Memorandum

To: KPFF Consulting Engineers

From: Larry Karpack P.E., Shaina Sabatine P.E., Marissa Karpack E.I.T.

Date: May 9, 2017

Re: North Shore Levee, Aberdeen and Hoquiam, WA - Hydraulic Analysis and Floodplain Mapping

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INTRODUCTION

Watershed Science & Engineering (WSE) was retained by KPFF Consulting Engineers on behalf of the Cities of Aberdeen and Hoquiam to provide hydraulic engineering support for design of the North Shore Levee in Aberdeen and Hoquiam, WA. The proposed North Shore Levee will provide flood protection for areas of the Cities located north of Grays Harbor, west of the Wishkah River, and east of the Hoquiam River. Figure 1 shows the project area, location of the proposed levee, and the potential flooding sources.

Portions of the Cities of Aberdeen and Hoquiam that are within the "effective" Federal Emergency Management Agency (FEMA) 100-year floodplain but would be protected from flooding by the North Shore Levee will be eligible for a map revision which will change the 1% annual chance flood (base flood) boundary. A Conditional Letter of Map Revision (CLOMR) request is being prepared for submittal to FEMA to show the effect that the proposed North Shore Levee will have on the floodplain extents. To support the design and accreditation of the proposed levee and the CLOMR submittal, WSE conducted analyses to determine the base flood elevation (BFE) and hydraulic effects of the proposed levee, prepared floodplain mapping for the with-levee condition, and evaluated the potential for scour along the levee.

There are four potential flooding sources near the proposed North Shore Levee; Grays Harbor (coastal flooding) and the Chehalis, Wishkah, and Hoquiam Rivers (riverine flooding). Therefore, both coastal and riverine hydraulic conditions were evaluated to determine the BFE for the proposed levee. The hydraulic analyses are described below.

FLOODING SOURCES AND PAST STUDIES

As noted above, there are four potential flooding sources in proximity to the proposed North Shore Levee: Grays Harbor, Chehalis River, Wishkah River, and Hoquiam River. All of these flooding sources have been previously studied and mapped by FEMA. The effective floodplain mapping for Grays Harbor coastal flooding was recently updated as part of the Grays Harbor County Flood Insurance Study (effective February 3, 2017). The effective mapping for the Chehalis River was developed based on a 1978 study by the US Army Corps of Engineers (Corps). The Chehalis River mapping is currently under review by FEMA. The effective floodplain mapping for the Wishkah River was developed based on a hydraulic study by the CH2M Hill in 1981. Data from that study were obtained from FEMA archives and used for the current work. The effective floodplain mapping for the Hoquiam River was completed in 1975. Data, modeling, and mapping from that study are not available from the FEMA archives.

POTENTIAL FOR COASTAL FLOODING FROM GRAYS HARBOR

Tidal flood frequency analyses for Grays Harbor were recently updated by STARR for FEMA. STARR determined the 100-year tidal flood elevation in Grays Harbor to be 12.0 feet $NAVD_{88}$[1]. STARR further determined the Coastal Total Water Level (TWL) along the area of the proposed North Shore Levee to be 13.0 feet $NAVD_{88}$. The effective FEMA mapping for Aberdeen, Hoquiam, and Grays Harbor shows flood elevations of 13.0 feet along the shoreline. Upstream of the first railroad bridge on the Hoquiam River and the East Wishkah Street Bridge on the Wishkah River the effective maps show water levels rising to 14.0 feet $NAVD_{88}$. It is believed that the change from the 13.0 to 14.0 foot BFE is a mapping error as described in detail in Appendix A.

The proposed North Shore levee will not affect coastal stillwater elevations as stillwater elevations are not affected by land features. The proposed North Shore levee is also set quite far back from the shoreline of Grays Harbor. As such the proposed levee will not affect any of the transects used in the recent FEMA Coastal mapping study and therefore the proposed levee will not affect TWL. Since the proposed levee will not affect either stillwater or total water levels in Grays Harbor no new coastal analyses were required for the CLOMR.

POTENTIAL FOR RIVERINE FLOODING FROM THE CHEHALIS RIVER

The Chehalis River flows west through Aberdeen into Grays Harbor. The river separates the main downtown area of Aberdeen from South Aberdeen as shown in Figure 1. The effective FEMA mapping for the Chehalis River was developed based on a 1978 hydraulic study by the Corps. Model information from the effective Chehalis River study was requested from FEMA but nothing was available. More recently the lower Chehalis River has been modeled in detail for the Chehalis River Basin Flood Authority (CRBFA) and Corps (WSE, 2012) and again for the CRBFA in 2014 (WSE, 2014). The Chehalis River hydraulic model developed and refined in those studies is currently being used by FEMA’s consultant Strategic Alliance for Risk Reduction (STARR) to prepare updated floodplain maps for the Chehalis River. While those maps are not yet final, and the current work by STARR does not yet supersede the effective study, the recent modeling does provide a reliable depiction of flood water levels and inundation extents within Aberdeen.

The latest hydraulic modeling done for the CRBFA simulated the 100-year flood on the Chehalis River. That model used elevation 8.47 feet $NAVD_{88}$, or mean higher high water (MHHW) in Grays Harbor, as its downstream boundary. The CRBFA modeling showed 100-year Chehalis River water levels ranging from 8.5 to 9.6 feet $NAVD_{88}$ from Grays Harbor to the Highway 101 Bridge. The CRBFA modeling and analysis is a reasonable representation of Chehalis River flooding under moderately high tidal conditions. Under

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[1] All surveying and design work for the proposed North Shore Levee is being performed in the $NAVD_{88}$ datum. Elevations cited herein are in $NAVD_{88}$ with any necessary conversions from $NGVD_{29}$ based on a +3.5 foot conversion factor.

higher tide conditions the effects of Chehalis River flows are drowned out by tidal water surface elevations. As shown on the effective FEMA mapping (Figure 2), the highest water levels on the Chehalis River through Aberdeen are totally controlled by coastal water surface elevations in Grays Harbor.

POTENTIAL FOR RIVERINE FLOODING FROM THE WISHKAH RIVER

The Wishkah River flows south to its confluence with the Chehalis River in Aberdeen. The river separates the main downtown area of Aberdeen from East Aberdeen as shown in Figure 1. Water surface elevations taken from the modeling done for the effective FEMA study of the Wishkah River (CH2M Hill, 1981) indicate that in a 100-year event some flow could overtop the right bank of the Wishkah River between the Market Street Bridge and Arthur Street (RM 0.4 to RM 1.3) and flow southwest toward downtown Aberdeen. The proposed North Shore Levee, and particularly the portion of the levee upstream of Market Street, would prevent Wishkah River overflows from reaching downtown Aberdeen. Although the Wishkah River does not produce the controlling BFE along the levee (the coastal BFE described above is higher) a hydraulic analysis was conducted to evaluate the effects of the proposed North Shore Levee on Wishkah River hydraulic conditions. Changes made to the effective Wishkah River model for use in the new hydraulic analysis are outlined in Appendix B. For detailed explanation of the methods and results of the hydraulic analysis see Appendix C.

POTENTIAL FOR RIVERINE FLOODING FROM THE HOQUIAM RIVER

The Hoquiam River flows south to its mouth at Grays Harbor as shown in Figure 1. Supporting data for the effective FEMA study of the Hoquiam River could not be located by FEMA and thus it is not possible to review the modeling or analysis. However, the effective study indicates that the entire length of the Hoquiam River, at t least as far upstream as the Little Hoquiam River, is controlled by backwater flooding from Grays Harbor. The proposed North Shore Levee would prevent Hoquiam River overflows (from tidal or riverine sources) from reaching the portion of the City of Hoquiam east of the Hoquiam River. A hydraulic analysis was conducted to evaluate the effects of the proposed North Shore Levee on Hoquiam River hydraulic conditions. The finding of the hydraulic analysis of the Hoquiam River is that the highest water levels in this area are controlled by the coastal BFE. For detailed explanation of the methods and results of the hydraulic analysis see Appendix C.

BASE FLOOD ELEVATION AND LEVEE FREEBOARD

Coastal flooding from Grays Harbor produces the controlling BFE throughout the project area. The 100-year BFE from Total Water Level (TWL) is 13.0 feet and the 100-year stillwater level is 12.0 feet throughout the area of the proposed North Shore Levee. The 100-year TWL was used by FEMA to map the 100-year floodplain throughout Aberdeen and Hoquiam. FEMA requirements for levee freeboard in coastal floodplains require a minimum of one foot of freeboard above TWL or two feet of freeboard above the stillwater level (USACE, 2010). Therefore, the required design crest elevation for the North Shore Levee must be 14.0 feet $NAVD_{88}$ or higher.

EFFECTS OF THE LEVEE ON THE 100-YEAR FLOODPLAIN

GRAYS HARBOR

The proposed North Shore Levee is 8.39 miles long, extending from A Street in Hoquiam on its west terminus to Arthur Street in Aberdeen on its east terminus. The top of the waterward side of the levee is as depicted in Figure 1. Without the levee, some portions of Aberdeen and Hoquiam could be flooded due to high tidal water levels during a 100-year coastal flood. With the levee, a significant portion of downtown Aberdeen and east Hoquiam will be protected from flooding as shown in Figure 2. The levee will prevent flooding from any of the external flooding sources (Grays Harbor, Chehalis River, Wishkah River, Hoquiam River) and the interior drainage is designed to completely drain the 100-year storm, as described in the interior drainage analysis report (KPFF, 2017).

By preventing flooding in Aberdeen and Hoquiam the proposed North Shore Levee will reduce floodplain storage during extreme high tide events. However, because the amount of floodplain storage lost due to the placement of the North Shore Levee is insignificant relative to the immense volume of water in a tidal cycle, the proposed North Shore Levee will not affect base flood elevations seaward of the levee.

CHEHALIS RIVER

Hydraulic modeling of the Chehalis River shows it would not overtop its right bank during a 100-year flood. Thus the Chehalis River would not, by itself, flood the area of the proposed levee. Therefore, the proposed levee will have no effect on Chehalis River hydraulic conditions.

WISHKAH RIVER

Updated hydraulic modeling of the Wishkah River under existing conditions shows it would not overtop its right bank anywhere along the proposed levee in the 100-year riverine flood. Thus the proposed levee has no effect on Wishkah River hydraulic conditions. Tidal flooding progressing up the Wishkah River could, however, cause overbank flooding along the Wishkah River. The eastern side of the proposed North Shore Levee follows the right bank of the Wishkah River north up until South E Street and then generally follows the alignment of East Market Street until Stanton Street, where it follows the right bank of the Wishkah River until Arthur Street. Hydraulic analysis indicates that water levels due to tides backing up the Wishkah River would not be significantly affected by the proposed levee (e.g. water surface elevation differences in WSE’s estimated 100-year tide of 0.02 feet or less).

The Wishkah River hydraulic analysis included an evaluation of the Wishkah River floodway. Analysis of the floodway found it to be fully contained within the river channel (i.e the floodway encroachments can be delineated at the channel banks) and thus the proposed levee, which is wholly landward of the channel banks will have no effect on the floodway. Furthermore, within the entire modeled length of the Wishkah River (to river mile 2.2), the coastal BFEs exceed the riverine BFEs and therefore the floodway would not be shown on revised FEMA floodplain mapping.

HOQUIAM RIVER

Hydraulic modeling of the Hoquiam River under existing conditions shows it would not overtop its left bank at any location during a 100-year riverine flood. Thus the proposed levee has no effect on Hoquiam River hydraulic conditions. The western side of the proposed North Shore Levee follows the left bank of the Hoquiam River to Aberdeen Avenue, where it follows 21st Street to Highway 101 West. It then follows Highway 101 West to the left bank, which it follows until just north of A Street.

LEVEE SCOUR POTENTIAL

POTENTIAL FOR SCOUR FROM THE WISHKAH RIVER

Flow velocities along the Wishkah River portion of the proposed North Shore Levee during a 100-year event were analyzed using the proposed conditions HEC-RAS model. In general, velocities along the levee are negligible (<0.2 fps). The maximum simulated velocity was 1.3 fps, which occurred along the floodwall portion of the levee upstream of Market Street. Such low velocities are not expected to cause any scour and as such existing or proposed vegetative cover is adequate to mitigate any potential erosion.

POTENTIAL FOR SCOUR FROM THE HOQUIAM RIVER

Flow velocities along the Hoquiam River portion of the proposed North Shore Levee during a 100-year event were analyzed using the proposed conditions HEC-RAS model. In general, velocities along the levee are negligible (<0.2 fps). The maximum simulated velocity was 0.9 fps, which occurs near Riverside Avenue. Such low velocities are not expected to cause any scour and as such existing or proposed vegetative cover is adequate to mitigate any potential erosion.

SUMMARY

Conclusions drawn from hydraulic analysis are presented here by WSE to support design of the North Shore Levee in Aberdeen and Hoquiam, WA and future FEMA accreditation/CLOMR mapping for the project. The proposed levee will provide flood protection for part of the Cites of Aberdeen and Hoquiam, reducing the Special Flood Hazard Area (SFHA) within the cities. The 100-year base flood elevation along the levee is controlled by coastal flooding from Grays Harbor which has a total water level of 13.0 feet $NAVD_{88}$ (FEMA, 2017). The 100-year riverine flood on the Chehalis, Wishkah, and Hoquiam Rivers results in water surface elevations lower than 13.0 feet $NAVD_{88}$ throughout the project area. As such, the proposed North Shore Levee will not change the effective base flood elevation with the exception of the areas protected from flooding by the levee. In the interior of the levee some areas would still be subject to flooding during a 100-year event due to interior drainage as shown on Figure xx. The design crest elevation of the proposed levee will be a minimum of 14.0 feet $NAVD_{88}$, providing one foot of freeboard above the coastal TWL base flood elevation and two feet of freeboard above the coastal stillwater elevation. Actual levee design elevations may be higher to provide additional freeboard, to allow for settling of the levee, and/or to account for potential effects of climate change.

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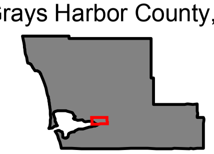
Grays Harbor County, WA



North Shore Levee
Project Area and Proposed Levee Alignment

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Scale: 1:31,295
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09 May 2017

WATERSHED SCIENCE & ENGINEERING

Figure 1

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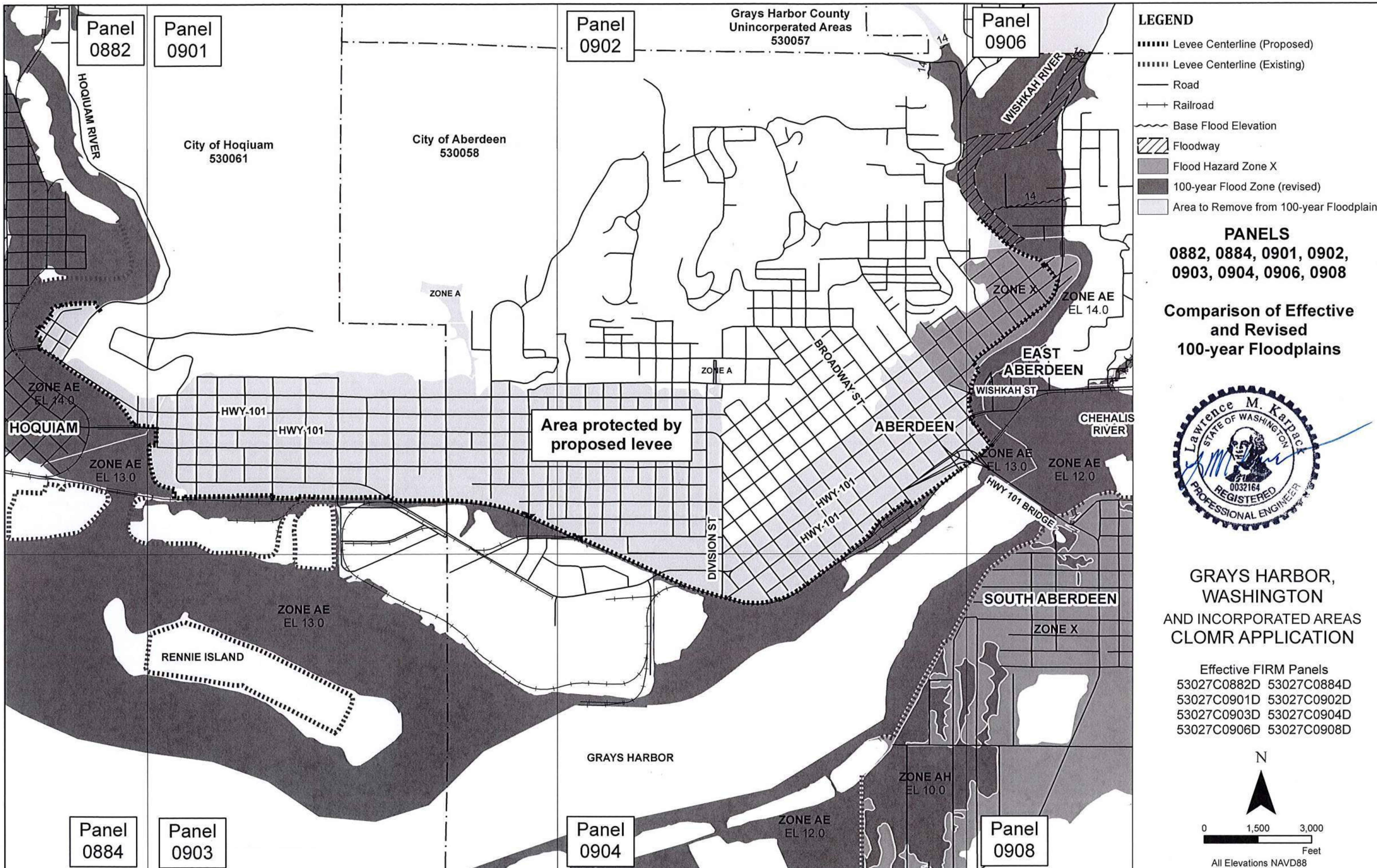
Panel 0882
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Panel 0902
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Panel 0904
Panel 0908
Grays Harbor County
Unincorperated Areas
530057
City of Hoqiuam
530061
City of Aberdeen
530058
HOQUIAM RIVER
WISHKAH RIVER
CHEHALIS RIVER
HOQUIAM
ABERDEEN
EAST ABERDEEN
SOUTH ABERDEEN
GRAYS HARBOR
RENNIE ISLAND
Area protected by proposed levee
HWY 101
HWY 101 BRIDGE
BROADWAY ST
DIVISION ST
WISHKAH ST
ZONE A
ZONE X
ZONE AE EL 14.0
ZONE AE EL 13.0
ZONE AE EL 12.0
ZONE AH EL 10.0
14
LEGEND
Levee Centerline (Proposed)
Levee Centerline (Existing)
Road
Railroad
Base Flood Elevation
Floodway
Flood Hazard Zone X
100-year Flood Zone (revised)
Area to Remove from 100-year Floodplain
PANELS
0882, 0884, 0901, 0902,
0903, 0904, 0906, 0908
Comparison of Effective
and Revised
100-year Floodplains
Lawrence M. Karpack
STATE OF WASHINGTON
0032164
REGISTERED
PROFESSIONAL ENGINEER
GRAYS HARBOR,
WASHINGTON
AND INCORPORATED AREAS
CLOMR APPLICATION
Effective FIRM Panels
53027C0882D 53027C0884D
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Feet
All Elevations NAVD88

Figure 2

Appendix A: Observations on Effective FEMA Mapping near Grays Harbor

The currently effective FEMA floodplain mapping of the areas along Grays Harbor and the Wishkah and Hoquiam Rivers near Hoquiam and Aberdeen WA appears to be inconsistent with the effective flood studies for Grays Harbor and the Wishkah River. The current maps reflect numerous different studies completed from the early 1970s through 2016. Based on a thorough review of the data sources and studies used to prepare the current FEMA maps the following observations were made:

1) Prior to the recent release of the new effective study for Grays Harbor County (on February 3, 2017) the study area we are interested in for the North Shore Levee project (as shown on Figure 1) was all mapped based on either a coastal floodplain mapping study conducted by the US Army Corps of Engineers in the 1970s or a riverine hydraulic study of the Wishkah River conducted in the early 1980s by CH2M Hill. Figure 2 shows a FIRMETTTE produced from the original FEMA mapping that was replaced by the new mapping earlier this year. Of particular note on this map is that the entire mapping area, up to Arthur Street on the Wishkah River, is mapped at elevation 10 feet NGVD (which corresponds to 13.5 feet NAVD). This is the 100-year coastal flood elevation determined by the USACE in their 1970s era study. It is also significant to note that there is a white "gutter line" on the Wishkah River near Arthur Street near the first BFE on the Wishkah River. This gutter line is likely the breakpoint between the Coastal mapping and the Riverine mapping.

2) Hydraulic modeling for the Wishkah River Study was performed in 1981 by CH2M Hill using HEC-2. The modeling assumed a downstream boundary condition on the Wishkah River equal to the 10-year coastal flood or 8.8 feet NGVD (which corresponds to 12.3 feet NAVD). Using that starting water level the Wishkah River model reached an elevation of 10 feet NGVD between model cross sections C and D which is at about the same location as the 10 foot NGVD BFE on the Wishkah River. This fact supports the assumption that the gutter line shown on the earlier effective maps was there to differentiate between the coastal and riverine studies.

3) At some point the earlier effective studies were converted from NGVD to NAVD. This was done by adding the conversion factor (approximately 3.5 feet) and then rounding the converted elevations to whole foot increments. Thus the 10 foot NGVD BFE on the original FEMA maps was converted to 14 feet NAVD. Figure 3 shows a section of the new effective maps showing a 14 foot NAVD BFE line in approximately the same location as the previous 10 foot NGVD BFE, just downstream of Wishkah River model cross section D.

4) In about 2013 FEMA contracted with STARR to produce a new Coastal Floodplain Mapping study of the southwest Washington Coast including Grays Harbor. The STARR Analysis determined a 100-year stillwater level of 12.0 feet in Grays Harbor and a 100-year Total Water Level (TWL) of 13.0 feet for the shoreline between Hoquiam and Aberdeen. The 13.0 foot TWL was mapped as the new Coastal BFE throughout much of the Aberdeen Hoquiam North Shore Levee project area. This included the north shore of Grays Harbor between Hoquiam and Aberdeen and

extending up the Hoquiam River to the first railroad bridge (see Figure 4) and up the Wishkah River to the East Wishkah Street Bridge (see Figure 5). Gutter lines on the new effective maps indicate that area upstream of these bridges were mapped based on some other data source.

5) Upstream of the East Wishkah Street Bridge on the Wishkah River, the new effective maps show a BFE of 14 feet NAVD. The Flood Insurance Study Report indicates that the upstream mapping was done based on riverine studies. However, the above observations suggest the mapping upstream of East Wishkah Street to Arthur Street on the Wishkah River is actually a remnant of the earlier coastal study (with the rounded conversion from NGVD to NAVD). Upstream of the 14 foot BFE shown in Figure 3 the new mapping corresponds fairly closely to the 1981 Wishkah River model.

6) Upstream of the railroad bridge on the Hoquiam River the new effective maps also show a BFE of 14 feet NAVD. The FIS Report does not identify the source of these data. Unfortunately FEMA does not have documentation or any of the modeling and analysis conducted for the Hoquiam River so it is not possible to review that to see where the riverine hydraulics would start to have an effect on Hoquiam River BFEs. However, new riverine hydraulic analyses conducted for the Aberdeen-Hoquiam North Shore Levee indicates that the tidal boundary condition controls water levels in the Hoquiam River to at least 3 miles upstream of the mouth.

7) Although the 1981 Wishkah River Hydraulic Model study used the 10-year tide level as a downstream boundary condition current FEMA guidance (see screen shot below) suggests that for modeling of river-coastal zones the downstream boundary of the riverine model should be taken as Mean Higher High Water (which in Grays Harbor is 8.47 feet NAVD). Modeling of the Wishkah and Hoquiam Rivers using a downstream boundary condition of MHHW would push the boundary between the coastal and riverine mapping even further upstream, such that the entire floodplain within the proposed Aberdeen-Hoquiam North Shore Levee Project area would be well within the Coastal Mapping Zone, with the newly determined 13.0 foot BFE.

Guidelines and Specifications for Flood Hazard Mapping Partners [November 2009]

- The arrival times of flood peaks are similar for the two combining watersheds; and
- The likelihood of both watersheds being covered by the storm being modeled is high.

If gage records are available for the basins, the Mapping Partner performing the hydraulic analysis should obtain guidance from the RPO on coincidence of peak flows using streamflow records.

When the downstream boundary of a modeled stream is within a coastal tidal reach, the tidal boundary of the model is taken as equal to the Mean Higher High Water (MHHW) level of the nearby tide station. Location of tide station(s) must be verified to represent true downstream conditions. The tide level can be transferable to other locations along open coast; however, tide level at an estuary station is not transferable to locations beyond the estuary.

Significance of Above Observations for North Shore Levee Hydraulic Analysis

Given the above observations, it can be concluded that the entire area of the proposed levee project is within the coastal flood study area and not within a riverine study area. The proposed Aberdeen-Hoquiam North Shore Levee will not cause any changes to coastal stillwater or total water levels in Grays Harbor because the proposed levee is set sufficiently far from the shoreline as to not be subjected to any waves (and stillwater levels are independent of shore features). Thus, no revised modeling or analysis of the Coastal Floodplain is needed.

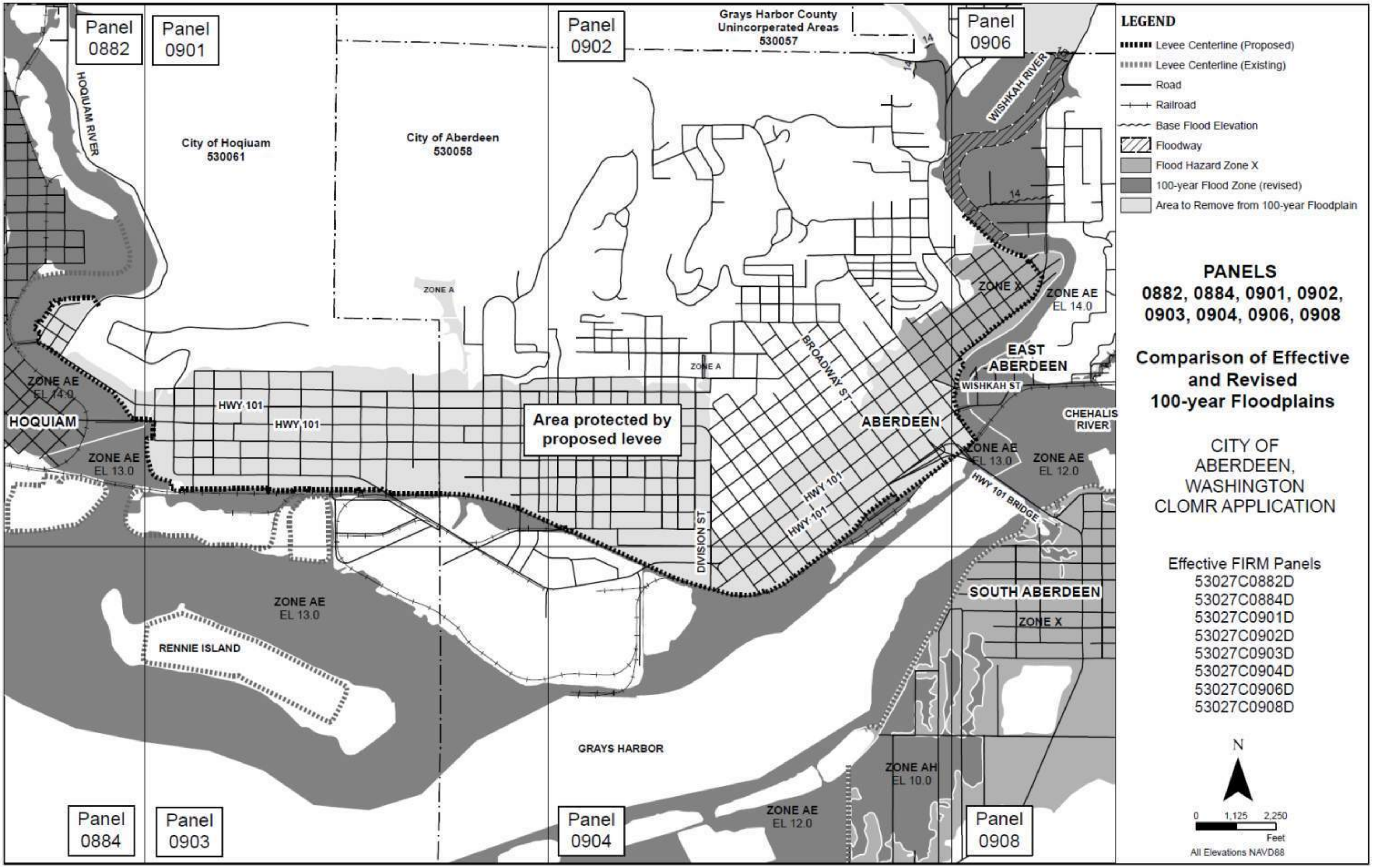


Figure 1: Study Area for Proposed Aberdeen-Hoquiam North Shore Levee

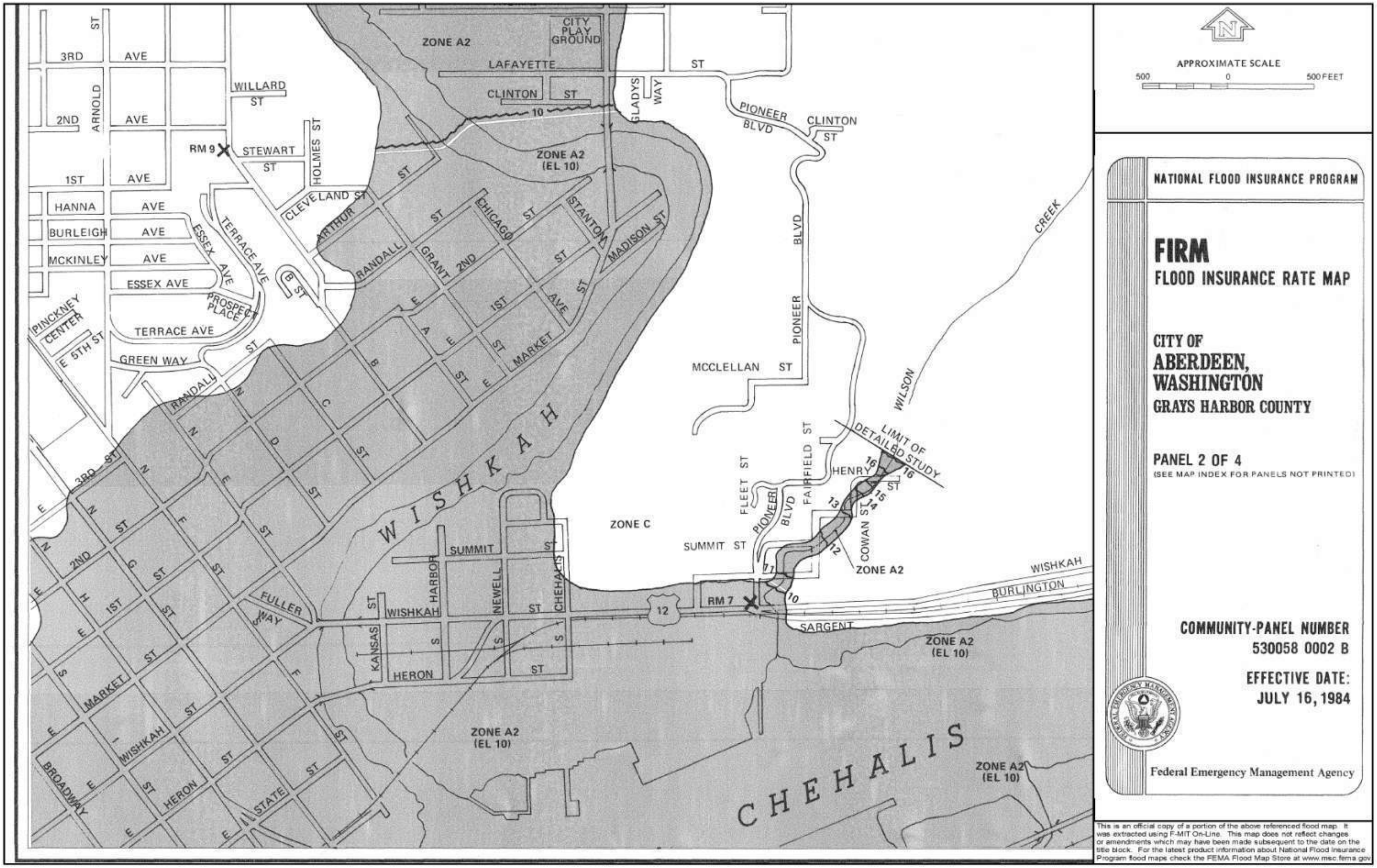


Figure 2: FIRMETTE for Wishkah River portion of study area based on previously (pre 2017) effective FIS

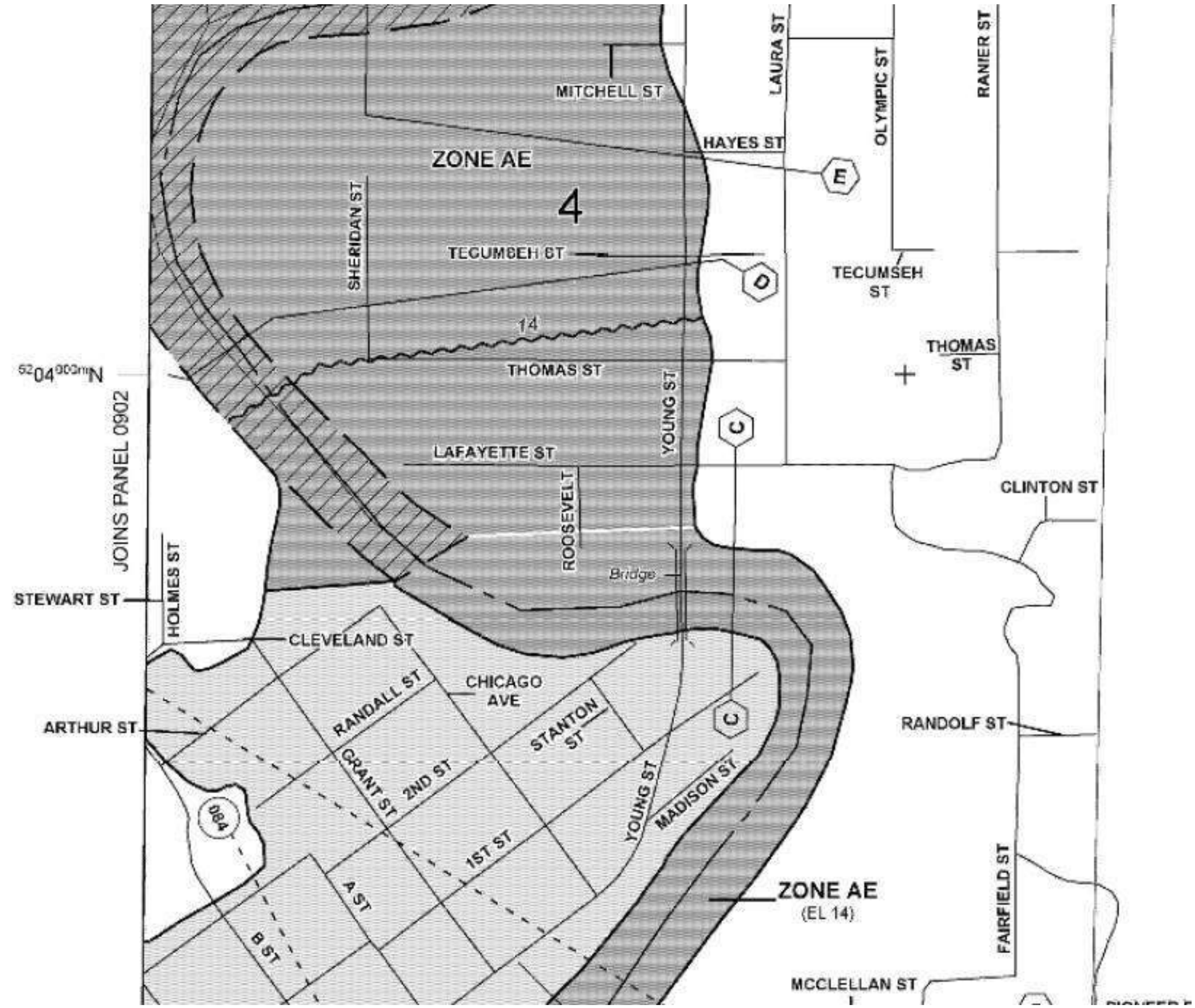
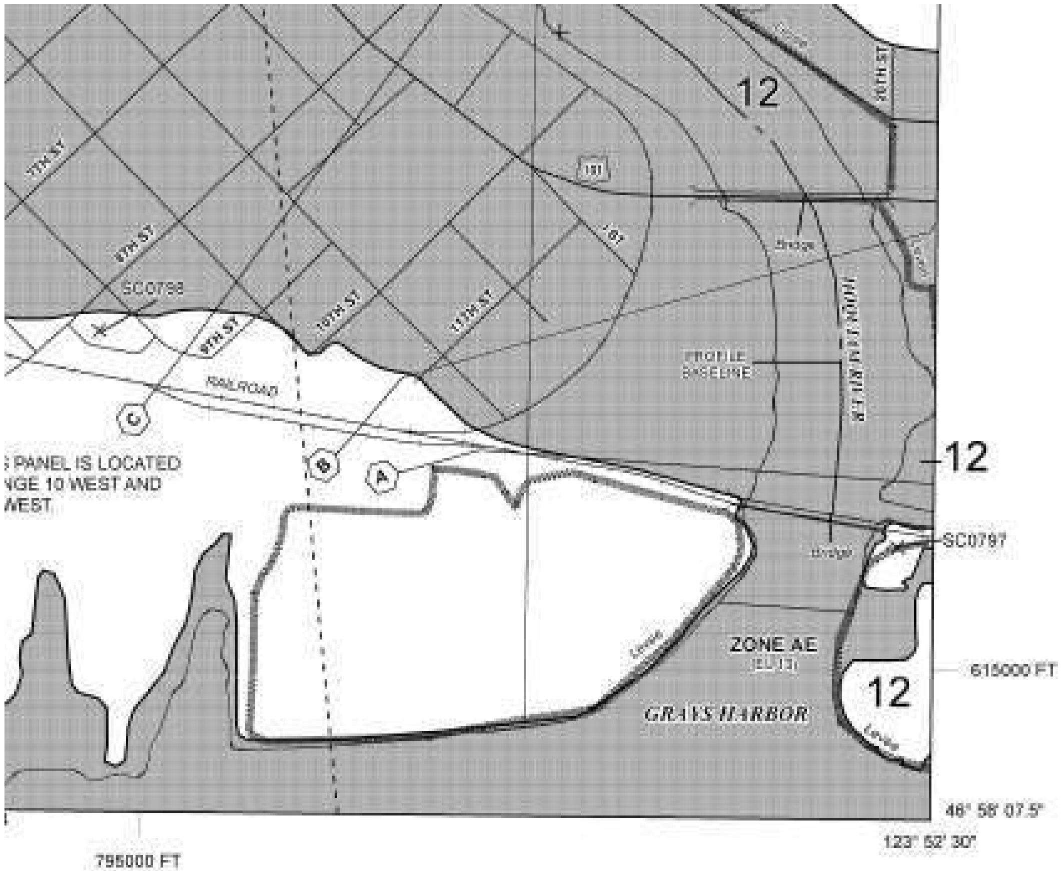
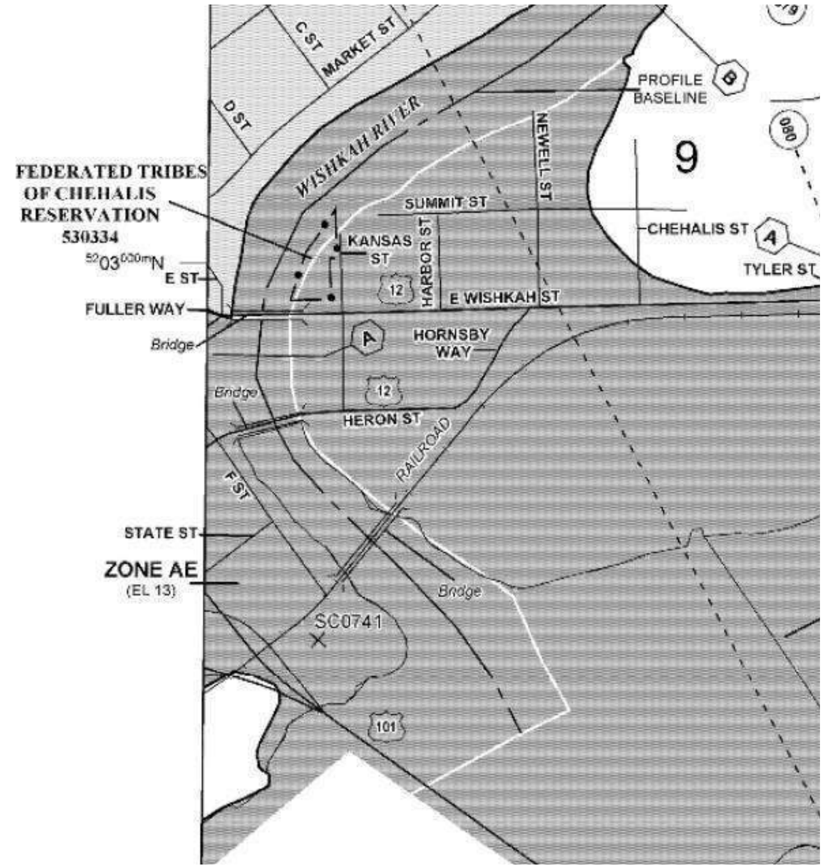


Figure 3: Screen Shot of February 3, 2017 Effective FEMA Maps showing 14 Foot BFE





Figures 4 and 5: New Effective Mapping for Grays Harbor showing 13.0 foot BFE extending upstream on Hoquiam and Wishkah Rivers

APPENDIX B: WISHKAH RIVER HEC-RAS MODELING FOR ABERDEEN HOQUIAM NORTH SHORE LEVEE

Data were obtained from the FEMA Archives for the Wishkah River from the Aberdeen Flood Insurance Study. Hard copy data in PDF format were obtained. Watershed Science and Engineering used these data as follows:

1. PDF file of HEC-2 model inputs were hand entered into Excel file “Wishkah HEC-2 Model Cross sections.xlsx”
2. Model data were exported from Excel to a text file named “Wishkah.dat”
3. HEC-2 data in Wishkah.dat were imported into a new HEC-RAS Version 4.1 Project named “WishkahRiver.prj”
4. Duplicate Effective FEMA Model was created as contained in the following HEC-RAS files:
   - Plan File: WishkahRiver.p01
   - Geometry File: WishkahRiver.g01
   - Flow File: WishkahRiver.f01
5. Corrected Effective Model was created by adjusting all elevation data from NGVD to NAVD using a correction of 3.51 feet. Additional flow data based on new USGS regression equations were included for evaluation. Corrected Effective Model is contained in the following HEC-RAS files:
   - Plan File: WishkahRiver.p02
   - Geometry File: WishkahRiver.g02
   - Flow File: WishkahRiver.f02
6. Existing Conditions model was created by adding additional interpolated cross sections and adjusting overbank elevation data to match new topographic data for Aberdeen. Additional flow data were included for sensitivity analyses. Existing Conditions Model is contained in the following HEC-RAS files:
   - Plan File: WishkahRiver.p03
   - Geometry File: WishkahRiver.g03
   - Flow File: WishkahRiver.f03
7. All files described above are contained in a zip archive file named “Wishkah River HEC-RAS 4_1.zip” which is attached.
8. Because the existing conditions model did not adequately show flow at the site of the proposed Aberdeen Northside Levee a Proposed Condition Model was not created in HEC-RAS Version 4.1. Instead, the existing conditions model was converted to a HEC-RAS version 5.0.3 model and the 2D Modeling capabilities of that version of HEC-RAS were used to evaluate conditions along the proposed levee.
9. An Alternate Existing Conditions model was created by trimming the cross sections at the top of the right bank, and adding the Hoquiam River as a 1D reach. 2D flow areas were added on the overbanks, with one 2D area connecting the Hoquiam left over bank with the Wishkah right over bank. Lateral structures were added between cross sections to transfer flow from the main

channel to the overbanks. The Alternate Existing Conditions Model is contained in the following HEC-RAS files:

- Plan File: NShoreLevee.p03
- Geometry File: NShoreLevee.g01
- Flow File: NShoreLevee.u03

10. A Proposed Conditions model was created by adding the proposed Aberdeen Northside Levee to the Alternate Existing Conditions Model. The levee was incorporated by raising the surface in the terrain file for 2D computation. The Proposed Conditions Model is contained in the following HEC-RAS files:
    - Plan File: NShoreLevee.p07
    - Geometry File: NShoreLevee.g02
    - Flow File: NShoreLevee.u07
11. The Proposed Conditions Model was used to evaluate flow velocity and depth conditions along the proposed levee for design of the levee protection. It is specifically noted that since the BFEs along the proposed levee are controlled by tidal flooding the Wishkah River models were not used in any way to develop data to support the proposed CLOIMR mapping.

Appendix C: North Shore Levee Hydraulic Modeling

OVERVIEW

A levee is proposed to be built along the Wishkah and Hoquiam Rivers and Grays Harbor to provide flood protection to portions of the Cities of Aberdeen and Hoquiam in Grays Harbor County, WA (Figure 1). The levee is designed to protect landward areas from coastal or riverine flooding. A HEC-RAS hydraulic model was developed by Watershed Science & Engineering (WSE) to inform the design and evaluate potential hydraulic impacts of the proposed levee. The hydraulic model includes approximately 2.2 miles of the Wishkah River, 2.2 miles of the Hoquiam River, 3.6 miles of shoreline along Grays Harbor, and overbank areas including the portions of the cities of Aberdeen and Hoquiam that would be protected by the levee. The model was used to determine whether the 100-year recurrence interval flood event in this area is controlled by coastal (tidal) or riverine flooding.

HYDRAULIC MODEL DEVELOPMENT

A HEC-RAS combined 1D/2D model was developed for this project. The main channels of the Wishkah and Hoquiam Rivers were defined as 1D model reaches (cross sections defined with station/elevation data) and overbank areas as 2D model areas (computational cells that route flow over underlying topography). Locations of cross sections and extents of 2D areas are shown in Figure 1. This 1D/2D model configuration allowed the seven bridges in the project area to be modeled in detail (in the 1D reaches) while also routing overbank flow more accurately through the 2D areas (detailed bridge modeling capabilities are not yet available within HEC-RAS 2D model areas).

MODEL SETUP AND TOPOGRAPHIC SURFACE

The HEC-RAS 1D/2D model includes the areas of Aberdeen and Hoquiam that will be protected by the proposed levees as well as reaches of the Hoquiam and Wishkah Rivers and Grays Harbor (Figure 1). The model does not include the Chehalis River as previous analyses by WSE (WSE, 2014) and others found that Chehalis River water levels do not overtop the channel banks in this area and thus will not be affected by the proposed levee. The 1D portion of the HEC-RAS model for both the Wishkah and Hoquiam Rivers used cross section surveys completed in May 2016 by Wilson Engineering. The surveys begin at the mouth of each river and extend upstream more than 2 miles.

The topographic surface for the 2D portion of the model was created by combining a 3 foot by 3 foot gridded ground (above water) surface created using aerial photogrammetry in June 2016 by David Smith & Associates, Inc. and coastal bathymetry of Grays Harbor produced in December 2005 by the National Oceanic and Atmospheric Administration Center for Tsunami Research (NOAA, 2005). Houses and other structures mapped in the photogrammetric data were included in the topographic surface as high ground. The final topographic surface and all elevation data reported herein are referenced to the NAVD 1988 vertical datum.

Two model configurations were prepared: one for the existing condition and one for the with-levee condition. With the exception of the inclusion of the levee in the 2D model terrain, all model elements (grid cells, model parameters, boundary conditions, etc.) were kept the same between the two configurations.

ROUGHNESS

Manning's *n* values representing the surface roughness were defined throughout the model domain based on engineering judgement. For the 1D area, a single roughness value was used for the channel and another for the overbanks. For the 2D portion of the model, a composite value was defined in overbank areas to represent a wide variety of land cover types including lawns, streets, sidewalks, etc. Note that roughness is less important during tidally-driven flood events than during riverine-driven flood events because flow velocities are much lower. Because the 100-year flood elevation in this area is controlled by tidal flooding (see Model Results section), definition of Manning's *n* roughness at this level of detail was determined to be sufficient. It is not expected that the overbank roughness values for the 1D and 2D portions of the model will match as the method of flow routing and topographic definition is different between the two methods.

Table 1: Model Roughness Values

LAND COVER AREA CATEGORY	1D/2D	MANNING'S ROUGHNESS (N VALUE)
River Channel	1D	0.035
Overbank	1D	0.07
Overbank (Composite)	2D	0.04
Harbor	2D	0.03

BOUNDARY CONDITIONS AND MODEL INPUTS

River inflows are defined at the upstream ends of both 1D rivers and a tidal boundary condition is defined at downstream-most cross sections. Where the 2D model boundary does not extend all the way to high ground (i.e. at the upstream ends of the model and along the edges of the model domain), boundary conditions are imposed to allow water to enter and/or exit the model as needed to accurately model these areas.

Tide Boundary Conditions

A long term tidal record at Aberdeen was synthesized using predicted tides and meteorological data including wind and barometric pressure from Bowerman Airport, described in detail in Appendix 1. Frequency analysis on the synthesized tide time series provided the 100-year tide level (stillwater elevation). Note that the 100-year tide level estimated by WSE is different from the 100-year tide level estimated by FEMA for the recent Grays Harbor floodplain mapping study (FEMA, 2017). The higher WSE estimated value will be used for levee design to provide a factor of safety, but the CLOMR does not propose to update the coastal floodplain mapping. By using the higher 100-year tide level for design, the levee will provide additional freeboard, allow for settlement, and provide additional protection in the event of future sea level rise due to climate change. When modeling riverine flooding, the mean higher high water (MHHW) level from the NOAA tidal gage at Aberdeen (Station 9441187) was used as the downstream boundary condition. The tide level boundary conditions are summarized in Table 2. For the WSE 100-year tide, a stage hydrograph was created by scaling a tide event measured at the gage at Aberdeen to the 100-year peak. The resultant 100-year tide hydrograph is shown in Figure 2.

Table 2 –Water Surface Elevations for Grays Harbor at Aberdeen

EVENT	WATER SURFACE ELEVATION (FEET NAVD88)
100-year Tide (WSE Analysis)	13.1[1]
100-year Tide (FEMA Effective Study)	12.1[1]
Mean Higher High Water (MHHW)	8.47

Note: [1] Maximum water surface elevation

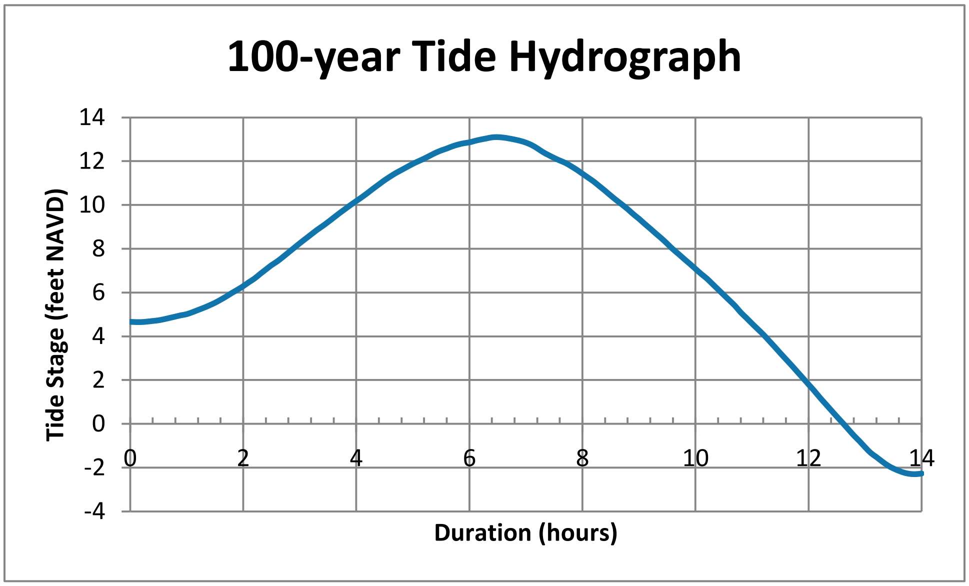


Figure 2 - Tide Hydrograph Scaled to 100-year Event.

Wishkah River and Hoquiam River Flow Boundary Conditions

The 100-year flow at the mouth of the Hoquiam and Wishkah Rivers was estimated using the USGS regional regression equations (Sumioka et al, 1998), with the results shown in Table 3.

The Washington Department of Ecology (DOE) operated a flow gage on the Wishkah River near Nisson from April 2005 to September 2013. The rating curve for the DOE gage only extended to a stage of 12 feet, even though that stage was regularly exceeded. WSE extended the rating curve for the DOE gage using a simple HEC-RAS 1D model created using channel slope and cross section data for the gage location provided by the DOE. The Nisson gage data with the extended rating curve were transposed from the gage site to the mouth of both the Wishkah and Hoquiam Rivers. This was done by scaling the Nisson data based on the ratio of its drainage area times the mean annual precipitation in the basin upstream of the gage site to the same quantity for the basin at the mouth of each river. A longer time series of flow data, extending from October 2002 through September 2016, was created by transposing data from the USGS gage on the Humptulips River below Highway 101 to the Wishkah River. Based on a regression analysis for the period of overlapping record (2007 – 2013) the data from the Humptulips were transposed to the Wishkah using the following equation:

$$Q_{Wishkah} = Q_{Humptulips} \times 0.414 + 42.7 \text{ cfs}$$

This computed long term stream flow record was used to compute the 10% exceedance winter flow in the Wishkah and Hoquiam Rivers, as shown in Table 3. These data were used with the tidal data in Table 2 as input to the hydraulic simulations.

Table 3 –Flows on the Wishkah and Hoquiam Rivers at Their Mouths

Event	Wishkah River Flow (cfs)	Hoquiam River Flow (cfs)
100-year Flow	20,500	14,800
10% Exceedance Winter Flow	1,800	1,300

Determination of the 100-year flood Event

The 100-year flood is an event with a 1% chance of occurring in any given year. In this area, a 100-year flood event can result from riverine flooding, coastal flooding, or a combination of the two. To determine the highest water surface elevations during a 100-year event, river flows and tides were modeled in combinations that have a 1% chance of jointly occurring, assuming they are independent events and that flood events on the Wishkah and Hoquiam Rivers occur concurrently (i.e. their peak flows occur at the same time). Each combination of river flows and tide levels is termed a '100-year event candidate'. Resulting water surface elevations were then examined to determine the maximum water surface elevations throughout the project area.

MODEL RESULTS AND INTERPRETATION

Two 100-year event candidates were modeled to ensure that the maximum water surface elevations at all locations were captured: 1) MHHW tidal boundary with 100-year river flows (i.e. riverine flooding conditions), and 2) 100-year tide hydrograph with 10% exceedance winter river flows (i.e. coastal flooding conditions). Figures 3 through 6 show the maximum water surface elevation for the two 100-year event candidate simulations for existing and with-levee conditions.

100-year Flood Elevation

Simulations of 100-year riverine flooding on the Wishkah and Hoquiam Rivers, with a MHHW tidal boundary, showed that flow did not overtop the river banks at any locations along the proposed north side levee. The maximum water surface elevation at any point along the proposed levee was 10.7 feet on the Wishkah River and 8.6 feet on the Hoquiam River.

Simulation of either the FEMA or WSE 100-year tide with 10% exceedance winter flows produces higher water levels and greater flood extents throughout the proposed levee area than the 100-year riverine simulations (with MHHW boundary). This indicates that within the project area, the 100-year BFE is wholly controlled by coastal flooding. The maximum 100-year water level in the study area under any of the simulations was 13.2 feet (assuming the WSE 100-year tide level and 10% exceedance flows). With the addition of 2 feet of freeboard the recommended levee design crest is 15.2 feet NAVD.

Levee Scour

Flow velocities along the Hoquiam and Wishkah River portions of the proposed North Shore Levee during a 100-year event were analyzed using the proposed conditions HEC-RAS model. In general velocities along the levee are negligible (<0.2 fps). The maximum simulated velocities are 1.3 fps near the Wishkah River and 0.9 fps near the Hoquiam River. Such low velocities are not expected to cause scour and as such the existing or proposed vegetative cover is adequate to mitigate any potential erosion.

SUMMARY AND CONCLUSIONS

A HEC-RAS 1D/2D model was developed for the Wishkah and Hoquiam Rivers near Aberdeen and Hoquiam, WA to analyze a levee proposed to protect portions of both cities from tidal and riverine flooding. The model was used to determine whether the 100-year flood in this area is controlled by coastal or riverine flooding, which is necessary to determine the base flood elevation for levee design. Two candidate 100-year flood events comprised of combinations of independent river flow and tide events were modeled. The modeling revealed that 100-year flood water levels in the project area are controlled by coastal flooding. The hydraulic model was also used to evaluate the potential for scour along the proposed levee. Velocities along the levee were found to be negligible in the modeled scenarios and thus the existing and proposed vegetative cover is adequate to mitigate potential erosion of the levee.

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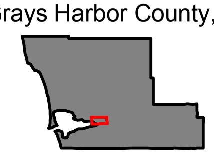
Grays Harbor County, WA



North Shore Levee
Proposed Levee Alignment and Model Set Up

0 2,000 4,000 Feet

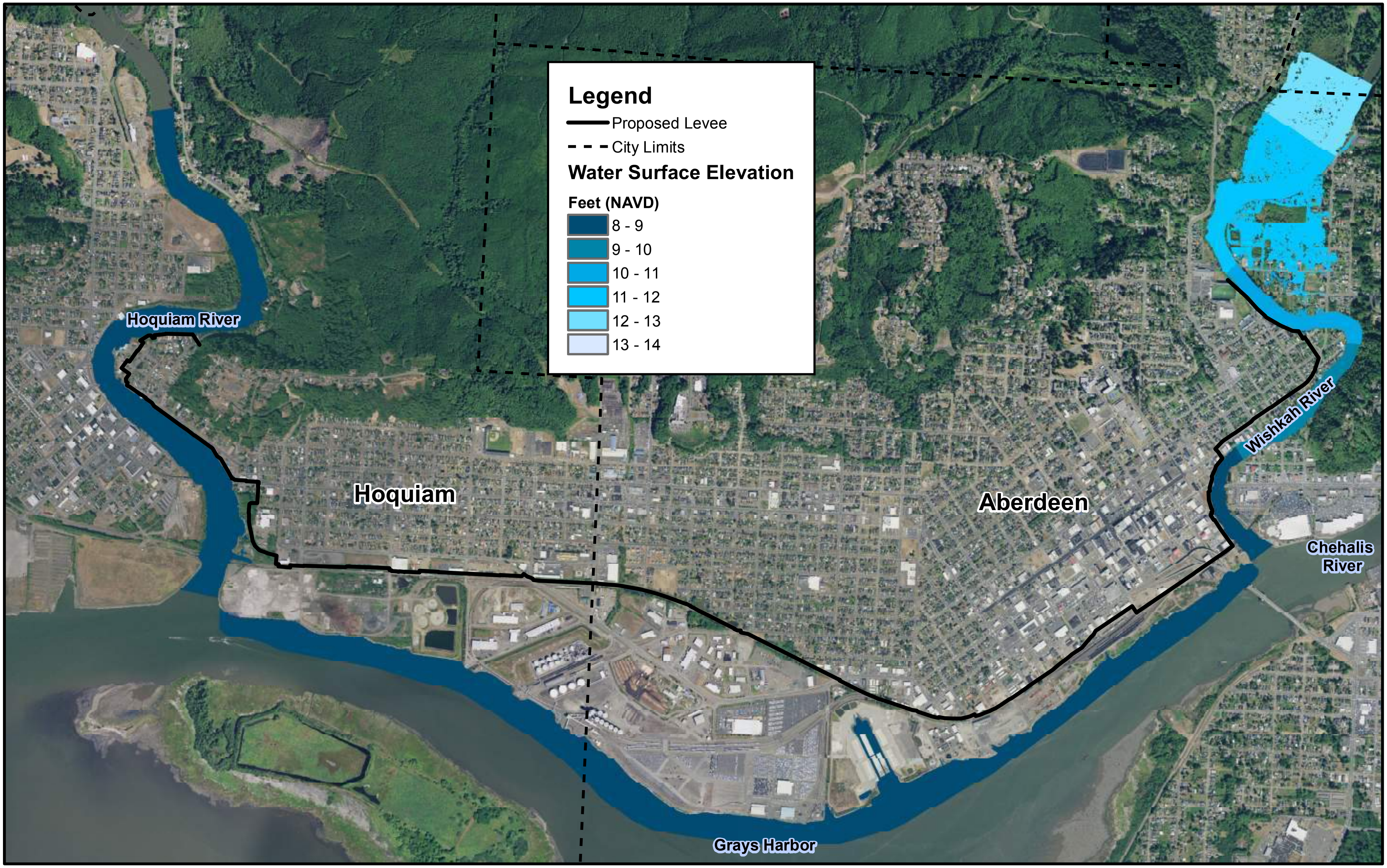
N

Scale: 1:31,295
NAD 1983 HARN
StatePlane Washington
South FIPS 4602 Feet

09 May 2017

WATERSHED SCIENCE & ENGINEERING

Figure 1



Grays Harbor County, WA

North Shore Levee

Simulated Maximum Water Surface Elevation - Existing Condition

100-year Event Candidate: Mean Higher High Water with 100-year Flows

0 1,750 3,500 Feet

N

Scale: 1:28,000
NAD 1983 HARN
StatePlane Washington
South FIPS 4602 Feet

09 May 2017

WATERSHED SCIENCE & ENGINEERING

Figure 3

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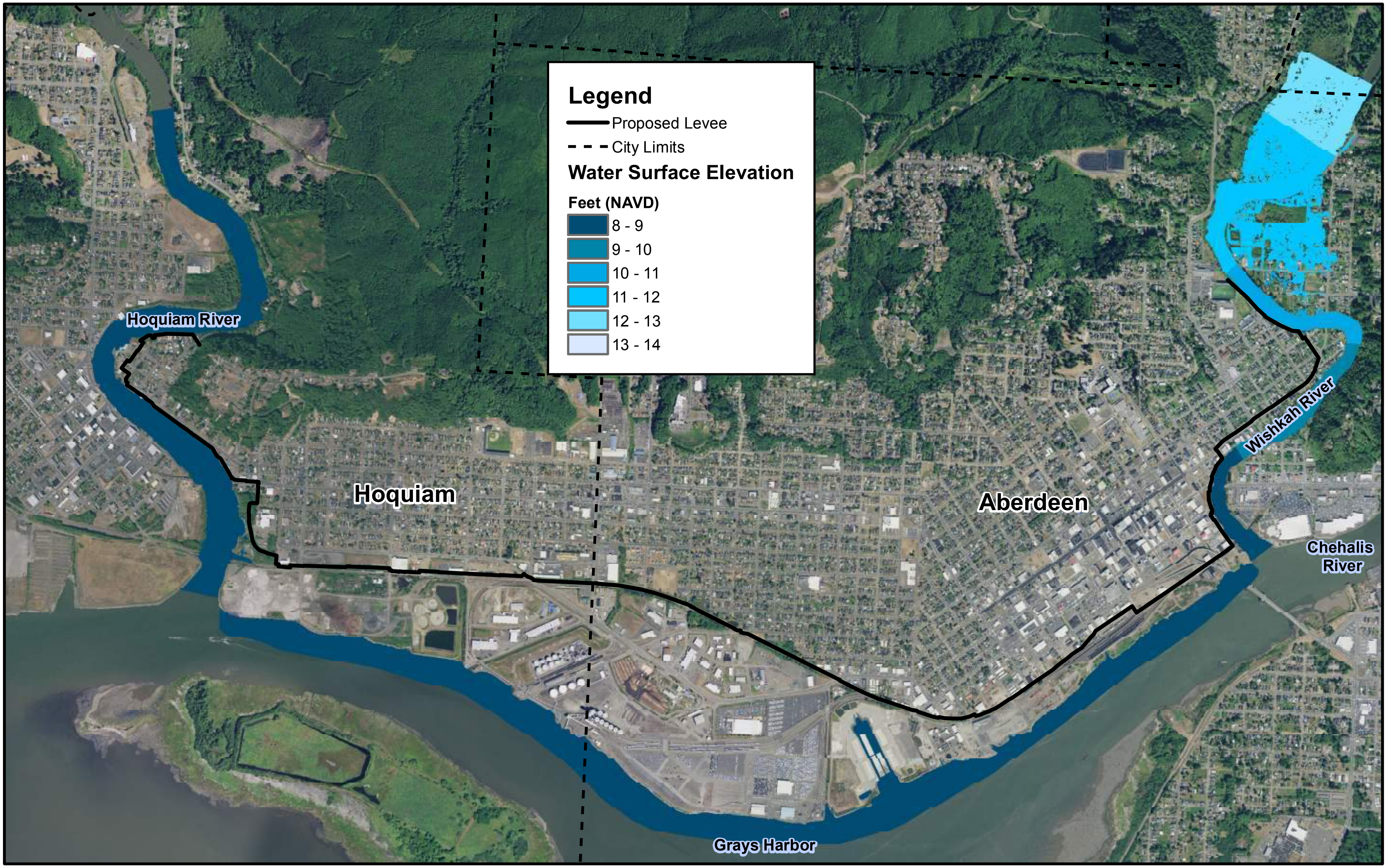

Grays Harbor County, WA

North Shore Levee
Simulated Maximum Water Surface Elevation - Levee Condition

100-year Event Candidate: Mean Higher High Water with 100-year Flows

0 1,750 3,500 Feet

N

Scale: 1:28,000
NAD 1983 HARN
StatePlane Washington
South FIPS 4602 Feet

09 May 2017

WATERSHED SCIENCE & ENGINEERING

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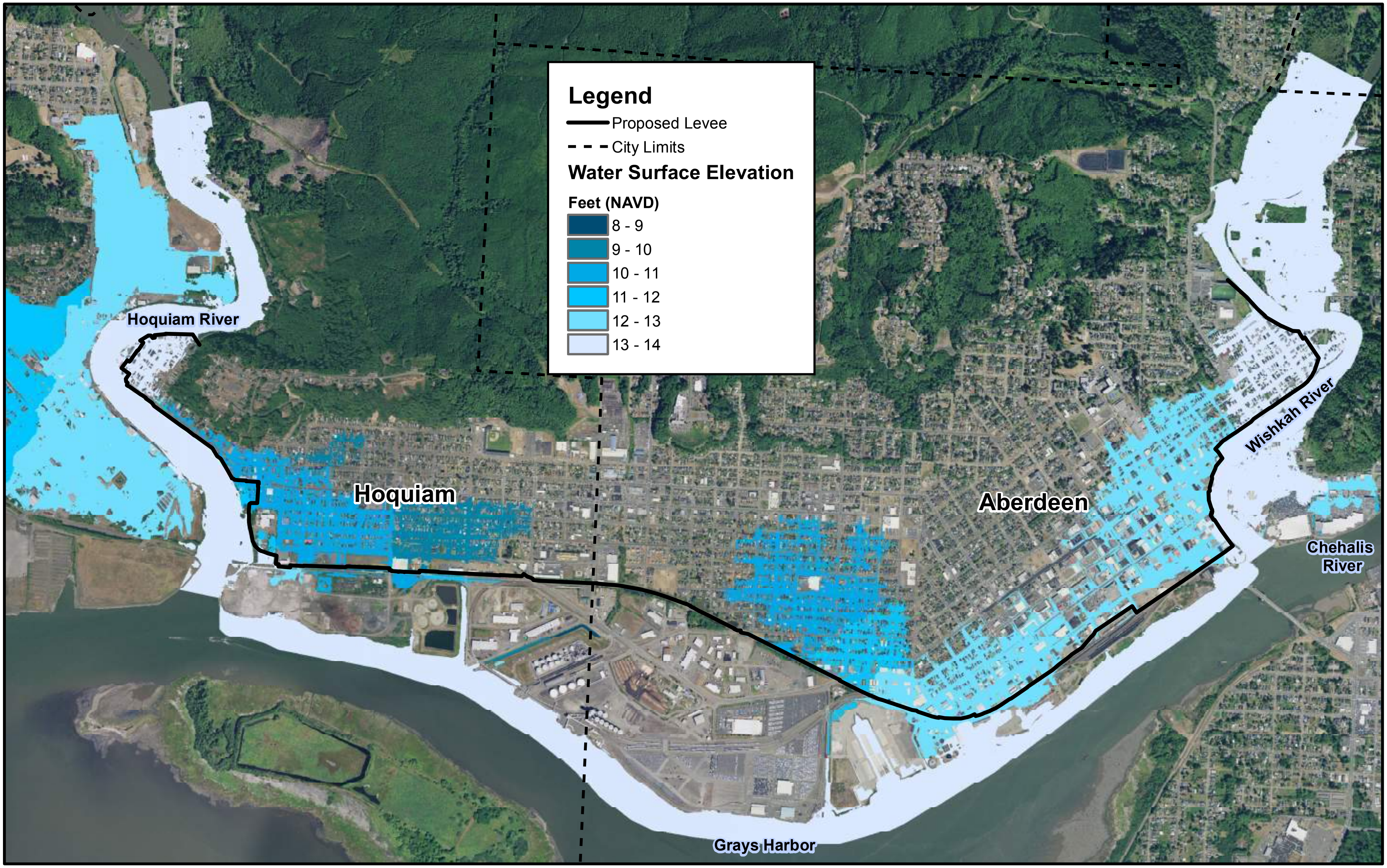
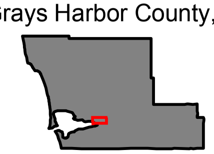
Figure 4





North Shore Levee
Simulated Maximum Water Surface Elevation - Existing Condition
100-year Event Candidate: 100-year Tide with 10% Exceedance Winter Flows

0 1,750 3,500 Feet
N

Scale: 1:28,000
NAD 1983 HARN
StatePlane Washington
South FIPS 4602 Feet

09 May 2017

WATERSHED SCIENCE & ENGINEERING

Figure 5

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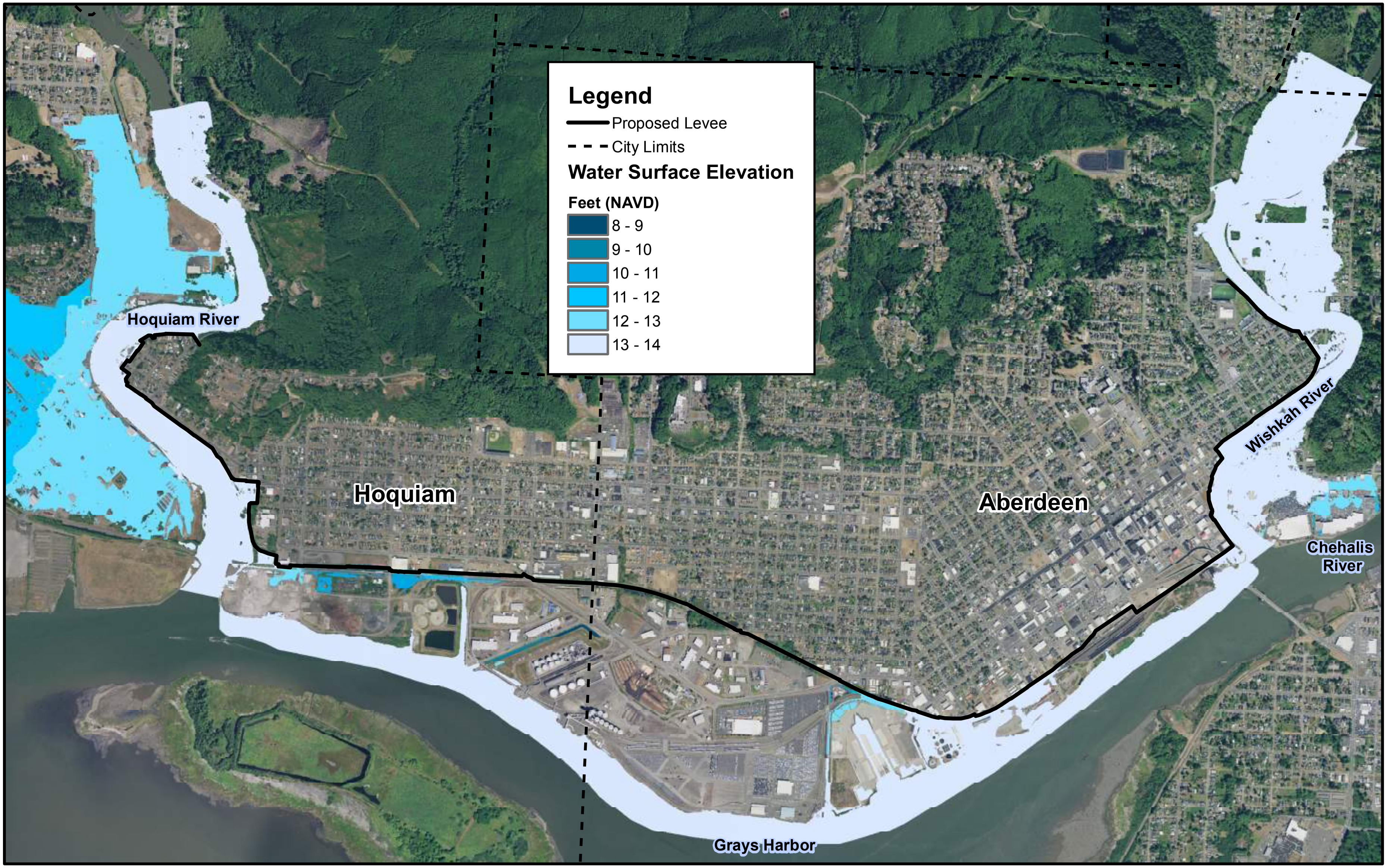
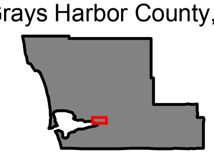


Grays Harbor County, WA



North Shore Levee
Simulated Maximum Water Surface Elevation - Levee Condition
100-year Event Candidate: 100-year Tide with 10% Exceedance Winter Flows

0 1,750 3,500 Feet

N

Scale: 1:28,000
NAD 1983 HARN
StatePlane Washington
South FIPS 4602 Feet

09 May 2017

WATERSHED SCIENCE & ENGINEERING

C:\Egnyte\Private\shaina\16-010 North Shore Levee\GIS\MXD\Figures\Appendix Figures\Figure 6 - Levee WSEL - 100yrTide 10%Q.mxd 5/9/2017 1:34:41 PM

Figure 6

Appendix 1: Grays Harbor Tide and Total Water Level Determination

To model flood levels near the mouths the Hoquiam and Wishkah Rivers, it is necessary to determine water levels at various recurrence intervals in Grays Harbor near the Cities of Hoquiam and Aberdeen. Observed tide level data are available from a NOAA gage near Aberdeen for February 2004 to December 2005. Observed tide level data are also available from a NOAA gage near Westport for March 2006 through the present. However, neither of these data sets is long enough for a reliable stage-frequency analysis and the Westport gage likely does not accurately represent conditions near Aberdeen. Therefore, a long term time series of tidal stillwater level was synthesized for a location near the mouth of the Wishkah River. Stillwater level computations include the effects of astronomical tide, El Niño, barometric pressure, and wind setup. Computations were also made for the total water level which adds the effects of waves, specifically wave setup and wave run up. Frequency analysis on the synthesized stillwater level and total water level results provide necessary boundary conditions for hydrodynamic modeling near the mouths of the Hoquiam and Wishkah rivers.

TOTAL WATER LEVEL COMPONENTS

ASTRONOMICAL TIDES

Astronomical tide data were computed using Xtide, a program based on the NOAA National Ocean Service equations and local harmonic constituents to create tide level time series at various locations (Flater, 2016). Astronomical tide was computed for the Aberdeen, Grays Harbor site from January 1, 1973 through June 30, 2016 at one hour intervals.

EL NINO SEA LEVEL ANOMALY

Monthly mean sea level data with average seasonal cycles and long term linear trends removed are available for a NOAA Sea Level Trend monitoring station at Toke Point (station 9440910). This measurement, also known as sea level anomaly, quantifies the impact of El Niño and Southern Oscillation on sea level. An hourly time series was created by linearly interpolating between the monthly sea level anomaly data. Because sea level fluctuations due to El Niño occur over large spatial and temporal scales, this data was considered representative of the project area at Aberdeen.

INVERSE BAROMETRIC EFFECT

Sea level atmospheric pressure data are available from NOAA National Climatic Data Center. Hourly data from Hoquiam Airport Station 72792794225 were used for January 1, 1973 to December 18, 1990. Hourly data from Bowerman Airport Station 72792394225 were used for January 8, 1991 to June 30, 2016. Gaps in the data were filled using linear interpolation. The impact of sea level pressure on total water level was calculated using the equation given in the Aviso User Handbook (2008),

$$IB = -9.948(P_{atm} - 1013.3)$$

where IB = inverse barometer effect (mm)

P_{atm} = sea level atmospheric pressure (mbar)

WIND SETUP

Wind setup was calculated based on wind speed and direction measured at Bowerman Airport and gaps were filled using linear interpolation similar to the sea level pressure data. Wind setup was calculated using the method for a flat bottom as described in Dean and Dalrymple (1991),

$$\bar{\eta}_{wind} = h_0(\sqrt{1+2A}-1)$$

where $\bar{\eta}_{wind}$ = wind setup (m),

h_0 = water depth (m), and

A = ratio of shear to hydrostatic forces = $\frac{1.2\tau_w l}{\rho g h_0^2}$

in which l = fetch length (m),

ρ = mass density of water (kg/m^2), and

τ_w = wind shear stress at the water surface $= \rho k W|W|$

in which W = wind speed at a 10 m height above the surface (m/s), and

k = wind stress coefficient, calculated using the formulation in Van Dorn (1953),

$$k = \begin{cases} 1.2 \times 10^{-6}, & |W| \leq W_c \\ 1.2 \times 10^{-6} + 2.25 \times 10^{-6}\left(1 - \frac{W_c}{|W|}\right)^2, & |W| > W_c \end{cases}$$

where W_c = critical wind speed, 5.6 m/s.

The average water depth used was dependent on tide level. The fetch was also dependent on tide as parts of Grays Harbor can be submerged or dry depending on the water surface elevation. The relationship between tide level, average water depth, and fetch was determined using the astronomical tide level and Grays Harbor bathymetric data produced in December 2005 by the National Oceanic and Atmospheric Administration Center for Tsunami Research (NOAA, 2005). Winds directed away from the study area shoreline were not considered for this analysis.

WAVE HINDCAST EQUATIONS

Peak wave period and significant wave height were determined using equations from Hurdle and Stive (1989),

$$\frac{gH_0}{{U_A}^2} = 0.25\tanh\left(0.6\left(\frac{h}{g{U_A}^2}\right)^{0.75}\right)\cdot\tanh^{0.5}\left(\frac{4.3\cdot 10^{-5}\left(\frac{gX}{{U_A}^2}\right)}{\tanh^2\left(0.6\left(\frac{gh}{{U_A}^2}\right)^{0.75}\right)}\right)$$

$$\frac{gT_P}{U_A} = 8.3\tanh\left(0.76\left(\frac{gh}{gU_A{}^2}\right)^{0.375}\right)\cdot\tanh^{0.5}\left(\frac{4.1\cdot 10^{-5}\left(\frac{gX}{U_A{}^2}\right)}{\tanh^3\left(0.76\left(\frac{gX}{U_A{}^2}\right)^{0.375}\right)}\right)$$

where H_0 = significant wave height (ft),

T_p = spectral peak wave period (s),

X = fetch length (ft), and

h = water depth (ft)

U_A = Adjusted wind speed given by the equation from the US Army Corps of Engineers Shore Protection Manual (1984) (converted to ft/s),

$$U_a = 0.589\, U_{10}{}^{1.23}$$

in which U_{10} = wind speed measured in mph 10 meters above the surface.

The fetch length was estimated for various wind directions in GIS based on aerial photography. The average depth for each fetch length was determined using available bathymetric data for Grays Harbor.

The calculated significant wave height, H_0, was then reduced by a sheltering coefficient. This coefficient combined diffraction around Rennie Island and removal of waves propagating offshore. The sheltering coefficient was dependent on the direction of the wind relative to the shore normal. Waves were analyzed at Transects 2, 7, and 8 surveyed by Wilson Engineering in May 2016 and shown in Figure 1. Transects 2, 7, and 8 were selected because they align with the largest fetch in Grays Harbor and are not sheltered by Rennie Island, and would thus result in the largest wave setup and runup of any transects within the project area.

Figure 1: Grays Harbor shoreline survey transects.

Wave Runup

Incident wave runup was calculated using the reduced significant wave height at the toe of the slope and the equation given by FEMA Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States (2005),

$$R = H_0 \begin{Bmatrix} 1.77\gamma_r\gamma_b\gamma_\beta\gamma_P\xi_0 & 0.5 \leq \gamma_P\xi \leq 1.8 \\ \gamma_r\gamma_b\gamma_\beta\gamma_P\left(4.3 - \dfrac{1.6}{\sqrt{\xi_0}}\right) & 1.8 \leq \gamma_P\xi \end{Bmatrix}$$

where R = 2% runup height (ft),

H_0 = significant wave height (ft),

γ_r = reduction factor for influence of surface roughness,

γ_b = reduction factor for influence of berm,

γ_β = reduction factor for influence of angled wave attack ,

γ_P = reduction factor for influence of structure permeability, and

ξ_0 = Iribarren Number for significant wave = $\dfrac{m}{\sqrt{\dfrac{2\pi H_0}{gT_P{}^2}}}$

in which m = beach profile slope, and

T_P = spectral peak period (s).

The roughness, berm, and permeability reduction factors and beach profile slope were determined from the shoreline transects and aerial photographs. The reduction factor for angled wave attack was calculated assuming that the waves approached from the measured wind direction using the FEMA (2005) equation,

$$\gamma_\beta = \begin{cases} 1.0, & 0 < |\beta| < 10° \\ \cos(|\beta| - 10°)\,, & 10° < |\beta| < 63° \\ 0.63, & |\beta| > 63° \end{cases}$$

where β is the angle of wave approach with β=0º for normally incident waves.

Wave Setup

Static wave setup calculations used the spectral peak period and significant wave height from the wave hindcast equations. Wave setup was calculated using the direct integration method described in FEMA Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States (2005),

$$\bar{\eta} = 4\left(\frac{H_0}{26.2}\right)^{0.8}\left(\frac{T_P}{20}\right)^{0.4}\left(\frac{m}{0.01}\right)^2$$

where $\bar{\eta}$ = static wave setup (ft)

H_0 = significant wave height (ft),

T_p = spectral peak wave period (s), and

m = nearshore slope.

ASTORIA AIRPORT METEOROLOGICAL DATA

To evaluate the effect of errors and gaps in the Bowerman Airport meteorological data, computations of the total water level components described above were repeated using meteorological data from NOAA National Climatic Data Center Astoria Airport Station 72791094224. The analysis was completed for the same time period. Changing the source of wind speed, wind direction, and sea level pressure to the Astoria gage produced lower 100-year stillwater and total water levels than estimated using the Bowerman Airport data. Because the Bowerman Airport gage is much closer to the project site and produced more conservative water levels, Bowerman Airport meteorological inputs were used for the final analysis.

RESIDUALS AND TOTAL WATER LEVELS

The sum of the El Niño, inverse barometer, and wind setup effects is the modeled residual. The residuals were added to the astronomical tide to create a synthetic time series of stillwater levels. Finally, the computed stillwater level, wave runup, and wave setup were summed to produce a total water level time series for the period January 1973 through June 2016.

VALIDATION OF MODELED STILLWATER LEVELS

Because tide gages measure stillwater level and not total water level, validation of the synthesized total water level data with an observed dataset was not possible. However, the computed stillwater levels were compared to the observed stillwater levels for the Aberdeen tide gage period of record in Figure 3. The computed residuals were also compared to the observed residuals in Figure 2. Note that the stillwater comparison is not provided for each transect as transect-specific computations were only used for wave setup and runup calculations. The correlation between the observed and computed residuals supports the computation of El Niño, inverse barometer, and wind setup effects, although the correlation is weak. In particular, the correlation is weakest with the highest observed residuals. However, because residual is only a small component of tide, the correlation between the observed and computed stillwater level is strong.

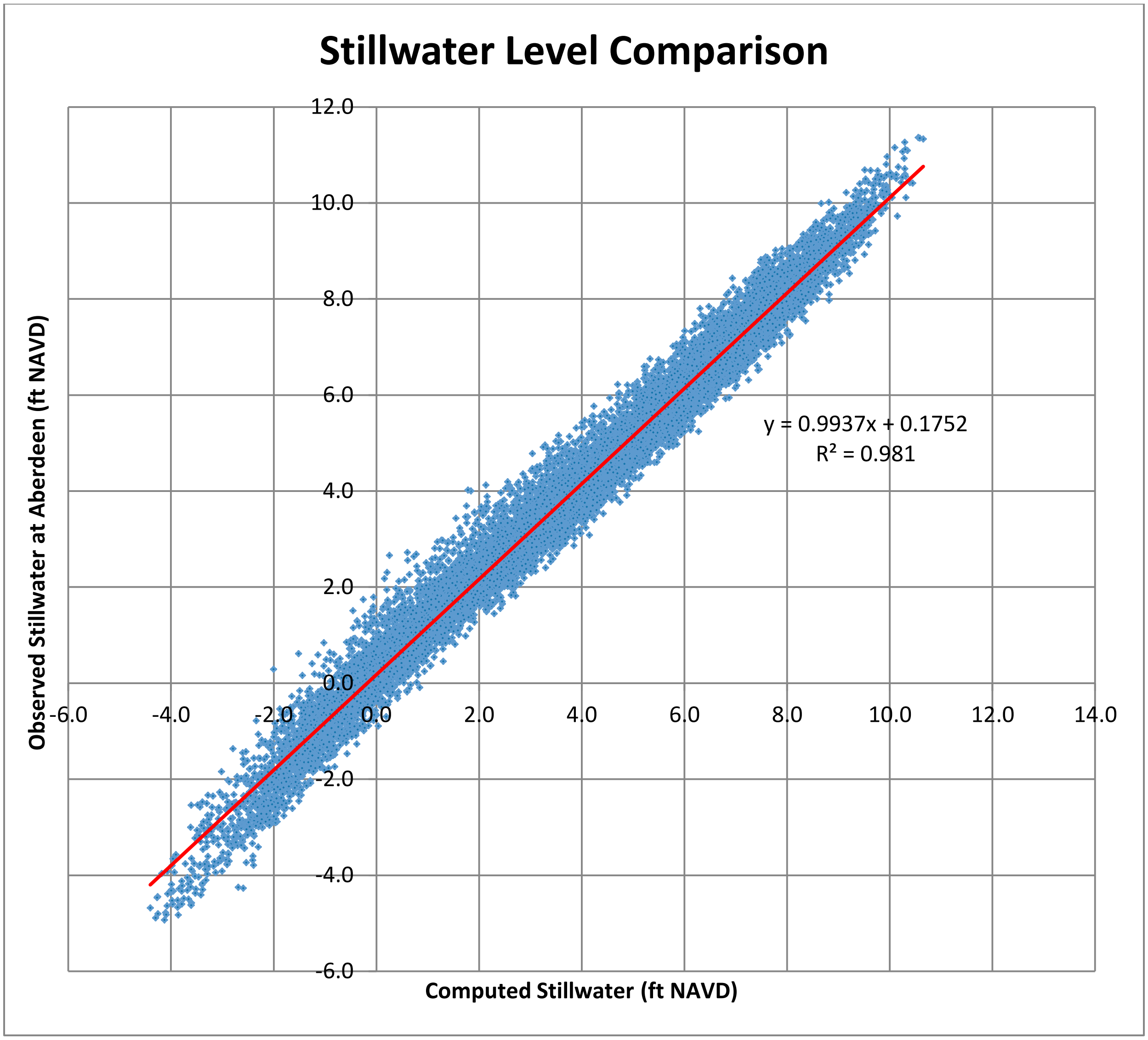


Figure 2: Comparison of computed and observed stillwater levels at Aberdeen.

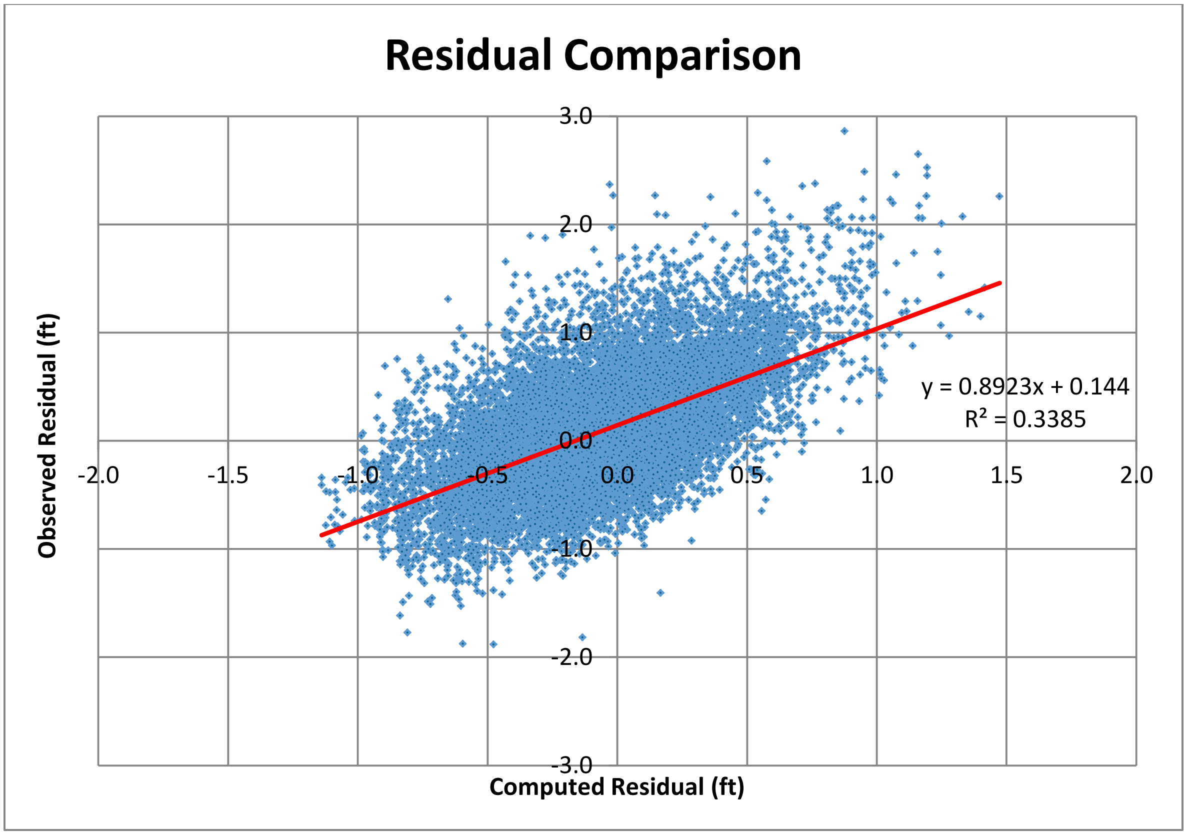


Figure 3: Comparison of computed and observed residuals at Aberdeen.

FREQUENCY ANALYSIS

Frequency analyses were performed on the long term synthesized tide records to determine the tide and total water level at Aberdeen for various return periods. The data were analyzed using the Generalized Pareto Distribution and results are shown in Figures 4 5, and 6. The resultant water levels for various recurrence intervals are summarized in Table 1 below.

Table 1: Computed stillwater and total water levels at Transects 2, 7, and 8 for various return intervals.

	Transect 2		Transect 7		Transect 8	
Recurrence Interval	Stillwater Level (ft NAVD)	Total Water Level (ft NAVD)	Stillwater Level (ft NAVD)	Total Water Level (ft NAVD)	Stillwater Level (ft NAVD)	Total Water Level (ft NAVD)
500 year	13.4	15.4	13.4	14.0	13.4	14.2
100 year	13.1	15.1	13.1	13.8	13.1	14.0
25 year	12.7	14.6	12.7	13.5	12.7	13.7
10 year	12.3	14.0	12.3	13.1	12.3	13.3
5 year	11.9	13.3	11.9	12.6	11.9	12.8
2 year	11.1	12.0	11.1	11.7	11.1	11.7

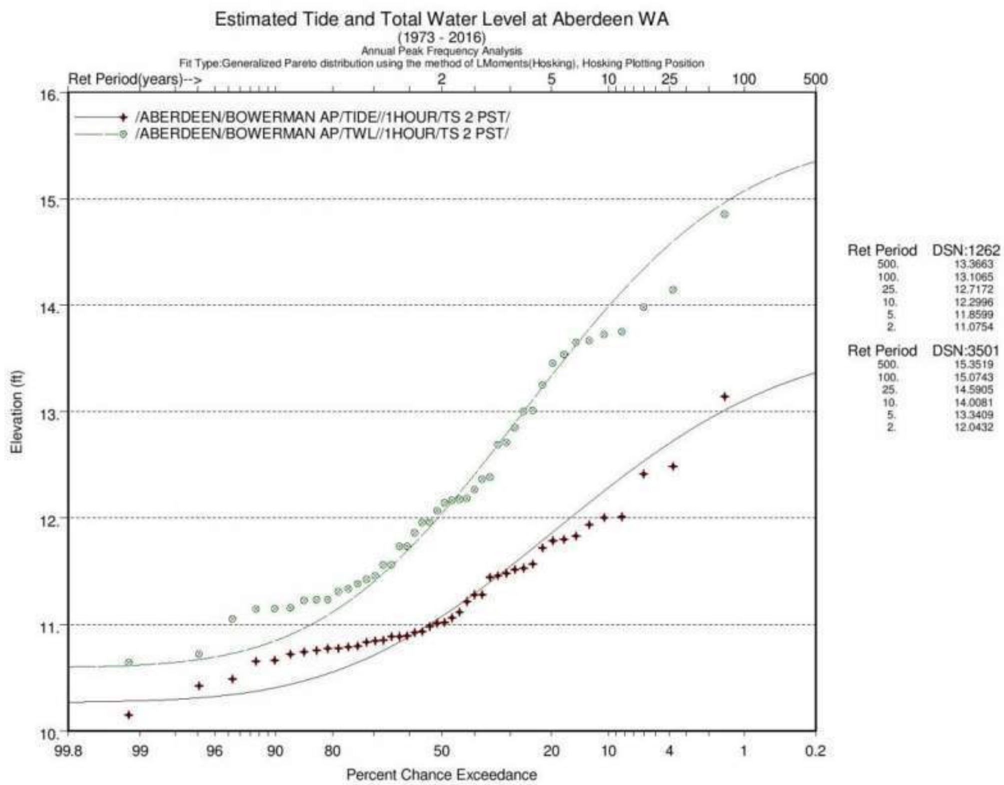


Figure 4: Frequency analysis of stillwater (tide) and total water level for Transect 2.

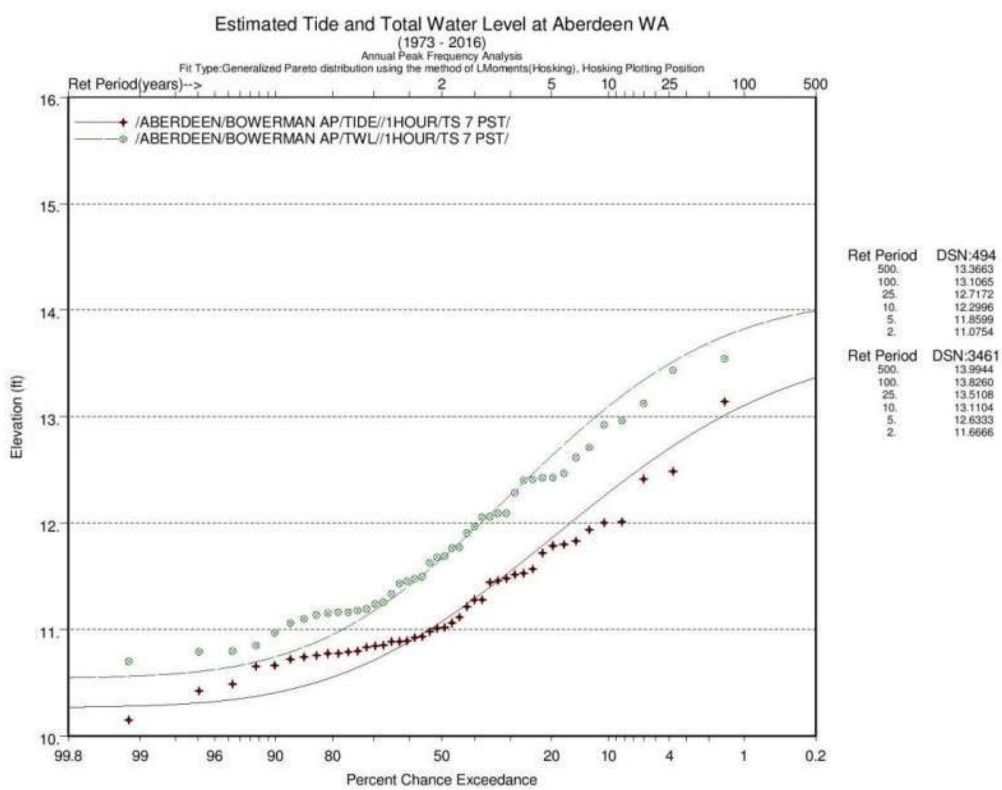


Figure 5: Frequency analysis of stillwater (tide) and total water level for Transect 7.

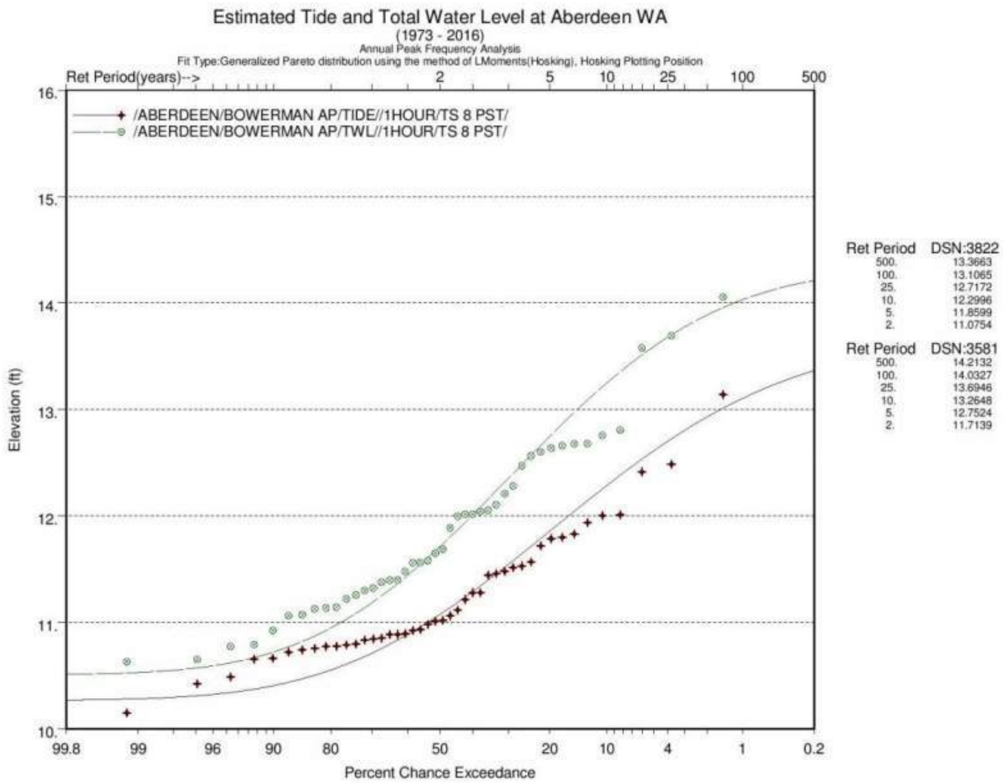


Figure 6: Frequency analysis of stillwater (tide) and total water level for Transect 8.

Based on the data in Table 1 and Figures 4 through 6, the 100-year tide (stillwater) level in Grays Harbor near the proposed Aberdeen-Hoquiam North Shore Levee was estimated to be 13.1 feet NAVD. The stillwater level is the same at all three transects evaluated. The 100-year total water level changes by transect ranging from a maximum of 15.1 feet at Transect 2 to a minimum of 13.8 feet at Transect 7. It should be noted again that these three transects have the greatest exposure to wind and wave action and as such will have the highest total water levels of any point in study area. While Transect 2 has the highest estimated 100-year TWL, it is felt that this water level would be isolated to the specific location of this transect which is very exposed to waves and has a steep bank slope. However, Transect 2's location near the mouth of the Hoquiam River is also quite distant from the proposed levee. Considering the exposure characteristics of the shoreline across the project area, a 100-year TWL of 14.0 feet was felt to better represent potential TWLs along the levee.

SUMMARY

A long term time series of tidal (stillwater) levels was created for a project area along Grays Harbor near Aberdeen and Hoquiam, WA. Historical stillwater levels were estimated based on astronomical tides plus a tidal residual capturing the effects of El Niño, inverse barometer, and wind setup. A frequency analysis was conducted on the computed stillwater results to estimate stillwater levels at various recurrence frequencies. Computed total water levels, which also include the effects of wave setup and runup, were computed for three shoreline transects near Hoquiam and Aberdeen. Frequency analyses were conducted on the computed water levels to determine design tidal and total water levels along this section of Grays Harbor.

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