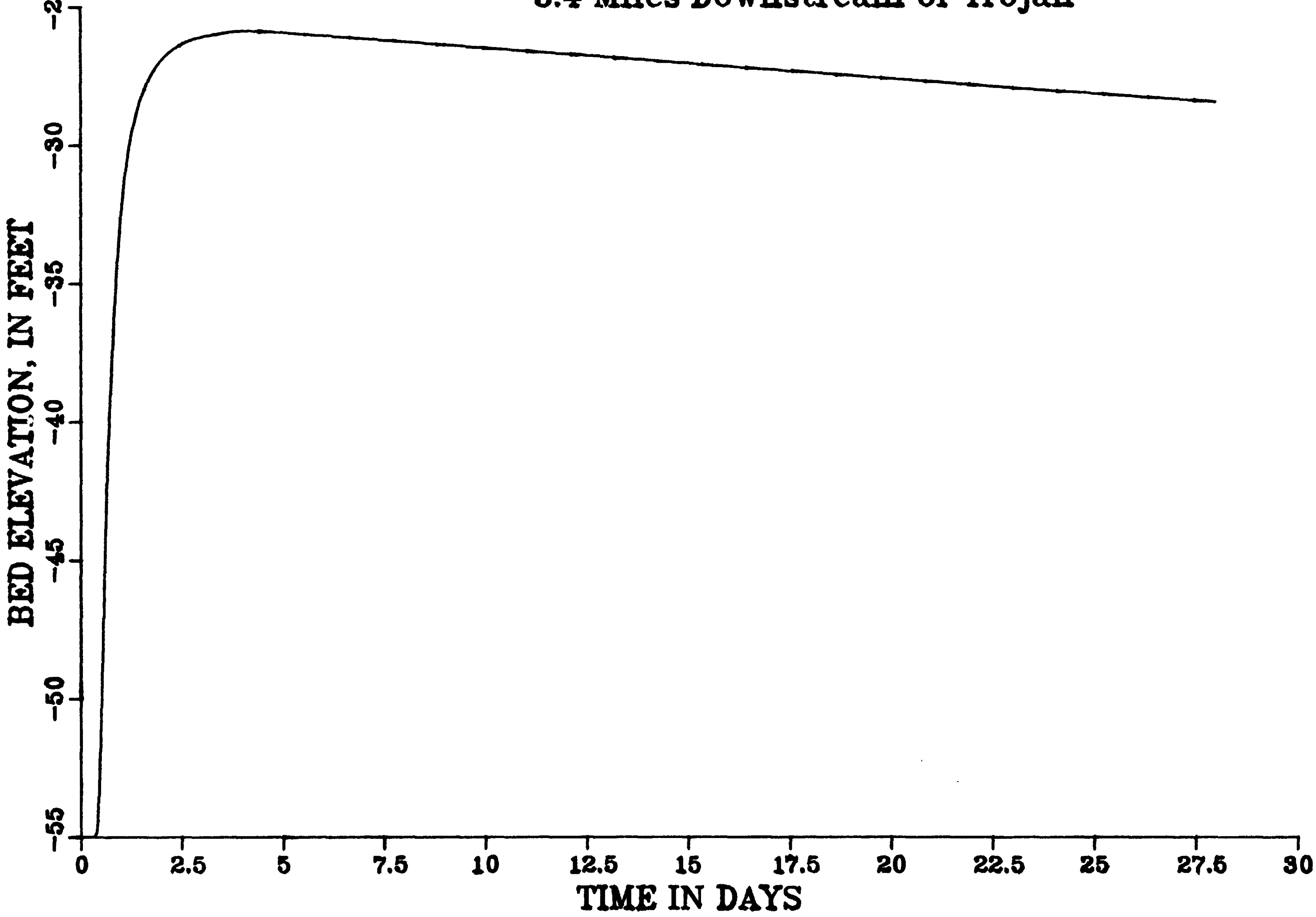


Figure 31
Columbia River at Mile 69.1
Predicted Sediment Discharge
3.4 Miles Downstream of Trojan

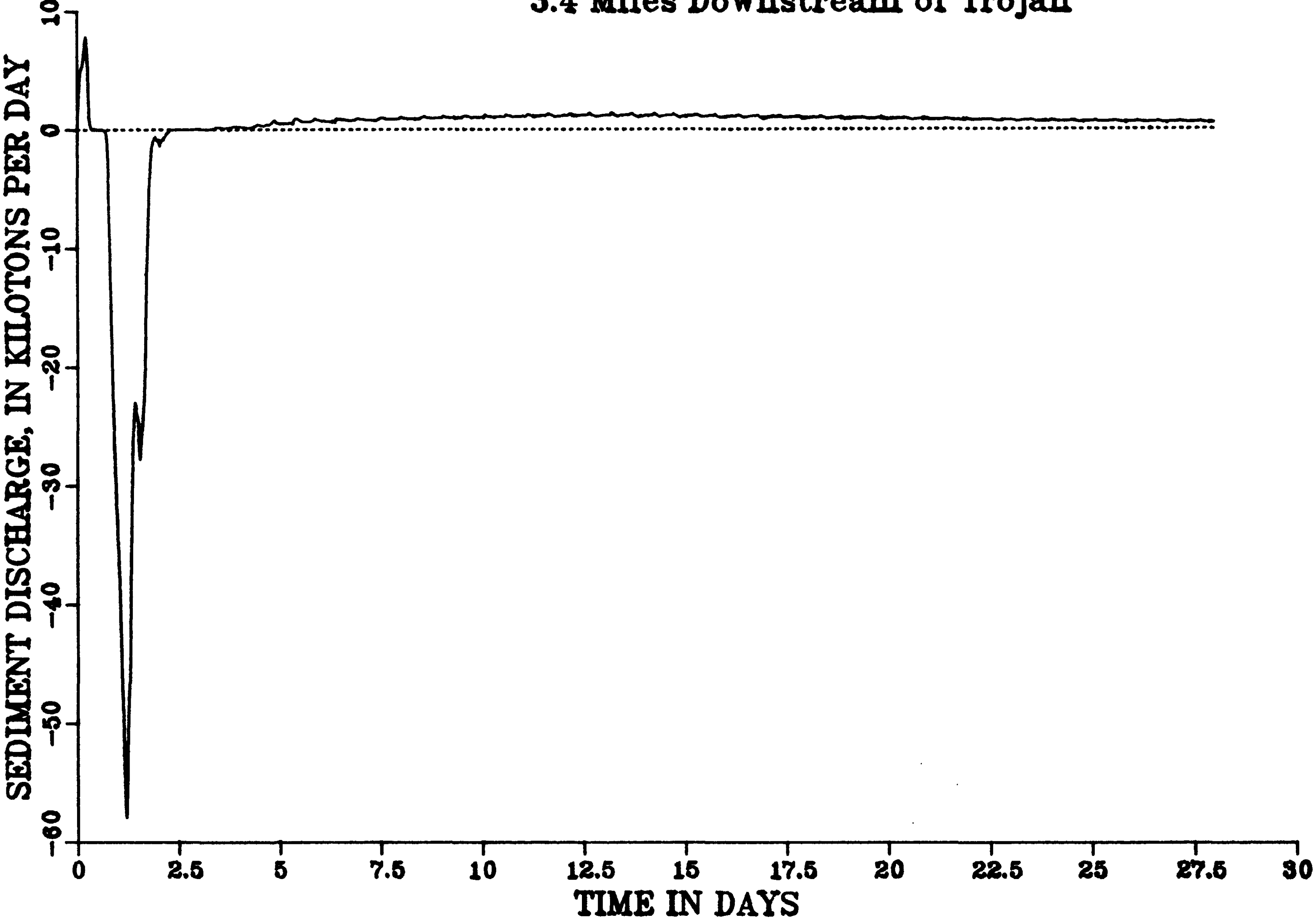


Figure 32
Columbia River at Mile 75.1
Predicted Water Surface Elevation
2.6 Miles Upstream of Trojan

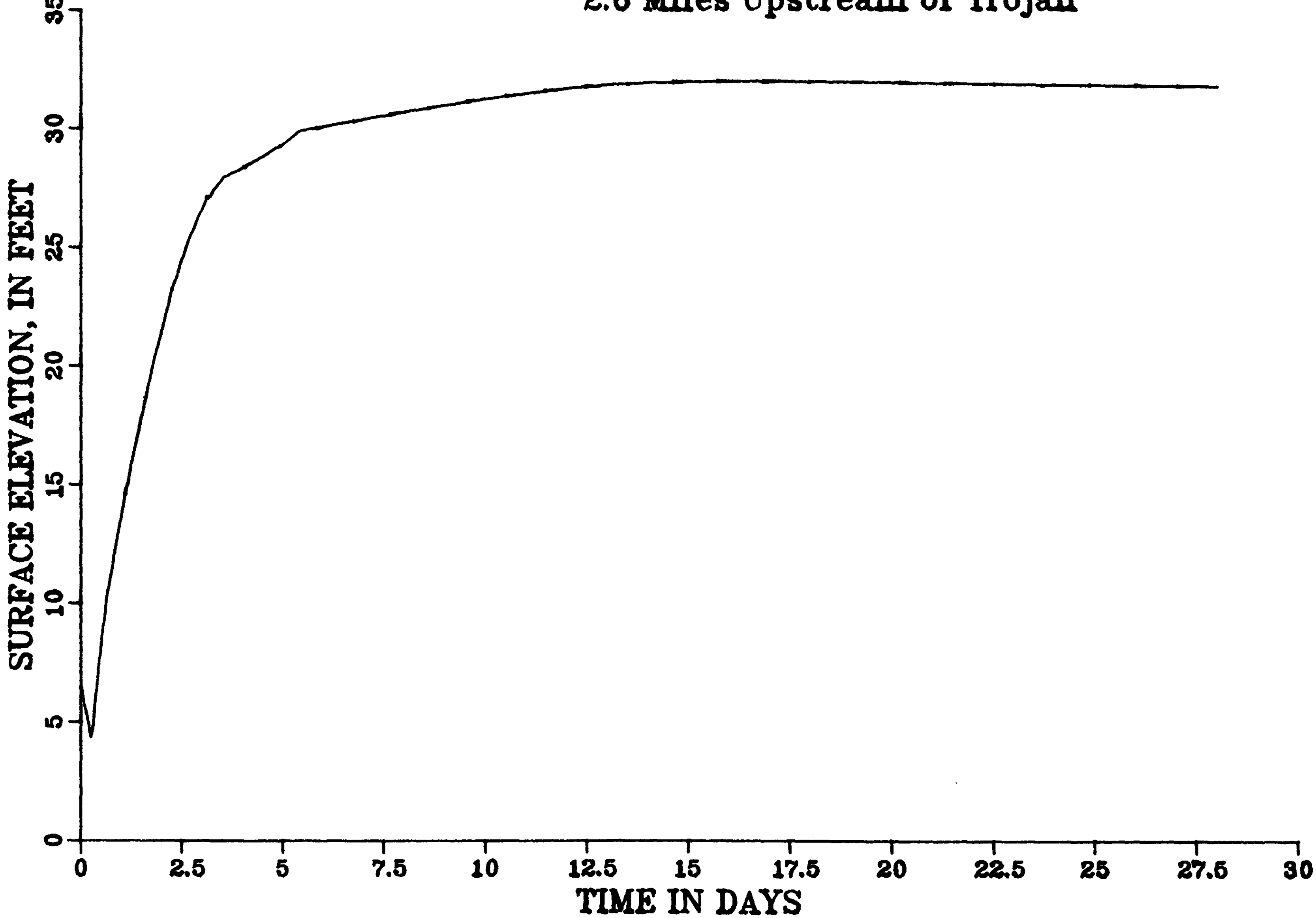


Figure 33
Columbia River at Mile 75.1
Predicted Total Discharge
2.6 Miles Upstream of Trojan

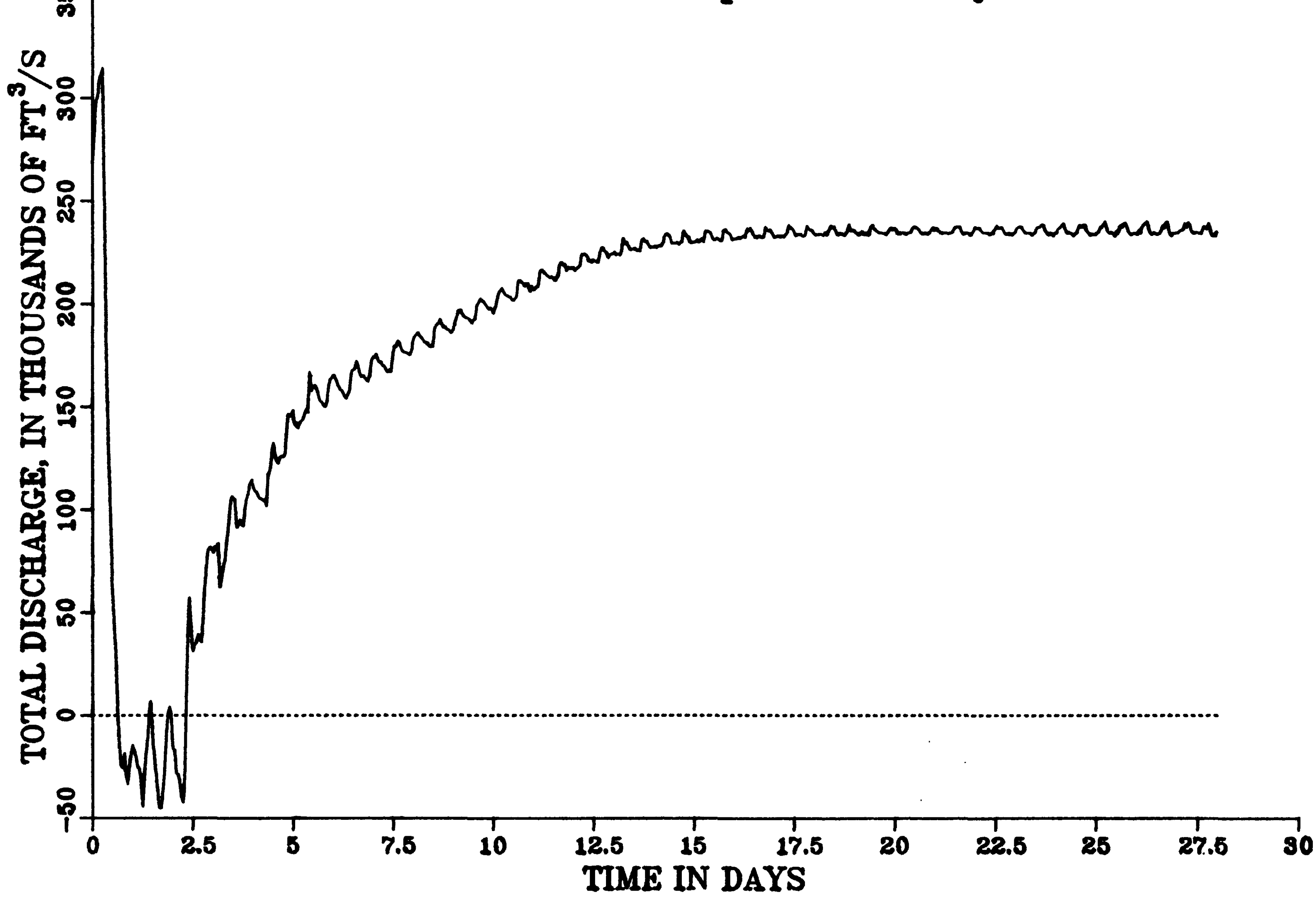


Figure 34
Columbia River at Mile 75.1
Predicated Bed Elevation
2.6 Miles Upstream of Trojan

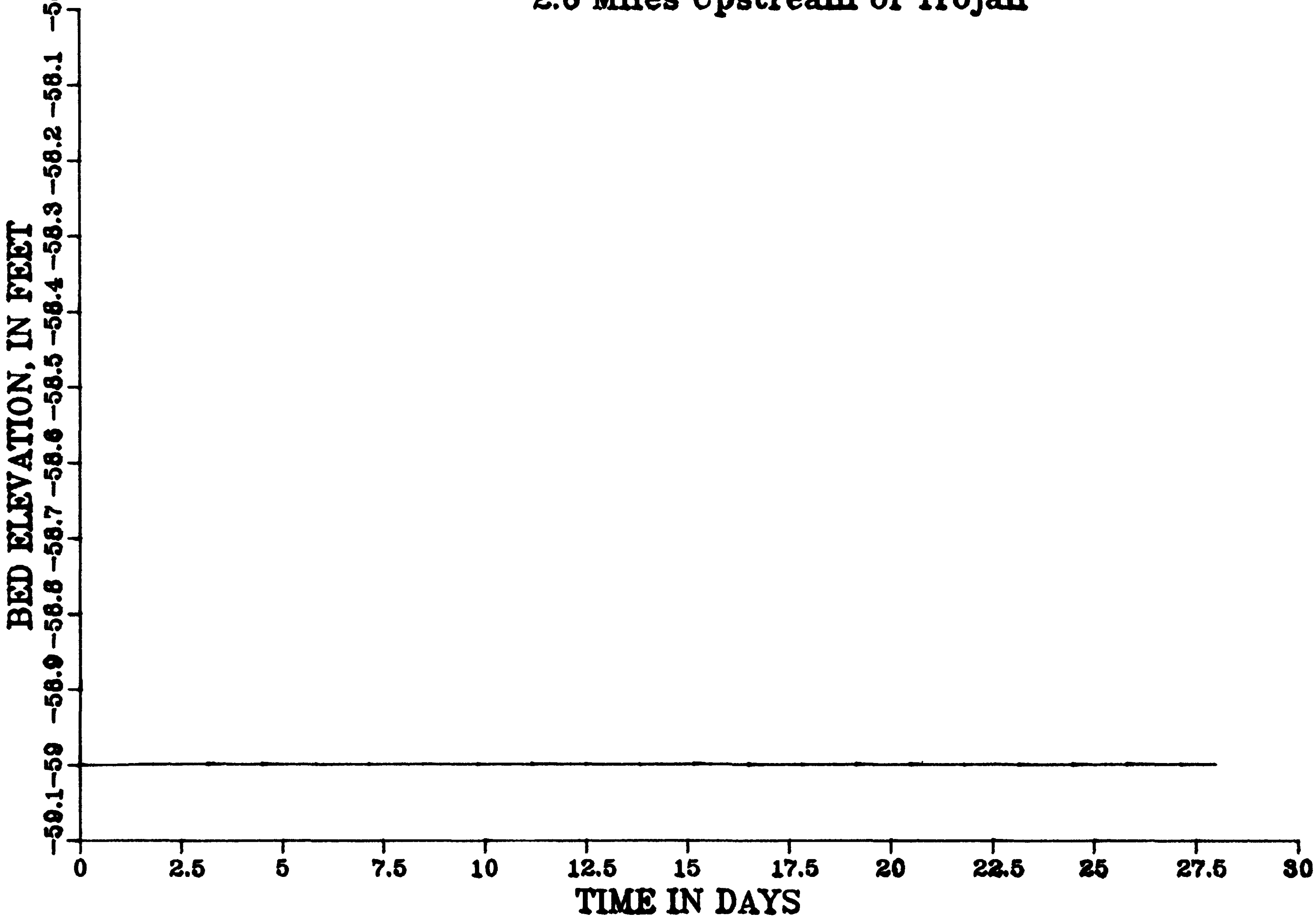


Figure 35
Columbia River at Mile 75.1
Predicted Sediment Discharge
2.6 Miles Upstream of Trojan

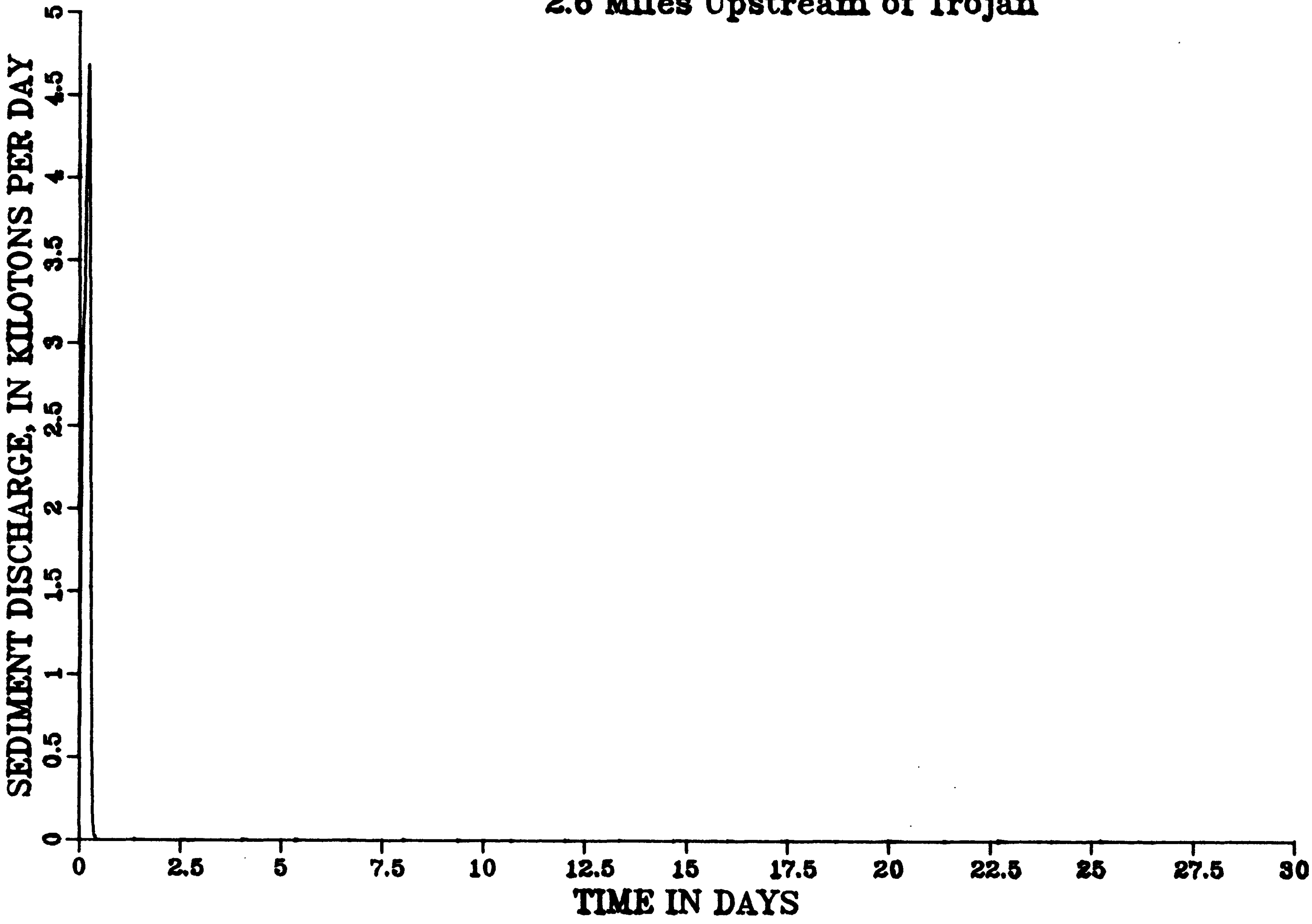


Figure 36
Columbia River at Mile 106.5
Predicted Water Surface Elevation
Portland, Oregon at Interstate Bridge

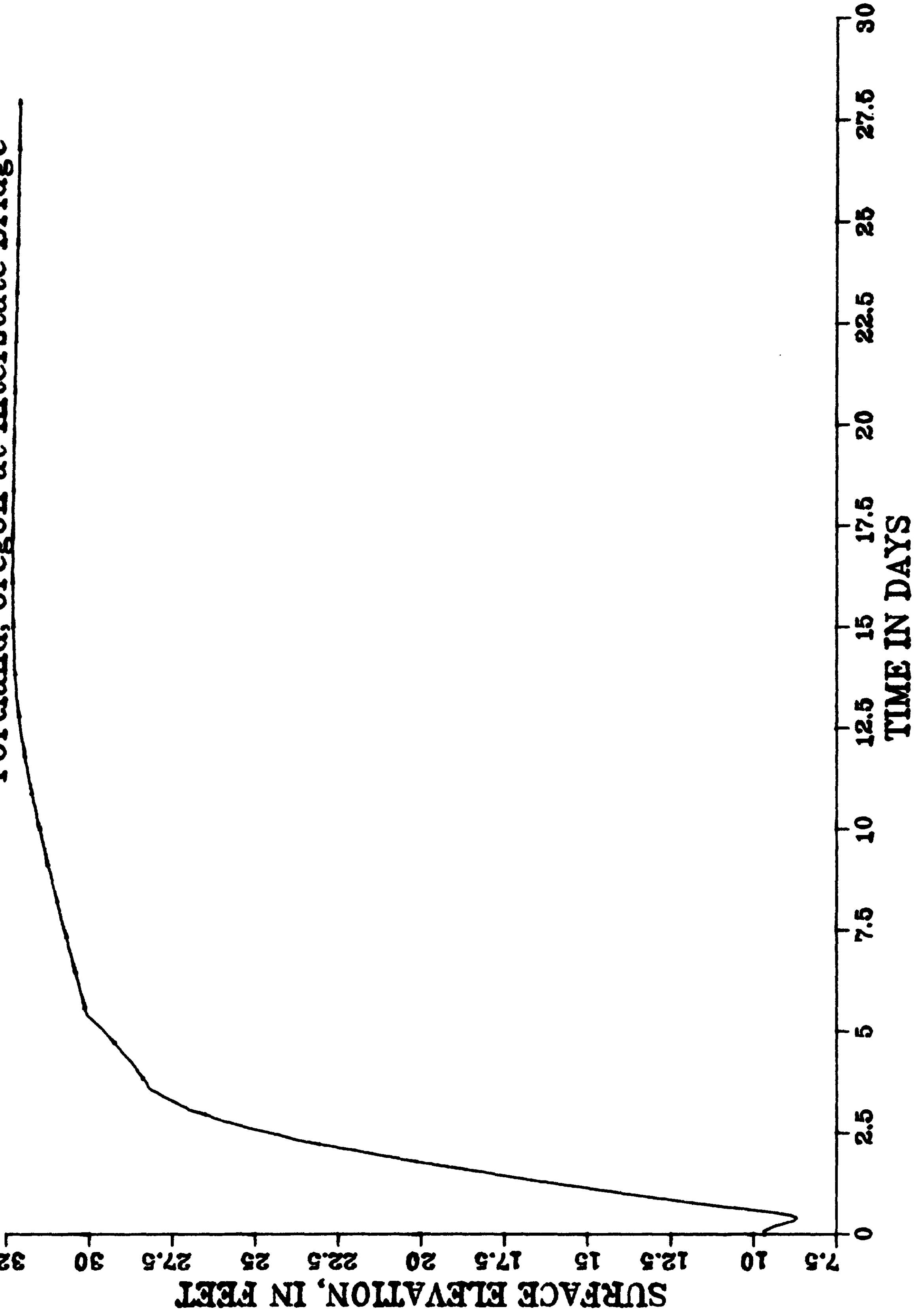


Figure 37
Columbia River at Mile 106.5
Predicted Total Discharge
Portland, Oregon at Interstate Bridge