Impacts of Columbia River Discharge on Salmonid Habitat: 1. A Nonstationary Fluvial Tide Model

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[1] This is the first part of a two-part investigation that applies nonstationary time series analysis methods and the St. Venant equations to the problem of understanding juvenile salmonid access to favorable shallow-water habitat in a tidal river. Habitat access is a function of river stage, tidal range, and the distribution of bed elevation. Part 1 models nonstationary tidal properties: species amplitudes and phases and tidal range. Part 2 models low-frequency river stage in the Lower Columbia River and reconstructs historical water levels, using the tidal model from part 1. To incorporate the nonstationary frictional effects of variable river discharge into the tidal model, we decompose the tidal wave into tidal species and calculate daily tidal range. Our one-dimensional tidal model is based on analytic wave solutions to the linearized St. Venant equation and uses six coefficients per tidal species to represent the upstream evolution of the frictionally damped tidal wave. The form of the coefficients is derived from the St. Venant equations, but their values are determined objectively from the data. About 50 station-years of surface elevation data collected (1981–2000) below Bonneville Dam (235 km from the ocean) were processed with a wavelet filter bank to retrieve time series of tidal species properties. A min-max filter was used to estimate daily tidal range. Tidal range, diurnal, and semidiurnal amplitudes were predicted with mean root mean square errors <30 mm, which is significantly more accurate than predictions obtained from harmonic analysis. Thus despite the compact form of our solution, we model nonstationary fluvial tidal properties with a high level of accuracy.

INDEX TERMS: 4560 Oceanography: Physical: Surface waves and tides (1255); 4227 Oceanography: General: Diurnal, seasonal, and annual cycles; 4235 Oceanography: General: Estuarine processes; 4215 Oceanography: General: Climate and interannual variability (3309);

KEYWORDS: Columbia River, tidal prediction, nonstationary tides, salmon habitat


1. Introduction

[2] The Columbia River (CR), as a major river in North America, is vital to North West American economy (e.g., fisheries, hydropower, ship-traffic). Less understood is the role of the CR in North East Pacific ecosystem dynamics. Populations of CR Basin salmon have diminished to a small fraction of their former diversity and abundance [Bottom et al., 2001]. Traditionally, environmental assessments have focused on obvious habitat changes upriver of the most seaward dam at Bonneville, 235 km from the ocean. Simenstad et al. [1990] and Independent Scientific Group [1999] indicate, however, the fundamental importance of tidal-fluvial and estuarine processes to salmonids. Despite its significance, relatively little attention has been paid to the tidal-fluvial portion of the system between Bonneville Dam and the upstream limits of salinity intrusion at about river kilometer (rkm) 15–30. In this portion, hereafter referred to as the Lower Columbia River (LCR), marsh, freshwater swamp, and seasonal floodplains are present [Thomas, 1983]. These shallow-water habitats not only supply organic matter to the estuary [Sherwood et al., 1990], but also provide migrating juvenile salmon with food resources, protection from predators, and an opportunity to prepare for the transition to marine conditions [Bottom et al., 2001].

[3] The availability of tidal-fluvial, shallow-water habitat in the LCR depends on the distribution of riverbed elevation (the hypsometric curve), river stage, and tidal range. Dredging, filling, and dike construction since the late nineteenth century have significantly decreased shallow-water habitat area [Thomas, 1983]. Also, climate change, flow regulation, and irrigation diversion have changed the magnitude and shape of the annual flow hydrograph, reducing peak flow by more than 40% and peak river stage by 0.5 up to 2.5 m during the spring and summer migration of juvenile salmo-
Because of the frictional interaction of river flow and tidal range, decreased spring-summer flows have decreased river stage, displacing habitat to lower elevations, and increased tidal range. Neither of these impacts has previously been quantified. Thus there is a need to assess the impacts of historical changes in both tidal range and low-frequency river stage in the LCR.

This is part 1 of a two-part investigation that focuses on the effects of historical changes in river discharge on stage, tidal range, and the availability of shallow-water habitat in the LCR. In order to determine the dependence of water surface elevation on river flow, we decompose water levels into low-frequency river stage and tidal variations. The objective of part 1 is to develop a nonstationary tidal model that captures, in a simple form, nonlinear interactions of variable discharge and ocean tidal forcing. In part 2, we present a river stage model for the CR that enables us, using the tidal range model from part 1, to reconstruct historical water levels and assess their impacts on the availability of shallow-water habitat. To accomplish these objectives, it is necessary to untangle the nonlinear interactions of flow and tides by representing stage and tidal properties in terms of external (fluvial and ocean tidal) forcing only.

We seek a representation of river tides that is, like harmonic analysis, extremely compact, yet powerful in its ability to hindcast or predict tides. In addition, a spatial model is desirable, so that tidal properties, analyzed at a finite number of locations, can be calculated throughout the river. Because of the need to calculate tides for periods of 1–120 years for a variety of scenarios, we have elected to use an approach that is closer to harmonic analysis than numerical modeling. Numerical models are not usually set up for such long-term predictions, and it is not a trivial task to elaborate on existing numerical models of the Columbia to do so [Baptista et al., 1999; Salerno and Markman, 1991]. A harmonic analysis relies on the assumption that a tidal wave consists of a sum of sine waves with constant amplitudes and phases, so that the tidal wave is stationary. Furthermore, each sine wave or tidal constituent, oscillates with an a priori known frequency derived from the tidal potential and nonlinear interactions. The phase and amplitude coefficients are determined from the data via a least squares fit. While the harmonic analysis has been very successful for stationary tides at a coastal station, it has shortcomings for very nonstationary tidal records [Jay and Flinchem, 1999]. The stationarity assumption is invalid when tides interact with variable river flow, as is the case for the LCR [Godin, 1984; Jay and Flinchem, 1997]. This can be illustrated by Vancouver (km 171) tides: during low-flow seasons the tidal amplitude can be as large as 1 m, while tides are almost unmeasurable during high-flow periods, because of the river flow damping.

Our tidal model applies wavelet transform tidal methods [Flinchem and Jay, 2000] and the dynamical model of Jay [1991] to analyze the nonstationary effects of variable river flow on tides. The dynamical model is an analytic solution to the linearized St. Venant equations, which incorporates fractional effects of river discharge due to bottom stresses. This approach is valid as long as the tidal amplitude to depth ratio is small and wave properties vary slowly relative to the tidal period. These conditions are usually satisfied in the CR. Further, in wavelet tidal analysis, a tidal wave is composed of tidal species, as is also done in the species concordance method [Simon, 1991]. Each tidal species consists of multiple, closely spaced tidal constituents. Both the wavelet and species concordance approaches rely on the nonlinear relationships between tidal species at an analysis station and at another reference station where the tide is well known and nearly stationary. The species concordance method requires long data records to take into account all possible combinations of tides and flow. Further, the model coefficients in the species concordance method do not allow a simple physical spatial interpretation, so that it is not straightforward to generalize a spatial model from coefficients determined for a few stations.

Analysis of the nonlinear interactions of river flow and neap-spring tidal forcing provides a simple model of the spatial evolution of tidal properties. The resulting closed-form model is sufficient to predict tidal species properties and tidal range throughout the system. Our study offers a new vision for the prediction of riverine tides and is, thus, a response to Godin’s recent conclusion: “Improved predictions of river tides will become possible when more careful consideration is given to fluctuations in river discharge, implying that short-time predictions should be considered, not conventional tide tables” [Godin, 1999].

2. Setting

The CR has the second largest flow in western North America, with an average discharge of \( \sim 7500 \text{ m}^3 \text{s}^{-1} \) [Sherwood et al., 1990]. The drainage basin encloses an area of 660,500 km\(^2\), and includes two subbasins (Figure 1).
The Interior Subbasin drains a large and mostly arid landscape, including parts of the Cascades, the Rocky Mountains in the United States and Canada, and the interior ranges of British Columbia. The Coastal Subbasin drains high precipitation terrain in Oregon and Washington west of the Cascade Mountains, including parts of the Oregon Coast Range. Although the Coastal Subbasin includes only 8% of the total surface area, it contributes roughly 25% to the total CR flow. The tidal-fluvial section of the CR system below Bonneville Dam is included within the coastal subbasin.

2.1. River Flow Variability

There are three characteristic timescales of CR flow variation: (1) interannual and lower frequency, (2) seasonal, and (3) daily and weekly variations caused by fluctuations in electric power demand ("power peaking"). Intannual and lower-frequency flow fluctuations are related to climatic variation [Latif and Barnett, 1994] and play a major role in the habitat investigation discussed in part 2. CR flows over the last 140 years show both interdecadal variability (the Pacific Decadal Oscillation [Mantua et al., 1997]) and a long-term decrease. Although irrigation depletion is responsible for part of the decline in river flow, the impact of long-term climate change is of equal magnitude [Sherwood et al., 1990; Bottom et al., 2001]. Before flow regulation, annual maximum discharge was usually observed during the May-July freshet period, due to snowmelt mainly in the Interior Subbasin. Transient winter high-flow events occur when heavy snowmelt and rainfall in the western subbasin accompany warm and intense storms. During November to March, river flow may fluctuate on timescales of days to weeks.

Flow regulation also now causes spring freshet flows to follow a different time history than they would in the absence of flow regulation (Figure 2). Reservoir storage (amounting to ~60% of mean annual flow volume) has greatly reduced spring freshet amplitude, increased fall and winter flows, and decreased seasonal flow variability [Bottom et al., 2001]. The maximum monthly mean flows during the spring freshets have been reduced by an average of 7500 m³ s⁻¹, and now seldom exceed 15,000 m³ s⁻¹ [Bottom et al., 2001].

Finally, an irregular daily power-peaking cycle introduces a pseudodiurnal tide, propagating seaward from Bonneville Dam at km 234. The power-peaking cycle also exhibits weekly fluctuations due to lower-power demand on weekends. Power peaking is often suppressed during high-flow periods, because water is spilled when power demand drops. Power-peaking fluctuations propagate as waves [Wiele and Smith, 1996], but differ from tides in that they are broadband, not frequency-limited signals. All these annual changes in flow cycle have an impact on the tidal properties of the LCR.

2.2. Tidal Processes

The tidal range in the LCR is ~1.7–3.6 m at the ocean entrance and increases to a maximum between 2.0 and 4.0 m, at Astoria (km 29). It then decreases in the landward direction to an average smaller than 0.2 m above Vancouver (km 171). The tide has a mixed character with a ratio of semidiurnal to diurnal amplitude of 1.5 at the estuary mouth. CR tides are nonstationary landward of km 30, so that a description of mean properties in terms of tidal constituents is an approximation. The principal lunar component (M₂) increases from 0.82 m at the mouth of the river to 0.95 m at Tongue Point during low-flow season [Jay, 1984], and then steadily decreases landward. The lunar-solar component (K₁) is nearly constant at 0.4 m over the lower 30 km, before landward damping occurs. Tidal propagation in the main channel is weakly nonlinear with respect to depth fluctuations, since the amplitude depth ratio is ~0.1 in the estuary and decreases thereafter. Nonlinear tidal interactions (self-damping) generate even overides. The ratio of M₂ to its first overtide, M₄, is 30–50 in the lower estuary, and decreases to 3–10 in the tidal river [Giese and Jay, 1989]. There is an abrupt 180° phase change in M₄ at km 35, suggesting that strong river flow dominates fluvial overide generation landward of this point, whereas the incoming ocean wave and frictional effects associated with tidal flats are important in the estuay [Jay and Musiak, 1996].

The main tidal species in the LCR are diurnal (D₁) and semidiurnal (D₂). Landward of km 35, significant energy is transferred to the quarter-diurnal wave (D₄) due to bedstress interaction with the semidiurnal wave. The tertiary species (D₆) results from frictional interaction of D₁ and D₂, and is usually smaller than D₄ and not simulated here. [Jay et al. [1990] suggested that the energy budget for the LCR exhibits three reaches: (1) the tidally dominated lower estuary from the ocean entrance up to ~km 15, (2) an intermediate, dissipation-minimum between km 15 and 50, and (3) a tidal-fluvial reach landward of km 50. In the first regime, energy for circulation is derived primarily from barotropic tides. Both tidal and fluvial energy are important in the second reach, although dissipation remains small. The upstream limits of salinity intrusion and a long-term locus of deposition are found in the dissipation-minimum region [Giese and Jay, 1989]. Dissipation in the tidal river is derived mainly from the river flow. Our analysis separates the reach landward of km 140 from the rest of the tidal-
3. Nonstationary Fluvial Tide Model

[15] The strategy employed here to describe CR fluvial tides is to use an analytical solution to model the dependence of tidal amplitude and phase on upriver location and river flow. The model coefficients for each species are determined by regression analysis to optimize the prediction power of the model. The spatial pattern of the coefficients is consistent between species and yields a clear physical interpretation.

3.1. Theory of Fluvial Tides

[16] The distinct and complex motion of riverine tides [see e.g., Godin, 1984] is caused by interactions among tidal constituents and freshwater discharge. They can be understood by the analysis of the governing St. Venant equations:

\[ \frac{\partial Q}{\partial t} + \frac{\partial}{\partial x} \left( \frac{Q^2}{A} \right) + gA \frac{\partial z}{\partial x} + bT = 0, \quad (1a) \]

\[ \frac{\partial Q}{\partial x} + b \frac{\partial z}{\partial t} = 0, \quad (1b) \]

where \( x \) is the along-channel distance (m); \( x = 0 \) at estuary entrance, \( x \) decreases landward; \( t \) is the time (s); \( z(x,t) \) is the tidal surface elevation (m); \( Q(x,t) \) is cross-sectionally integrated tidal transport (m³ s⁻¹); \( z \) and \( Q \) are complex numbers; \( A(x,Q_R) = bh \) is the channel cross-sectional area (m²); \( b(x,Q_R) \) is the channel width (m); \( h(x,Q_R) \) is mean channel depth (m); \( Q_R \) is the river flow transport (m³ s⁻¹); \( g \) is the gravitational acceleration (9.81 m s⁻²); and \( T \) is the bedstress divided by water density (m² s⁻²).

[17] The cross-sectionally integrated momentum equation (1a) indicates that the local acceleration (first term from left) is due to the convective acceleration (second term), water surface slope (third term), and friction (fourth term). The cross-sectionally integrated continuity equation (1b) shows along-channel changes in water transport are balanced by temporal changes in water surface elevation. Equations (1a) and (1b) together suggest that the propagation of a tidal wave is determined by the balance of inertia, friction, and topography. With slowly varying cross-sectional channel area and in the absence of friction, gravitational forces balance local acceleration, resulting in a dynamic wave [Lamb, 1932]. Energy flux is conserved, and Green's Law applies. Green's Law relates tidal amplitude inversely to \( b \) and \( h \) as \( b^{-1/2}h^{-1/4} \) [Green, 1837]. In most estuaries, however, friction and topographic funneling cause the tidal wave to deviate from this form. The LCR is divergent from the ocean entrance to rkm 11, convergent up to rkm 50, and weakly convergent thereafter [Giese and Jay, 1989]. Following the work of Lanzoni and Seminara [1998], the LCR as a whole can be classified as a "strongly dissipative and strongly convergent" estuary, which is similar to that in the work of Jay's [1991] "critical convergence" regime. In this regime, inertia is negligible, which causes tidal propagation to approach a diffusive condition, as in the Fraser River and many other river estuaries [LeBlond, 1978; Jay, 1991]. Friedrichs and Aubrey [1994] showed that tidal wave distortion in a strongly convergent channel can be approximately described by a first-order differential equation, but the "critical convergence" regime is more realistic for the CR, where channel cross-section convergence rate is small landward of rkm 50.

[18] A key point in modeling frictional effects on tides is the representation of the bottom stress [Godin, 1991]:

\[ T = c_D |U|U, \quad (2a) \]

where \( c_D \) is the drag coefficient and \( U \) is the flow velocity. Using an expansion in odd powers, Godin elucidated various interaction mechanisms of tidal constituents, and the generation of odd overtones. The Tschebyschev polynomial approach [Dronkers, 1964] provides an intuitively appealing explanation of the change in character of tidal interaction with river flow. With this approach, the bedstress can be expressed as

\[ T \approx c_D \left( p_0 U_0^2 + p_1 U_0 U + p_2 U^2 + p_3 U^3 / U_0^2 \right) / \pi, \quad (2b) \]

where \( U_0 \) is a flow scale determined by half the velocity range and \( p_i \), \( i = 0,1,2,3 \) are the Tschebyschev coefficients, which depend on the ratio of tidal current amplitude \( U_T \) to river flow currents \( U_k \) (see also Figure 3). The coefficient \( p_0 \) is close to zero and vanishes in the formation of the wave equation, where equation (1a) is differentiated. The second \( (p_1) \) term on the left-hand side in equation (2b) describes linear self-damping. The third \( (p_2) \) term expresses quadratic interaction of the tidal wave with river flow. The last \( (p_3) \) term describes cubic interactions. Thus the Tschebyschev coefficients \( p_1,p_2,p_3 \) (Figure 3) determine, together with river and tidal flow, the bedstress characteristics. The coefficients \( p_1 \) and \( p_2 \) decrease with increasing \( U_k/U_T \) until they converge to zero for \( U_k < U_T \). This is the case further upriver where there are no current reversals (roughly landward of Beaver, rkm 87). Then \( p_0 = p_1 = p_2 = 0 \) and \( p_3 = \pi \), so that the bedstress becomes \( T = c_D U_T^2 \), an even function in \( U \). If \( U_k \) approaches zero, \( p_0 = p_2 = 0 \), \( p_1 = 16/15 \), and \( p_3 = 32/15 \) so that \( T \) is an odd function in \( U \). This bedstress approach can also be used to solve the St. Venant equations [Jay, 1991]; we follow it, therefore in our analysis.

3.2. Analytic Solutions

[19] To derive an approximate solution to equations (1a) and (1b), we expand them in the small-perturbation parameter, the ratio of tidal amplitude to depth \( \varepsilon = |z|/h \). Keeping only lowest and first-order terms and subtracting the time derivative of the momentum equation (1a) from the location derivative of the continuity equation (1b) results in the governing wave equation:

\[ \frac{\partial^2 Q}{\partial x^2} - \frac{1}{b h} \frac{\partial Q}{\partial x} - \frac{2}{1} \frac{U_R}{h} \frac{\partial^2 Q}{\partial x \partial t} + \frac{1}{gh} \frac{A}{dx} \frac{\partial Q}{\partial t} - \frac{1}{gh} \frac{Q}{\partial t} - \frac{b}{gh} \frac{\partial T}{\partial t} = 0. \quad (3) \]

The terms from left to right are due to the pressure gradient, pressure gradient and topography, convective accelerations (two terms), local acceleration, and bedstress. This formulation and its approximate solution below use the
Figure 3. Tschebyschev coefficients $p_1$, $p_2$, and $p_3$ and the integral $P_3$ (defined in section 5.2.2) as function of upriver distance. The coefficients $p_1$, $p_2$, and $p_3$ determine the magnitude of linear, quadratic, and cubic frictional interaction, respectively. The following assumptions were made for this plot: current reversal occurs only seaward of Beaver, the ratio of tidal to river flow currents at the estuary entrance is 5 (corresponding to average river discharge of 7500 m$^3$ s$^{-1}$ and average tidal transport amplitude of 37,500 m$^3$ s$^{-1}$ at the estuary entrance), and tidal currents decrease linearly with $x$.

The following assumptions: (1) The tidal transport $Q$ is one-dimensional in $x$. (2) The channel geometry is tidally invariant, and exponentially varying in depth and width, although the convergence rate may be a function of $x$. The effects of tidal flats, bifurcations, and islands are neglected, though they are quite prominent in the first 50 km. (3) River flow enters only at one source far upriver and varies slowly relative to the tides. This is fulfilled to the lowest order, although power-peaking cycles and river flows from the Willamette River (rkm 165) and other tributaries can cause occasional violations of this assumption. (4) For forcing mechanisms, effects of wind stresses and baroclinic forcing are neglected. Both are significant forcing mechanisms in the estuary, but are small farther landward [Jay, 1987; Jay and Musiak, 1996]. (5) For interactions of tidal species, it is assumed that each tidal species can be treated in isolation, aside from frictional generation of overtides by both $D_1$ and $D_2$, and the influence of $D_2$ on $D_1$.

[21] In our implementation of equation (4), tidal flats are neglected and only lowest order terms are kept. Thus small corrections to the wave number and phase due to convective accelerations are not considered here. Because the depth convergence rate in the LCR is small, the transformed $x'$ coordinate $x' = x/(g^{1/2}h)$ [Jay, 1991] is directly proportional to $x$, in the absence of tidal flats. The use of undistorted coordinates may have, however, a small effect on the coefficients determined from the data. The boundary conditions are: (1) the amplitude and phase are known at the estuary mouth, and (2) the wave vanishes for large $x$, so that the reflective wave is absent. The complex wave number $q = \kappa + ir$ governs wave propagation. For critical convergence, $q$ is given to lowest order by [Jay, 1991, equation (22)]:

$$q = \kappa + ir \quad \text{with} \quad \kappa = -r = \frac{1}{c_0} \sqrt{F \omega},$$

where $c_0$ is the inviscid wave speed, i.e., $c_0 = (gh)^{1/2}$.

$$F = \frac{U_0 c_D}{H \pi} P(u_R, u_T),$$

$$P \equiv p_1 + p_2 u_R + p_3 \left(3 u_T^2 \frac{1}{2} u_T^2\right),$$

where $U_0$ is the velocity scale arising from the bedstress representation [Dronkers, 1964] (m s$^{-1}$); $H$ is the depth scale equal to 10 m; $u_T$ is the amplitude of scaled tidal velocity (dimensionless); $u_R$ is the scaled river flow velocity (dimensionless); and $p_i$ is the Tschebyschev coefficients, where $i = 1, 2, 3$. 

$$z(x, t) = \text{Re} \left(-iB q \frac{\exp(i \omega t - iq x)}{A(x)}\right),$$

where $\omega$ is the tidal frequency (s$^{-1}$), $B$ is the complex constant determined by boundary conditions (m), $q = \kappa + ir$ is the complex wave number (m$^{-1}$), $\kappa$ is the wave number (m$^{-1}$), and $r$ is the damping modulus (m$^{-1}$).
Note that $F$ is a friction factor that arises from the bedstress linearization of equation (2b) [Jay, 1991]. With this linearization, the bedstress becomes $T = QF/h$. The factor $P$ in $F$ contains the tidal parts of the Tschebyschev polynomial. Using this solution, we develop a simple regression model applicable to each tidal species.

### 3.3. Regression Model for Normalized Amplitude and Phase

Practical application of the above model requires manipulation of equations (4)–(5c) into a form allowing a regression analysis to determine modified forms of the $p_i$, $i = 1, 2, 3$, for each station and species. The oscillatory character of $z(x,t)$ is taken into account by considering tides to be the sum of a small number of species. Each species is harmonic despite the subtidal evolution of the wave, with amplitude $|z|$ and phase $\varphi$. Normalizing by the incoming ocean tide, one obtains the log-normalized amplitude $Z$ and phase difference $\Delta \varphi$:

$$Z(x) = \log \left( \frac{|z(x,t)|}{|z(0,t)|} \right) = \frac{1}{2} \log \left( \frac{A(0)}{A(x)} \right) + rx,$$

$$\Delta \varphi(x) = \arg(z(x,t)) - \arg(z(0,t)) = -\kappa x. \quad (6a)$$

$Z$ and $\Delta \varphi$ should be related to river flow and forcing ocean tides. To find the simplest and most physical linear regression model, we examine more closely the damping modulus $r = -\kappa$ (see Appendix A):

$$r = \frac{1}{\sqrt{c_0}} \frac{\sqrt{F_2}}{\phi_2} \left[ p_1 + p_2 u_R + p_1 \left( \frac{3u_R^2 + 1}{2} \right) \right] \approx c_i' u_R + c_2' \frac{u_R^2}{u_R^2} \approx c_1' u_R + c_2' \left( \frac{c_0}{U_0 h} \right) \frac{z_0^2}{u_R^2}, \quad (7)$$

where the coefficients $c_i'$ and $c_2'$ are defined in Appendix A. For the derivation of equation (7), it is assumed that $u_R$ is at least of the same order of magnitude of $u_T$, which is valid roughly landward of rkm 87. Although tidal transport can exceed river discharge seaward of rkm 87, equation (7) captures usefully tidal and river flow forcing, as shown below. The tidal current velocity was estimated using tidal theory for an inviscid wave with $u_T \approx z_0 c_p(h/U_0)$ where $z_0 = z_0(t) = z(0,t)$. A more accurate estimate can be derived from the continuity equation, which requires, however, knowledge of the wave number. Nevertheless, the continuity equation suggests that the amplitudes of tidal height and transport are, in the absence of a reflected wave, directly proportional to one another.

The coefficients $c_1'$ and $c_2'$ are proportional to the square root of the tidal frequency and vary with $u_R/u_T$. To lowest order, they are proportional to $p_2$ and $p_3$, respectively. If the coefficients $d_1$, $d_2$, and $d_0$ are defined as:

$$d_1(x) = c_1' x, \quad (8a)$$

$$d_2(x) = c_2' \left( \frac{c_0}{U_0} \right)^2, \quad (8b)$$

$$d_0(x) = \frac{1}{2} \log \left( \frac{A(0)}{A(x)} \right), \quad (8c)$$

the log-normalized amplitude $Z$ can be modeled linearly in the parameters $d_0$, $d_1$, $d_2$, using $u_R$ and $z_0$ as the only input variables. The coefficient $c_i'$ is a function of $x$ because $c_i'$ depends on the Tschebyschev coefficients. Thus $d_i$ will only vary linearly with $x$ further upriver where the Tschebyschev coefficients are constant.

Equations (6a) and (6c), and $r = -\kappa$ suggest that $Z - \Delta \varphi = d_0$ so that the phase difference can be modeled by analogy to $Z$. Defining an offset coefficient, $d_0'$ in equation (6b) for $\Delta \varphi$ (due to the simplifications made in equation (7)) yields for the phase difference $\Delta \varphi = -\kappa x + d_0'$. The complete analysis bears for $Z$ and $\Delta \varphi$ for the dominant tidal species:

$$Z(x) \approx d \cdot X^T, \quad (9a)$$

$$\Delta \varphi(x) \approx d' \cdot X^T, \quad (9b)$$

where $d = (d_0, d_1, d_2)$ is the amplitude coefficient; $d' = (d_0', d_1', d_2')$ is the phase coefficient;

$$X = \left( 1, u_R, \frac{1}{\sqrt{u_R}} \left[ \frac{z_0}{h} \right]^2 \right);$$

center dots are the matrix multiplication operator; $X^T$ is the transpose of $X$.

The coefficient $d_0$ is primarily determined by geometry, the divergence/convergence of the channel cross section. Coefficient $d_1$ is determined by nonlinear interactions with river flow, while $d_2$ represents nonlinear interactions due to neap-spring variability. In principle, $d_i = -d_i$, $i = 1, 2$. Optimal modeling of the data, however, requires that $d_i$ remain distinct from the $d_i$. This model applies only to major tidal species. Thus there is still a need to estimate the behavior of the tidal range and overtides.

### 3.4. Tidal Range

Daily tidal range is estimated as the difference between maximum and minimum heights during one 12.42 hours tidal period. The mean tidal range is approximately twice the amplitude of the dominant tidal constituent, $M_2$. The actual range, however, is dependent on the phases and amplitudes of all the larger constituents, as manifested in the grouping formula given by the U.S. Coast and Geodetic Survey [1952, p. 10]. Since major tidal constituents in the ocean ($M_2$, $S_2$, $N_2$, $K_1$, and $O_1$) are stationary, the tidal range can be easily predicted from harmonic properties near the estuary mouth. Further upriver, the tidal species interact through the bedstress with each other and with the variable river flow. This causes the generation of nonstationary tides, complicating significantly the analysis and prediction of the tidal range in the tidal-fluvial part of the system.

A lowest order dependency of the range coefficients $d_i$ on the semidiurnal and diurnal coefficients, $d_{Di}$ and $d_{Di}$, can be derived by approximating the log-normalized tidal range $Z_R$ as:

$$Z_R(x) \approx \log \left( \frac{|D_2(x)| + \cos(\gamma)|D_1(x)|}{|D_2(0)| + \cos(\gamma)|D_1(0)|} \right). \quad (10a)$$

where $\gamma$ is the phase angle between the diurnal and semidiurnal wave. For the lowest order estimate, we assume that...
the tidal range is only composed of the $D_1$ and $D_2$ waves. This is justified when constructing a daily analysis range on a tidal day, so that constructive superposition of $D_1$ and $D_2$ can take place. Setting now $D_1(0) = \beta D_2(0)$ where $D_2$, $D_1$ is the tidal height (in meters) of the semidiurnal, diurnal tidal species, respectively, and $f = \beta \cos(\gamma)$, where $0 < f < \beta$, we obtain the following approximation (see Appendix B):

$$Z_s(x) \equiv \log \left( \frac{\exp(D_2 \cdot X^2) + f \exp(D_1 \cdot X^2)}{1 + f} \right)$$

(10b)

This approximation uses the fact that $f \ll 1$ and $|D_2 - D_1| \ll 1$, which are justified because the semidiurnal tide is dominant over the diurnal tide, as described in Appendix B. From this derivation, we estimate the following lowest-order value for the range coefficients:

$$d_k = (1 - f)d_{D_1} + f d_{D_2}.$$  

(11)

Consequently, range can also be predicted using a formula like (9a). We expect the $d_k$ to be between the coefficients of the major tidal species, but closer to $d_{D_1} = 0.12$. The influence of overtides causes some deviations from equation (10b), which do not, however, interfere with the analysis based on equation (9a).

5. Overtide Properties

[31] River flow effects on fluvial overtides are fundamentally different from effects on $D_1$ and $D_2$ because overtides are generated, as well as damped, due to the frictional energy transfer between frequencies. A $D_4$ model is derived in Appendix C. The results are summarized as follows:

$$D_4(x,t) = |D_4(x,t)| \exp[i(\omega_4t - \kappa_4x + \lambda)],$$

(12)

where $\omega_4 = 2\omega_2$, $\kappa_4 \approx 2\kappa_2$, $|D_4|$ is linear in $U_r|D_2|^2$ and $|D_2|^2$, and $\lambda$ is a response delay. The phase difference $\Delta \phi_4 = \arg(D_4(x,t)) - 2\arg(D_2(0,t))$ can be modeled in analogy to equation (9b) with

$$d'_{D_n} = 2d_{D_n}, \quad i, j = 1, 2.$$  

(13)

[32] Similarly, simple forced wave solutions like equation (12) can be obtained for other overtides, which are, however, not discussed here. In section 4, we introduce the methodology necessary to extract $D_1$, $D_2$, $D_4$, and $R$ properties from tidal height data.

4. Data Analysis Methods

[33] Tidal damping by fluctuating river flow renders tidal propagation a nonstationary process, requiring appropriate data analysis methods [Jay and Flinchem, 1997, 1999]. Also, quasi-stochastic forcing, due to dam-released high-frequency discharge waves, modifies the natural tidal frequency spectrum. The dilemma in analyzing nonstationary processes is the need to extract instantaneous information about frequencies, while the definition of “frequency” itself implies some time extent. Thus there is a trade-off between the length of the time window used to analyze data and the precision with which the filter can retrieve frequency information, as described by the Heisenberg uncertainty principle [Landau and Lifshitz, 1977]. The least squares fit, employed in harmonic analysis, responds inconsistently to nonstationary variance, when the short windows needed here are used [Jay and Flinchem, 1999]. For nonstationary data, a substantial fraction of the variance would be lost due to the assumption of constant phases and amplitudes at discrete frequencies. Further, the frequency response characteristics of a harmonic analysis are undefined in the sense that the response in any one frequency band depends on all other bands of the signal. To optimally extract tidal species properties for nonstationary tidal data, we employ wavelet filters [Flinchem and Jay, 2000]. We use a nonlinear filter that determines daily extrema to retrieve tidal range, with the resulting range estimates smoothed over a small number of wave cycles.

4.1. Continuous Wavelet Transform

[34] The continuous wavelet transform $\Psi$ is the convolution of a time series with a scaled wavelet. A wavelet is an oscillating function with zero mean and finite energy and duration [see e.g., Kaiser, 1994]. The scaling depends on the analysis period $s$ (equivalent to the analysis frequency $1/s$) and is characterized by the time dilatation of $1/s$. Like Jay and Flinchem [1997], we use a Kaiser-windowed complex exponential as our basis wavelet filter. The Kaiser window is employed because it minimizes energy leakage into sidelobes [Kaiser, 1966]. The wavelet $\Psi_L$ has the following form:

$$\Psi_L(t,s) = N_L(s)\sqrt{\frac{1 - \left(\frac{t}{L}\right)^2}{2\pi L^2}} \exp\left(2\pi i s t^2\right),$$

(14)

where $L_0$ is a zero order modified Bessel function of the first kind, $\beta = 6.755$ determines the frequency roll-off, and $L$ establishes the wavelet length relative to $s$, and $N_L(s)$ is chosen such that the maximal response to a unit wave is one. A wavelet transform $y_L$ is then defined by:

$$y_L(t,s) = [z^*\text{conj}(\Psi_L(t,s))](t),$$

(15)

where $*$ is the convolution operator, $\text{conj}(\cdot)$ is the complex conjugate of the argument, and $z$ is the shape elevation record. In conventional wavelet transforms, the length of the wavelet filter is proportional to the analysis timescale $s$, so that higher frequencies have a relatively short filter length with reduced frequency resolution. We have increased the filter length for frequencies higher than $D_2$ to improve the frequency resolution and reduce noise, at a small cost in temporal resolution. The filter length is selected, so that (1) it corresponds to the timescale of nonstationary processes (e.g., changes in river discharge) and (2) the filter responds primarily to particular tidal species. A filter that is too long yields a frequency resolution that is too narrow, along with poor time resolution. If a filter is too short, it has good time resolution, but poor frequency resolution; it samples multiple tidal species. Filter lengths of 84 h and 168 h were used for $D_2$ and $D_1$, respectively (equivalent to $7s$). For frequencies above $D_2$, the filter length is $20s$ long; for frequencies below $D_1$, the filter length is $5.6s$ long. These choices provide time resolution consonant with the time
variability of the river flow, and sufficient frequency resolution to separate tidal species.

4.2. Tidal Range Filter

Daily tidal range can be smaller than the daily low-frequency subtidal variation (due to changes in river discharge). If low-frequency components are not removed, a range filter could measure the subtidal variation rather than the variation due to ocean tides. Thus the stage record must be high-pass filtered, before analyzing tidal range:

\[ y(t) = [z * \text{filter}_{HP}](t), \]

where \( \text{filter}_{HP} \) is a high-pass filter. The filter length should eliminate as much as possible the subtidal signal without attenuating the tidal signal; a 74h filter was used. Nonlinear maximum and minimum filters can then be constructed to determine tidal range by:

\[ y_{\text{max}}(t) = \max_{j} (bw(t_j - t)y(t_j)), \]
\[ y_{\text{min}}(t) = \min_{j} (bw(t_j - t)y(t_j)), \]

where \( y(t_j) \) is the variable \( y \) sampled at point \( t_j \), and \( bw(t) \) is a square window with unit amplitude, centered at \( t = 0 \). The min-max filter must be slightly longer than a tidal period defined by the time from higher high water to the subsequent higher high water to capture the full daily tidal range. To be able to center the window, we chose an odd length of 27h. The smoothed tidal range \( R \) is retrieved by the operation

\[ R(t) = [(y_{\text{max}} - y_{\text{min}}) * \text{filter}_{LP}](t), \]

where \( \text{filter}_{LP} \) represents a 4-day low-pass filter. The final filtering produces a signal smoothed over the same time-scales as the wavelet filter for the \( D_2 \) tide. Hourly sampling does not perfectly capture the extrema to produce an ideal estimate of range, so that we have used more frequently sampled data where available.

4.3. Data

Hourly (or more frequent) tide gauge data recorded between 1980 and 2000 were available from 20 stations along the LCR (Figure 4 and Table 1) from the National Ocean Service (NOS), the U.S. Geological Survey, and the U.S. Weather Service. The record length for the stations varied between several months (e.g., Knappa) to >20 years.

<table>
<thead>
<tr>
<th>Station</th>
<th>Year of Record</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jetty A</td>
<td>1981</td>
</tr>
<tr>
<td>Ft. Stevens</td>
<td>1981</td>
</tr>
<tr>
<td>Knappo</td>
<td>1981</td>
</tr>
<tr>
<td>Astoria</td>
<td>1981 - 2000</td>
</tr>
<tr>
<td>Altoona</td>
<td>1981</td>
</tr>
<tr>
<td>Knappa</td>
<td>1981</td>
</tr>
<tr>
<td>Skamokawa</td>
<td>1981, 1997 - 2000</td>
</tr>
<tr>
<td>Cathlamet</td>
<td>1981</td>
</tr>
<tr>
<td>Wauna</td>
<td>1981</td>
</tr>
<tr>
<td>Beaver</td>
<td>1981, 1997 - 2000</td>
</tr>
<tr>
<td>Longview</td>
<td>1997 - 2000</td>
</tr>
<tr>
<td>Rainier</td>
<td>1981</td>
</tr>
<tr>
<td>Kalama</td>
<td>1981</td>
</tr>
<tr>
<td>Columbia City</td>
<td>1981</td>
</tr>
<tr>
<td>St. Helens</td>
<td>1999, 2000</td>
</tr>
<tr>
<td>Vancouver</td>
<td>1997 - 2000</td>
</tr>
<tr>
<td>Washougal</td>
<td>1981</td>
</tr>
<tr>
<td>Multnomah</td>
<td>1981</td>
</tr>
<tr>
<td>Warrendale</td>
<td>1981</td>
</tr>
</tbody>
</table>
Table 2. Tidal Constituents at Fort Stevens With Amplitudes Greater Than 0.05 m and Overtide Constituents M3, M4, M6, and M8

<table>
<thead>
<tr>
<th>Tidal Constituent</th>
<th>Amplitude, m</th>
<th>Phases, deg Relative to Greenwich</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSA</td>
<td>0.1062</td>
<td>107.29</td>
</tr>
<tr>
<td>O1</td>
<td>0.2578</td>
<td>121.05</td>
</tr>
<tr>
<td>P1</td>
<td>0.1235</td>
<td>127.36</td>
</tr>
<tr>
<td>K1</td>
<td>0.4127</td>
<td>128.45</td>
</tr>
<tr>
<td>N2</td>
<td>0.1890</td>
<td>359.13</td>
</tr>
<tr>
<td>M2</td>
<td>0.9352</td>
<td>19.07</td>
</tr>
<tr>
<td>S2</td>
<td>0.2397</td>
<td>38.26</td>
</tr>
<tr>
<td>K2</td>
<td>0.0694</td>
<td>36.47</td>
</tr>
<tr>
<td>M3</td>
<td>0.0033</td>
<td>279.56</td>
</tr>
<tr>
<td>M4</td>
<td>0.0253</td>
<td>127.21</td>
</tr>
<tr>
<td>M6</td>
<td>0.0060</td>
<td>76.92</td>
</tr>
<tr>
<td>M8</td>
<td>0.0006</td>
<td>295.57</td>
</tr>
</tbody>
</table>

(Astoria). We chose Ft. Stevens at rkm 13 as reference station for a number of reasons: (1) tides at this location are only weakly influenced by river flow (i.e., are nearly stationary), (2) the tide gauge is close to the main channel and as such measures the dominant tide which propagates into the river, and (3) the record was long enough to confidently retrieve harmonic constants. Ft. Stevens tidal data were not available for the entire 1980–2000 period. Incoming ocean tides were, therefore, predicted using harmonic constants from Ft. Stevens, derived by a harmonic analysis (Table 2). The dominant tidal constituent is M2, followed by K1; with amplitudes of 0.94 and 0.41 m, respectively. Overtide constituents are much smaller than the major diurnal and semidiurnal constituents. Figure 5 shows a typical scaleogram for this station. Most energy is in the semidiurnal and diurnal frequency bands, which vary quasiperiodically with the 2 weekly neap-spring cycle. The quarterdiurnal species has the third largest amplitude, however, all overides are significantly lower than the dominant tidal species.

[37] Daily flow values for Beaver (rkm 87) before 1991 were estimated using a routing formula that involves the main stem CR flow at Bonneville and flow from tributaries below rkm 234 [Orem, 1968]. On average, the flow at Bonneville is >75% of the flow at Beaver. Since 1991, daily flows have been measured at Beaver (rkm 87) by the U.S. Geological Survey. Because flow is a reasonable flow estimate for the analysis for all gauging stations, because the tidal wave interacts to lowest order with river flow at Beaver. There are several reasons why the daily river flow values remain uncertain; there is (1) unmeasured flow below Beaver of ~1–10% of Beaver flow, (2) random error in measured daily flow as high as 5%, especially at Beaver, where stage is tidally influenced, and (3) direct precipitation on the river below Beaver which may increase flow by ~5%. Random errors and uncertainties due to tributary inflows are reduced by smoothing river discharge over the timescales of the wavelet filters.

5. Results and Discussion

[38] Results presented here emphasize the predictions of D1, D2, and R amplitudes, because these are important for

Figure 5. (top) Scaleogram for tides at reference station Ft. Stevens. The horizontal lines indicate energy at a particular frequency. (bottom) Three most energetic frequency bands. The semidiurnal (D2) band is significantly larger in amplitude than the diurnal (D1) species. The quarterdiurnal species (D4) is even weaker than D1. Note the neap-spring cycle of ~2 weeks of all species.
the shallow-water habitat characteristics of interest in part 2. Our approach, however, can also predict \( D_1 \) and \( D_2 \) phases as well as overtide characteristics.

5.1. Modulation of the Oceanic Tidal Frequency Spectrum

Frictional dissipation causes estuarine tidal propagation to be a nonlinear phenomenon, steepening and distorting the tidal wave (Figure 6, second and third panels) [Parker, 1991]. First panel in Figure 6 shows a tidal height record at Astoria (km 29), a station less influenced by river flow than more landward stations. The sequence of panels shows tidal height records of stations with increasing upriver distance during a period in 1981 when most data records are available. The last panel in Figure 6 shows the least tidally influenced surface elevation record at Bonneville Dam (km 234), where water surface elevation is dominated by power peaking and river flow. The stations between Astoria and Bonneville in Figure 6 demonstrate intermediate properties between these extremes.

The generation of overides and damping control the evolution of the tidal frequency spectrum up to about Columbia City (km 135). The ocean tidal wave was accordingly modulated with upriver distance (Figures 6 and 7). The dominant overtide species was \( D_4 \), generated from \( D_2 \) by frictional interaction. The strength of \( D_4 \) relative to the sixth diurnal species at Columbia City suggests that quadratic interactions (related to \( P_2 \) and river flow) are more significant than cubic interactions. This is consistent with the behavior of \( P_2 \) and \( P_3 \) in equation (2b) for high discharge rates.

Landward of Columbia City (km 135), the form and evolution of the tidal spectrum change, suggesting additional physical processes are at work. In this region, a lesser degree of wave steepening and an increase in \( D_1 \) amplitudes suggest the superposition of seaward propagating downriver discharge waves from Bonneville Dam (Figure 6, second to fourth panels). Interference of tidal and discharge waves is, for example, evident on day 68 as far as downriver at Columbia City (Figure 7, second and fourth panels). Generally, the tide changed its character from nearly stationary and band-limited at Astoria (where horizontal lines in Figure 7 indicate wave processes) to nonstationary and broadband in the reach above Portland (km 170), where vertical patterns dominate the scaleogram.

Irregular power-peaking cycles at Bonneville Dam generate the event-like fluctuation in Figure 7; they are often larger than the tides for approximately 60 km downriver from the dam. At Bonneville, energy was mainly present at frequencies below two cycles per day; probably only the \( D_2 \) component is primarily tidal. Stochastic high-frequency discharge waves (frequencies greater than two cycles per day) were rapidly damped out and merely contribute to spectral background noise downriver from Washougal (km 190). Irregular waves with periods of 1–4 days traveled downstream and still had an excursion on the order of 0.05 m at Columbia City (Figure 7).

In summary, the spatial variations of LCR water level frequency spectra suggest that tidal energy input (modulated by frictional interactions) dominates the frequency spectrum from the estuary entrance to at least km 135. Further, the influence of discharge waves was weak seaward of Columbia City at km 135. Consistent with this qualitative assessment, tidal coefficients \( d_n, n = 0, 1, 2 \), from the estuary entrance up to Vancouver (km 171) displayed relatively little variability, compared to the variability of these for the reach landward of Vancouver.

5.2. Model Coefficients

We determined the three amplitude coefficients \( d_0 \), \( d_1 \), and \( d_2 \) from equation (9a) as function of \( x \) for the \( D_1 \) and \( D_2 \) amplitudes and tidal range. With knowledge of these
coefficients we are able to predict the dominant tidal amplitudes and range, which are relevant to part 2. The spatial variability of these coefficients also provides important information about tidal processes.

5.2.1. River Flow Coefficient \( d_1 \)

[46] Data analyses confirm the approximately linear relationship (9a) between river flow and log-normalized tidal amplitudes; equation (7) implies further that the damping modulus should be linearly dependent on discharge. Tidal damping also grows with upriver distance, since the damping modulus increases with \( x \).

[47] The Tschebyschev coefficients are constant where \( u_R \geq u_T \), so that equation (8a) provides a linear relationship between the \( d_1 \) and upriver location. Close to the estuary, the influence of coefficient \( p_2 \) is small, so that \( d_1 \) decreases only slowly for the first 50 km (Figures 3 and 8). With increasing upriver distance, the tidal influence weakens and \( p_2 \) steadily increases, causing \( d_1 \) to become more negative (Figure 8). At the point where the current does not reverse anymore (roughly landward of Beaver, km 87, for average flow conditions), \( p_2 = \pi \) and is constant thereafter. The interannual variation in the \( d_1 \) coefficient at Beaver is likely explained by the fact that currents reverse only during low-flow seasons, and current reversal is more frequent in low-flow years. From Beaver landward, the slope of \( d_1 \) with \( x \) is maximal and nearly constant, as expected from the form of \( p_2 \) in Figure 3.

[48] The drag coefficient \( C_D \) can be estimated from equation (8a) and the definition of \( C'_1(x) \) in Appendix A. The average calculated \( C_D \) for semidurnal amplitudes was \( 5.4 \times 10^{-3} \), with a standard deviation of \( 1.5 \times 10^{-3} \). For our determination of \( C_D \), we used all available data (~10 station-years) in the reach from km 100 to 175 where currents do not reverse, but tidal amplitudes are still large enough to allow such an analysis. Our estimated drag coefficient is somewhat larger than the value estimated by Giese and Jay [1989], \( C_D = 3 \times 10^{-3} \). The latter estimate was made, however, based on model performance in a more seaward reach from km 20 to 135. Our value may be greater, either because bed forms are larger farther upriver (increasing bed roughness), or because the Giese and Jay model did not include topographic convergence in the wave number, altering both wave propagation and damping.

[49] For the coefficient \( d_1 \), the regression analysis provided similar results for \( D_1 \) and \( D_2 \) amplitude, and tidal range (Figure 8). The coefficient for tidal range \( d_{1R} \) is between \( d_{1D_1} \) and \( d_{1D_2} \) as equation (11) suggests. Thus tidal range reflects the influence of both species. The ratio of \( D_2 \) to \( D_1 \) river flow coefficient should be approximately \( \sqrt{2} \) according to equation (8a) and Appendix A. This is because the damping modulus is proportional to the square root of tidal frequency, according to equation (5a). The ratio, however, is close to unity, perhaps because the complex interactions that damp the smaller \( D_1 \) species are not fully reflected in equation (7).

[50] The more complex damping of \( D_1 \) probably in part also causes greater variability in the calculated values of
amplitude is due to the quadratic bed stress term and causes a more rapid decrease in amplitude for larger incoming tidal ranges [Jay et al., 1990; Godin, 1991]. Although the ocean tides in the $d_2$ term of the regression model (9a) should in principle be modeled by half the tidal range, the best results for $D_2$ were achieved by modeling incoming ocean tides with only the semidiurnal ocean amplitude $D_2(0)$. This is plausible, considering the dominant character of the semidiurnal wave. The coefficient $d_2$ decreased up to roughly Beaver (rkm 87) and thereafter varied about a constant value, with a relative standard deviation of $\sim 30\%$ (Figure 9).

The behavior of $d_1$ with upriver distance further suggests the division of the system into four regimes. In the reach seaward of rkm 50, the slope of $d_1(x)$ is nearly constant. The slope changes in a second reach between rkm 50 and 90 to another nearly constant value. This value is smaller (more negative, corresponding to a larger value of $c_D$) than the value from the first regime, and defines the third regime roughly from rkm 90 to 175. Further upriver, in the fourth regime, calculated $d_1$ is erratic, indicating that tidal influence is weaker than power peaking from Bonneville Dam. This fourfold division modifies the three-regime description of the channel defined by Jay et al. [1990]. Our first regime includes the tidally dominated and dissipation-minimum region, while the transition zone begins at the landward end of the dissipation-minimum region. The fluvial energy region as defined by Jay et al. [1990] is here divided into ocean-tidal and dam-wave influence regimes.

5.2.2. Neap-Spring Coefficient $d_2$

The neap-spring variation of the normalized tidal amplitude is due to the quadratic bedstress term and causes a more rapid decrease in amplitude for larger incoming tidal ranges [Jay et al., 1990; Godin, 1991]. Although the ocean tides in the $d_2$ term of the regression model (9a) should in principle be modeled by half the tidal range, the best results for $D_2$ were achieved by modeling incoming ocean tides with only the semidiurnal ocean amplitude $D_2(0)$. This is plausible, considering the dominant character of the semidiurnal wave. The coefficient $d_2$ decreased up to roughly Beaver (rkm 87) and thereafter varied about a constant value, with a relative standard deviation of $\sim 30\%$ (Figure 9).

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Therefore small uncertainties in $d_0$ do not significantly affect model behavior. Since the simplifications involved in equations (7) - (9b). Since the model is, however, robust against uncertainties at high-river discharge because tidal range is weaker further upriver and for larger discharge. Thus tidal range is small and the absolute errors remain small.

5.2.3. **Geometry Coefficient $d_0$**

[54] The coefficient $d_0$ represents geometry properties in equation (8c) through $\log(A(0)/A(x))$, which describes the topographic funneling. Because friction dominates wave propagation, amplitude variations related to topographic funneling do not obey Green’s Law [Jay, 1991]. For exponentially convergent geometry, $d_0$ increases linearly with increasing $x$. For constant cross section, roughly the case landward of rkm 60 [Giese and Jay, 1989], $d_0$ should converge to a constant value. The somewhat erratic results for $d_0$ suggest random error or small-scale spatial variability (Figure 10). Changes in $d_0$ may also be due to the presence of intertidal areas (not considered in our model) seaward of rkm 60, the dependence of cross section on river stage, uncertainties in river flow values, and perhaps also the simplifications involved in equations (7) - (9b). Since the cross-sectional area decreases by no more than a factor of 2 landward of rkm 50, $d_0$ should be positive and small. Taking $d_0 = 0.15 \approx 1/2\log(4/3)$ (compare also to equation (8c)) for both $D_1$ and $D_2$ is consistent with our analysis results, considering that the standard deviation of $d_0D_1$ is $\sim 0.2$. This value is close to zero and enters exponentially the model. Therefore small uncertainties in $d_0$ do not significantly affect model behavior.

5.3. **Reconstruction of $D_2$, $D_1$, and $R$ Amplitudes**

[55] The modeled and observed tidal amplitudes are generally in agreement for stations from Jetty A (rkm 5) to Vancouver (rkm 171) (Table 3 and Figure 11). Using the coefficients calculated for data from each year and station (the "specific coefficients"), the root mean square (rms) relative error is $< 3.5\%$ for $D_2$ and tidal range $R$, but higher for $D_1$ (\sim 9\%). If tidal amplitudes are reconstructed using the coefficients taken from the fitted curves (Figures 8 - 10), hereafter referred to as the “universal coefficients,” the average error increased to 9, 8, and 17\% for $D_2$, $R$, and $D_1$, respectively. Absolute errors are likely more important than relative errors for evaluating tide predictions; they do not become unbounded when tides are small. The rms absolute errors for the specific coefficients are all between 25 and 30 mm, but decrease during high-flow periods, as tidal amplitudes decrease. As with the relative errors, use of the universal coefficients slightly more than doubles the errors. Although use of the universal coefficients causes somewhat larger errors in hindcasts, the curves from which they are defined allow prediction for any combination of flow and tidal input at any point seaward of Vancouver. Averaged over all stations, the modeled amplitudes are underestimated by 0.9, 3.0, and 0.3\% for $D_2$, $D_1$, and $R$, respectively, which is an acceptable bias considering the relative modeling error. It is likely that a bias arises from the regression analysis in log space. This interpretation is supported by the fact that $D_1$ has both the largest bias and the largest rms error [see the work of Kukulka, 2002].

[56] These relatively small error ranges validate the usage of the model for hindcasting historic conditions employing the universal coefficients. As discussed in the previous section, the relatively large $D_1$ error is not a major issue for predicting $R$ (in part 2), because $D_1$ is a smaller contribution to the tidal range than $D_2$; errors for $R$ are comparable to those for $D_2$. Neap-spring variations are well resolved for $D_2$ and $R$. Uncertainties in river discharge and

![Figure 9. The neap-spring coefficient $d_2$ as a function of upriver distance for semi-diurnal, diurnal, and tidal range amplitude (solid dots). The curve shows a linear regression for the reaches from rkm 5 to 50 and the average from rkm 50 to 171. In the transition region, the linear curves are connected with a cubic spline. The tidal amplitude $z_0$ was scaled by 1 m and not by $h$ (compare to equation (9)).](image-url)
Figure 10. The geometry coefficient $d_0$ as a function of upriver distance for semidiurnal, diurnal, and tidal range amplitude (solid dots). The line shows the average of the coefficients for rkm 5 to 171.

Variations in channel cross section with flow do not appear to significantly affect the model accuracy. One reason why flow-related variations in cross-sectional area are of little importance is the presence of flood control dikes. These dikes prevent significant overbank flow for flows <24,000 m$^3$ s$^{-1}$, stabilizing the width of the river. Flows exceeded this level only twice during the 1970–2000 period, for a total of less than a week. Hindcasts for earlier periods, however, might be affected by width variation not accounted for by 1980–2000 data.

It is also useful to compare the results achieved here with those of previous studies. Our approach of objectively fitting coefficients for each tidal species provides better results than obtained from the semianalytical model of Giese and Jay [1989]. In this model $c_D$ was the only parameter, and it varied systematically with $Q_R$. That model's $D_2$ amplitude prediction error was ~5% for stations seaward of rkm 90, and greater errors were found for tidal height predictions further landward. The largest improvement is made for $D_2$ amplitudes far upriver and during periods with high river discharge. Thus model results can be improved through an objective, data-driven approach to the representation of frictional energy. This confirms that the form of the bedstress representation (including effects of river discharge and neap-spring variability) is crucial to achieving accurate predictions.

The method is also more accurate and compact than conventional harmonic predictions. Using harmonic analysis, the rms error for semidiurnal amplitudes averaged over all station-years was 83 mm (compare to 28 mm), the average rms error for the diurnal amplitude was 56 mm (compare to 25 mm). Generally, harmonic predictions function poorly when $Q_R$ is large or changes abruptly (Figure 12). To predict the semidiurnal amplitude of the 300-day record at Columbia City, 14 constituents were included by the harmonic analysis, translating to 28 coefficients (two per constituent). The addition of further constituents corresponding to numerous, small nonlinear interactions would not yield improved prediction power for nonstationary flows [Godin, 1991]. Our method uses at each location only river flow, incoming ocean tides, and three dynamically meaningful coefficients per species.

A major advantage of our method is that we can interpret the spatial distribution of the model coefficients, which is modeled by the universal coefficients. This allows predictions of $D_1$, $D_2$, and tidal range amplitude at arbitrary upriver locations. This approach has also been applied to two stations in the Fraser River, with comparable results to the CR (not shown). This suggests that our tidal model can be applied to a variety of tidal rivers with strong fluvial forcing.

5.4. Phases

Our method can also provide robust hindcasts of phases and overtide properties. The analysis of phases is limited to data records with consistent time control. Unlike elevation errors, timing errors are often not obvious from inspection of a tidal record, but have a major impact on the phase analysis. For example, a timing error of 1 hour, as frequently occurs during the transition from daylight standard time to daylight saving time, can cause significant errors in the phase predictions.

### Table 3. Average RMS Error for all Stations From Jetty A to Vancouver

<table>
<thead>
<tr>
<th>Tidal Amplitude</th>
<th>Error for Specific Coefficients</th>
<th>Error for Universal Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Absolute, mm</td>
<td>Relative, %</td>
</tr>
<tr>
<td>$D_2$</td>
<td>28</td>
<td>3.9</td>
</tr>
<tr>
<td>$D_1$</td>
<td>25</td>
<td>9.2</td>
</tr>
<tr>
<td>$R$</td>
<td>30</td>
<td>3.4</td>
</tr>
</tbody>
</table>
ard time in the fall, introduces a temporary 60° phase error in $D_4$. Such an error is large enough to obscure the dynamical signal we seek.

The 1981 NOS tidal records provide a data set with both a substantial number of stations and a consistent time control. Though the smaller number of station-years limits statistical certainty in determination of the $d_i$ the resulting $d_{i,D}$ and the $d_{i,D}$ patterns are consistent with our theoretical development (Figure 13). The coefficients $d_{i,D}$ are greater than $d_{i,D}$ in agreement with equation (5a), because the wave number is proportional to $\omega^{1/2}$. As suggested by equations (5a) and (7), the magnitudes of the $d_i$ are close to those of the $d_i$ for $i = 1, 2$, but phase and amplitude coefficients have opposite signs. As with $d_i$, the phase coefficients $d_{i,D}$ increase in magnitude with upriver distance. The relative errors of reconstructed phases are generally similar to the relative errors of $D_1$ and $D_2$ amplitudes (Figure 14). The $D_1$ phase error landward of ~rk3 135, however, increased significantly. This could be because nonlinear interactions among tidal species have not been considered in our analysis. In addition, diurnal discharge waves from Bonneville Dam interferes further upriver with the $D_1$ wave. Finally, phase predictions become meaningless as tidal amplitudes approach zero at upriver stations during high-flow periods.

5.5. Overtides: $D_4$

The first overtide ($D_4$) of the $D_2$ wave is considered here as representative of the problem of modeling overides. Using the theoretical development from section 3.5, we are able to predict $D_4$ with an average rms error of <1 mm, which translates to a relative error of 12%. Model results for Altoona, Beaver, Columbia City, and Vancouver are shown in Figure 11b. Because of the nonstationary behavior of river tides, this degree of predictability could not have been achieved with conventional harmonic tidal analysis.

The results for the phase coefficients of the $D_4$ wave are summarized in Table 4, showing the ratio of the phase coefficients where $d_{i,D}/d_{i,D}$, where $i = 1, 2$. Where phase differences are very small (close to the reference station at rkm 13) the ratio is sensitive to the small denominator, and results are erratic. The negative numbers, however, are probably due to the influence of the incoming ocean $D_4$ wave up to ~rk3 50. As noted by Jay and Musiak [1996], the $D_4$ wave undergoes an abrupt 180° phase change at about rkm 35 as the forced wave becomes dominant over the free wave of oceanic origin. After the incident $D_4$ wave is damped and the fluvial forcing becomes dominant, the ratio $d_{i,D}/d_{i,D}$ is $O(2)$, as predicted by equation (13). The mean ratio $d_{i,D}/d_{i,D}$ of the phase neap-spring coefficient landward from rkm 53 is 1.6 with a standard deviation of 1.0. The greater standard deviation of the neap-spring ratio is due to the variability of the neap-spring coefficients, as discussed in section 5.2.2. That $D_4$ phase and amplitude can be successfully modeled suggests that the method employed herein can be applied to other major overtides to complete our new approach to nonstationary tidal analysis and hindcasting.

6. Summary and Conclusions

We have developed a new method for modeling and hindcasting nonstationary tides. By applying wavelet tidal
Figure 11b. Observed (solid) and modeled (shaded) tidal amplitudes (in meters) for \( R \) and \( D_4 \) at the stations (a) Altoona, (b) Beaver, and (c) Columbia City in 1981, and (d) Vancouver in 1997. The \( x \) axes show days from 1 January.

Figure 12. Observed semidiurnal tidal amplitude at Columbia City compared to predictions (top) from HA and from equation (9a) and (bottom) river discharge.
Figure 13. The flow coefficients for the $D_2$ and $D_1$ tidal wave number determined for the year 1981.

Figure 14. Observed (solid) and modeled (shaded) $D_2$ and $D_1$ tidal phase differences (in degrees) between reference station at Ft. Stevens and stations at (a) Altoona, (b) Beaver, and (c) Columbia City in 1981. The x axes show days from 1 January.
analysis methods, an analytical fluvial tide model, and an objective determination of model coefficients to the LCR, we have compactly defined the interactions of tides and river flow in the LCR. In part 2, these results will be used to assess historical changes of salmonid-favorable shallow-water habitat.

[65] The model is based on an analytical solution for incident tidal waves in frictional, convergent channels [Jay, 1991]. We have decomposed the bedstress, such that (1) the effects of variable river flow and incoming ocean tides are represented in a manner consistent with the underlying physics, and (2) the model coefficients can be determined from tidal height data by linear regression analysis. From ∼50 station-years of surface elevation records for the LCR, we extracted D1, D2, and D3 amplitudes and phases by CWT methods [Flinchem and Jay, 2000]. Smoothed tidal range was retrieved by a min-max filter. Tidal species phase and amplitude for each station can be predicted with only six, dynamically meaningful, coefficients, river flow, and incoming ocean tides. A spatial interpretation of the three coefficients for the diurnal and semidiurnal tidal amplitudes allowed describing each coefficient as simple function of upriver location. Thus each dominant species amplitude was parameterized with three simple spatial curves throughout the whole river channel. This description of nonstationary river tides represents a new level of compactness. Further, we showed that this compactness does not compromise prediction accuracy. In reconstructing tidal amplitudes, the rms model prediction error was <30 mm for R, D2, and D3 amplitude, which is significantly more accurate than predictions obtained from harmonic analysis. One of the model’s strengths is the rapid, accurate tidal hindcasts for a very broad range of river discharge. To the extent that future river flow is known, the model can be used as a tidal prediction tool. In conclusion, we have modeled nonstationary fluvial tidal properties with a previously unattained level of compactness and a functional level of accuracy.

Appendix A: Simplifications for Regression Model in Section 3.3

[66] For the following development, it is assumed that p2 ≫ p3 and p2 ≫ p1, so that (p1/p2) uR ≪ 1 and p3/p2 (3uR + 0.5uR²/ur) ≪ 1. This assumption is valid roughly landward of rkm 60, where ur is of the same order of magnitude of uR or larger. Even seaward of rkm 60 where these assumptions do not hold, an analysis based on these assumptions still provides practically useful results.

\[
\frac{1}{c_0} \left( \frac{c_p U_0 \omega}{2H \pi} \right) \left( \sqrt{p_1 + p_2 u_R + p_3 \left( 3u_R^2 + \frac{1}{2} u_R^2 \right)} \right)
\]

\[
\approx \frac{1}{c_0} \left( \frac{c_p U_0 \omega}{2H \pi} \right) \sqrt{p_2 u_R \left( 1 + \frac{p_3}{p_2} \left( 3u_R^2 + \frac{1}{2} u_R^2 \right) \right)}^{1/2}
\]

\[
\approx \frac{1}{c_0} \left( \frac{c_p U_0 \omega}{2H \pi} \right) \sqrt{p_2 u_R \left( \frac{3p_3}{2p_2} + \frac{u_R^2}{4p_2} \right) \sqrt{u_R} + \frac{u_R^2}{2\sqrt{p_2} \sqrt{u_R}}}
\]

\[
\approx c_1 u_R + c_2 \frac{u_R^2}{\sqrt{u_R}}
\]

with

\[
c_1 = \frac{1}{c_0} \left( \frac{c_p U_0 \omega}{2H \pi} \right) \frac{p_3}{2\sqrt{p_2} \sqrt{u_R}}\frac{1}{\sqrt{u_R}}
\]

\[
c_2 = \frac{1}{c_0} \left( \frac{c_p U_0 \omega}{2H \pi} \right) \frac{p_3}{2\sqrt{p_2} \sqrt{u_R}}\frac{1}{\sqrt{u_R}}
\]

Appendix B: Development of First-Order Relationship for ZR in Section 3.4

[67] For the development, the following assumptions are used:


[69] This is justified, because f = \beta \cos(\gamma) with \beta = D_1(0) D_2(0) < 1; since D_2 is the dominant species (see section 2.2) and 1 ≥ |\cos(\gamma)|.

[70] Assumption 2: |d_{D_1} - d_{D_2}| ≪ 1, so that \{d_{D_1} - d_{D_2}\} \cdot X^I ≪ 1.

[71] This is suggested by the results shown in section 5. The theory also motivates this relationship considering: d_{D_0} and d_{D_2} (equation (15)), d_{D_0} and d_{D_2}, i = 1, 2, are only distinguished by the frequency term \omega^{1/2} involved in c_1 (see Appendix A).

\[
Z_R(x) \equiv \log \left( \frac{\exp(d_{D_1} \cdot X^1) + f \exp(d_{D_2} \cdot X^I)}{1 + f} \right) \tag{B1}
\]

\[
= \log \left( \frac{\exp(d_{D_1} \cdot X^1) + f \exp(\{d_{D_1} - d_{D_2}\} \cdot X^I)}{1 + f} \right) - \log(1 + f) \tag{B2}
\]

\[
\approx d_{D_1} \cdot X^1 + \log(1 + f) \exp(\{d_{D_1} - d_{D_2}\} \cdot X^I) - f \tag{B3}
\]

\[
\approx d_{D_1} \cdot X^1 + f \log(1 + f) \exp(\{d_{D_1} - d_{D_2}\} \cdot X^I) - f \tag{B4}
\]

\[
\approx d_{D_2} \cdot X^I + f(1 + \{d_{D_1} - d_{D_2}\} \cdot X^I) - f \tag{B5}
\]

\[
\approx (1 - f) d_{D_2} + f d_{D_2} \cdot X^I \tag{B6}
\]
Appendix C: Derivation of a $D_4$ Model

[72] River flow effects on fluvial overtones are fundamentally different from effects on $D_1$ and $D_2$ because overtones are generated, as well as damped, due to the frictional energy transfer between frequencies. The wave equation (equivalent to equation (3)) for overtones is an inhomogeneous differential equation because of the bed stress forcing term $T_{4,\text{forcing}}$:

$$T_{4,\text{forcing}} = \frac{bc D}{gh^2} (p_2 + p_3 u_h)Q_2^2,$$  \hspace{1cm} \text{(C1)}

where the subscripts “2” indicates a semi-diurnal component and “4” a quartetidal one. The semi-diurnal transport can be specified as:

$$Q_2(x,t) = |Q_2| \exp(i[\omega_2 t - \varphi_2(x)]).$$  \hspace{1cm} \text{(C2)}

Now the forcing term $\Phi(x,t)$ due to the semi-diurnal wave in the wave equation (3) is:

$$\Phi(x,t) = \frac{\partial}{\partial t} T_{4,\text{forcing}} = iCQ_2^2,$$

with

$$C = 2\omega_2 \frac{bc D}{gh^2} (p_2 + p_3 u_h).$$  \hspace{1cm} \text{(C3)}

Using the same assumptions as for the $D_1$ and $D_2$ waves and the same linearization procedure of the bed stress (leading to the friction factor $F$) yields the following $D_4$ wave equation:

$$\frac{\partial^2 Q_4}{\partial x^2} - \frac{1}{U_b^2} \frac{\partial Q_4}{\partial x} - 2 \frac{1}{U_b^2} \frac{\partial^2 Q_4}{\partial x \partial t} + \frac{1}{U_b^2} \frac{\partial Q_4}{\partial t} + \frac{1}{U_b^2} \frac{\partial^2 Q_4}{\partial t^2} + \frac{F}{gh^2} \frac{\partial Q_4}{\partial t} = \Phi(x,t).$$  \hspace{1cm} \text{(C4)}

Note that equation (C4) has the same terms as the wave equation for a dominant tidal species, besides the forcing term $\Phi$.

[73] Further upstream, where the incident oceanic quartetidal wave ($D_4$) has lost most of its energy, the $D_4$ wave solution to equation (3) is a forced wave with its amplitude also linear in $U_b[|D_2|^2 + |D_3|^2]$, and oscillating with $D_2^3/|D_2| \exp(i\lambda)$, where $\lambda$ is a phase delay relative to $D_2$ forcing. Sufficiently far upriver, we have:

$$D_4(x,t) \approx |D_4(x,t)| \exp(i[\omega_4 t - \kappa_4 x + \lambda]),$$  \hspace{1cm} \text{(C5)}

where $\omega_4 = 2\omega_2$.

$$\kappa_4 \approx 2k_2,$$  \hspace{1cm} \text{(C6)}

$|D_4|$ is linear in $U_b[|D_2|^2 + |D_3|^2]$. Note that if the response delay $\lambda$ is nearly independent of river flow and tidal range, the phase of $D_4$ may be modeled by analogy to equation (9b). Then the phase difference $\Delta \varphi_4 = \arg(D_4(x,t)) - 2\arg(D_2(0,t))$ should lead to flow and neap-spring coefficients, so that

$$d_i d_i = 2d_i d_i, \quad i = 1,2.$$

[74] Forced wave solutions similar to equation (C5) can be obtained for other overtones, which are, however, not discussed here.

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