Appendix H
Engineered Protection Design Analysis for Engineered Caps and Area-Specific Technology

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ABBREVIATIONS

BODR	*Basis of Design Report*
AIS	Automatic Identification System
ENR	enhanced natural recovery
ENR/AC	Enhanced Natural Recovery/Activated Carbon
EPA	U.S. Environmental Protection Agency
ft/sec	feet per second
FS	Feasibility Study
FNC	federal navigation channel
H:V	horizontal to vertical (ratio)
LAT	lowest astronomical tide
LDW	Lower Duwamish Waterway
MHHW	mean higher high water
MLLW	mean lower low water
mph	miles per hour
MSL	mean sea level
RAA	remedial action area
RD	remedial design
ROD	*Record of Decision*
SLR	sea level rise

1 Introduction

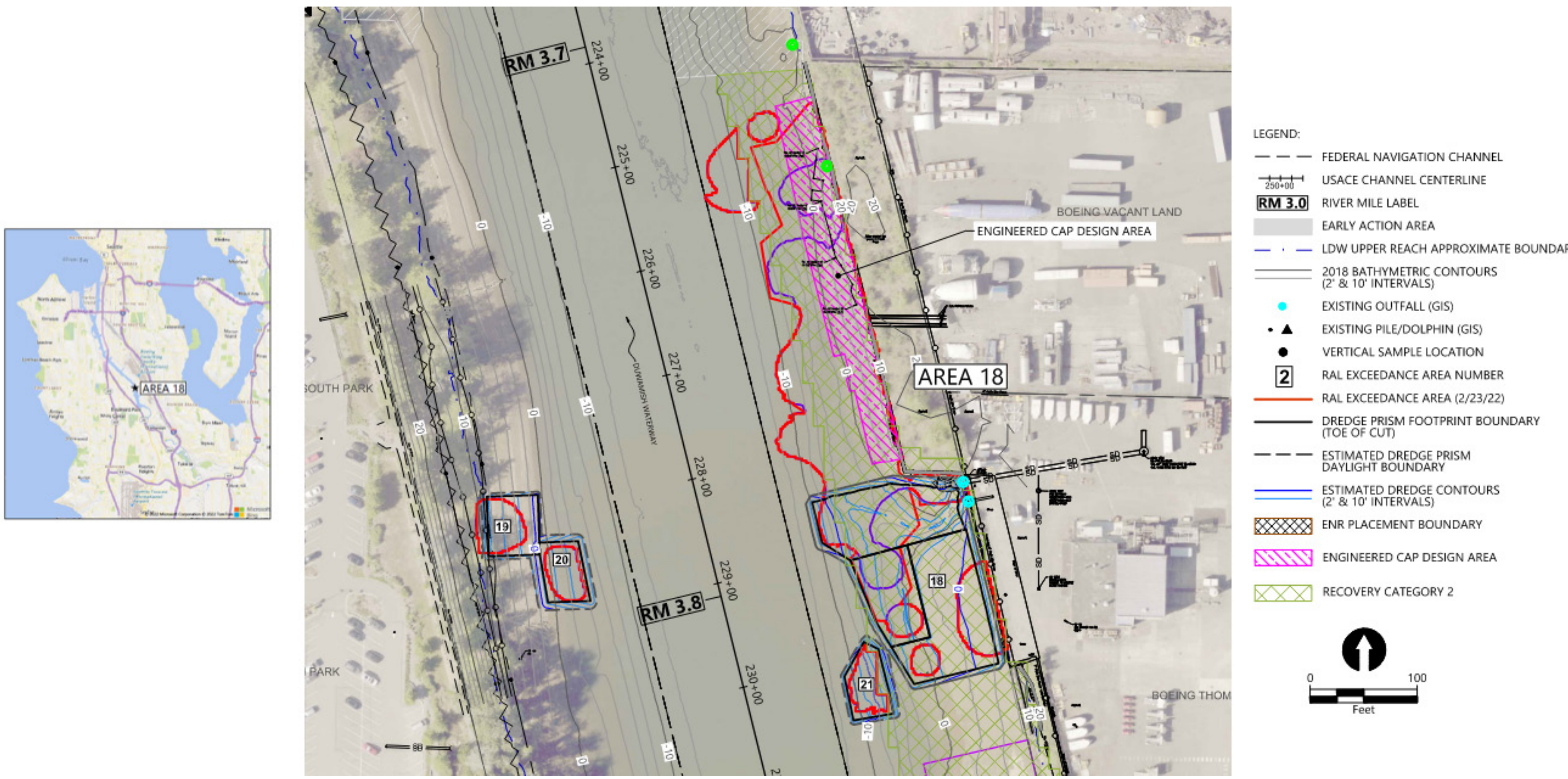
This appendix describes the design of erosion protection for a representative example engineered cap (using remedial action area [RAA] 18 as the example cap design area). RAA 18 is located in the upper reach of the Lower Duwamish Waterway (LDW) between river miles 3.7 and 3.8 between the federal navigation channel (FNC) and the bulkhead along the eastern bank of the LDW (Figure H-1). Cap contaminant mobility assessments and example design are described in Appendix G and summarized in Section 10.3.1 of the Intermediate (60%) Remedial Design (RD) *Basis of Design Report* (BODR).

Within the upper reach, capping is a technology that can be assigned in certain areas with deep contamination and compatible final surface elevations, in accordance with the *Record of Decision* (ROD; EPA 2014). As discussed in Section 6.1.3 of the BODR, the preferred approach to the northern portion of RAA 18 is to defer in-water remedial action; however, the engineered capping erosion protection analysis is presented in this appendix as representative design for engineered capping within intertidal areas with a similar range of physical conditions.

As discussed in Section 2.2 of the BODR, 60% RD identified that the sediment cleanup remedy at RAA 27 (Container Properties; RM 4.1E) extends up the adjacent bank. The adjacent bank consists of debris and manufactured materials that help protect the bank slope from potential erosive forces, but the bank at RAA 27 is not an engineered slope. Chemistry data underneath the bank debris and armoring materials could not be collected; however, chemistry data at the toe of the bank slope indicate potential for contaminated sediment underneath at least part of the bank surface. Because there is uncertainty whether sediment underneath the bank debris and armor material at RAA 27 is contaminated, the conservative remedial technology of engineered capping will be applied to the bank portion of this RAA. Therefore, erosion protection design for the bank area at RAA 27 is assumed necessary. The same methodologies used for example cap design erosion protection (outlined in Sections 2.1 through 2.7) were also used for the erosion protection cap design for the bank areas of RAA 27 and are summarized in Section 2.8.

The erosion protection analyses also describe the methods that will be used for erosion protection design for other remedial technology applications (e.g., area-specific technologies). Area-specific technologies consisting of cover material over structural offset areas are identified for portions of RAAs 18, 24, and 26. These are areas adjacent to existing structures or armored slopes and where dredging offsets are required because dredging may cause structural instability of existing structures or armored banks. No dredging will occur within the dredging offset areas, and area-specific technology will be applied to these small limited areas..

Figure H-1
Vicinity Map: Example Cap Design Area (RAA 18)



Note:
Cross section A-A′ is presented in Figure H-4.

The primary objective of the erosion protection layer in an engineered cap is to prevent exposure and erosion of the underlying chemical isolation layer. The potential for erosion of the sediment cap depends on the erosive processes likely to occur in the LDW, as well as the materials comprising the cap layers. Potential erosive processes that may act on the sediment cap within the upper reach of the LDW include the following:

- Localized propwash from vessels
- Waves generated by passing vessels (wakes)
- Hydrodynamic flows in the LDW resulting from discharge of tributaries and other discharges, as well as from typical river circulation conditions
- Wind-generated waves due to storm events

Each of these potential erosion processes was evaluated independently to determine the design requirements for the cap erosion protection component. The cap erosion protection layer was then designed to withstand erosion under the range of anticipated conditions for each process. This appendix presents the results of this design analysis.

While the analysis in this appendix focuses on RAA 18 as an example, the methodology for the erosion protection layer design is appropriate for other areas of the upper reach where capping may be the technology applied.

As described in Palermo et al. (1998):

> The cap component for stabilization/erosion protection has a dual function. On the one hand, this component of the cap is intended to stabilize the contaminated sediments being capped and prevent them from being resuspended and transported offsite. The other function of this component is to make the cap itself resistant to erosion. These functions may be accomplished by a single component, or may require two separate components in an in-situ cap.

Methods for designing cap erosion protection (i.e., armor layer) are presented in Appendix A of Palermo et al. (1998). The cap armor material gradation and thickness must also be designed to stabilize and protect the underlying physical and chemical isolation layers from erosion (based on an evaluation of each potential erosional source). The erosion resistance design must account for the forces along the edge of the cap as well as on the surface of the cap to prevent scour for both typical flows and anticipated flood events.

The armor layer of the cap has been designed to provide stabilization of underlying finer grained cap materials (as well as sediment) to prevent the vertical migration of those materials through the armor layer, termed piping (Palermo et al. 1998). As described in the RD Work Plan, the cap design considers the physical, chemical, hydrodynamic, and hydrogeological properties (LDWG 2019).

Climate change is expected to affect the greater Puget Sound region and, relevant to the LDW, includes sea level rise (SLR), changes in precipitation patterns, and overall hydrological changes. Climate change adaptation generally focuses on evaluating a system's vulnerability to climate change and implementing adaptation measures, when warranted, to ensure the remedy continues to remain effective at meeting the ROD objectives (EPA 2014). As such, an evaluation of the long-term effects of SLR and climate change on cap integrity is also discussed in this appendix.

2 Erosion Protection Design for Example and RAA 27 Engineered Caps

This section presents an evaluation of the following design criteria as related to erosive forces in the vicinity of example cap design area (RAA 18). This same evaluation methodology is then applied to RAA 27 to develop the erosion protection design at that location:

- Selection of design vessels
- Review of bathymetry, water levels, and potential changes due to SLR
- Predictive modeling to estimate stable particle size for propwash forces, wake forces, hydrodynamic forces, and wind-generated waves

2.1 Selection of Design Vessels

A propwash and vessel wake analysis was conducted to evaluate the stable particle sizes to resist propwash from vessels in the upper reach. Propwash and wake forces are related to specific characteristics of the vessel being considered, including vessel size, vessel power, vessel propeller size, operational speeds, and depth of the propeller beneath the water line. As such, a “design” vessel or vessels must be selected so that propwash and wake forces can be estimated. Vessel traffic data were obtained through the Automatic Identification System (AIS). The AIS vessel data are collected by the U.S. Coast Guard through onboard navigation safety devices that transmit and monitor vessel locations and characteristics of large vessels. These data were downloaded via MarineCadastre.gov (BOEM and NOAA 2021).

The design vessel selection consisted of the following components:

- Vessel activity was evaluated to establish the types and sizes of vessels that utilize the upper reach.
- Vessel characteristics (e.g., draft, propeller type, dimensions) were obtained for representative vessels; outlined below.
- Vessel operating information and assumptions (e.g., operating horsepower and vessel location and orientation) were selected to correspond with each representative vessel.

The available AIS data for 2020 were plotted. A portion of the data, from October 2020, is presented in Figures H-2 and H-3, showing AIS designated vessel types and vessel speeds, respectively. A total of 87 unique vessels were identified that transited near the example cap design area during the year. Of those unique vessels, three representative design vessels were selected for analysis:

- Capt. Cae Tug
  - The largest tug to transit the area in 2020 (92 feet long)

- Westrac II Tug
  - An average-sized tug (74 feet long), selected to represent the more typical tugs that frequent the area; the average length for tugs that transited the area in 2020 was 72 feet
- Arctic Pride Yacht
  - One of the largest pleasure vessels to transit the area in 2020 at 126 feet long; there were three larger vessels (up to 150 feet long), but Arctic Pride transited more frequently

Table H-1 outlines the specifications of the three design vessels used in the erosion protection basis of design.

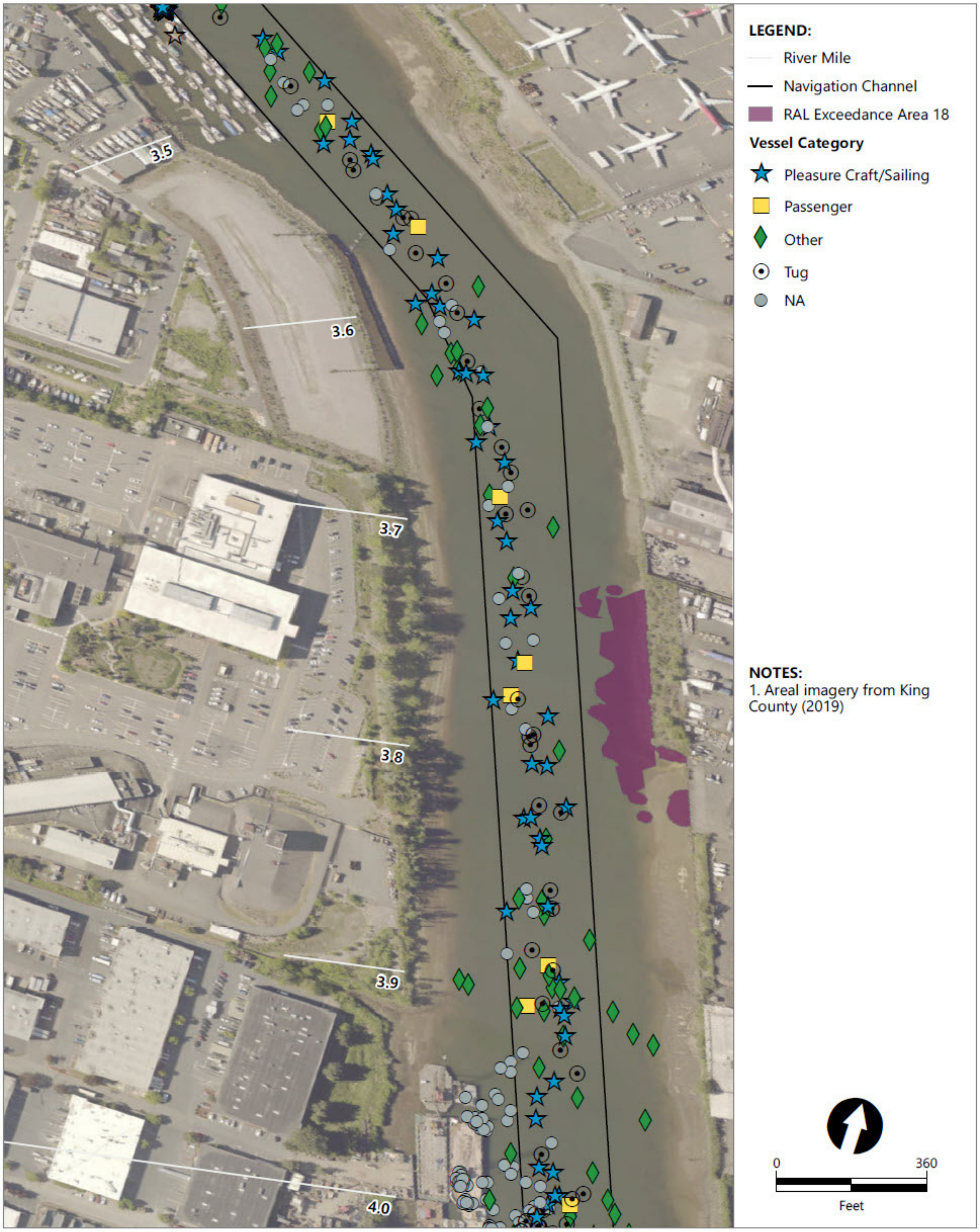
Table H-1
Design Vessel Specifications

Vessel Characteristic	Capt. Cae Tug	Westrac II Tug	Arctic Pride Yacht
Owner/operator	DeForge Maritime Towing	Western Towboat Company	Private Recreational Vessel
Length	92 feet	74 feet	126 feet
Draft	11 feet	14 feet	6 feet
Propeller diameter	7.25 feet	6.3 feet	4 feet
Horsepower per propeller[1]	1400 hp	1250 hp	1250 hp
Operational speed	4 to 8 knots	4 to 8 knots	4 to 8 knots

Notes:
1. This characteristic is used because the propwash analysis utilizes a single propeller.
hp: horsepower

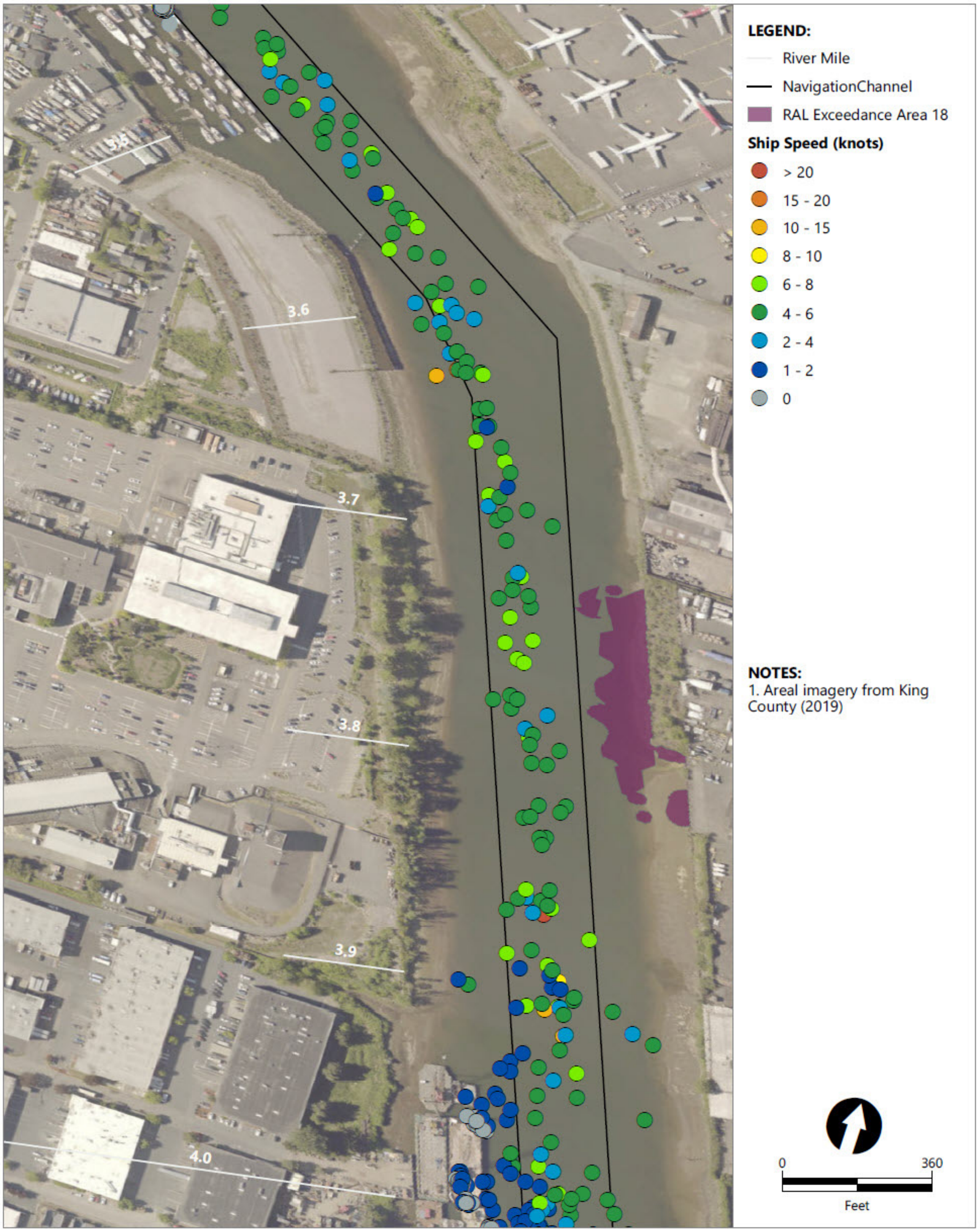
The Feasibility Study (FS) propeller-induced riverbed scour analysis (Appendix C, Part 7 of the FS), used the J. T. Quigg tug, with a length of 100 feet, for the evaluation near the example cap design area. This vessel is similar to the Capt. Cae Tug, with similar specifications. Because the FS analysis was performed in 2009, the design vessels were updated to reflect more recent usage data.

Figure H-2
AIS Vessel Categories: October 2020



Note:
Categories are based on AIS Ship Types.

Figure H-3
Vessel Speeds: October 2020



2.2 Bathymetry, Water Levels, and Sea Level Rise Impact

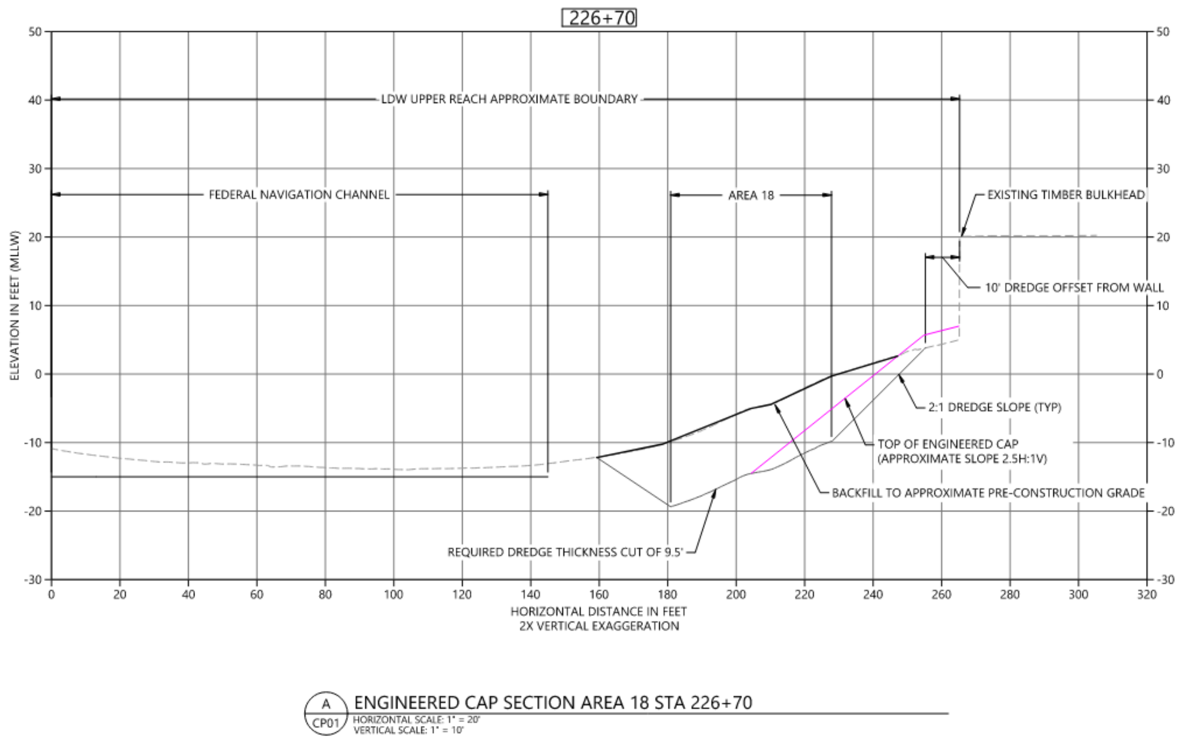
Erosion protection layer stability under vessel propwash and wakes is dependent on the configuration of the navigation channel and the water depths in which the vessels are operating. The upper reach is tidally influenced and experiences a large range of water levels. Table H-2 outlines the tidal datums for the Seattle, Washington, National Oceanic and Atmospheric Administration Tidal Station (944130).

Table H-2
Seattle Tidal Datums

Datum	Water Level (feet MLLW)
Highest Astronomical Tide (HAT)	13.3
Mean Higher High Water (MHHW)	11.3
Mean Sea Level (MSL)	6.6
North American Vertical Datum of 1988 (NAVD88)	2.3
Mean Lower Low Water (MLLW)	0
Lowest Astronomical Tide (LAT)	-4.3

Survey data were used to develop a cross section perpendicular to the FNC, through the middle of the example cap design area (Figure H-4). The authorized elevation for the FNC is -15 feet mean lower low water (MLLW). The eastern channel boundary is approximately 60 feet from the edge of the proposed example cap design area.

Figure H-4
Cross Section A-A′ Through Example Cap Design Area (RAA 18)



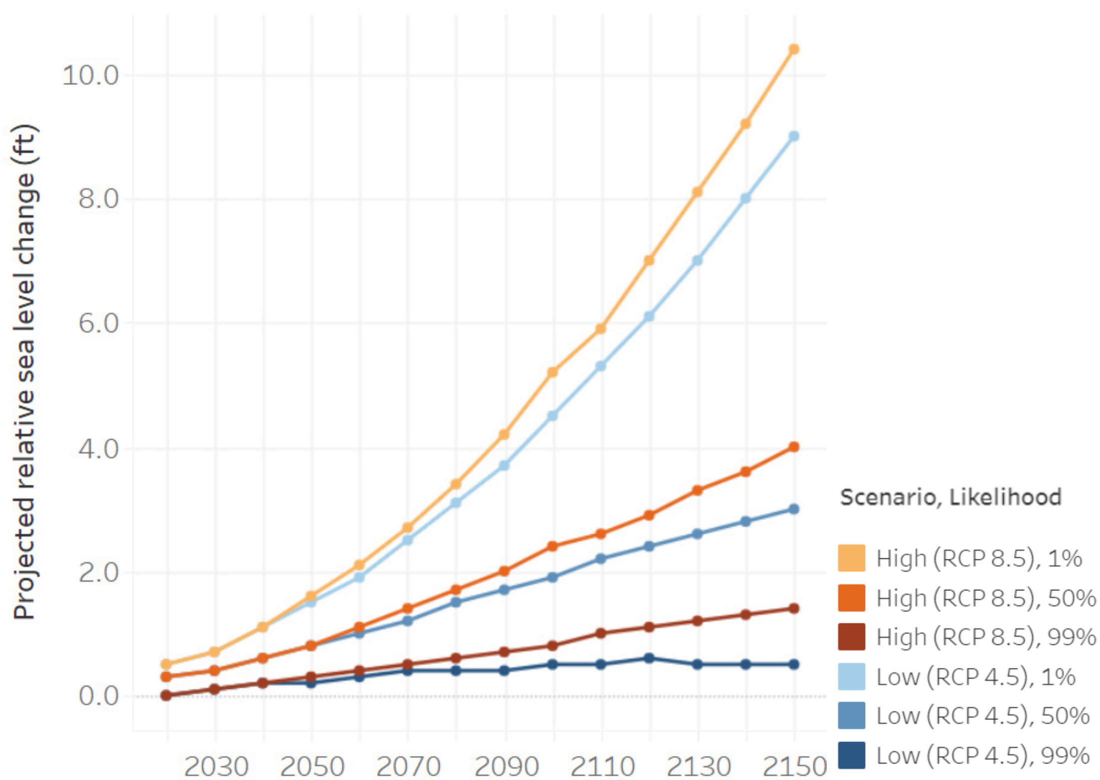
Notes:
Bathymetric survey by Northwest Hydro performed between April 18, 2019, and May 15, 2019. Additional bathymetric survey by Northwest Hydro performed June 2020 to fill data gaps. Composite data updated December 23, 2020.
The location of cross section A-'A' is shown in Figure H-1.

As described in Section 11.4.1 of the BODR, climate change is expected to increase sea levels over the next few hundred years. An increase in mean sea level (MSL) will correspond to an increase in design water levels at the site. In the future, SLR will increase the water depths within the upper reach. The projected changes in sea level have been assessed in accordance with Washington State Department of Ecology guidance.

Figure H-5 shows the projected SLR for various potential scenarios for the upper reach. The figure presents the projected SLR under the low and high predictions for greenhouse gas scenarios (Representative Concentration Pathways 4.5 and 8.5) for the 1%, 50%, and 99% likelihood of occurrence. While there is no industry standard for the application of SLR projections, other projects in Puget Sound have incorporated the 50% central estimate for the design of site elevations. Based on the projections and using the 50% central estimate, the relative sea level is predicted to rise between 1.9 and 2.4 feet by 2100 (Miller et al. 2018).

SLR will have different effects on the erosive forces acting on the cap, as discussed in Sections 2.3 through 2.6. Propwash forces are expected to be lower with SLR due to the larger propeller clearance as water depths increase. Wake forces are not expected to change with SLR because wake heights are not expected to change. Hydrodynamic forces are expected to be lower with SLR due to the larger flow area under the same flow volumes because flow is controlled by the upstream restriction at the Interstate 5 crossing of the Green River as described in 11.4.2 of the BODR. Wind-generated waves are not expected to be affected by SLR because they are limited by fetch lengths and the narrow shape of the waterway, which would not materially change under SLR.

Figure H-5
Sea Level Rise Projections for the LDW (Washington Coastal 2022)



2.3 Predictive Modeling to Estimate Stable Particle Size for Propwash Forces

As a vessel or boat moves through the water, the propeller produces an underwater jet. This turbulent jet is known as propwash. Where the jet reaches the mudline, it can contribute to resuspension or movement of bottom particles. Potential propwash effects of representative vessels that operate near and around the example cap design area were evaluated in accordance with Appendix A of U.S. Environmental Protection Agency's (EPA's) *Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Palermo et al. 1998) cap armor layer design guidance.

The propwash velocity was calculated using the method developed by Blaauw and van De Kaa (1978). The stable particle size under these velocities was calculated based on a method by Blaauw et al. (1984) and additional research by Maynord (1984); both methods are presented in Appendix A of EPA's *Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Palermo et al. 1998): *Armor Layer Design*. The model considers physical vessel characteristics (e.g., propeller diameter, depth of propeller shaft, and total engine horsepower) and operational and site conditions (e.g., applied horsepower and water depth) to estimate propeller-induced bottom velocities at various distances behind the propeller. The model is used to predict the particle size that would be stable when subjected to the steady-state propwash (i.e., the vessel is essentially stationary or maneuvering at a very low speed) from the modeled vessel.

Equation 6 from Appendix A of Palermo et al. (1998) predicts the propeller velocity at any location below (z distance) and aft of (x distance) the vessel propeller:

$$V_x = 2.78 \times U_0 \times \frac{D_0}{x} \exp\left(-15.43\left(\frac{z}{x}\right)^2\right)$$

where:

V_x	=	propwash velocity at location x and z (fps)
D_0	=	adjusted propeller diameter (function of propeller type and diameter)
x	=	horizontal distance aft of propeller (feet)
z	=	distance from axis of propeller (feet)
U_0	=	propwash jet velocity (fps) at the propeller (Equation 4 from Appendix A of Palermo et al. [1998])

The above equation was used to compute propwash velocities for the selected design vessels based on their specifications and operating conditions. For each scenario, bathymetric data were compiled to apply water depths and shoreline orientations (distances and slopes) such that realistic scenarios were analyzed. Propwash velocities at the sediment bed surface were calculated by applying jet velocities to the water depths and local bathymetry data and determining the velocity of the jet where it would meet the sediment bed mudline.

The example design area (RAA 18) is located east of the FNC (Figure H-1), and as Figures H-2 and H-3 show, the design vessels are not expected to transit directly over the area. A plan view analysis of the bottom velocities and stable sediments was conducted to estimate how a transiting vessel could affect the proposed cap (see Attachment H-1). The scenarios evaluated and results are outlined in Table H-3. All scenarios conservatively assume the design vessels are operating with the propeller located at the eastern boundary of the FNC (mudline elevation of -15 feet MLLW), which is approximately 60 feet from the edge of the example design area (elevation of -4 feet MLLW). PIANC (2015) suggests using 5% to 15 % of the installed power for the main propellers for transiting vessels. For this analysis, 20% and 30% applied power were used to calculate the propwash velocities as an extra measure of conservatism above the PIANC guidelines and to more closely compare to the FS assumptions, which were based in part on interviews with local vessel captains. Although vessels typically operate at some sailing speed, which acts to significantly reduce the duration and magnitude of the propwash acting on the waterway bottom, for purposes of this analysis, static vessel conditions (stationary vessel) were used for evaluating potential propwash forces, which is another conservative factor.

In addition, sailing scenarios for larger vessels at low tidal elevations (e.g., MLLW) were not modeled because the large vessel drafts (drafting greater than 11 feet) would make navigation unsafe due to small propeller clearances. The modeled scenarios are considered to be conservative because of the additive conservative assumptions used to develop the modeled scenarios.

As shown in Table H-3, because the example cap design area is 60 feet, or more, farther east of the FNC (Figure H-1), the predicted bottom velocities and required stable sediment D_{50} are relatively small (0.5 foot per second [ft/sec] and <0.25 inch).

Future SLR conditions are not expected to increase the stable particle size required based on propwash. The stable particle size due to propwash forces increases as propeller jet induced bottom velocities increase. With SLR the water depths will increase, therefore increasing the propeller clearance and reducing the bottom velocities, ultimately requiring a smaller particle size to be stable.

Table H-3
Bottom Velocities and Stable Sediment Size

Attachment H-1 Figure No.	Design Vessel	Water Level (feet MLLW)	Applied Power	Max. Bottom Velocity in Cap Design Area (ft/sec)	Stable D_{50} in Cap Design Area (inches)
1a	Capt. Cae Tug	MHHW (11.3)	20%	0.5	0.25
1b	Capt. Cae Tug	MSL[1] (6.6)	20%	0.5	0.25
2a	Westrac II Tug	MHHW (11.3)	20%	<0.5	<0.25
2b	Westrac II Tug	MSL[1] (6.6)	20%	<0.5	<0.25
3a	Arctic Pride Yacht	MHHW (11.3)	20%	<0.5	<0.25
3b	Arctic Pride Yacht	MLLW (0)	20%	<0.5	<0.25

Attachment H-1 Figure No.	Design Vessel	Water Level (feet MLLW)	Applied Power	Max. Bottom Velocity in Cap Design Area (ft/sec)	Stable D_{50} in Cap Design Area (inches)
4a	Capt. Cae Tug	MHHW (11.3)	30%	0.5	0.25
4b	Capt. Cae Tug	MSL[1] (6.6)	30%	0.5	0.25
5a	Westrac II Tug	MHHW (11.3)	30%	0.5	0.25
5b	Westrac II Tug	MSL[1] (6.6)	30%	0.5	0.25
6a	Arctic Pride Yacht	MHHW (11.3)	30%	<0.5	<0.25
6b	Arctic Pride Yacht	MLLW (0)	30%	<0.5	<0.25

Notes:
All scenarios assume the design vessel is operating with the propeller on the edge of the FNC (elevation of -15 feet MLLW), which is approximately 60 feet from the edge of the example cap design area (elevation of -4 feet MLLW).
1. Capt. Cae Tug and Westrac II Tug were not analyzed at MLLW; given their larger drafts, it is unlikely they would operate with such small propeller clearances.

D_{50}: median particle size
ft/sec: feet per second
MHHW: mean higher high water
MLLW: mean lower low water
MSL: mean sea level

2.4 Predictive Modeling to Estimate Stable Particle Size for Wake Forces

Estimates of vessel-induced wake heights were completed through an evaluation of ship traffic patterns within the FNC adjacent to the example design area and calculations of vessel wakes based on type of vessel, operational speed, and water depths.

Based on the vessel speed and locations shown in Figure H-3, the design vessels were assumed to be operating at speeds between 4 and 8 knots (4.6 and 9.2 miles per hour [mph]) within the FNC[1], as close as 60 feet to the edge of the cap from the potential sailing line along the eastern edge of the FNC. The analysis used the Weggel and Sorensen (1986) methodology to predict vessel wakes. The Weggel-Sorensen method is an empirical model (developed from available laboratory and field data on vessel-generated wakes) to predict maximum wake height as a function of vessel speed, vessel geometry, water depth, and distance from the sailing line. This model is applicable for various vessel types (ranging from tugboats to large tankers), vessel speeds, and water depths. The method calculates the wake height generated at the bow of a vessel as a function of the vessel speed, distance from the sailing line, water depth, vessel displacement volume, and vessel hull geometry (i.e., vessel length, beam, and draft). The method has been widely tested on different vessels and is recommended for use

[1] The Duwamish River has a 7-knot speed limit; AIS data indicate that some vessels exceed this limit.

with conditions having a Froude number between 0.2 and 0.8, which was met. The non-dimensional Froude number used in this method is defined as follows:

$$\mathrm{Fr} = \frac{v}{\sqrt{g \times l_w}}$$

where:

Fr	=	Froude number
v	=	vessel velocity (ft/sec)
g	=	vcceleration due to gravity foot per second squared
l_w	=	water depth (feet)

Design vessel wake heights were estimated to be between 0 and 1.2 feet with a period up to 2.2 seconds (see Table H-4).

Waves (or wakes) break in shallow water when the ratio of wave height to water depth surpasses 0.78 (Dean and Dalrymple 1991). The wide tidal range means the example cap design area is sometimes fully inundated, and at lower tidal levels, parts of the cap design area are above the water surface. At a MLLW tidal elevation, the maximum water depth along the edge of the cap design area is approximately 4 feet. At the lowest astronomical tide (LAT; -4.3 feet MLLW), the entire the cap design area would be above the water surface. As the water surface rises and falls over the cap design area, every portion of the proposed cap area will fall within the wave breaking zone. For waves breaking on the cap, the rubble-mound revetment module (USACE 2004) from the Automated Coastal Engineering System developed by the U.S. Army Corps of Engineers (USACE 1992) was used to compute the median particle size (D_{50}) that is stable for the predicted wake height based on the proposed placement slope.

A bracketing analysis that considered flatter and steeper restored slopes was conducted; Figure H-4 depicts conditions where the cap would be placed at a 2.5 horizontal to 1 vertical (2.5H:1V) slope, with a backfill placed over with a finished grade slope of approximately 6H:1V, and the cap placed on existing grade near the bulkhead wall with a slope of 6H:1V. This area was considered to represent the range of typical slope conditions that are anticipated for backfill in the upper reach. If different cap slope angles are determined necessary during 60% design, this bracketing analysis will be revisited and updated as appropriate.

Based on these analyses, a stable median particle size diameter (D_{50}) of 3 inches would withstand vessel wakes that break on top of a 6H:1V backfill layer, and, assuming the backfill was eroded away, a 2.5H:1V erosion protection layer would require a D_{50} of 5 inches to withstand the breaking vessel wakes (summarized in Table H-4).

Table H-4
Vessel Wakes and Stable Sediment Sizes

Vessel	Vessel Speed (mph)	Distance from Sailing Line (feet)	Wake Height (feet)	Wake Period (seconds)	Slope	Stable Armor Stone Size D_{50} (inches)
Capt. Cae Tug (MLLW at edge of FNC)	9.2	60	1.2	2.2	Backfill: 6H:1V	2.8
					Erosion Protection Layer: 2.5H:1V	4.8
Westrac II Tug (MLLW at edge of FNC)	9.2	60	1.2	2.2	Backfill: 6H:1V	2.8
					Erosion Protection Layer: 2.5H:1V	4.7
Arctic Pride Yacht (MLLW at edge of FNC)	9.2	60	1.0	2.2	Backfill: 6H:1V	2.4
					Erosion Protection Layer: 2.5H:1V	4.0

Notes:
D_{50}: median particle size
FNC: federal navigation channel
H:V: horizontal to vertical (ratio)
MLLW: mean lower low water

The wake heights are not expected to increase or decrease with the addition of SLR to the waterway. Therefore, required stable sediment sizes for future SLR conditions are not expected to change. Additionally, SLR will reduce the size of the intertidal zone, as the water level range moves farther up the existing bulkhead wall on the shoreline and to the east of the cap design area (see Figure H-4). Therefore, SLR conditions are not expected to modify the evaluation scenario or resulting stable particle size for the example erosion protection layer at the cap design area.

2.5 Predictive Modeling to Estimate Stable Particle Size for Hydrodynamic Forces

Stable particle sizes to resist hydrodynamic flows (i.e., river currents) were assessed for the example cap design area. The 100-year flow event was modeled for the *Lower Duwamish Waterway Sediment Transport Modeling Report* in 2008 (QEA), and the velocity results from the hydrodynamic model cell that includes the cap design area were used to estimate the stable particle size.

The stable particle size was estimated using a method developed by Maynord (1988) *for Stable Riprap Size for Open Channel Flows*, which is presented in Appendix A of EPA's *Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Palermo et al. 1998).

The stable particle size is estimated utilizing the following equation:

$$D_{50} = S_f C_S C_V C_T C_G d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5}$$

where:

D_{50}	=	characteristic riprap size of which 50% is finer by weight
S_f	=	safety factor (1.5)
C_S	=	stability coefficient for incipient failure (0.3 for angular rock)
C_V	=	velocity distribution coefficient (1.0 for straight channels)
C_T	=	blanket thickness coefficient (1.0 for flood flows)
C_G	=	gradation coefficient = $(D_{85}/D_{15})^{1/3}$ (typically 1.2 to 1.5)
d	=	local depth
γ_w	=	unit weight of water
γ_s	=	unit weight of stone
V	=	local depth averaged velocity
K_1	=	side slope correction factor = 0.97 (defined below)
g	=	gravitational constant

$$K_1 = \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \Phi}}$$

where:

K_1	=	side slope correction factor
θ	=	angle of side slope with horizontal (2.5H:1V = 21.8 degrees)
Φ	=	angle of repose of riprap material (typically 40 degrees)

The modeled 100-year velocity over the cap design area was 1.5 ft/sec, which results in an estimated 0.1-inch stable particle size. Note that the resulting stable particle size is smaller than that required for propwash, even though the 100-year velocity was larger than the propwash velocity. This is due to the propwash velocity being a turbulent jet, which results in a larger stable grain size for the 0.5 ft/sec velocity as compared to the 0.1-inch stable particle size for river flow at 1.5 ft/sec.

As described in section 11.4.2 of the BODR, hydrodynamic forces are expected to be lower with SLR due to the larger flow area under the same flow volumes. A larger flow area will reduce the velocities, therefore reducing the required stable sediment size.

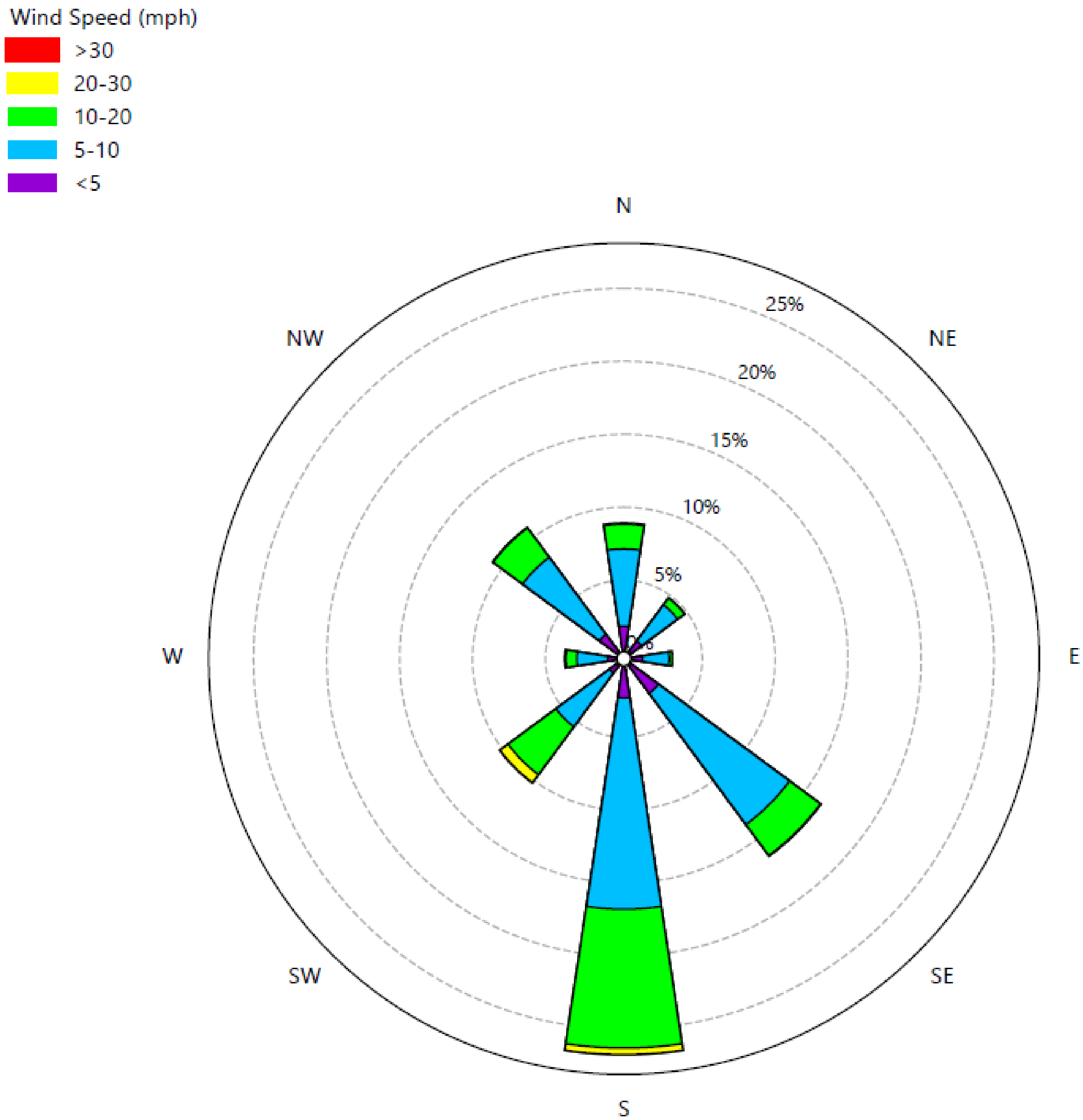
2.6 Predictive Modeling to Estimate Stable Particle Size for Wind-Generated Waves

Wind-generated waves are a result of wind blowing over the water surface. Such waves become larger due to continuous wind in an unobstructed single direction, over long distances (fetch[2]). To estimate the wind-generated wave height, the Automated Coastal Engineering System Wave Prediction module was used, which uses wind speed, water depth, and effective fetch distance (Leenknecht et al. 1992).

Measured wind direction and intensity data from King County International Airport, approximately 1.5 miles north of the upper reach, are shown in the wind rose in Figure H-6.

[2] Fetch refers to the unobstructed over-water distance in the wind direction of interest. Fetch distance can be very long in large open-water locations (e.g., oceans) and is very short where land masses and other wind obstructions (e.g., buildings, bridges) limit the ability of wind shear stress to act for sustained distance on the water surface.

Figure H-6
Wind Rose: King County International Airport (1943 Through 2022)



Notes:
1. Maximum recorded wind speed: 62 mph
2. Wind data are presented as the "blowing from" direction

An extreme analysis was conducted for the 79 years of wind data to find the 100-year wind speeds at various directions. The example cap design area is near a bend in the LDW that has two fetches, one from the northwest and one from the south. The 100-year wind speeds are 43 mph from the northwest and 62 mph from the south. The fetch from the northwest is approximately 1.7 miles and the fetch from the south is approximately 0.9 mile. However, given the waterway is narrow, with a low width to length ratio, effective fetch factors were included to reduce the fetch lengths to 0.2 and 0.3 mile, respectively (Ippen 1966). Utilizing the FNC depth of 26.3 feet at mean higher high water (MHHW), maximum wave heights for the 100-year wind speeds are 0.3 foot from the northwest and 0.6 foot from the south. SLR, discussed in the Section 2.2, would not materially increase the width of the river, and therefore would not change the predicted future wave heights

Wind-generated wave heights are smaller than the predicted possible wake heights caused by transiting vessels (Section 2.4). Thus, wind-generated waves will not govern the size of the erosion protection layer aggregate.

2.7 Recommended Armor Material Size, Layer Thickness, and Filter Material Size

The example cap design area is expected to be made up of a chemical isolation layer protected by an overlying erosion protection (armor) layer for cap stability on a 2.5H:1V slope, with habitat-compatible sand backfill placed on top to return the area to existing grade, at approximately 6H:1V. Because the habitat-compatible layer will be the topmost layer, the armor material is not expected to be directly exposed to erosive forces. However, the armor layer is designed to resist erosive forces as a "backstop" in case the habitat-compatible layer is displaced. Habitat-compatible material is within a size range that can be moved by design-level erosive forces whereas armor material will be stable when subjected to the same forces.

The armor layer material size is driven by the largest particle size that is stable against a range of erosive forces in the upper reach, including hydrodynamic forces, wind-generated waves, vessel-generated propwash, and wakes. The primary design criterion for erosion protection is breaking wakes caused by vessel transit (Section 2.4) and requires a median stable particle size (i.e., D_{50}) of 3 inches when placed on typical flatter slopes (6H:1V) and 5 inches when placed on steeper slopes (2.5H:1V).

Guidance from Appendix A of Palermo et al. (1998) was used to determine the minimum thickness of the armor layer of the cap. Based on the above stable particle size estimates, the armor size is dictated by the wake forces (Section 2.4) and requires a D_{50} of 3 to 5 inches. From the guidance, the armor layer thickness should be two times the D_{50} size, therefore, an armor material with a D_{50} of 3 inches would need to be a minimum of 6 inches thick and an armor material with a D_{50} of 5 inches would need to be a minimum of 10 inches thick. The armor design will be refined, if appropriate, during 60% RD if it is determined that armor layers are needed for capping areas that are subject to different erosion forces.

Development of gradations of cap materials will consider the design D_{50} values and criteria from the U.S. Army Corps of Engineers Engineering Manual 1110-2-2300 - *General Design and Construction Considerations for Earth and Rock-Fill Dams* (USACE 2004). In addition, the potential for vertical migration of one granular material through another (often referred to as "piping") will also be considered, as recommended by the *Guidance for In-Situ Subaqueous Capping of Contaminated Sediment* (Palermo et al. 1998). The potential for piping can be minimized through the use of well-graded materials for the armor and chemical isolation layers. The compatibility of the two materials in combination is verified below in accordance with geotechnical filter criteria (Terzaghi and Peck 1967) and Palermo et al. (1998).

Standard geotechnical filter criteria presented by Terzaghi and Peck (1967) provide recommended particle size ratios between base and overlying materials (e.g., sand chemical isolation and overlying erosion protection materials). The primary filter criterion particle size relationship primarily applicable to subaqueous capping materials is the ratio of the D_{15} of the armor stone to the D_{85} of the base layer. This relationship relates to the ability of the base layer material (e.g., sand) to pass through the void spaces in the overlying larger material (e.g., erosion protection armor stone). Compliance with the recommended filter criteria minimizes the potential for wash out of the base material by the creation of internal filters in the armor stone voids.

The Terzaghi filter criteria recommend the following relationship to prevent material loss through the armor layer:

$$d_{15(Armor)} < 5d_{85(Base)}$$

where:

d_x = particle diameter, such that x percent of the sediment particles are smaller, by weight

The cap armor and chemical isolation material specifications will be developed so that a separate filter layer will not be required.

The specific armor and filter material gradations will be selected during Intermediate (60%) and Pre-Final 90% RD. A number of factors will be considered in developing the complete material specifications, including the following:

- Local availability of materials
- Material processing effort required to meet specifications
- Cap material placement equipment and limitations
- Required quantities
- Fines content relative to water quality (turbidity)
- Well-graded materials

2.8 RAA 27 Erosion Protection Design

The forces acting on a cap (as outlined in Sections 2.3 through 2.6) within the LDW upper reach are propwash, vessel-generated wake waves, hydrodynamic waves (river currents), and wind-generated waves. These forces were evaluated for RAA 27 and are summarized in this section.

If the need for a cap at RAA 27 is confirmed following post-excavation sampling, then an engineered cap will be placed over the steeper sloped riverbank portions of the RAA, nearest the shoreline. The riverbank slopes are approximately 270 to 330 feet from the navigation channel, between elevations from approximately +2 to +20 feet MLLW with an existing 2H:1V riverbank slope. Similar to the RAA 18 example cap evaluation, propwash and vessel-generated waves are the largest forces on the engineered cap, and the design evaluation is summarized here.

Propwash forces were analyzed utilizing the Captain Cae Tug, as it was determined to be the worst-case scenario in Section 2.3. The following similar conservative assumptions were made for the RAA 27 analysis as were made for the example cap design in Section 2.3:

- The vessel is operating at the edge of the FNC closest to RAA 27, resulting in the propeller distance of approximately 250 feet from the cap.
- The vessel is transiting, not turning, the propeller towards the shoreline.
- 20% and 30% applied power were evaluated.
- MHHW (+11.4 feet MLLW) and MSL (+6.6 feet MLLW) water levels were evaluated.

Attachment H-2 shows the plan view results of the analysis at +2 feet MLLW for the velocity and stable sediment size. Ultimately, the site being located at least 250 feet from the navigation channel places it beyond the typical range of transiting propwash. Therefore, all scenarios resulted in a D_{50} stable sediment size of less than 0.25 inch at the cap area for propwash forces.

For the vessel wake force analysis, two scenarios were evaluated to account for the varying distances from the sailing line. The results are outlined in Table H-5. The resulting stable median particle size (D_{50}) for the cap area is 2.7 inches.

Table H-5
Vessel Wakes and Stable Particle Sizes: RAA 27

Vessel	Vessel Speed (mph)	Distance from Sailing Line (feet)	Wake Height (feet)	Wake Period (seconds)	Slope	Stable Median Particle Stone Size D_{50} (inches)
Captain Cae Tug (MLLW at edge of FNC)	9.2	270	0.5	2.2	2H:1V	2.7
	9.2	300	0.4	2.2	2H:1V	2.4

Notes"
D_{50}: median particle size
FNC: federal navigation channel
H:V: horizontal to vertical (ratio)
MLLW: mean lower low water
mph: miles per hour

The methodology for determination of armor gradation, layer thickness, and filter material considerations are outlined in Section 2.7.

The primary design criterion for erosion protection at the bank area of RAA 27 is breaking wakes caused by vessel transiting past RAA 27 and requires a median stable particle size (i.e., D_{50}) of 2.7 inches when placed on a 2H:1V slope. An idealized gradation is outlined in Table H-6 and will be refined during Pre-Final 90% RD based on locally available materials. Given a D_{50} of 2.7 inches, the armor layer should have a minimum thickness of 6 inches.

Table H-6
RAA 27 Idealized Armor Gradation

Percent Passing	Size (inches)
100	4.5
85	3.4
50	2.7
15	2.0
0	1.4

Given the size of the armor layer material, a filter material would be required between the armor and isolation sand layer to prevent loss of the isolation sand between the armor material interstices. Based on the criteria outlined in Section 2.7 a filter material with a D_{50} of 0.4 inch would be required for the armor outlined in Table H-6. RAA 27 armor and filter will be further refined during the Pre-Final 90% RD. It is important to note that geotechnical slope stability considerations may end up resulting in a larger design D_{50} size than needed to protect against erosive forces.

3 Area-Specific Technology – Cover Material Design

In addition to engineered caps, an erosion protection design analysis was performed to address stability considerations for area-specific technology application. As described in BODR Section 10.5, use of area-specific technology is proposed for structural offset areas where dredging may cause structural instability of existing structures or armored banks that exceed the enhanced natural recovery (ENR) upper limit concentration; no dredging will be conducted in these structural offset areas. The area-specific technology in structural offset areas (RAAs 18, 24, and 26) will consist of placing a clean cover material (Drawing C157 of Appendix D). Both structural offset areas are above +4 feet MLLW, within the surf zone, and RAA 24 has an approximate slope of 10H:1V, while RAA 26 has an approximate slope of 4H:1V.

The clean cover material would consist of a gravelly sand (similar to ENR material).While ENR remedial technology is not intended to remain stable and is allowed to mix or move around, area-specific technology is intended to be more stable than the ENR technology and will be used in structural offset areas that exceed the ENR upper limit concentration. Should the structural offset area contaminant concentration be below the ENR upper limit, then ENR remedial technology would be used instead of area-specific technology.

The four forces evaluated in Sections 2.3 through 2.6 were propwash, vessel-generated wake waves, hydrodynamic currents, and wind-generated waves. As hydrodynamic and wind-generated waves were determined to be smaller than the vessel wake forces for the example cap design area, and the structural offset areas are within the surf zone, only vessel forces were analyzed for the area-specific technology cover material design. Both propwash and wake forces were analyzed for RAAs 24 and 26.

As discussed in Section 2.2, SLR will have different effects on the erosive forces acting on the cover. Propwash forces are expected to be even lower with SLR due to the larger propeller clearance as water depths increase, and wake forces are not expected to change with SLR because there is no expectation for the wake heights to change, either.

Propwash forces were analyzed utilizing the Captain Cae Tug, as it was determined to be the worst-case scenario in Section 2.3. The following similar assumptions were made for RAAs 24 and 26 analyses as the example engineered cap in Section 2.3:

- The vessel is operating at the edge of the FNC, resulting in the propeller being approximately 140 feet from the edge of the offset areas.
- The vessel is transiting, not turning the propeller toward the shoreline.
- 20% and 30% applied power were evaluated.
- MHHW (+11.4 feet MLLW) and MSL (+6.6 feet MLLW) water levels were evaluated.

Attachment H-3 shows the plan view results of the analysis at +4 feet MLLW for the velocity and stable sediment size. Ultimately, the site being located 140 feet from the navigation channel places it beyond the typical range of transiting propwash. Therefore, all scenarios resulted in a D_{50} sediment size of less than 0.25 inch at the cap area. The same design methodology as presented in Section 2.4 was used to estimate wakes and stable sediment sizes based on new erosion force variables for the structural offset areas. To simplify the analysis, only the Captain Cae Tug was evaluated because it creates the largest wakes at equal speeds compared to the other vessels analyzed.

Table H-7 outlines the variables analyzed and resulting sediment size.

Table H-7
Captain Cae Tug Wakes and Cover Sediment Sizes for Structural Offset Areas

Vessel Speed (mph)	Sail Line Distance (feet)	Water Depth (feet)	Wake Height (feet)	Wave Period (seconds)	Cover Material D_{50} (inches)	
					Slope: 4H:1V	Slope: 10H:1V
7	140	13	0	0	0.0	0.0
8	140	13	0.3	1.9	**0.6**	0.4
9	140	13	0.7	2.1	1.3	0.8

Notes:
D_{50}: medial particle size
H:V: horizontal to vertical ratio
mph: miles per hour

The further sail line distance results in lower wake heights and, therefore, smaller cover material size requirements compared to the erosion protection for the example cap design area (Section 2.4). Lower vessel speeds were also analyzed to account for more typical events within the upper reach versus the highest speed recorded in the AIS data. Additionally, a higher allowable damage level was used compared to the erosion protection example cap design; essentially, a lower factor of safety was used to allow for some movement, but not failure, of the proposed cover material, assuming a thickness of at least 1 foot. This design approach is considered reasonable because the area-specific technology is not intended to be an engineered cap that allows no movement of the erosion protection layer.

Vessels operating at 7 mph or lower result in essentially zero wake; therefore, an armor cover is unnecessary, and a cover consisting of sand material would be sufficient. For higher and less frequent vessel speeds, the cover material size increases. Cover material size also increases as the placement slope becomes steeper. A vessel operating speed of 8 mph was selected as a very conservative condition because it is the vessel navigation speed limit for the area. For the steepest slope in RAA 26, this results in a gravel material with D_{50} of 0.6 inch.

Because the stable sediment size (0.6-inch) analysis is a conservative methodology, and the material does not need to meet the erosion protection requirements of an engineered cap, a mix of sand and the stable gravel is proposed. A mixture of approximately 50% sand and 50% gravel is proposed to meet the cover (sand) and protection (gravel) needs.

The cover material will include sand in the gradation to perform as discussed in Appendix Q and allow for blending of granular activated carbon with the sand fraction when needed. To provide protection during wake events, a gravel fraction is needed with the sand, creating a gravelly sand mix similar to the material placed at the Enhanced Natural Recovery/Activated Carbon (ENR/AC) Pilot Study intertidal plot but with slightly larger gravel and at a higher fraction to increase the stability of the area-specific technology cover material. The intertidal ENR/AC Pilot Study plot was monitored for 3 years and was found to have remained in place and performed as intended under various physical conditions (e.g., wakes and waves and propwash) over the 3-year study (Wood et al. 2021). The cover material proposed for the structural offset is larger than the ENR/AC Pilot Study ENR material because the wake and sediment size analyses are both conservative evaluations.

The stable gravel gradation and cover design gradation blended with 50% sand are outlined in Table H-8. The gravelly sand mix cover should be placed at a thickness of 1 foot.

Table H-8
Area-Specific Technology Material Gradations

U.S. Standard Sieve Size	Proposed Percent Passing by Dry Weight	
	Gravel	50% Gravel Mixed with 50% Sand
1-1/2 inch	100	100
3/4 inch	85	90
1/2 inch	50	70 to 75
3/8 inch	15	50 to 60
U.S. No. 4 (0.187 inch)	0	30 to 50
U.S. No. 10 (0.079 inch)	N/A	20 to 50
U.S. No. 200 (0.003 inch)	N/A	0 to 2

Notes:
N/A: not applicable

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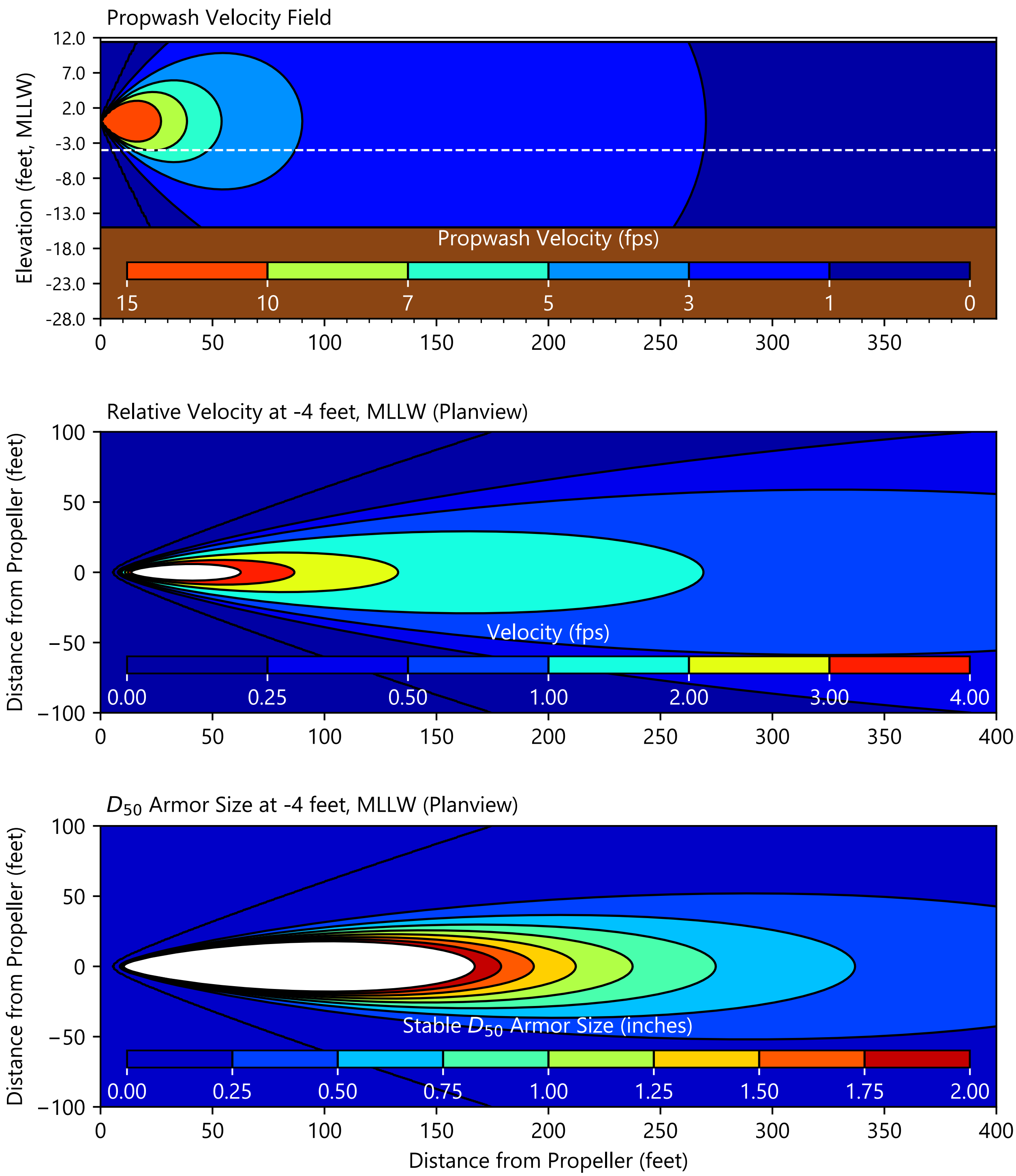
Attachment H-1
Bottom Velocity and Sediment Figures
(RAA 18)



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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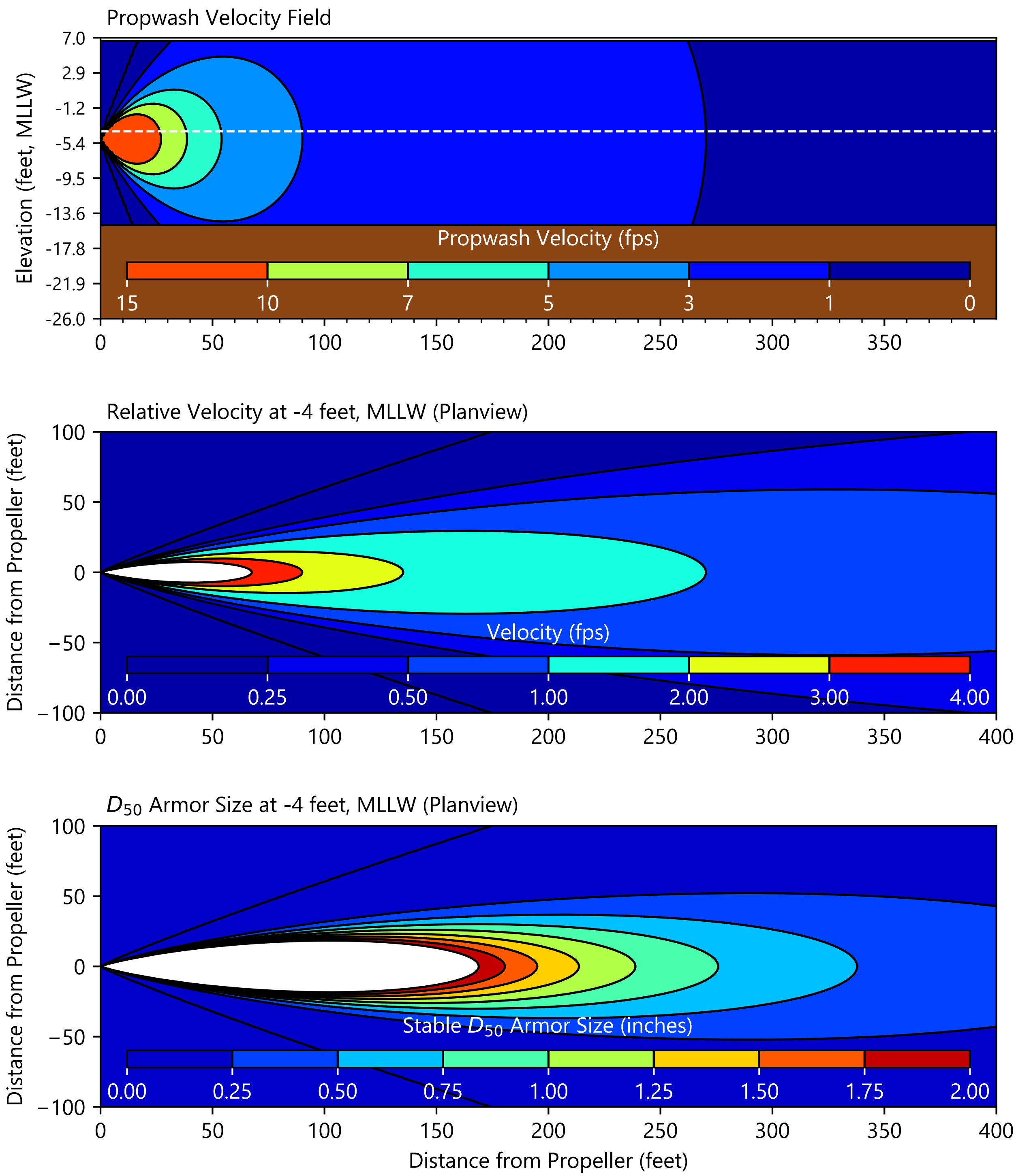
ANCHOR QEA

Figure 1a
Capt Cae Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MHHW



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
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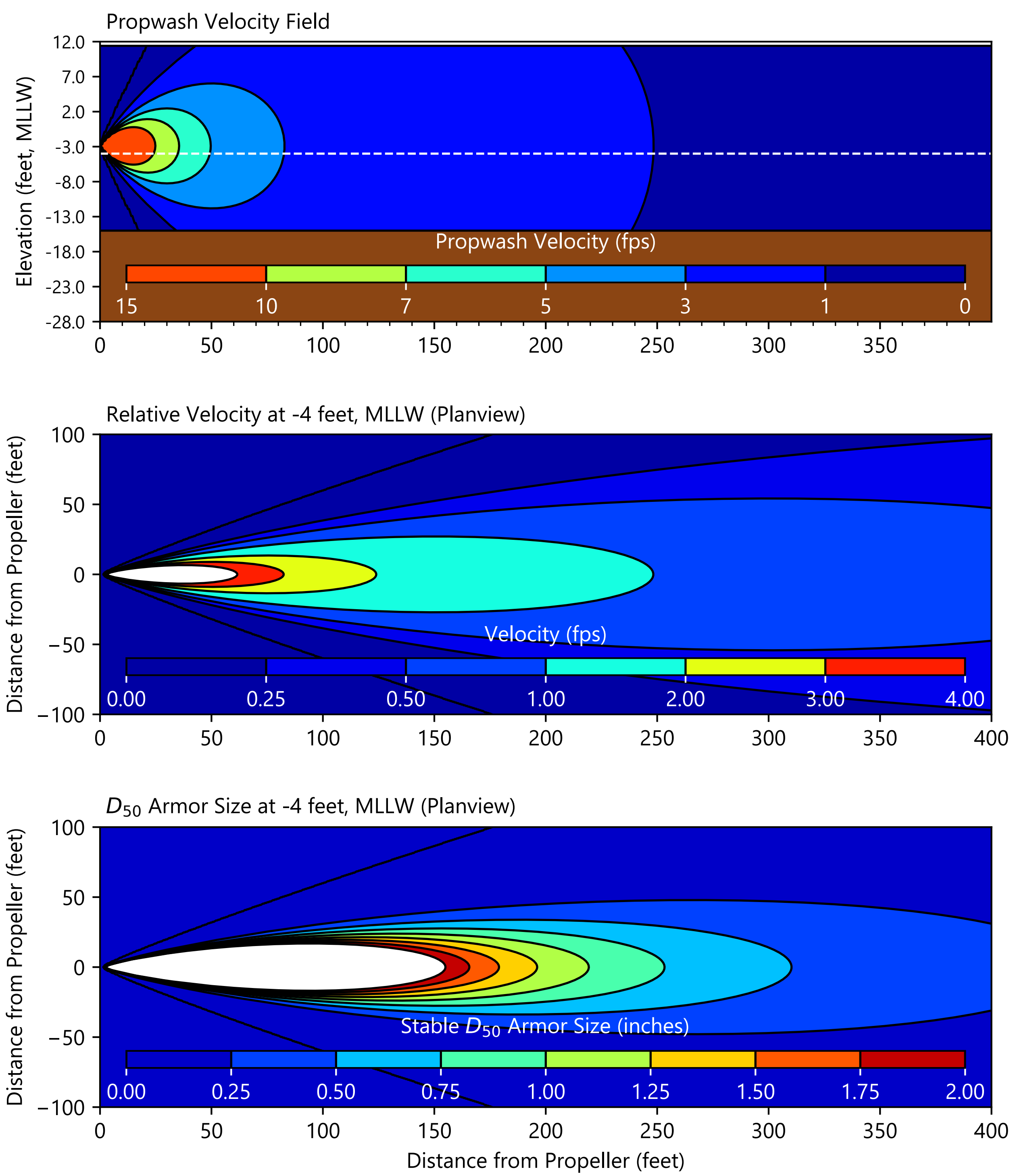
Figure 1b
Capt Cae Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MSL



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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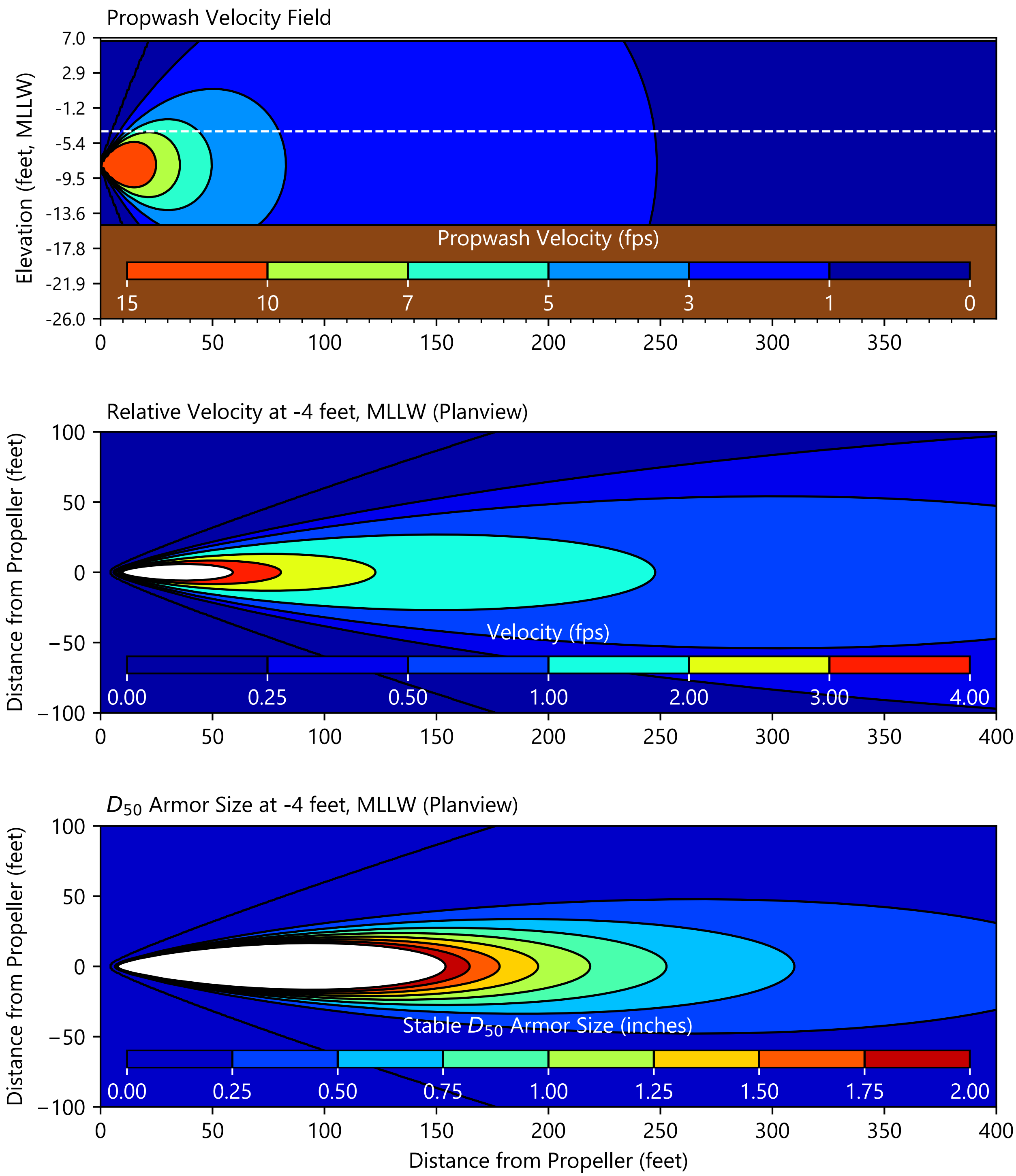
ANCHOR QEA

Figure 2a
Westrac II Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MHHW
60% Remedial Design Basis of Design Report



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
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The C_3 coefficient = 0.55 for no sediment movement.

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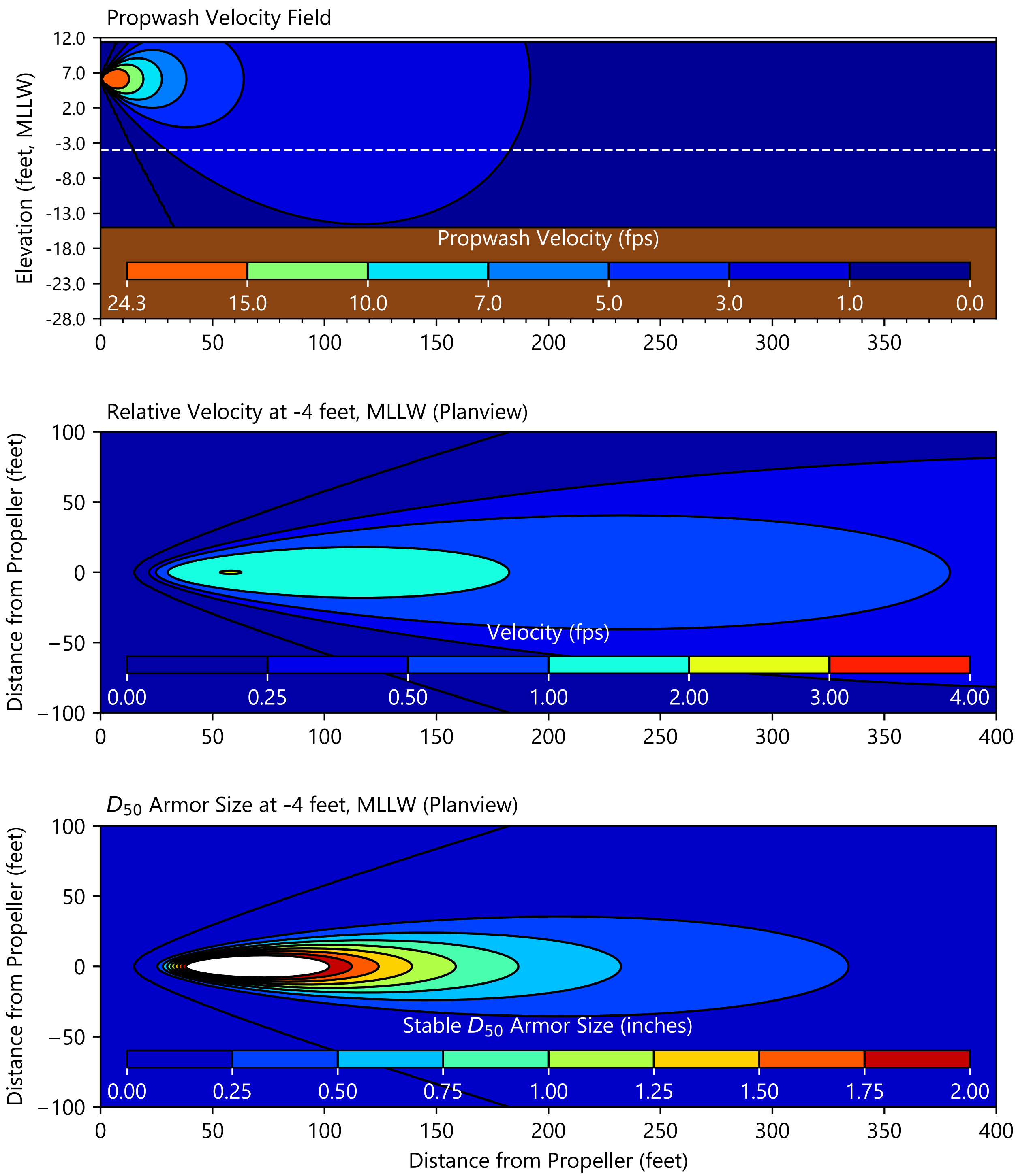
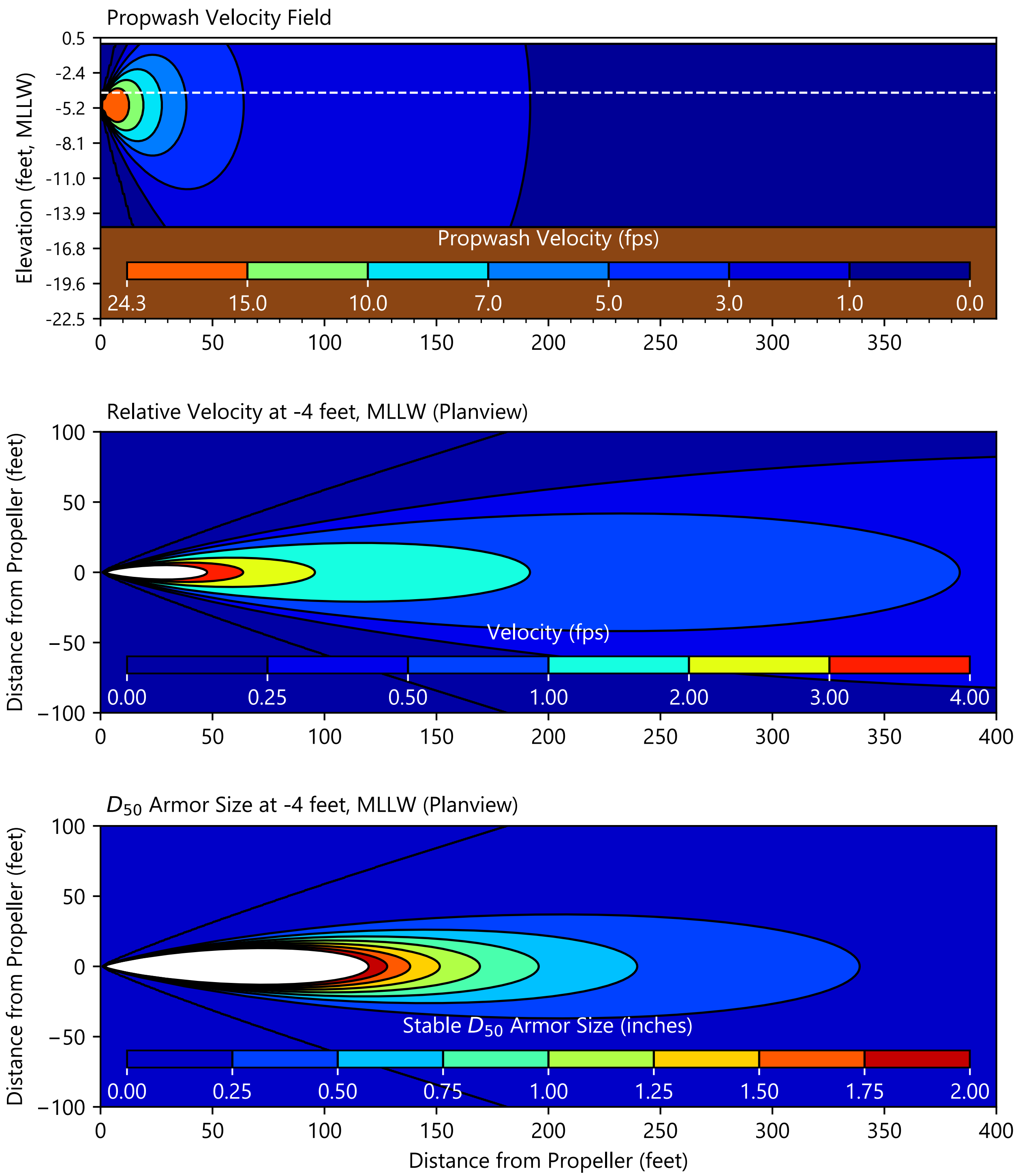
ANCHOR QEA

Figure 2b
Westrac II Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MSL



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

Figure 3a
Arctic Pride Yacht at 0 knots, using 20% Applied Power
Transect: Nav Channel at MHHW



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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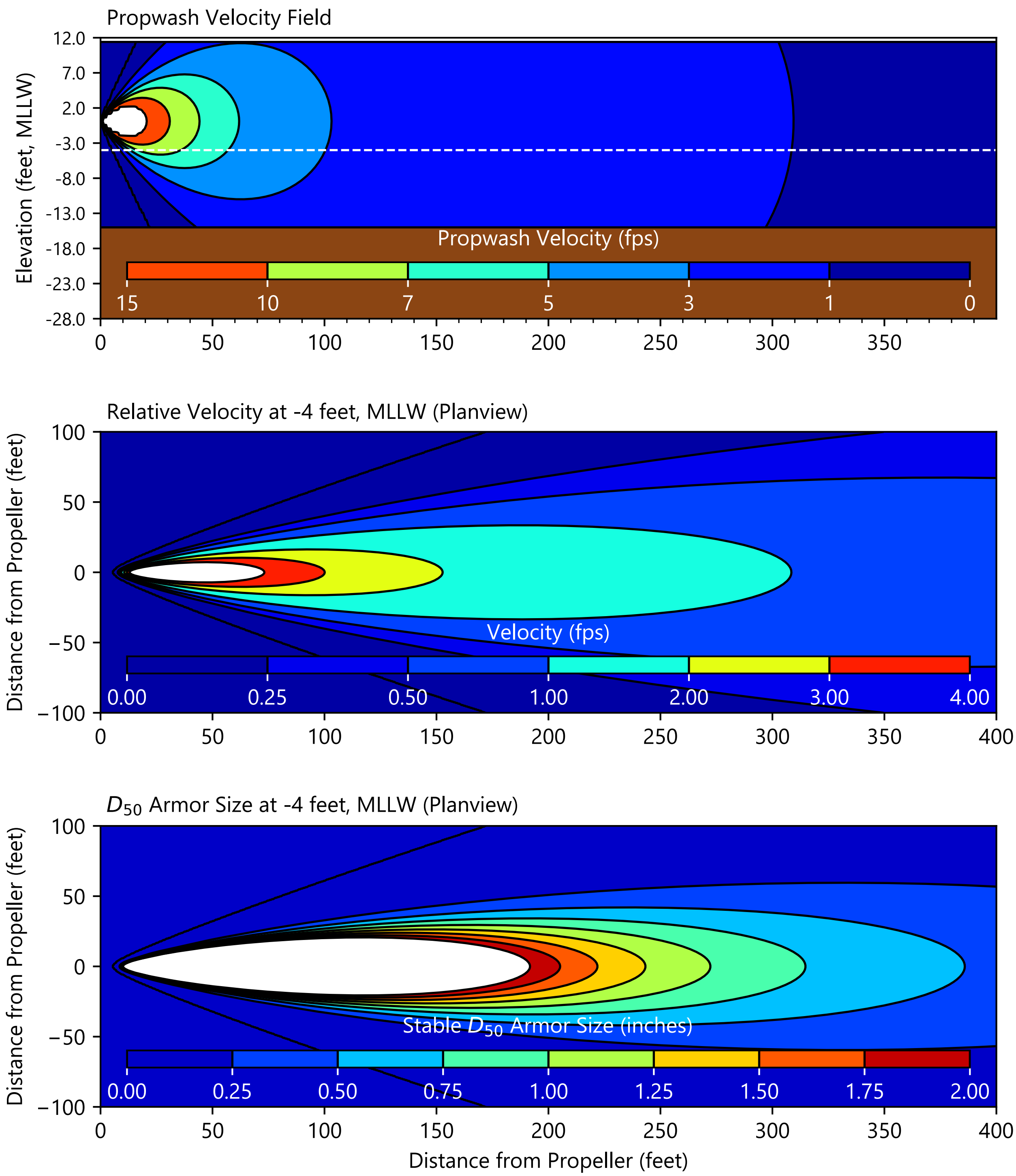
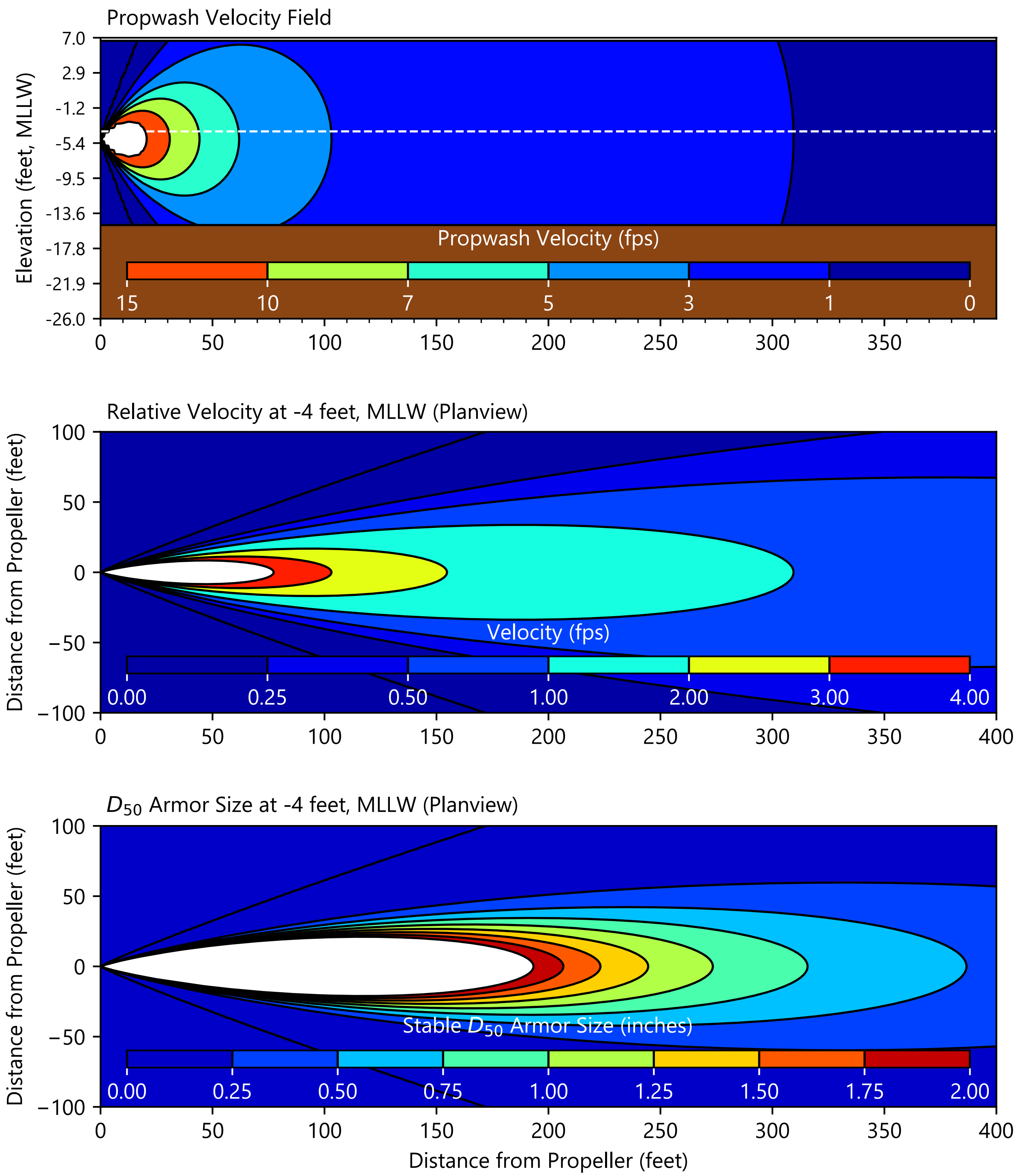
Figure 3b
Arctic Pride Yacht at 0 knots, using 20% Applied Power
Transect: Nav Channel at MLLW

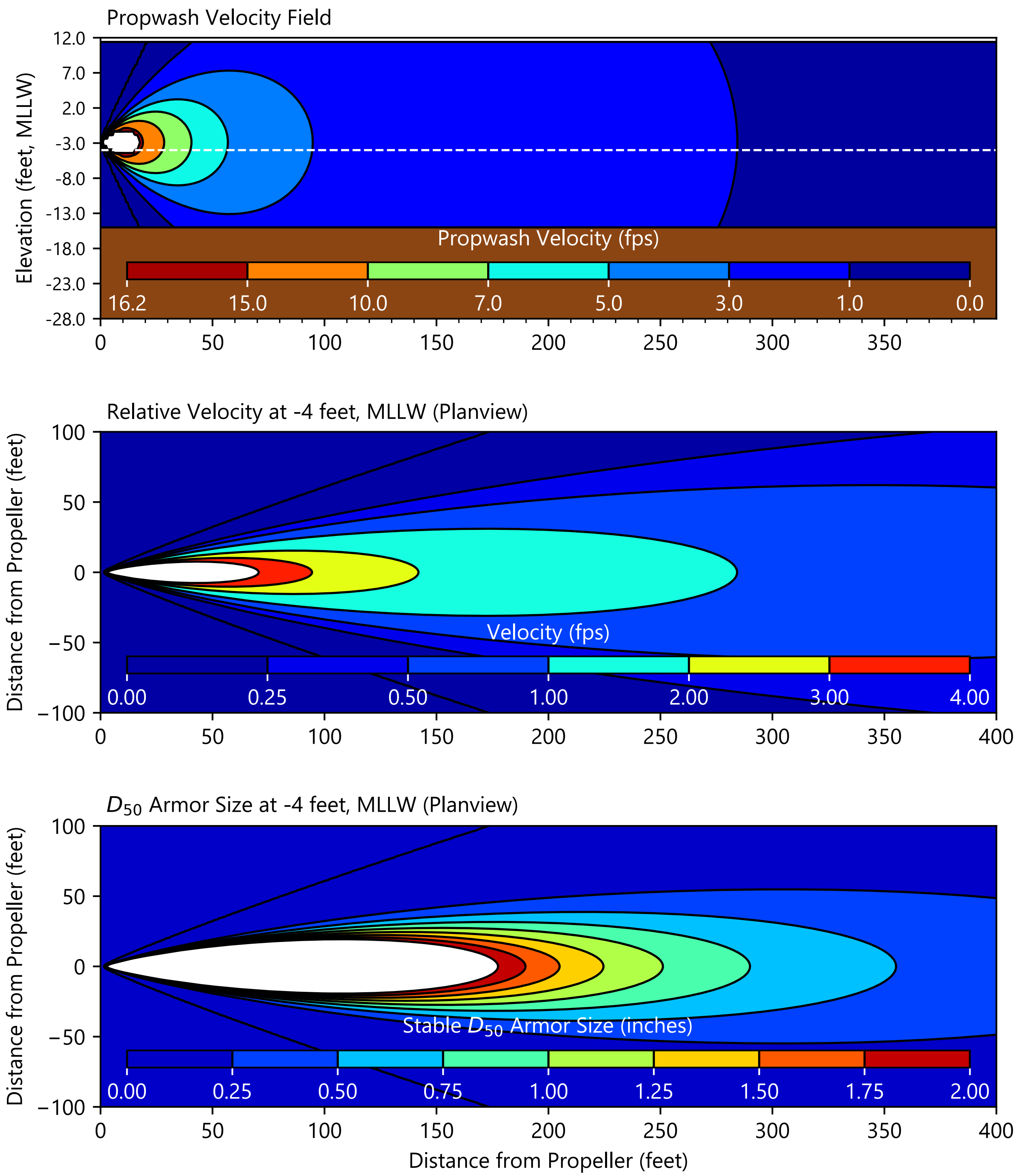


Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

Figure 4a
Capt Cae Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MHHW



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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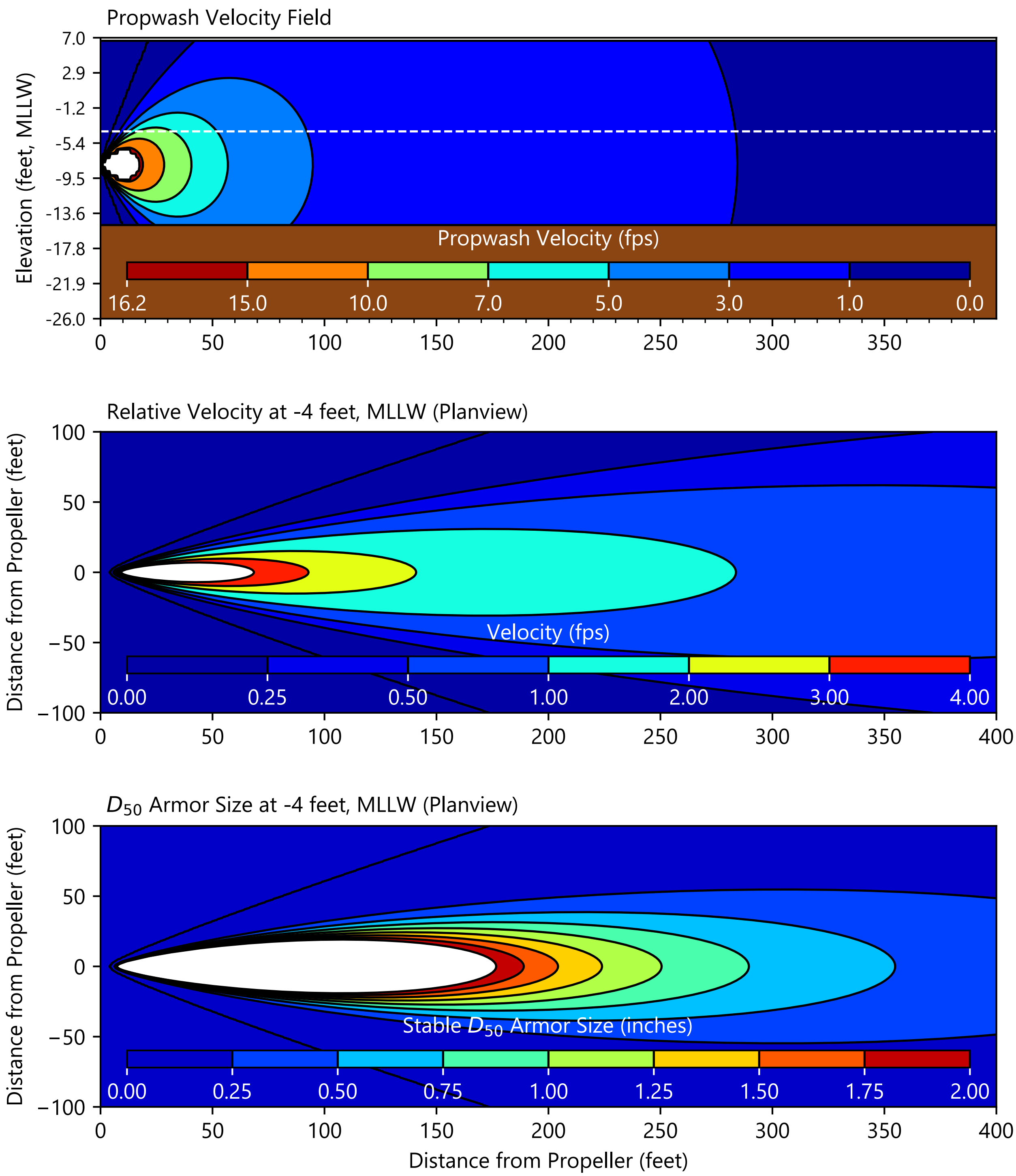
Figure 5a
Westrac II Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MHHW



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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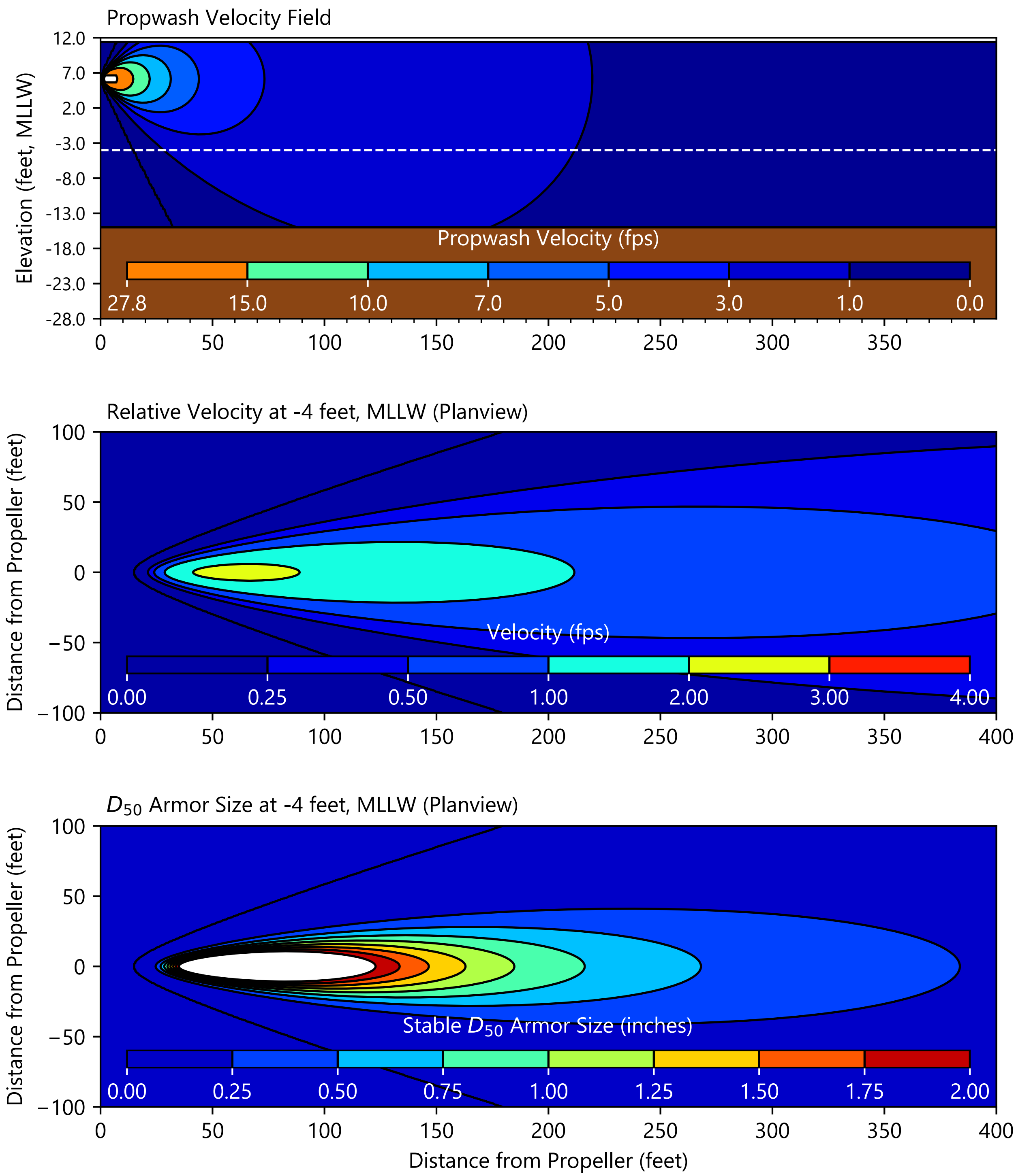
Figure 5b
Westrac II Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MSL



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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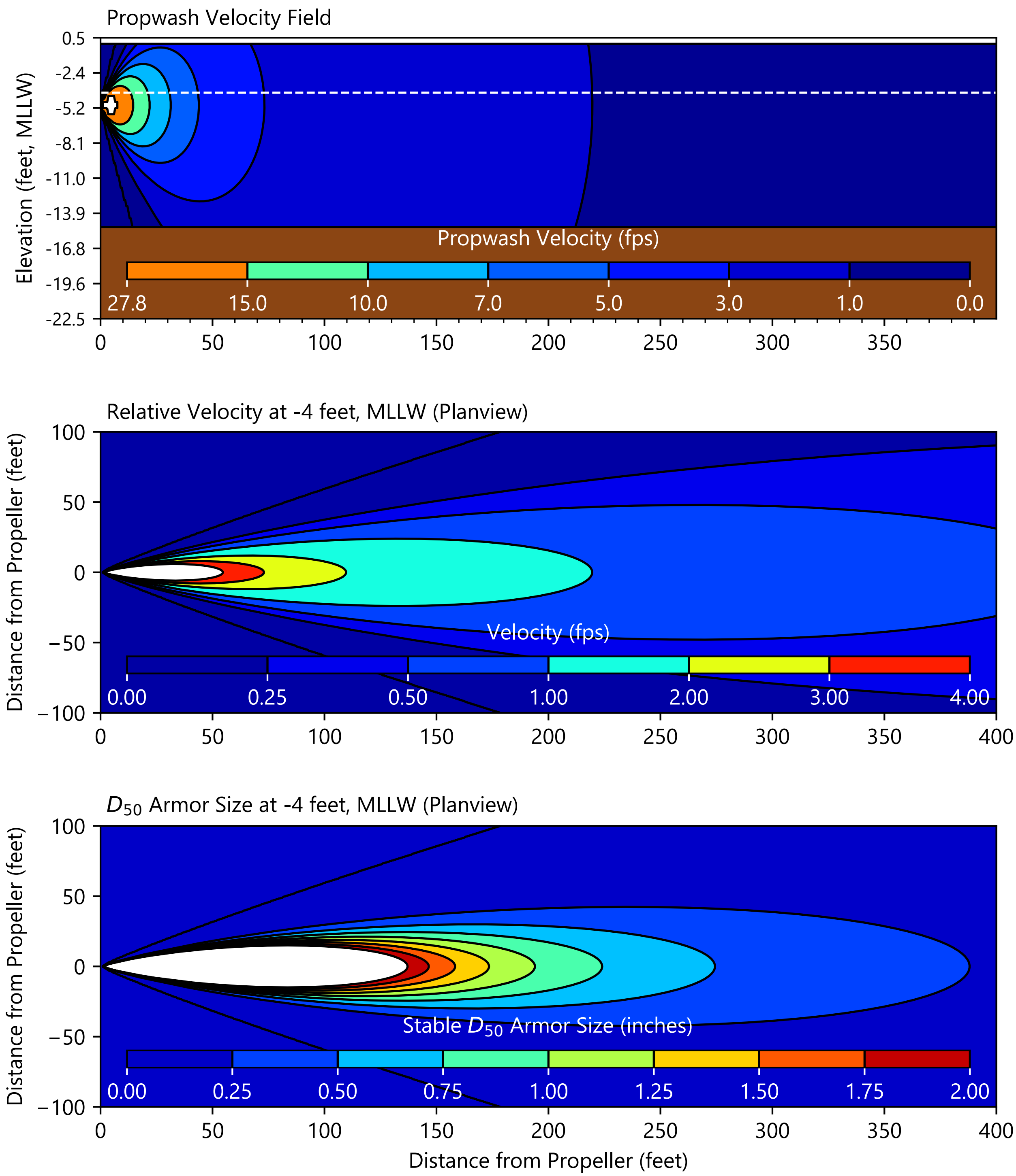
Figure 6a
Arctic Pride Yacht at 0 knots, using 30% Applied Power
Transect: Nav Channel at MHHW



Notes:
The white dashed line at -4 ft MLLW shows the location of the planview plane.
The -4 ft MLLW elevation is approximately 60 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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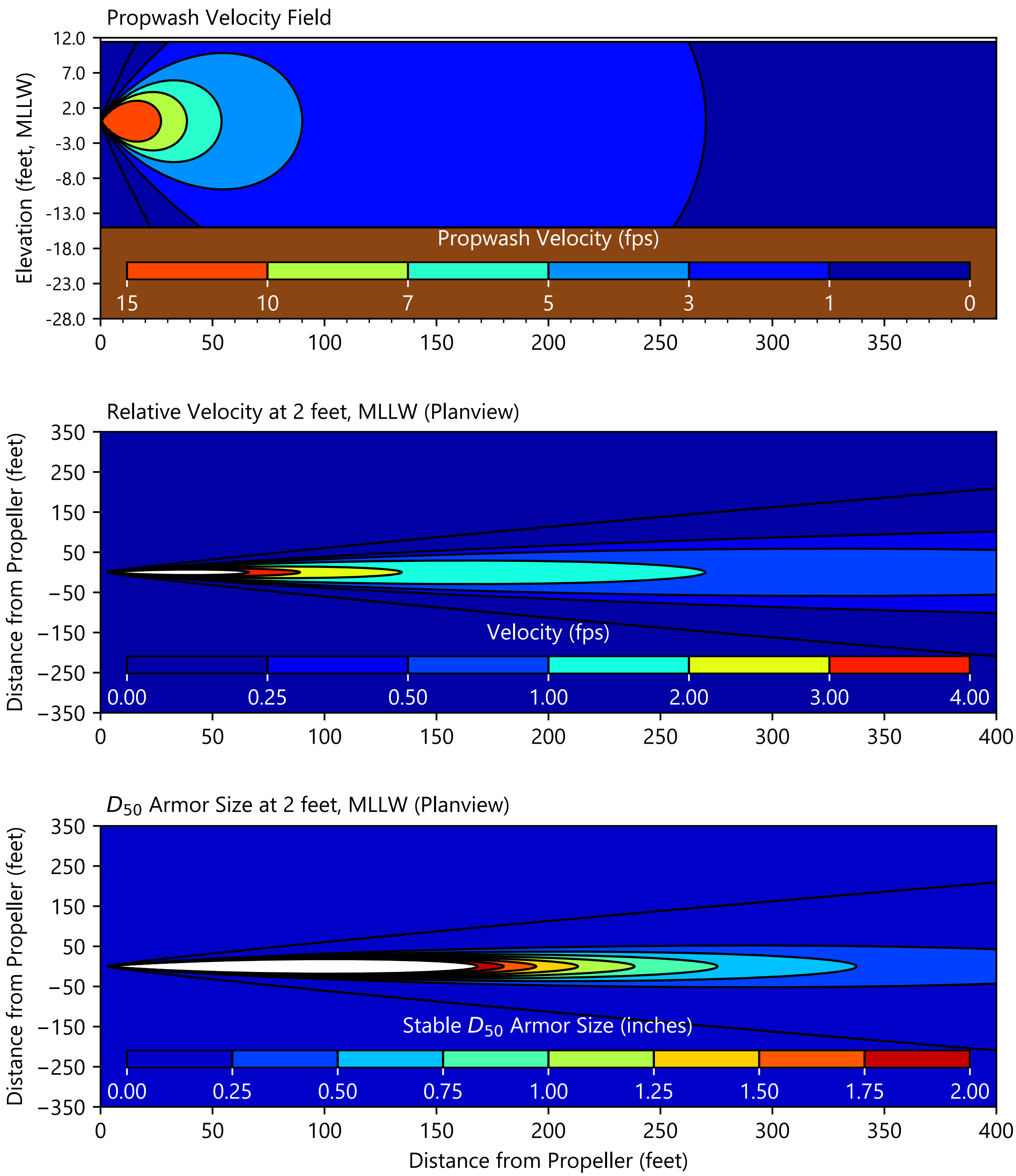
ANCHOR QEA

Figure 6b
Arctic Pride Yacht at 0 knots, using 30% Applied Power
Transect: Nav Channel at MLLW

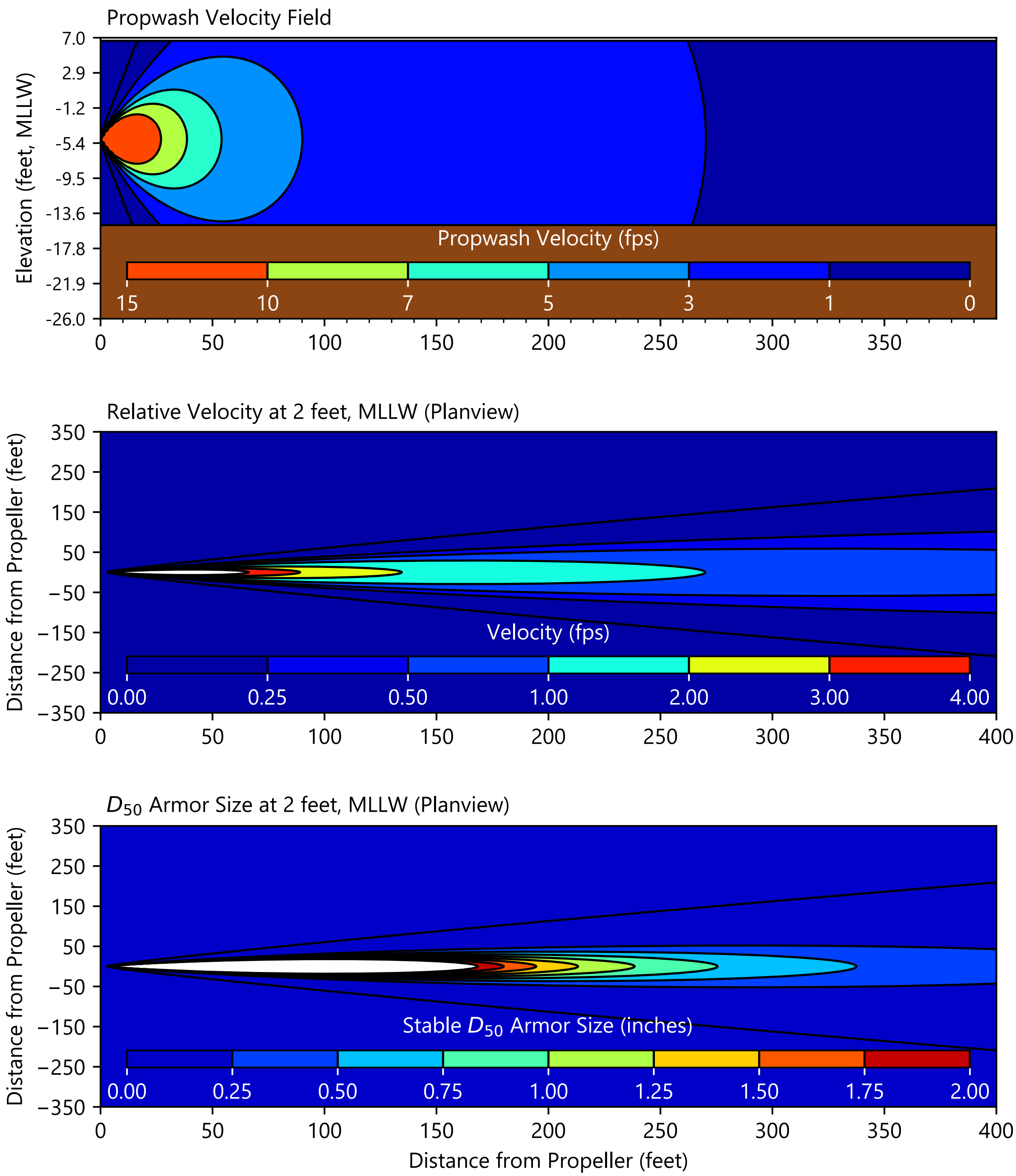
Attachment H-2
Bottom Velocity and Sediment Figures
(RAA 27)



Notes:
The +2 ft MLLW elevation is approximately 300 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

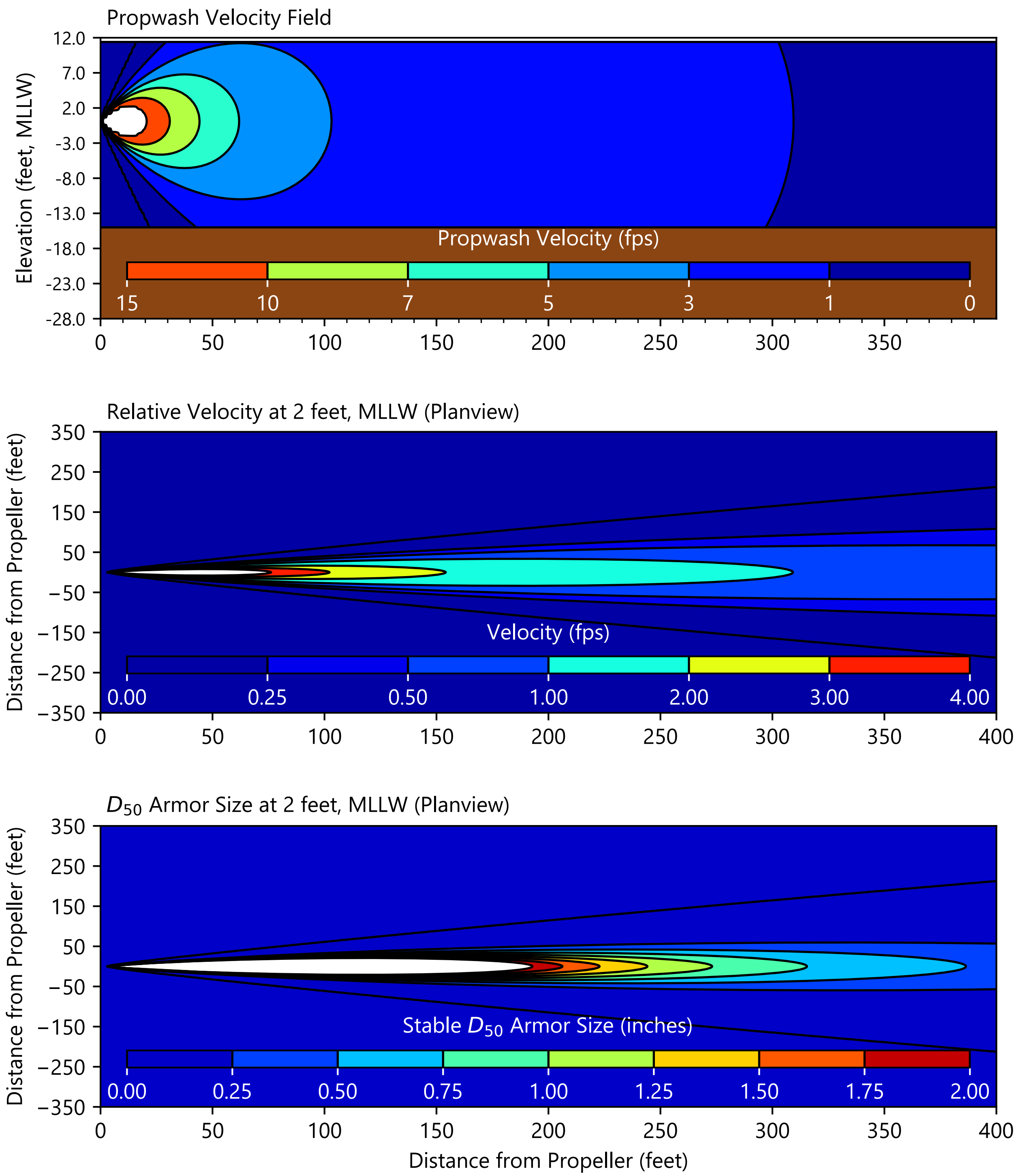
ANCHOR QEA

Figure 1a
Capt Cae Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MHHW



Notes:
The +2 ft MLLW elevation is approximately 300 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

Figure 1b
Capt Cae Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MSL



Notes:
The +2 ft MLLW elevation is approximately 300 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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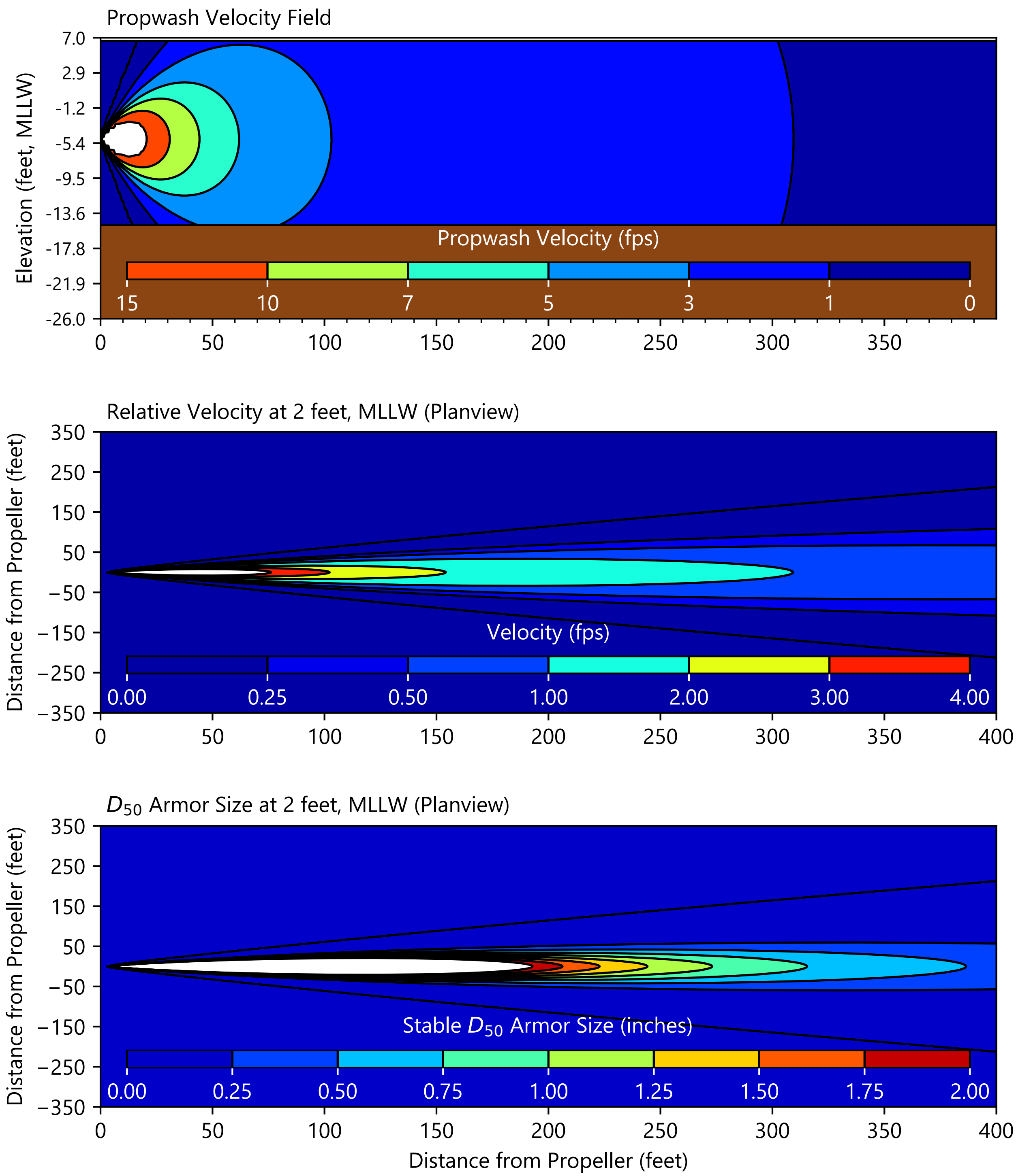
ANCHOR QEA

Figure 2a
Capt Cae Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MHHW



Notes:
The +2 ft MLLW elevation is approximately 300 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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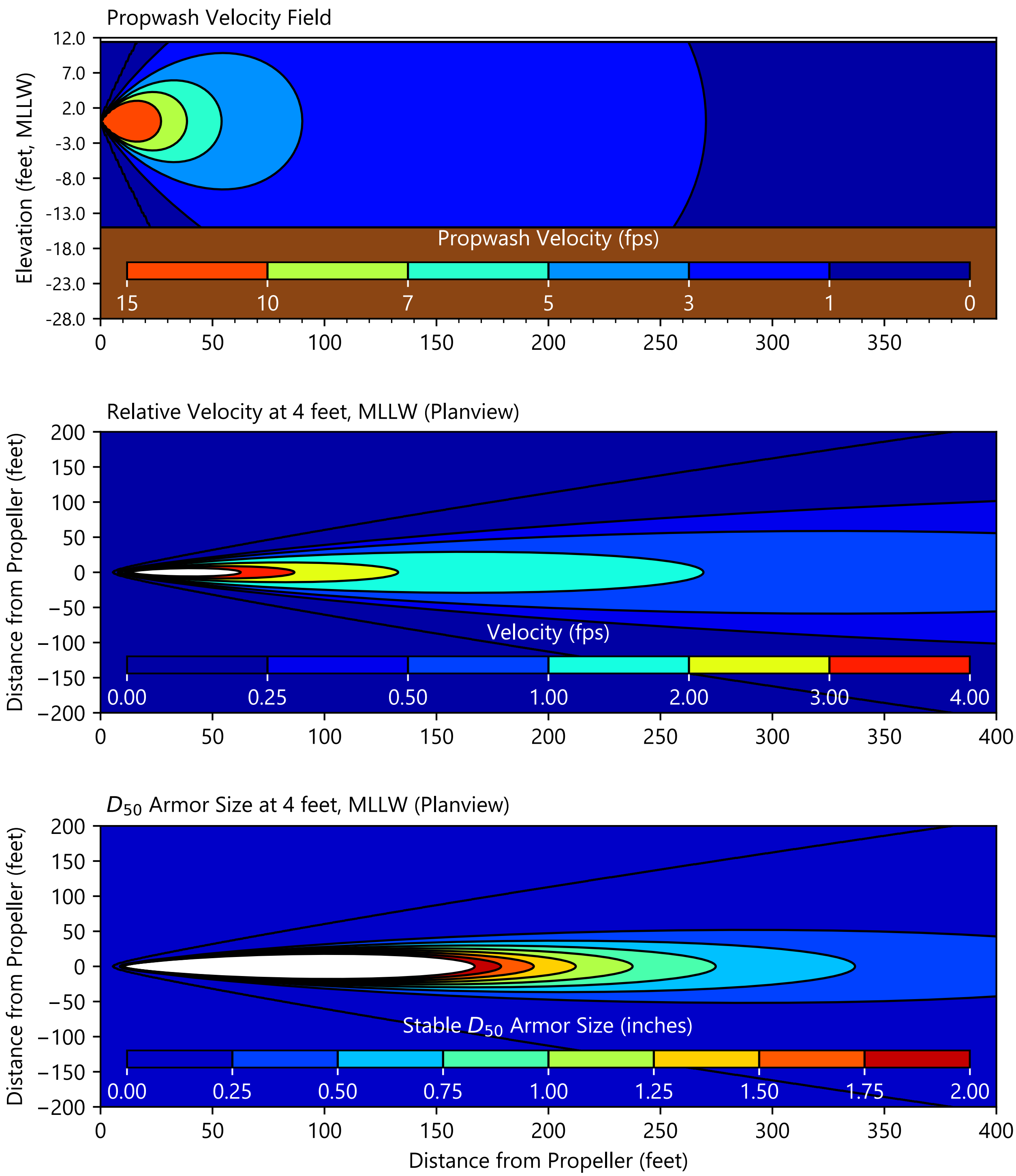
Figure 2b
Capt Cae Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MSL

Attachment H-3
Bottom Velocity and Sediment Figures
(RAAs 24 and 26)



Notes:
The +4 ft MLLW elevation is approximately 140 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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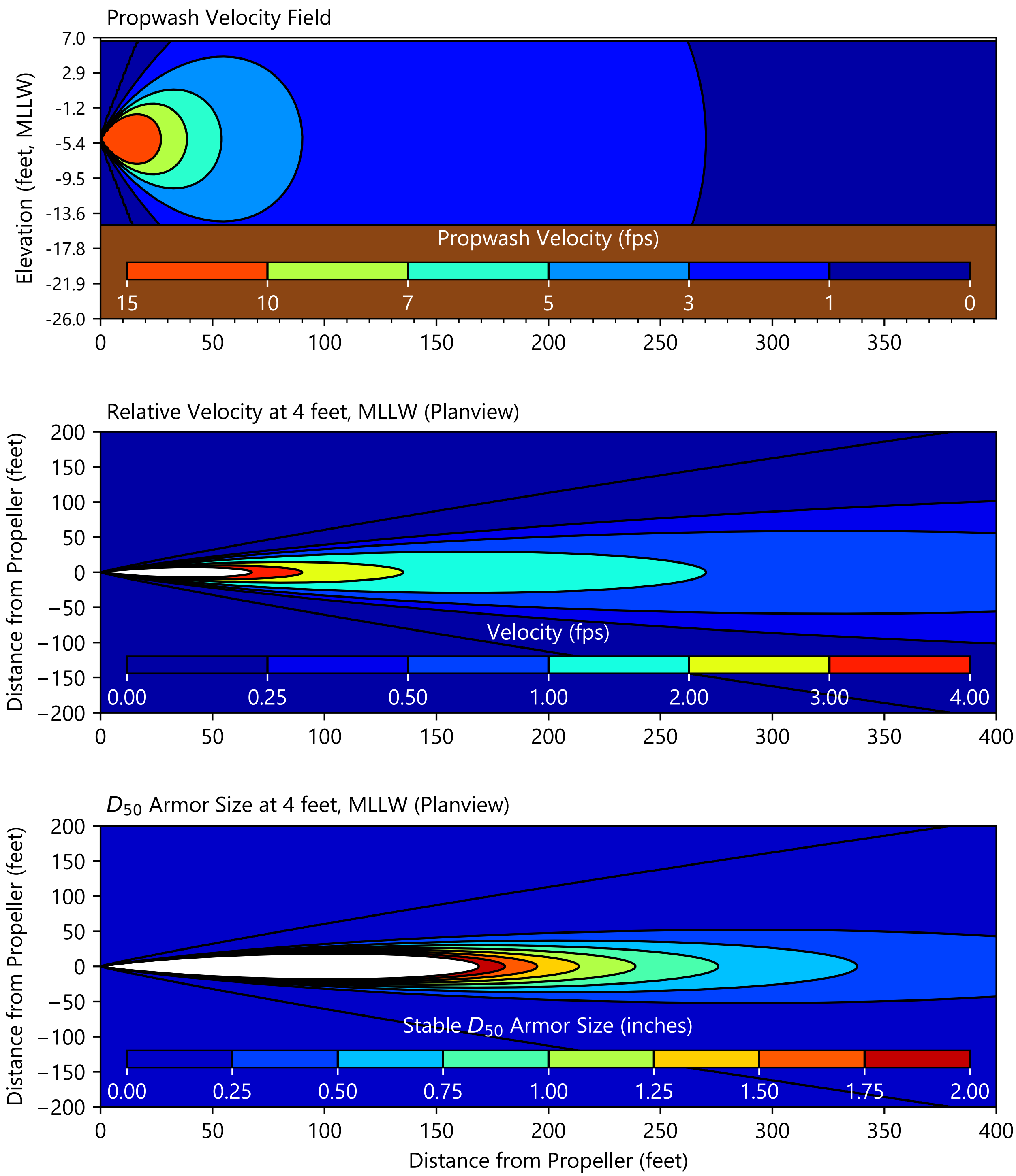
File Path: C:\Users\acannon\OneDrive - ANCHOR QEA\LDW Propwash\Areas 24 and 26\Propwash\LDW-Blaaw_and_Kaa_Propwash_Field_Analysis_v7_0-area24and26.py

Figure 1a
Capt Cae Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MHHW



Notes:
The +4 ft MLLW elevation is approximately 140 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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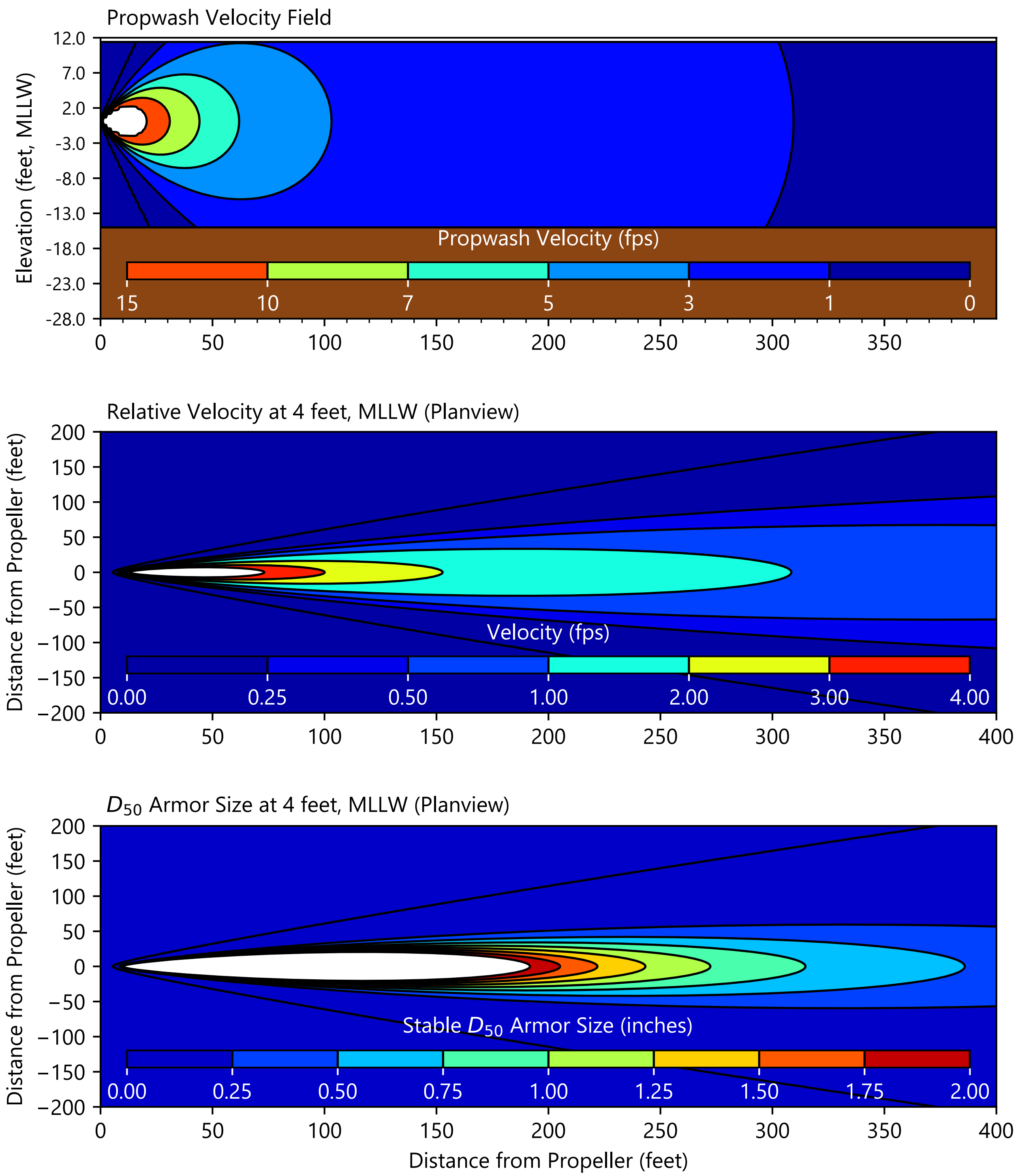
ANCHOR QEA

Figure 1b
Capt Cae Tug at 0 knots, using 20% Applied Power
Transect: Nav Channel at MSL



Notes:
The +4 ft MLLW elevation is approximately 140 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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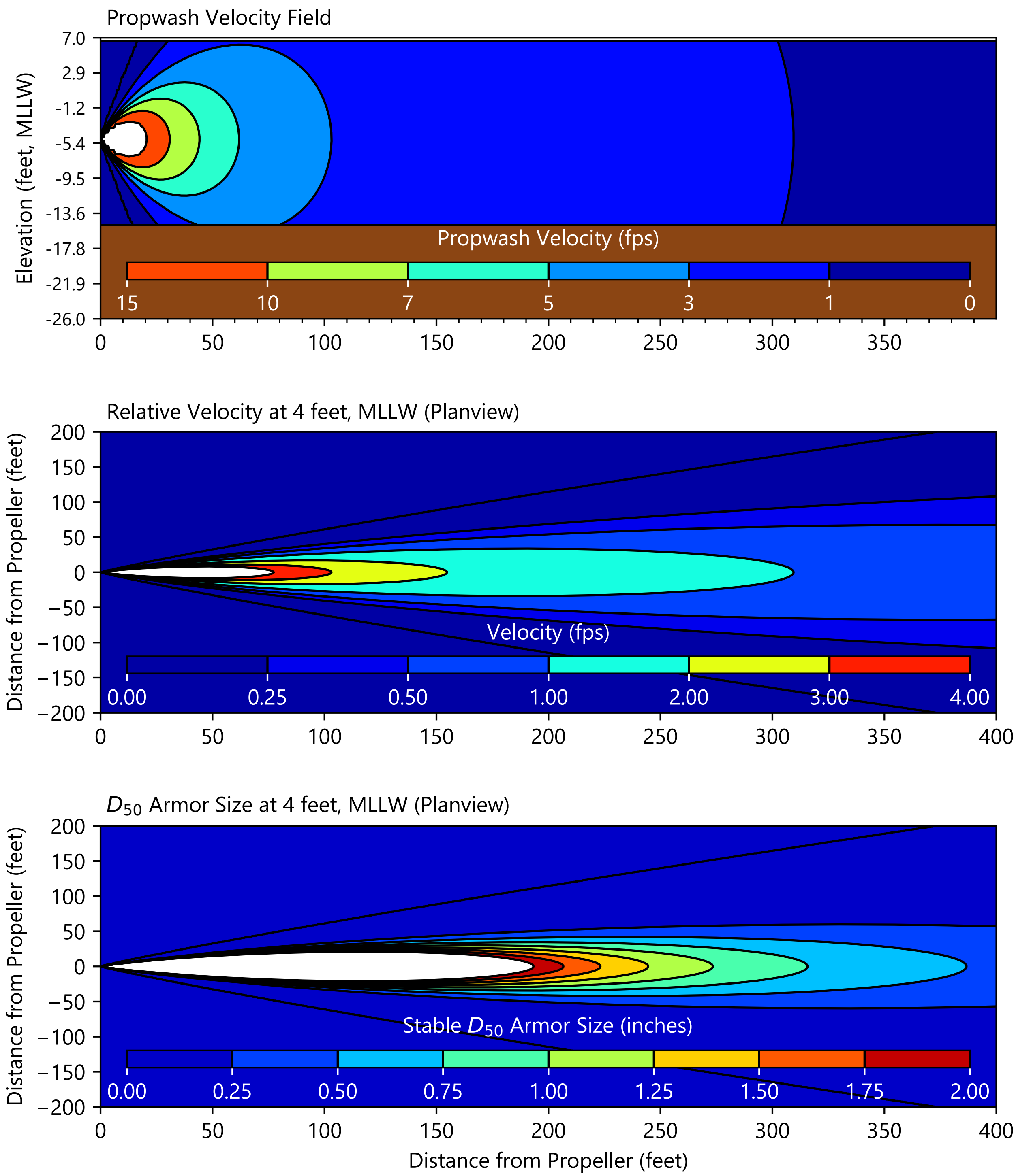
ANCHOR QEA

Figure 2a
Capt Cae Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MHHW



Notes:
The +4 ft MLLW elevation is approximately 140 feet perpendicular to the sail line.
The propwash analysis is based on the Maynord 1998 capping methodology.
The C_3 coefficient = 0.55 for no sediment movement.

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Figure 2b
Capt Cae Tug at 0 knots, using 30% Applied Power
Transect: Nav Channel at MSL