U.S. Army Corps of Engineers
Charleston District

APPENDIX A

CHARLESTON HARBOR POST 45
CHARLESTON, SOUTH CAROLINA

Engineering

June 2015

Engineering Appendix

Attachments Available via hyperlink

A-1 2012 Hydrodynamic Model Calibration Plots
<http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_1_Hydrodynamic_Model_Calibration_Plots.pdf>

A-2 2004 Hydrodynamic Model Validation Plots
<http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_2_2004_Hydrodynamic_Model_Validation_Plots.pdf>

A-3 Suspended Sediment Concentration Plots

<http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_3_Suspended_Sediment_Concentration_Plots.pdf>

A-4 Water Quality Validation Plots

<http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_4_Water_Quality_Validation_Plots.pdf

Engineering Appendix

1.0 Existing and Historic Conditions

1.1 History

Charleston Harbor is the largest seaport in South Carolina. Charleston Harbor is formed by the confluence of the Cooper, Ashley and Wando Rivers. It includes the tidal estuary of the lower 12 miles of the Cooper River and the four miles of open bay between the confluence of the Ashley and Cooper Rivers and the Atlantic Ocean. The Cooper River contributes most of the freshwater inflow to the system and is the largest of the estuaries, extending about 57 miles from the harbor entrance to the Jefferies Hydroelectric Station at Pinopolis, SC. Two granite jetties protect the entrance to the harbor, 2,900 feet apart, which extend from Sullivan and Morris Islands, respectively. The harbor is approximately 2 miles wide between the entrance channel and the junction of the Ashley and Cooper Rivers.

Federal involvement in the harbor began over 160 years ago when the River and Harbor Act (RHA) of 1852 initially authorized navigation improvements at Charleston.

In the 1940's, the South Carolina Public Service Authority (SCPSA) constructed the Santee-Cooper hydroelectric project, which diverted water from the Santee River Watershed Basin into the Cooper River. Figure 1.1.1. provides a map of the watershed and major water features. The project formed two lakes that were connected by a diversion canal. Lake Marion, formed by Wilson Dam, is located on the Santee River near Pineville, SC. Lake Moultrie, formed by Jefferies Hydroelectric Station near Pinopolis, SC, is located at what were once the headwaters of the Cooper River. A canal connects the two lakes and diverts freshwater flow from the Santee River Basin into the Cooper River. Inflows into Charleston Harbor prior to the 1942 diversion were approximately 261 cfs from the Ashley River, approximately 82 cfs from the Wando River and approximately 72 cfs from the Cooper River at its headwaters. Diversion of the Santee Basin flows via Lake Moultrie and the Jefferies Hydroelectric Station into the Cooper River increased flows to a total average inflow of about 15,600 cfs into Charleston Harbor. The increased inflow caused the character of the harbor to change from vertically well-mixed to a more stratified salinity condition. The increased freshwater inflow into the salt water of the harbor resulted in density currents, which trapped sediments in suspension until deposited on the harbor floor.

After diversion, shoaling and dredging quantities increased from approximately 110,000 CY/yr to over 10 million CY/yr in the Charleston Harbor navigation channel. This was compounded by the reduced availability of disposal areas in the region for dredge material. Improved dredging and disposal methods stabilized the dredged quantity at about 7.5 million CY/year, but higher maintenance costs to keep the harbor and Cooper River navigable for port traffic necessitated some action. The U. S. Army Corps of Engineers (USACE), also referred herein as "the Corps", proposed rediverting a majority of the flows from the Cooper River back to the

Santee River, thereby reducing sedimentation in the harbor. Under PL90-483, the Cooper River Rediversion Project (CRRP) was completed in 1985 with a canal from Lake Moultrie to the Santee River near St. Stephen, South Carolina. Dredging records from 1988 to 1994 indicate average annual maintenance dredging was reduced to approximately 1.8 million CY throughout the inner harbor.

Bushy Park is a freshwater reservoir located in the upper reaches of the Cooper River and used by local industry and municipal for water supply (see Figure 1.1.1). Concerns within the Cooper River watershed about salinity levels in the harbor and water supply needs prompted detailed physical and numerical modeling to study the effects of the proposed rediversion. Between the 1950's and the 1970's, two physical models were developed at the U. S. Army Engineer Waterways Experiment Station (WES, now known as ERDC- Engineer Research and Development Center). The model was part of a series of studies of both Charleston Harbor and Bushy Park Reservoir to determine effects of the proposed Cooper River Rediversion on salinity concentrations. The studies addressed mixing and sediment flushing in the harbor, as well as the inflow, tidal and meteorological effects on salinity conditions in the Bushy Park Reservoir.

As directed by a CRRP requirement, prototype tests were conducted before and after project completion in order to determine the maximum amount of fresh water (weekly average discharge) that could be released from Lake Moultrie into Cooper River without causing stratification and the resultant sediment trapping density currents in Charleston Harbor. The agreement with SCPSA prior to the start of the project set a weekly average of 3,000 cfs as the fresh water discharge from Pinopolis. This would maintain a well-mixed harbor regimen. However, further tests proved that stratification would also be prevented with average discharge up to 4,500 cfs. The tests also indicated that salinity intrusion protection of Bushy Park Reservoir could be more efficiently accomplished with the nonstructural monitoring system with 4,500 cfs inflow as opposed to 3,000 cfs. The weekly average flow requirement of 4,500 cfs is the contractual agreement between the Corps and SCPSA.

The Charleston Harbor federal navigation channel is identified by reaches as seen in Figure 1.1.2. Widths of the existing channel template are shown in Table 1.1. Design depth is 47 feet mean lower low water (MLLW) in the entrance channel, 45 feet MLLW from the Mount Pleasant Range throughout Charleston Harbor, the Wando River, and the Cooper River. A new feature to the project, the SCSPA is constructing a 65.8-acre berth and access channel at the former Navy Base on Daniel Island Reach, as shown in Figure 1.1.3. Expected completion of the terminal, berth and access channel is 2019. EIS for the proposed terminal was approved with USACE Record of Decision in April 2007. The berthing and access area extends approximately 850 feet from the edge of the proposed wharf to the edge of the existing Federal navigation channel. The berthing area and access channel will be constructed by the SCSPA to correspond to the existing authorized depth of the Federal project and portions of the access channel that are also part of the turning basin will be rolled into the Federal channel as part of future authorization of the recommendations of this study.

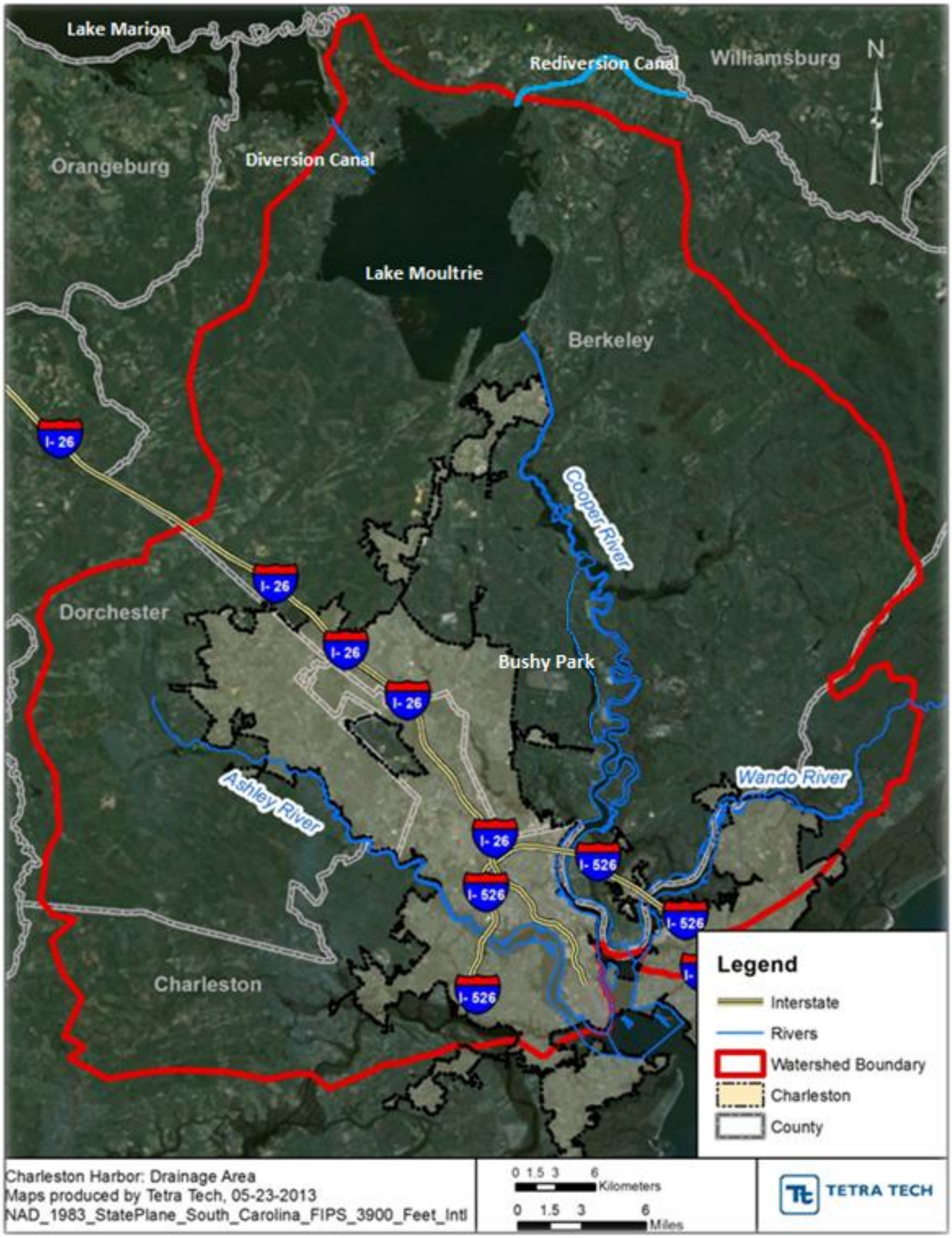

Figure 1.1.1 Charleston Harbor Watershed

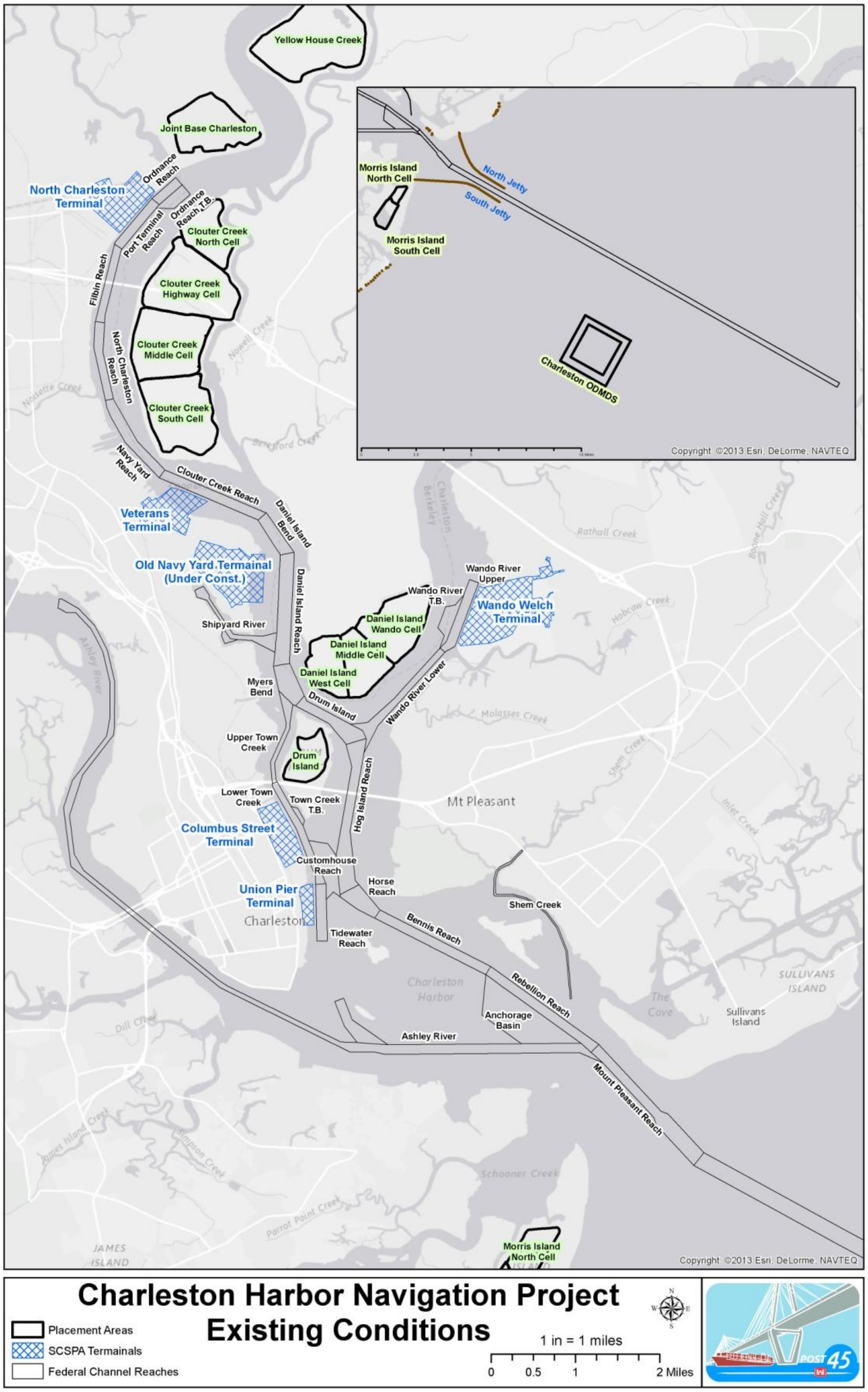

Figure 1.1.2 Charleston Harbor Federal Channel Reaches

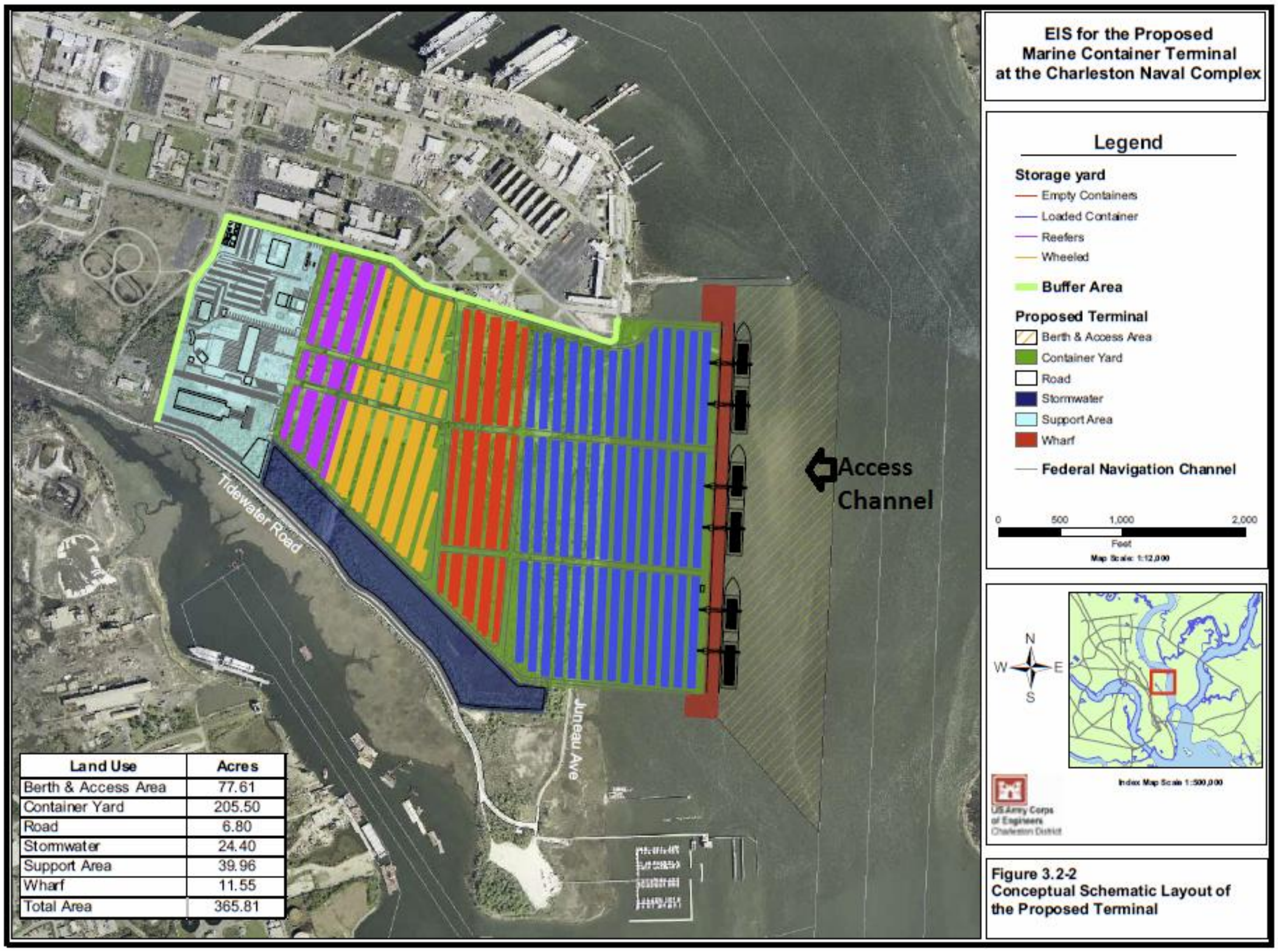

Figure 1.1.3 Location of New SCSPA Terminal Access Channel

Table 1.1 Charleston Harbor Federal Channel Dimensions by Reaches (* channel widths vary, table denotes dimensions traversing upstream not including bend easings)

Reach	New Stationing (2011)		Historical Stationing(pre-2011)		Start Width* (feet)	End Width* (feet)	Length miles	depth MLLW (ft)
	Start Station	End Station	Start Station	End Station				
Fort Sumter Reach	0+00	900+00	-900+00	0+00	1000	1000	17.5	47
Mount Pleasant Reach	900+00	995+18	0+00	95+21	1000	600	1.8	45
Rebellion Reach	995+18	1077+91	95+21	177+99	600	600	1.6	45
Bennis Reach	1077+91	1155+87	177+99	256+57	600	600	1.5	45
Horse Reach	155+87	1178+12	256+57	285+62	600	600	0.6	45
Hog Island Reach	1178+12	1272+82	286+62	375+79	600	600	1.6	45
Drum Island Reach	1272+82	1316+91	375+79	420+71	600	600	0.8	45
Meyers Bend Reach	1316+91	1342+47	420+71	446+25	880	880	0.5	45
Daniel Island Reach	1342+47	1412+40	446+25	518+09	880	880	1.4	45
Daniel Island Bend	1412+40	1440+56	518+09	543+50	800	700	0.5	45
Clouter Creek Reach	1440+56	1508+78	543+50	612+66	600	600	1.3	45
Navy Yard Reach	1508+78	1566+35	612+66	668+13	600	675	1.1	45
North Charleston Reach	1566+35	1615+65	668+13	722+01	500	500	1	45
Filbin Creek Reach	1615+65	1664+42	722+01	768+34	500	500	0.9	45
Port Terminal Reach	1664+42	1701+05	768+34	804+96	600	600	0.7	45
Ordnance Reach	1701+05	1720+53	804+96	824+44	600	600	0.4	45
Ordnance Reach Turning Basin	1698+50	1720+53			800	800	0.4	45
Wando River Lower Reach	0+00	71+49	0+00	76+00	400	400	1.4	45
Wando River Upper Reach	71+49	119+78	76+00	123+43	600	600	0.9	45
Wando River Turning Basin	71+00	109+00	86+56	106+72	800	800	0.4	45
Tidewater Reach	0+00	35+73			650	650	0.7	40
Custom House Reach	26+00	64+00			1385	1385	0.5	45
Town Creek Lower Reach	35+73	105+82	38+68	73+76	450	450	1.1	45
Town Creek Upper Reach	105+82	161+27	105+91	161+24	250	250	1	16
Town Creek Turning Basin	74+00	87+86	73+76	105+91	300	300	0.25	35
Shipyard River	0+00	64+77	0+00	64+77	300	200	1.1	45-30

1.2 Shoaling Rates

Multiple sources of the sediments cause shoaling in the harbor. These are sediments discharged from Pinopolis Dam; biogenic sources in the estuary (e.g., diatom phytoplankton, marsh vegetation); stormwater runoff from the surrounding watershed; shoreline erosion; ocean sediments; and other unknown sources. This is further discussed in section 3.4.3 of this appendix.

Shoaling does not occur in every reach; rather it occurs in particular reaches and at varying rates. In 2012, a very basic sediment budget using SBAS was done based on dredging records and disposal. Figure 1.2.1 shows the high shoal areas, those areas that are regularly dredged for maintenance, throughout the project limits. The existing project is maintained to the authorized project depth of 45 MLLW (47 feet MLLW for the entrance channel). In addition, two feet of advanced maintenance and two feet of allowable overdepth are authorized. Some specific reaches have a four foot or six foot authorized advanced maintenance due to accelerated shoaling rates in those areas (Table 1.2.1 as justified in the Environmental Assessment Charleston Harbor Additional Advanced Maintenance Dredging, 2009. Documented approval by memorandum for record from CESAD-PDO dated Nov, 14 2012).

Table 1.2.1 Charleston Harbor Areas Requiring Additional Advanced Maintenance

	Station Start	Station End	Required Depth	Authorized Advanced Maintenance	Allowable Overdepth
Drum Island Reach	1281+00	1296+00	45'	6'	2'
Lower Wando Reach	0+00	30+00	45'	4'	2'
Lower Town Creek Reach	41+69	73+45	45'	4'	2'
Ordnance Reach	1693+00	1720+53	45'	4'	2'
Ordnance Reach Turning Basin	1698+34	1720+53	45'	4'	2'
Wando Turning Basin	81+35	101+51	45'	4'	2'

SAC has authorization for the entire reach indicated in the table. However, only the portion that has shoaled is dredged to the advance maintenance depth (it may be only a portion of the length on the left or right side of the channel). High shoal areas predominantly occur in the inside or fill bank of a bend.

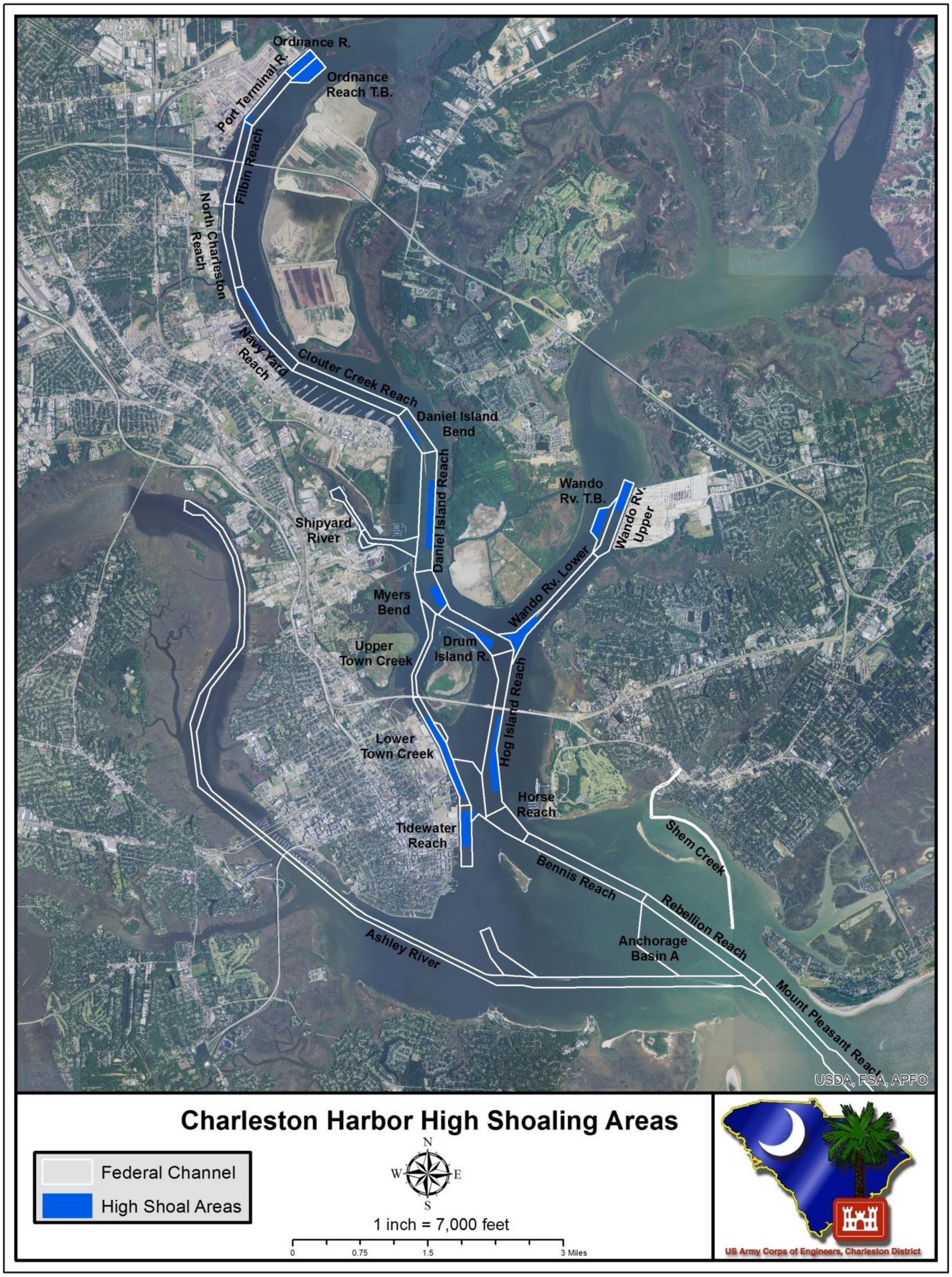

Figure 1.2.1 Charleston Harbor High Shoaling Areas

In order to determine the existing shoaling rates, dredging records in Charleston Harbor occurring between 1998 and 2011 were selected. The last deepening project began in June 1999 and completed in May 2004. Thus, the period of record includes periods when the last deepening occurred, but the new work quantities were not included in the estimate of maintenance rates.

1.2.1 Computation Methodology of Shoaling Rates

The dredging records during the period of study were exported to a file geodatabase. For each reach in the Charleston Harbor project, ArcGIS "Select by Location" was used to select each dredge event with a polygon centroid located within the reach. Each selected set of dredge event polygons was exported as a polygon feature class within the database.

The records of each polygon were exported to an MS Excel spreadsheet. By reach, the records were sorted by date. After sorting, the time since last dredging in months was determined. For each Maintenance Work dredging record, the volume dredged was divided by the time of last dredging for "NET" and "OVERDEPTH" quantities. This resulted in a volumetric shoal rate in units of cubic yards/month (CY/MO).

The start and stop stationing of each reach, recorded in the Excel file, was used to determine a distance along the shoal. The "NET" and "OVERDEPTH" monthly shoal rates were then divided by this number. This resulted in a unit monthly shoal rate in CY/LF.MO. For each reach, the "NET" and "OVERDEPTH" monthly shoal rate and monthly unit shoal rate were averaged. The overdepth rate is reported in the summary for each reach in Table 1.2.2.

Shoals with multiple events occurring within three months of each other were analyzed slightly differently. These instances represent multiple shoals within the same reach. Showing large volumes dredged in each shoal of the reach over short periods of time skews the shoaling rate. For these events, the volume of each dredging was added, and the sum divided by the sum of time since last dredging. This is appropriate because it reflects the manner in which maintenance dredging is performed. To capture maintenance dredging after the project had reached full authorized depth in all reaches, the average shoaling rate after the last new work dredging events in 2004 was computed. The max was a one year max observed for a specific reach and the min was a one year minimum for a specific reach while the average covers the entire period of record.

Table 1.2.2 Maintenance Dredging

Charleston Reach Extents											
Reach	**New Stationing**		**Historical Stationing**		**Area (ac)**	**Section**	**Cycle Length (mos)**	**Average Shoaling Rate Following May 2004 (CY/MO)**	**Average Shoaling Rate (CY/LF.MO)**	**MAX Observed Shoaling Rate (CY/MO)**	**MIN Observed Shoaling Rate (CY/MO)**
	Start Station	**End Station**	**Start Station**	**End Station**							
Fort Sumter Reach	0+00	900+00	- 900+00	0+00	2070	EC	24	33,911	2.426	53,576	15,092
Mount Pleasant Reach	900+00	995+18	0+00	95+21	165	CHL	15	No Maintenance Dredging			
Rebellion Reach	995+18	1077+91	95+21	177+99	115	CHL	15	No Maintenance Dredging			
Bennis Reach	1077+91	1155+87	177+99	256+57	107	CHL	15	No Maintenance Dredging			
Horse Reach	155+87	1178+12	256+57	285+62	51	CHL	15	No Maintenance Dredging			
Hog Island Reach	1178+12	1272+82	286+62	375+79	162	CHL	15	9,787	2.148	11,488	8,178
Drum Island Reach	1272+82	1316+91	375+79	420+71	84	CHL	15	7,658	4.445	8,487	6,842
Meyers Bend Reach	1316+91	1342+47	420+71	446+25	66	CHL	15	1,974	1.135	3,103	624
Daniel Island Reach	1342+47	1412+40	446+25	518+09	144	CHU	19	14,607	4.776	34,597	3,413
Daniel Island Bend	1412+40	1440+56	518+09	543+50	49	CHU	19	875	0.000	875	875
Clouter Creek Reach	1440+56	1508+78	543+50	612+66	94	CHU	19	No Maintenance Dredging			
Navy Yard Reach	1508+78	1566+35	612+66	668+13	84	CHU	19	2,394	1.293	3,307	1,515
North Charleston Reach	1566+35	1615+65	668+13	722+01	61	CHU	19	No Maintenance Dredging			
Filbin Creek Reach	1615+65	1664+42	722+01	768+34	57	CHU	19	542	0.476	624	460
Port Terminal Reach	1664+42	1701+05	768+34	804+96	50	CHU	19	370	0.616	370	370
Ordnance Reach	1701+05	1720+53	804+96	824+44	27	CHU	19	12,045	4.863	18,066	8,155
Ordnance Reach Turning Basin	1698+50	1720+53			73	CHU	19	27,287	12.272	36,016	21,643
Wando River Lower Reach	0+00	71+49	0+00	76+00	81	CHL	15	4,848	1.880	7,045	3,110
Wando River Upper Reach	71+49	119+78	76+00	123+43	65	CHL	15	7,788	3.594	10,915	5,841
Wando River Turn Basin	71+00	109+00	86+56	106+72	39	CHL	15	7,126	3.563	8,256	5,952
Tidewater Reach	0+00	35+73			53	CHL	15	1,813	0.885	1,813	1,813
Custom House Reach	26+00	64+00			94	CHL	15	4,279	3.566	5,038	3,559
Town Creek Lower Reach	35+73	105+82	38+68	73+76	70	CHL	15	17,685	6.036	24,865	11,250
Town Creek Upper Reach	105+82	161+27	105+91	161+24	42	CHL	15	No Maintenance or New Work Dredging			
Town Creek Turning Basin	74+00	87+86	73+76	105+91	7	CHL	15	Maintenance Dredging Reported in Town Creek Lower Reach			
Shipyard River	0+00	64+77	0+00	64+77	67	CHU	19	14,042	4.636	26,225	5,908

1.2.2 Verification of Quantities

In order to verify that the historic records in the database were accurate, the USACE GIS Analyst selected a representative sample of the before- and after-dredging surveys and computed the quantities using ArcGIS. The computed quantities were then compared to the database. Figure 1.2.2 indicates the work flow chart used for the evaluation.

Additionally, manual comparison of geodatabase records was compared to the Dredging History: Federal Channel Maintenance Dredging spreadsheet developed by the Operations Branch. Transcription errors were corrected and any remaining differences in quantities were researched. In some cases, comparison to the handwritten records kept by the section determined whether the geodatabase was correct or the Dredging History spreadsheet.

1.2.3 Non- USACE Dredging

Maintenance dredging is also performed by the property owners in the berthing areas of the port terminals and private docks and piers. Dredging in the inner harbor is typically done by pipeline dredges with disposal in the upland confined disposal areas. Some areas of the lower harbor reaches are dredged by clamshell dredges, loaded into barges and taken to the Offshore Dredged Material Disposal Site (ODMDS). Hopper dredges are used to maintain the entrance channel, with material taken to the ODMDS. These quantities are considered in the life cycle management of disposal area capacity and maintenance.

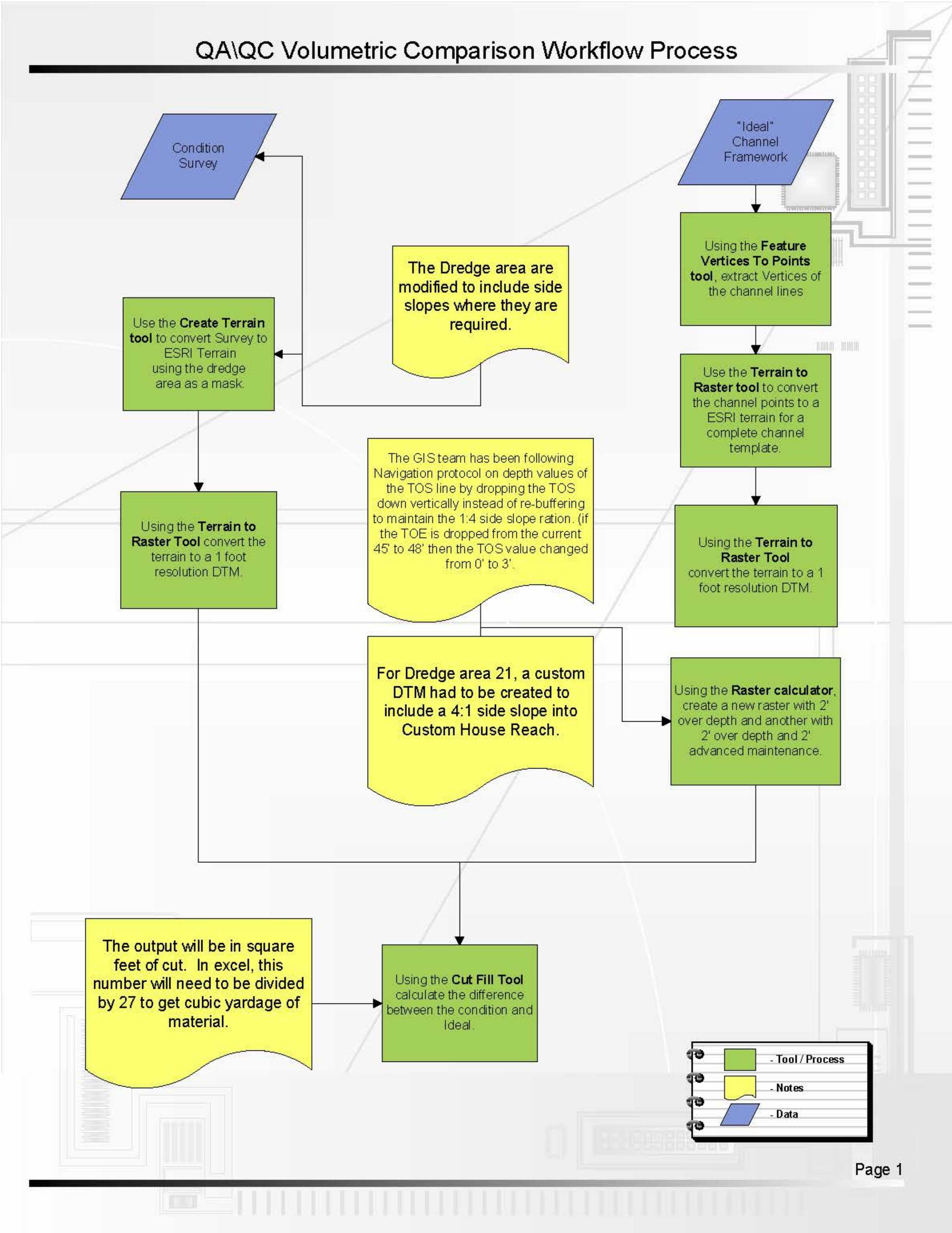

Figure 1.2.2 Shoaling Rate Determination Workflow Diagram

1.3 Shoreline Changes

1.3.1 Objective

The objective of this analysis was to determine recent historic changes in the shoreline features within Charleston Harbor, SC where channel modifications of deepening and widening have occurred. Primary areas of concern included Crab Bank, Shutes Folly, Fort Sumter and the southern shore of Sullivan's Island adjacent to Fort Moultrie (see Figure 1.3.1). Methodology was a GIS analysis using aerial imagery from 1994 to 2011 from SCDNR and NRCS in order to detect shoreline changes within the harbor.

1.3.2 Data

Imagery obtained and used for the study was found on SCDNR's public GIS data clearinghouse, the data clearinghouse held imagery that provided full coverage the study area except the latest data available was 2006. http://www.dnr.sc.gov/GIS/gisdownload.html The earliest readily available spatial data was 1994 and represents an after deepening condition of the 40-foot channel deepening project. The dataset from 1999 represents a condition just prior to the 45-foot deepening project and the 2006 data represents the condition after the 45-foot deepening. The 2011 imagery represents approximately 6 years after the completion of 45-foot project. The National Resource Conservation Service also provides a GIS data clearinghouse to the public. National Agriculture Inventory Products (NAIP) Imagery was used to supplement the final data needed for the 2011 temporal interval. http://datagateway.nrcs.usda.gov/

	Date	Source		Resolution	Image type
-	1994	SCDNR	1m	False Color IR	DOQQ
-	1999	SCDNR	1m	False Color IR	DOQQ
-	2006	SCDNR	1m	False Color IR	DOQQ
-	2011	NAIP	1m	R,G,B, IR	DOQQ

1.3.3 Software and Tools

- ERDAS Imagine 2011:
  AutoSync - auto rectification of orthorectified imagery
- ESRI ArcMap 10.1:
  Editor Tool - General editing of features used for digitization of all vector features required by DSAS tools
- DSAS 4.3 – Digital Shoreline Assessment Tool (DSAS) a USGS tool

1.3.4 Study Area

The shoreline assessment for Charleston Harbor included all three major rivers that converge there (the Ashley, Cooper and Wando Rivers). The Wando River was studied from the convergence with the Cooper River up to the I-526 overpass. The Ashley River was studied from the convergence with Charleston Harbor to the James Island Connector (State Road 30) overpass. The Cooper River was studied from Charleston Harbor jetties at the mouth of Charleston Harbor up to a point slightly north of Bushy Park. Particular areas of concern within the study area are Fort Sumter, Sullivan's Island adjacent to Fort Moultrie, Crab Bank, and

Castle Pinckney (Figure 1.3.1).

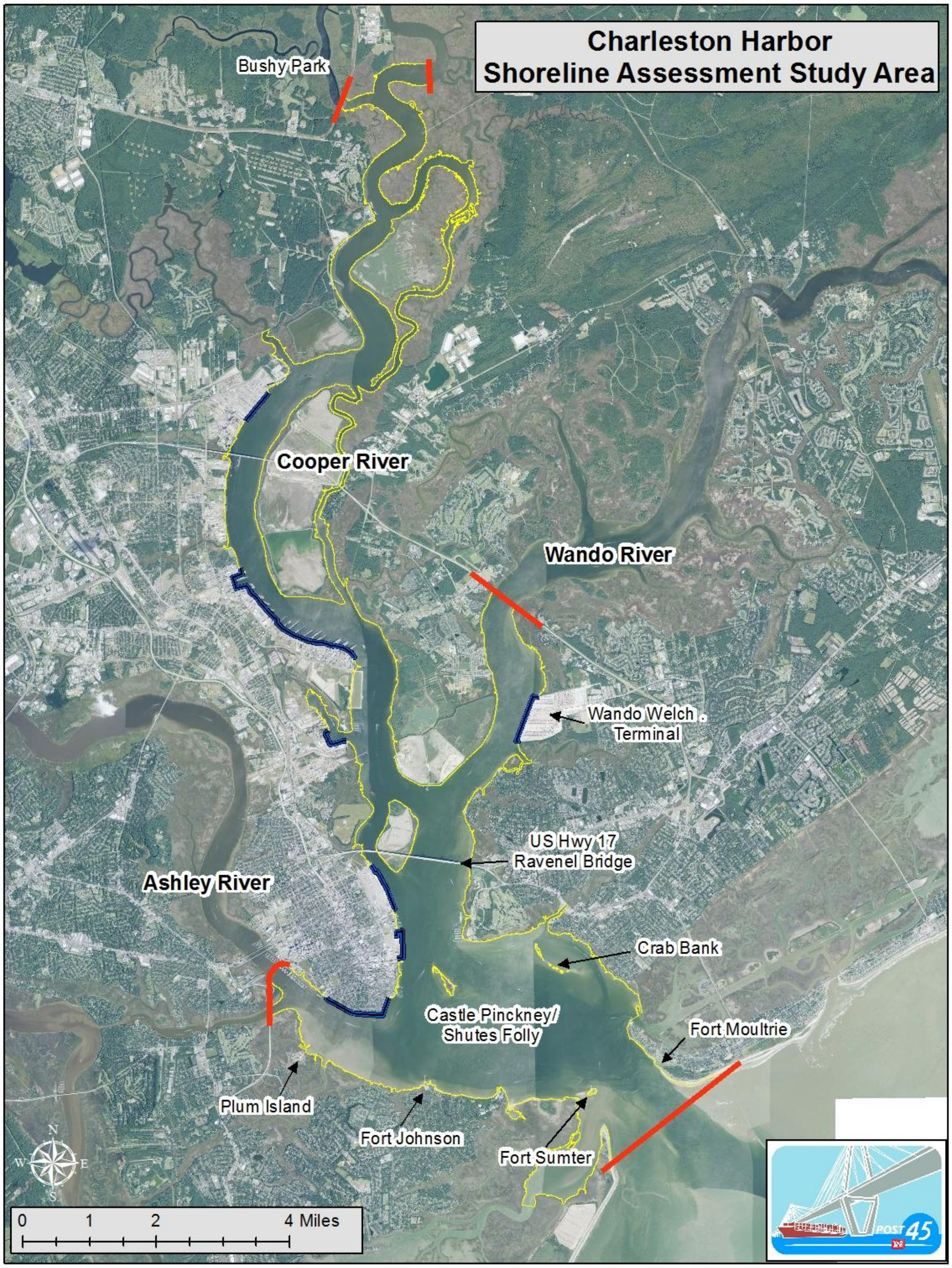

Figure 1.3.1. Shoreline Assessment Study Area

1.3.5 Study Methodology

During initial data mining for this study no LiDAR data was found that covered the full study area over all temporal dates needed, which led to data mining for aerial imagery that held full coverage of the study area over the temporal range. This aerial imagery data was used to digitize shorelines. The digitized shorelines were then compared to measured distances and changes in polygon areas to determine the changes that have occurred to the shoreline over the period of study.

SCDNR Imagery from 1994, 1999, and 2006 were downloaded and the Digital Orthophoto Quarter Quads (DOQQs) mosaic formed using ERDAS IMAGINE to create a single image dataset for each year. NRCS NAIP imagery from 2011 was used as the last data set for assessment. The DOQQs were downloaded and mosaic formed in the same manner as the SCDNR datasets. (Figure 1.3.2).

All data sets have 1m spatial resolution, but are widely varied in their spatial accuracy. Positional inaccuracies of as great as 10m existed between images when compared to a baseline dataset. The baseline dataset used for the comparison was NRCS NAIP imagery of the study area from 2011.

The inaccuracies were created due to differing methodologies in the original ortho-rectification of the imagery. This could be due to several factors including the elevation models used in the initial processing and also the original base imagery used to initially rectify the images. Changes in ortho-rectification algorithms over the various time periods of the imagery can also create discrepancies between the datasets.

The processes used to by the Autosync tool in ERDAS Imagine Suite take a base image and creates thousands of points picked by the software that can be found in both images and then re-rectifies the second image to the first. All images used were run through the Autosync tool and re-rectified to the NRCS NAIP 2011 image. This brought the overall image error down to 2.3m.

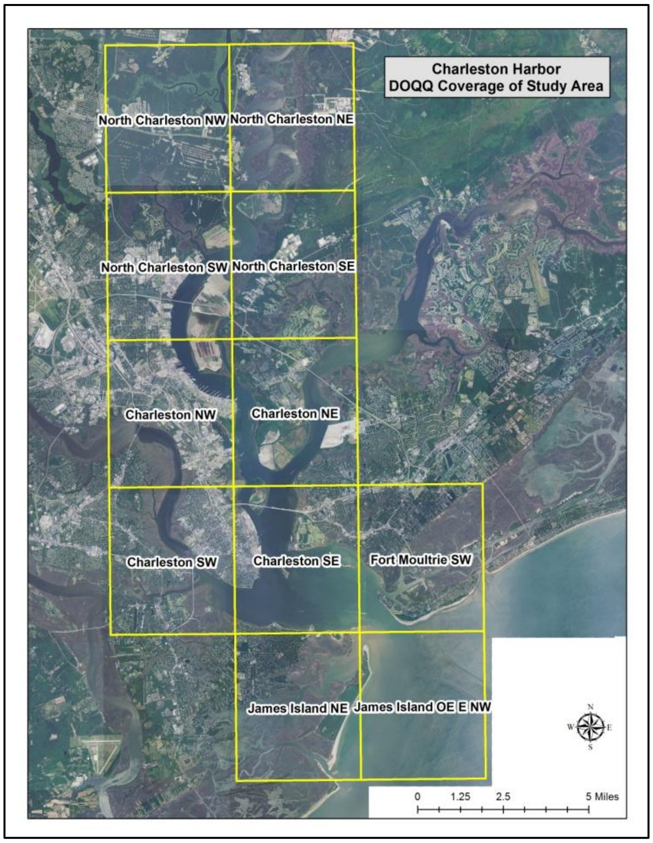

Figure 1.3.2 DOQQ Coverage of Study Area

The positional accuracy was determined by locating multiple features that were evenly distributed and definable throughout all datasets. These features were digitized using visual interpretation within the ESRI ArcMap platform. Similar features within all temporal datasets were digitized and the positional difference between the input image and the base image was calculated. An average positional accuracy was calculated for each dataset by determining the mean distance of all digitized points to the matching point on the base image.

ERDAS IMAGINE is a remote sensing application with raster graphics editor abilities designed by ERDAS for geospatial applications. ERDAS IMAGINE technical specialists were contacted to determine the most accurate and efficient means of correcting the spatial error between the different datasets. AutoSync was recommended to correct the issues of spatial inaccuracies between datasets.

> "IMAGINE AutoSync is an add-on module for ERDAS IMAGINE that gives users the capability of generating highly accurate geometric models from two or more images of potentially dissimilar type, such as data from different sensors or with different resolution. This method can be used to improve the registration between already georeferenced data sets, or it can be used to correlate new raw imagery to an existing georeferenced image base to quickly georeference the raw imagery. IMAGINE AutoSync generates thousands of tie points between the images automatically allowing

for the output images of the process to align more closely with the initial reference image.” (www.erdas.com)

AutoSync processes were performed on the 1994, 1999, and 2006 datasets to match the datasets to the 2011 NAIP imagery. Another positional accuracy was calculated to determine the spatial accuracy of the Autosync process.

Auto Sync parameters:

- Geometric Model Type: Polynomial
- Maximum Polynomial Order: 1
- RMS (Root Mean Square) Threshold: 5
- Projection: Same as Reference Image
- Minimum Point Match Quality - .99
- Correlation Size – 20
- Least Square Size - 30
- After initial processing the RMS threshold was reset to three and the model was re-run to gain a more accurate correlation to the base image

Processing of all the images thru Autosync provided a positional accuracy of 2.33 meters across all four data sets. See Table 1.3.1 for all circular error ratings. Time frames were selected based on channel changes: 1994 represents the condition prior to the last deepening; 1999 was during deepening construction; 2006 was a post deepening condition and 2011 was the existing condition at the time of evaluation.

Table 1.3.1 AutoSync Circular Error Results

Auto Sync Results 2011 NAIP imagery used as the reference image to sync all others.						
Imagery Year	Sync'ed to	RMSE	Std. Dev.	# of Pts	Er. Threshold	Circular Error
1994	2011	1.95	0.76	129	3	2.71 m
1999	2011	2.05	0.75	179	3	2.39 m
2006	2011	1.64	0.687	449	3	1.88 m
2011	N/A	N/A	N/A	N/A	N/A	N/A
Overall						2.33 m

An un-supervised classification of the images was tested to help provide a consistent and repeatable shoreline feature. This was done to separate the image into two classes: land and water. The process proved to be more difficult than expected due to large areas of false classification. The false classification was mainly due to sun angle on the water features having the reflectance values close to that of dry sand and therefore being falsely coded as land features. The ERDAS classification process was discontinued in favor of image interpretation and heads-up digitization of the shoreline within the ArcMap platform for all four datasets. Heads-up digitization is the method of manually tracing geographic features from another dataset (such as an aerial or satellite image). It is usually used with creating vector data from raster data. The digitization standards set forth in a protocol and pilot mapping project preformed by SCDNR and SCGS in the "Digital Mapping of Estuarine Shorelines in South Carolina" (Howard et al, 2011) were followed as closely as possible in the creation of the shoreline in this study.

A map scale of 1:12000 or 1 inch= 1000 ft was used when digitizing all shoreline features. All datasets were set to display in a False Color Infra-Red band configuration. Due to inconsistencies in the imagery, it was decided that the vegetation line would be used as the "shoreline" in areas where the tidal marsh joined the water features (Figure 1.3.3). In areas where a sand/beach joined the water to create a shoreline, the "Wet line" along the beach material was digitized and represented the shoreline of that area (Figure 1.3.4). These shoreline digitization methods were used in the SCDNR/SCGS shoreline study and were followed in this study due to the large tidal range within the study area and unknown tide levels, and the variations in the composition of the shoreline. Hardened shoreline features such as SCSPA Wharfs and armored shoreline segments (Rip-rap) were separated out as "Hardened Shoreline" and excluded from the DSAS analysis (Figure 1.3.5). Some areas of hardened shoreline were digitized as regular shoreline due to erosion factors removing material from behind the erosion prevention structures.

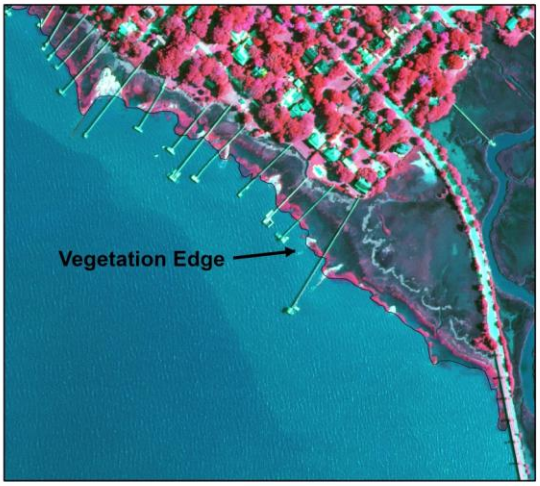

Figure 1.3.3. Digitization of vegetation edge

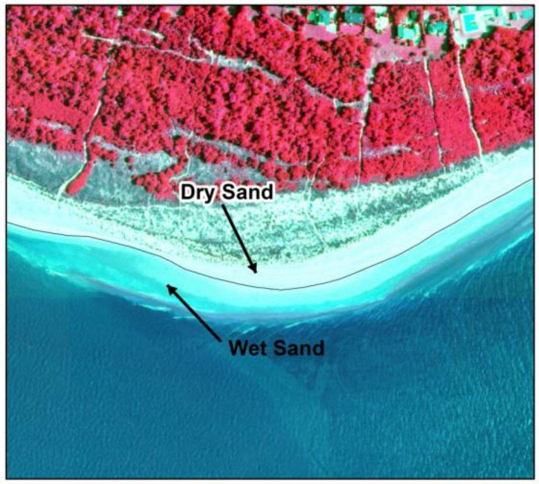

Figure 1.3.4. Wet/Dry line on beach face/sand shore

Figure 1.3.5. Hardened Shoreline, seawall, rip-rap

After all shorelines had been digitized using the above methodology, USGS' DSAS tool was used for the final shoreline assessment to determine the changes during the varying time periods. The Digital Shoreline Analysis System (DSAS) is computer software that computes rate-of-change statistics from multiple historic shoreline positions residing in a GIS. It is also useful for computing rates of change for just about any other boundary change problem that incorporates a clearly-identified feature position at discrete times.

DSAS has two different statistical Methods that were used in this assessment: End Point Ratio: (EPR) and Linear Regression Ratio: (LRR).

- End Point Ratio: (EPR)

"The end point rate is calculated by dividing the distance of shoreline movement by the time interval elapsed between the oldest and most recent shoreline." (Himmelstoss, E.A. 2009)

Parameters used:

- 10 meter transect spacing
- 250 meter transect lengths
- Shoreline measured based on first intersection of transect
- Transect cast direction: auto-detect (based on attribute of baseline)
- Default data uncertainty: 2.33 m (average circular error of all datasets)
- Shoreline intersection threshold: 2
- Confidence interval: 95%
-

- Linear Regression Ratio: (LRR)

"A linear regression rate-of-change statistic can be determined by fitting a least-squares regression line to all shoreline points for a particular transect." (Himmelstoss, E.A. 2009)

Parameters used:

- 20 meter transect spacing (10 m spacing overwhelmed the DSAS program and caused it to malfunction)
- 250 m transect lengths
- Shoreline measured based on first intersection of transect
- Transect cast direction: auto-detect
- Default data uncertainty: 2.33 m (average circular error of all datasets)
- Shoreline intersection threshold: not used (using with LRR caused the program to malfunction due to the large number of cross-sections)
- Confidence interval: 95%

1.3.6 Study Results:

Initial DSAS statistics were run for the End Point Ratio (EPR) calculations to compare the rates of change between data sets in chronological order. The EPR calculations were then run on the first and last shoreline datasets to determine a rate over the entire study temporal interval. Lastly a Linear Regression Rate was calculated to determine the statistically smoothed rate of change using all shoreline features over the full temporal range. Negative values in both the EPR and LRR results are representative of loss of shoreline or erosion. (Table 1.3.2)

All shoreline regression or accretion in a linear manner calculations were completed using DSAS, which computes in meters as its standard output. US units of acreage were used when calculating area of features that were unsuitable for DSAS calculations, since they were not linear features.

Table 1.3.2 DSAS Results

Pair Date	Change Rate (m/yr)	Statistic Method
1994-1999	-0.657	End Point Ratio
1999-2006	0.555	End Point Ratio
2006-2011	-0.605	End Point Ratio
1994-2011	-0.146	End Point Ratio
All Shoreline Iterations	-0.050	Linear Regression Rate

For the smaller, more dynamic features, such as the islands within the study area, the DSAS statistical calculation was not used, but instead the total surface area was calculated and compared across the differing temporal intervals. This was done to help address several features or parts of features that are migrating within the study area and would consistently show loss on the front face and gain on the back face. Thus the total exposed surface area was compared, but not the exact positioning of the feature or to what extent it migrates over the temporal interval. Feature area results can be seen in Table 1.3.3.

Table 1.3.3. Feature Area Results:

AREA YEAR	Crab Bank (SQ FT)	Crab Bank (AC)	Castle Pinckney (SQ FT)	Castle Pinckney (AC)	Fort Sumter (SQ FT)	Fort Sumter (AC)	Drum Island (SQ FT)	Drum Island (AC)	Main Harbor (water area) (SQ FT)	Main Harbor (water area) (AC)
1994	781266	17.94	1301690	29.88	203388	4.67	12266700	281.60	670011010	15,381.34
1999	565391	12.98	1062780	24.40	212680	4.88	12060800	276.88	681643010	15,648.37
2006	358917	8.24	1017670	23.36	220457	5.06	11934300	273.97	672035970	15,427.82
2011	218185	5.01	850508	19.52	241195	5.54	11717700	269.00	708561980	16,266.34
	LOSS/GAIN									
1994-1999	-215,875	-4.96	-238,910	-5.48	9,292	0.21	-205,900	-4.73	-11,632,000	-267.03
1999-2006	-206,474	-4.74	-45,110	-1.04	7,777	0.18	-126,500	-2.90	9,607,040	220.55
2006-2011	-140,732	-3.23	-167,162	-3.84	20,738	0.48	-216,600	-4.97	-36,526,010	-838.52
1994-2011	-563,081	-12.93	-451,182	-10.36	37,807	0.87	-549,000	-12.60	-38,550,970	-885.01

The 1994 - 1999 temporal interval showed the most drastic erosion along the north-west shoreline of James Island along the marsh in front of Plum Island (-12 – 6 m/ yr) and the area adjacent to the James Island Yacht Club (-3.8 – 4 m/yr) , the face of Sullivan's Island seaward of Fort Moultrie (-19 - -2 m/yr) , the north edge of James Island west of Fort Sumter (-7 - -2 m/yr) , the eastern shore of the harbor under the US17 Ravenel Bridge (-9 - -0.8 m yr), and the tidal mud flat upriver from the SCSPA Wando Terminal(-12 – 2 m/yr). Areas of accretion were along the mouth of Shem Creek and the marsh front behind Crab Bank, and the seaward face of Sullivan's Island directly adjacent to the Charleston Harbor North Jetty. There were mixed areas of tidal marsh and mud flat change in the Upper Yellowhouse Creek area. (Figure 1.3.6)

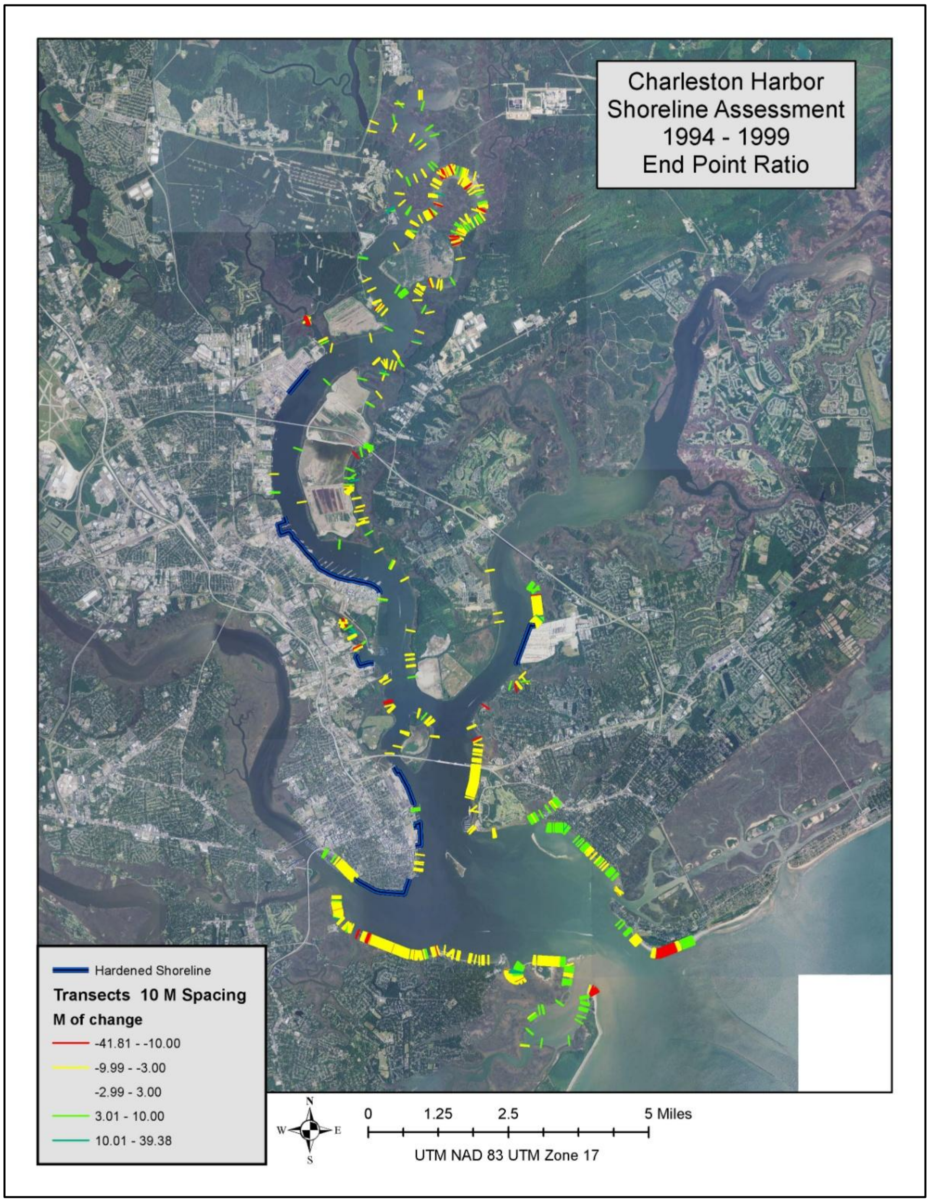

Figure 1.3.6 End Point Ratio: Shoreline Change from 1994-1999

The 1999 – 2006 temporal intervals showed only a few areas of erosion within the study area. The northeast face of James Island west of Fort Sumter (-8 - -1 m/yr) and the tidal marsh edge north of the SCSPA Wando Terminal (-6.8 - -1 m/yr) both showed erosion outside of the uncertainty interval. The seaward face of Sullivan's Island showed growth (-3 – 15 m/yr) in the beach and the marsh along the southern and northern bank of the Ashley River showed accretion (-3 – 11 m/yr). The tidal marsh areas North of the Wando Terminal (-2 – 13 m/yr) and the marsh along the east side of the harbor under the US17 Ravenel Bridge (-2 – 7 m/yr) both showed growth during this time interval. Continued area of mixed change appeared in the marsh area in Upper Yellowhouse Creek (Figure 1.3.7).

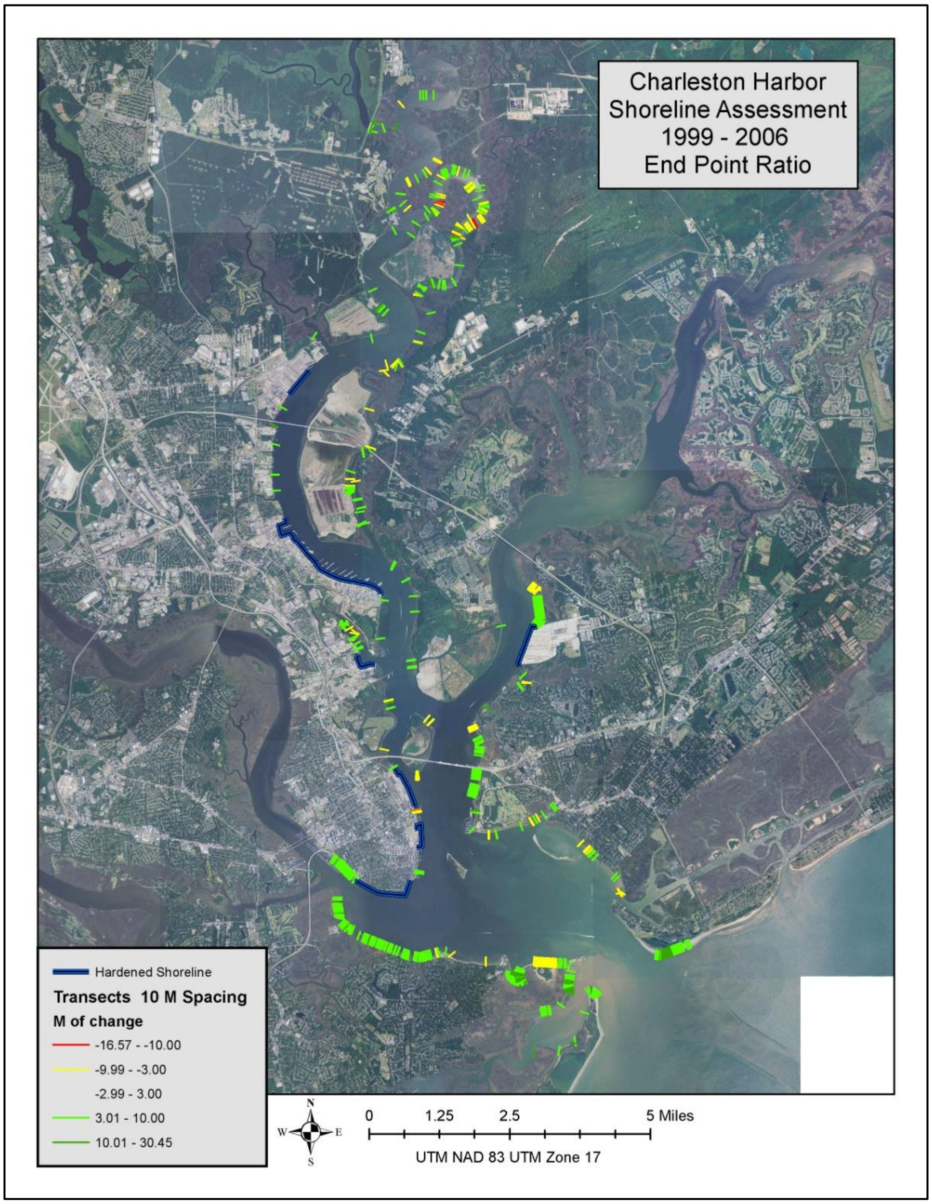

Figure 1.3.7 End Point Ratio: Shoreline Change from 1999-2006

The 2006 – 2011 temporal intervals showed only a few areas of accretion within the study area. The areas of growth were along the seaward face of Sullivan's Island (-2 – 20 m/yr) and the seaward face of James Island, South of Fort Sumter (0.11 – 20 m/yr). Continued loss occurred along James Island west of Fort Sumter (0.7 - -8 m/yr). Consistent loss also occurred along the marsh edge behind crab bank and along the northern edge of James Island in front of Plum Island and Fort Johnson (-13 – 4 m/yr). There were other smaller areas of loss throughout the study area. Upper Yellowhouse Creek continued to be in a state of change with areas of both erosion and accretion (Figure 1.3.8).

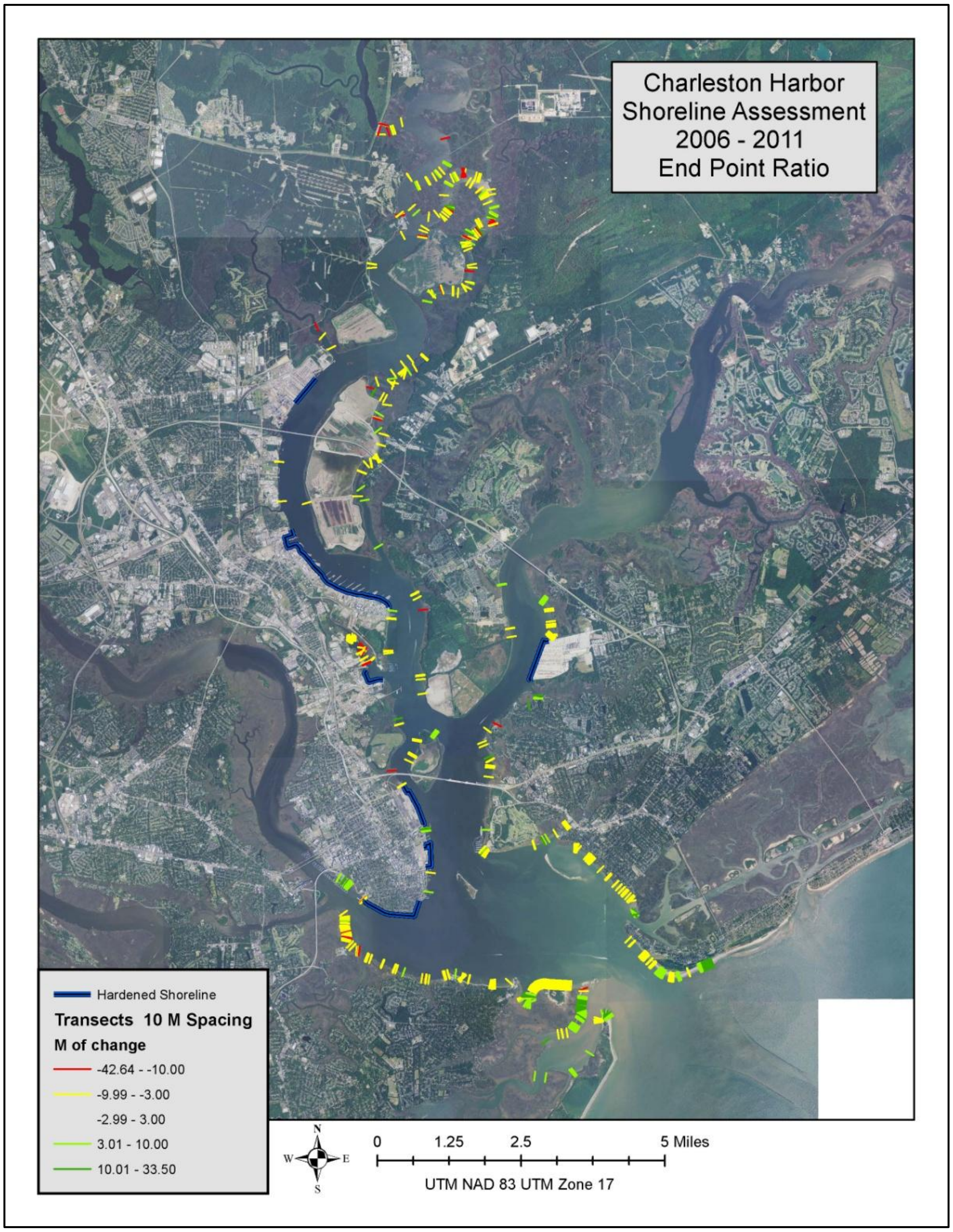

Figure 1.3.8 End Point Ratio: Shoreline Change from 2006-2011

The first and last shoreline intervals 1994 to 2011 were run to see the EPR for the whole study time period. The areas that showed growth over the study interval were the seaward face of Sullivan's Island just inward of the Charleston Harbor Jetties, the seaward face of James Island south of Fort Sumter, and the small inlet behind Fort Johnson (-1.4 – 14 m/yr). Areas of shoreline loss over the study period were the north edge of James Island just west of Fort Sumter (-8 – 1.5 m/yr) and sparse areas of Upper Yellowhouse Creek (Figure 1.3.9).

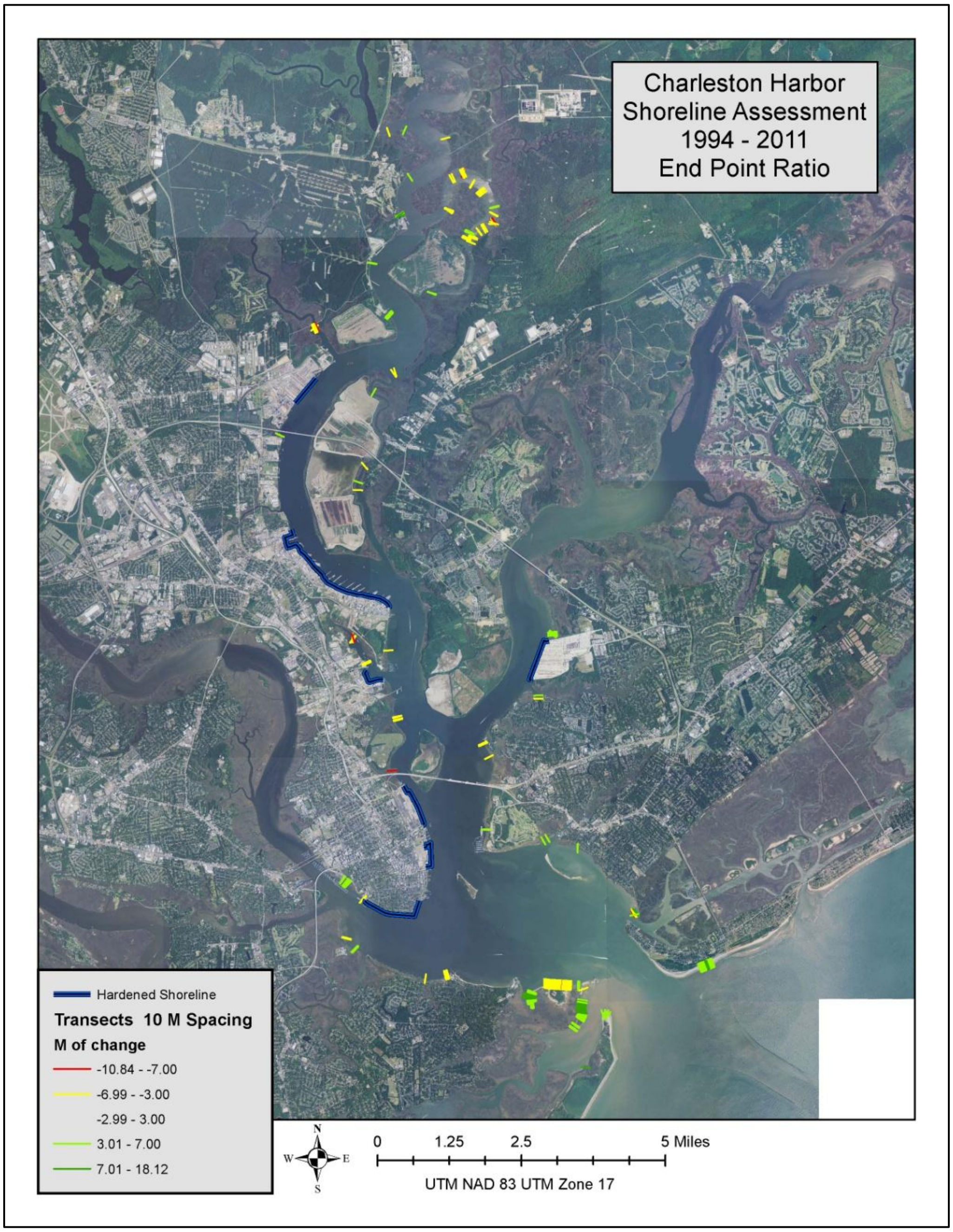

Figure 1.3.9 End Point Ratio: Shoreline Change from 1994-2011

The Linear Regression statistic results from all shorelines combined, 1994 to 2011, provided the same picture as the 1994-2011 EPR with some areas showing a more extreme rate of accretion or erosion. The additional areas of change seen through this model were areas of erosion between Fort Johnson and the James Island Yacht Club (Figure 1.3.10).

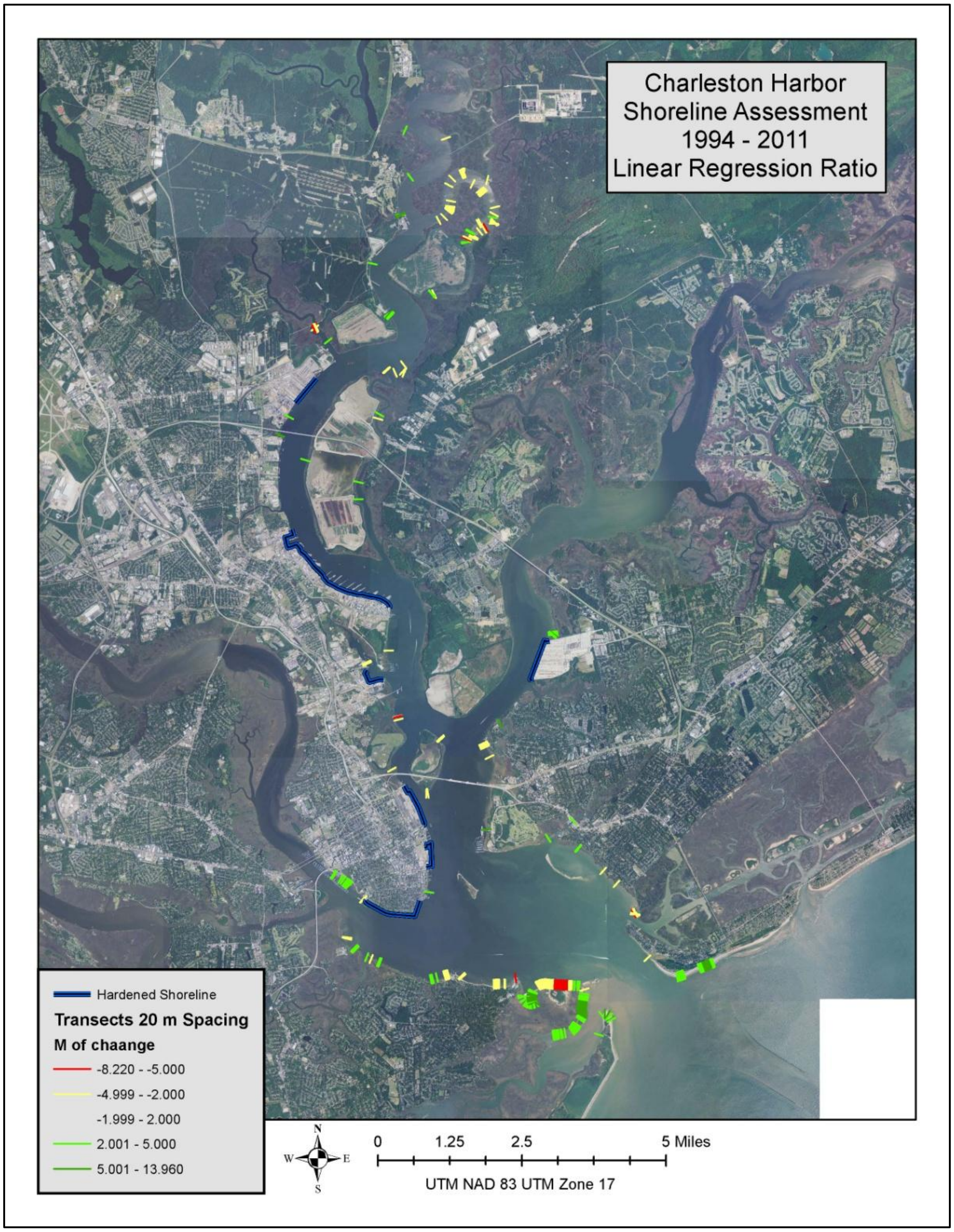

Figure 1.3.10 Shoreline Change Results from Linear Regression Analysis 1994-2011

Surface calculation results for three areas of Interest within Charleston Harbor are presented in Figures 1.3.11 through 1.3.13 and discussed in the next paragraphs.

As shown in Figure 1.3.11, the northern side (facing the channel) and the eastern side (facing the ocean) of Fort Sumter is armored with rip-rap and appeared to remain stable throughout the study interval (1994-2011). The tidal marsh/sand flat area to the landward side of the island appears to be slowly accreting material in a very dynamic way.

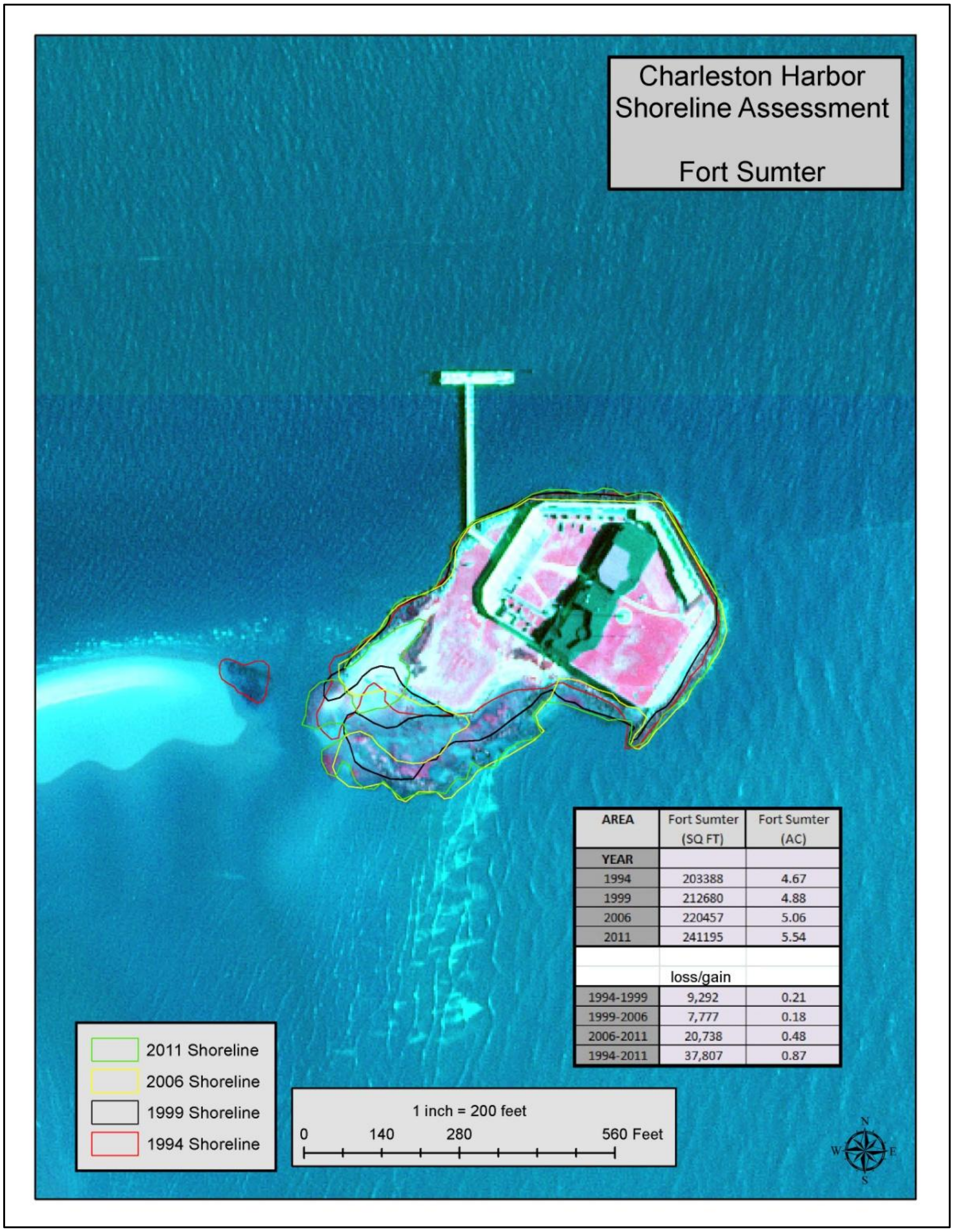

Figure 1.3.11 Fort Sumter Surface Change Results

Crab Bank was originally created as an inner-harbor dredge spoil area by the USACE in the early 1900s. As seen in the shoreline outlines in Figure 1.3.12, the surface area above the high tide line on Crab Bank has been condensing and migrating to the northeast towards Mount Pleasant. As the island has migrated over the last twenty years, it has become more tide and waved washed leaving a significantly smaller area exposed during high tide and wind events.

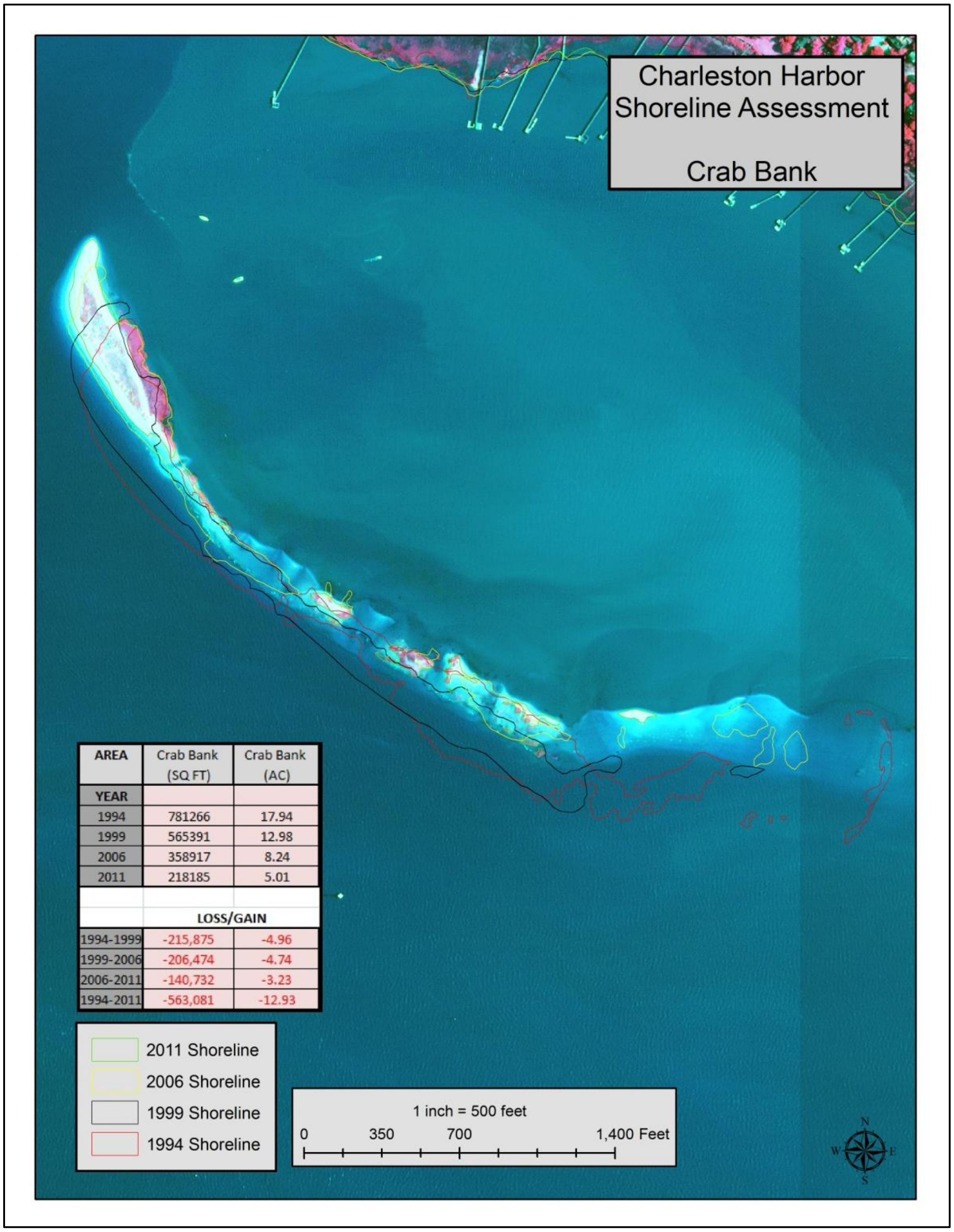

Figure 1.3.12 Crab Bank Surface Change Results

Shutes Folly area above the high tide line has been slowly reducing on all sides, the small tidal sand spit on the northern point appear to be migrating towards the main northern shoreline of the island. The USACE constructed a 294-foot rip-rap armored breakwater at elevation 7 MLW and a top width of 12 feet at a distance no closer than 30 feet to the face of Castle Pinckney in 1999. The structure appears to be holding and preventing any further erosion at Castle Pinckney end (Figure 1.3.13).

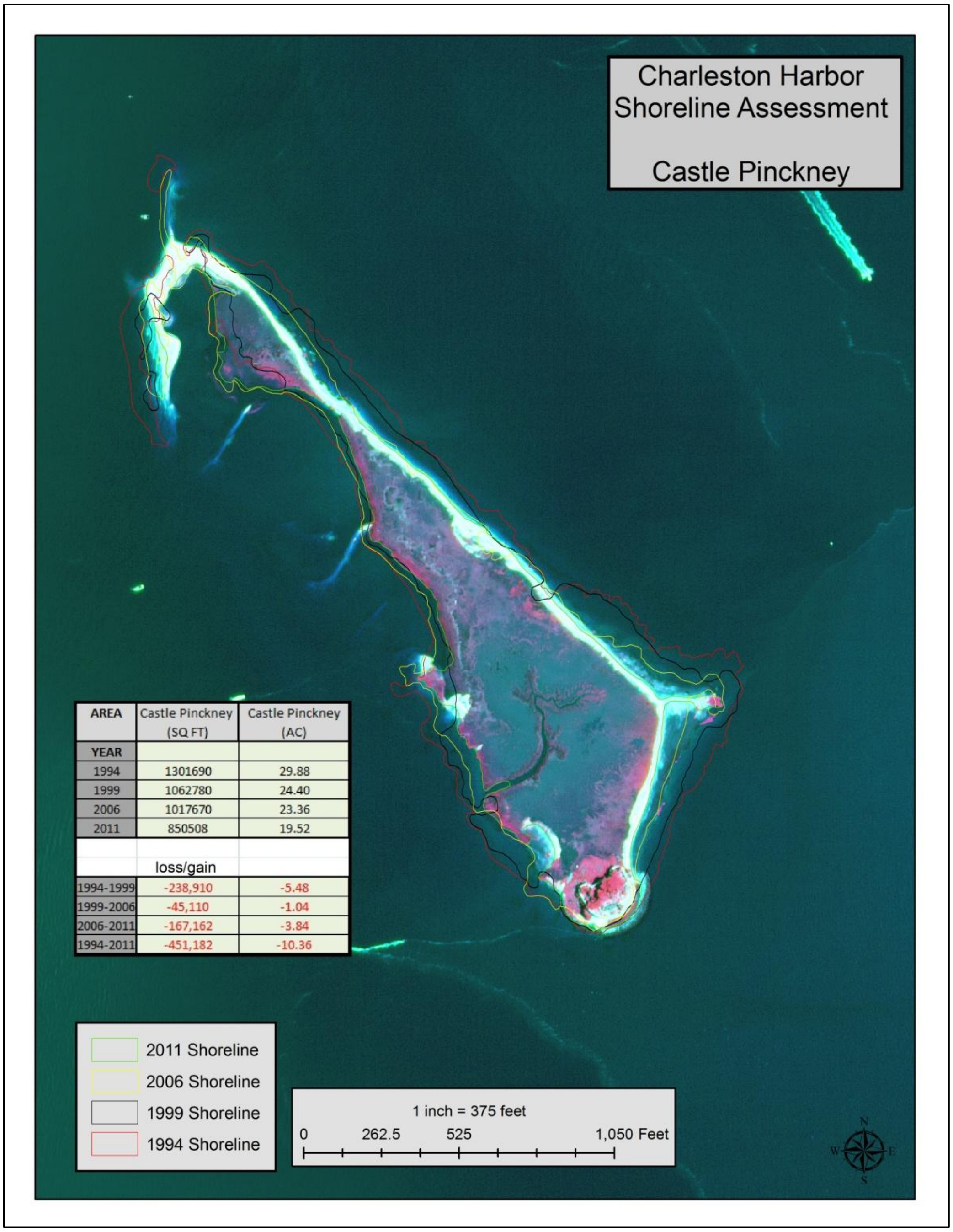

Figure 1.3.13 Shutes Folly Surface Change Results

Contributing factors to shoreline changes are discussed in section 2.5.

1.4 Salinity

1.4.1 History

Charleston Harbor conditions were monitored by USACE Waterways Experiment Station Hydraulics Laboratory, Estuaries Division (WES-HE-P, now part of ERDC Coastal and

Hydraulics Lab) both before and immediately after the Cooper River rediversion in 1985 and again in 1987 for comparison purposes. Background salinity conditions were previously compiled for the Bushy Park Reservoir from the USGS gages that had been there since 1980. A schematic numerical model was developed to examine salinity intrusion and applied to predict harbor–deepening shoaling effects. From these analyses WES developed a salinity alert system of gages and required discharges from Jefferies Hydroelectric Station, based on various tide and salinity thresholds, to push the salinity down river and away from the entrance of the Bushy Park Reservoir. There was also an alternative for relocation of the entrance canal to Bushy Park, but the gage alert system was deemed more cost effective until monitoring analysis indicated the need for canal relocation. The USGS was contracted to install and monitor a system of satellite-telemetered water level and water quality monitors, providing real-time data.

1.4.2 Monitoring System

The salinity alert system for protecting the reservoir, described in Design Memorandum No. 15 Water Monitoring Plan of the CRRP, was implemented when the project was completed in 1985. A system of tide (water stage) gages and water quality monitoring stations has been installed which provides advance warning of a salinity threat to the reservoir. The system measures Specific Conductance (an indicator of salinity), which is a measure of the ability of water to transmit an electrical current and is proportional to the amount of dissolved solids in the water; thus, the greater the conductance, the greater the salinity. Specific conductance is standardized to 25 degrees Celsius. Provisions have been included in the contract between the Corps and the S.C. Public Service Authority, which permit emergency flow releases to repel salinity intrusion.

The monitoring plan consists of a system of real-time gages (Figure 1.4.1), which transmit salinity and tide information to a U.S. Geological Survey (USGS) computer in Columbia, SC. USGS personnel use the tide and salinity gage data to determine if additional flow releases are necessary to prevent salinity intrusion in Bushy Park Reservoir. The system is monitored 24 hours a day, 7 days a week by USGS. USACE contracts USGS to maintain and monitor the system, and to call South Carolina Public Service Authority (SCPSA) when flow releases from Pinopolis Dam are required. Charleston District personnel also monitor the tide and salinity gages during working hours as a back up to USGS. The system has successfully protected Bushy Park Reservoir from salinity intrusion since the construction of the Cooper River rediversion (September 1985); a period, which included multiple droughts and a direct hit of major hurricane (Hugo) in 1989. The following sections describe the existing salinity gages in the system and their locations.

1.4.3. Salinity Gages

- **Gage 02172020 (West Branch Cooper River at Pimlico)** is located near Moncks Corner, SC. (above Bushy Park entrance) (Berkeley County) Latitude 33^{o} 05’3’’, Longitude 79^{o}56’57’’ Hydrologic Unit 03050201, at Pimlico on right bank, 1.1 miles upstream from Seaboard Coast Line Railroad bridge, 2.1 miles downstream from Molly Branch, 7.8 miles southwest of Moncks Corner .

- **Gage 02172040 (Back River at Dupont Intake)** is located near Kittredge, SC. (Berkeley County) Latitude 33°03’49’’, Longitude 79°57’26’’, Hydrologic Unit 03050201, on left bank of Durham Canal, the entrance to Bushy Park Reservoir, 0.5 miles upstream of Secondary Road 9.

- **Gage 02172050 (Cooper River at Dean Hall or Goose Creek)** is located near Goose Creek, SC. (Berkeley County) Latitude 33°03’27’’, Longitude 79°56’11’’, Hydrologic Unit 03050201, on right bank, below Bushy Park Entrance, 6.2 miles downstream from Seaboard Coast Line Railroad Bridge, 7.4 miles upstream from Goose Creek.

- **Gage 02172053 (Cooper River at Mobay)** is located at North Charleston, SC (Berkeley County) Latitude 32°59’00”, Longitude 79°55’23”, Hydrologic Unit 03050201, on right bank of Cooper River, 9.9 mi from confluence of East and West Branch Cooper River.

- **Gage 021720711 (Cooper River at Customs House)** is located at Charleston, SC. (Charleston County) Latitude 32°46’44’’, Longitude 79°55’26’’, Hydrologic Unit 03050201, at South Carolina State Ports Authority Dock, 0.25 miles east of Customs House at Charleston.

1.4.4 Salinity Trends

This analysis to assess trends in salinity changes in the river used the Pimlico (2020), Mobay (2053) and Goose Creek (2050) gages located on the Cooper River for analysis. The Dupont (2040) gage is located within the Durham Canal to Bushy Park. The Customs House gage does not measure specific conductance, but is used to determine a tide factor. When the threshold for the tide factor is reached, it is assumed that it may be a precursor for a salinity alert and thus a tidal alert is called and releases from Pinopolis Dam are required.

Comparison of specific conductance maximums, minimums and averages at the three gages are shown in Table 1.4.1. The periods of analysis were selected based on the significant changes in the Charleston Harbor and are explained below:

- Pre-rediversion is the period prior to July 1986 when there were no regulated releases through Pinopolis Dam into the Cooper River.

- Post-rediversion, August 1986 until May of 1988 represents a significant hydrodynamic transition period for the harbor as it adjusted to a regulated weekly average of 4500 cfs. This is prior to the harbor deepening and represents when only maintenance dredging was performed in the federal channel.

Figure 1.4.1 USGS Gages Location Map

- June 1988 to April 1992 represents the harbor conditions while the construction of the federal channel was deepened from -35 feet MLLW to -40 MLLW. Dredging did not occur during the entire time frame. Annual federal funding was received by the District to do a portion of the new work dredging each year until the entire channel was at authorized depth.

- From May 1992 until June 1999 is a period in which no changes were made to the federal channel, other than annual maintenance.

- July 1999 to May 2004 represents the period of the last harbor deepening from - 40 MLLW to -45 MLLW.

- June 2004 to December 2011 is another period in which no changes were made to the federal channel, other than annual maintenance.

Table 1.4.1 Specific Conductance at USGS Gages Throughout Changes in Charleston Harbor

	Specific Conductance (micro mhos/cm)				**Specific Conductance** (micro mhos/cm)				**Specific Conductance** (micro mhos/cm)				**Avg Daily Flow**
	MAX	**MIN**	**AVG**		**MAX**	**MIN**	**AVG**		**MAX**	**MIN**	**AVG**		**CFS**
Period of Analysis	**Mobay gage POR:1983- Present**				**Goose Creek gage POR:1970- Present**				**Pimlico gage POR:1983- Present**				**Tailrace canal**
					(Alert level is 1550)				**(Alert level is 180)**				**POR 1978-present**
Pre Rediversion (Oct 1983-July 1986)	33700	60	2620		4270	31	111		334	60	93		12906
Post Rediversion (August 1986- May 1988)	35200	111	3854		3710	30	206		285	40	116		4364
During Deepening to 40ft (Jun 1988-Apr 1992)	25400	100	3192		2890	62	194		318	75	106		5286
Post Deepening to 40ft (May 1992-Jun 1999)	37500	31	2733		3030	46	170		286	62	97		5126
During Deepening to 45ft (July 1999-May 2004)	26900	64	3426		2180	59	224		220	59	125		4796
Post Deepening to 45 ft (June 2004 – Dec 2011)	29500	83	4744		2630	71	244		219	61	109		4893

(Use minimums with caution as they may reflect gage error at low levels. (For example it is not likely that Mobay would go as low as 31 micro mhos/cm).

As explained in the main report and in Appendix L, USACE used the Cowardin classification system to delineate various wetland types. Marine and estuarine wetlands are designated those having average annual a salinity concentrations over 5 parts per thousand (ppt). Brackish marshes have salinities between 0.5 and 5.0 ppt. Freshwater marshes are those with salinities less than 0.5 ppt. Salinity (in ppt) was derived from specific conductance using the following formula provided by USGS. Table 1.4.2 shows the specific conductance (SC) in units of micromhos/cm values above that were converted to Salinity (S) in units of ppt.

$$S = \frac{.47413}{\frac{1}{0.001SC^{1.07}} - 0.7464 \times 10^{-3}}$$

Where S= salinity in ppt; and SC = specific conductance in micromhos/cm

Table 1.4.2 Salinity at USGS Gages throughout changes in Charleston Harbor

	Salinity (PPT)				Salinity(PPT)				Salinity (PPT)				Avg Daily Flow
	MAX	MIN	AVG		MAX	MIN	AVG		MAX	MIN	AVG		CFS
Period of Analysis	**Mobay gage POR:1983- present**				**Goose Creek gage POR:1970- present**				**Pimlico gage POR:1983- present**				**Tailrace canal**
													POR 1978-present
Pre Rediversion (Oct 1983-July 1986)	21.12	0.02	1.33		2.25	0.01	0.05		0.15	0.02	0.04		12906
Post Rediversion (August 1986- May 1988)	22.16	0.05	2.01		1.93	0.01	0.09		0.12	0.02	0.05		4364
During Deepening to 40ft (Jun 1988-Apr 1992)	15.47	0.04	1.65		1.48	0.02	0.08		0.14	0.03	0.04		5286
Post Deepening to 40ft (May 1992-Jun 1999)	23.77	0.01	1.39		1.56	0.02	0.07		0.12	0.02	0.04		5126
During Deepening to 45ft (July 1999-May 2004)	16.48	0.03	1.78		1.09	0.02	0.10		0.09	0.02	0.05		4796
Post deepening to 45 ft (June 2004 – Dec 2011)	18.23	0.03	2.66		1.34	0.03	0.10		0.09	0.02	0.04		4893

A trend of increasing maximum salinity levels is observed, which would be affected by the drought years that occur in 1988, 2002, 2007, 2008 and 2011. However, there is little to no change in average for the Goose Creek and Pimlico gages.

While presenting the average, minimum, and maximum concentrations is valuable in determining trends over time, percentiles provide another means to analyze the data. A review of the Mobay gage hourly data resulted in the discovery of numerous blocks of missing data that may bias the percentiles. This occurred predominantly when levels were dropping, so percentiles is not the best method for analyzing the data. Table 1.4.3., and 1.4.4 demonstrate the since rediversion in 1986 for Pimlico and Goose Creek gages.

Table 1.4.3 Pimlico Salinity Percentiles

USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC						
Salinity	Aug 86-May88	Jun88-Apr92	May92-Jun99	Jul99-May04	Jun04-Dec11	Aug 86-Dec11
1st percentile	0.025	0.030	0.026	0.025	0.029	0.026
10th percentile	0.033	0.033	0.029	0.030	0.033	0.033
50th percentile	0.048	0.039	0.037	0.050	0.039	0.039
90th percentile	0.053	0.052	0.044	0.063	0.055	0.055
99th percentile	0.058	0.060	0.062	0.068	0.062	0.065

Table1.4.4 Goose Creek Salinity Percentiles

USGS 02172050 Cooper River near Goose Creek, SC						
Salinity	Jul86-May88	Jun88-Apr92	May92-Jun99	Jul99-May04	Jun04-Dec11	Jul86-Dec11
1st percentile	0.019	0.030	0.027	0.029	0.033	0.029
10th percentile	0.041	0.036	0.034	0.041	0.042	0.042
50th percentile	0.066	0.059	0.049	0.071	0.075	0.063
90th percentile	0.154	0.142	0.123	0.166	0.207	0.188
99th percentile	0.370	0.378	0.339	0.403	0.458	0.405

There has been variability of salinity values within the system and these are dependent upon factors other than changes in the federal channel depth. For example, drought conditions have a distinct impact on salinity levels, as noted by the jump in the July 1999 – 2004 time frame and continued though 2011. There have been several instances of reduced flows of a 3000 cfs weekly average requested by Santee Cooper due to drought conditions during 2002 drought and from 2004 to 2011 period that impact the salinity of the river and lower harbor (October 23, 2007 through April 10, 2008; June 27, 2008 through August, 2008; November 8, 2011 through January 19, 2012). These periods of drought and reduced flows had higher instances of salinity alerts than normal conditions. This is further discussed in the next section.

1.4.5 Salinity Alerts

Documented in Memorandum for Record from WES-HE-P, the alert system requires minimum releases for various level alerts of specific conductance thresholds at salinity gages along the Cooper River and also for a tide factor at Customs House. These are outlined below in Table 1.4.5.

Table 1.4.5 Specific Conductance Thresholds for Required releases

SPECIFIC CONDUCTANCE THRESHOLDS FOR KEY STATIONS.		
CONDUCTANCE THRESHOLD (Micro Mhos/cm)		
	Level 1	Level 2
Station	Alert Value	Alert Value
Dean Hall (Goose Creek) 02172050	1,550	1,900
Dupont Intake 02172040	260	300
Pimlico 02172020	180	200
Converted to Salinity ppt		
	Level 1	Level2
Station	Alert Value	Alert Value
Dean Hall 02172050	0.76	0.94
Dupont Intake 02172040	0.11	0.13
Pimlico 02172020	0.08	0.08

The Tide Factor for determining required release to prevent salinity intrusion is based on the following formula.

Tide Factor = (Mean Tide Level – 17.9) +Tide Range

Where:

Mean Tide Level = Average of hourly values from 0600 preceding day to 0500 current day.

Tide Range = The highest value minus the lowest value from the same period used to compute the mean tide.

If the Tide Factor us greater than 10, a Level 3 response will be issued. If the Tide Factor is greater than 13, a Level 4 response will be issued.

The required flow releases associated with each level alert are

- Level 1 Response – 4,000 cfs minimum per hour for 15 hours.
- Level 2 Response – 6,000 cfs minimum per hour for 15 hours.
- Level 3 Response – requires an average daily flow of at least 3,000 cfs to be released prior to 2400 hours of the current day.

- Level 4 Response – requires an average daily flow of at least 4,000 cfs to be released prior to 2400 hours of the current day.

Level 3 and level 4 responses do not require SCPSA to release for a fixed period of time. These responses do require SCPSA to release a specific volume of water, but also allows flexibility in performing the release. The average daily flows for Level 3 or 4 responses is based on a 24 hour time period from 2400 hours the previous day to 2400 hours the current day.

There is no trend for alerts after past deepening projects. The primary influencing factor appears to be droughts, During low flow conditions, there are often increases in alerts. During the 2007-2008 time frame there were 22 alerts of Pimlico and Goose Creek (previously known as Dean Hall) combined and a total of 65 alerts of all gages (tidal or salinity). During the summer of 2008 there were 17 alerts between Goose Creek and Pimlico gages combined and a total of 32 alerts of all gages (tidal or salinity). During the Nov – Dec 31, 2011 time frame, there were a total of 16 alerts of all gages (tidal or salinity). Figure 1.4.2 demonstrates the salinity alerts at the Goose Creek and Pimlico gages.

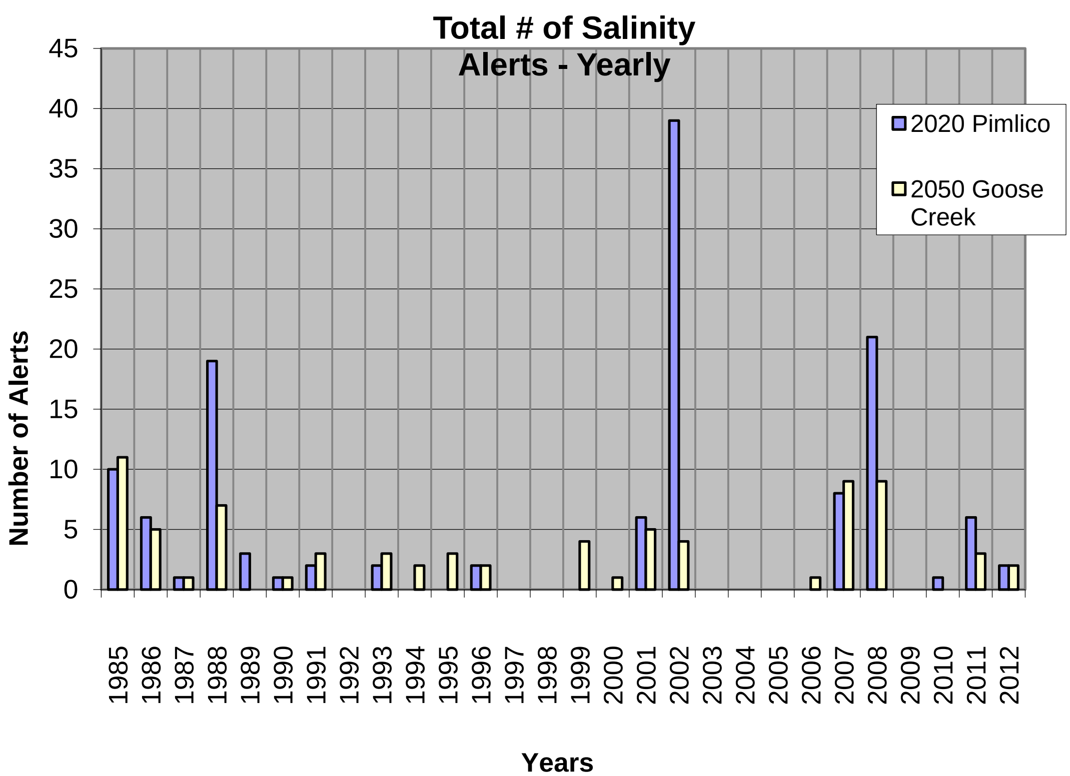

Figure 1.4.2. Salinity alerts at Goose Creek (2050) and Pimlico (2020) gages

Overall, the historic records indicate August through December time period seem to be when there are the most salinity alerts (Table 1.4.6). With the exception of 1988, 2002, 2007, 2008 and 2011(all drought years when flows were reduced from Pinopolis to 3000 cfs for some portion of the year), overall alerts for these gages have decreased drastically since the late 1980's. Note the higher number of alerts at Dupont (2040) over Goose Creek (2050). These occurred in drought years and discussions with USGS seem to indicate it has to have something to do with residual salinity within Bushy Park. However, the alert system has successfully reacted to the situation.

Table 1.4.6 Alert totals by month

2020	**Pimlico**													
		jan	**feb**	**mar**	**apr**	**may**	**jun**	**jul**	**aug**	**sep**	**oct**	**nov**	**dec**	**tot**
1985-1989.		0	0	0	1	1	3	12	9	3	7	1	2	39
1990-1999		0	0	0	0	0	1	3	0	1	1	0	1	7
2000-2009		8	5	0	2	12	9	10	14	4	4	1	5	74
2010-2012		1	1	0	0	1	0	0	0	0	0	3	3	9
total		**9**	**6**	**0**	**3**	**14**	**13**	**25**	**23**	**8**	**12**	**5**	**11**	**129**
2040	**Dupont**													
		jan	**feb**	**mar**	**apr**	**may**	**jun**	**jul**	**aug**	**sep**	**oct**	**nov**	**dec**	**tot**
1985-1989.		**0**	**0**	**1**	**2**	**4**	**14**	**10**	**8**	**3**	**9**	**4**	**2**	**57**
1990-1999		**1**	**1**	**1**	**0**	**1**	**1**	**1**	**15**	**8**	**6**	**5**	**5**	**45**
2000-2009		**16**	**13**	**3**	**5**	**0**	**4**	**18**	**23**	**7**	**12**	**15**	**14**	**130**
2010-2012		**1**	**2**	**1**	**1**	**3**	**4**	**0**	**2**	**3**	**4**	**9**	**13**	**43**
total		**18**	**16**	**6**	**8**	**8**	**23**	**29**	**48**	**21**	**31**	**33**	**34**	**275**
2050	**Goose Creek**													
		jan	**feb**	**mar**	**apr**	**may**	**jun**	**jul**	**aug**	**sep**	**oct**	**nov**	**dec**	**tot**
1985-1989.		1	0	0	1	1	6	4	1	3	7	0	0	24
1990-1999		1	0	1	0	1	2	0	3	5	5	0	0	18
2000-2009		2	2	4	3	3	0	1	5	2	1	2	4	29
2010-2012		0	0	0	0	0	1	0	2	0	0	1	1	5
total		**4**	**2**	**5**	**4**	**5**	**9**	**5**	**11**	**10**	**13**	**3**	**5**	**76**

Ambient conditions, release patterns, drought, as well as, the tides are other factors that appear to have an impact on alerts. A review of all gages within the alert system is shown on Table 1.4.7.

Based on the historical records, overall the September/October time period is when the higher number of tidal alerts was observed (Table 1.4.8). The most tidal alerts of 76 occurred in 2009 with the majority (25) of those occurring in September. Of the 1999 total 74 tidal alerts, 18 occurred in September and 17 in October. Overall from 1991 to Year 2009, tidal alerts have increased when compared to earlier years but there is not a constant rise and averages to approximately 44.7 tidal alerts per year with a median of 43. While it varies up and down from one year to the next, storm season is a vital factor, as are seasonal high tides, as might be expected. Tidal alert responses are generally performed within the required weekly flow releases and do not result in additional weekly average flows.

While tide factor is the first indicator of potential salinity changes, there have not always been noticeable salinity changes when tidal alerts exist, as shown in Table 1.4.9. The exception to this is the years of significant salinity alerts which generally were years of major droughts (2002, 2007 and 2008), then the tidal and salinity alerts coincided.

Table 1.4.7. Summary of Alerts

Total # of Alerts	Salinity	Salinity	Salinity	Tidal
Yearly	Pimlico	Dupont	Goose Creek	Customs House
1985	10	18	11	0
1986	6	11	5	8
1987	1	2	1	16
1988	19	25	7	8
1989	3	1	0	8
1990	1	1	1	7
1991	2	6	3	43
1992	0	2	0	42
1993	2	6	3	39
1994	0	1	2	45
1995	0	6	3	64
1996	2	5	2	28
1997	0	7	0	39
1998	0	5	0	25
1999	0	6	4	74
2000	0	1	1	59
2001	6	13	5	25
2002	39	17	4	42
2003	0	0	0	39
2004	0	0	0	22
2005	0	2	0	55
2006	0	5	1	26
2007	8	32	9	55
2008	21	53	9	49
2009	0	7	0	76
2010	1	4	0	47
2011	6	22	3	44
2012	2	17	2	71
Total of All	129	275	76	1056

Drought years highlighted in yellow.

Table 1.4.8 Summary of tidal alerts by month

710-711 Tide Gage														
		jan	**feb**	**mar**	**apr**	**may**	**jun**	**jul**	**aug**	**sep**	**oct**	**nov**	**dec**	**tot**
1985-1989		2	2	3	9	0	1	1	2	7	6	4	3	40
1990-1999		14	22	4	16	31	28	15	40	85	93	32	26	406
2000-2009		4	9	14	16	27	22	15	37	138	99	49	18	448
2010-2012		4	2	5	6	12	16	5	25	24	27	29	7	162
total		**24**	**35**	**26**	**47**	**70**	**67**	**36**	**104**	**254**	**225**	**114**	**54**	**1056**

Table 1.4.9. Summary of Concurrent tidal and salinity alerts

	# Days concurrent alerts
Year	Tidal and Salinity
2002	25
2003	1
2004	0
2005	2
2006	4
2007	22
2008	20
2009	1
2010	2
2011	10

2.0 Design Considerations

All geodetic and hydraulic data comply with ER 1110-2-8160 *Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums.* Design considerations of a federal navigation project include the factors that are considered in determining the depth, widths, angles of bends and turning basin sizes needed to safely and efficiently transport the vessels. Quantities of dredged material and the placement or disposal of the material is an important component to the overall management of the sediment in the system. These are addressed in the following sections.

2.1 Channel Modifications

Engineering guidance EM 1110-2-1613 “Hydraulic Design of Deep-Draft Navigation Projects (herein referred to as “guidance” or EM 1613) identifies the following main factors for navigation channel design:

(1) Design ship beam, length, and draft.
(2) Local piloted ship control.
(3) Channel cross section and alignment.
(4) River and tidal currents.
(5) Navigation traffic pattern (one- or two-way).
(6) Vessel traffic intensity and congestion.
(7) Wind and wave effects.
(8) Visibility.
(9) Quality and spacing of navigation aids.
(10) Composition of channel bed and banks.
(11) Variability of channel and currents.
(12) Speed of design ship.

2.1.1 Design Vessel

Two containership design vessels were selected for this study and used in the engineering and design considerations. Due to the air draft restriction of the Don Holt Bridge (I-526) a different design vessel was selected for the North Charleston Terminal than has been selected for the Wando and new Navy Base Terminal (NBT) at the former Navy Base. The upper reach of the channel that extends from the NBT (Daniel Island Bend) to the North Charleston Terminal (NCT) and Ordnance turning basin has a design vessel with a 1100 feet length, 141 feet beam and draft of 48 feet. The design vessel selected for the lower harbor and Wando River is 1200 feet in length, approximately 160 feet beam and 50 feet in depth.

2.1.2 Tabletop Exercise

A tabletop exercise using a scaled cartographic representation of the harbor, federal channels and scaled design vessels was done in conjunction with Charleston Harbor Pilots Associations and the Charleston tug pilots to discuss existing problem areas and potential modifications they felt were needed to navigate the post-Panamax ships (see vessel discussion in Economics appendix) expected to call on the Port of Charleston and the subsequent channel modifications needed. The Pilots' requests were evaluated based on EM 1110-2-1613. USGS velocity measurements from 2004 (best available data) resulted in a maximum of 2.5 fps or 1.5 knots (the point at where the formula changes in the guidance).

Taking into account parameters that govern ship navigation, channel geometry is classified into types of cross sections for evaluation of the channel-width criteria. Based on cross-sections, the channel reaches in the federal navigation channel would be classified as dredged channel trench as shown in Figure 8-1 of EM1110-2-1613 (Figure 2.1.1), although the trench is very shallow as compared to the full cross- section of the river (Figure 2.1.2).

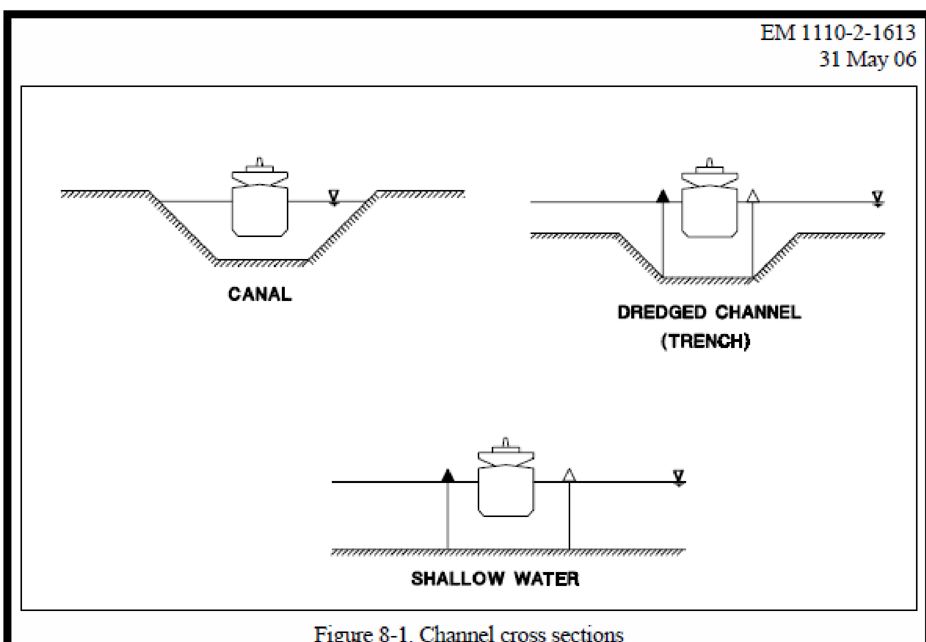

Figure 2.1.1 Channel cross-section

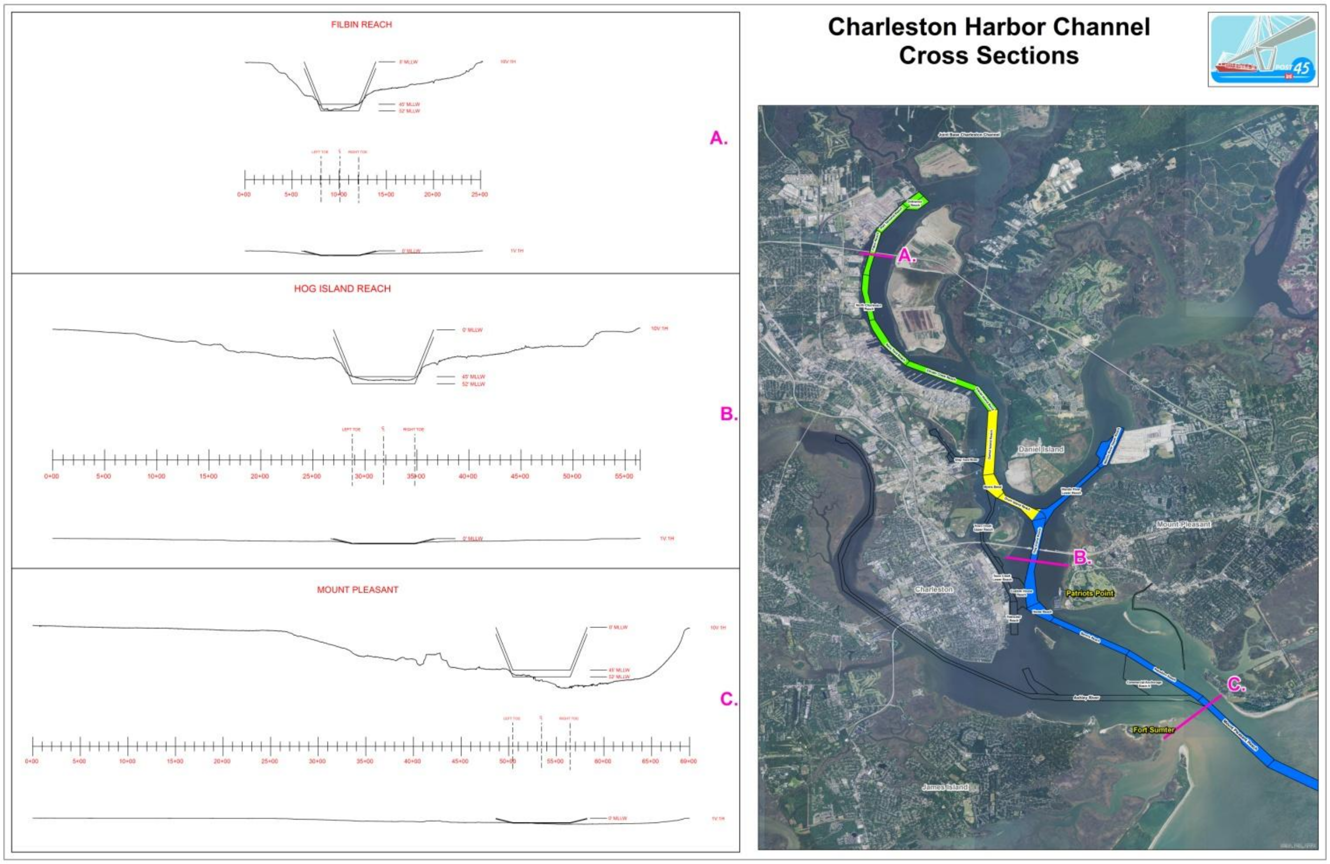

Figure 2.1.2 Cross-sections of Channel within River

The items requested by the pilots are listed below and shown on Figure 2.1.3 and include a evaluation based on the guidance. These will be further evaluated during ship simulation.

1. Widen the Drum Island bends for better turning because it currently has limited steerage on bulker ships. The pilots requested a 100 feet cutoff turn width increase from point of intersection. The angle of the bend in Drum island reach, according to Table 8.4 of the guidance, it could range from zero increase in width to 2 times the beam width increase in width. Using a design vessel of beam width of 160’ would result in up to a 320 feet increase in width. The pilots’ request is within the guidance range and will help straighten the reach, and minimize the cut in shallow areas adjacent to Drum Island.

2. Widen the end from Drum to Myers for better turning because it currently has limited steerage on bulker ships. Pilots requested a 200 feet cutoff turn width increase from point of intersection. The overall angle from the guidance in Table 8.4 would suggest a range of .7 to 1.0 times the beam width. Using design vessel of 160 feet beam, results in a range from 112 to 160 feet. However, since according to the centerline, there is technically another turn within that transition, a range of 1.0 to 2.0 times beam width increase in that area (160 to 320 feet) would be justified.

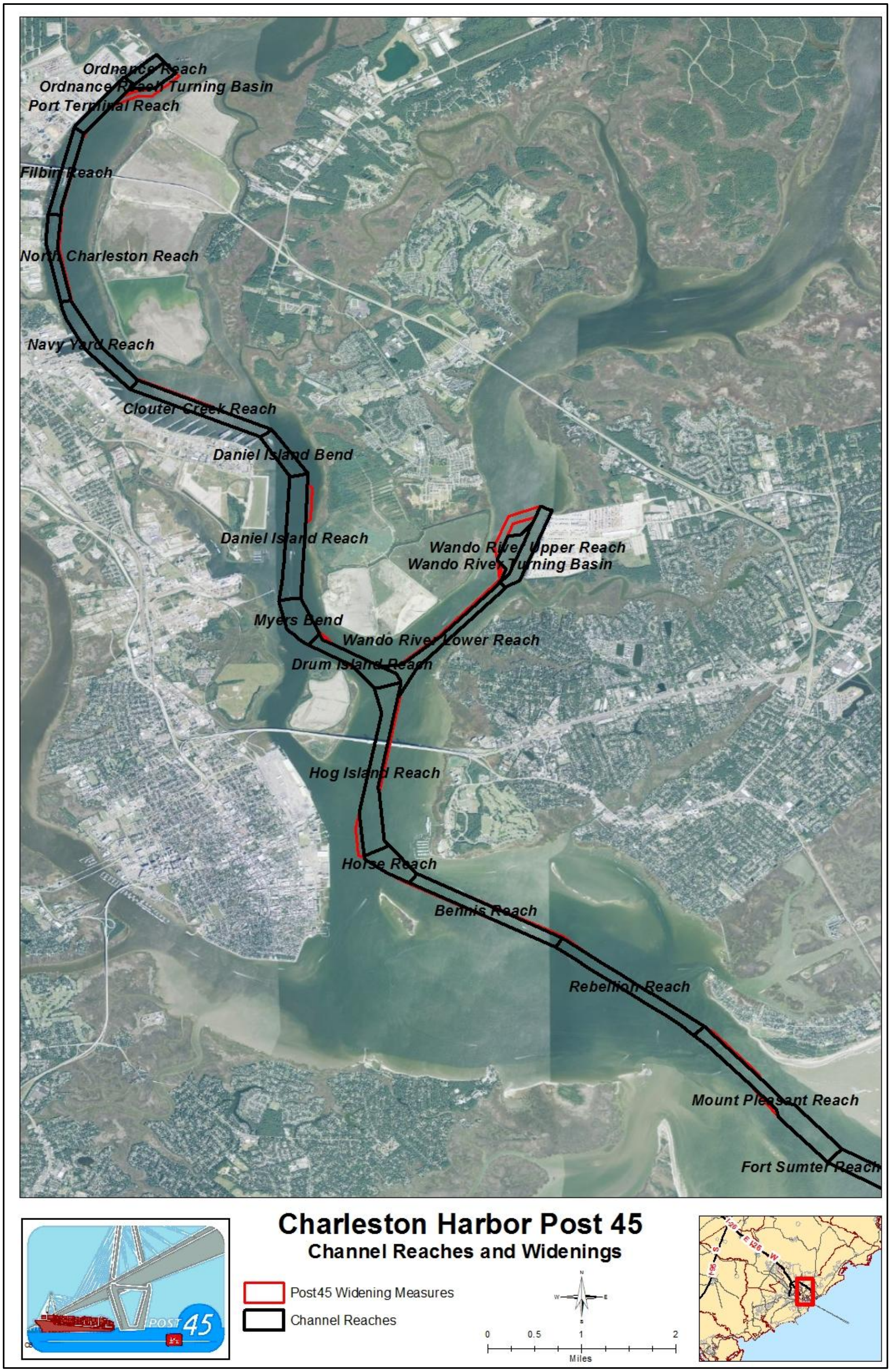

Figure 2.1.3 Channel Reaches and Widenings

3. Widen Wando Reach 100 feet since transit is difficult during extreme wind conditions. Ships are crabbing even with tugs. Wando reach is presently 400 wide at its narrowest. USGS velocity measurements resulted in a maximum of 2.5 fps or 1.5 knots.   Table 8-2 formula in the guidance criteria is 3.25 or 4.0 times beam width, which indicates width could be 520’ to 640’ for beam of 160 feet.

4. Enlarge the turning basins for better maneuverability. Guidance states widths should be 1.2 times (for currents of 0.5 knots or less – which is highly unlikely) to 1.5 times (for currents 0.5 knots to 1.5 knots) the ship length. Currents greater than 1.5 knots should be sized by ship simulation. Basin length along the prevailing current direction is dependent on currents. It was assumed length would be equal to width and maximum sizes were used to estimate new work dredging quantities, costs and for computer model simulations to determine shoaling rates, and environmental impacts. Turning basins will be evaluated during PED to confirm or revise turning basin dimensions.

4a. Ordnance turning basin for ships calling on NCT, using design vessel of 1100’, the resultant turning basin size would range from 1320’ and 1650’ widths. Basin length along the prevailing current direction is dependent on currents.   Maximum size of 1650’ was assumed for feasibility study.

4b. The new SCSPA terminal at the former Navy Base (NBT) will require a turning basin. The previous deepening study justified a turning basin in the reach when the terminal was located on the opposite bank on Daniel Island. Since the terminal on Daniel Island was never constructed, the turning basin was never constructed and now the new terminal is located where the authorized turning basin was to have been constructed. Thus, a turning basin for this reach will now have to be located within the federal channel, portions of the access channel to the new terminal and, if necessary, slightly out of channel to the east. The turning basin was sized using a design vessel length of 1200’, which results in 1440’ and 1800’ widths. Maximum size of 1800’ was assumed for feasibility study.

4c. Wando turning basin was sized using design vessel of 1200’, which results in 1440’ and 1800’ widths.   Maximum size of 1800’ was assumed for feasibility study.

5. Widen angles at turning basins for best maneuverability with increased currents.   Guidance indicates end angles are 45 degrees or less, depending on local shoaling tendencies, but also states that modifications are acceptable to permit better sediment flushing characteristics or accommodate local operational considerations. Without ship simulation, determination of the angles is based on discussions with the pilots.

5a. Wando turning basin angle is presently 30 degrees. The pilots also requested that the turning basin be centered on the terminal for better access to the north end of the terminal. This also moves the turning basin into deeper water.

5b. NCT is at a 37.5 degree angle, but the pilots indicate difficulty in maneuvering and requested a lesser angle transition. To minimize dredging quantities, instead of changing the entire turning basin transition angle, the turning basin dimension used the same angle from the channel; however, added a 300 foot cutoff turn width increase from point of intersection.

5c. Since the majority of the new turning basin at the new NBT terminal is located within the channel and terminal access area, the angle for the portion to the east outside the federal channel will remain the guidance recommended 45 degrees. The turning basin was aligned with the center of the terminal dock. The pilots' request of extending upstream end of the turning basin to interest Daniel Island Bend would potentially add additional 600 feet. Prior to installation of the contractions dikes this was an area of high shoaling rate. Increasing the channel width in that area will once again increase shoaling rates and therefore was not considered as part of this study.

6. Widen North Charleston and Filbin reaches by 50 feet to the east. Presently these reaches are 500 feet except for the upper portion of North Charleston reach above R 54A which is 600 feet. The guidance allows 4 times the beam width which results in 564 feet width using the design vessel beam of 141'. Thus, an additional 50 feet in the Filbin and lower North Charleston reaches is within guidance.

7. Add 50 feet to the east side of Clouter Creek Reach for passing lane. Using a design vessel of 150 feet beam and the guidance of 5.5 (for less than 1.5 knots) and 6.5 (for greater than 1.5 knots) would result in 825' to 975 feet width. To meet guidance would have a significant amount of new cut required and proximity of shallow flats. However, 50 feet to pass a smaller ship with a larger ship was included in the feasibility study. Ship simulation will determine if additional 50 feet is enough to pass ships safely.

8. Widen Hog Island by 100 feet to the east for two way traffic. Using design vessel of 160 feet beam and the guidance of 5.5 (for less than 1.5 knots) and 6.5 (for greater than 1.5 knots) would result in 880 to 1040 feet width, depending on currents. Presently this reach is 600 feet, therefore, 100 feet more is well within guidance allowance. Study assumes the 100 feet and ship simulation phase will assess if this is acceptable.

9. Widen Bennis Reach by 100 feet to the east for two way traffic. Using a design vessel of 160 feet beam and the guidance of 5.5 (for less than 1.5 knots) and 6.5 (for greater than 1.5 knots would result in 880' to 1040 feet width, depending on currents. Presently this reach is 600 feet, thus 100 feet more is well within guidance allowance, although it would not meet minimum of guidance recommendation. Study assumes the 100 feet and ship simulation will assess if this is acceptable.

An overall review of the federal channel as compared to the guidance is shown in Table 2.1.1. Some reaches vary in width so there is a minimum width and maximum width ratio computation. Some ratios do not meet the guidance criteria outlined in table 8-2, but pilots are able to maneuver, thus reducing federal costs.

Table 2.1.1. Channel Width Comparison (proposed)

Reach	New Stationing		Width (feet) min	Width (feet) max	Length (miles)	depth MLLW (feet)	design vessel beam (feet)	design vessel length (feet)	width/beam ratio min	width/beam ratio max	channel length/vessel length ratio
	Start Station	End Station									
Fort Sumter Reach	0+00	900+00	1000	1000	17.5	47	160	1200	6.3	6.3	77.0
Mount Pleasant Reach	900+00	995+18	600	1000	1.8	45	160	1200	3.8	6.3	7.9
Rebellion Reach	995+18	1077+91	600	600	1.6	45	160	1200	3.8	3.8	7.0
Bennis Reach	1077+91	1155+87	700	700	1.5	45	160	1200	4.4	4.4	6.6
Horse Reach	155+87	1178+12	600	600	0.6	45	160	1200	3.8	3.8	2.6
Hog Island Reach	1178+12	1272+82	700	700	1.6	45	160	1200	4.4	4.4	7.0
Drum Island Reach	1272+82	1316+91	600	600	0.8	45	160	1200	3.8	3.8	3.5
Meyers Bend Reach	1316+91	1342+47	880	880	0.5	45	160	1200	5.5	5.5	2.2
Daniel Is. Reach	1342+47	1412+40	880	880	1.4	45	160	1200	5.5	5.5	6.2
Daniel Island Bend	1412+40	1440+56	700	780	0.5	45	141	1100	5.0	5.7	2.4
Clouter Creek Reach	1440+56	1508+78	600	650	1.3	45	141	1100	4.3	4.6	6.2
Navy Yard Reach	1508+78	1566+35	600	700	1.1	45	141	1100	4.3	5.0	5.3
North Charleston Reach	1566+35	1615+65	550	550	1	45	141	1100	3.9	3.9	4.8
Filbin Creek Reach	1615+65	1664+42	550	550	0.9	45	141	1100	3.9	3.9	4.3
Port Terminal Reach	1664+42	1701+05	600	600	0.7	45	141	1100	4.3	4.3	3.4
Ordnance Reach	1701+05	1720+53	600	600	0.4	45	141	1100	4.3	4.3	1.9
Ordnance Reach TB	1698+50	1720+53	1050	1050	0.4	45	141	1100	na	na	1.9
Wando R. Lower Reach	0+00	71+49	500	500	1.4	45	160	1200	3.1	3.1	6.2
Wando River Upper Reach	71+49	119+78	600	600	0.9	45	160	1200	3.8	3.8	4.0
Wando River TB	71+00	109+00	1200	1200	0.4	45	160	1200	Na	na	1.8
Tidewater Reach	0+00	35+73	650	650	0.7	40	na	na	na	na	na
Custom House Reach	26+00	64+00	1385	1385	0.5	45	160	1200	8.7	8.7	2.2
Town Creek Lower Reach	35+73	105+82	450	450	1.1	45	160	1200	2.8	2.8	4.8
Town Creek Upper Reach	105+82	161+27	250	250	1	16	na	na	na	na	na
Town Creek T B	74+00	87+86	300	300	0.25	35	na	na	na	na	na
Shipyard River	0+00	64+77	1200-200	1200-200	1.1	45-30	na	na	na	na	na

Guidance indicates channel width can vary from 2.0 to 6.0 times the beam width for one way traffic. Trench conditions indicates for currents between 0.5 to 1.5 knots a ratio of 3.25 is recommended and for 1.5 to 3 knots, 4.0 is the recommended ratio. Wando River Lower reach existing has a 2.5 ratio, which is why it is one area proposed for widening and proposed widening will bring it to 3.1. Town Creek does not meet the 3.25 ratio criteria either; however, ships in this area are being docked, not in transit. All other reaches meet the 3.25 ratio. Filbin (3.3), Bennis (3.8), and Rebellion (3.8) reaches do not meet the 4.0 ratio. Filbin is proposed for widening which will increase the ratio to 3.9. Bennis is proposed for widening as well and will have a 4.4 ratio, but the widening is also for passing purposes.

Guidance recommends straight segments between turns should be at least five times the length of the design ship. The current navigation channel does not meet this criteria on the Drum Island, North Charleston, Filbin or Port Terminal reaches. Harbor pilots indicated the proposed changes were sufficient, however, only ship simulation will conclude if this is an issue. Ship simulation will assess whether these maximum widening measures are necessary or if they can be reduced to minimize the footprint and thereby save new work and maintenance dredging costs and environmental impacts. Conversely, it will also determine if greater measures need to be included for the safety and maneuverability of vessels.

2.1.3 Contraction Dike at Daniel Island Bend

To ameliorate shoaling, three contraction dikes were constructed in the Cooper River in 1959. Contraction dikes are special cases of training dikes designed to focus flows in certain areas. Focusing the flow keeps the sediment in suspension and reducing shoaling. Two contraction dikes were constructed on the east and west banks just south of the Shipyard River entrance. There is another contraction dike located along the west side of Daniel Island Bend just upstream of Daniel Island reach and the new terminal. The contraction dike on the east side was removed and relocated to the west side upstream of Shipyard River entrance and the other two contraction dikes were rehabilitated to elevation 9.0 MLW as part of the 1998 authorized project. These dikes have been previously studied by ERDC and proven to reduce shoaling in Daniel Island Reach by 50%. A potential risk was identified that additional deepening may alter the top of existing side slope and it may impact the Daniel Island Bend contraction dike. Preliminary assessment of the top of bank footprint showed that at the existing depth of 45, the channel template with a 4 on 1 side slope may impact the contraction dike.

Further investigation of the bathymetry data at channel cross- section adjacent to the contraction dike (Figure 2.1.4), indicates that although the template may show the contraction dike in the footprint (red line), the actual ground elevation (blue) is lower and flattens out before the contraction dike. The existing side slope does not actually impact the contraction dike and the existing depth in this area is already 50 to 52 feet. Therefore, the template would run out of earthen side slope before reaching the contraction dike. It is not expected the contraction dike would be impacted with the existing channel alignment at a greater depth (green line). Therefore, there is no reason to shift the channel in this reach.

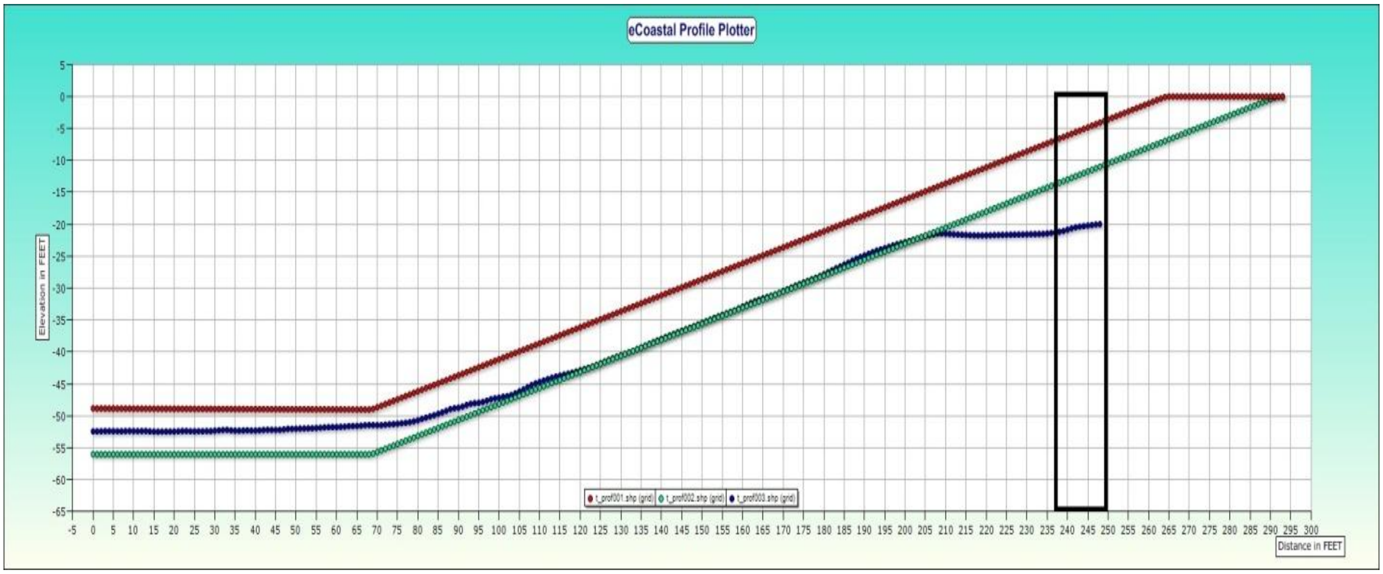

Figure 2.1.4 Cross- section at Contraction Dike

2.2 Sea Level Change

Table 2.2.1 shows the tide range from the NOAA Benchmark Sheet for 8665530, Charleston SC located near the Customs House,published 4/29/2003, based on tidal epoch 1983-2001, :

(source : http://tidesandcurrents.noaa.gov/benchmarks.html?id=8665530)

Table 2.2.1 Tide Range

Elevations of tidal datums referred to Mean Lower Low Water (MLLW),			
		meters	feet
HIGHEST OBSERVED WATER LEVEL (09/21/1989)		3.817	12.52
MEAN HIGHER HIGH WATER MHHW		1.757	5.76
MEAN HIGH WATER MHW		1.648	5.41
North American Vertical Datum NAVD88		0.957	3.14
MEAN SEA LEVEL MSL		0.891	2.92
MEAN TIDE LEVEL MTL		0.853	2.80
MEAN LOW WATER MLW		0.057	0.19
MEAN LOWER LOW WATER MLLW		0	0.00
LOWEST OBSERVED WATER LEVEL (03/13/1993)		-1.245	-4.08
Based on North American Vertical Datum (NAVD88)			

Recent climate research by the Intergovernmental Panel on Climate Change (IPCC) predicts continued or accelerated global warming for the 21st Century and possibly beyond, which will cause a continued or accelerated rise in global mean sea-level. USACE Engineering Regulation ER 1110-2-8162 “Incorporating Sea Level Change into Civil Works Programs” was developed with the assistance of coastal scientists from the NOAA National Ocean Service and the US Geological Survey. Please refer to the ER for explanation of how and why sea level is to be applied to USACE projects. Planning studies and engineering designs are to evaluate the entire range of possible future rates of sea-level change (SLC), represented by three scenarios of “low”, “intermediate’ and “high” sea-level change. (Figure 2.2.1). The use of sea level change scenarios as opposed to individual scenario probabilities underscores the uncertainty in how local relative sea levels will actually play out into the future. At any location, changes in local relative sea level (LRSL) reflect the integrated effects of global mean sea level (GMSL) change plus local or regional changes of geologic, oceanographic, or atmospheric origin.

- “**Low**” rate of sea-level change is equal to the historic rate of SLC.

- “**Intermediate**” rate of sea-level (ISL) change is based on the modified NRC curve I and Using the current estimate of 1.7 mm/year for GMSL change, the following equations

$$E(t) = 0.0017t + bt^2$$

in which t represents years, starting in 1986, b is a constant, and E(t) is the eustatic sea level change, in meters, as a function of t.

Manipulating the above equation to account for the fact that it was developed for eustatic sea level change starting in 1992, while projects will actually be constructed at some date after 1992, results in equation

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)$$

where t_1 is the time between the project’s construction date and 1992 and t_2 is the time between a future date at which one wants an estimate for sea level change and 1992 (or $t_2 = t_1$ + number of years after construction)

- “**High**” rate of sea-level change (HSL) is based on the modified NRC curve III and the above equations.

Using the USACE Institute of Water Resources (IWR) Sea Level Change calculator spreadsheet the trend at Charleston is estimated to be 2.94 mm/yr based on CO-OPS gage 8665530. 2012 tidal data is used for the analysis of existing conditions. Estimating construction completion of 2021, and a 50 year project life, starting with 2012 (thus estimate the increase in 59 years) – the rates of change relative to Charleston Harbor are as follows: “low” rate of change is 0.57 feet, the “intermediate” is 1.08 feet and the “high” is 2.74 feet (Table 2.2.2).

Table 2.2.2 Sea Level Change Rates

Sea Level Change (feet)			
Year	Low	Int	High
2012	0.00	0.00	0.00
2015	0.03	0.04	0.08
2020	0.08	0.11	0.22
2025	0.13	0.19	0.38
2030	0.17	0.27	0.56
2035	0.22	0.35	0.76
2040	0.27	0.44	0.98
2045	0.32	0.53	1.21
2050	0.37	0.63	1.47
2055	0.41	0.73	1.74
2060	0.46	0.84	2.03
2065	0.51	0.95	2.34
2070	0.56	1.06	2.67
2075	0.61	1.18	3.01

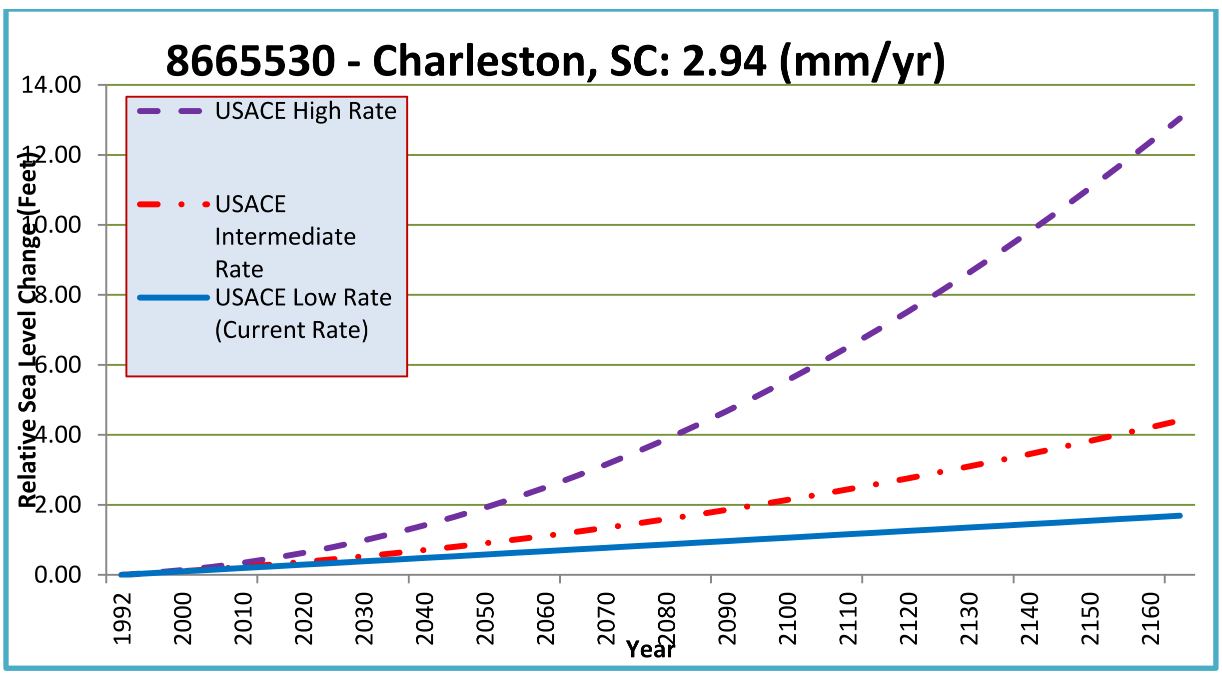

Figure 2.2.1 Relative Sea Level Changes for Charleston Harbor

The purpose and function of the navigation channel is not highly sensitive to the rate of sea level change. According to ER 1110-2-8162, paragraph 6.(d).(1), when plan performance is not highly sensitive to the rate of sea level change the study can work within a single scenario, typically the "most-likely" or historic (low) rate and identify the preferred alternative under that scenario. The preferred alternative's performance would then be evaluated under the other scenarios to determine its overall potential performance. This method is employed in this study.

2.3 Beneficial Use

Engineering analysis of any potential uses of dredge material was postponed to Preconstruction, Engineering and Design (PED) phase under the new SMART planning requirement to limit feasibility study scope and duration. Some discussion of potential options recommended during public scoping and discussions with various environmental agencies include:

- Crab bank
  - Brown Pelican nesting, black skimmers, royal terns
  - Also for shorebirds and wading birds.
- Sandbar complex b/w east end of southern jetty and Cummings point
- Morris Island Lighthouse
- Shutes Folly/ Castle Pinckney
- Feeder berms for barrier islands
- Offshore fish habitat berms
- Augmenting ODMDS berms (consideration also for mitigation)
- Fort Sumter (consideration also for mitigation)
- Creation of a new island south of the south jetty
- Nearshore placement off Morris Island

These options are discussed more thoroughly in the main report and environmental appendices. Mitigation options to offset project impacts using dredge material of rock from the entrance channel to create hardbottom mounds adjacent and parallel to the federal channel are discussed in the Environmental Appendices. This also is a lesser cost than taking material to the ODMDS.

2.4 Ship Simulation

Under new requirements to limit scope and schedule duration, it was determined that ship simulation was only required on the alternative selected. Ship simulations area a scaling tool and are not necessary to make a planning decision and will therefore be accomplished during the preconstruction, engineering, and design (PED) phase. Maximum widening measures based on discussions with the pilots were assumed for the determination of the recommended plan. These will be refined in the ship simulation to determine if lesser widening can be used. Thus it was determined that there was little risk in moving ship simulation to PED phase.

However, the post processor (discussed in section 3.4.6) to process the output files of numerical model into usable tables included a module to generate an ascii file for use with POLARIS (ship's bridge simulator). Options include selection of the type of tide (spring or neap), latitude and longitude of the user selected tile, and selection of vertical layers (full water column, top half of water column…). Example output files were provided to ERDC for confirmation they would be acceptable for the ship simulation that would occur in PED phase. Concurrence was provided via email from Keith Martin, Research Physicist, Coastal and Hydraulics Laboratory.

2.5 Shoreline Change Contributors

Tides and sea level change, river currents, ship wakes, tropical and subtropical storms, shoreline changes (riprap protection) and wind generated waves, all contribute a part in the erosion of shorelines of the Charleston Harbor.

Studies done by Waterways Experiment Station (now ERDC) of shoreline erosion near Hobcaw Point on the Wando River in Charleston Harbor (Teeter et al. 1997) considered there to be five possible causes of shoreline retreat:

- shoreline construction (diking of Daniel Island- applicable only to Wando River) ,
- channel modifications
- Cooper River Rediversion (that affect sediments, flows, waves and geometry),
- vessel wakes
- wind generated waves.

While none of the five factors assessed could be ruled out, the study concluded "waves produced by container vessels do not appear to be as important as wind waves or even waves produced by smaller displacement vessels in generating shear stress forces on the sediment bed. Vessel waves are solitary and infrequent in comparison to wind waves." It was also noted that sea level rising in the harbor also contributes to land loss along the shoreline.

As the channel is a small portion of the cross- sectional area of the harbor (see Figure 2.1.2), past deepening projects are not considered primary drivers of shoreline change.

As a component of the current study, USACE analyzed wind generated waves (section 2.5.1) and vessel wake based on a presently unpublished study entitled "Use of AIS and AISAP for Analysis of Vessel-Wakes in Charleston Harbor: A Case Study". The case study addressed future vessel generated waves under existing and deepened channels at four separate areas of concern (section 2.5.2). Then a comparison of the waves generated by waves and those by vessel was done to confirm that the wind generated waves are more impactful than the vessel wakes.

2.5.1 Winds in Charleston Harbor

Winds can be described by their speed, direction, and duration.

The National Oceanic and Atmospheric Administration (NOAA) operates a weather station in Charleston Harbor which collect 6-minute wind data. This station records wind speed and direction at the shore. A wind rose was generated using the hourly averaged data recorded between January 2010 and December 2011 to visualize the distribution of winds which pass over Charleston Harbor (See Figure 2.5.1).

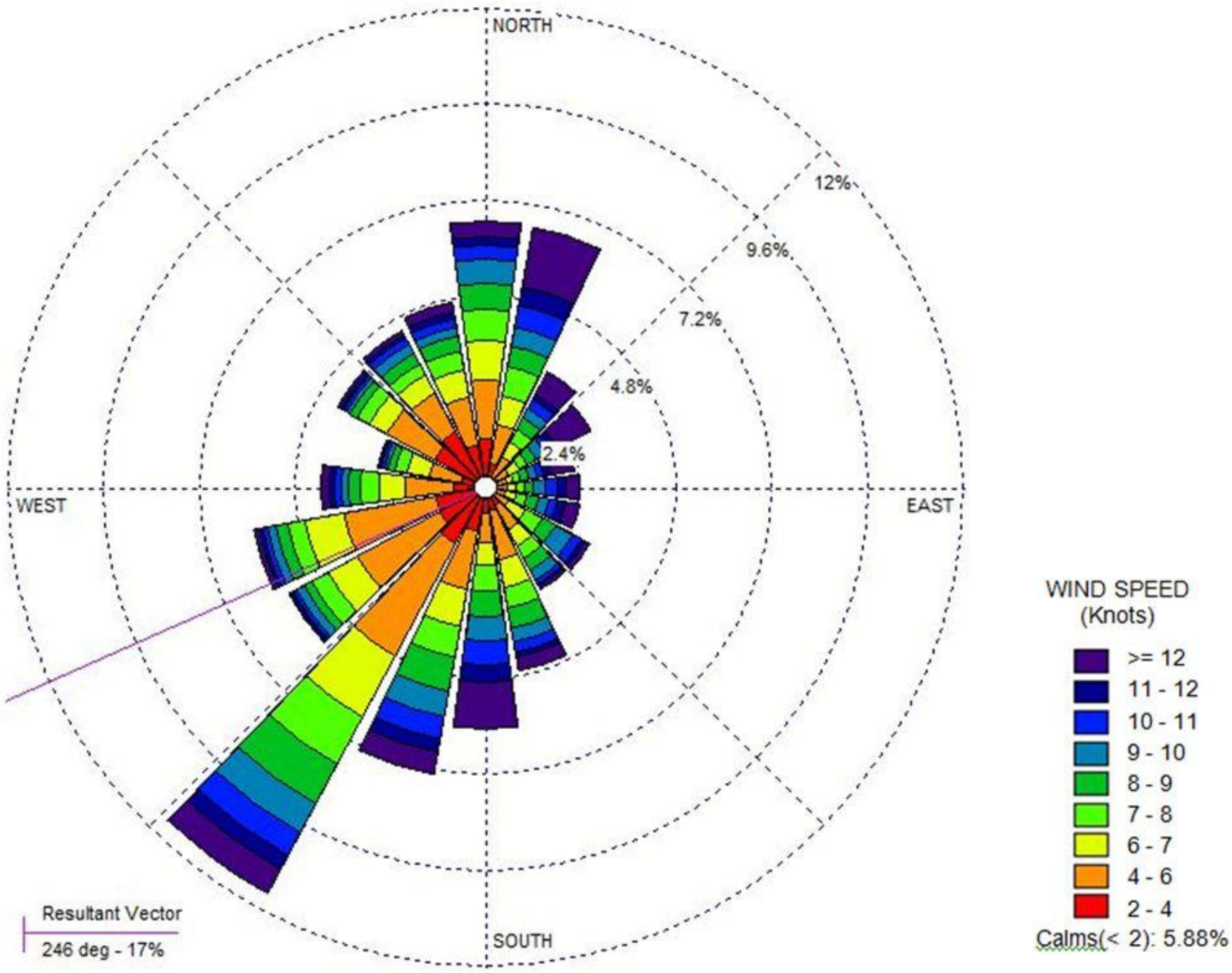

Figure 2.5.1. Wind Rose for Charleston Harbor Depicting Wind Direction and Speed Frequency

The distribution of wind speeds varies by direction (Refer to Figure 2.5.1. This figure is known as a wind rose). The total winds over Charleston Harbor, regardless of angle of approach, have the distribution by wind speed class shown in Figure 2.5.2. Three petals of the wind rose from Figure 2.5.1 are shown as frequency distributions in Figure 2.5.3. The petals selected reflect the three key directions: the largest number of winds, the highest speed winds and those with longest fetch (distance to travel). The largest number of winds in Charleston Harbor come from the southwest, while the most high-speed winds (fastest 10% of winds) come from the north-northeast direction (Wando River). Winds entering the harbor from open ocean (south-east) have the potential to travel the furthest distance before reaching a shoreline.

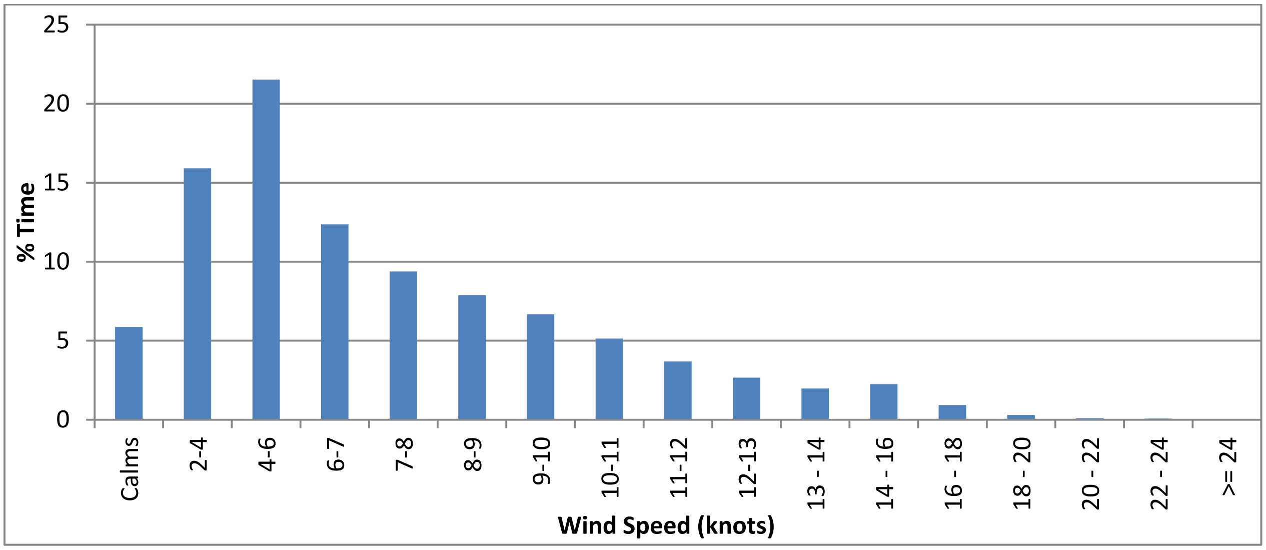

Figure 2.5.2. Wind Speed Frequency Distribution in Charleston Harbor from all directions

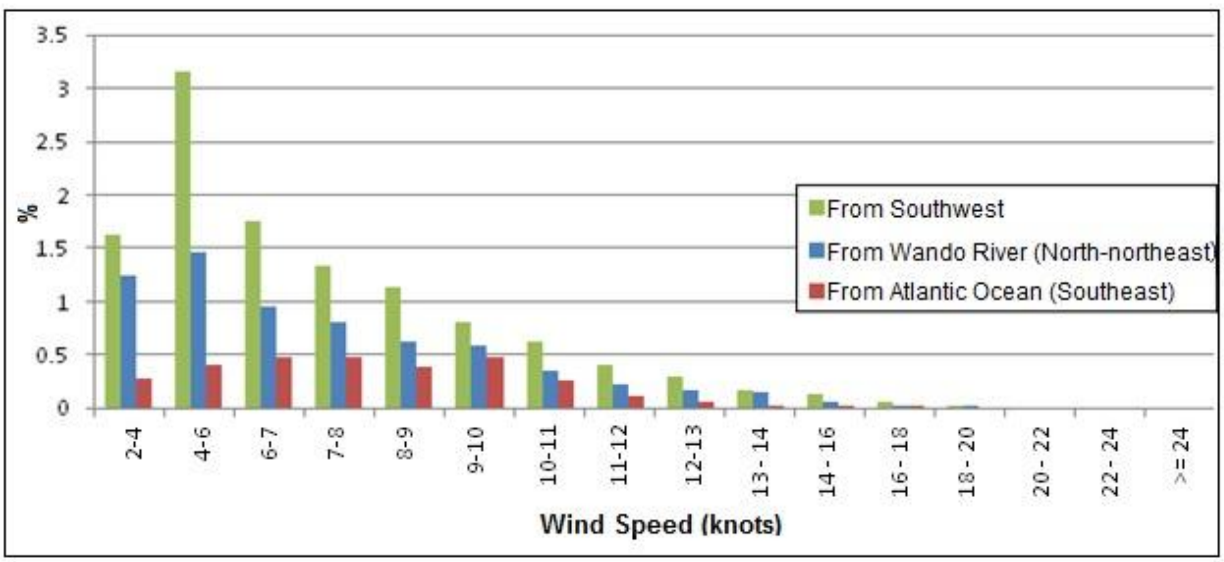

Figure 2.5.3. Wind Speed Frequency Distribution in Charleston Harbor comparing three key directions

As winds move over water, the friction between the air and water generate waves. The length of open water a wind blows over will affect the size of wave produced (USACE, 2008). The path a wind follows over water is called its “fetch”. Fetch distances vary within Charleston Harbor depending on the angle the wind is blowing from and the location of interest. Some representative fetches are shown in Figure 2.5.4 for four areas of concern near the harbor entrance, where erosion has historically been a concern.

A wind could create a variety of wave heights according to the fetch distance available for it to travel over. The increase of wave height with fetch distance varies with a curved shape as shown in Figure 2.5.5 for an 8 kt and an 24 kt wind, each with a 51 min duration. At large fetch lengths in deep water, the wave height becomes dependent only on duration of the wind.

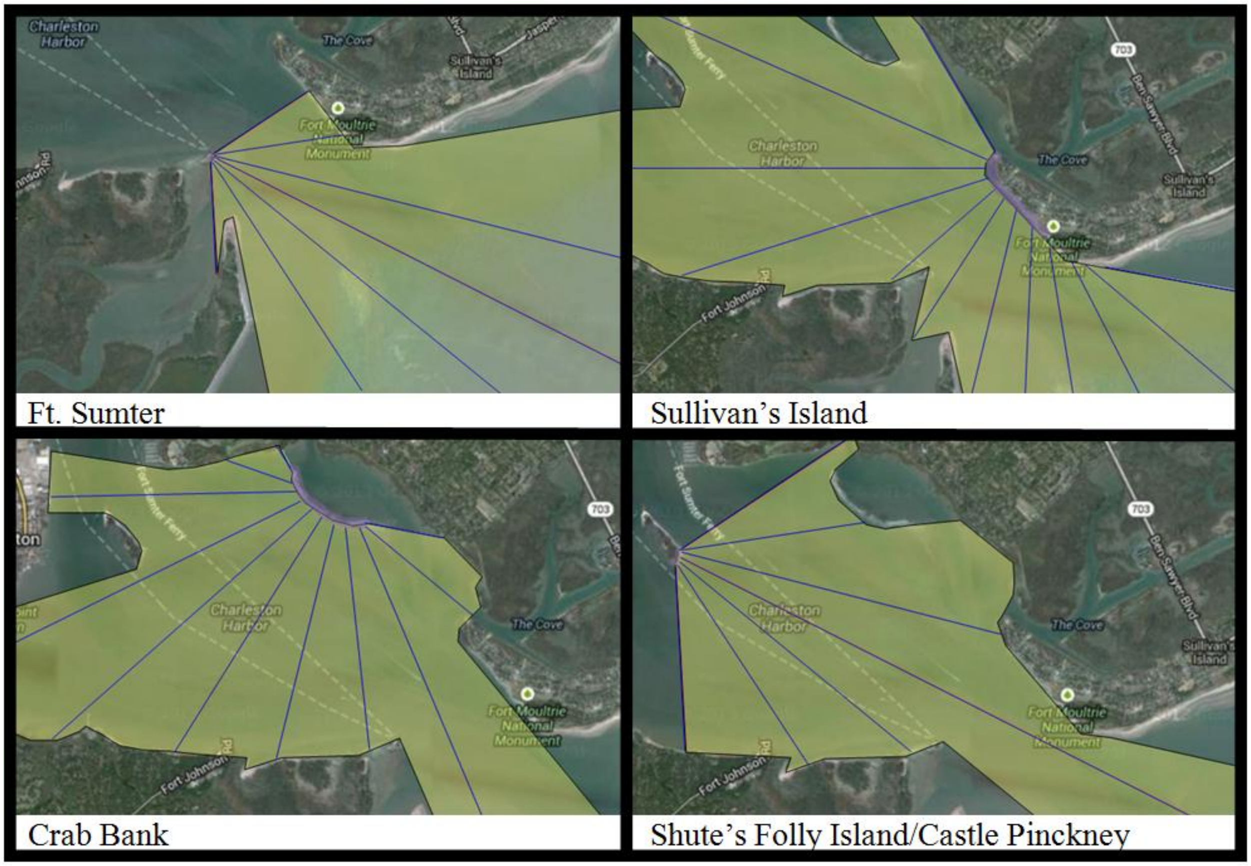

Figure 2.5.4. Representative Fetch Lines for Four Areas of Concern in Charleston Harbor

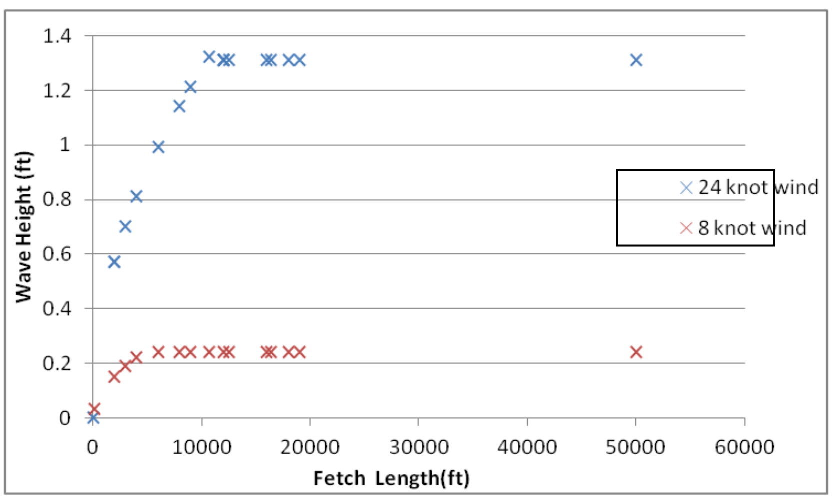

Figure 2.5.5. Wave Height Variation with Fetch Length for a 51 min Duration Wind at 8 kt and 24 kt

Wind waves do not decay until the wind stops, the wind changes direction, or the wave travels outside the locality over which the wind exists. This means that winds can also create a variety of wave heights for a single location according to the duration of the wind. The duration of winds in Charleston Harbor vary greatly. Winds can be separated into constant wind events lasting as long as the following conditions are met (USACE, 2008):

$$|Ui - \overline{U}| < 4.86\ kn$$

$$|Di - \overline{D}| < 15°$$

Where U_i is wind speed at some instant, i, D_i is wind direction at the same instant, and $\overline{U}$ and $\overline{D}$ are the average of preceding consecutive hourly wind speeds and direction, respectively.

It was determined from the NOAA 6-minute wind data that constant wind events total more than two-thirds of the year and of those events, 96% are sustained for less than 5 hr before significantly changing speed or direction. The longest wind duration observed exceeded 22 hr at approximately 12 kt. The average duration of constant winds was 51 min.

Wind wave heights were calculated using the Coastal Engineering Design and Analysis System (CEDAS) ACES tool from Veritech Inc. with inputs from the NOAA gage wind data. ACES calculates wind wave heights by determining the time required for a wave height, H_{m0}, to be limited by fetch, F. If the duration, t, of the wind with speed, U, is less than the time required to be limited by fetch, then the wind wave is considered “fetch limited” and the following equation is used:

$$H_{m0}=0.0000851\ (U^2/g)(gt/U)^{5/7}$$

If the duration of the wind is greater or equal then the time required to be limited by fetch, then the wind wave is considered “duration limited” and the following equation is used:

$$H_{m0}=0.0016\ (U^2/g)(gF/U^2)^{1/2}$$

Finally, ACES calculates the fully developed wave height, H_{fd}, which represents the maximum wave height physically possible for a given wind speed with infinite fetch and duration to ensure that it is not exceeded using the following equation:

$$H_{fd}=0.2433(U^2/g)$$

If H_{fd} is exceeded, then the maximum wave height output from ACES, H_{m0}, will be limited to equal that instead of the duration or fetch limited H_{m0}.

Maximum wind wave height calculated for Charleston Harbor from the NOAA wind data was produced by a constant wind which reached 24 kt and pushed for over 14 hr at 180 degrees (due north) to Sullivan’s Island. Since this maximum wind condition came from offshore, it could have travelled over a long fetch without disturbance and result in a maximum wave height of 11.88 ft (170 mile fetch), but that is unlikely due to the variability allowed from the constant wind criteria and the narrow entrance to Charleston Harbor. If a fetch of only 6 miles is instead selected, for example, then the wave height would be 2.2 ft. It is not known what the appropriate fetch should be in this scenario due to a lack of data but given that such wind speed and duration conditions were measured, demonstrates the importance wind waves can have in harbor wave spectrums. The average speed of winds from NOAA data was 6.8 kt and the resultant direction was from 246 degrees. This average wind is capable of producing a wave 0.18 ft high at Crab Bank, the area of concern which is affected by the longest fetch (~3.5 miles) from 246 degrees.

Waves are created along the fetches shown in Figure 2.5.4 headed to Fort Sumter and Shute's Folly Island for more than 22% of the time. The distribution of wave heights estimated for the portion of wind spectrum reaching the areas of concern is shown in Figure 2.5.6. All wave heights were estimated for 51 min duration winds. The range of wind directions considered are the same for Fort Sumter and Shute's Folly Island but fetch lengths are different for the two sites which leads to slight differences in wave height distribution and average wave height estimation.

Similarly, Crab Bank and Sullivan's Island have the same wind directions of interest and are thus both affected by 64% of the wind spectrum. Their respective estimated wind wave height distributions are also shown in Figure 2.5.6.

Despite wind wave dependence on fetch and duration, the wave height distribution at Shute's Folly Island shown in Figure 2.5.7 takes nearly the identical shape to the wind speed distribution shown in Figure 2.5.3 for the three selected wind directions. This demonstrates the importance of wind speed in estimation of wind-generated waves.

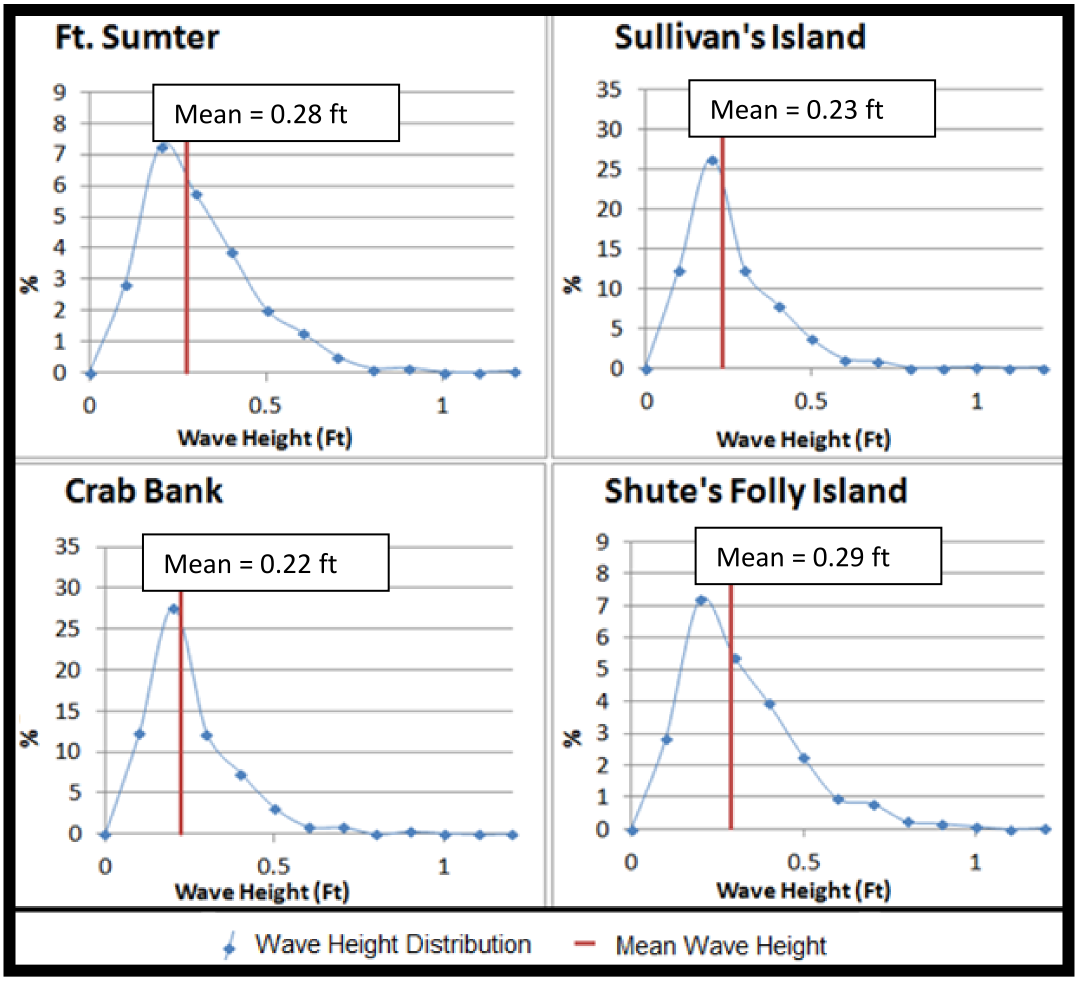

Figure 2.5.6. Distribution Significant Wave Height H_{m0} based on fetch lengths at Area of Concern (as shown in purple and yellow in Figure 2.5.4) for Wind-generated Waves due to 51 min Duration Winds

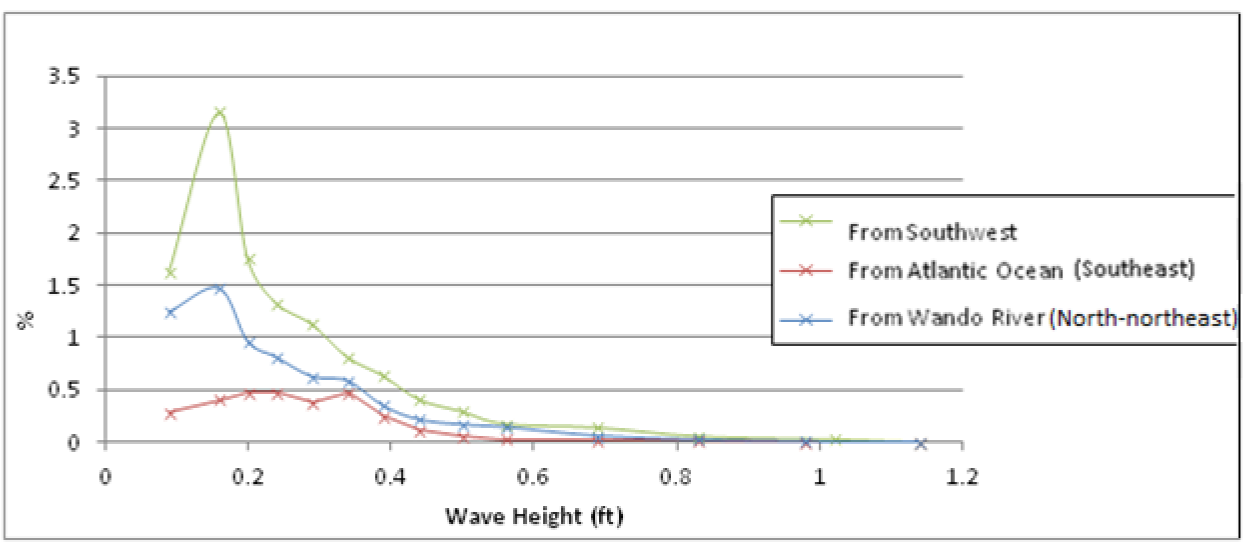

Figure 2.5.7. Shute's Folly Island Wave Height Distribution Caused from Three Key Wind Directions

The average of the significant wave heights (Figure 2.5.6) for each location calculated from the wave height distributions are higher than the significant wave height calculated for an average wind condition, 6.8 kt and from 246 degrees at Crab Bank which had the longest fetch and therefore the highest significant wave height. This demonstrates the impact of large wave outliers on potential impacts

Wind gusts are also measured by the NOAA gage as “the maximum 5 second moving scalar average of wind speed that occurred during the previous hour” (NOAA, 2008). Gusts were not analyzed in detail but are expected to have little effect on shoreline change due to their short duration. A quick calculation in ACES inputting a fetch of 5 mi, wind speed of 40 kt (as was the highest recorded between 2010-2011), and a duration of 5 sec suggests a significant wave height of 0.03 ft. Increasing the duration to 6 min increases the estimated wave height to 0.65 ft although such wind conditions are uncommon.

2.5.2 Vessels in Charleston Harbor

Charleston Harbor is transited by many vessels each year. Commercial vessels and large privately owned vessels carry transmitters which output geospatial and qualitative vessel information called Automatic Identification System (AIS) data. The AIS data is collected by the US Coast Guard and available by request.

As vessels travel through water they produce transverse and divergent waves from the bow and stern, as seen in Figure 2.5.8. These waves are often visible from the shore and raise public concerns of sediment movement and habitat disruption at nearby shorelines.

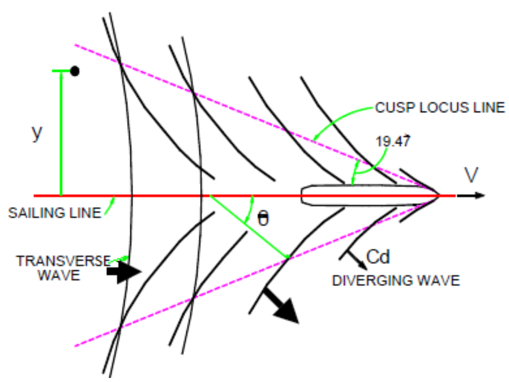

Figure 2.5.8. Vessel Generated Wave System, top view (Kriebel et al., 2003)

As waves produced by vessels travel outward from the sides of vessels they will contact shorelines if there is not a long enough distance for the waves to dissipate beforehand. The size of waves created by vessel movement are affected by the size of the vessel, shape of vessel hull, direction of tidal current, speed of travel, and shape of the channel.

Larger vessels are expected to call on Charleston Harbor as a result of the Panama Canal expansion. Charleston Harbor is one of the deepest harbors in the nation and could potentially receive calls from the new larger vessels, called “Post Panamax” (see Economics Appendix). It is of concern what the effects of these larger vessels would have on the environment. Multiple deepening alternatives are under consideration and the waves generated by vessels in each were predicted for comparison. It was hypothesized that comparing vessel waves from the current vessel fleet in the existing harbor to predicted future vessel fleets in an unchanged and a deepened harbor would identify any potential vessel-generated wave impacts that would result from the deepening project which would have the potential to increase erosion at sensitive areas.

AIS data from Charleston Harbor vessel transits between 2010 and 2011 was analyzed (McCartney and Scully, 2014) to determine the existing vessel conditions. AIS data includes vessel dimensions, speed, and transit heading (Figure 2.5.10), amongst other information. The AIS data was used to estimate waves generated by vessels and to estimate future vessel waves for comparison. The harbor was separated into four reaches based on vessel patterns revealed through the AIS data (Figure 2.5.9).

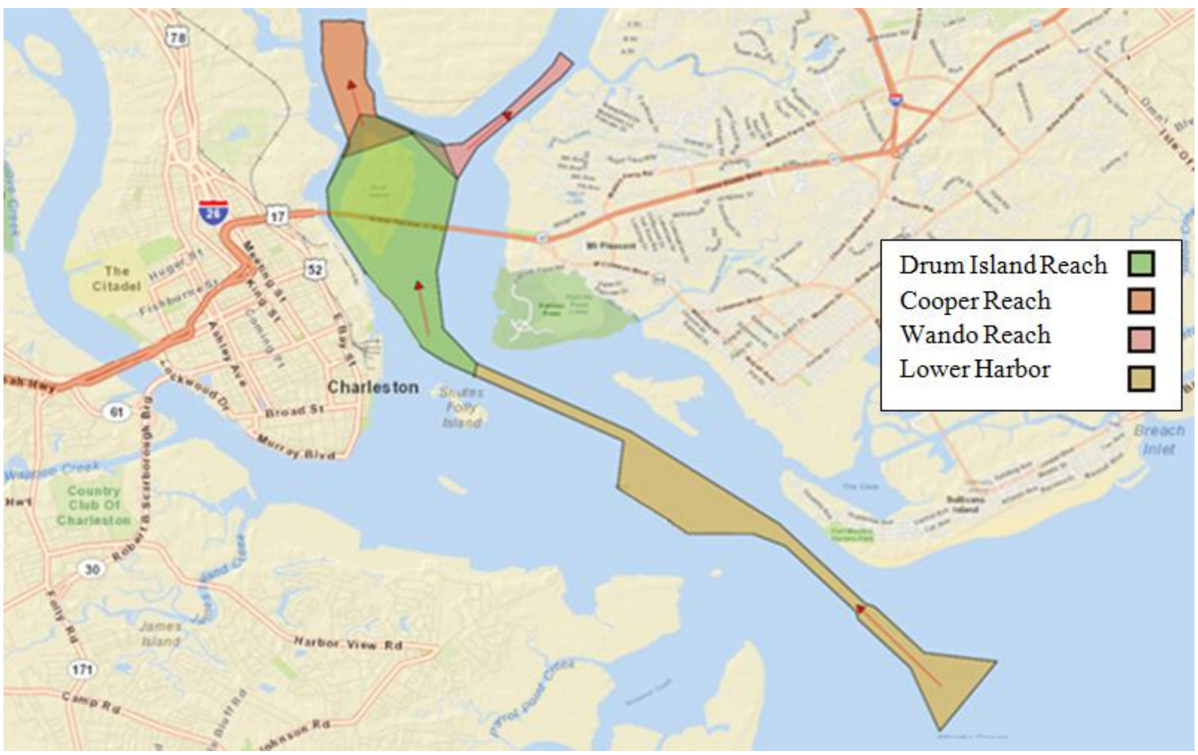

Figure 2.5.9. Areas of Interest (four reaches) Along the Federal Channel, (McCartney and Scully, 2014)

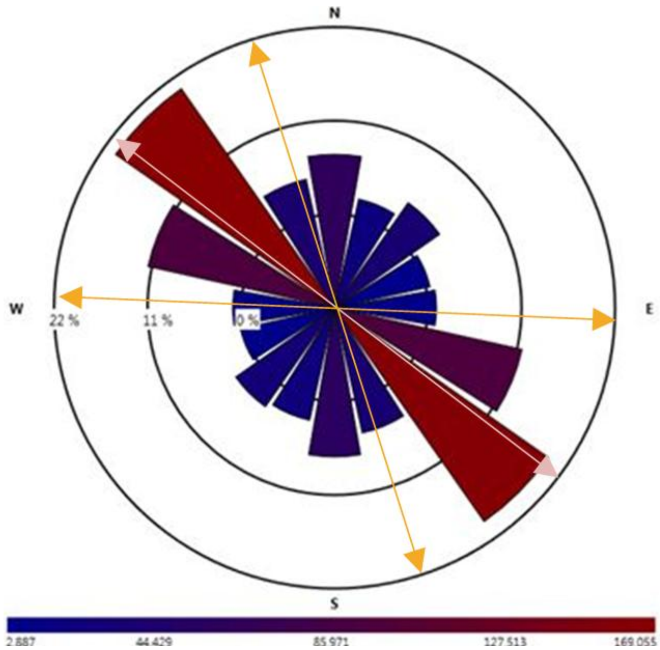

Figure 2.5.10. Frequency of Vessel Headings.

The count of AIS reports in each direction are color-coded. Average direction of vessels in Charleston Harbor is represented by the light red arrows while orange arrows show correlating direction of wave propagation (~35.26° from sailing line).

Some unitless parameters used to describe vessel shape are entrance length, L_e, block coefficient, C_B, and hull coefficients, α and β. These parameters can be found in tables of standard values by vessel type, or calculated using (Kriebel and Seelig, 2005, USACE, 2008, Maynord, 2007):

$$C_B = \frac{\nabla}{L*B*d} \quad \text{Eq (1)}$$

$$B/L_e = 1.11C_B - 0.33 \quad \text{Eq (2)}$$

$$\beta = 1 + 8 * \tanh^3[0.45\,(L/Le - 2)] \quad \text{Eq (3)}$$

$$\alpha = 2.5(1 - C_B) \quad \text{Eq (4)}$$

Refer to Figure 2.5.11 for depiction and definition of vessel dimensions. ∇ is the displacement of the vessel. C_B ranges from 0.6 to 0.8 for cargo ships (PIANC, 2002).

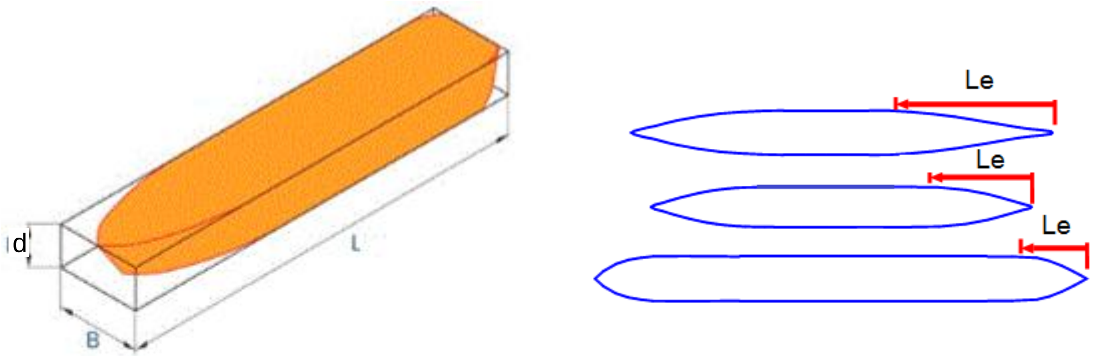

Figure 2.5.11. Vessel Dimensions (length, L, beam, B, draft, d) from PIANC, 2002, and Entrance Length, Le for Varying Vessel Shapes, modified from Kriebel et al., 2003

In order to compare estimated existing vessel generated waves with future vessel generated waves, McCartney and Scully, 2014, separated vessels transiting Charleston Harbor into beam classes with name conventions and average dimensions shown in Table 2.5.2.1.

Table 2.5.2.1 Average Vessel Class Dimensions (2010 - 2011) (McCartney and Scully, 2014)

Vessel Class	Draft (ft)	Length (ft)	Beam (ft)
Sub Panamax	30.1	611.4	89.8
Panamax	34.6	816.4	104
PP Generation I*	38.9	906.7	126.2
PP Generation II*	40.5	1030.9	141.3
PP Generation III*	43	1200	158.3

*PP = Post Panamax

Channel dimensions are important in the assessment of vessel waves because they govern which vessels are able to physically transit through waters. Blockage factor is the ratio of wetted cross-sectional area of a vessel to the wetted cross-sectional area of a channel. The blockage factor changes spatially and temporally according to tides and the vessel operation. Pilots are able to alter their draft during transits by varying the level of ballast water and by changing speed within allowable ranges. They can also unload some cargo offshore or load lighter initially as they expect will be necessary to have an acceptable draft for their transits. Commercial vessels will load as heavy and draft as deep as they safely can in order to maximize economic efficiency.

Table 2.5.2.2. Predicted Vessel Class Dimensions (McCartney and Scully, 2014)

Alternative	Vessel Class	Draft (ft)	Length (ft)	Beam (ft)
3 ft Deepening	Sub Panamax	30.1	611.4	89.8
	Panamax	34.6	816.4	104
	PP Generation I	41.9	906.7	126.2
	PP Generation II	43.5	1030.9	141.3
	PP Generation III	46	1200	158.3
5 ft Deepening	Sub Panamax	30.1	611.4	89.8
	Panamax	34.6	816.4	104
	PP Generation I	43.9	906.7	126.2
	PP Generation II	45.5	1030.9	141.3
	PP Generation III	48	1200	158.3
7 ft Deepening	Sub Panamax	30.1	611.4	89.8
	Panamax	34.6	816.4	104
	PP Generation I	45.9	906.7	126.2
	PP Generation II	47.5	1030.9	141.3
	PP Generation III	50	1200	158.3

If blockage factor is increased, such as happens with shoal formation, vessels will interact more with channel boundaries having larger environmental impacts and decreasing safety. A blockage factor of more than 0.02 or less is considered a fully unconfined channel while 0.5 is a highly confined channel (Maynord, 2007, USACE, 2008).

Charleston Harbor is a wide harbor with a deep channel maintained for commercial navigation. Some cross-sections are shown in Figure 2.5.12. The blockage factor is small, ranging from 0.14 to less than 0.02 depending on the vessel class, and considered “unconfined” for the purpose of this analysis.

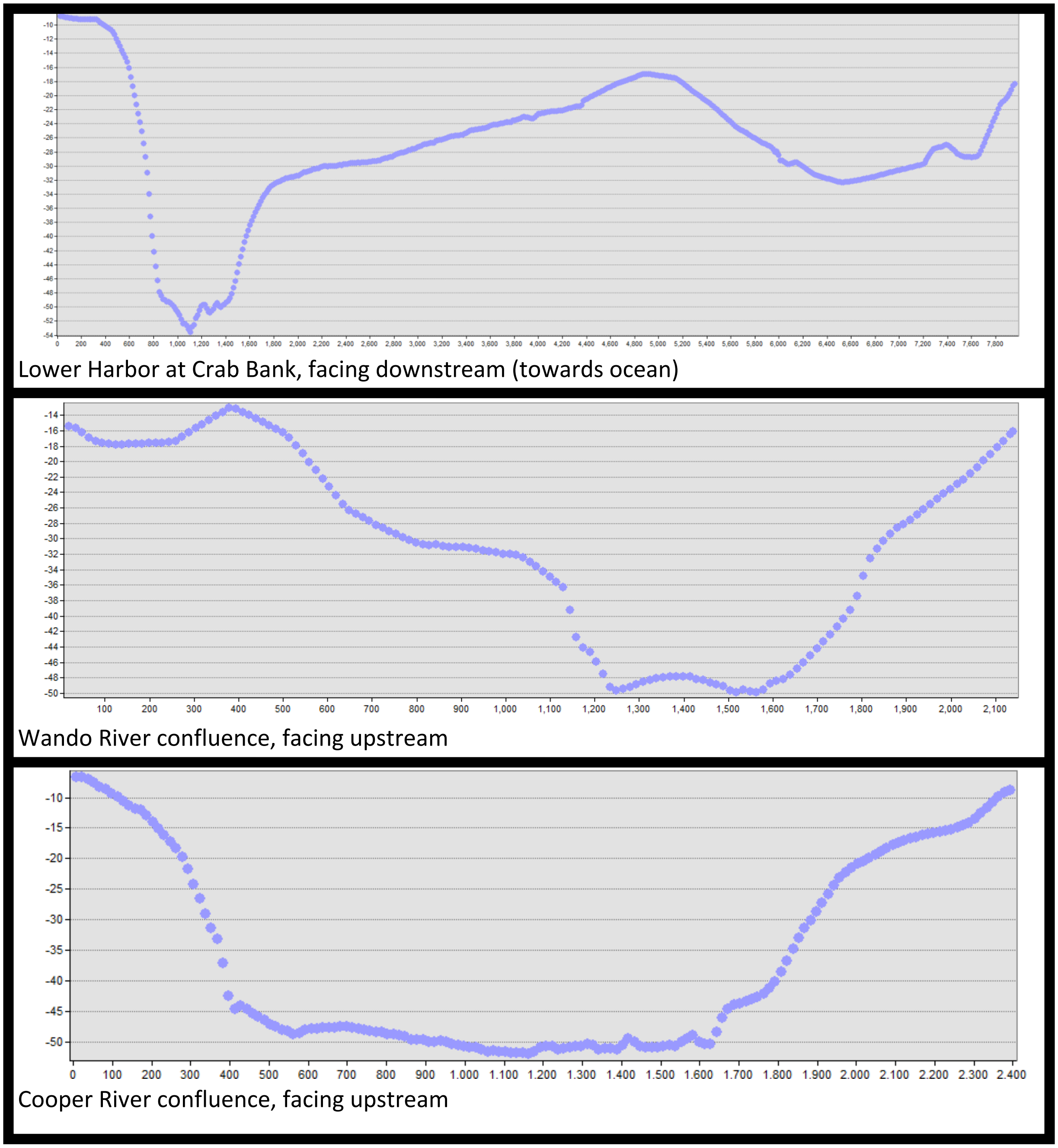

Figure 2.5.12. Cross-sections of Charleston Harbor from GPS Data

McCartney and Scully, 2014, predicted that deepening the harbor would allow larger vessels to transit the harbor and they estimated future vessel class dimensions as shown in Table 2.5.2.2 based on AIS data and economic vessel fleet projections. Harbor deepening and changing vessel fleets could alter the blockage factor. The cross-sections in Figure 2.5.12 and vessel fleet projections were used to estimate the change in blockage factor. Deepening the harbor from 45 ft MLLW, affects the blockage factor differently for each vessel class since the larger vessel classes are expected to draft deeper in a deepened harbor, thus altering their cross-sectional area. Using the draft predictions of McCartney and Scully, B_f would increase with 7 ft deepening for Sub Panamax and Panamax classes by approximately 16% while decreasing for PP Gen 1, PP Gen 2, and PP Gen 3 by approximately 2%, 1%, and 0.5% respectively in the lower area of the harbor near Crab Bank. In other areas of the harbor, similar variations are expected due to consistent federal channel width and depth along the main vessel paths.

Table 2.5.2.3. Vessel Average Speeds over Ground (kt) Based on AIS data (Data obtained from the USCG and analysis performed by district)

Vessel Class	Lower Harbor	Drum Island	Wando	Cooper	Average of Full Area of Interest
Sub Panamax	12.0	8.9	8.4	8.3	10.1
Panamax	12.5	8.4	8.1	8.3	10.3
PP Generation I	12.3	8.5	7.3	8.8	10.1
PP Generation II	11.9	8.4	7.1	6.7	9.7
PP Generation III	11.9	8.4	7.1	6.7	9.7

The speed of vessel travel to be used in wave calculations consists of the speed over ground (see Table 2.5.2.3) adjusted by any tidal influences to find the vessel velocity relative to water. For example, a vessel travelling in waters with a current of 4 kt and a speed over ground of 12 kt in the same direction would equate to a vessel travelling at 8 kt relative to water, while the same current and vessel heading in opposite directions would equate to a vessel travelling at 16 kt relative to water. The largest waves will be produced by a vessel travelling against the current. Both speed and depth of draft affect the size of waves created as vessels travel through water. Wave parameters can be calculated using the following equations (Kriebel and Seelig, 2005, USACE, 2008):

$$F^* = F_{L*}\exp^{(\alpha d/D)} \qquad \text{Eq (5)}$$

$$gH/v^2 = \beta(F^* - 0.1)^2(y/L)^{-1/3} \qquad \text{Eq (6)}$$

$$F_L = v/(gL)^{1/2} \qquad \text{Eq (7)}$$

Where,
- g = Acceleration due to gravity, ft/s^2
- F^* = Modified Froude number, Eq (5)
- H = Wave height, ft, Eq (6)
- v = Ship velocity relative to water , ft/s
- y = Distance from sailing line, ft
- D = Depth of water, ft
- F_L = Length-based Froude number, Eq (7)

Kriebel and Seelig, 2005, suggest using a y=L for analyses of wave height with Eq (6). Due to the assessment of multiple vessel classes with varied lengths, an approximate average of the vessel class lengths was suggested by McCartney and Scully, 2014, thus leading to an assessment at a distance from the sailing line, y, of 975 ft.

Eq (6) was used to calculate wave height using data shown in Tables 2.5.2 for the existing harbor and deepening harbor predictions. Results are presented in Table 2.5.2.4.

Table 2.5.2.4. Maximum Wave Heights (ft) Produced 975 ft from Sailing Line at Low Tide in Lower Harbor (McCartney and Scully, 2014)

	Deepening Alternative			
Vessel Class	**No Deepening**	**Deepened 3 ft**	**Deepened 5 ft**	**Deepened 7 ft**
Sub Panamax	0.374	0.344	0.328	0.313
Panamax	0.608	0.552	0.520	0.491
PP Generation I	0.446	0.460	0.469	0.477
PP Generation II	0.353	0.363	0.370	0.376
PP Generation III	0.362	0.369	0.374	0.378

Each wave carries energy, E, imparted by the vessel. Energy per unit wave crest, can be calculated using the equation:

$$E = \rho g H^2/8$$

Where ρ is the density of water, g is acceleration due to gravity, and H is the wave height.

According to economic projections which are presented in Figure 2.5.13, the number of ships expected to call on Charleston in the future will be highest if the harbor is not deepened.

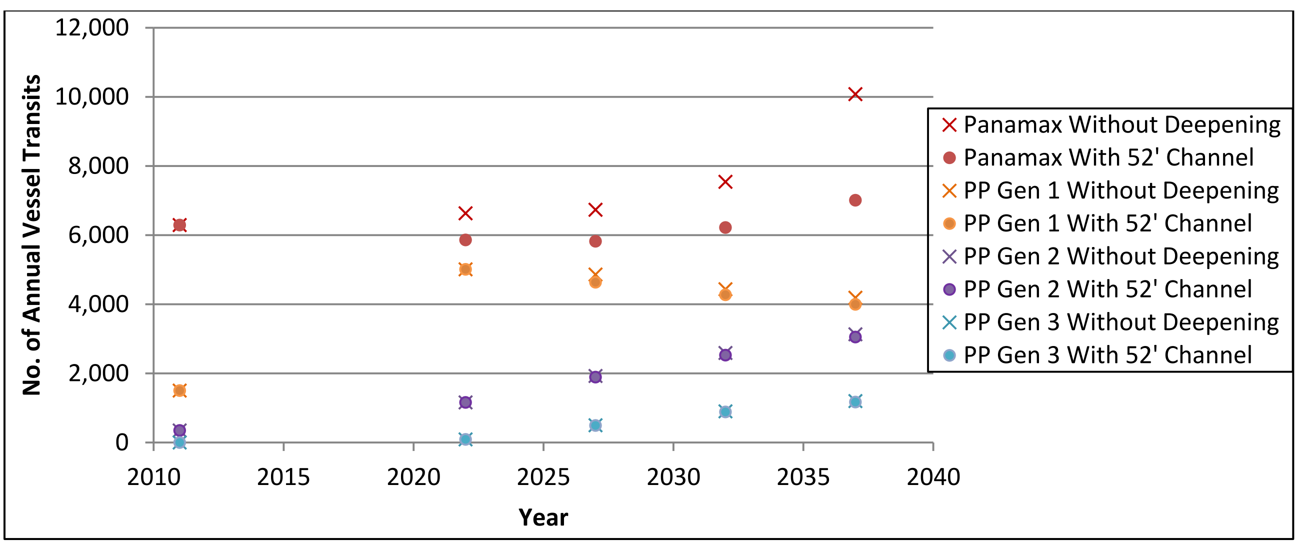

Figure 2.5.13. Vessel Population Transit Predictions With (7 ft) and Without Deepening by Vessel Class

Wave celerity, C, is the speed with which a wave propagates (Kamphuis, 2010). It can be used to calculate wavelength iteratively using Eq (11). The wavelength, L_w, was calculated in order to verify that waves in Charleston Harbor could be treated as deepwater waves, and thus would have a group velocity, C_g, equivalent to 0.5C.

$$F = v/(gD)^{1/2} \quad \text{Eq (8)}$$

$$\theta = 35.27*(1\text{-}\exp^{(12(F-1))}) \quad \text{Eq (9)}$$

$$C = v \cos \theta \quad \text{Eq (10)}$$

$$C^2 = (gL_w/2\pi) \tanh (2\pi D/L_w) \quad \text{Eq (11)}$$

Where, F = Froude number, Eq (8)
θ = Angle of wave propagation, degrees, Eq (9)
C = Celerity, ft/s, Eq (10)
L_w = Wavelength, ft, Eq (11)

The power generated by a ship is converted to the production of waves. These waves have a power per unit crest width given by the equation:

$$P = E*C_g$$

The resulting power per vessel transit can be calculated for all vessels within the vessel population and aggregated to a representative annual power, P_{class}, depicting the power imparted to the harbor system by each vessel class for each deepening scenario by reach.

$$P_{class} = E*n_{ships}*C_g$$

Assuming each vessel within a vessel class transits a similar distance since they all transit to the existing ports according to observed GPS path data and the defined federal channel, it has been determined that the relative contribution of power from the Panamax vessels is greatest of the commercial vessel classes. Relative wave power variations for each alternative and year by including the contribution by each vessel class are provided in Figure 2.5.15 for all vessel classes combined. As shown in Figure 2.5.14, if no deepening occurs, wave power will still be approximately 34% higher in 2022 than in 2011 and approximately 90% higher in 2037 than in 2011.

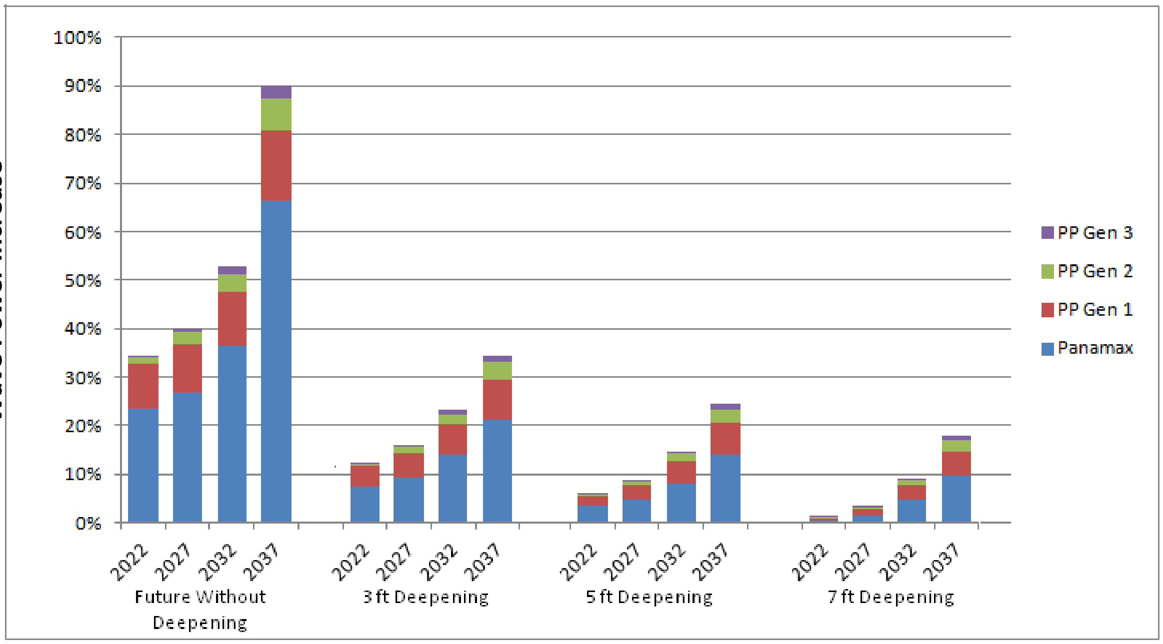

Figure 2.5.14. Predicted Vessel-generated Wave Power Increase by Alternative for Lower Harbor Relative to 2011 Vessel-generated Wave Power for All Vessel Classes

The tidal influences which affect speed will also affect the potential of vessel-generated waves to reach sensitive shorelines. As vessel-generated waves approach the shore they decay with a theoretical decay rate of 1/3 as they move away from the sailing line. This type of decay curve is shown in Figure 2.5.15.

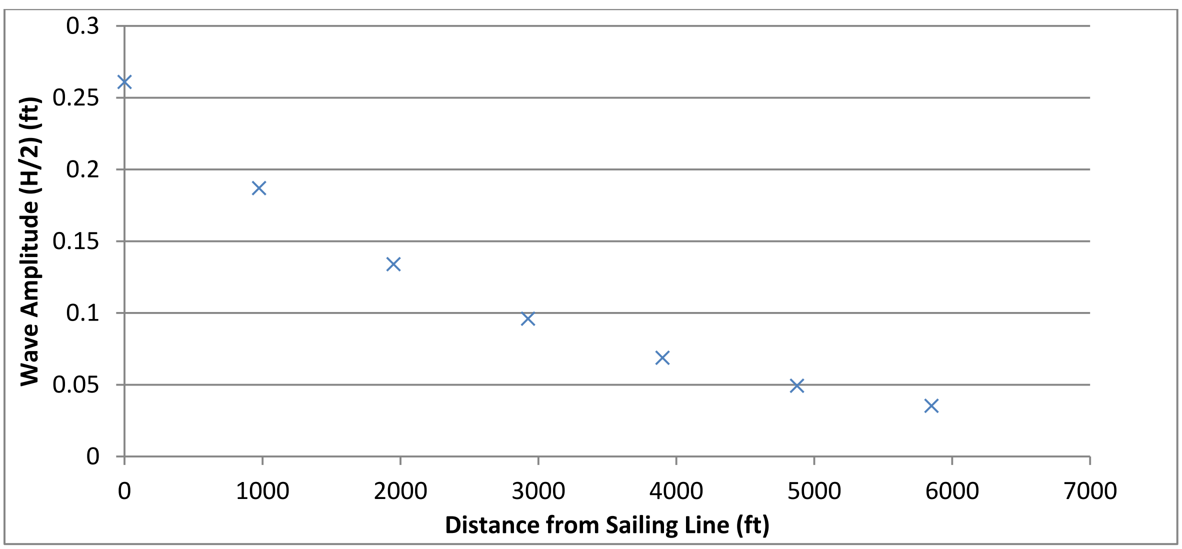

Figure 2.5.15. Wave Decay with Distance from Sailing Line for Sub Panamax Vessel in Existing Harbor

The flat slope at erosion sites of concern allow for waves generated at lower tide elevations to break far from fragile vegetation. During high tide, waves impact regions higher on the shore, as depicted in Figure 2.5.16. The period of high tide, corresponding to the period of transit of large vessels, is therefore of interest at Crab Bank and other sensitive regions.

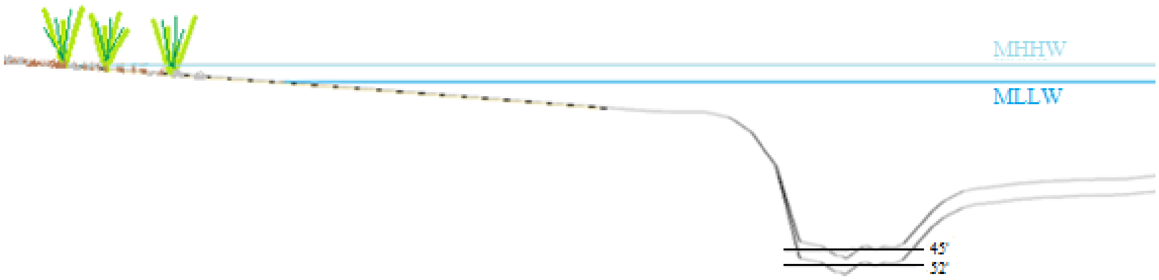

Figure 2.5.16. Crab Bank Cross-section with authorized depth of 45 ft compared to a 7 ft deepened harbor (52 ft depth) at low tide and high tide, neglecting any advance maintenance dredging for simplicity

The largest waves which reach the shore were estimated and are shown in Table 2.5.2.5. The largest calculated waves were all attributed to the Panamax vessel class in the lower harbor since they transit at the highest speeds of all vessel classes in any of the reaches, according to AIS data.

The tidal regime in Charleston is semi-diurnal with a range of 5.77 ft and a period of T=12.42 hr. For a nominally 45 ft deep channel, as Charleston Harbor is authorized, depth available ranges from 45 ft at low tide to approximately 50.77 ft at high tide. By deepening the channel to 52 ft nominally, the available depth at high tide would increase to 57.77 ft.

Table 2.5.2.5. Vessel wave height estimates near shore in lower harbor

Area of Concern (See Section 2.5.1)	Scenario	Distance to Shore (ft)	Wave Height Near Shore (ft)
Fort Sumter	Existing	2925	0.31
	2037 Without Deepening		0.31
	2037 With 7 ft Deepening		0.25
Shute's Folly Island	Existing	1950	0.44
	2037 Without Deepening		0.44
	2037 With 7 ft Deepening		0.35
Crab Bank	Existing	2925	0.31
	2037 Without Deepening		0.31
	2037 With 7 ft Deepening		0.25
Sullivan's Island	Existing	1950	0.44
	2037 Without Deepening		0.44
	2037 With 7 ft Deepening		0.35

The largest ships observed calling on Charleston had a 48 ft nominal draft, and could not safely transit at low tide (GCaptain, 2011). To accommodate UKC, a minimum of 2 ft should be added to the draft of the vessel, giving an apparent draft of 50 ft. This apparent draft is 5 ft above MLLW and limits these large vessels to call at high tide, as shown in Figure 2.5.17 as the time period between t_1 and t_2.

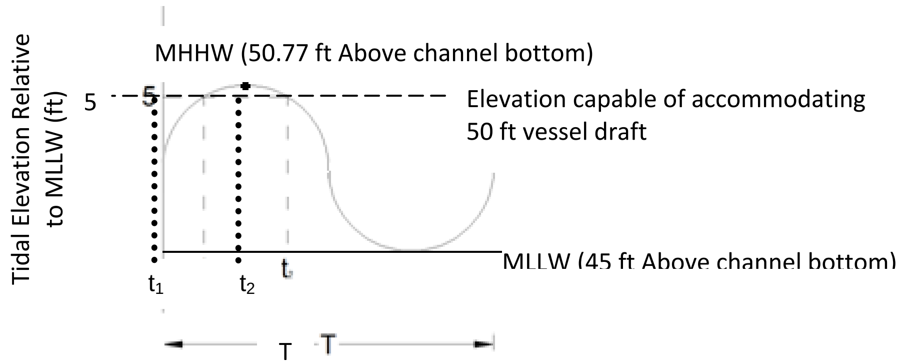

Figure 2.5.17. Tidal Range for one Tidal Period (T=12.42 hr), 50 ft Apparent Draft Vessel in a 45 ft Harbor

The difference in time between the dotted lines indicating $time_1$ and $time_2$ is nominally 4 hrs or 33% of the tidal period, T. This period indicates when 5 ft of tide or 50 ft of channel depth is available. In a year, Charleston would expect a number of tide cycles computed as:

$$365 \text{ days/yr} * 24 \text{ hrs} * 1/(12.4 \text{ hr/tide}) = 705.3 \text{ cycles/yr}$$

All large vessels calling on Charleston Harbor must travel with lighter loads to reduce draft or arrive in the time span of high tide. Over the year, concentrated vessel-generated wave energy at high tide occurs 705 times.

The full tidal range will accommodate a vessel with 48 ft draft, 50 ft apparent draft, if in a 52 ft deep harbor as depicted in Figure 2.5.18 below (i.e. t_2-t_1=T).

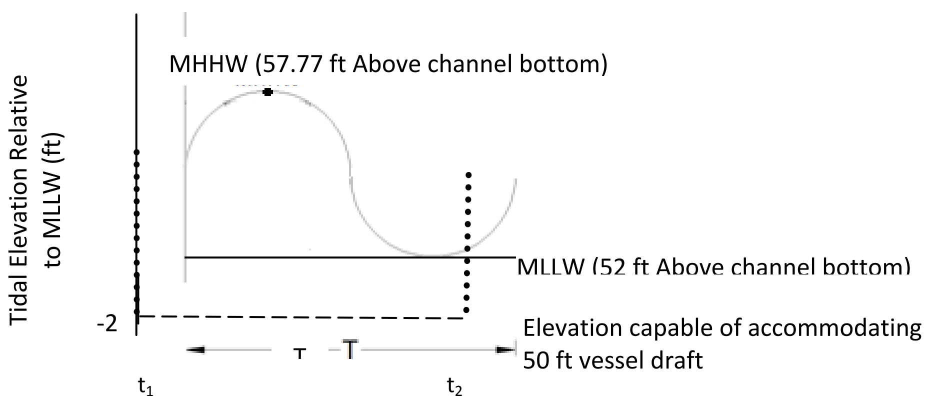

Figure 2.5.18. Tidal range for one tidal period (T=12.42 hr), 50 ft apparent draft vessel in a 52 ft deep harbor

To compare the effects of vessel-generated waves over a given time period, including tidal constraints, the concept of power density is applied:

$$\text{Power Density} = (n_{tides}/T_T)\ [\sum P_{class} / (t_2 - t_1)]$$

Where, $\sum P_{class}$ is the sum of power of all vessel-generated waves by vessels of a particular size during a timeframe of interest, T_T, which contains a given number of tides, n_{tides}. The time frame during each tidal period which accommodates all vessels of that size is described as the difference from t_2 to t_1 (refer to Figures 2.5.17 and 2.5.18). As discussed previously, the range t_2 to t_1 is 4 hr for the vessel with 50 ft apparent draft in a 45 ft channel and 12.4 hr for the same vessel in a 52 ft channel.

For the regions where marshes and shorelines are threatened by vessel activity, the power density is decreasing with each incremental foot of deepening. This occurs because power generation (P) is constant for a certain unchanging size of vessel, the number of ship transits, n_{ships}, is decreasing due to a transfer of cargo to larger vessels as deepening permits which requires less total vessels than the existing channel conditions. The range of time (t_2-t_1) for which the harbor can accommodate vessels of a certain size class is expected to also increase with deepening since the harbor would have a larger available depth for safe UKC.

A relative comparison of the power density for the current Post Panamax Generation II vessel fleet in a 45 ft channel with the predicted vessel fleet in 2037 for a 45 ft and 52 ft channel is made in Table 2.5.2.6.

Table 2.5.2.6. Power Density at y = 975 ft Comparison for Post Panamax Generation II in Charleston Harbor

	n_{tides}/T_T (tides/yr)	P_{class} (Relative to 2011)	t_2-t_1 (hr/tidal cycle)	Power Density*
Existing	705.3	1	4	1.63
Future Without Deepening	705.3	1.88	4	3.06
Future With 7 ft Deepening	705.3	1.9	12.4	1

*Values provided in terms of relative power density

Deepening the harbor will decrease the power density by increasing the time period for which available depth accommodates large vessels. This implies that large vessels will be more likely to pass areas of concern at times when the water surface elevation is lower than high tide, lessening the impact to sensitive marsh habitats and shorelines. The reduction in the total number of vessels expected to call on the port after deepening versus the “Future Without Deepening” scenario will also reduce the likelihood of vessels passing at critical high tides.

The large wave heights produced by Panamax vessels relative to other vessel classes can be attributed mainly to their higher travel speeds. Deepening the harbor increases wave height for PP vessel classes by fractions of an inch but reduces the wave height from Panamax vessels by over an inch. Since the largest vessel class population is also the Panamax vessels, and that population is expected to shrink over time and with deepening, due to a cargo shift to a few larger vessels, the total energy imparted on the system is expected to be reduced appreciably. The decrease in wave height predicted for smaller vessel classes in a deepened harbor is larger than the predicted increase in wave height for PP vessel classes in a deepened harbor which results in a potential impact reduction when compared to the future harbor without deepening.

The comparison in Table 2.5.2.6 emphasizes the potential effects of harbor deepening as determined using power density and representative tidal data. The expected increase in power generation by vessels of increased size for a deepened harbor is not predicted to have a larger input to the harbor system, and was determined to potentially reduce the energy generation density due to the larger available depth, as seen from t_2-t_1 increasing to the full tidal period in Table 2.5.2.6.

An increase of blockage factor for Sub Panamax and Panamax vessels as predicted would reduce the erosive impacts of vessel-generated waves from those vessels. Since Charleston Harbor is unconfined in its current state, and moves further from confined channel conditions the further it is deepened, it is unlikely that these changes to the blockage factor will have any impact on the effects of vessels on the shoreline.

Regardless of uncertainty in model inputs, the relative contribution between vessel classes provides insight into the changes for the future which can be expected. The equations support the hypothesis that increased vessel sizes transiting in a deepened harbor throughout the day would be less impactful than vessels of increased size transiting in the current harbor over the limited window of high tide. Additionally, the increase in number of Post-Panamax vessels is expected to provide a significant decrease in the number of Panamax and other smaller commercial vessels, which are known to travel at increased velocities and generate larger waves, providing another benefit from harbor deepening.

2.5.3 Comparison of Waves Generated by Winds and Vessels

A comparison of vessel wake data and wind data was performed. Constant wind waves ranging up to 24 kt and gusts up to 40 kt are produced consistently throughout the harbor and offshore of Charleston, South Carolina. Storms in Charleston Harbor can also have significant impacts to shorelines and key habitat. Water level increased 6.9 ft above MHHW in 1989 from Hurricane Hugo and 1.9 ft above MHHW from Hurricane Floyd, remaining elevated for multiple days leading up to the storms and exerting additional energy to sensitive regions above the high water line (Zervas et al., 2000). Winds that travel over Charleston Harbor push waves to the shore which increase the longer they blow and the longer the available fetch distance. Waves produced by vessels dissipate as they move away from the transiting vessel towards the shore. Waves generated by winds and waves generated by vessels are estimated to be of similar heights on average in Charleston Harbor but occur with extremely different distributions and frequencies.

Whether erosion or accretion may be caused from vessel- or wind-generated waves is difficult to quantify due to wave addition and cancelling effects and was not feasible to determine as part of this study. Tidal effects can cancel out on average due to their back and forth trend. Vessels follow a similar path as the tides in Charleston Harbor but may occur out of sync with the tides and can thus have varied erosion and accretion effects. Winds occur from, and generate waves propagating in, all directions and are thus very difficult to assess for erosion and accretion effects.

From the wind distributions presented in this appendix and the wave heights in Table 2.5.2.5, equivalent wind generated waves and vessel generated wave frequencies were analyzed.

Fort Sumter and Crab Bank currently experience maximum average wave heights from passing vessels equivalent to a 9.25 kt wind blowing over a 5000 ft fetch in the harbor. Equivalent or larger wind waves occur for 107 days per year, and 29.4 days per year at the angles of concern for Fort Sumter. Equivalent or larger wind waves occur at the angles of concern for Crab Bank 47 days per year.

At Sullivan’s Island and Shute’s Folly Island, the existing conditions lead to an estimated vessel wake equivalent to an 13.25 kt wind over 5000 ft fetch due to similar distances from the sailing line of vessels. Equivalent or larger wind waves occur for 14.6 days per year at Shute’s Folly Island within angles of concern and 19.3 days per year within Sullivan’s Island angles of concern.

In a deepened harbor, the maximum average wave height created by vessels is predicted to be shorter than existing conditions since vessel-generated waves are greatly affected by the depth of water they are traveling through. Wind wave height is independent of water depth since waves are generated due to surface forces. Therefore, the maximum average wave height reaching Fort Sumter and Crab Bank from passing vessels is equivalent to an 8.25 kt wind blowing over a 5000 ft fetch; slightly smaller than in a shallower harbor. Equivalent or larger wind waves occur for 137 days per year, and wind waves exceed the size of those minimum conditions 38 days per year at the angles of concern for Fort Sumter and 66.8 days per year at angles of concern for Crab Bank.

The maximum average wave height reaching Sullivan’s Island and Shute’s Folly Island in a deepened harbor is estimated to be equivalent to an 11 kt wind blowing over a 5000 ft fetch. A wind producing waves equivalent to those conditions occurs 33.2 days per year at the angles of concern for Sullivan’s Island and 21.2 days per year at angles of concern for Shute’s Folly Island.

These equivalent wind-generated wave frequencies were compared to the frequency of vessels passing each area of concern in Table 2.5.3.1. Due to the level of uncertainty in this type of analyses, an order of magnitude comparison was used to generally compare the total effects in any given year which can be attributed to vessels and wind. Since wave energy is determined directly from wave heights, the ratios determined in Table 2.5.3.1 would be the same if performed on an energy basis.

Table 2.5.3.1. Comparison of wind waves to vessel waves frequency

Area of Concern	Scenario	H_{vessel} Near Shore (ft)	Length of Area (ft)	% Exceeding[1]	Equiv Days - Wind[2]	Equiv Days - Vessels[3]	Ratio per Ft of Shore[4]
Fort Sumter	Existing	0.31	600	8.05	29.4	2.95	2.2E+04
	2037 Without Deepening	0.31	600	8.05	29.4	6.81	9.6E+03
	2037 With 7 ft Deepening	0.25	600	10.4	38.0	5.65	1.5E+04
Shute's Folly Island	Existing	0.44	1300	4.0	14.6	6.30	1.1E+04
	2037 Without Deepening	0.44	1300	4.0	14.6	14.56	4.8E+03
	2037 With 7 ft Deepening	0.35	1300	5.8	21.2	12.09	8.4E+03
Crab Bank	Existing	0.31	4300	12.9	47.0	20.90	3.6E+04
	2037 Without Deepening	0.31	4300	12.9	47.0	48.18	1.6E+04
	2037 With 7 ft Deepening	0.25	4300	18.3	66.8	40.01	2.7E+04
Sullivan's Island	Existing	0.44	5000	5.3	19.3	24.30	1.5E+04
	2037 Without Deepening	0.44	5000	5.3	19.3	56.08	6.4E+03
	2037 With 7 ft Deepening	0.35	5000	9.1	33.2	46.57	1.3E+04

[1]This is the percent of time winds from NOAA data are predicted to create waves larger than the near shore vessel wave within the angles of concern shown in Figure2.5.4

[2]This is the "% Exceeding" multiplied by 365 days to describe the total time each year in which the wind is creating waves equivalent or larger than the near shore wave height at the Area of Concern

[3]This is the total amount of time in days each year in which vessels are passing the Area of Concern, assuming vessel speed of 12 knots and using total vessel transits from 2011 - Existing (8500 transits), 2037 – Without Deepening (19600 transits) and 2037 – With 7 ft Deepening (16277 transits)

[4]Ratio of number of wind waves per year (equal or larger) to vessel waves per year impacting each foot of the area of concern (Assuming "Equiv Days – Wind" are exerted at all points, while the "Equiv Days – Vessels" are divided by "Length of Area" and the average period of wind waves = 1 sec, average period of vessel waves = 3.7 sec, as determined using the ACES tool)

It was estimated that winds produce waves which reach each foot of the areas of concern 10^3-10^4 times as often as vessels. The vessel-generated wave height compared were the maximum calculated wave height including maximum tidal effects while the wind waves compared did not include tidal effects. The use of these assumptions means that the ratio determined could be underestimating the size of waves generated by winds and overestimating the size of waves generated by vessels. Additionally, the wave heights of wind-generated waves range much higher than any estimated vessel-generated waves at a distance of at least 975 ft from the vessel sailing line. The ratios determined in Table 2.5.3.1 do not include adjustments for the range of wave heights, and only compares the number of waves occurring due to winds which meet or exceed the height of waves generated by vessels.

Overall, erosion of Charleston Harbor shorelines is controlled predominately by wind waves and tidal currents. The relative infrequency of cargo vessel waves compared with wind waves means that they are only a minor factor contributing to shoreline changes and erosion. Under existing channel conditions, it has been shown that the vessel generated wave power will increase 34% in 2022 and 90% in 2037 over the 2011 condition due to the number of ships. Deepening the federal navigation channel will reduce the shoreline impact of vessel generated waves by the reduction of the number of ships and the range of tides that the ships can traverse the inner harbor (reference Figure 2.5.14).

2.6 New Work Quantity Computations

New work volume quantities for all reaches in the Charleston Harbor as a part of the Post 45 Feasibility Study were computed for estimating disposal needs and costs. In addition to deepening the existing channel alignment, widening measures were also included as alternatives for the study to accommodate future larger cargo vessel traffic in the harbor. New work dredging quantities were determined for both deepening and widening options of the Charleston Harbor.

All calculated dredge volumes were based on Chapter 10, Construction Dredging Measurement, Payment, and Clearance Surveys of USACE Engineering Manual 1110-2-1003, Hydrographic Surveying. Hypack software was used to determine volume quantities listed in Table 2.6.1 and Table 2.6.2. Quantity amounts are listed in cubic yards and were derived from bathymetric surveys of each reach in the Charleston Harbor. Areas requiring advanced maintenance beyond the normal 2’ were taken into account while determining quantities (refer to table 1.2.1). These areas include Lower Wando Reach, Wando Turning Basin, Drum Island Reach, Lower Town Creek Reach, Ordnance Reach and Ordnance Turning Basin. The assumption was made to keep the extents of these high shoaling areas consistent with what has been occurring under existing conditions. All channel reaches will adopt the existing channel side slope of one vertical on four horizontal. Table 2.6.1 and Table 2.6.2 show total quantities classified by segments at the bottom of both tables. Segment 1 consists of Mount Pleasant Reach, Rebellion Reach, Bennis Reach, Horse Reach, Hog Island Reach, Wando River Lower Reach, Wando River Upper Reach and Wando River Turning Basin. Segment 2 consists of Drum Island Reach, Myers Bend and Daniel Island Reach. Segment 3 consists of Daniel Island Bend, Clouter Creek Reach, Navy Yard Reach, North Charleston Reach, Filbin Creek Reach, Port Terminal Reach, Ordnance Reach and Ordnance Reach Turning Basin.

The base condition alternative (Table 2.6.1) is defined as simply deepening the existing channel with no modifications to the extents of the channel toes. The new work quantity totals in the table were calculated by deducting the existing maintenance quantity. The depths listed in each column identify the project depth with the actual depth in parentheses (actual depth + 2' advanced maintenance+ 2' overdepth). The quantities listed only identify new work material.

Table 2.6.1. New Work Dredge Quantities

			New Work Quantity Total at Project Depth (Actual Depth), in Cubic Yards						
Reach	Start Station	End Station	46' (50')	47' (51')	48' (52')	49' (53')	50' (54')	51' (55')	52' (56')
Mount Pleasant Reach	900+00	995+18	23,898	82,884	196,530	339,976	496,990	663,967	840,083
Rebellion Reach	995+18	1077+91	59,059	162,828	312,491	490,390	681,413	879,372	1,081,341
Bennis Reach	1077+91	1155+87	65,660	190,886	358,119	541,378	736,374	940,017	1,151,213
Horse Reach	1155+87	1179+00	12,121	32,473	70471	113,459	189961	262,230	264070
Hog Island Reach	1178+23	1273+12	110,293	263,455	458,544	682,292	934,485	1,208,111	1,493,972
Wando River Lower Reach	0+00	71+49	257,793	382,814	523,589	674,605	832,524	995,252	1,162,064
Wando River Upper Reach	71+49	119+78	46,250	107,415	192,468	296,509	407,901	521,548	636,251
Wando River Turning Basin	70+76	109+00	64,892	137,968	214,504	292,086	370,560	449,902	530,097
Drum Island Reach	1273+12	1317+21	122,117	196,833	293,901	410,476	539,927	678,524	825,796
Myers Bend	1317+21	1342+77	45,505	113,857	204,567	311,315	425,781	544,925	666,402
Daniel Island Reach	1342+77	1412+71	125,375	300,709	519,440	773,667	1,041,015	1,314,719	1,592,690
Daniel Island Bend	1412+71	1440+86	15,962	37,045	74,551				
Clouter Creek Reach	1140+86	1509+00	96,155	232,407	389,959				
Navy Yard Reach	1509+00	1566+65	81,661	211,072	358,816				
North Charleston Reach	1566+65	1615+95	33,372	109,877	225,645				
Filbin Creek Reach	1615+95	1664+72	23,387	69,348	156,072				
Port Terminal Reach	1664+72	1701+00	27,374	78,918	160,376				
Ordnance Reach	1701+00	1720+83			30,989	72,331	118,091		
Ordnance Reach Turning Basin	1698+65	1720+83			56,845	116,170	176,617		
Segment 1			639,966	1,360,724	2,326,716	3,430,695	4,650,208	5,920,399	7,2459,091
Segment 2			292,997	611,399	1,017,908	1,495,458	2,006,723	2,538,169	3,084,888
Segment 3			277,910	738,666	1,453,254	188,501	294,707		
Total			1,210,873	2,710,788	4,797,877	5,114,654	6,951,639	8,458,567	10,343,979

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The quantities are further broken out for base, advanced and overdepth volumes for the recommended plan in the Coast Engineering appendix.

The widening alternatives (Table 2.6.2) include all channel widening measures outside of the current toe of the Federal Channel. Since the widening measures alter the existing Federal Channel, any dredging that occurs in these areas is considered new work. Therefore, all quantities listed under the widening tab are new work computations. The depths listed in each column identify the project depth with the actual depth in parentheses (actual depth + 2' advanced maintenance+ 2' overdepth).

Table 2.6.2. Maximum Widening New Work Quantities

	New Work Quantity Total for Maximum Widening Option at Project Depth (Actual Depth), in Cubic Yards						
Reach	46' (50')	47' (51')	48' (52')	49' (53')	50' (54')	51' (55')	52' (56')
Bennis Reach	304,765	387,562	464,394	536,426	622,016	699,743	791,645
Hog Island Reach Lower	6,326	16,413	31,715	59,308	88,390	117,165	146,515
Hog Island Reach Upper	240,316	273,906	308,492	344,025	380,523	417,985	456,433
Wando River Lower Reach	355,247	398,317	442,018	478,658	523,742	560,655	607,006
Wando River Turning Basin	2,112,790	2,219,130	2,324,725	2,431,067	2,538,148	2,645,972	2,754,536
Drum Island Reach	56,724	62,654	68,765	75,295	81,815	85,012	91,677
Myers Bend	126,331	142,829	153,015	159,756	170,097	176,806	187,287
Daniel Island Reach	435,426	465,124	494,980	525,720	556,413	587,891	619,267
Clouter Creek Reach	143,280	167,650	193,191				
North Charleston Reach	245,331	276,119	307,048				
Filbin Creek Reach	192,131	220,283	249,348				
Filbin-Port Terminal Bend	24,357	27,924	31,692				
Ordnance Reach Turning Basin	1,133,901	1,193,600	1,253,007	1,311,876	1,372,696		
Segment 1	3,019,444	3,295,328	3,571,344	3,849,484	4,152,819	4,441,520	4,756,135
Segment 2	618,481	670,607	716,760	760,770	808,326	849,709	898,231
Segment 3	1,739,000	1,885,577	2,034,285	1,311,876	1,372,696		
Total	5,376,925	5,851,511	6,322,390	5,922,131	6,333,840	5,291,229	5,654,366

The entrance channel presently extends from station 0+00 to 900+00 – approximately 17 miles. Deepening will require an extension of approximately 3 miles.

In April 2014, quantities in the Entrance Channel (EC) for the Post 45 Feasibility Study were revised based on a new configuration. Rock discovered in portions of the entrance channel requires overdepth dredging for safety. Initially, entrance channel was assumed 2-ft deeper than harbor + 2-ft of advanced maintenance + 2-ft of allowable overdepth. Following the identification of rock, the entrance channel was subdivided into two segments (Figure 2.6.1).

EC1 extends from station 0+00 to station 365+00, which is approximately 1000' beyond the existing maintenance shoals in the Entrance Channel. Quantities are based on 2-ft for vertical motion, 2-ft of advanced maintenance & 2-ft of allowable overdepth. EC-1 would consist of the area where normal maintenance dredging activity was conducted prior to the deepening.

EC-2 extends from station 365+00 to the new proposed seaward extension. Quantities based on 2-ft for vertical motion, 1-ft of required overdepth (rock) & 2-ft of allowable overdepth. In order to reduce impacts to hardbottom habitat within the margins of the channel and reduce new work dredging costs, USACE proposed an idea to shorten the width of the wings by keeping the current 4:1 slope consistent at deeper depths. The reduction of material to be dredged based on the change resulted in a reduction of approximately 400,000 CY.

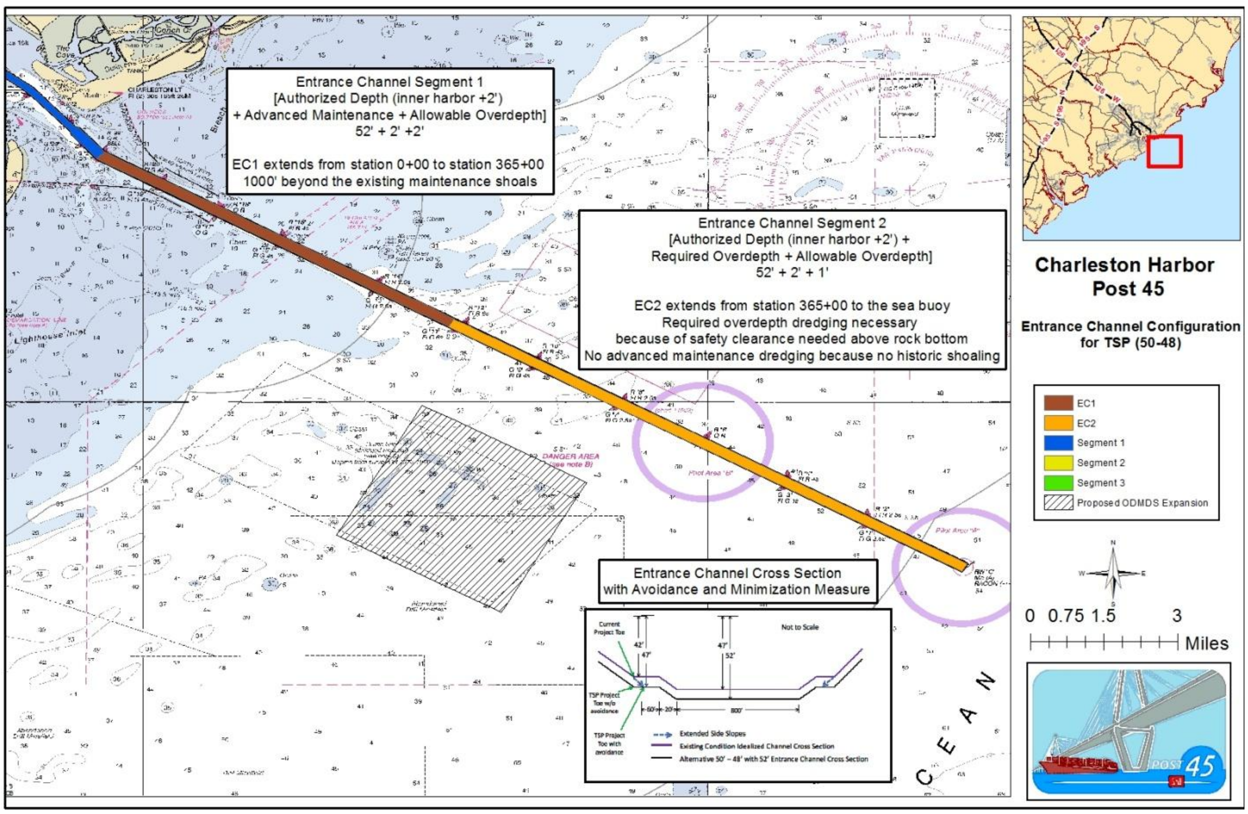


Figure 2.6.1. Entrance Channel Segments

Advanced Channel Design was used in Hypack to create EC templates at each depth. Quantities were then determined for each template by using condition surveys and calculating volumes in EC-1 & EC-2 by both the TIN Modeler & Cross Section and Volumes. The resulting quantities can be seen below in Table 2.6.3.

All volume calculations were verified through an in-house District Quality Control procedure, using GIS capabilities. Dredge volumes were compared using the same survey data sets, but differing volume calculation methodology contained in Hypack and ArcGIS.

Table 2.6.3. Entrance Channel New Work Quantities

New Work Quantities (CY)					
EC-1			EC-2		
48'+2'+2'			**48'+1'+2'**		
50'	Pay	1,243,305	50'	Pay	161,779
52'	Advanced	1,530,809	51'	Target	330,452
54'	Overdepth	2,241,907	53'	Overdepth	1,843,251
	Total	5,016,021		Total	2,335,482
50'+2'+2'			**50'+1'+2'**		
52'	Pay	2,115,292	52'	Pay	985,547
54'	Advanced	3,137,133	53'	Target	1,147,391
56'	Overdepth	2,589,717	55'	Overdepth	3,377,348
	Total	7,842,142		Total	4,103,095
52'+2'+2'			**52'+1'+2'**		
54'	Pay	6,319,792	54'	Pay	4,039,854
56'	Advanced	1,522,638	55'	Target	1,697,136
58'	Overdepth	2,626,133	57'	Overdepth	3,992,821
	Total	10,468,563		Total	9,729,811

2.7 Dredged Material Disposal

A preliminary assessment was performed to document the management of all planned Federal and non-federal maintenance material, the dredged material generated from deepening the Federal channel (new work), as well as the improvements by the state. Based on the additional capacity need for new work material, a current and anticipated capacity in each upland disposal area was examined. The goal of the examination was to ensure that the upland disposal areas have adequate capacity for the new work and are available the future maintenance needs. The current capacity of the cells without any dike maintenance is not sufficient for placement of the new work material nor is barging new work material offshore to the ODMDS economically feasible for most of the Upper Harbor reaches. Therefore, a plan that encompasses both the Federally owned disposal areas, as well as the disposal areas owned and actively used by the SCSPA was derived. Based on the evaluation, the increased dredged material from the Post 45 Upper Harbor should be separated and placed into three of the upland disposal sites: Yellow House Creek DA (state owned), Daniel Island DA (state owned), and Clouter Creek DA (state and federally owned). The capacity calculations for the plan are based on the cells requiring maintenance to raise the dikes to achieve the required capacity. The new work material is divided based on the geometry of each cell and the location of the shoaled reaches. Figure 2.7.1 shows the breakout of the disposal plan.

This is discussed in more detail in Environmental Appendices or the Dredged Material Management Plan Preliminary Assessment (DMMP PA).

2.8 Coastal Assessments

Coastal analysis for the proposed project has been postponed to PED phase under the new SMART planning requirement to limit the scope of the feasibility phase to those analyses that are determining factors in the decision process. It is accepted that the initial construction and maintenance of the Charleston Harbor navigation channel and jetties has affected the adjacent shorelines and altered the pattern of sediment transport from a natural condition. USACE has already mitigated for the impacts of the jetties and federal project by significantly reducing the cost sharing requirements of the City of Folly Beach for the Shore Protection Project. The 57% acknowledged impact of the jetties is subtracted from Folly Beach’s cost share – reducing it from 35% to 15%. This has resulted in close to $60 million in federal investment since 1993 when the initial shore protection project for Folly Beach was constructed.

There are no changes being proposed to the jetties and the majority of the proposed extension of the entrance channel is already at a depth of 52 below MLLW or deeper. The proposed changes to the entrance channel, when viewed in perspective of both the historical changes and large expanse of the ocean, are expected to result in negligible changes to the waves and currents that transport sediment. The Corps is committed to assessing whether the proposed action will impact the coastal shoreline and the analysis in PED phase will be done to verify that it does not. PED phase will revise costs, benefits and mitigation of the recommended plan if the results of coastal erosion analysis indicate significant impacts.

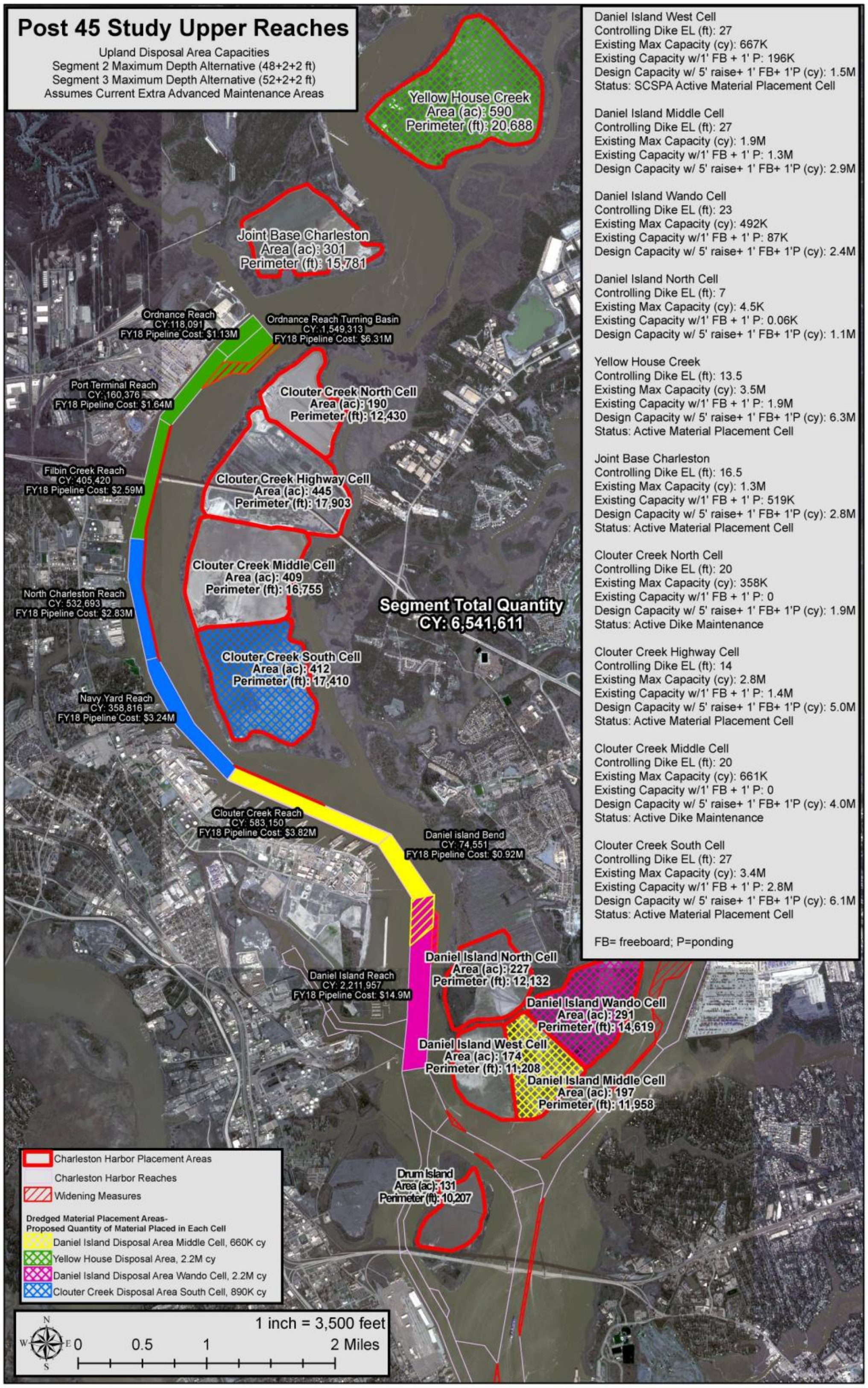


Figure 2.7.1 Dredged Material Placement Areas – Post 45 Proposed New Work Material

3.0 Hydrodynamic Numerical Modeling

3.1 Model Selection - Hydrodynamic/Salinity/sedimentation

3.1.1 Study Goals

Goals of the numerical modeling were to characterize the existing hydrodynamic, salinity, dissolved oxygen (DO) concentrations and sedimentation patterns in the Charleston Harbor System (CHS) estuary, and to estimate the effects of the proposed Post 45 project on the estuary hydrodynamics, salinity, DO concentrations and sedimentation patterns. The model would be calibrated such that it reasonably represents the existing hydrodynamic patterns in the lower Cooper River and Charleston Harbor, as well as salinity patterns, DO concentrations and sedimentation rates. The calibrated model would be used to predict the project-related effects on currents, salinity, DO and sedimentation for various channel modification alternatives. Results of the alternatives modeling would be used for as input for the ship simulation study, evaluation of project effects on salinity and DO and prediction of shoaling rates to estimate changes in navigation channel maintenance dredging quantities. The Environmental Impact Statement (EIS) would use the model results to assist the evaluation of the potential project effects on natural resources of concern (e.g., wetlands, essential fish habitat, threatened and endangered species, etc.).

Numerical models used for previous USACE studies were the TABS- MD collection and performed by USACE WES now the Environmental Research and Development Center (ERDC). ERDC indicated the Charleston Harbor models were no longer available. A new model was necessary.

3.1.2 Model Selection

The model selected for this study would be one of the primary tools used to help answer the following questions:

- How would channel improvements change water quality? (e.g. dissolved oxygen)
- How would channel improvements change currents?
- How would channel improvements change sediment transport and suspended sediment concentrations?
- How would channel improvements change shoaling in the federal navigation channel?
- How would channel improvements change salinity intrusion?
- How would channel improvements change adjacent marsh/surface elevations?

3.1.3 Model Requirements

In addition to the need to answer the above questions, the model code needed to satisfy the following criteria:

- The model code must be approved by the US Army Corps of Engineers for use in USACE studies.
- The model code must be widely applied, verified and peer reviewed from applications in other estuarine systems;

- The model code must be accepted by the US Environmental Protection Agency (EPA) and SCDHEC;
- The model code must be capable of simulating 3D hydrodynamics, salinity, temperature, dissolved oxygen and sediment transport; and

The application of the model code for simulating the CHS must also meet the following needs:

- The model domain must include the Ashley, Cooper and Wando Rivers as well as the Charleston Harbor;
- The model application must be capable of assessing compliance with SCDHEC dissolved oxygen standards;
- To properly represent the tidal flows in the estuary the model domain must include intertidal marsh areas, and the wetting and drying capabilities of the model must be enabled.

USACE Charleston District selected the Environmental Fluid Dynamics Code (EFDC) to meet these needs.

3.1.4 EFDC Description

As described by EPA (2004), the EFDC is a state-of-the-art model that can be used to simulate aquatic systems in one, two, and three dimensions. It has evolved over the past two decades to become one of the most widely used and technically defensible hydrodynamic models in the world. EFDC uses stretched or sigma vertical coordinates and Cartesian or curvilinear, orthogonal horizontal coordinates to represent the physical characteristics of a waterbody. It solves three-dimensional, vertically hydrostatic, free surface, turbulent averaged equations of motion for a variable-density fluid. Dynamically-coupled transport equations for turbulent kinetic energy, turbulent length scale, salinity and temperature are also solved. The EFDC model allows for drying and wetting in shallow areas by a mass conservation scheme. The physics of the EFDC model and many aspects of the computational scheme are equivalent to the widely used Blumberg-Mellor model and U. S. Army Corps of Engineers' Chesapeake Bay model.

The EFDC model has been used by SCDHEC and EPA Region 4 in other basins for evaluating surface water hydrodynamics, sediment transport and water quality problems. The EFDC model was originally developed by Dr. John Hamrick at the Virginia Institute of Marine Science and is considered public-domain software. EFDC is currently supported by Tetra Tech for the EPA Office of Research and Development (ORD), EPA Region 4, and EPA Headquarters.

In addition to hydrodynamic, salinity and temperature transport simulation capabilities, EFDC is capable of simulating the transport and fate of multiple size classes of cohesive and noncohesive suspended sediment, including bed deposition and resuspension (Tetra Tech, 1999). Water column transport is based on the same advection-diffusion scheme used for salinity and temperature. This feature is useful for evaluating the project effects on shoaling patterns in the federal navigation project.

The water quality component of EFDC is based on water quality kinetics from the Chesapeake Bay Water Quality model, CE-Qual-ICM. The water quality component simulates the impacts of oxygen consuming loads from various sources in the watershed on dissolved oxygen in the impaired sediments.

The EFDC model has been used for other USACE feasibility studies, most recently in Savannah, Georgia, and Jacksonville, Florida. Discussions with these districts indicated that the studies resulted in favorable application of the model and acceptance by environmental agencies in meeting the same goals and answering the same questions.

3.1.4.1 Recent Charleston Harbor System Models

3.1.4.1.1 BCD-COG Dissolved Oxygen TMDL Model Study

The Cooper and Ashley Rivers have both been identified as impaired for dissolved oxygen under Section 303(d) of the 1972 Clean Water Act. As a result, multiple model studies have been completed to determine a Total Maximum Daily Load (TMDL) for DO in these tributaries. Most recently, the Berkeley-Charleston-Dorchester Council of Governments (BCD-COG) contracted to develop a 3-D model to represent 3-D in-stream hydrodynamics and water quality in the CHS to establish TMDL allocation alternatives. The model selected for the CHS TMDL study was the Environmental Fluid Dynamics Code (EFDC).

This model study used a 2004 monitoring data set for model calibration, and it used a 1996 data set from an earlier monitoring study to validate the model. The 1996 and 2004 datasets for the CHS represent similar hydrological periods, but the 2004 data provided special studies directed at measuring rates, loads, and boundaries needed for a successful calibration of a 3-D hydrodynamic and water quality model. The 2004 data set includes sufficient data for calibration of a 3-D model, particularly surface, mid-depth, and bottom continuous measurements of DO, temperature and salinity at multiple locations.

As described by Tetra Tech and JJG (2008), the special studies included site-specific measurements of SOD (sediment oxygen demand), water column production and respiration, oxygen diffusion/reaeration, sediment nutrient flux, and effluent mixing. In addition to the special studies, the following major efforts occurred simultaneously: continuous water quality and water level, continuous flow monitoring on all three major branches of the CHS, discrete water quality sampling, non-point source sampling during wet and dry weather conditions, LTBOD (long-term biochemical oxygen demand) rates and kinetics, and marsh import and export of nutrients and organic material. All of the data were used in the model development for rates, kinetics, loads, boundaries, and/or constants.

3.1.4.1.2 Charleston Naval Complex Marine Container Terminal EIS Model (new Navy Base Terminal located at the former Navy Base)

In 2004, the South Carolina State Port Authority initiated an Environmental impact Study (EIS) for a proposed Marine Container Terminal at the Charleston Naval Complex (now known as Navy Base Terminal or NBT) in order to evaluate potential impacts of the proposed project on water levels, currents, salinity and sedimentation patterns in the lower Cooper River. Requirements of the numerical modeling were to characterize the existing hydrodynamic, salinity, DO and sedimentation patterns in the Lower Cooper River. The model would be calibrated to reasonably represent the existing hydrodynamic patterns in the lower Cooper River and Charleston Harbor, as well as salinity patterns and sedimentation rates. The calibrated model would be used to predict the related impacts to currents, salinity and sedimentation resulting from the proposed terminal. Results of the modeling would be used for ship simulation.

The EFDC model was selected to meet these requirements. For the EIS, the BCD-COG DO TMDL model was used to assess the project effects on DO in the Lower Cooper River. According to conversations with the contractor, the BCD-COG DO TMDL model grid was refined and the model calibrated and validated to characterize the existing hydrodynamic, salinity and sedimentation patterns. Data sets from a previous 1996 hydrodynamic model, as well as new data collection in 2003, 2004 and 2005 were used for calibration and confirmation of the model.

Technical review of the Charleston Naval Complex Marine Container Terminal EIS (NBT) model for the Lower Cooper River performed by the USACE included external peer review by Dr. Allen Teeter of Computational Hydraulics and Transport, LLC (CHT). Dr. Teeter is formerly of the USACE Engineer Research and Development Center (ERDC) and performed the hydrodynamic modeling for the 1996 Feasibility Study for Charleston Harbor Deepening and Widening that was completed in 2005.

3.1.5 Decision Process

The Corps had the opportunity to use the existing measured data sets and adapt these fully-developed models for the Charleston Harbor Post 45 Feasibility Study, rather than having to spend additional time and funds collecting data and developing new model applications from scratch.

As the accepted model by the local counties, municipalities, state and federal agencies involved in water quality concerns for the CHS, USACE Charleston chose to use the existing EFDC model to evaluate water quality impacts of the various channel modification alternatives of the Charleston Harbor Post 45 Feasibility study. However, modification of the model grid to increase the number of cells was necessary to insure it will adequately resolve the federal navigation channel and the proposed channel modification alternatives. Using an existing hydrodynamic model, not creating a new separate water quality model, and not needing to collect a large amount of additional water quality data for model calibration was an overall benefit to study cost and schedule, while still meeting goals and needs of the study.

While the Naval Base Terminal EIS study did generate a new model grid for the characterization of the harbor, the area of interest was focused on the site of the proposed marine container terminal. USACE Charleston used the same procedure of starting with the

BCD-COG DO TMDL model and refining the grid for the entire authorized navigation project to insure a sensitivity to channel modifications and to meet the needs of the ERDC Ship Simulation Study. Additional data collection was collected for validation of this new model grid; however, building on an existing model and not creating a new separate hydrodynamic, salinity and sedimentation model will be an overall benefit to the study cost and schedule.

3.1.6 Conclusion

Interagency Coordination Team (ICT) meetings with SCDHEC-BOW, USEPA, NOAA- NMFS, USFWS, SCDNR, SCDHEC-OCRM, and USGS occurred to discuss the model applications and needs for impact assessment.

3.2 Data Collection

Since this study involved refinement of the EFDC grid to capture the federal channel and the proposed widening alternatives, the model required recalibration and verification. New field data collection would be required.

3.2.1 Data for Model Input, Calibration and Validation

The BCDCOG DO TMDL model was calibrated for hydrodynamics, temperature and salinity with 2004 data and validated using 1996 data. Also described in Section 3.1.4.2, the EFDC model was also modified for the proposed CNCMCT EIS and calibrated for hydrodynamics, salinity and sedimentation using 2004 data and validated using 1996 data.

The 2004 field data collection to support the setup and calibration of the BCD-COG DO TMDL model was sufficient to confirm the calibration of the Water Quality Module. The 2004 field data collection was comprehensive, including the boundary forcing data, calibration data and sampling and analysis to estimate important model variables (e.g., LTBOD and SOD measurements, etc.). This measured data set was accepted by the EPA and SCDHEC for the purposes of supporting a DO TMDL (the TMDL was issued in 2013). The hydraulics and the water quality kinetics of the CHS have not substantially changed since the 2004 data collection effort. Therefore, a water quality model calibrated to the 2004 conditions is appropriate for use in evaluating the proposed project effects, and no additional field data collection was required to support the Water Quality Model.

The 2004 data set did not, however, include the necessary information to set up and calibrate a sediment transport model for the navigation channel. Also, the 2003/2004 data collected for the CNCMCT EIS model study was limited to the Lower Cooper River and did not include the Wando River or the harbor area. Therefore, additional field data collection in 2012 was needed for purposes of setting up and calibrating the Engineering Model.

This section describes the monitoring data reused from the previous studies to support the model studies, as well as additional data collected to support the 2012/2013 Engineering Model setup and calibration.

3.2.1.1 Bathymetric Data

New data in the form of multi-beam and single beam surveys performed by the USACE Charleston District was collected for existing condition bathymetric data to determine bottom elevation of each horizontal cell within and adjacent to the federal channel. Data outside the federal channel was supplemented by NOAA survey data collected in 2009 to the 12 ft MLLW contour and 2010 bathymetric data collected by Coastal Carolina University under contract with USACE. In response to comments received during model review in March and April 2013, bathymetry in the upper Cooper River was updated with new survey data collected by the USACE in the Cooper River upstream from the federal channel.

3.2.1.2 Pinopolis Dam Flow

The flow from Lake Moultrie was specified from measured data collected by the USGS in the tailrace of Pinopolis Dam.

3.2.1.4 Point Source Discharges

Point source dischargers that were used for the BCD-COG DO TMDL model were used as input loads for the Water Quality Model.

3.2.1.5 Offshore Water Surface Elevation Data

The water surface elevation time series at the Custom House gage on the Cooper River (Station ID8665530) was used at the open boundary in the Atlantic Ocean. Similar to the previous studies, the phase and amplitude of the offshore boundary water levels was adjusted during calibration.

3.2.1.6 Temperature

The freshwater flows and the open boundary tidal flow exchange heat energy within the model domain. Therefore, the temperatures of the freshwater flows and the tidal flow affect the water temperature in the model domain.

The BCD-COG DO TMDL model used the USGS temperature data listed in Table 3.2.1 for the 2004 calibration comparisons and boundary conditions. For the offshore open boundary, the boundary was initially based on 1996 data measured at the Customs House gage, and then it was adjusted iteratively until achieving good agreement to 2004 data observed at the Cooper River Hwy 17 gage.

There are new sources of data that are closer to the offshore model boundary than the USGS gages, and therefore they provided a better source of boundary salinity and temperature. The Carolinas Regional Coastal Ocean Observing System (Carolinas RCOOS) deployed a buoy offshore from Capers Island in February 2005 (Station CAP2). Also, the National Buoy Data Center (NBDC) maintains a buoy at Station 41004 (EDISTO).

The existing USGS gages collecting continuous temperature data were used for 2012/2013 validation of the model (see Figure 3.2.1) from the USGS gages listed in Table 3.2.2 that are collecting gage height and mid-depth temperature. In addition, the study reinstalled USGS gage 02172100 (Fort Sumter) for calibration of the model for temperature and water levels.

Table 3.2.1 Gages used for 2004 model temperature calibration and boundary conditions

USGS Gage	**Description**
02172002	Lake Moultrie Tail Race at Moncks Corner, SC (upstream boundary condition)
02172020	W Branch Cooper River at Pimlico
02172040	Durham Canal
02172050	Cooper River near Goose Creek (Dean Hall)
02172053	Cooper River at Mobay
021720677	Cooper River at I-526 (Filbin Creek)
021720696	Wando River at Cainhoy (NLS) (boundary condition)
021720698	Wando River at I-526 (above Mount Pleasant)
021720709	Cooper River at Hwy 17 (boundary condition)
02172100	Fort Sumter on Cooper River (NLS)
021720869	Ashley River at I-526
02172084	Ashley River at Bakers Landing (NLS)
02172080	Ashley River at Summerville (NLS) (boundary condition)

*(NLS) indicates no longer in service

Table 3.2.2 Continuous USGS gages in operation for 2012

Station	**Station**	**Parameters**
2172001	Lake Marion near Pinopolis, (Tailrace)	Gage height
2172002	Lake Marion Tailrace canal	Discharge, Velocity, Gage height
2172020	West Branch Cooper River @ Pimlico near Moncks Corner, SC	Specific conductance, temperature, Gage height
2172040	Back River at Dupont Intake nr Kittredge sc	Specific conductance, temperature, Gage height
2172050	Cooper River near Goose Creek, SC	Specific conductance, Dissolved Oxygen, temperature, Gage height
2172053	Cooper River at Mobay	Specific conductance, temperature, Gage height
21720677	Cooper River at I-526	Specific conductance, Dissolved Oxygen, temperature, Gage height
21720698	Wando River above Mount Pleasant (I-526)	Specific conductance, Dissolved Oxygen, temperature, Gage height
21720709	Cooper River at Hwy 17	Specific conductance, Dissolved Oxygen, temperature, Gage height
21720710	Cooper River at Customs House	Specific conductance, temperature, Gage height
21720869	Ashley River near North Charleston (I-526)	Specific conductance, Dissolved Oxygen, temperature, Gage height

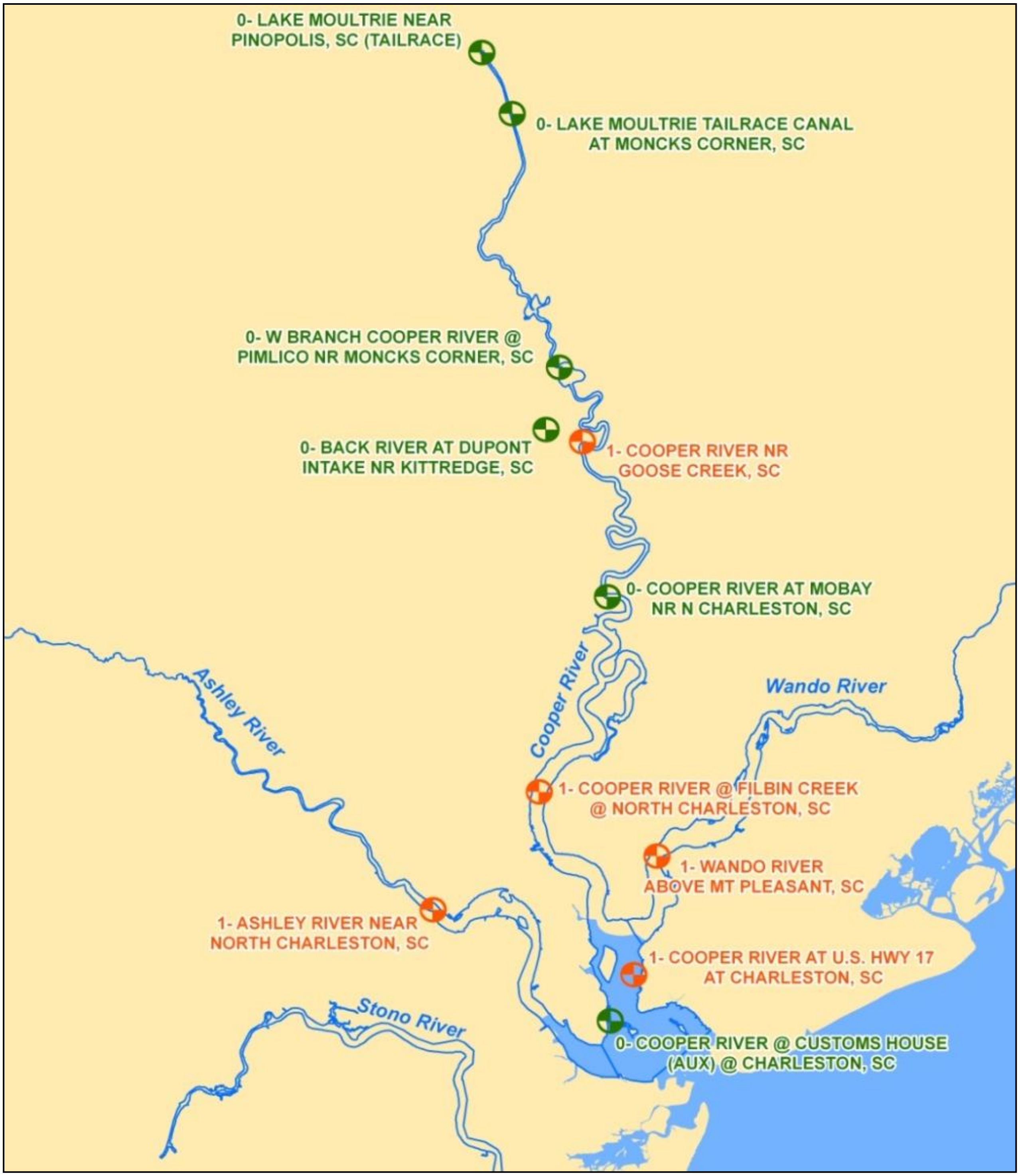


Figure 3.2.1 Continuous USGS monitoring gages in operation for 2012 (Orange indicates DO monitoring)

3.2.1.7 Salinity

3.2.1.7.1 Offshore Open Boundary

The open boundary salinity concentration for 2012 was based on measured data from the Carolinas Regional Coastal Ocean Observing System. The data from a buoy about 5 miles offshore from Capers Island (Station CAP2) was used as the basis for developing the model boundary concentrations. The observed Station CAP2 data used for 2012 were collected from 2005 through 2012. Therefore, measured offshore data were not available for the 2004 period.

3.2.1.7.2 Inshore Data

Existing USGS gages collecting continuous data for salinity were from the USGS gages listed in Table 3.2.2 that collected gage height and mid-depth specific conductance. In addition, the study reinstalled USGS gage 02172100 (Fort Sumter) for salinity and water levels.

3.2.1.8 Dissolved Oxygen

Table 3.2.3 lists the USGS gages for DO that are in operation as of 2012. The BCD-COG DO TMDL model was calibrated using 2004 data from the discrete monitoring sites collected by JJG listed in Table 3.2.4 and the continuous data collected by USGS at the stations listed in Table 3.2.5. The locations of these monitoring stations are shown in Figure 3.2.2.

Table 3.2.3 Gages in operation for 2012 DO validation

USGS Gage	Description
02172050	Cooper River near Goose Creek, SC
021720677	Cooper River at I-526
021720698	Wando River above Mount Pleasant (I-526)
0021720869	Ashley River near North Charleston (I-526)
21720709	Cooper River at Hwy 17

Figure 3.2.2 Location of 2004 Discrete Sampling Stations and Continuous Monitoring Gages

Table 3.2.4 Discrete sampling stations used for 2004 dissolved oxygen calibration

Station	Station Name
JJG-WQ-A1	Ashley River near the Mouth
JJG-WQ-A2	Ashley River downstream of I-526 Bridge (near 021720869)
JJG-WQ-A3	Ashley River at Magnolia Gardens (near 02172084)
JJG-WQ-A4	Ashley River upstream of Dorchester
JJG-WQ-A5	Upper Ashley River at Hwy 17 (near 02172080)
JJG-WQ-C1	Lower Cooper River
JJG-WQ-C2	Cooper River at the I-526 Bridge (near 021720677)
JJG-WQ-C3	Cooper River downstream of William Steam Plant
JJG-WQ-C4	Cooper River upstream of Mobay (near 02172053)
JJG-WQ-C5	Cooper River at Dean Hall (near 02172050)
JJG-WQ-H1	Charleston Harbor at the Mouth (near 02172100)
JJG-WQ-H2	Charleston Harbor - Center Channel
JJG-WQ-H3	Charleston Harbor upstream of Hwy 17
JJG-WQ-W1	Wando River at the I-526 Bridge (near 021720698)
JJG-WQ-W2	Wando River near Cainhoy (near 021720696)

Table 3.2.5 Continuous gages used for 2004 dissolved oxygen calibration

Station	Station Name
02172002	Lake Moultrie Tail Race at Moncks Corner, SC
02172020	Cooper River near Moncks Corner
02172050	Cooper River near Goose Creek, SC
02172053	Cooper River at Mobay
02172067.7	Cooper River at I-526
02172069.6	Wando River at Cainhoy
02172069.8	Wando River above Mount Pleasant (I-526)
02172070.9	Cooper River at Hwy 17
02172080	Ashley River near Summerville
02172084	Ashley River at Bakers Landing
02172086.9	Ashley River near North Charleston (I-526)
02172100	Charleston Harbor at Fort Sumter

3.2.1.9. Currents and Flows

Table 3.2.6 lists the gages collecting current and flow data that were used for the BCD-COG DO TMDL model calibration.

Table 3.2.6 Continuous gages used for 2004 current and flow calibration

Station	Station Name
02172002	Lake Moultrie Tail Race at Moncks Corner, SC
02172053	Cooper River at Mobay (no longer in service)
021720696	Wando River at Cainhoy (no longer in service)
021720869	Ashley River near North Charleston (I-526) (no longer in service)

Only the Lake Moultrie Tailrace gage was still collecting flow data at the time of this study. USACE reinstalled flow measurements at the Cooper River at Mobay gage for the calibration period. In addition, new Acoustic Doppler Current Profiler (ADCP) measurements at 16 locations shown in Figure 3.2.3 were collected. This data collection included a combination of continuous Acoustic Doppler Velocity Meters (ADVM) at 4 locations and discrete current measurements across the channel cross-section at 12 locations. The advantage of continuous ADVMs is that they show a continuous record of the variation between spring and neap tide conditions, they can show the effects of atmospheric conditions (if the monitoring period includes significant wind or passage of a pressure front), and they also provide an estimate of the net current and net flow at the gage location. The continuous gage locations were selected to enable measurement of the flows at the entrance to the harbor in the Mount Pleasant Range, Lower Cooper River in the Daniel island reach, Wando River at the terminal, and the entrance to the Ashley River at James Island Connector bridge. This allowed a characterization of the tidal and net flows from all of the major tributaries to the harbor.

The transect measurements were completed using a boat-mounted ADCP. The transects were performed at the continuous current meter locations, which allowed the continuous current speed and direction data to also be used to estimate continuous flows through the channel. Additional transects were located at problematic shoaling areas and areas where navigation is most difficult, in order to characterize the current patterns in these areas of interest.

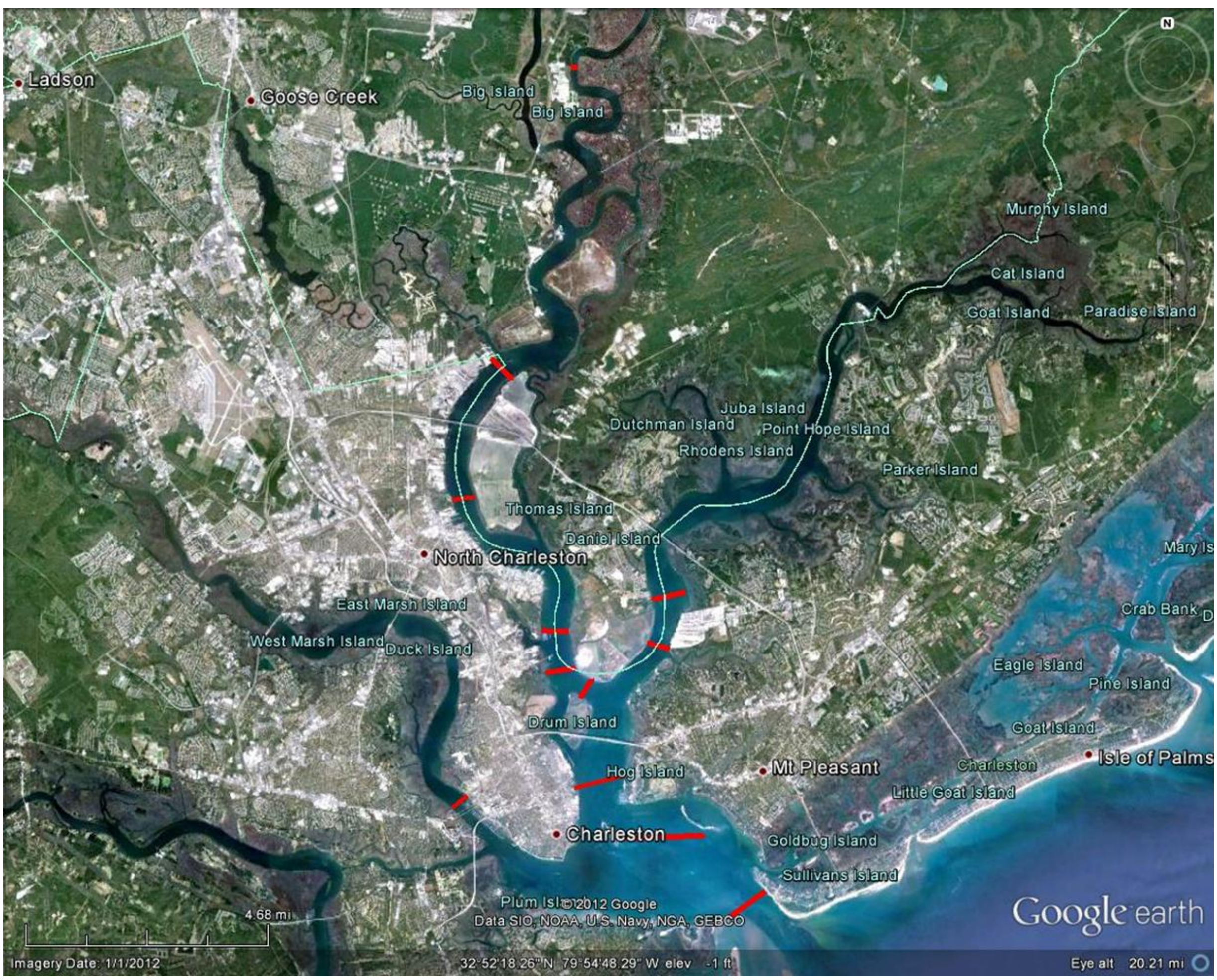


Figure 3.2.3 Location of 2012 ADCP current and flow monitoring transects

3.2.1.10 Suspended Sediments and Bed Sediments

The BCD-COG DO TMDL model did not simulate sediment transport, however the CNCMCT EIS model did. The suspended sediment concentration for the freshwater inflow at the upstream boundary and the suspended sediment concentration for the offshore open boundary were the same values as used in a previous sedimentation model study conducted by the USACE Waterways Experiment Station (WES) that was used to assess sedimentation at the Columbus Street terminal (Teeter et al., 2000).

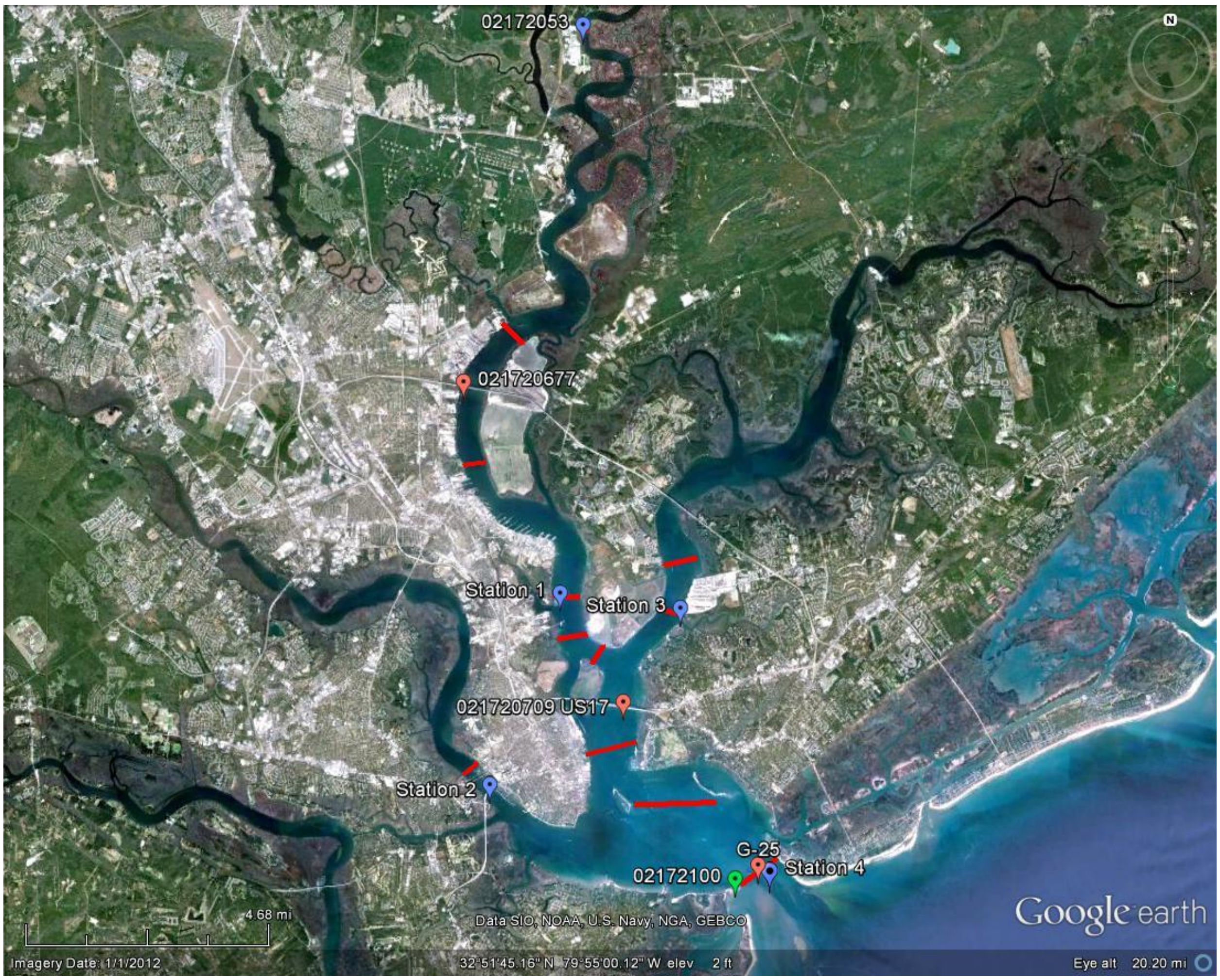


Figure 3.2.4 Locations of index velocity stations equipped with ADVM's (blue icons) and water quality stations (red and green icons).

For the 2012 monitoring, water samples were collected for laboratory analysis of TSS and salinity concentration (Figure 3.2.4). During the ADCP monitoring study, water samples were taken at flood and ebb tide at each transect with 3 vertical samples in the water column (surface, mid-depth, and near-bottom). These samples were collected during both spring and neap tide conditions. Additionally, optical backscatter sensors (OBS) were placed near bottom at the 4 USGS gages along the navigation channel (Fort Sumter- 02172100, Cooper River at

US 17-021720709, Cooper R at I-526 021720677, and Wando R. at I-526 – 021720698) for 60 days. Sources of data for grain size, density (or moisture content), and organic content of the typical sediment bed load into Charleston Harbor include, but are not limited to: a) 2010 report evaluating Lower Harbor and Entrance Channel material for ocean disposal, b) 2004 report evaluating Town Creek and Lower Harbor material for ocean disposal, c) 1994 sediment results used for numerical modeling throughout the channel, and d) 1994 sediment sampling results from the Federal channel (specifically upper harbor).

3.3 Model Development

Tetra Tech LLC was contracted to modify the BCDCOG EFDC model to better represent the federal navigation channel, increase vertical resolution and enable the sediment transport module. The scope included a convergence test to identify the most appropriate level of horizontal and vertical grid resolution in the federal channel and the minimum model time step. Following completion of the convergence testing, a model grid was developed to represent 2012 conditions in the CHS. This grid incorporated the most recent available bathymetric data.

Following development of the model grid, the scope required setup and calibration of the following model components:

 Hydrodynamic module

o 2012 Calibration – Water level, flow, current, salinity and temperature comparison to observed May–June 2012 conditions (measured by the U.S. Geological Survey [USGS] under a previous scope of work)

o 2004 Validation – Water level, flow, current, salinity and temperature comparison to observed April–October 2004 conditions (data collected in support of the BCDCOG model development under a previous scope of work)

 Sediment transport module

o 2012 Calibration – Based on suspended sediment comparisons to observed May–June 2012 conditions (measured by the USGS) and deposition comparisons to long-term maintenance dredging rates in the federal navigation channel

o 2003 Validation – Based on suspended sediment comparisons to observed 2003 conditions (measured by Applied Technology and Management in support of the Environmental Impact Statement for the Charleston Naval Complex Marine Container Terminal)

 Water quality module

o 2004 Calibration Confirmation – comparison to 2004 observed data to confirm that the quality of the model calibration is similar to that completed for the TMDL study (Tetra Tech 2008)

The scope included a sensitivity analysis. This sensitivity analysis demonstrates the model sensitivity to changes in key model variables.

The calibration model was reviewed by agencies and the USACE Jacksonville and Savannah Districts through the USACE Agency Technical Review. Comments received during these reviews were incorporated into the model and the calibration was finalized.

3.3.1 Grid Refinement

Two primary changes were made to the BCDCOG model grid. First, the grid was refined to more accurately represent the federal channel and provide greater resolution across the Cooper and Wando Rivers. Second, the grid bottom elevations were revised using recently collected bathymetry data. Following these revisions, a convergence test of the model grid was completed.

As in the previous BCD-COG model effort, a curvilinear, orthogonal grid was used to approximate the physical dimensions of the meandering estuary system (Tetra Tech 2008). The model grid cells are now aligned along the toe of the federal channel in Charleston Harbor, the Cooper River, the Wando River, and the entrance channel. The model grid was also updated to represent the defined reaches provided by the USACE. To allow for the necessary resolution of currents to support a ship simulation study, the grid includes a minimum of three cells across the federal channel. Additional cells were also added to the grid to represent future widening scenarios on the basis of the data provided by the USACE. These changes increased the total number of horizontal grid cells to 3,648. Also, based on comments received from agency and USACE review, the model grid was adjusted to better match the jetty alignment along the emergent parts of the jetties, and the mask was also adjusted in this area to better represent the jetties. The model grid is shown in Figure 3.3.1. and an example of the refinement is shown on Figure 3.3.1.A.

No changes were made to the BCDCOG model grid in the Ashley River, the Back River, Tidewater Reach, Town Creek Reach, Shipyard River, upstream of the federal channel, or in tidal creeks. Slight modifications were made to the Cooper River upstream of the federal channel and in the Wando River to the east, or upstream, of the federal channel. These changes on the Cooper and Wando Rivers were necessary to transition from the high resolution grid along the federal channel to the lower resolution grid upstream from the federal channel.

The vertical layers of the three-dimensional model are represented by a fixed number of layers for each horizontal grid cell. The vertical thickness of each layer changes dynamically in time, depending on the water depth in the grid cell. This is accomplished by transforming the Cartesian vertical coordinate of the model equations to a time variable stretched vertical coordinate, referred to as the sigma coordinate. Four, six, eight, and ten layers were tested and six vertical layers were selected for the initial model calibration.

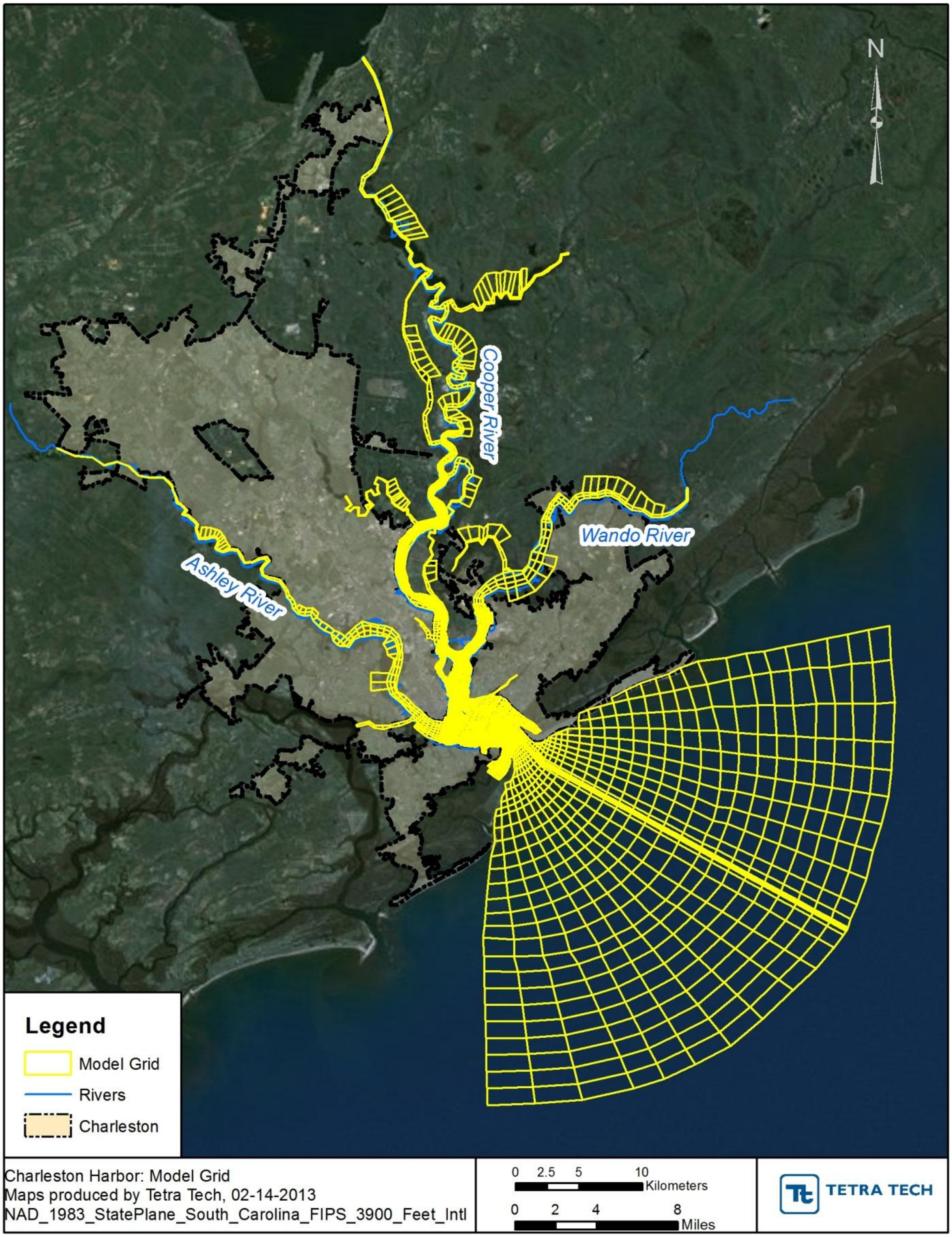


Figure 3.3.1 Overall EFDC Grid

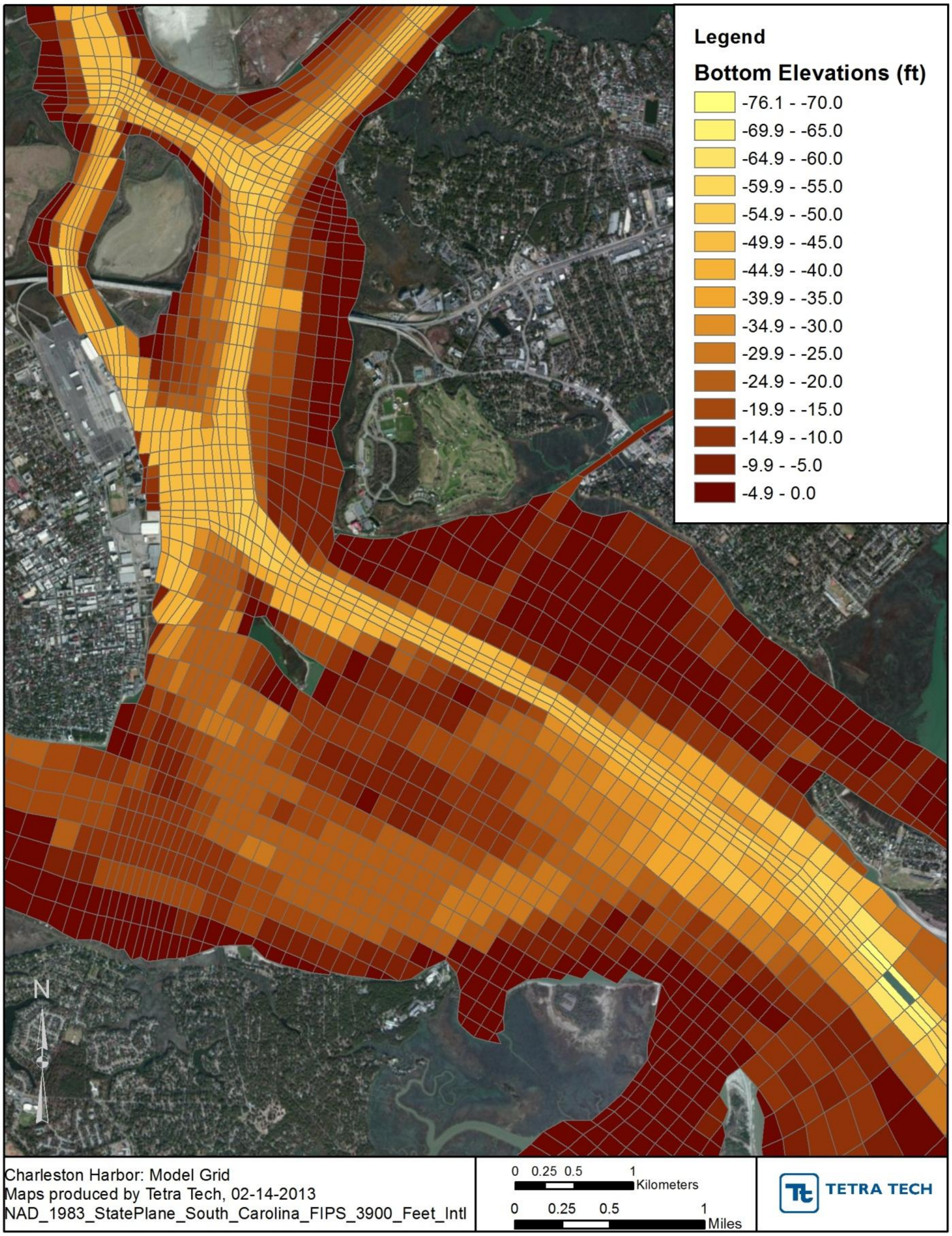


Figure 3.3.1.A Enlargement of EFDC Grid showing refinement of channel

3.3.2 Bathymetry

The existing federal channel includes a 17-mile-long, 47-foot-deep, 800-foot-wide entrance channel. Inside the harbor, the channel transitions to a depth of 45 feet and has varying widths between 500 and 900 feet. These depths are relative to a tidal datum of mean low-low water (MLLW).

USACE provided a compiled bathymetry data set to assign bottom elevations to the horizontal grid cells. This data set was compiled from three sources: USACE Charleston District multibeam surveys from 2011 to 2013 covering the federal channel and the Cooper River upstream of the federal channel; National Oceanic and Atmospheric Administration (NOAA) survey data collected in 2009 covering offshore areas and tributaries; and 2010 bathymetry data collected by Coastal Carolina University. These data encompass the main harbor and the channel areas. In areas outside the coverage of those three data sets, the depths from the previously developed BCDCOG model grid were used (these data are based on historic National Ocean Survey and NOAA survey data).

USACE merged the three individual bathymetry surveys into one data set. Before merging the data, they were converted to a common vertical datum of MLLW with units of feet. Because the USACE and NOAA record water depths as positive integers, the bathymetry data was converted to negative elevation values. In addition, the Coastal Carolina University surveys data were converted from units of meters relative to the NAVD88 vertical datum to units of feet relative to the MLLW tidal datum. A translation of -2.9 feet was added to the elevation value to convert from NAVD88 to MLLW. Overlapping data were removed with preference given to the data set with the most recent data.

The data from the BCDCOG model grid are referenced to mean sea level (MSL) at the Custom House. These elevations were converted to MLLW at the Custom House by adding 2.95 feet to the elevations. The adjusted bathymetry data were then used for interpolating the bottom elevations using the EFDC Grid Editor software.

In response to comments received during model review in March and April 2013, bathymetry in the upper Cooper River was updated with new survey data collected by the USACE in the Cooper River upstream from the federal channel. The survey data was averaged across the river cross-section of the upper Cooper River where the grid is represented by a single grid cell, and the averaged data was interpolated onto the model grid. These changes to upstream bathymetry improved the salinity calibration in the upper Cooper River.

3.3.3 Model Convergence

A convergence test was performed to identify the most appropriate level of horizontal and vertical grid resolution in the federal navigation channel and the minimum model time step (USACE 2012b). Three aspects of the model were tested:

(1) Time step—to ensure that further refinement of the time step size does not significantly affect model results.

(2) Grid resolution—to ensure that the further refinement of the model grid in the federal navigation channel does not significantly affect model current velocity results. This included

a. Evaluation of the number of cells in the horizontal direction

b. Evaluation of the number of cells in the vertical direction

(3) Offshore boundary location—to ensure that the boundary is sufficiently seaward to avoid boundary conditions from influencing the predicted changes in salinity and water quality caused by the proposed channel deepening.

The results of the convergence tests indicate that a 1.5-second time step should be used with three cells in the federal channel. The test results also indicate that the model was not sensitive to the number of vertical layers (with 1 to 2 percent changes between 4 and 10 vertical layers) in the grid, and the offshore boundary should be extended to the end of the federal channel (Tetra Tech 2012). Subsequent model calibration testing found that a 3-second time step would ensure numerical stability in the model.

3.3.4 Marsh Areas

The CHS includes large areas of intertidal marsh. Large intertidal areas affect the magnitude and timing of tidal flows through the main stem tributaries; therefore, they are important to include in the model. The BCDCOG model represented the intertidal marsh areas with model grid cells that are allowed to become wet or dry on the basis of water level elevation in adjacent cells. This approach was initially found to be unstable after refining the model grid in the channel and increasing vertical resolution to 6 layers. Therefore, for the draft model calibration the marsh areas were simulated similar to riverine cells by lowering the marsh elevation to -3 feet MLLW. The marsh cells were assigned a bottom roughness height of 0.04 meters.

For the final model calibration the model stability was improved by low-pass filtering the offshore water level boundary to remove high frequency noise from the observed water level data. Filtering the offshore water surface elevations and implementing horizontal momentum diffusion stabilized the model to allow the wetting and drying of intertidal marsh areas to be used. Therefore, for the final model calibration the marsh elevations in the upper Cooper River were raised so that they are intertidal (with varying elevations between 0 and 2.5 feet MLLW), and the marsh areas on the upper Cooper River were increased to be consistent with the total marsh area shown in the geographic information system (GIS) coverage. The marshes in the lower part of the system, including the Ashley and Wando Rivers, were left at a low elevation (-3 feet MLLW) to avoid affecting the good salinity calibration in the lower estuary.

3.3.5 Boundary Conditions

The boundaries used for the BCDCOG model were extended in time from 2004 through July 2012. Four categories of inputs were used to define the boundaries for this modeling effort:

meteorological inputs, local watershed inputs, offshore inputs, and point source inputs. These inputs and their data sources are described in the following sections.

3.3.5.1 Local Watershed Inputs

The Loading Simulation Program in C++ (LSPC) watershed model was used to establish watershed and overland flows and loads to the CHS. LSPC is a comprehensive data management and modeling system that is capable of representing loading, both flow and water quality, from nonpoint and point sources, and simulating in-stream routing. The LSPC model description and calibration is described in detail by Tetra Tech (2008) and is not repeated here. The LSPC model was reviewed by DHEC during development and was ultimately approved for use by DHEC in TMDL development.

Meteorological inputs to the LSPC were extended to provide daily average freshwater flows from the watershed to the EFDC model for the period from December 1995 through July 2012. The sensitivity of salinity to watershed flows, and other model inputs, is described later in Section 8.0. The freshwater inflows in the model were assigned a constant salinity concentration of 0.05 ppt to account for the conductivity observed in freshwater areas.

Water temperatures from tributaries were not simulated using LSPC. Instead, daily average water temperature was calculated from 7 USGS stations (02172020, 02172040, 02172050, 02172053, 02172067.7, 02172069.8, and 02172070.9). A single daily average temperature was used for all the watershed inputs.

Land use characteristics input to the LSPC model were based on the 2001 USGS Multi-Resolution Land Characteristics (MRLC) data set. Although the BCDCOG model uses the 2001 MRLC data set, the most recent data set available is the 2006 MRLC. The difference in stormwater runoff volumes and watershed pollutant loads between the 2001 and 2006 land uses is assumed to have a minimal effect on the prediction of in-stream water quality for two reasons: (1) stormwater management practices employed during this period were intended to mitigate changes in land use from new development; and (2) the prediction of Cooper River and Wando River DO by the BCDCOG DO TMDL model was found to be insensitive to changes in watershed loads. In regard to the first item, South Carolina’s Sediment, Erosion, and Stormwater Management Program has been in existence since 1991. This program requires that anyone doing any construction or land-disturbing activity in South Carolina must apply for a stormwater management and sediment control permit from SCDHEC. SCDHEC is charged with making sure that stormwater runoff during and after construction projects will not have an adverse effect on water quality in South Carolina. The stormwater program requires developing and implementing a plan to control stormwater runoff and sediment to prevent them from entering water bodies in the state. Given the stormwater program’s requirements for stormwater management, it is reasonable to assume that the change in stormwater runoff loads between the 2001 and 2006 land uses will have a minimal effect on the model prediction of in-stream water quality in the CHS. This assumption is supported by the fact that sensitivity testing by Tetra Tech (2008) found that the model prediction of DO in the Cooper River and the Wando River was not sensitive to ±20 percent changes in nonpoint source loads.

Therefore, the results of the LSPC model that were used in the BCDCOG DO TMDL modeling effort for the CHS are still appropriate for modeling water quality in the CHS, and this study will also use the LSPC model results for input to the EFDC model. A comparison of land use changes between the 2001 and 2006 datasets found the greatest subwatershed change in the Wando River watershed, where forested area had decreased by 5.3 percent. These changes are small as compared to the variations tested in the sensitivity analysis (±20 percent). For this reason, updating the watershed model on the basis of the 2006 data would not have a significant effect on the model calibration.

DO boundary conditions for the freshwater boundaries are based on continuous measurements of temperature and DO. For Pinopolis Dam and upstream Ashley River percentages of saturation of 85 and 40 percent, respectively, were determined from measurements. DO boundary conditions were established using the computed percentage of saturation (based on temperature) for periods when DO measurement data are not available. Other freshwater DO boundary conditions were established using temperature at those locations and a percentage of saturation of 80 percent. For other water quality variables (including ammonia [NH3], nitrate [NO3], dissolved organic nitrogen [DON], total organic carbon [TOC], dissolved organic phosphorus [DOP], and orthophosphate [PO4]), boundaries are set at constant concentrations, listed in Table 3.3.5.1. These values were applied to daily average flows to define loads for the 2004 water quality validation of the EFDC model. They remain unchanged from those values used for the BCDCOG model study (Tetra Tech 2008).

Table 3.3.5.1 Water Quality model watershed inflow boundary concentrations (mg/L).

Boundary input	Ashley River	Pinopolis Dam	Goose Creek	Wando River	East Branch of the Cooper River	Elliot's Cut	Cooper River at Wadboo Swamp
NH_3	0.015	0.025	0.060	0.025	0.025	0.025	0.025
NO_3	0.02	0.02	0.02	0.02	0.02	0.01	0.02
TOC	24	4.4	11	8.0	8.0	0.5	4.4
DON	1.00	0.350	0.730	0.470	0.470	0.375	0.350
DOP	0.060	0.015	0.050	0.006	0.006	0.246	0.015
PO_4	0.150	0.035	0.119	0.014	0.014	0.020	0.035

3.3.5.2 Meteorological Inputs

Hydrologic conditions depend on precipitation events and evapotranspiration in the watershed. Daily meteorological records from the Charleston International Airport (WBAN 13880) were used to define hydrologic conditions in the CHS. Precipitation records from 1980, along with maximum and minimum temperatures for the same period, were collected from the National Climatic Data Center and input to the LSPC watershed model over the watershed area. Several precipitation stations were available in the CHS; but from data analysis, the airport records were found to be the most complete. Therefore, only Charleston International Airport data were used for the meteorological inputs. The inputs initially developed for the BCDCOG model were extended to include data through July 2012. The LSCP model was then used to simulate January 1, 2003, through July 31, 2012, to provide the watershed inputs to the EFDC model.

The EFDC model also requires direct input of meteorological conditions: wind speed, wind direction, atmospheric pressure, air temperature, relative humidity, rainfall, evaporation, solar radiation, and cloud cover. These inputs were also generated using hourly data from the Charleston International Airport.

3.3.5.3 Offshore Open Boundary Inputs

3.3.5.3.1 Water Levels

The water surface elevations measured by NOAA at a gage installed at Custom House on the Cooper River (Station ID 8665530) were used to develop the model offshore tidal boundary conditions. The 6-minute interval time series of water levels are referenced to the MLLW vertical datum. A 2-hour low-pass filter was applied to the water level data from Station 8665530 to remove high frequency noise. Because the observed water level data were not measured at the offshore boundary location (roughly 18 miles offshore in the Atlantic Ocean), the input water levels cannot be specified by direct measurements and must be determined as part of the model calibration process.

Additionally, the EFDC model for the CHS employs a radiation-separation condition at the open boundary. This allows transients generated inside the model domain to be transmitted outward (rather than reflected from the open boundary, which occurs with an elevation specified boundary condition). Using this type boundary condition also requires iterative model simulations to determine the appropriate input boundary conditions.

By trial and error, the water surface elevation, including its phase and amplitude, at the open boundary was adjusted such that the simulated water surface elevation at Custom House matched the measured data. The adjusted water surface elevation at the open boundary for 2012 is shown in Figure 3.3.2. The same process was used to determine the boundary conditions for 2004, shown in Figure 3.3.3.

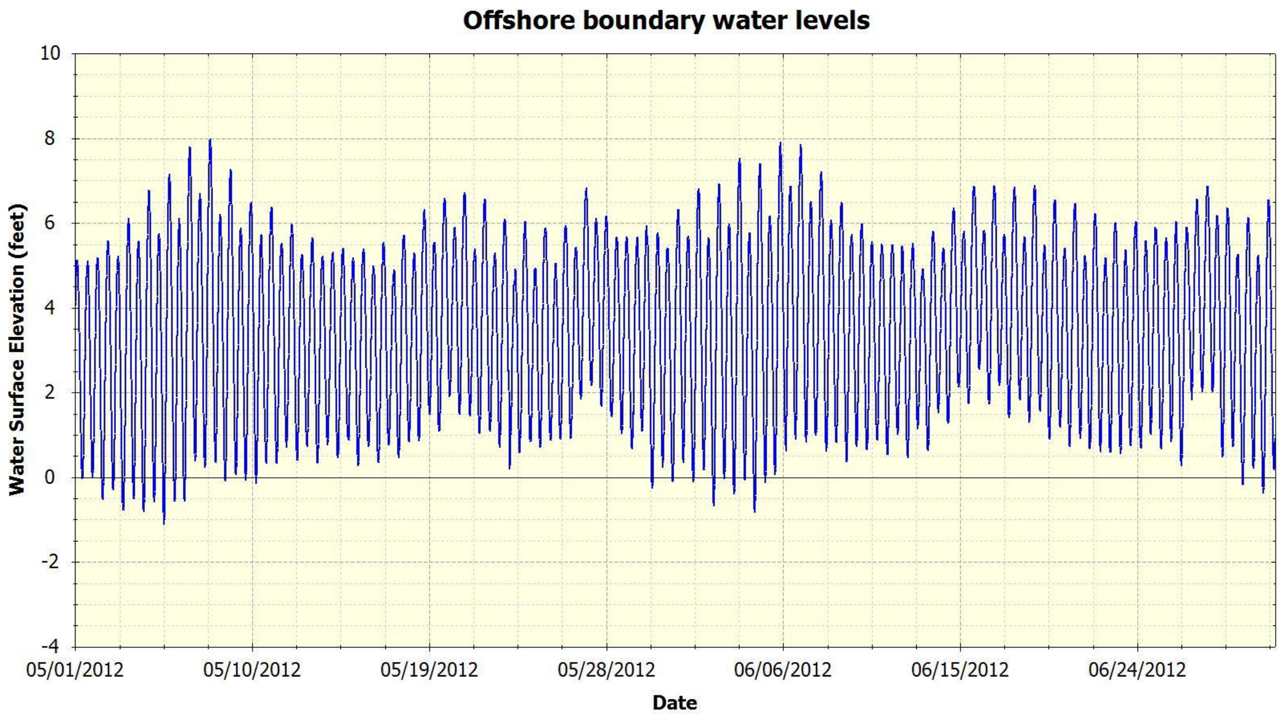


Figure 3.3.2. Offshore boundary water levels for 2012 simulation

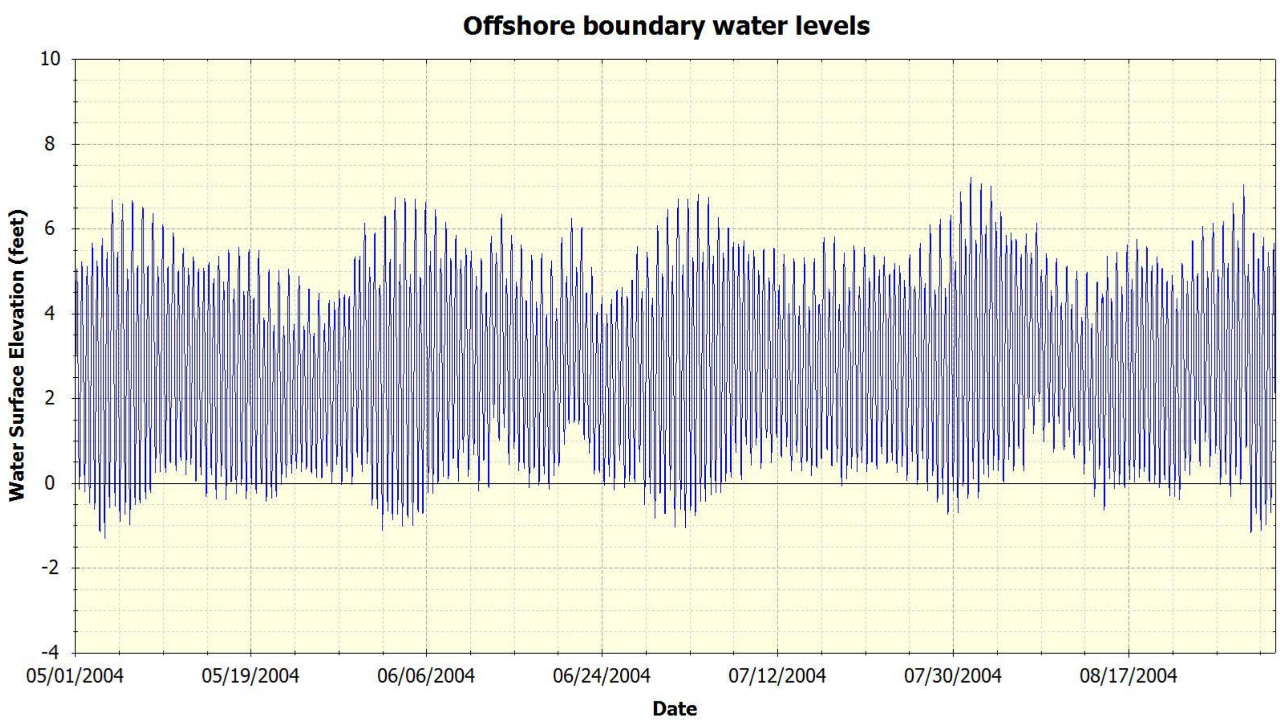


Figure 3.3.3. Offshore boundary water levels for 2004 simulation

3.3.5.3.2 Salinity

The open boundary salinity concentration for 2012 is based on measured data from the Carolinas Regional Coastal Ocean Observing System. The data from a buoy about 5 miles offshore from Capers Island (Station CAP2) was used as the basis for developing the model boundary concentrations, shown in Figure 3.3.4. Average offshore salinity in South Carolina has been documented at 36.6 ppt (Laurie and Chamberlain 2003). Average salinity at Station CAP2 for the 2012 time period was 34.11 ppt. A constant increase of 0.5 ppt was added to the buoy time series data to create the boundary salinity for the 2012 calibration simulation (an increase determined iteratively during the model calibration process). Also, a multiplier was applied to the boundary salinity for each layer, as follows (from the bottom layer to the surface layer): 1.075, 1.075, 1.05, 1.025, 1.0, and 1.0.

The observed Station CAP2 data used for 2012 were collected from 2005 through 2012. Therefore, measured offshore data are not available for the 2004 period. When observed data was not available at Station CAP2, for 2004, a constant salinity was used for the boundary (33.5 ppt at the surface and 34.5 ppt at the bottom) based on the BCDCOG model.

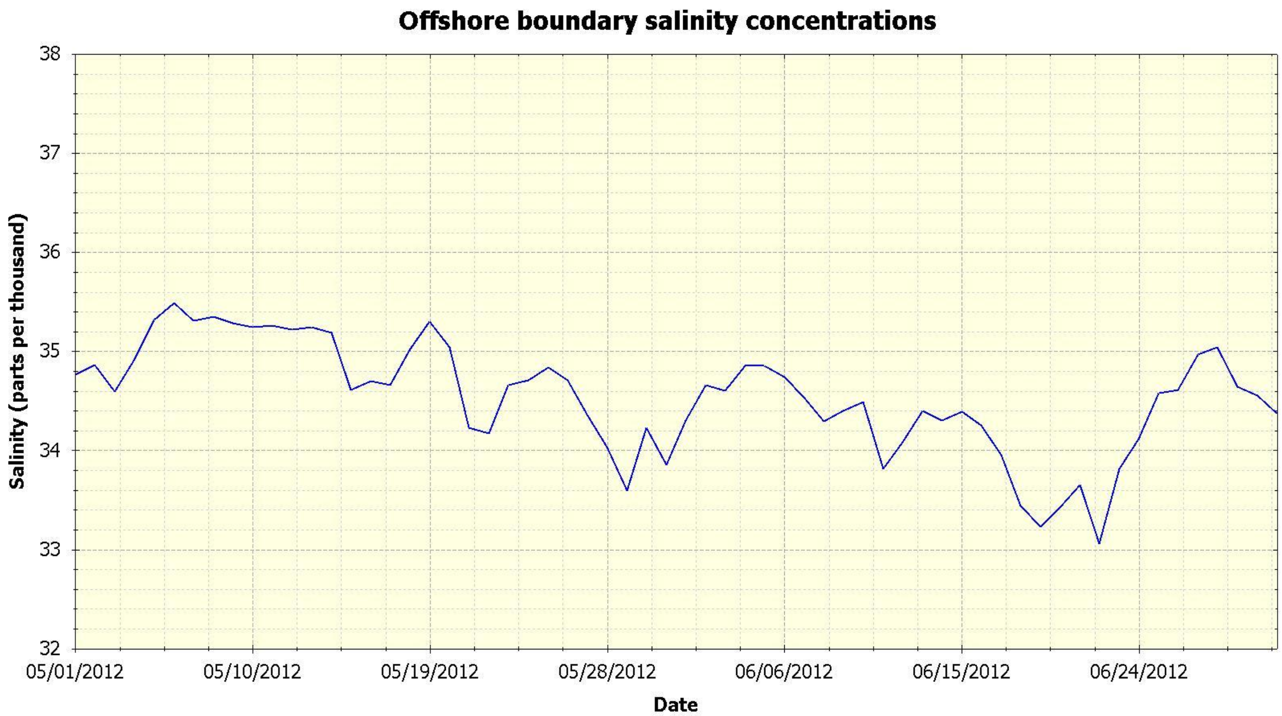


Figure 3.3.4. Offshore surface boundary salinity concentrations for 2012 simulation

3.3.5.3.3 Temperature

The open boundary temperature for 2012 is also based on measured data from the Carolinas Regional Coastal Ocean Observing System. The daily averaged data from buoy CAP2 was used for the model boundary temperatures, shown in Figure 3.3.4. The water temperature illustrated in Figure 3.3.5 was applied uniformly in the water column. The 2004 temperature boundary conditions, shown in Figure 3.3.6, were developed as part of the BCDCOG model calibration by iteratively adjusting the boundary until achieving good agreement between simulated and observed temperatures at the Hwy 17 bridge on the Cooper River.

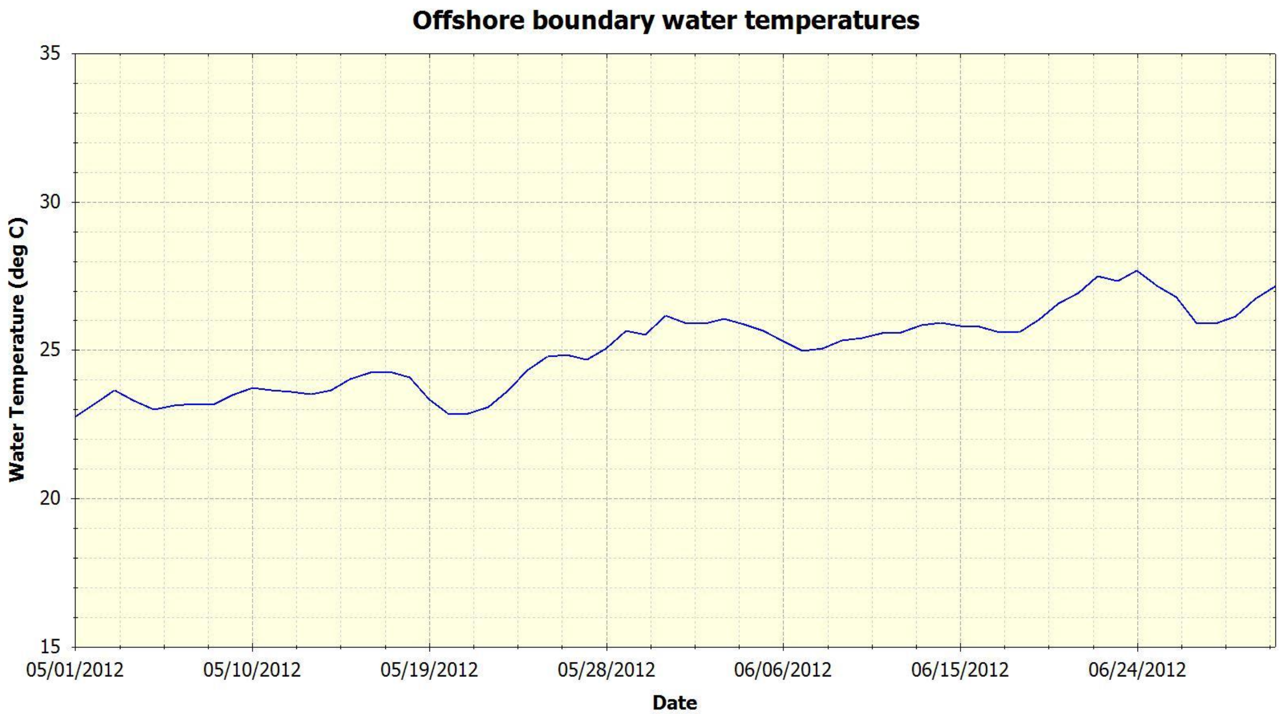


Figure 3.3.5. Offshore average boundary temperatures for 2012 simulation

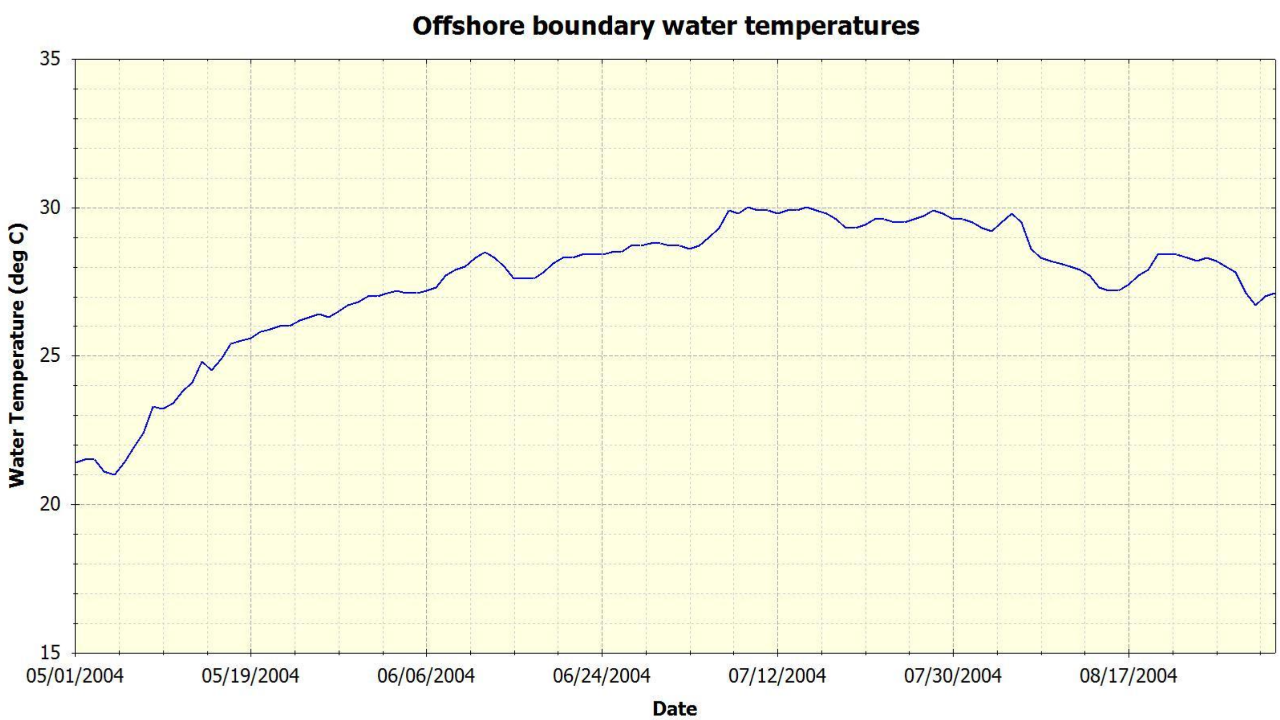


Figure 3.3.6. Offshore average boundary temperatures for 2004 simulation

3.3.5.3.4 Water Quality

The open boundary percentage of saturation of 85% was considered and DO boundary conditions calculated based on salinity and temperature input as boundary conditions. Other freshwater DO boundary conditions were established based on temperature at those locations and a percentage of saturation of 80%. The approach described here is consistent with the TMDL model. The percentage of DO saturation applied at the offshore boundary in the TMDL model was based on limited measurements from offshore sampling and default values for background DO saturation used by EPA for TMDL studies. The same values have been used in nearby model studies, such as the model study completed for the Savannah Harbor TMDL and the Savannah Harbor Expansion Project.

The upstream DO boundary at the Ashley River was increased from 45% (in the BCDCOG model) to 65% of DO saturation. This change was made to address comments regarding the simulated low DO effect cause by high freshwater flows that cause the model to stratify in the upper Ashley River. In addition to changes in the DO boundary, the upper Ashley River benthic zone was extended down to I-526, effectively reducing the sediment oxygen demand (SOD) in this area from 1.2 to 0.7 $g/m^2/d$. This area of concern is far removed from the area of interest for the Post 45 Study such that it would not be expected to affect the ability of the model to assess potential deepening impacts on DO.

For other variables, open water boundaries were set at constant concentrations, listed in Table 3.3.5.2., that are based on similar systems in the region (Tetra Tech 2008).

Table 3.3.5.2. Water quality model offshore boundary concentrations (mg/L)

Variable	Open boundary concentration
NH_3	0.16
NO_3	0.01
TOC	0.50
DON	0.33
DOP	0.03
PO_4	0.02

3.3.5.4 Pinopolis Dam Freshwater Inputs

The USGS has measured discharge data from the Pinopolis Dam (USGS gage 02172002 in the tailrace canal below Lake Moultrie) dating back to 1978. The model boundary flow into the tailrace canal at the upstream end of the West Branch of the Cooper River is specified according to this measured data collected by the USGS and the Jefferies Powerhouse, as shown in Figure 3.3.7. Daily averaged water temperature from the USGS gage was also specified at the model boundary.

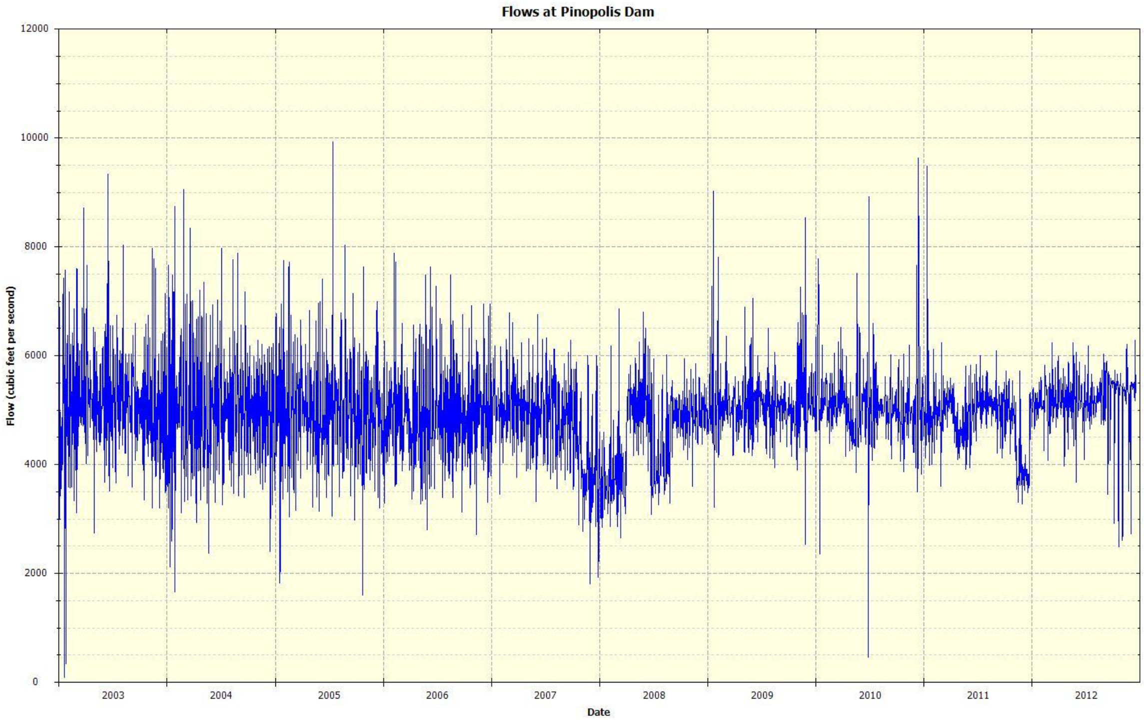


Figure 3.3.7. Flows at Pinopolis Dam for 2002 – 2012

The hourly freshwater flow measurements from the Pinopolis Dam (USGS gage 02172002 in the tailrace canal below Lake Moultrie) were used for the model calibration period. During calibration these data were compared with hourly flows calculated from the Jefferies Power Station at Pinopolis Dam. A comparison of hourly flows between USGS measured data and the flows reported by Jefferies Power Station illustrated differences between 12 to 25%. A Waterways Experiment Station study of leakage at Pinopolis Dam found the average leakage is about 305 cfs. Most of this leakage appears to occur through the turbines while they are shut down. Because this leakage flow is unaccounted for in the generation flow reports from Jefferies Power Station and occurs only in periods when power generation is not occurring, it is assumed that the leakage flow is about 150 cfs on average. This accounts for about a 3 percent increase in flow from the Pinopolis Dam. The boundary flow in the final model calibration is based on the USGS data minus 13 percent to account for the difference between the Santee Cooper reported generation flows (plus the leakage flow), and the USGS flows. Similar to the watershed inflows; concentrations of salinity from Pinopolis Dam were assumed to be zero ppt. This is important when comparing model (or simulated) results with measured (observed) data upstream of Mobay, in areas of the estuary with salinity concentrations less than 1.0 ppt.

3.3.5.5 Point Source Inputs

The point source inputs include 18 National Pollutant Discharge Elimination System (NPDES) permitted discharges (Table 3.3.5.3), consistent with facilities considered in the TMDL model. These discharges were input to the model based on monthly discharge monitoring reports (DMRs). The hydrodynamic model includes only the 12 discharges with average flows greater than 1 million gallons per day (mgd). The peak tidal flow rates at Mobay are greater than 60,000 cfs. The net downriver flow from the Pinopolis Dam averages 4,500 cfs. By comparison, the aggregate flow from other discharges is less than 12 cfs. Given the difference in magnitude of flow rates, flows from discharges less than 1 mgd will have an insignificant effect on the system hydrodynamics. The water quality model includes constituent mass loading for all 18 discharges.

Generally, NPDES permitted facilities in the CHS are required to monitor and report only 5-day biochemical oxygen demand (BOD_5) and ammonia from their effluent discharge. Daily measured concentrations and reported monthly average loads for 2004 were used to develop the point source load files for the model where available; otherwise, maximum permitted values were used. Average model loads for the NPDES facilities are listed in Table 3.3.5.5.1. The BOD_5 loads were converted to carbonaceous BOD_5 ($CBOD_5$) using a conversion ratio of 2.67 (the ratio of mass of oxygen consumed per mass of carbon assimilated). $CBOD_5$ was converted to ultimate CBOD ($CBOD_u$) f-ratios calculated from long-term BOD (LTBOD) sampling and analysis for the NPDES dischargers (these calculations are documented in Appendix A of the BCDCOG model report [Tetra Tech 2008]). For those NPDES dischargers that collected no LTBOD data in 2004 (those with average flows less than 1 mgd), f-ratios previously calculated by SCDHEC were used.

The loads for the 12 largest dischargers are shown in Appendix F of the BCDCOG model report (Tetra Tech 2008). For all NPDES discharges, only loads for organic carbon, ammonia, and DO are included. Partitioning of the BOD between labile and refractory components is based on the results of the LTBOD sampling and analysis. For the Summerville and Lower Berkeley facilities, an f-ratio of 3.8 was used on the basis of an average of similar facilities in the CHS.

Table 3.3.5.3 NPDES Point Source Discharges in the Hydrodynamic and Water Quality Model.

Permit name	NPDES number	Stream	Model grid* I	Model grid* J	Avg flow (mgd)	f-ratio	Avg $CBOD_u$ (lbs/day)	Avg NH_3 (lbs/day)	Avg DO (lbs/day)
Mead Westvaco	SC0001759	Cooper	56	79	27.12	8.7	22,784	110.55	1,218
North Charleston Sewer District	SC0024783	Cooper	55	52	12.82	4.43	2,858	895.66	716
Plum Island WWTP	SC0021229	Cooper	71	14	19.96	3.2	3,269	258.11	1,003
Lower BCW&SA	SC0046060	Cooper	62	110	7.27	3.8	1,766	131.63	459
Mount Pleasant	SC0040771	Cooper	107	32	4.39	3.2	464	225.35	198
Bayer	SC0003441	Cooper	65	127	2.28	4	529	101.12	125
Summerville	SC0037541	Ashley	44	67	5.37	3.8	488	7.32	386
BP (Amoco)	SC0028584	Cooper	64	115	4.22	6.9	1,437	12	214
Moncks Corner WWTF	SC0021598	West Cooper	20	167	1.04	4.1	970	131.97	42
Lower Dorchester	SC0038822	Ashley	44	52	1.76	2.2	95	1.69	99
DAK (Dupont)	SC0026506	Cooper	65	157	0.87	7	546	5.49	36
Williams Station Steam Plant1	SC0003883	Cooper	65	142	420	NA	18,940	0	10,230
Central BCW&SA	SC0039764	West Cooper	30	167	0.35	1.52	131	58	--
King's Grant	SC0021911	Ashley	44	63	0.238	1.52	89	40	--
CPW Daniel Island	SC0047074	Cooper	63	63	0.5	1.52	31	4	--
Middleton Inn	SC0039063	Ashley	44	49	0.014	1.52	5	2	--
CR Bard	SC0035190	Tail Race	9	167	0.192	1.52	41	0	--
Teal	SC0030350	Ashley	44	78	0.03	1.52	4	2	--

Note: * The listed I and J values are the indices of the model grid cells where these discharges are located

3.3.5.6 Marsh Loads and Sediment Oxygen Demand

As described in the BCDCOG model report (Tetra Tech 2008), the exchange of nutrients between marshes and open water of the CHS is based on 2004 field studies conducted by Dr. Dan Tufford of the University of South Carolina. Dr. Tufford sampled flood and ebb flows from two historic rice fields on the Cooper River, Mulberry Plantation, and Dean's Hall as part of a study to evaluate the nutrient and sediment fluxes from freshwater marshes to the Cooper River. The calibrated water quality model uses a TOC load of 7.73 pounds per acre per day, with 80 percent of the TOC split into the refractory TOC variable, and 20 percent of the TOC split into the labile TOC variable consistent with the TMDL model.

Agency review of the initial draft of this report questioned whether different types of marsh vegetation (e.g., salt water marsh versus freshwater marsh) are important to consider in determining the appropriate marsh loading rates. The relative importance of this variable was evaluated based on the previously completed sensitivity analysis for the BCDCOG TMDL model. The sensitivity analysis of the BCDCOG TMDL model indicates that ±20 percent variations in the marsh loading rates caused only –0.04 and +0.03 mg/L changes in 50th percentile DO in the Cooper River. This indicates that the model is not sensitive to changes in the marsh loading rates and because of this, it is reasonable to assume that there is not a large enough difference in export of TOC between salt and freshwater marshes to have a significant effect on the model calibration.

In addition to the loading rates, the marsh areas also include a uniform SOD rate. This rate was adjusted from that used in the BCDCOG TMDL model in order to improve the DO predictions. The marsh SOD was reduced from 1.2 to 0.7 $g/m^2/d$. This is within the range of values typical for SOD in estuaries. SOD rates for sandy bottom range between 0.2 and 1 $g/m^2/d$, and SOD rates for estuarine range between 1 and 2 $g/m^2/d$ (Chapra 1997). Also, intertidal areas exposed to light and air can have very small SOD rates. The marsh cells in the model are large areas that aggregate a range of bottom types that will exert different SOD rates, including tidal creeks with bottom elevations below MLLW, as well as intertidal marsh areas that become dry every tidal cycle. As a result, the marsh SOD in the model is a calibration parameter that is determined through iterative adjustment and evaluation of the model results. The calibrated value of 0.7 $g/m^2/d$ is within the range of SOD values in the literature.

3.4. Hydrodynamic Model Calibration and Validation

The calibration methodology for the CHS EFDC model included graphical time series comparisons (qualitative) and statistical calculations (quantitative). The calibration methodology was parameter specific, starting with the following:

- Water surface elevation
- Currents/flows
- Temperature
- Salinity

Each of these parameters has its importance in the determination of success for the model calibration and confirmation. The order in which the hydrodynamic model is calibrated is performed to address issues such as bathymetry, friction, tidal volume, cross-sectional area, and heat budget before salinity is calibrated. Salinity is the predominant signal in the model to ensure that mass is being moved horizontally and vertically with the appropriate timing and direction.

WRDB Graph was used to plot time series of the EFDC model output files (*.BMD) and the measured data files (*.DB) (Wilson 2013). Time series graphical comparisons are useful to visualize key trends in the data compared to that of the model. Water levels were compared to USGS data to determine if the spring and neap variations in tidal amplitudes are appropriately simulated. Seasonal fluctuations of temperature and salinity were compared to the continuous USGS data to determine if the model is simulating these variables appropriately.

A variety of model fit statistics are available for evaluating model performance (Tetra Tech 2006). WRDB Graph was also used to perform the statistical calculations using only interpolated simulated values at the measured points. For the statistical evaluations, in addition to 5th, 50th and 95th percentiles, the following calculations were generated:

Mean error: $ME = \overline{P} - \overline{O}$

Absolute mean error: $MEA = \frac{\sum_{i=1}^{n}|P_i - O_i|}{n}$

RMS error: $RMSE = \sqrt{\frac{\sum_{i=1}^{n}(P_i - O_i)^2}{n}}$

Normalized RMS error: Norm $RMSE = \frac{RMSE}{\sqrt{\frac{\sum PO}{n}}}$

Coefficient of determination: $R^2 = \frac{(n\sum_{i=1}^{n}(P_i - O_i)) - (\sum_{i=1}^{n} O_i \times \sum_{i=1}^{n} P_i)}{\sqrt{(n\sum_{i=1}^{n} P_i) - (\sum_{i=1}^{n} P_i)^2} \times \sqrt{(n\sum_{i=1}^{n} O_i) - (\sum_{i=1}^{n} O_i)^2}}$

Index of Agreement: $Index\ of\ Agrmt = 1 - \frac{\sum_{i=1}^{n}(O_i - P_i)^2}{\sum_{i=1}^{n}(|P_i - \bar{O}| + |O_i - \bar{O}|)^2}$

where P is the predicted value, O is the observed value and n is the number of data points.

The coefficient of determination, R^2, measures the tendency of the predicted and observed values to vary together linearly. It can range from –1 to 1, with negative values indicating that the observed and predicted tend to vary inversely. It should be recognized that even if the correlation is close to 1, the predicted and observed might not match each other; they only tend to vary similarly (Stow et al. 2003).

The root mean squared error, mean error, and absolute mean error are all measures of the size of the discrepancies between predicted and observed values. Values near zero indicate a close match. The mean error is a measure of aggregate model bias, although values near zero can be misleading because negative and positive discrepancies can cancel each other. The absolute mean error and the root mean squared error both accommodate the shortcoming of the average error by considering the magnitude rather than the direction of each discrepancy. The root mean square error can be normalized to compare model performance between locations. The index of agreement is a standardized measure of the degree of simulation error with 1 being a perfect match. Together, these five statistics provide an indication of model prediction accuracy (Stow et al. 2003, Tetra Tech 2006).

3.4.1 2012 Model Calibration

The hydrodynamic model simulated the April through June 2012 period for the model calibration. Measured data collected by the USGS during this period were used for the calibration comparisons. Comparisons to water levels, flows, currents, salinity, and temperature are provided below.

3.4.1.1 Water Levels

The model calibration includes comparison to the 11 USGS water level gages listed in Table 3.4.1 and shown in Figure 3.4.1. The grid location (I,J) and river mile where measured data was compared with model output is also provided in Table 3.4.1.1. As mentioned in Section 3.3.5.3, the offshore open-boundary water levels were adjusted to obtain good agreement between the simulated and observed values at the Custom House gage.

Plots of simulated and observed water level time series are provided in Attachment A-1 (Figures A-1 through A-9). Comparison statistics are listed in Table 3.4.1.2. To visualize a summary of the model performance along the length of the Cooper Rivers, the percentiles from Table 3.4.1.2 are plotted versus river mile in Figure 3.4.2. Figure 3.4.3 presents the water surface elevation percentiles from the harbor up the Wando River. These figures show that the model is accurately simulating the overall tide range from Fort Sumter up the Cooper and Wando Rivers.

As would be expected, the agreement of simulated and observed values is best at locations in the harbor with increased variability moving up the Wando and Cooper Rivers, although a good quality of fit is attained throughout. The range of water surface elevation in areas upstream on the Wando and Cooper Rivers is influenced by a number of factors including channel geometry and the extent and depth of marshes. Marshes in the headwaters act similar to sponges, absorbing and dampening the effect of fluctuating water levels. These reasons account for differences between simulated and observed values in the West Branch of the Cooper River and near Goose Creek (river miles 57.16 and 50.57 respectively). In meandering, riverine sections of the Cooper River, channel geometry in the model restricts the range of fluctuations in water level. This explains why the greatest variability between simulated and observed water surface elevations is seen at Mobay (river mile 41.04). Adjustments were made to the bathymetry in the upper Cooper River based on comments received in March and April 2013 to decrease the effects of these factors and improve the calibration of water levels. The results presented here incorporate those changes described in previous sections of this report.

Table 3.4.1.1. 2012 Water Level Gage Locations

USGS station ID	Description	River mile	I	J
02172020	West Branch Cooper River at Pimlico near Moncks Corner	57.16	46	167
02172050	Cooper River near Goose Creek	50.57	65	161
02172053	Cooper River at Mobay	41.04	65	128
2172067.7	Cooper River at I-526	31.00	56	79
2172069.8	Wando River above Mount Pleasant (I-526)	27.84	75	68
2172070.9	Cooper River at Hwy 17	23.48	81	38
2172071.1	Cooper River at Custom House (AUX)	21.78	70	21
2172086.9	Ashley River near North Charleston (I-526)	32.77	44	17
2172100	Charleston Harbor at Fort Sumter	18.94	115	19
Site 4	Bottom mounted ADVM at Mouth to Charleston Harbor	18.84	117	23

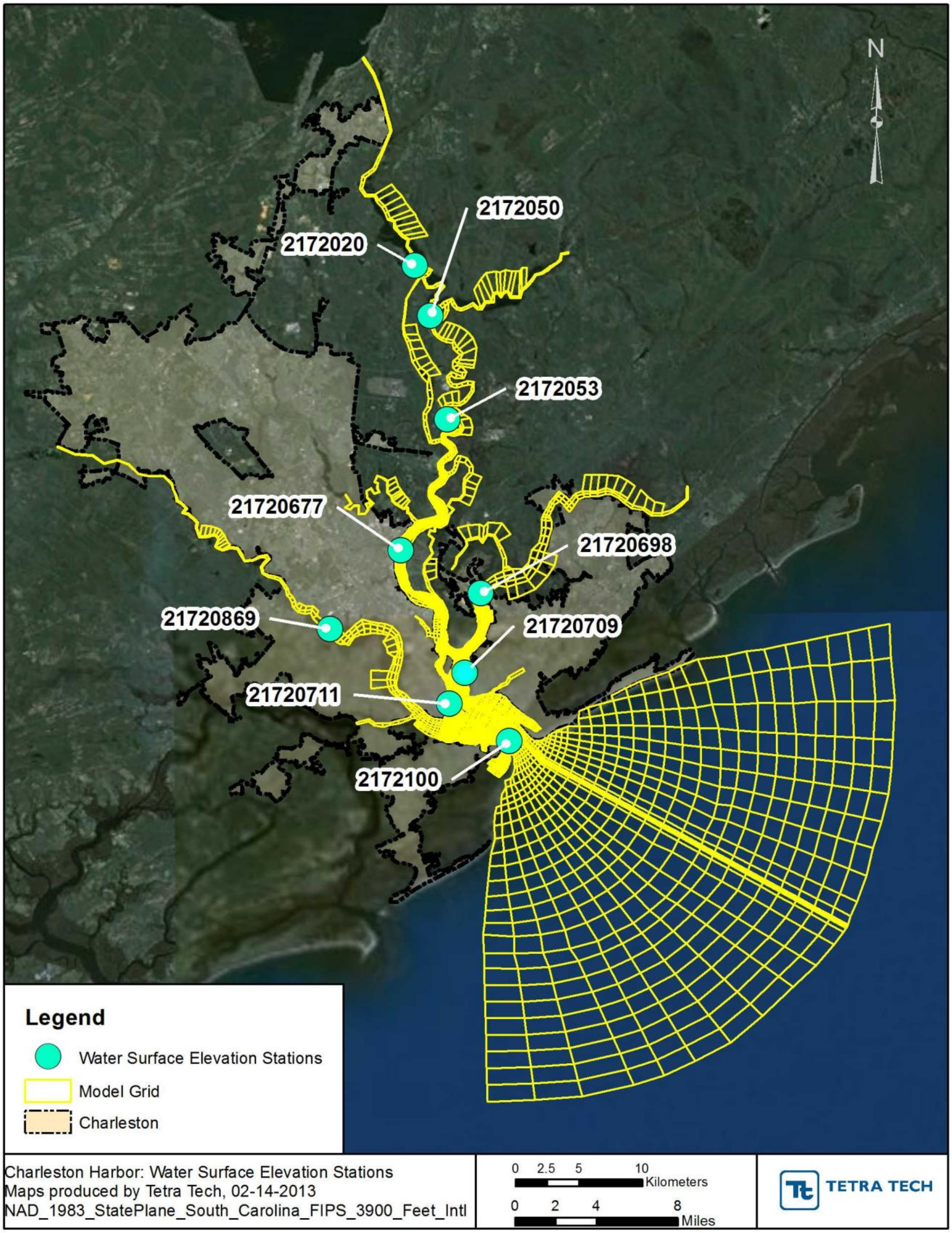


Figure 3.4.1 Water surface elevation gage locations for 2012

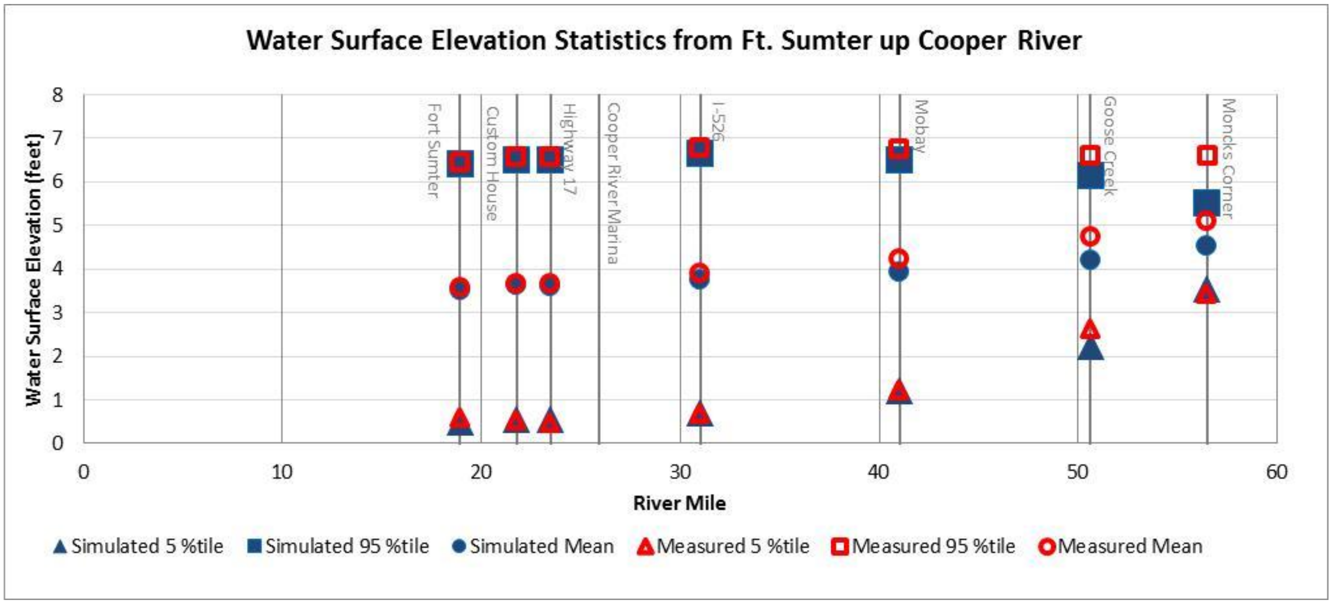


Figure 3.4.2. Longitudinal profile of 2012 water surface elevation (feet, MLLW at Customs House) statistics from Fort Sumter to Pinopolis Dam

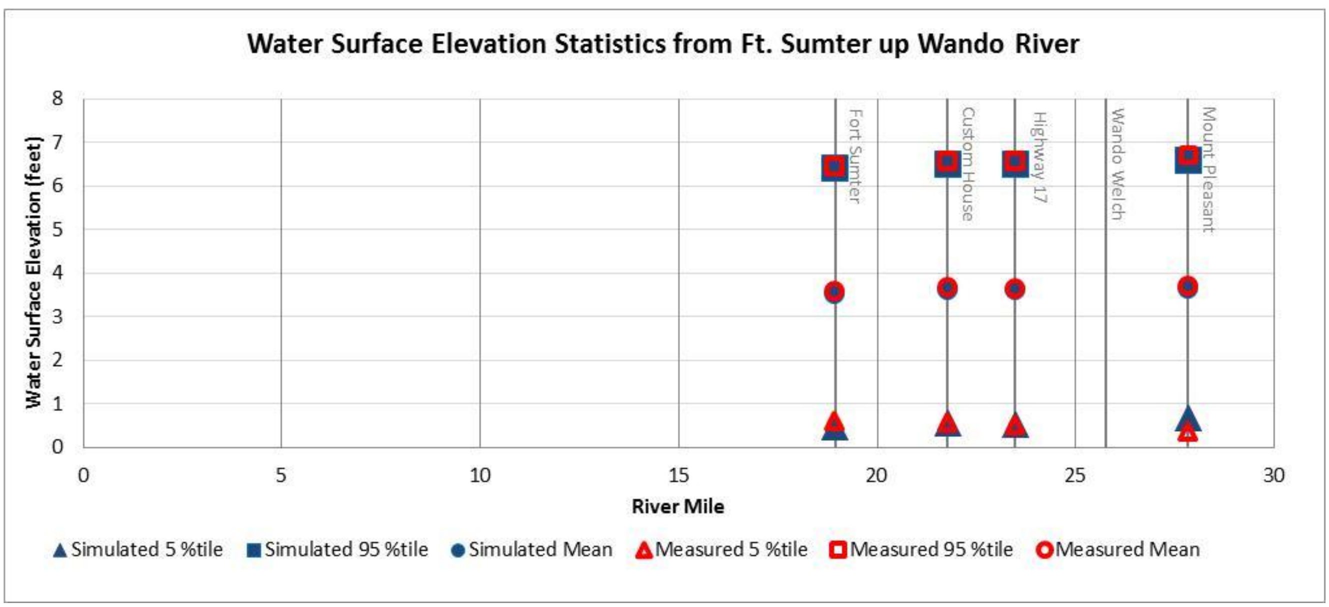


Figure 3.4.3. Longitudinal profile of 2012 water surface elevation (feet, MLLW at Customs House) statistics from Charleston Harbor up Wando River

Table 3.4.1.2. 2012 Water surface elevation (feet, MLLW at Customs House) comparison statistics for the period from 5/01/2012 – 07/01/2012

Station	River Mile	Measured			Simulated			Difference			R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
		Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile					
USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC	57.16	5.09	3.46	6.60	4.51	3.54	5.51	-0.58	0.08	-1.09	0.87	0.62	0.73	0.15	0.81
USGS 02172050 Cooper River near Goose Creek, SC	50.57	4.72	2.63	6.58	4.19	2.22	6.13	-0.54	-0.41	-0.45	0.96	0.54	0.60	0.13	0.95
USGS 02172053 Cooper River at Mobay	41.04	4.22	1.25	6.74	3.92	1.21	6.50	-0.30	-0.04	-0.24	0.95	0.40	0.50	0.11	0.98
USGS 02172067.7 Cooper River at I-526	31.00	3.90	0.69	6.76	3.75	0.71	6.64	-0.15	0.03	-0.12	0.97	0.30	0.37	0.09	0.99
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3.69	0.37	6.71	3.64	0.67	6.60	-0.05	0.30	-0.10	0.96	0.32	0.41	0.10	0.99
USGS 02172070.9 Cooper River at Hwy 17	23.48	3.65	0.50	6.57	3.60	0.54	6.49	-0.06	0.04	-0.07	0.99	0.15	0.19	0.05	1.0
USGS 02172071.1 Cooper River at Custom House (AUX)	21.78	3.66	0.55	6.57	3.62	0.56	6.51	-0.05	0.01	-0.07	0.99	0.12	0.15	0.04	1.0
USGS 02172086.9 Ashley River near North Charleston	32.77	3.69	0.36	6.61	3.66	0.32	6.72	-0.03	-0.04	0.11	0.98	0.22	0.27	0.06	1.0
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	3.58	0.60	6.44	3.51	0.48	6.40	-0.07	-0.12	-0.04	0.99	0.12	0.19	0.05	1.0

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

3.4.1.2 Flows

The model calibration includes comparison to flows measured by the USGS at 12 transects in the Cooper River, the Wando River, and the harbor, as shown in Figure 3.4.4. The flows at these locations were measured by a vessel-mounted Acoustic Doppler Current Profiler (ADCP), which measures currents through the water column as the vessel moves across the river.

Plots of simulated and observed flow time series are provided in Attachment A-1 (Figures A-10 through A-34). In these plots the ebb flow is positive and the flood flow is negative. The model simulates similar magnitudes of ebb and flood flow rates as shown by the measured data for most of the comparisons. For some transects (such as Transect 1 on May 7, 2012), the model predicted peak ebb flow is lower than the observed data, and the ebb flow curve is flatter than the observed data. Although the simulated ebb flow for these plots is not as peaked as the observed ebb flow, for many of these comparisons the total area under the curve (i.e., total volume of water passed through the transect) is similar in magnitude. These comparisons demonstrate that the model is transporting the appropriate tidal prism (i.e., the total volume of water passed on each tide) through the Cooper and Wando Rivers.

3.4.1.3 Currents

The model calibration includes comparison to currents measured by the USGS at five locations in the harbor and the Ashley, Cooper, and Wando Rivers, as shown in Figure 3.4.5. Current meters were installed at four new locations, and one station was installed at the existing USGS station at Mobay. The station numbers, location (latitude and longitude), equipment installed, river mile, grid location (I,J), and the station short name are listed in Table 3.4.1.3. Four stations have horizontally oriented (side-looking) acoustic Doppler velocity meters (ADVMs), as indicated by the SL designation in Table 3.4.1.3. The ADVM time series velocity data are averaged over 4 minutes and recorded every 15 minutes. The G-25 station data was measured by a bottom-mounted upward-looking Acoustic Doppler Current Profiler (ADCP). This instrument continuously measures currents through the entire water column. This data was post-processed to evaluate current time series near the bottom, mid-depth and surface of the water column.

Plots of simulated and observed current speed and velocity time series are provided in Attachment A-1 (Figures A-35 through A-38). Comparison statistics are listed in Table 3.4.1.4. The percentiles from Table 3.4.1.4 are plotted versus river mile in Figures 3.4.6 and 3.4.7. Overall, the model simulated current speeds are generally in agreement with the range of observed data. At the harbor entrance, the predicted current speeds at the surface, mid-depth and bottom are in very good agreement with the measured values. However, in the Ashley River the model tends to under-predict the currents. In the Cooper River, the model slightly over-predicts currents at the Cooper River Marina and slightly under-predicts currents at Mobay. In the Wando River the model is in good agreement with the measured current speeds.

The simulated north and east velocities components do not agree as well as the current speeds as a result of variations in the velocity directions. However, the current speeds along the channel are in good agreement, and the model is reasonably simulating the velocities in the Cooper and Wando Rivers.

Note that the simulated values are output from a single model layer that changes in thickness and elevation over time, whereas the measured data were collected at a fixed elevation. This discrepancy accounts for some of the variability between the measured and simulated values. Furthermore, velocities are highly localized phenomena that are dependent on the flow through the river cross-section and the water depth at that point in space. The model discretizes the estuary into large quadrilateral areas (grid cells) with an average depth assigned to the entire grid cell. Some of the differences between the measured and simulated currents are caused by the fact that the simulated currents are averaged over a grid cell (with an associated average depth), whereas the observed currents are measured at a point in space (with an associated depth at that point).

Table 3.4.1.3. 2012 Current Meter Locations

Station number	Latitude	Longitude	Equipment*	River mile	I	J	Station short name
324958079560000	32° 49’ 58” N	079° 56’ 00” W	500 kHz SL	25.95	56	57	Site 1 - Cooper River Marina
324647079572400	32° 46’ 47” N	079° 57’ 24” W	1,500 kHz SL	24.62	64	13	Site 2 - Ashley River
324943079533700	32° 49’ 43” N	079° 53’ 37” W	500 kHz SL	25.76	79	59	Site 3 - Wando River
02172053	32° 59’ 26” N	079° 55’ 33” W	500 kHz SL	41.04	65	128	Cooper River at Mobay
G-25	32° 45’ 04” N	079° 51’ 52” W	1200 kHz UL	18.84	117	23	Site 4 – Charleston Harbor mouth

Notes: * SL = side-looking ADVM; UL = up-looking ADCP

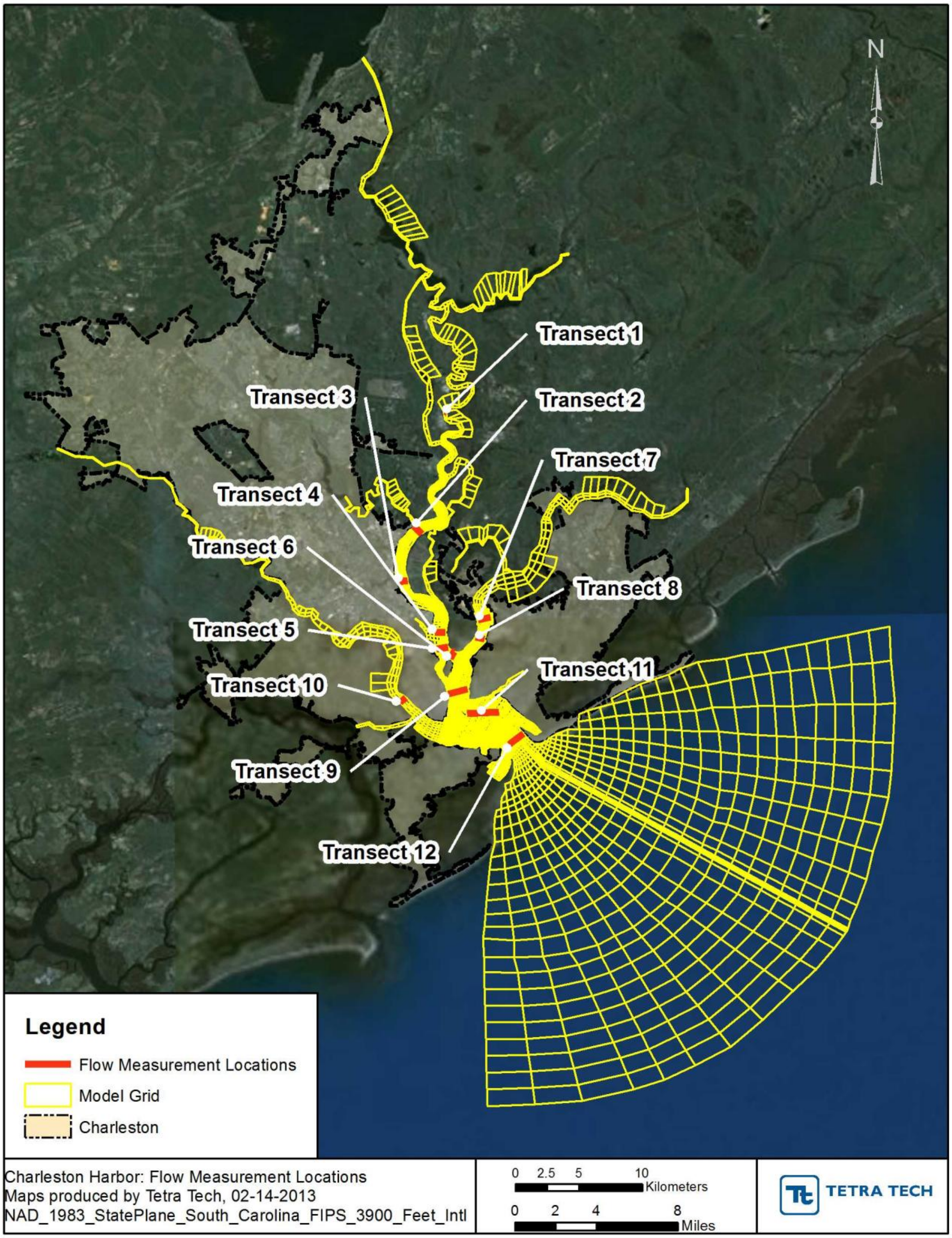


Figure 3.4.4. Flow measurement locations for 2012

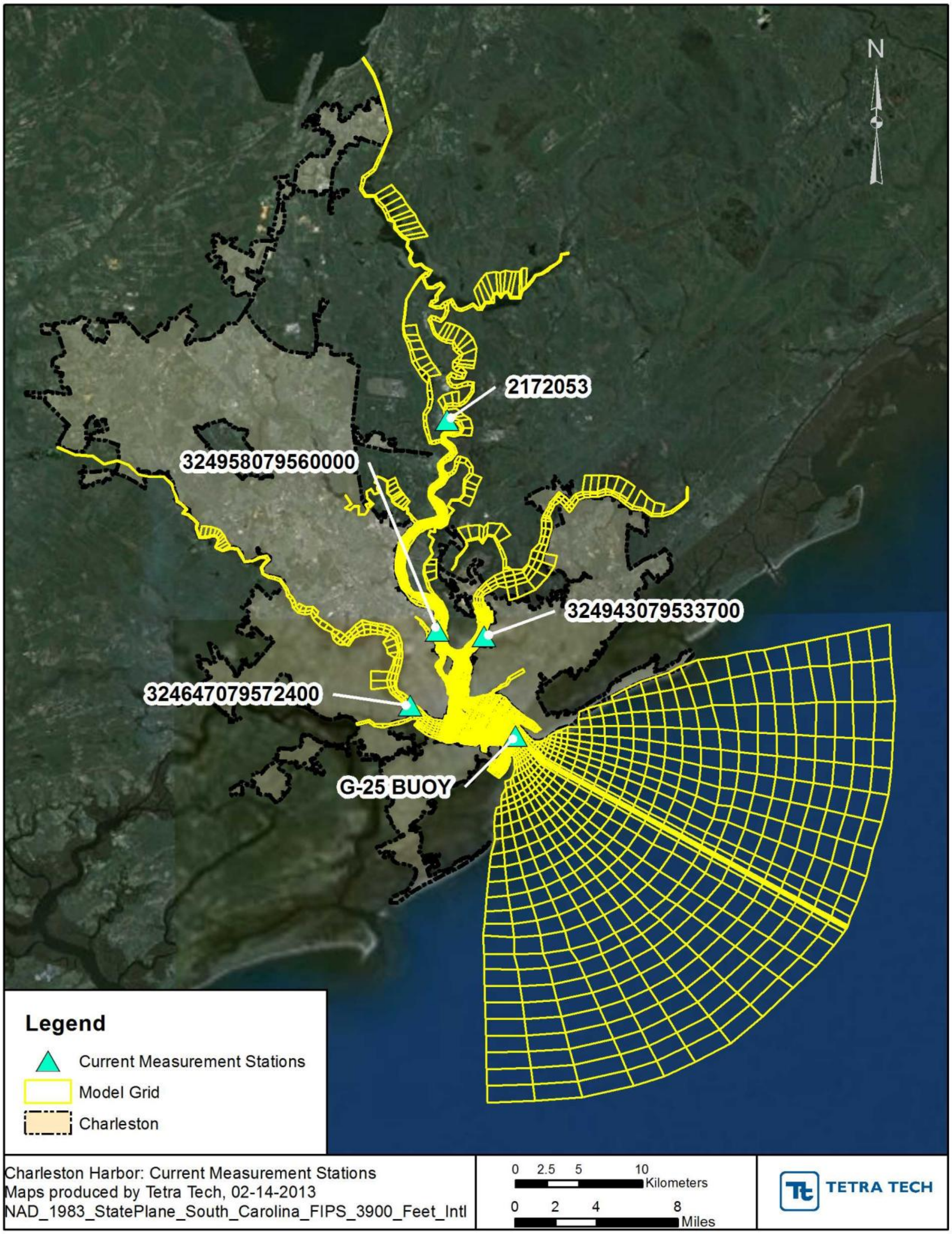


Figure 3.4.5. Continuous Current Measurement Locations for 2012

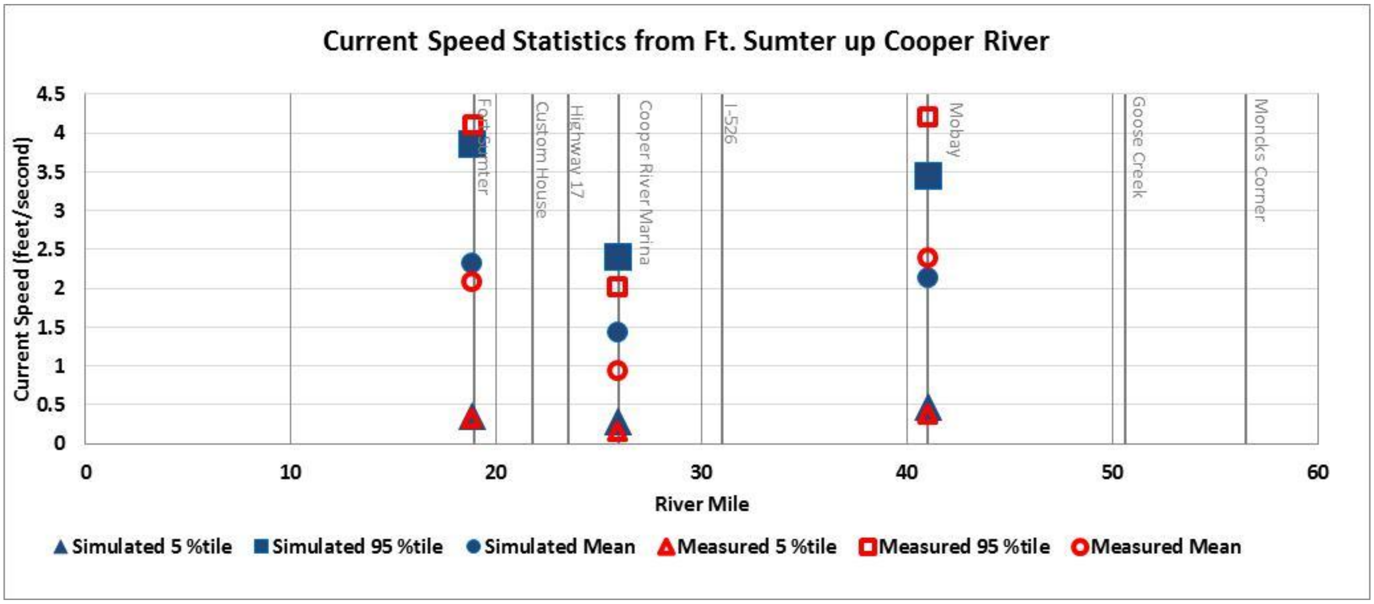


Figure 3.4.6. Longitudinal profile of 2012 current statistics from Fort Sumter to Pinopolis Dam

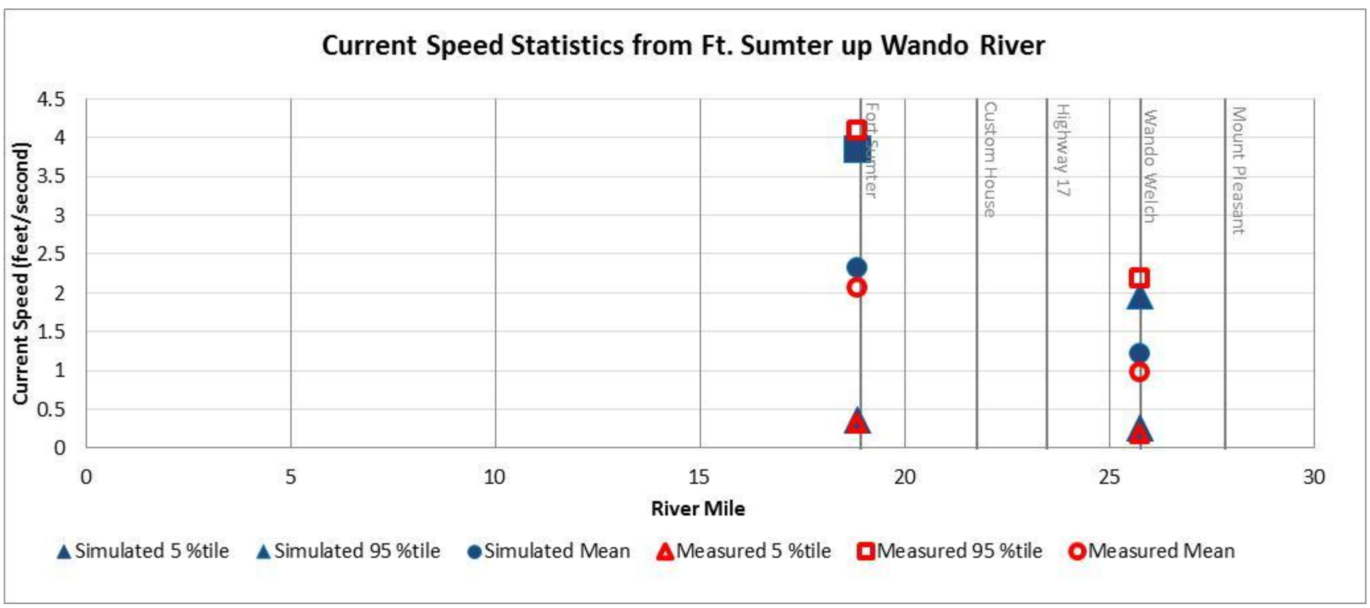


Figure 3.4.7. Longitudinal profile of 2012 current statistics from Charleston Harbor up Wando River

Table 3.4.1.4 2012 Current (feet per second) Comparison Statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Speed (feet/second) 05/01/2012 - 07/01/2012																
G-25 Surface*	18.84	4	2.42	0.49	4.71	2.61	0.39	4.57	0.19	-0.10	-0.14	0.31	0.88	1.06	0.17	0.85
G-25 Mid-depth*	18.84	3	2.07	0.31	4.10	2.32	0.36	3.86	0.25	0.05	-0.25	0.38	0.76	0.85	0.16	0.85
G-25 Bottom*	18.84	2	1.69	0.30	3.22	1.73	0.32	2.69	0.04	0.02	-0.53	0.46	0.52	0.65	0.14	0.78
324958079560000-Site 1 at Cooper River Marina, just downstream of Transect 4	25.95	4	0.93	0.15	2.01	1.43	0.29	2.41	0.50	0.14	0.40	0.24	0.66	0.81	0.66	0.64
324647079572400-Site 2 near Ashley Marina at James Island Expressway or Hwy 30	24.62	4	1.67	0.25	2.94	1.04	0.18	1.80	-0.63	-0.07	-1.14	0.67	0.67	0.81	0.56	0.70
324943079533700-Site 3 at POC Wando Welch, Transect 8	25.76	3	0.99	0.17	2.18	1.21	0.27	1.96	0.23	0.10	-0.22	0.43	0.43	0.53	0.45	0.77
USGS 02172053 Cooper River at Mobay	41.04	2	2.39	0.37	4.20	2.12	0.47	3.45	-0.27	0.10	-0.75	0.87	0.41	0.52	0.21	0.94
East Velocity (feet/second) 05/01/2012 - 07/01/2012																
G-25 Surface*	18.84	4	0.54	-2.12	3.88	0.24	-3.69	3.74	-0.30	-1.57	-0.14	0.72	0.91	1.07	0.13	0.95
G-25 Mid-depth*	18.84	3	0.22	-1.98	3.23	0.01	-3.13	2.98	-0.21	-1.15	-0.25	0.76	0.73	0.85	0.12	0.96
G-25 Bottom*	18.84	2	0.08	-1.80	2.47	-0.19	-2.38	1.87	-0.27	-0.58	-0.60	0.81	0.52	0.61	0.10	0.96
324958079560000-Site 1 at Cooper River Marina, just downstream of Transect 4	25.95	4	-0.11	-0.62	0.17	0.03	-0.14	0.13	0.14	0.49	-0.03	0.18	0.18	0.25	3.46	0.41
324647079572400-Site 2 near Ashley Marina at James Island Expressway or Hwy 30	24.62	5	0.08	-0.63	0.89	0.08	-1.09	1.20	0.01	-0.46	0.31	0.79	0.37	0.44	0.78	0.87
324943079533700-Site 3 at POC Wando Welch, Transect 8	25.76	5	-0.11	-1.81	1.20	-0.06	-0.68	0.65	0.05	1.13	-0.55	0.77	0.49	0.61	0.95	0.82
USGS 02172053 Cooper River at Mobay	41.04	4	0.04	-0.77	0.80	0.09	-0.78	0.99	0.05	-0.01	0.19	0.94	1.04	1.15	NaN	0.0

Table 3.4.1.4 continued. 2012 Current (feet per second) Comparison Statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
North Velocity (feet/second) 05/01/2012 - 07/01/2012																
G-25 Surface*	18.84	4	-0.16	-2.73	2.21	0.21	-1.63	2.71	0.36	1.10	0.50	0.83	0.60	0.71	0.10	0.95
G-25 Mid-depth*	18.84	3	-0.01	-2.52	2.11	0.12	-1.65	2.24	0.13	0.87	0.13	0.85	0.48	0.59	0.10	0.95
G-25 Bottom*	18.84	2	0.08	-2.04	1.94	0.05	-1.24	1.30	-0.03	0.80	-0.64	0.23	0.45	0.61	0.11	0.89
324958079560000-Site 1 at Cooper River Marina, just downstream of Transect 4	25.95	4	-0.36	-1.94	1.00	0.08	-2.18	2.29	0.44	-0.24	1.29	0.90	0.67	0.83	0.68	0.90
324647079572400-Site 2 near Ashley Marina at James Island Expressway or Hwy 30	24.62	5	0.05	-2.59	2.63	-0.08	-1.23	1.19	-0.13	1.37	-1.44	0.94	0.90	1.03	0.85	0.85
324943079533700-Site 3 at POC Wando Welch, Transect 8	25.76	5	-0.02	-1.04	0.93	-0.19	-1.81	1.54	-0.18	-0.78	0.61	0.70	0.71	0.80	1.02	0.80
USGS 02172053 Cooper River at Mobay	41.04	4	-0.38	-4.13	3.02	-0.31	-3.28	2.67	0.07	0.86	-0.35	0.97	0.44	0.55	0.23	0.99

*ADPC data at G-25 was not available after 06/11/2012, and therefore, the statistical analysis for G-25 is from 05/01/2012 to 06/11/2012.

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

3.4.1.4 Salinity

The model calibration includes comparison to salinity calculated from specific conductivity measured at the 11 USGS water level gages listed in Table 3.4.1.5 and shown in Figure 3.4.8. As mentioned in Section 3.3.5.3.2, the salinity at the offshore open boundary is based on data measured at a buoy (CAP2) about 5 miles offshore from Capers Island. CAP2 is 14.6 miles offshore from the G-25 Buoy.

Plots of simulated and observed salinity series are provided in Appendix A. It should be noted that there are two gages at the harbor entrance: one in shallow water near Fort Sumter and one surface instrument at the G-25 buoy in deep water adjacent to the federal navigation channel. The grid cell bottom elevation at the Fort Sumter gage is only -11 feet MLLW, and the grid cell bottom elevation at the G-25 buoy is -38 feet MLLW. The overall salinity intrusion to the estuary is more dependent on correctly simulating salinity in the deep entrance channel rather than accurately simulating the salinity in the shallow off-channel areas. Therefore, the model calibration focused on the agreement between the simulated and observed salinity at the G-25 buoy instead of the Fort Sumter gage. The good agreement at the G-25 buoy indicates that the model is reasonably simulating salinity at the harbor entrance.

Comparison statistics are listed in Table 3.4.1.6. Figure 3.4.9 presents the salinity percentiles from Fort Sumter up the Cooper River. Figure 3.4.10 presents the salinity percentiles from the harbor up the Wando River. These figures show that the model reasonably simulates the range of median salinity extending from values near 37 ppt at the ocean boundary to low concentrations in the Cooper River at Mobay. As shown in the longitudinal plots (Figures 3.4.9 and 3.4.10), the calibration represents the overall trends shown by the measured data. A closer look at differences in mean statistics shows ±10% differences between simulated and observed values; differences within range of instrument effort. In general, the model is slightly over predicting salinity in the Cooper River and slightly under predicting salinity in the Wando River, with mean differences of ±10%. The greatest variation between simulated and observed on the Cooper River at Mobay where observed salinity ranges from 0 to 16 ppt and the simulated salinity range from near 1 to 9 ppt. Even upstream of Mobay at Goose Creek, where observed salinity ranges from 0 to 1 ppt, the mean differences between simulated and observed salinity are only 3%.

Table 3.4.1.5 2012 salinity and temperature measurement locations

USGS station ID	Description	River mile	I	J
02172020	West Branch Cooper River at Pimlico near Moncks Corner	57.16	46	167
02172050	Cooper River near Goose Creek	50.57	65	161
02172053	Cooper River at Mobay	41.04	65	128
2172067.7	Cooper River at I-526	31.00	56	79
2172070.9	Cooper River at Hwy 17	23.48	81	38
2172071	Cooper River at Custom House	21.78	70	21
324943079533700	Wando River at Wando Welch Terminal	25.76	79	59
2172069.8	Wando River above Mount Pleasant (I-526)	27.84	75	68
2172086.9	Ashley River near North Charleston (I-526)	32.77	44	17
2172100	Charleston Harbor at Fort Sumter	18.94	115	19
324544079520700	G-25 Buoy at Fort Sumter	18.84	117	23

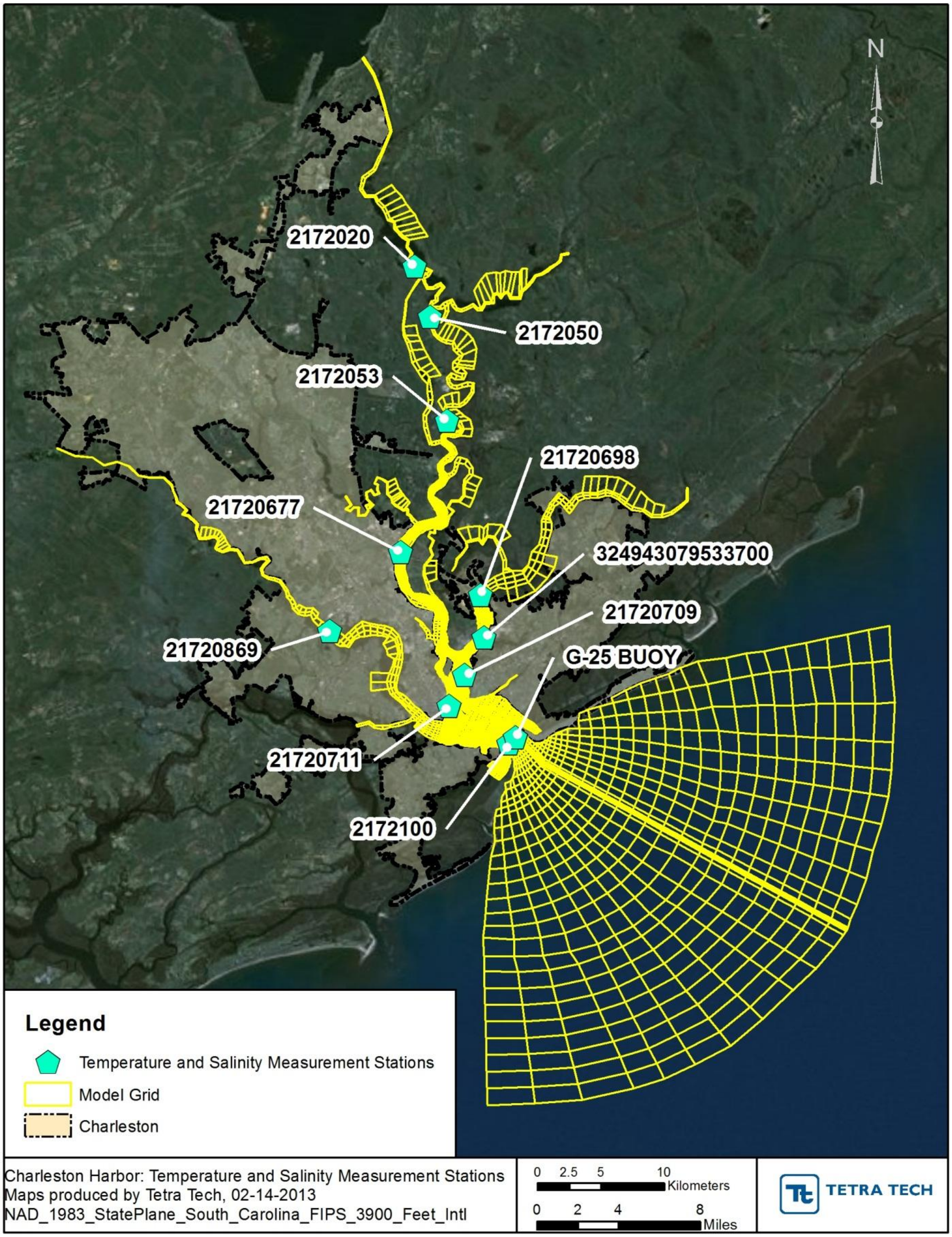


Figure 3.4.8 Continuous temperature and salinity measurement locations for 2012

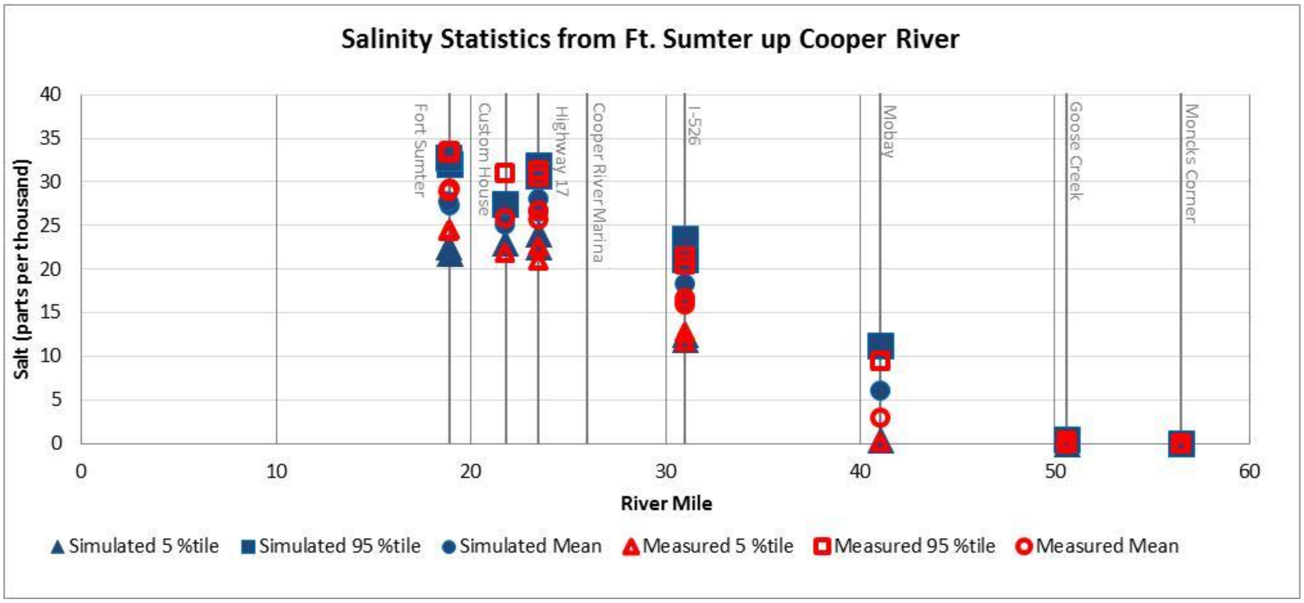


Figure 3.4.9 Longitudinal profile of 2012 salinity statistics from Fort Sumter to Pinopolis Dam

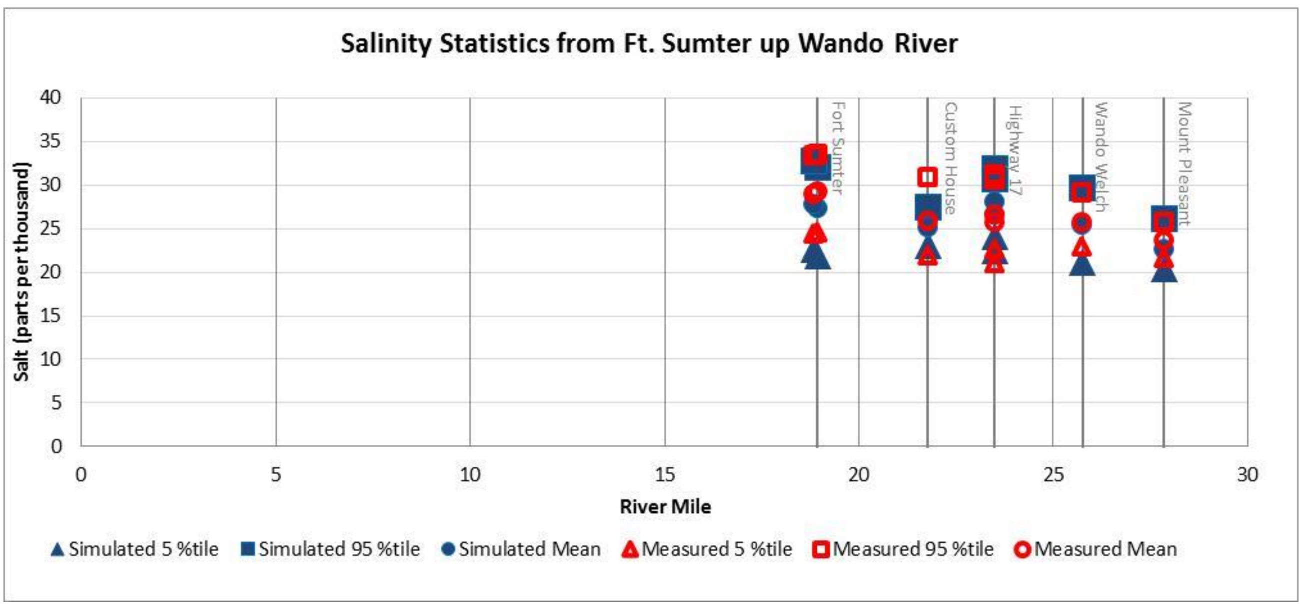


Figure 3.4.10. Longitudinal profile of 2012 salinity statistics from Charleston Harbor up Wando River

Table 3.1.1.6. 2012 salinity (ppt) comparison statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Calculated salinity (parts per thousand [ppt]) 05/01/2012 - 07/01/2012																
USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC	57.16	5	0.05	0.05	0.06	0.05	0.05	0.05	0.00	0.00	-0.01	0.31	0.00	0.01	0.11	0.14
USGS 02172050 Cooper River near Goose Creek, SC	50.57	5	0.13	0.06	0.34	0.11	0.05	0.41	-0.02	-0.01	0.07	0.58	0.08	0.17	0.93	0.73
USGS 02172053 Cooper River at Mobay	41.04	4	2.93	0.18	9.41	5.99	0.51	11.21	3.06	0.34	1.80	0.63	3.20	3.70	0.73	0.74
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3	23.59	21.52	25.69	22.59	20.31	26.06	-1.01	-1.20	0.37	0.44	1.46	1.69	0.07	0.73
USGS 324544079533700 Wando River at Wando Welch	25.76	2	25.69	22.84	29.01	25.31	21.04	29.65	-0.38	-1.79	0.64	0.74	1.29	1.57	0.06	0.89
USGS 02172086.9 Ashley River near North Charleston	32.77	5	15.98	3.07	23.75	16.86	13.16	19.04	0.88	10.09	-4.71	0.65	4.05	5.21	0.31	0.58
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	5	29.18	24.45	33.46	27.26	21.86	31.97	-1.93	-2.59	-1.49	0.54	2.38	2.96	0.11	0.78
Surface salinity calculated (ppt) 05/01/2012 - 07/01/2012																
USGS 02172071.0 Cooper River at Custom House	21.78	5	25.83	21.79	30.90	24.92	20.51	29.85	-0.91	-1.28	-1.04	0.66	1.68	2.03	0.08	0.87
G-25 Buoy near Fort Sumter	18.94	6	28.85	24.38	33.31	27.66	22.54	32.71	-1.19	-1.84	-0.60	0.76	1.64	1.95	0.07	0.89
Mid-depth salinity calculated (ppt) 05/01/2012 - 07/01/2012																
USGS 02172067.7 Cooper River at I-526	31.00	3	15.92	11.59	20.48	16.29	11.88	21.06	0.37	0.29	0.58	0.60	1.57	1.96	0.12	0.87
USGS 02172070.9 Cooper River at Hwy 17	23.48	3	25.63	20.89	30.53	26.59	22.50	30.62	0.97	1.61	0.09	0.62	1.66	2.08	0.08	0.86
Bottom salinity calculated (ppt) 05/01/2012 - 07/01/2012																
USGS 02172067.7 Cooper River at I-526	31.00	1	16.73	12.69	21.52	18.23	12.60	23.42	1.50	-0.10	1.90	0.55	2.21	2.80	0.16	0.80
USGS 02172070.9 Cooper River at Hwy 17	23.48	1	26.62	22.56	31.19	28.04	24.03	31.84	1.42	1.47	0.64	0.46	1.88	2.49	0.09	0.77

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

3.4.1.5 Temperature

The model calibration includes comparison to temperature measured at the 11 USGS water level gages listed in Table 3.4.1.5 and shown in Figure 3.4.8 (the same locations as the salinity data). Plots of simulated and observed temperature series are provided in Appendix A. Comparison statistics are listed in Table 3.4.1.7. Figure 3.4.11 presents the temperature percentiles from Fort Sumter up the Cooper River. Figure 3.4.12 presents the temperature percentiles from the harbor up the Wando River.

These figures and statistics illustrate that the model simulation provides a good representation of monthly and seasonal trends in water temperature throughout the CHS. Differences in mean temperature between simulated and observed values are within 0 to 3%; much less than error assumed from instrumentation. Assumptions in the model limit the simulations ability to match hourly fluctuations in observed values. As described in Section 3.3.5, temperature inputs were based on daily average water temperatures. Tributary temperatures were calculated from 7 USGS stations. Pinopolis Dam inputs were based on measured daily average values and point source inputs were based on DMRs.

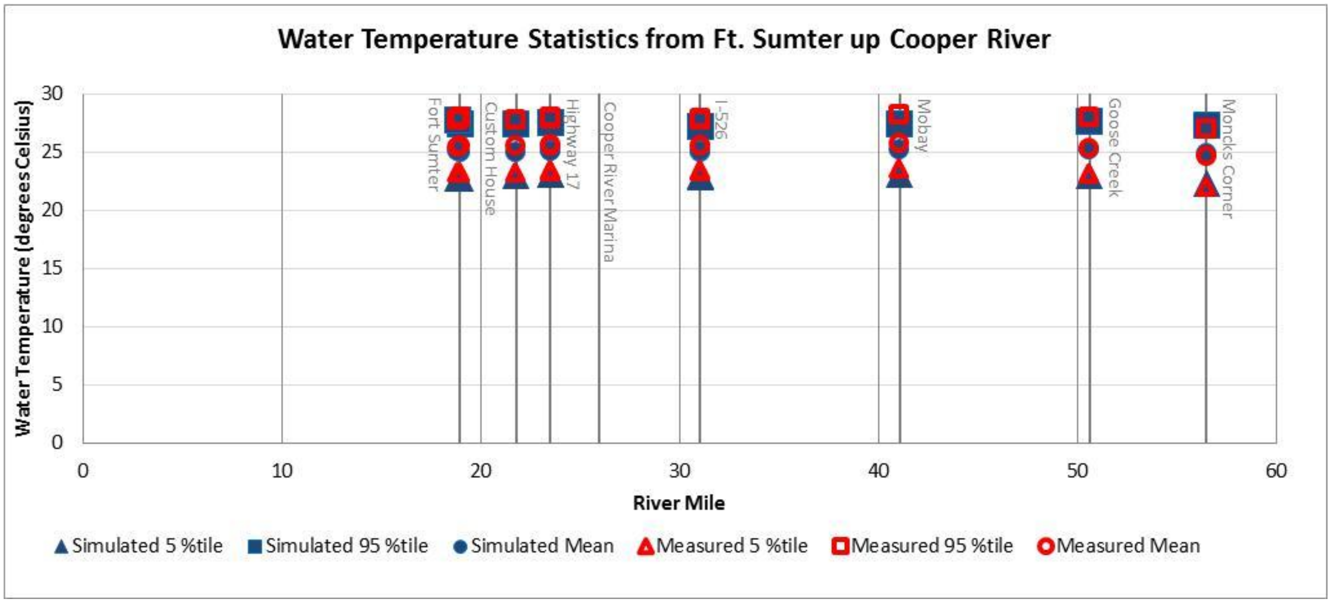


Figure 3.4.11 Longitudinal profile of 2012 temperature statistics from Fort Sumter to Pinopolis Dam

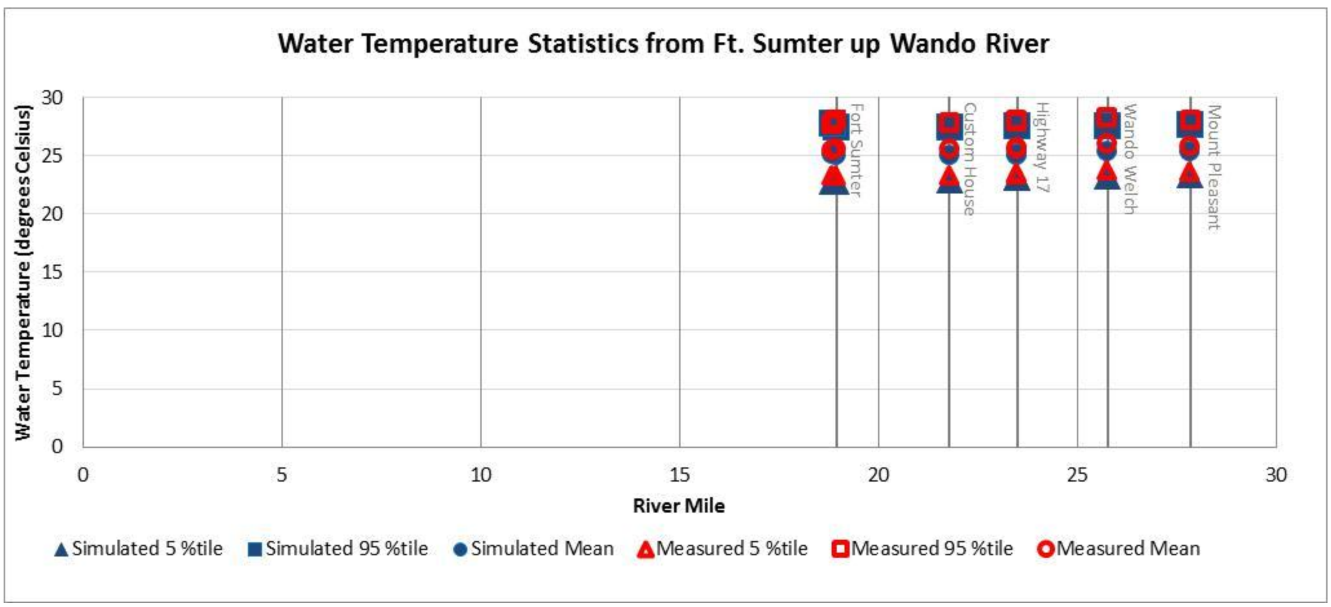


Figure 3.4.12 Longitudinal profile of 2012 temperature statistics from Charleston Harbor up Wando River

Table 3.4.1.7 2012 Temperature (degrees Celsius) Comparison statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Water temperature (degrees Celsius [ºC]) 05/01/2012 - 07/01/2012																
USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC	57.16	5	24.7	22.0	27.1	24.9	22.4	27.4	0.2	0.4	0.3	0.84	0.5	0.7	0.0	0.95
USGS 02172050 Cooper River near Goose Creek, SC	50.57	5	25.4	23.1	28.0	25.2	23.0	27.7	-0.2	-0.1	-0.3	0.91	0.4	0.5	0.0	0.97
USGS 02172053 Cooper River at Mobay	41.04	4	25.8	23.6	28.3	25.3	23.2	27.5	-0.5	-0.4	-0.8	0.85	0.6	0.7	0.0	0.92
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3	25.8	23.6	28.1	25.3	23.3	27.7	-0.5	-0.3	-0.4	0.95	0.5	0.6	0.0	0.95
USGS 324544079533700 Wando River at Wando Welch	25.76	2	26.1	23.7	28.3	25.3	23.3	27.6	-0.8	-0.4	-0.7	0.95	0.8	0.8	0.0	0.91
USGS 02172086.9 Ashley River near North Charleston	32.77	5	26.0	24.0	28.2	25.7	23.4	27.7	-0.3	-0.5	-0.6	0.69	0.7	0.8	0.0	0.89
USGS 02172071.1 Cooper River at Custom House (AUX)	21.78	5	25.6	23.3	27.8	25.1	23.0	27.4	-0.5	-0.3	-0.4	0.93	0.5	0.6	0.0	0.95
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	5	25.6	23.3	28.0	25.0	22.8	27.5	-0.6	-0.5	-0.5	0.89	0.6	0.7	0.0	0.93
Surface water temperature (ºC) 05/01/2012 - 07/01/2012																
G-25 Buoy near Fort Sumter	18.94	6	25.4	23.3	27.8	25.2	22.8	27.8	-0.3	-0.4	0.1	0.92	0.4	0.5	0.0	0.97
Mid-depth water temperature (ºC) 05/01/2012 - 07/01/2012																
USGS 02172067.7 Cooper River at I-526	31.00	3	25.6	23.4	27.9	25.1	23.0	27.2	-0.5	-0.4	-0.7	0.94	0.5	0.6	0.0	0.95
USGS 02172070.9 Cooper River at Hwy 17	23.48	3	25.7	23.5	28	25.2	23.2	27.6	-0.5	-0.3	-0.4	0.95	0.5	0.6	0.0	0.96
Bottom water temperature (ºC) 05/01/2012 - 07/01/2012																
USGS 02172067.7 Cooper River at I-526	31.00	1	25.6	23.5	27.8	25.1	22.9	27.2	-0.5	-0.6	-0.6	0.93	0.5	0.6	0.0	0.95
USGS 02172070.9 Cooper River at Hwy 17	23.48	1	25.6	23.4	27.9	25.1	23.1	27.6	-0.5	-0.3	-0.3	0.94	0.5	0.6	0.0	0.95

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

3.4.2 2004 Model Validation

The model validation includes comparisons to observed data from 2004 that were used to calibrate the BCDCOG DO TMDL model. The simulation results for this effort are comparable with the BCDCOD DO TMDL model calibration for water surface elevation. The two modeling efforts were compared by review of statistical tables and visual comparisons. The statistical comparisons made during this modeling effort are based on simulations from April through October 2004. As discussed in Section 3.3.2, the model bathymetry was also refined with more recent data. Differences from bathymetry refinements explain some variability between simulations seen in the statistical analysis.

3.4.2.1 Water Surface Elevation

For water surface elevation, the 2004 validation provides a reasonable representation of water levels throughout the CHS. The model validation includes comparison to measured water surface elevation at the 7 USGS water level gages listed in Table 3.4.2.1 with comparison statistics. Plots of simulated and observed water surface elevation are provided in Attachment B. Figure 3.4.13 presents the water surface elevation percentiles from Fort Sumter up the Cooper River. Figure 3.4.14 presents the water surface percentiles from the harbor up the Wando River.

The validation completed during this modeling effort is similar to the results from the previous BCDCOG DO TMDL model. The two simulations have consistent difference from observed data at stations in the harbor and lower riverine sections. Moving upstream on the Cooper River variability between simulated and observed water surface elevation increases. Comparing figures from the two simulations, water surface elevations are comparable. At Mobay in particular, differences between measured and simulated are similar when comparing figures and considering differences in the y-axis scale.

3.4.2.2 Salinity

The model validation includes comparison to measured data at the 10 USGS gages listed in Table 3.4.2.2 and illustrated in Figure 3.4.15. Table 3.4.2.2 includes comparison statistics. Plots of simulated and observed salinity series are provided in Attachment A-2. Figure 3.4.16 presents the salinity percentiles from Fort Sumter up the Cooper River. Figure 3.4.17 presents the salinity from the harbor up the Wando River. Validation of the 2012 Post 45 model with 2004 observed data found that trends in monthly salinity were begin captured during the summer of 2004; similar to what was found during the BCDCOG DO TMDL model effort.

The model validation results indicate that the model predicts more of a vertical gradient in salinity than indicated by the observed data (e.g., compare Figure B-11 to Figure B-13 for the Cooper River at I-526; compare Figure B-14 to Figure B-16 for the Cooper River at Hwy 17; and also compare Figure B-18 to Figure B-20 for the Wando River above Mount Pleasant). Based on model calibration testing, this was necessary in order to simulate the salinity intrusion to the upper Cooper River stations at Mobay and Goose Creek. Therefore, the model gives conservative estimates of project effects.

The model does not agree well with the observed data in the Wando River at Cainhoy (Figure B-17). This is mainly due to three factors. First, the tendency of the model to over-predict the vertical gradient may affect this comparison. Second, the large freshwater discharge events in August 2004 are likely under estimated by the LSPC flows in the boundary conditions (similar results are seen in the BCDCOG DO TMDL model calibration results). Third, the initial salinity condition in the Wando River used for the model calibration does not perform as well for the model validation period.

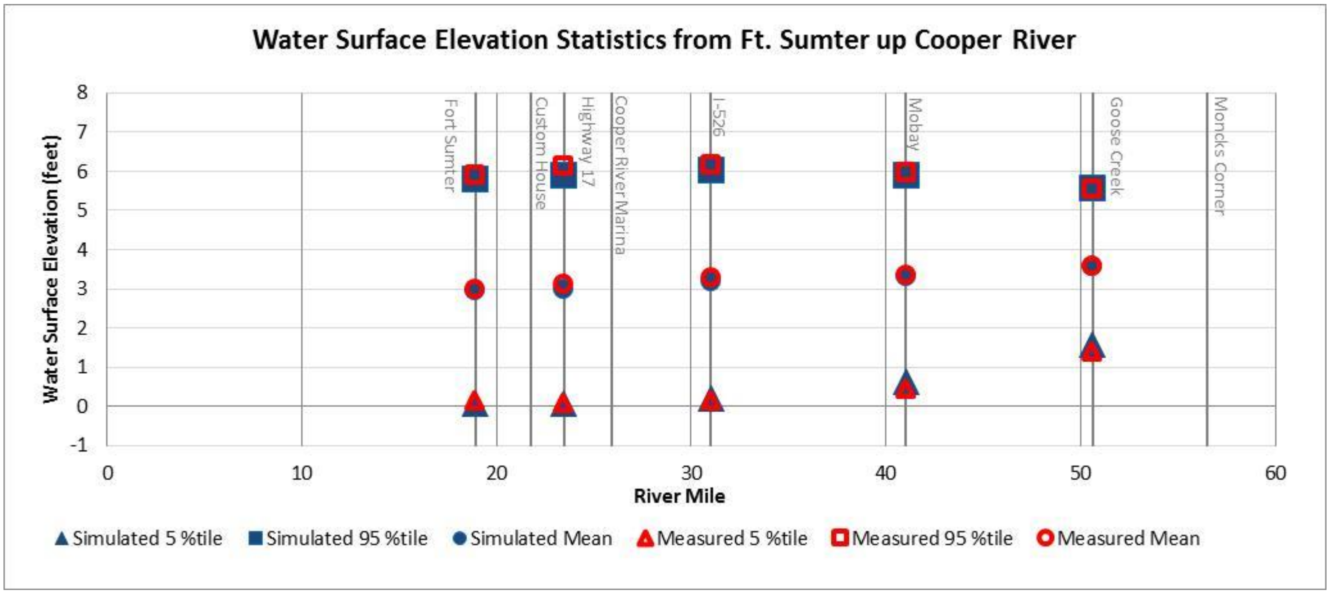


Figure 3.4.13 Longitudinal profile of 2004 water surface elevation (feet, MLLW at Customs House) statistics from Fort Sumter to Pinopolis Dam

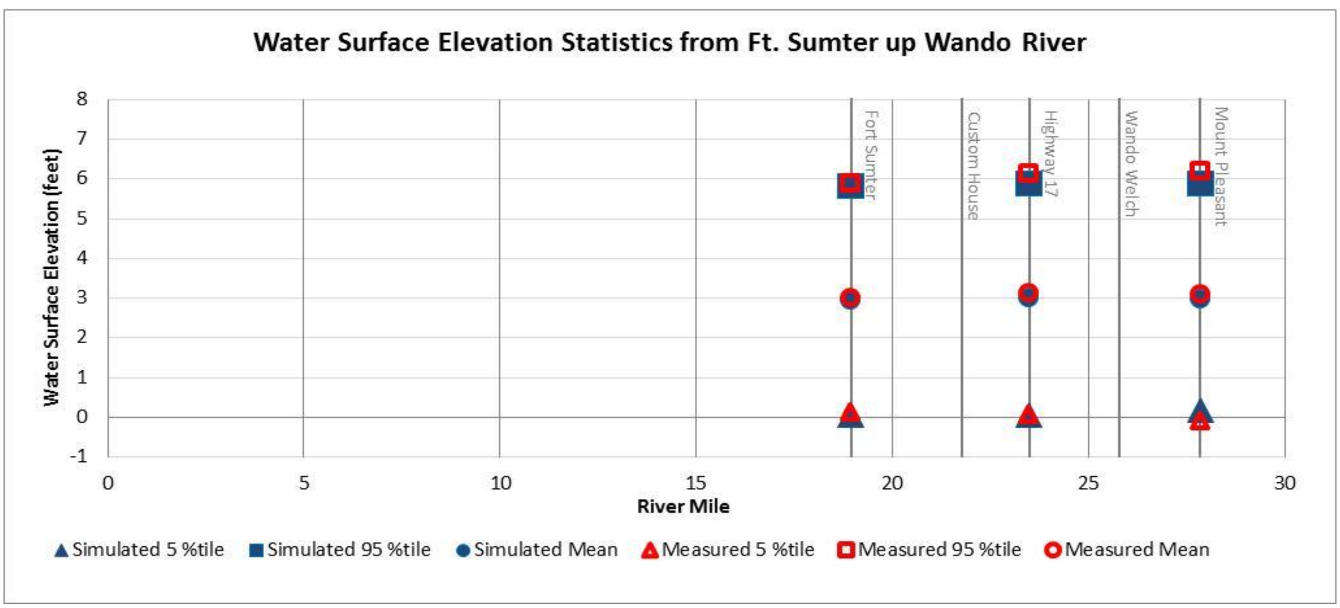


Figure 3.4.14 Longitudinal profile of 2004 water surface elevation (feet, MLLW at Customs House) statistics from Charleston Harbor up Wando River

Table 3.4.2.1 2004 water surface elevation (feet, MLLW at Customs House) comparison statistics for the period from 04/15/2004 – 11/01/2004

Station	River Mile	Measured			Simulated			Difference			R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
		Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile					
USGS 02172050 Cooper River near Goose Creek, SC	50.57	3.58	1.40	5.53	3.57	1.57	5.56	-0.01	0.18	0.03	0.96	0.20	0.25	0.07	0.99
USGS 02172053 Cooper River at Mobay	41.04	3.35	0.43	5.97	3.32	0.63	5.89	-0.03	0.21	-0.08	0.96	0.30	0.38	0.10	0.99
USGS 02172067.7 Cooper River at I-526	31.0	3.29	0.15	6.16	3.18	0.21	6.03	-0.10	0.06	-0.14	0.97	0.27	0.34	0.09	0.99
USGS 02172086.9 Ashley River near North Charleston	32.77	3.15	-0.09	6.14	3.10	-0.07	6.11	-0.05	0.02	-0.03	0.99	0.17	0.23	0.06	1.0
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3.11	-0.08	6.18	2.99	0.18	5.88	-0.13	0.26	-0.30	0.95	0.41	0.50	0.14	0.98
USGS 02172070.9 Cooper River at Hwy 17	23.48	3.12	0.07	6.14	2.99	0.07	5.89	-0.12	0.00	-0.24	0.99	0.21	0.25	0.07	1.0
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	2.99	0.13	5.88	2.95	0.07	5.81	-0.04	-0.06	-0.07	1.0	0.102	0.13	0.04	1.0

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

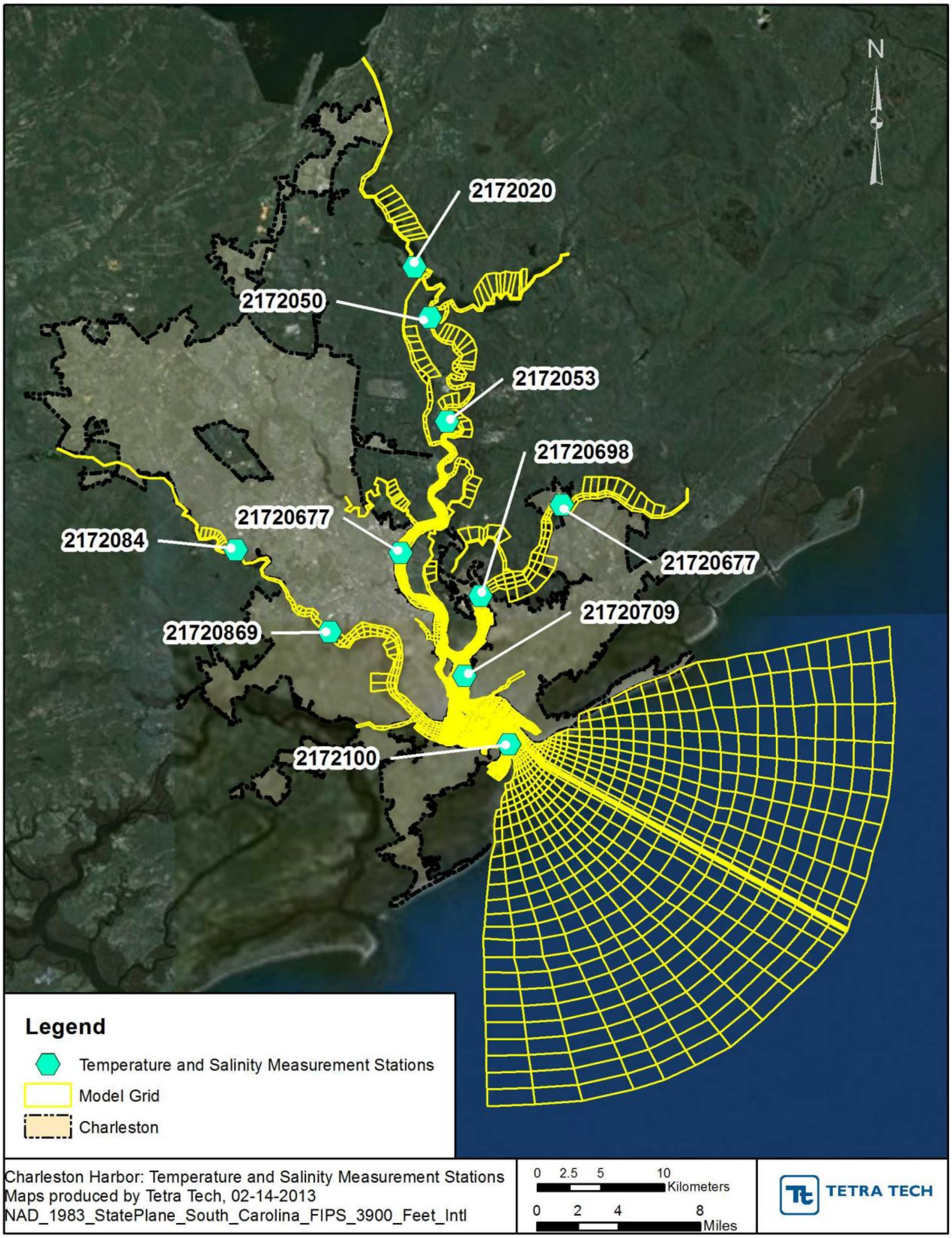


Figure 3.4.15 Continuous temperature and salinity measurement locations for 2004

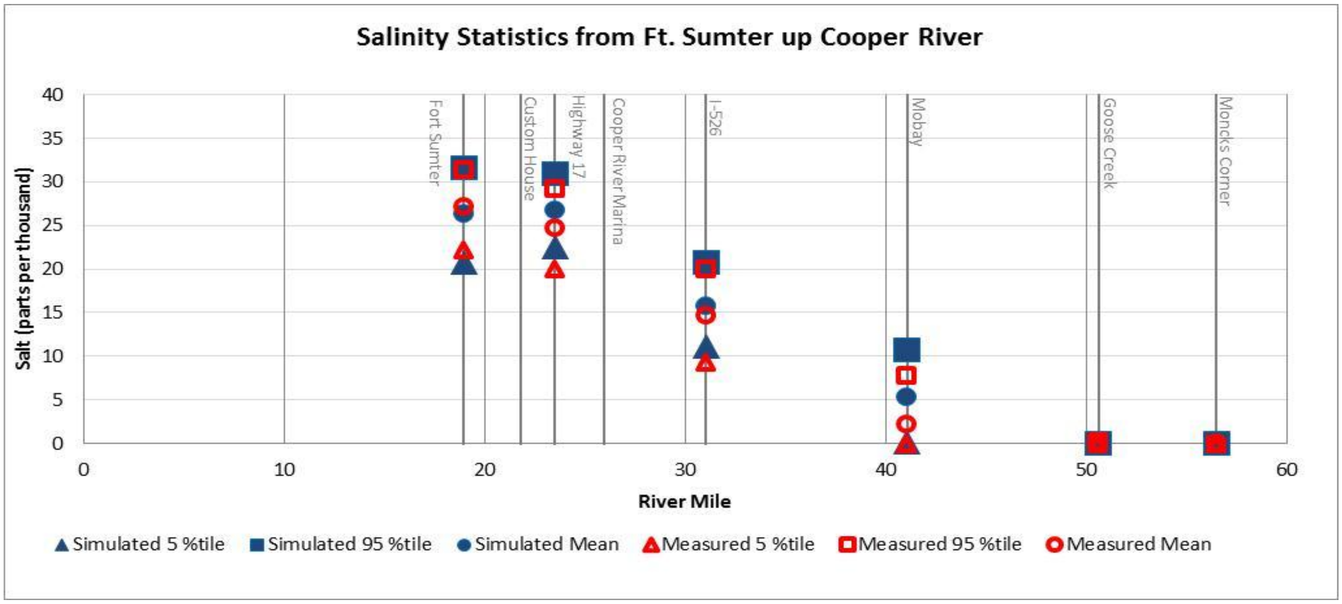


Figure 3.4.16 Longitudinal profile of 2004 salinity statistics from Fort Sumter to Pinopolis Dam

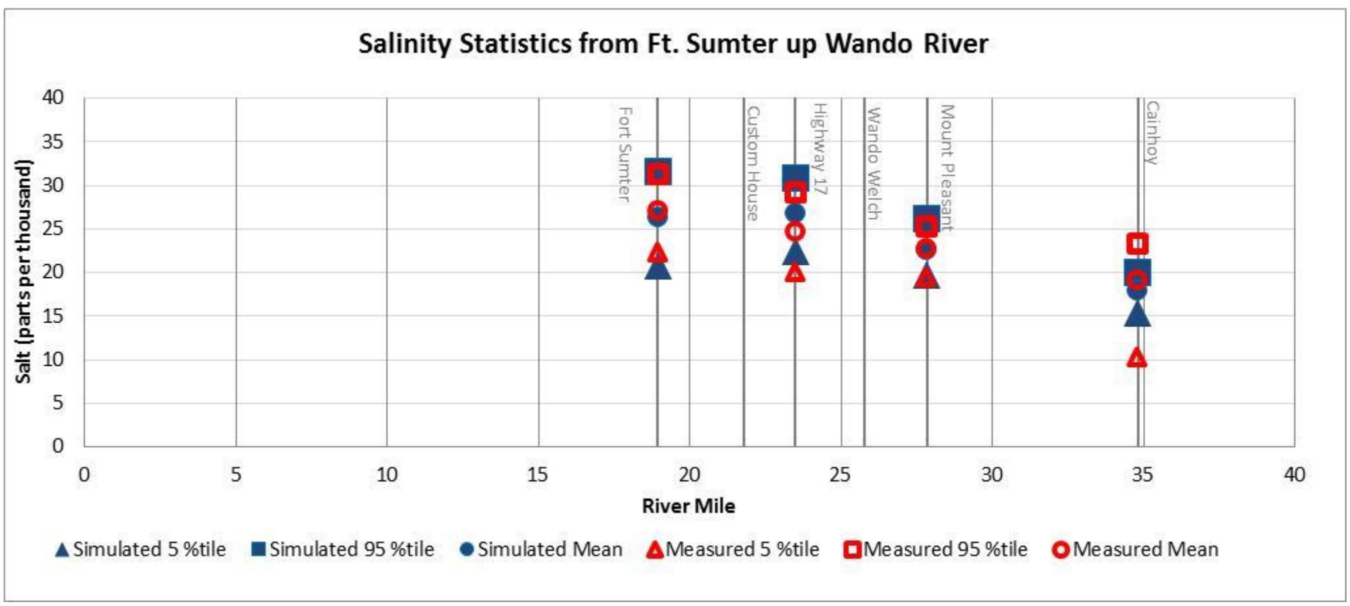


Figure 3.4.17 Longitudinal profile of 2004 salinity statistics from Charleston Harbor up Wando River

Table 3.4.2.2 2004 salinity (ppt) comparison statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Calculated salinity (parts per thousand [ppt]) 04/15/2004 - 11/01/2004																
USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC	57.16	5	0.04	0.04	0.04	0.05	0.05	0.05	0.01	0.01	0.01	0.01	0.01	0.01	0.27	0.07
USGS 02172050 Cooper River near Goose Creek, SC	50.57	5	0.08	0.04	0.20	0.06	0.05	0.09	-0.02	0.01	-0.10	0.27	0.03	0.07	0.85	0.65
USGS 02172053 Cooper River at Mobay	41.04	4	2.25	0.09	7.78	5.38	0.19	10.73	3.13	0.10	2.95	0.62	3.18	3.76	0.86	0.72
USGS 02172084 Ashley River at Bakers Landing	39.95	4	3.72	0.19	9.73	4.16	0.45	9.51	0.44	0.26	-0.22	0.60	1.54	2.02	0.43	0.87
Surface salinity calculated (ppt) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	6	13.93	8.50	18.50	10.19	7.18	13.96	-3.74	-1.32	-4.54	0.46	3.91	4.34	0.36	0.55
USGS 02172070.9 Cooper River at Hwy 17	23.48	6	24.05	19.95	28.36	22.16	17.32	26.62	-1.90	-2.62	-1.74	0.53	2.20	2.76	0.12	0.76
USGS 02172069.8 Wando River above Mount Pleasant	27.84	6	22.66	20.15	25.12	20.60	18.15	22.84	-2.06	-2.01	-2.28	0.34	2.24	2.54	0.12	0.58
USGS 02172086.9 Ashley River near North Charleston	32.77	6	15.80	7.09	22.44	14.17	6.85	18.91	-1.64	-0.25	-3.53	0.50	2.98	3.63	0.24	0.80
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	6	27.14	22.30	31.32	26.35	20.73	31.65	-0.80	-1.57	0.33	0.37	2.41	2.95	0.11	0.76
Mid-depth salinity calculated (ppt) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	3	14.66	9.35	19.95	15.81	11.08	20.73	1.15	1.73	0.78	0.6	1.87	2.37	0.15	0.85
USGS 02172070.9 Cooper River at Hwy 17	23.48	3	19.17	10.19	23.28	17.86	15.31	20.04	-1.31	5.12	-3.24	0.22	3.59	3.97	0.21	0.51
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3	22.68	19.40	25.26	22.52	19.73	26.13	-0.15	0.33	0.87	0.29	1.56	1.85	0.08	0.73
USGS 02172069.6 Wando River at Cainhoy	34.81	4	24.67	20.09	29.20	26.77	22.49	30.84	2.09	2.40	1.64	0.51	2.38	2.92	0.11	0.74

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

Table 3.4.2.2 continued. 2004 salinity (ppt) comparison statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Bottom salinity calculated (ppt) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	1	15.71	10.00	21.60	17.89	11.75	23.43	2.18	1.75	1.83	0.55	2.57	3.38	0.20	0.79
USGS 02172070.9 Cooper River at Hwy 17	23.48	1	25.85	21.19	30.44	28.63	24.07	32.57	2.78	2.88	2.13	0.55	2.89	3.42	0.13	0.70
USGS 02172069.8 Wando River above Mount Pleasant	27.84	1	23.08	19.67	25.90	22.91	19.81	27.15	-0.18	0.14	1.25	0.40	1.57	1.84	0.08	0.78
USGS 02172086.9 Ashley River near North Charleston	32.77	1	16.44	8.09	22.71	18.41	15.53	20.81	1.97	7.44	-1.89	0.40	2.97	3.97	0.23	0.58
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	1	28.38	23.63	32.57	29.83	23.90	34.58	1.45	0.27	2.01	0.52	2.22	2.77	0.10	0.80

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

3.4.2.3 Temperature

The model validation includes comparison to measured data at the 10 USGS gages listed in Table 3.4.2.3 and illustrated in Figure 3.4.15. Plots of simulated and observed temperature are provided in Attachment A-2. Figure 3.4.18 presents the salinity percentiles from Fort Sumter up the Cooper River. Figure 3.4.19 presents the salinity from the harbor up the Wando River.

The 2012 Post 45 modeling effort was found to represent water temperatures throughout the CHS. Time series figures in Attachment A-2 generally indicate that water temperatures are over predicted when compared in figures with 2004 observed values. However, visual comparisons of longitudinal plots in Figures 3.4.18 and 3.4.19 and review of statistical results in Table 3.4.2.3, indicate that simulated water temperatures are reasonable representing conditions in the CHS.

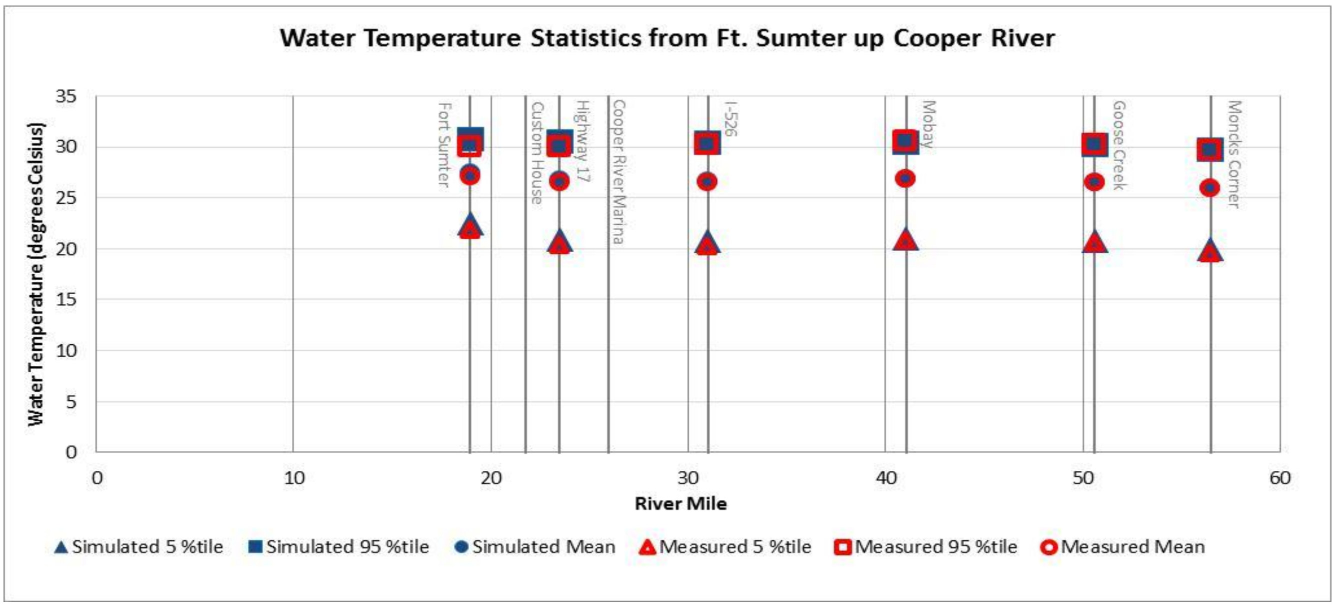


Figure 3.4.18. Longitudinal profile of 2004 temperature statistics from Fort Sumter to Pinopolis Dam

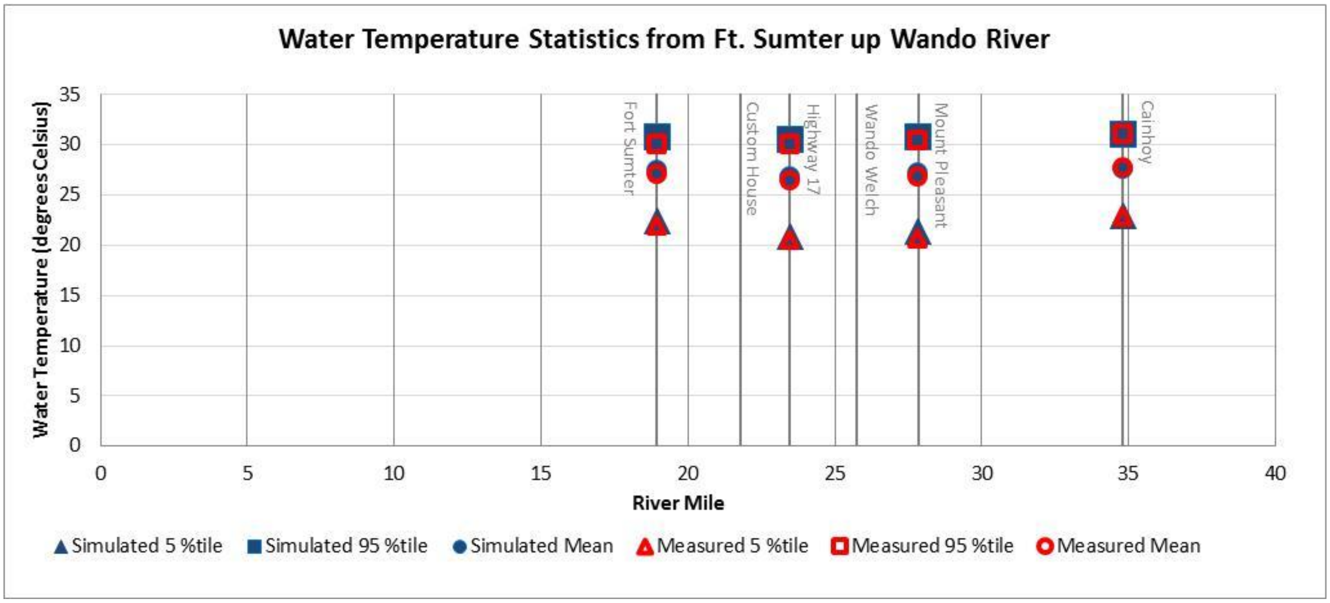


Figure 3.4.19. Longitudinal profile of 2004 temperature statistics from Charleston Harbor up Wando River

Table 3.4.2.3 2004 temperature (degrees Celsius) comparison statistics

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Water temperature (degrees Celsius [ºC]) 04/15/2004 - 11/01/2004																
USGS 02172020 Cooper River near Moncks Corner	57.16	5	26.0	19.6	29.6	26.0	20.0	29.7	0.0	0.4	0.1	0.97	0.4	0.5	0.0	0.99
USGS 02172050 Cooper River near Goose Creek, SC	50.57	5	26.5	20.7	30.2	26.5	20.8	30.1	0.0	0.1	-0.1	0.98	0.3	0.4	0.0	0.99
USGS 02172053 Cooper River at Mobay	41.04	4	26.9	21.0	30.5	26.9	21.0	30.5	0.0	0.0	0.0	0.99	0.3	0.4	0.0	1.0
USGS 02172084 Ashley River at Bakers Landing	39.95	4	27.0	22.0	30.6	27.0	22.0	30.8	0.0	0.0	0.2	0.95	0.5	0.6	0.0	0.99
Surface water temperature (ºC) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	6	26.5	20.3	30.2	26.8	21.2	30.6	0.3	0.9	0.4	0.98	0.4	0.5	0.0	0.99
USGS 02172070.9 Cooper River at Hwy 17	23.48	6	26.5	20.2	30.0	27.0	21.1	30.9	0.5	0.9	0.9	0.98	0.6	0.7	0.0	0.99
USGS 02172069.8 Wando River above Mount Pleasant	27.84	6	26.9	20.5	30.4	27.3	21.5	30.9	0.5	1.0	0.5	0.98	0.5	0.6	0.0	0.99
USGS 02172086.9 Ashley River near North Charleston	32.77	6	27.4	22.3	30.8	27.6	22.4	31.6	0.3	0.1	0.8	0.91	0.7	0.9	0.0	0.97
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	6	27.1	21.9	30.0	27.4	22.5	30.7	0.3	0.6	0.7	0.95	0.5	0.6	0.0	0.98
Mid-depth water temperature (ºC) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	3	26.5	20.3	30.2	26.6	20.8	30.4	0.1	0.5	0.2	0.99	0.3	0.4	0.0	1.0
USGS 02172070.9 Cooper River at Hwy 17	23.48	3	26.5	20.5	30.0	26.7	20.9	30.5	0.3	0.4	0.5	0.98	0.4	0.5	0.0	0.99
USGS 02172069.6 Wando River at Cainhoy	34.81	4	27.7	22.8	31.0	27.6	22.9	31.1	0.0	0.1	0.1	0.96	0.4	0.5	0.0	0.99
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3	26.9	20.6	30.4	27.2	21.4	30.8	0.3	0.8	0.4	0.98	0.4	0.5	0.0	0.99

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

Table 3.4.2.3 continued. 2004 temperature (degrees Celsius) comparison statistics

Station	River Mile	Layer	Measured			Simulated			Difference			R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
			Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile					
Bottom water temperature (ºC) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	1	26.5	20.3	30.1	26.6	20.7	30.4	0.1	0.4	0.3	0.99	0.3	0.4	0.0	1.0
USGS 02172070.9 Cooper River at Hwy 17	23.48	1	26.3	20.2	29.8	26.6	20.4	30.4	0.3	0.2	0.6	0.98	0.4	0.5	0.0	0.99
USGS 02172069.8 Wando River above Mount Pleasant	27.84	1	26.8	20.7	30.3	27.1	21.3	30.7	0.3	0.6	0.4	0.99	0.4	0.5	0.0	0.99
USGS 02172086.9 Ashley River near North Charleston	32.77	1	27.3	22.2	30.8	27.0	22.0	30.6	-0.3	-0.2	-0.2	0.93	0.6	0.8	0.0	0.98
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	1	27.1	21.8	29.9	27.1	22.0	30.3	0.1	0.2	0.4	0.93	0.5	0.6	0.0	0.98

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

3.4.3 Sediment Model Calibration and Validation

Following the hydrodynamic and salinity model calibration, the sediment transport component of the EFDC model was enabled. This section describes the setup and calibration of the three-dimensional sediment transport model.

3.4.3.1 Background

The primary purpose of the sediment transport model is to evaluate potential project effects on shoaling rates in the inner harbor federal navigation channel (i.e., this model study is not intended to address the shoaling in the offshore Fort Sumter Reach portion of the channel – see estimate for entrance channel shoaling in section 3.8.3). Brief summaries of the historic shoaling rates and estimates of sediment sources follow.

Before 1942, Charleston Harbor had been an almost self-maintaining harbor, and maintenance dredging of the federal navigation channel averaged 110,000 cubic yards per year. Based on the limited data available from that time period, the maintenance dredging was primarily removal of sandy material from the tops of several bars (Teeter et al. 1992).

The 1942 diversion of Santee River flows into the Cooper River dramatically increased shoaling in the federal navigation channel to as much as 10 million cubic yards per year before stabilizing at about 7.5 million cubic yards per year (Teeter 1989). The diversion increased average flows in the Cooper River from 600 cfs to about 15,600 cfs. This increased flow increased the sediment supply to the CHS and it reduced the vertical mixing, changing the estuary from a well-mixed regime to a partially mixed estuary regime. A partially mixed estuary has saline water intruding in the upstream direction along the bottom. This causes a predominance of flood currents along the bottom and a predominance of ebb currents near the surface. Suspended sediments settling toward the river bottom are transported back upstream by this circulation pattern, and as a result, this type of system is much more efficient at trapping sediments than a well-mixed estuary. The increase in project channel depth that occurred about the same time as the diversion did not have an important effect on the federal navigation channel shoaling (Teeter 1989).

The Cooper River Rediversion Project was completed in 1985 to counteract the high shoaling rates in Charleston Harbor. The rediversion lowered the Pinopolis Dam flows to a weekly average of 4,500 cfs. This decreased the sediment load and increased vertical mixing in the estuary, which resulted in about a 70 percent reduction in maintenance dredging rates.

On the basis of data from 2004 through 2012 provided by the USACE, present long-term average maintenance dredging of the inner harbor federal navigation channel (exclusive of the entrance channel) is about 1.6 million cubic yards per year. In addition to the federal navigation channel maintenance dredging, the SCSPA dredges approximately 340,000 cubic yards per year from its terminals in the harbor, and private facilities dredge approximately 230,000 cubic yards per year. The total inner harbor dredging is summarized in Table 3.4.3.1.

Table 3.4.3.1. Average Maintenance Dredging Rates

Dredging location	Average rate (million CY/yr)	Percentage of total
Federal navigation channel	1.62	74%
SCSPA terminals	0.34	16%
Private facilities	0.23	11%
Total dredging volume	2.19	

Sediment core data in the CHS indicate that the sediment bed varies in its composition, with the fraction of sand ranging between 5 and 88 percent, and the fraction of fine grained material (silts and clays) ranging between 12 and 95 percent. The fine-grained material in the estuary creates deposits of low-density unconsolidated mud. As described by Teeter (1989), this material has bulk densities between 1.22 and 1.05 grams per cubic centimeter (g/cm^3) and consistencies between that of mayonnaise and pea soup.

Multiple sources of the sediments cause shoaling in the harbor. These are sediments discharged from Pinopolis Dam; biogenic sources in the estuary (e.g., diatom phytoplankton, marsh vegetation); stormwater runoff from the surrounding watershed; shoreline erosion; ocean sediments; and other unknown sources. Table 3.4.3.2 summarizes estimates of these sources based on Teeter (1989) and Patterson (1983) and updated for this study.

Table 3.4.3.2. CHS sediment source estimates

Source	Average rate (million CY/yr)	Percentage of total
Inflow at Pinopolis Dam	0.24	11%
Plant production (marshes & diatom plankton)	0.37	17%
Storm water runoff	0.11	5%
Shoreline erosion	0.03	1%
Ocean and other sources	1.44	66%

Teeter (1989) estimates that the inflow of sediment from the discharge at Pinopolis Dam is about 0.24 million cubic yards per year. This is based on a study by the USGS (Patterson 1983) and adjusted for the post-rediversion average flow rate of 4,500 cfs. This represents only about 11 percent of the total sediment entering the CHS.

Biological activity can contribute both organic and inorganic sediment to the harbor. This material includes decaying plants and animals, and skeletal remains of diatoms and other plankton. Patterson (1983) estimates that erosion and biological activity in tidal marshes, combined with diatom production could account for 17 percent of sediment sources in the estuary, and Teeter (1989) used the same assumption. This estimate is highly uncertain.

Teeter et al. (2000) analyzed shoaling material at the Columbus Street Terminal. The organic fraction in these samples averages around 10 percent, which supports the conclusion that biological activity is a significant source of sediments. Although marsh areas may export some organic matter, marshes areas are also a sediment sink because low current velocity conditions in the marsh areas create a depositional environment for sediments. Therefore, the net effect of the marsh areas on the sediment budget is unknown. For this analysis, it is assumed that biological activity accounts for approximately 17 percent of sediment sources, the same estimate as that used by Teeter (1989).

Stormwater runoff contains suspended sediments and was estimated by Patterson (1983) to contribute 150,000 cubic yards per year to the harbor sediments. For this analysis, the storm water runoff sediment was estimated on the basis of 2001 land use types in the watershed, event mean concentrations (EMCs) for land use types given by Harper and Baker (2007), and stormwater flows predicted by the LSPC model. This yields an estimate of 110,000 cubic yards per year (5 percent of the total sources), which is similar in magnitude to the estimate made by Patterson (1983).

On the basis of analysis of National Ocean Survey data between 1933 and 1963 to determine changes in bottom elevation, Patterson (1983) estimated that shoreline erosion contributes 20,000 to 40,000 cubic yards per year.

The ocean is a source of fine sand and fine grain sediments that are carried into the harbor on flooding tides. Potential sources of fine-grained material from the ocean include sediment from the continental shelf and fluvial sediment discharge updrift from the CHS (Patterson 1983). Coastal storm events likely greatly increase this source as waves action increases suspended sediments along the coast and frontal passages cause subtidal variations in the water levels. The ocean source is unquantified, and it is combined with other unknown sources in Table 3.4.3.2. According to Teeter (1989), the unknown component could largely be composed of ocean sources.

Much of the material that shoals in the federal navigation channel is likely comprised of fine grained sediments already in the harbor. Teeter (1989) estimates that there is a large reservoir of unconsolidated mud on the floor of the estuary that is on the order of 20 to 30 million cubic yards in volume. These sediments are continuously resuspended by tidal currents and storm events and settle in lower energy areas. Unconsolidated mud also moves as a density current along the bottom and is generally not moved with the net estuarine circulation.

3.4.3.2 Sediment Boundary Conditions

The input boundary conditions to the sediment transport model are the input bed conditions, sediment concentrations for freshwater inflows, and sediment concentrations at the offshore open boundary. These input boundary conditions are described below.

3.4.3.2.1 Sediment Bed

On the basis of the following data received from the USACE, initial assumptions were made regarding sediment types in the CHS:

- South Carolina Department of Natural Resources SCECAP stations (1999–2011)
- Lower Town Creek sediment cores (2004)
- Sediment cores (1994 and 1996)
- Lower Harbor sediment cores (2010)
- Upper Harbor National Weapon Stations sediment cores (2009)
- Upper Harbor Pier P sediment cores (2011)
- Upper Harbor Pier Q sediment cores (2012)

The SCECAP data are based on grab samples of surficial sediments. The other six data sources are based on sediment core samples.

The percent of sand and non-cohesive material was plotted using GIS software to illustrate the dominant bed material (cohesive or non-cohesive). Figure 3.4.20 presents a map of these data. The darkest points represent sediment cores with more than 90 percent sand or non-cohesive materials. The lights points represent sediment cores with less than 30 percent sand or non-cohesive materials.

The sediment transport model was set up to include two classes of sediments: one class of cohesive sediments representing the silt and clay fraction, and one class of non-cohesive sediments representing the sand-sized particles. The initial distribution of cohesive and non-cohesive sediments assigned to the model bed is presented in Table 3.4.3.3. The grid plotted in Figure 3.4.20 illustrates the initial conditions for model calibration, where the lightest colored cells are dominated by cohesive sediments and the darkest cells are dominated by non-cohesive sediments.

The sediment bed was set up with 8 vertical layers, ranging in porosity from 0.85 at the surface layer (very low density sediments, with a bulk wet density of 1.3 g/cm^3), to 0.45 at the bottom layer (consolidated sediments with a bulk wet density of 1.9 g/cm^3).

Although generalized initial bed conditions were specified throughout the model based on the available sediment data, the appropriate existing bed conditions for each grid cell is not known *a priori*. Therefore, the model was run for a 4-day “spin-up” period during spring tide conditions that allowed initial adjustment of the sediment bed to the hydrodynamic conditions in the river. During this spin-up period the lower density surface layers of the sediment bed was eroded in high current areas. Following the spin-up, the model bed conditions were saved and used as the initial conditions for the model calibration simulation.

3.4.3.2.2 Inflow Boundary Concentrations

The model includes sediment concentrations specified for each of the freshwater inflows and the offshore open boundary. The sediment concentration for the inflow from Pinopolis Dam was specified on the basis of the sediment budget estimate given in Table 3.4.3.2. This is equivalent to a concentration of about 14 mg/L in the water discharged from the dam. This concentration is consistent with the values observed in areas upstream from the Tee that were collected as part of a previous study on the distribution of suspended sediments in the Cooper River (Althausen and Kjerfve 1992). This concentration was applied to the incoming flows at the Pinopolis Dam boundary.

Neiheisel and Weaver (1967) analyzed the composition of sediments in the Cooper River and concluded that only silt and clay are transported into the Cooper River from Lake Moultrie. Therefore, the Pinopolis Dam inflows were entirely assigned to the cohesive sediment class.

Sediment production from biological activity was added as a mass loading rate. The total loading rate estimated in Section 3.4.3.1 as 0.67 million cubic yards per year was added upstream from the federal navigation channel in the Ashley, Cooper and Wando Rivers. The fraction of this loading rate assigned to each river was apportioned by the fraction of the total estuary marsh area that occurs on each tributary.

Stormwater inflows from the LSPC model were assigned a constant concentration of 25 mg/L in the cohesive sediment class. This was not directly simulated by the LSPC model. The inflow concentration is based on the average EMC for the land uses in the watershed, as described in Section 3.4.3.1.

The offshore boundary concentrations were assigned a constant concentration of 15 mg/L of cohesive sediment. This boundary is sufficiently removed from the harbor that it does not have a significant effect on sediment transport in the harbor.

Table 3.4.3.3. Initial distribution of non-cohesive and cohesive sediments

% Non-cohesive	% Cohesive	Reach
30%	70%	Ashley River
80%	20%	Offshore
80%	20%	Charleston Harbor
30%	70%	Cooper River upstream from federal navigation channel
70%	30%	Wando and Cooper River marshes and tidal creeks
70%	30%	Wando River
70%	30%	Lower Cooper River and federal navigation channel

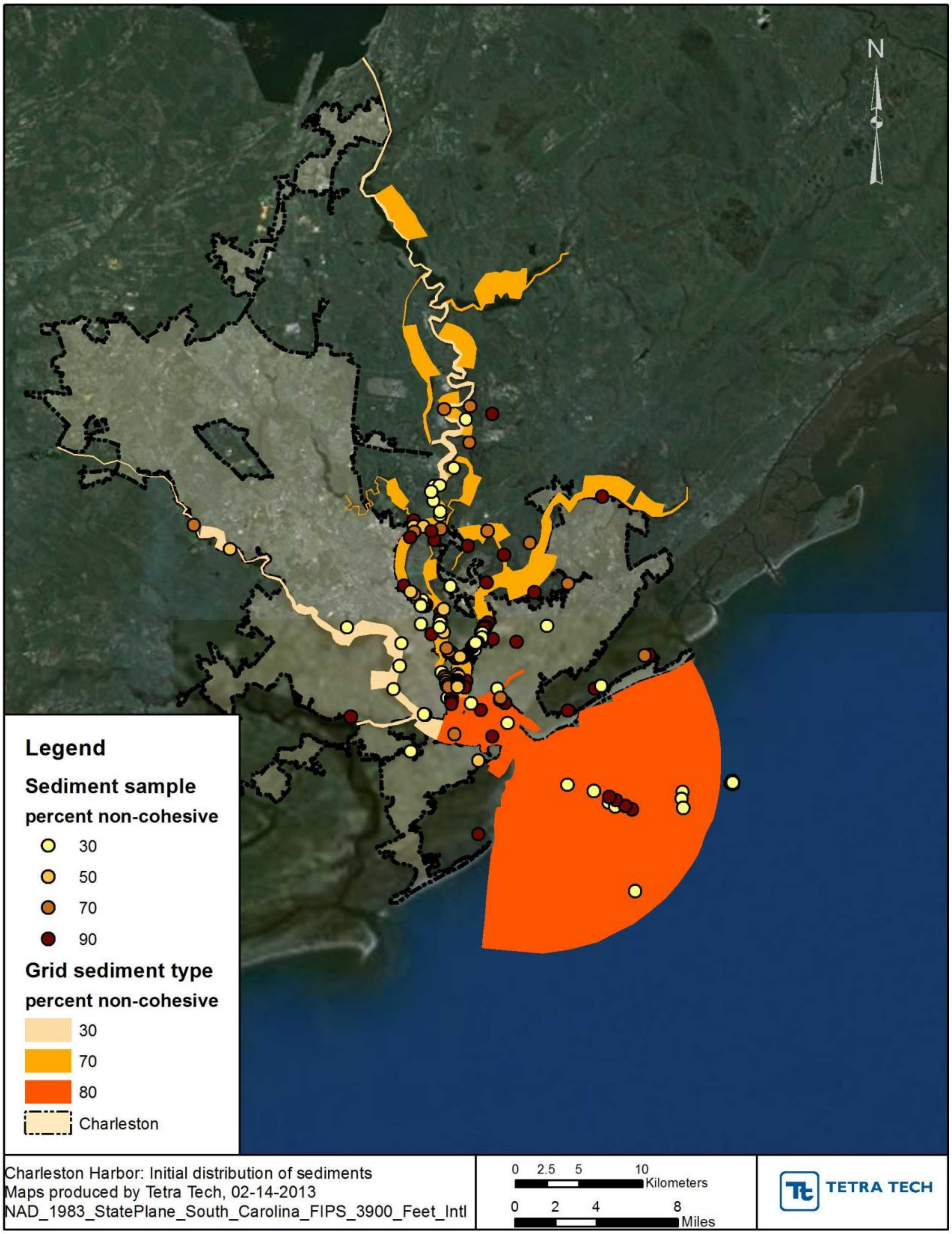


Figure 3.4.20 Initial distribution of non-cohesive and cohesive sediments

3.4.3.3 Sediment Model Options and Constants

The model options related to the sediment bed mechanics are listed in Table 3.4.3.4. Options for bed mechanics include: time invariant constant bed mechanical properties; simple consolidation calculation with constant coefficients; simple consolidation calculation with constant coefficients; simple consolidation calculation with variable coefficients; and complex consolidation with variable coefficients. The option for simple bed consolidation calculation with constant coefficients is used for this study. The deposited sediments are given an initial specified void ratio, and the bed consolidates to the minimum specified void ratio at a rate controlled by the input rate constant SEDVDRT.

The cohesive sediment input values are listed in Table 3.4.3.5. In this model, surface erosion of cohesive sediments occurs gradually when the flow-exerted bed stress is greater than a critical erosion or resuspension stress, τ_{ce}, which is dependent on the shear strength and density of the bed. The surface erosion is given by

$$J_o^r = w_r S_r = \frac{dm_e}{dt}\left(\frac{\tau_b - \tau_{ce}}{\tau_{ce}}\right)^{\alpha} : \ \tau_b \geq \tau_{ce}$$

where $\frac{dm_e}{dt}$ is the surface erosion rate per unit surface area of the bed, and τ_{ce} is the critical stress for surface erosion or resuspension (TAUR in Table 3.4.3.5). The base erosion rate and the critical stress for erosion depend on site-specific sediment characteristics and the degree of consolidation of the sediments. For this study, the selected sediment erosion rate option (IWRSP) calculates the base surface erosion rate of the sediment as a function of the bed density. This allows for faster erosion of low density deposits and slower erosion of dense consolidated sediments. The surface erosion rate varies with void ratio as follows:

$$\frac{dm_e}{dt} = \frac{dm_{e0}}{dt} exp\left(-\frac{1+\varepsilon_0}{1+\varepsilon}\right) \times \left(\frac{1+\varepsilon_0}{1+\varepsilon}\right)$$

where ε is the bed void ratio, ε_0 is the reference void ratio (VDRRSPO in Table 3.4.3.5), and $\frac{dm_{e0}}{dt}$ is the reference erosion rate per unit surface area (WRSPO in Table 3.4.3.5).

The constants for the noncohesive sediment class are listed in Table 3.4.3.6. The model internally calculates the non-cohesive sediment fall velocity and critical shear stress for erosion using the equations given by Van Rijn (1984a and 1984b). Equilibrium non-cohesive sediment concentration option 1 is used, which calculates the concentration using the approach described by Garcia and Parker (1991). Bedload option 2 is used, which employs the formulation of Engelund and Hansen (1967) to calculate bedload transport rates.

3.4.3.4 Sediment Model Calibration and Validation Comparisons

As described in Section 3.4.3.2.1, the model was used to simulate a 14-day spring tide condition spin-up period. The sediment bed conditions were saved and used as the initial conditions for the model calibration simulation. The model simulated the period from May 2, 2012 through May 30, 2012. This 28-day period includes two spring and neap tidal cycles and encompasses the period during which suspended sediment concentrations were sampled and analyzed by the USGS.

The model was calibrated by comparing the model results to measured suspended sediment concentrations and observed deposition rates. The primary variables adjusted during calibration were the cohesive sediment fall velocity, surface erosion rate, and critical shear stress for erosion. As explained by Lick (2009), comparison of simulated and observed suspended sediment concentration (SSC) alone can result in multiple solutions for these variables. Similar SSC results can be achieved with a higher erosion rate by also increasing the settling velocity. However, increasing the erosion rate and settling velocity will increase erosion and deposition patterns. Therefore, it is important to compare both SSC and deposition patterns when calibrating the sediment transport model to identify the appropriate values for settling velocity and surface erosion rate.

In 2012, the USGS collected discrete water samples for laboratory analysis of SSC at 12 locations in the CHS (Figure3.4.21). A total of 376 water samples were collected. The samples were collected during ebb and flood tides on May 7 and 8 (spring tide range conditions), and May 22 and 23 (neap tide range conditions). At each location, a vertical profile of samples was collected, including surface, mid-depth and bottom samples. A replicate sample was collected for each bottom sample using the same equipment and methods at the same location immediately after collecting the first sample. The variability in the replicate samples incorporates inhomogeneities in the water column SSC, and the variability inherent in laboratory handling and analysis of the samples. Note that the USGS analyzed the samples for SSC (USGS parameter code 80154), which is a different method than that used for total suspended sediments (TSS). USGS considers SSC as the most accurate way to measure the total amount of suspended material in a water sample collected from the flow in open channels. The TSS method is also commonly used, but can result in large errors (Gray and Glysson 2001). One notable difference is that TSS analysis is normally performed on an aliquot of the original sample. In contrast, the SSC method is performed on the entire sediment mass of the collected sample. Because the measured data are SSC, the model predicted sediment concentrations in this report are also referred to as SSC.

The relative percent difference (RPD) between the field and replicate samples (i.e., the difference between the field sample and replicate sample, divided by the average of the two values) reached up to 140 percent. The median absolute RPD of the field and replicate samples is 33 percent. Most of this variation is likely due to the high degree of variability in the river SSC, even when sampling the same location twice within one minute.

In 2003, ATM collected TSS samples in the Cooper River in support of the sediment transport modeling for the new terminal at the Naval Complex in North Charleston. TSS measurements from 2003 are used in this study to validation the model simulation of water column suspended sediment concentrations. The locations of the 2003 measurements used for comparison are shown in Figure 3.4.22. The 2003 data collection was focused on supporting and analysis of the proposed marine container terminal on Daniel Island Reach. Therefore, the data are limited and do not include sampling throughout the CHS.

Table 3.4.3.4. EFDC sediment bed constants

Variable	Value	Units
IBMECH: bed consolidation option	1	-
BEDPORC: porosity of depositing noncohesive sediment	0.45	-
SEDVDRD: void ratio of depositing cohesive sediment	9	-
SEDVDRM: minimum cohesive sediment bed void ratio	0.82	-
SEDVDRT: bed consolidation rate constant	10.E6	sec

Table 3.4.3.5. EFDC cohesive sediment constants

Variable	Value	Units
SDEN: sediment specific volume	3.85E-07	m^3/g
SSG: sediment specific gravity	2.6	-
WSEDO: constant sediment settling velocity	0.0005	m/s
TAUD: boundary stress below which deposition takes place	1.E-04	$(m/s)^2$
IWRSP: resuspension rate option	4	-
WRSPO: reference surface erosion rate	0.025	$g/m^2/s$
VDRRSPO: reference void ratio for resuspension rate	9	-
TAUR: boundary stress above which surface erosion occurs	2.5E-04	$(m/s)^2$
TEXP: exponent in erosion rate formula	1	-

Table 3.4.3.6. EFDC noncohesive sediment constants

Variable	Value	Units
SDEN: sediment specific volume	3.77E-07	m^3/g
SSG: sediment specific gravity	2.65	-
SNDDIA: representative diameter of sediment class	1.8E-4	m
ISNDEQ: reference equilibrium concentration option	1	-
ISBDLD: bed load option	2	-

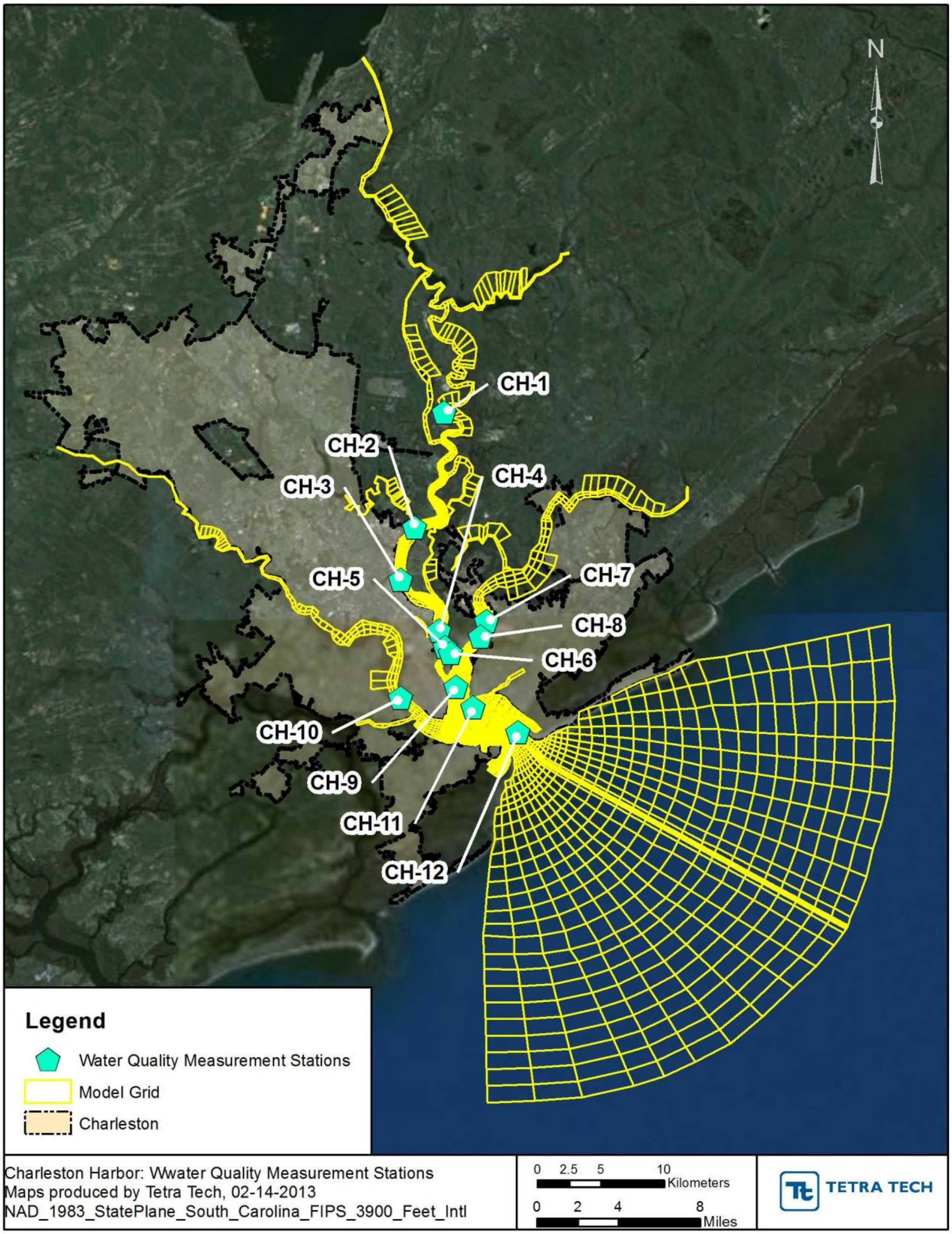


Figure 3.4.21 2012 SSC measurement locations

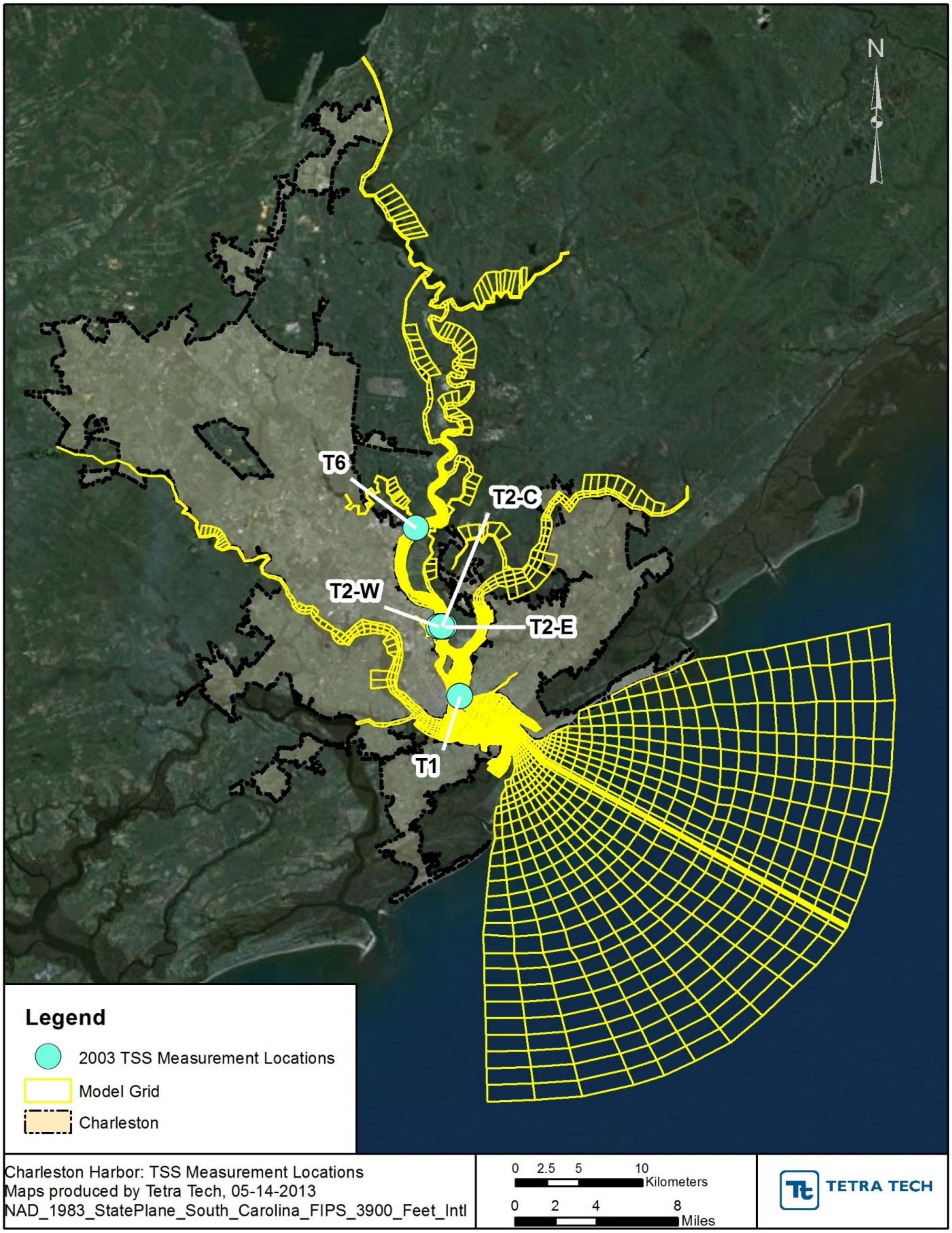


Figure 3.4.22. 2003 TSS measurement locations

Comparisons of measured and simulated SSC are provided in Attachment A-3 (a total of 24 plots). In these figures, measured samples below the Laboratory Reporting Level (LRL) of 15 mg/L are plotted at the LRL. The model over predicts the bed erosion in the upper Cooper River near Mobay (station CH-1 in Figures C-1 and C-2). This is well upstream from the federal navigation channel. At the upstream end of the federal navigation channel at Ordnance Range (station CH-2 in Figures C-3 and C-4), the model is close to the range of observed values. The model tends to slightly over predict concentrations during spring tide conditions and under predict concentrations during neap tide conditions. In Navy Yard Reach (station CH-3 in Figure C-5 and C-6) the model is in good agreement with observed concentrations during spring tide conditions and tends to under predict concentrations during neap tide conditions. In Daniel Island Reach, the model generally agrees with the observed concentrations for both spring and neap tide conditions (Figures C-7 and C-8), although it does not match the high near-bend concentrations in excess of 200 mg/L observed on May 7, 2012. The model is in good agreement the observed concentrations in Myers Bend and Drum Island Reach, as shown by stations CH-5 and CH-6 (Figures C-9 through C-12). The model is in good agreement with the observed concentrations upstream and downstream from the Wando Turning Basin (stations CH-7 and CH-8 in Figures C-13 through C-16), with the exception of one spring tide high SSC event at CH-7 with concentrations in excess of 100 mg/L. In the middle harbor, in Hog Island Reach (station CH-9 in Figures C-17 and C-18) the model is in general agreement with observed SSC during spring tide, but the model under predicts slightly during neap tide conditions. On the Ashley River (station CH-10 in Figures C19 and C-20), the model generally under predicts SSC. The model also tends to under predict concentrations in Bennis Reach and the ocean inlet (stations CH-11 and CH-12 in Figures C-21 through C-24). The under prediction near the ocean entrance is likely the results of the fact that coastal processes, such as wind waves and the wave induced effects on sediment erosion are not included. Therefore, the model should not be used to predict changes in the inlet area.

Overall the model is in general agreement with the observed SSC data. The model does not match individual observed concentrations accurately, but this is expected in a highly dynamic estuary where the observed replicate samples show a high degree of variability. At some stations the model over predicts during spring tide conditions and under predicts during neap tide conditions. The model results are generally smoother and more consistent than the observed data, as expected.

The simulated deposition rates are shown in Figure 3.4.23. For reference, the federal navigation channel reaches are shown in Figure 3.4.24. Figure 3.4.25 compares the simulated deposition rates in each reach of the federal navigation channel to the long-term (2004 through 2012) average maintenance dredging rates in the channel reaches.

The model correctly predicts the overall order of magnitudes of deposition in the federal navigation channel. This includes little to no deposition in the inlet and in the reaches between Daniel Island Bend and Port Terminal Reach. The model also correctly predicts the order of magnitude of deposition in the Wando Reaches and in the reaches near the Charleston peninsula. Overall, the total shoaling rate in the federal navigation channel is under predicted by the model by 19 percent.

However, there are important differences between the simulated and observed deposition rates that should be noted. The model under predicts the deposition in Ordnance Range and Ordnance Turning basin by more than 200,000 cubic yards per year. At Ordnance Reach the model correctly predicts a net upriver current along the bottom extending just upriver from this location, which should cause a turbidity maximum in this region. Nonetheless, the model predicted deposition is not as high as the historic maintenance dredging rate in this area.

The model does predict shoaling along the west side of Daniel Island. However, the sedimentation in this area is also well below the observed rates. Shoaling in this area is governed by a complex pattern of tidal flows through Daniel Island Bend and Daniel Island Reach, and this model may not be sufficiently high resolution to accurately predict the magnitude of shoaling in this area.

The model was used to simulate the 2003 time period to validate the model prediction of suspended sediment concentration. The comparisons of simulated and predicted suspended sediment concentrations are also shown in Attachment A-3 (Figures C-25 though C-38). The comparisons show reasonable agreement between the simulated and observed concentrations in the harbor at Horse Reach (Station T-1) and at the upstream end of the federal navigation channel at Ordnance Reach (Station T-6). However, the model under predicts suspended sediment concentrations in Daniel Island Reach. This is consistent with the fact that the model under predicts deposition in this area.

Overall, the model is sufficiently calibrated to estimate the *relative* changes (that is, the percent changes) in long-term maintenance dredging rates and SSC caused by the Post 45 Study. Given the uncertainty in the model predictions, the high degree of variability in the observed SSC data, and the uncertainty in the observed sedimentation rates (maintenance dredging records are assumed here to be indicative of long-term sedimentation rates in the channel); the model should not be used to predict absolute changes in shoaling rates.

Given that the sediment budget contains a high amount of uncertainty, the model should not be used to simulate sediment transport over long time periods. Instead, the best approach to modeling potential changes in suspended sediment concentrations and federal navigation channel shoaling rates is to simulate the resuspension and deposition of sediments over a one-month period that includes typical spring and neap tidal conditions. Because the model hydrodynamics are in reasonably good agreement with the observed tides and currents, the model should correctly predict areas of high and low current velocities that correspond to areas of low and high shoaling potential, respectively. This approach is suitable to identify the patterns of SSC and deposition in the federal navigation channel that are indicative of long term trends.

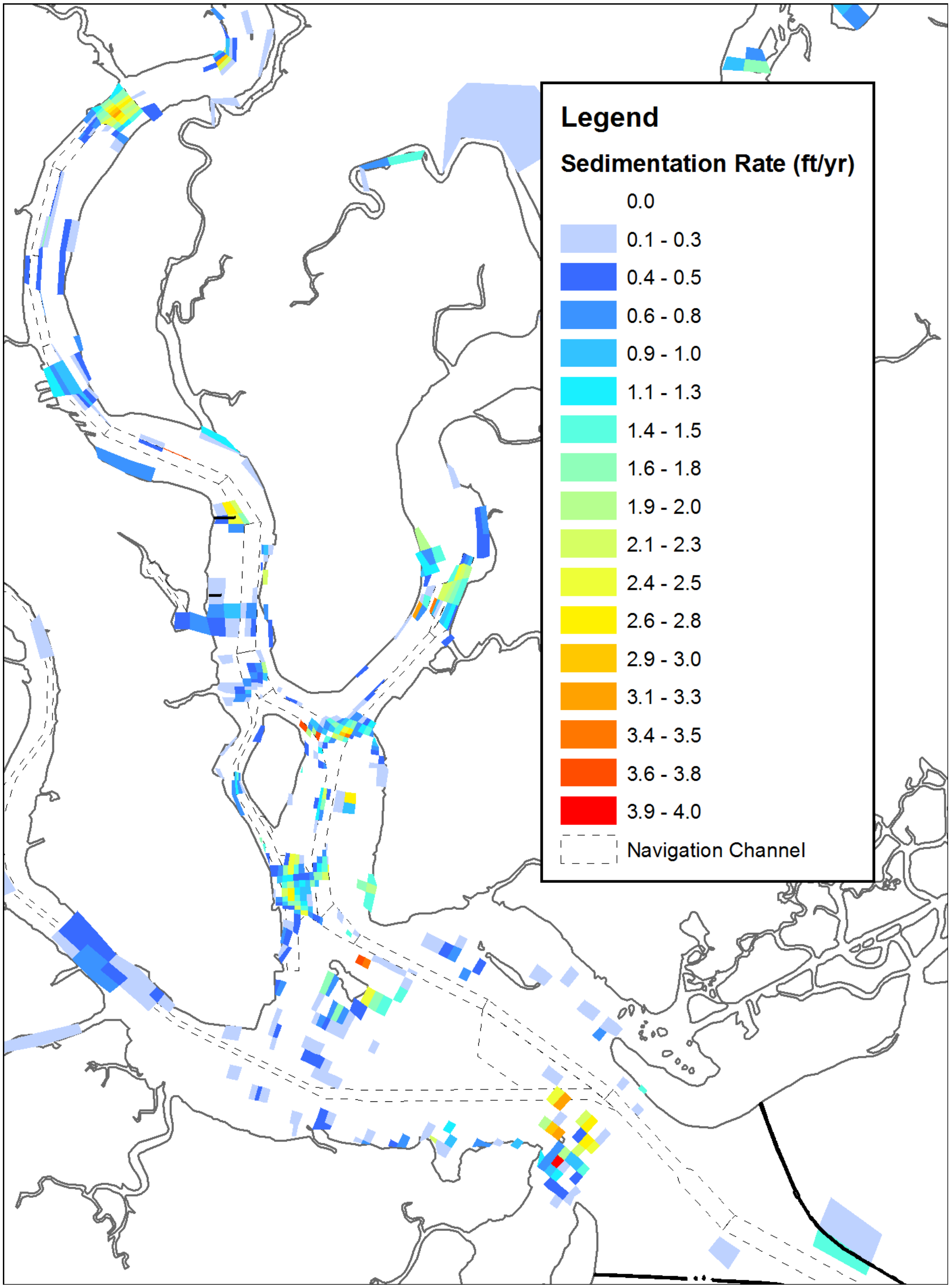


Figure 3.4.23. Simulated Deposition Rates

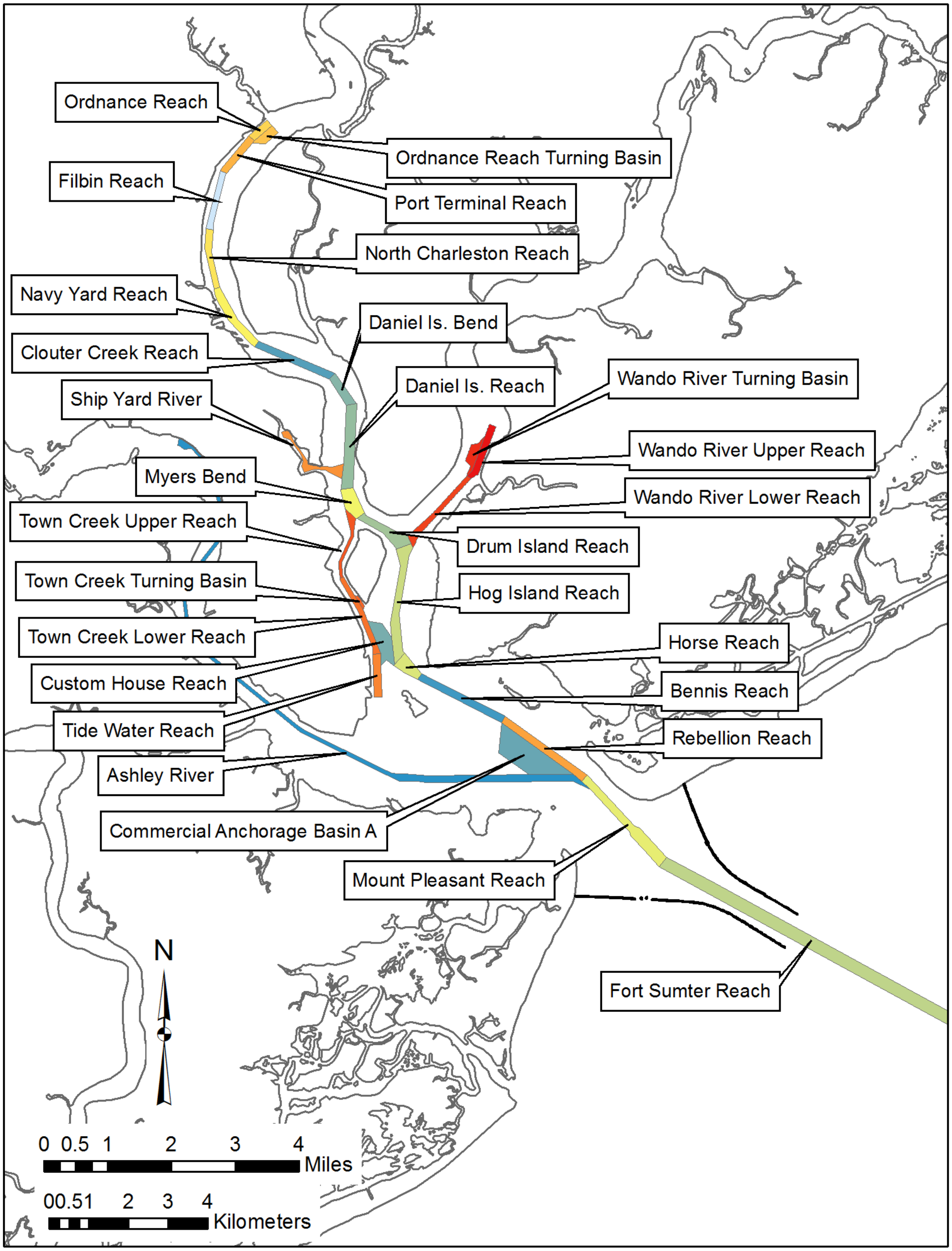


Figure 3.4.24. Charleston Harbor System Federal navigation channel reaches

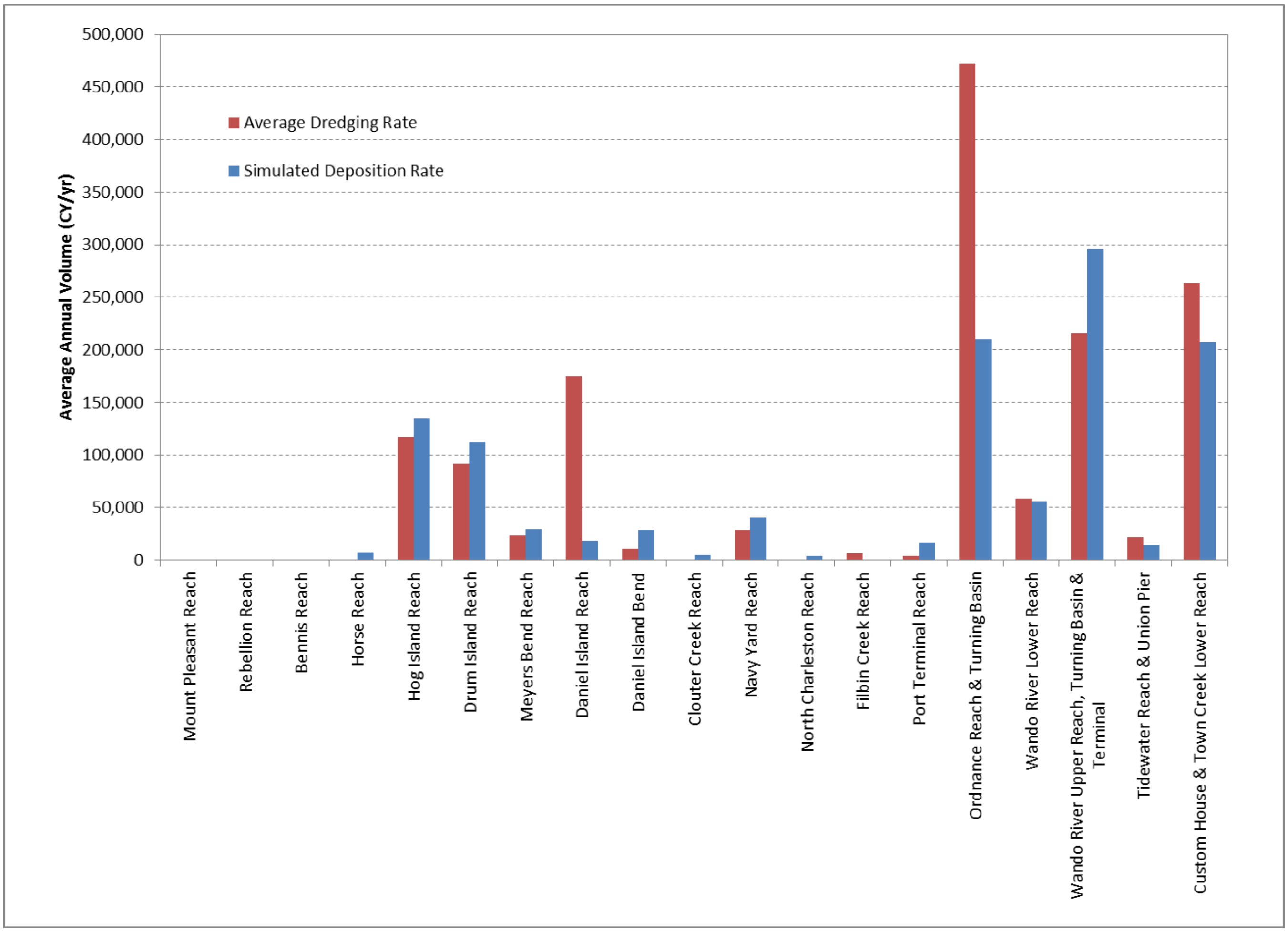


Figure 3.4.25. Observed and simulated sedimentation rates by federal navigation channel reach

3.4.4 Water Quality Model Validation

The water quality model results presented here are based on a validation of 2004 water quality conditions using the 2012 hydrodynamic model calibration parameters. The water quality parameters are the same parameters used in the final calibration of the BCDCOG DO TMDL model with exceptions made to address comments received by agencies and the USACE in March and April 2013. Changes to water quality parameters were described in previous sections. The same stations used for the calibration of the BCDCOG DO TMDL model were used to validate the water quality model for 2004. Table 3.4.4.1 lists the 2004 stations used to validate water quality and Figure 3.4.26 shows their location in the CHS. Longitudinal comparisons of observed and simulated values indicate the model is generally within range of measured DO in the CHS (Figures 3.4.27 and 3.4.28). Statistically (Table 3.4.4.2), the simulated DO is within the mean of measured values. The simulated DO is within range of the 5^{th} percentile along the Cooper River and at Fort Sumter. For the purpose of this modeling effort, DHEC has found these results are reasonable and would be expected to allow the SCSPA and USACE evaluate the effects of harbor deepening on DO in the CHS (Personal communication with Mr. Wade Cantrell, May 3, 2013). The results of this validation illustrate the model would be capable of evaluating relative changes in DO in areas of the CHS.

Comparing observed and simulated longitudinal Figures 3.4.29 through 3.4.41 and the time series figures in Attachment A-4 illustrate the model's ability to capture trends in various water quality parameters during critical summer months. In the previous BCDCOG DO TMDL modeling effort outputs for DO and total organic carbon at discrete monitoring stations (JJG stations) were similar to this modeling effort in the harbor and Cooper Rivers. Model results during this effort on the Ashley and Wando Rivers are slightly over predicting total organic carbon. Increased total organic carbon would be expected to lower DO. Though the model is under predicting DO, the range of modeled results is within range of the mean and lowest measured values, and comparable to the calibration results of the BCDCOG DO TMDL model (Figures 3.4.29, 3.4.30, and 3.4.31). DO deficit is also presented in Figures 3.4.32 and 3.4.33 to illustrate the difference between the system's capacity to carry DO and the actual (observed or simulated) DO.

The DO deficit is defined as the difference between the DO saturation concentration (a function of temperature and salinity), and the simulated DO. DO is a function of many variables, including salinity, temperature, reaeration, plant production, plant respiration, and oxidation of oxygen demanding materials. It is helpful to evaluate the DO deficit to filter out some of the effects of salinity and temperature variations on DO so that the effects of the other variables can be more directly evaluated. However, note that the DO deficit does not remove all the influence of temperature variation; multiple kinetic rates (the organic carbon oxidation rate, for example) and the SOD are still affected by temperature changes that, in turn, also affect the DO.

Loads influencing other water quality parameters (ammonia, total phosphorus, and chlorophyll-*a*) would be held constant during alternative evaluation. Because loads into the model will not be changed during alternative evaluations, the relative quality of simulated water quality is comparable to the BCDCOG model and adequate for use in this effort.

Table 3.4.4.1. 2004 Water Quality Measurement Locations

USGS station ID	Description	River mile	I	J
02172020	West Branch Cooper River at Pimlico near Moncks Corner	57.16	46	167
02172050	Cooper River near Goose Creek	50.57	65	161
02172053	Cooper River at Mobay	41.04	65	128
2172067.7	Cooper River at I-526	31.00	56	79
2172069.8	Wando River above Mount Pleasant (I-526)	27.84	75	68
2172070.9	Cooper River at Hwy 17	23.48	81	38
2172084	Ashley River at Bakers Landing	39.97	44	42
2172086.9	Ashley River near North Charleston (I-526)	32.77	44	17
2172100	Charleston Harbor at Fort Sumter	18.94	115	19
JJG-WQ-A1	Ashley River near mouth	25.57	22	6
JJG-WQ-A2	Ashley River downstream (ds) of 526 Bridge	31.55	4	7
JJG-WQ-A3	Ashley River at Magnolia Gardens	38.95	5	31
JJG-WQ-A4	Ashley River upstream of Dorchester	44.11	5	46
JJG-WQ-C1	Lower Cooper River	27.10	24	38
JJG-WQ-C2	Cooper River at 526 Bridge	30.87	23	50
JJG-WQ-C3	Cooper River downstream (ds) of Steam Plant	36.24	24	67
JJG-WQ-C4	Cooper River upstream (us) of Mobay	40.93	45	44
JJG-WQ-C5	Cooper River at Dean's Hall	49.93	45	78
JJG-WQ-H1	Charleston Harbor at the mouth	18.44	52	16
JJG-WQ-H2	Charleston Harbor – Channel Center	22.28	35	16
JJG-WQ-H3	Charleston Harbor upstream (us) of Hwy 17	24.18	31	26
JJG-WQ-W1	Wando River at 526 Bridge	27.90	32	40
JJG-WQ-W2	Wando River near Cainhoy	35.05	34	56

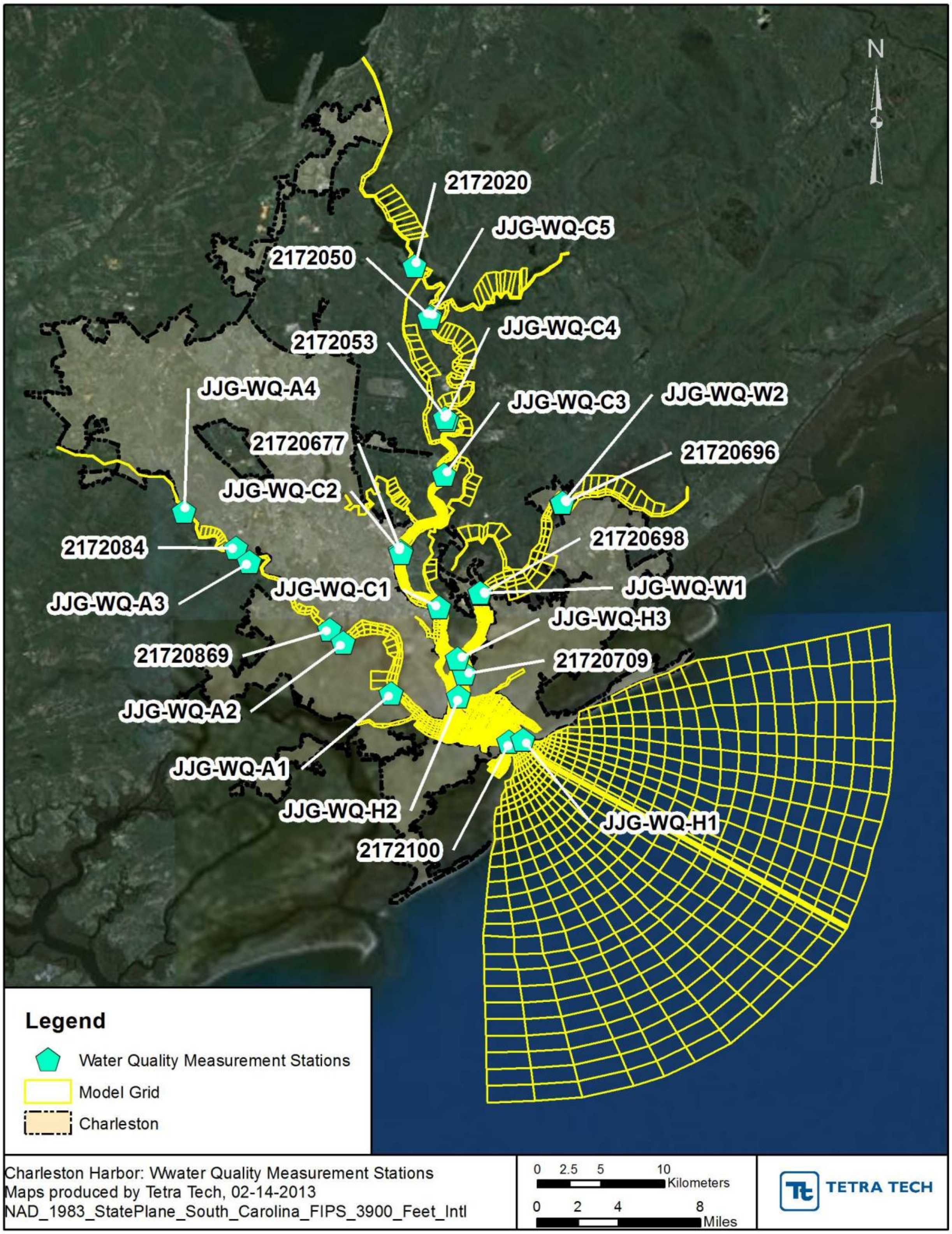


Figure 3.4.26. Water Quality Measurement Locations for 2004

Table 3.4.4.2. Summary percentile statistics for daily average dissolved oxygen (mg/L)

Station	River Mile	Layer	Measured Mean	Measured 5 %tile	Measured 95 %tile	Simulated Mean	Simulated 5 %tile	Simulated 95 %tile	Difference Mean	Difference 5 %tile	Difference 95 %tile	R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
Dissolved oxygen (mg/L) 04/15/2004 - 11/01/2004																
USGS 02172020 Cooper River near Moncks Corner	57.16	5	5.711	4.640	7.051	5.991	5.447	6.749	0.28	0.807	-0.302	0.20	0.567	0.690	0.118	0.60
USGS 02172050 Cooper River near Goose Creek, SC	50.57	5	5.664	4.678	7.057	5.690	4.869	7.244	0.026	0.192	0.188	0.59	0.392	0.502	0.088	0.87
USGS 02172053 Cooper River at Mobay	41.04	4	5.210	3.689	7.128	4.769	3.875	6.460	-0.442	0.185	-0.668	0.63	0.631	0.741	0.147	0.82
USGS 02172084 Ashley River at Bakers Landing	39.97	4	3.995	3.263	5.273	3.565	1.546	4.930	-0.430	-1.717	-0.343	0.03	0.921	1.160	0.306	0.47
Surface dissolved oxygen (mg/L) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	6	5.467	4.491	6.829	5.394	4.734	6.678	-0.073	0.243	-0.151	0.5	0.371	0.496	0.091	0.83
USGS 02172070.9 Cooper River at Hwy 17	23.48	6	5.673	4.644	6.719	5.561	5.138	6.044	-0.112	0.495	-0.674	0.33	0.486	0.549	0.098	0.62
USGS 02172069.8 Wando River above Mount Pleasant	27.84	6	5.454	4.495	6.591	5.563	5.007	6.600	0.109	0.513	0.010	0.15	0.519	0.634	0.115	0.62
USGS 02172086.9 Ashley River near North Charleston	32.77	6	4.661	3.344	6.158	5.698	4.793	6.650	1.037	1.449	0.492	0.00	1.166	1.397	0.271	0.32
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	6	5.449	4.591	7.105	5.488	4.959	6.103	0.039	0.368	-1.002	0.38	0.410	0.522	0.095	0.70
Mid-depth dissolved oxygen (mg/L) 04/15/2004 - 11/01/2004																
USGS 02172067.7 Cooper River at I-526	31.00	3	5.216	4.246	7.354	4.799	4.086	6.288	-0.417	-0.160	-1.065	0.74	0.490	0.593	0.118	0.83
USGS 02172070.9 Cooper River at Hwy 17	23.48	3	4.839	3.675	6.627	5.130	4.566	5.794	0.291	0.891	-0.833	0.45	0.670	0.769	0.154	0.63
USGS 02172069.6 Wando River at Cainhoy	35.05	4	5.215	3.895	6.356	5.539	4.758	6.220	0.324	0.863	-0.136	0.26	0.562	0.710	0.132	0.62
USGS 02172069.8 Wando River above Mount Pleasant	27.84	3	5.327	4.045	6.943	5.355	4.748	6.456	0.028	0.703	-0.487	0.46	0.449	0.550	0.103	0.77

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

Table 3.4.4.2 continued. Summary percentile statistics for daily average dissolved oxygen (mg/L)

Station	River Mile	Layer	Measured			Simulated			Difference			R^2	Mean Abs Err	RMS Err	Norm RMS Err	Index of Agrmt
			Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile	Mean	5 %tile	95 %tile					
Bottom dissolved oxygen (mg/L) 04/15/2004 - 11/01/2004																
USGS 02172086.9 Ashley River near North Charleston	32.77	1	4.659	3.467	5.927	4.109	2.082	5.940	-0.550	-1.386	0.012	0.02	1.149	1.432	0.328	0.28
USGS 02172100 Charleston Harbor at Fort Sumter	18.94	1	5.090	3.595	6.406	4.837	4.077	5.753	-0.252	0.482	-0.653	0.10	0.699	0.838	0.169	0.54

%tile = percentile; R^2= coefficient of determination; Abs Err = absolute mean error; RMS Err = root mean squared error; Norm RMS Err = normalized root mean square error; Agrmt = agreement.

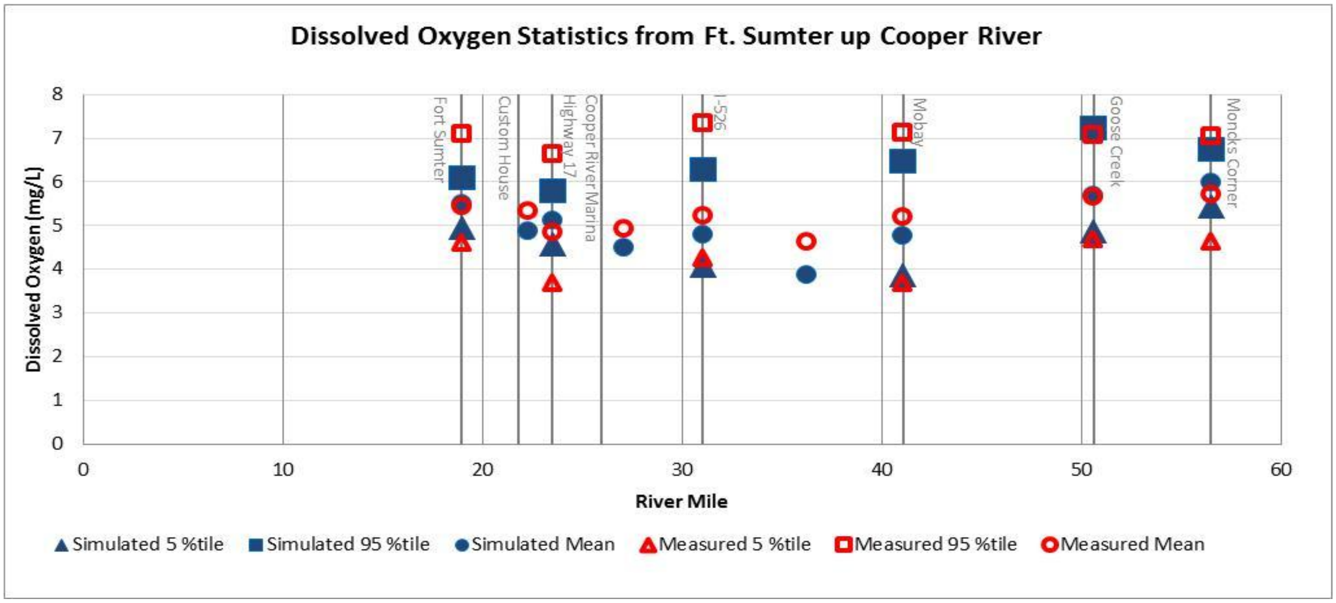


Figure 3.4.27. Longitudinal dissolved oxygen on the Cooper River

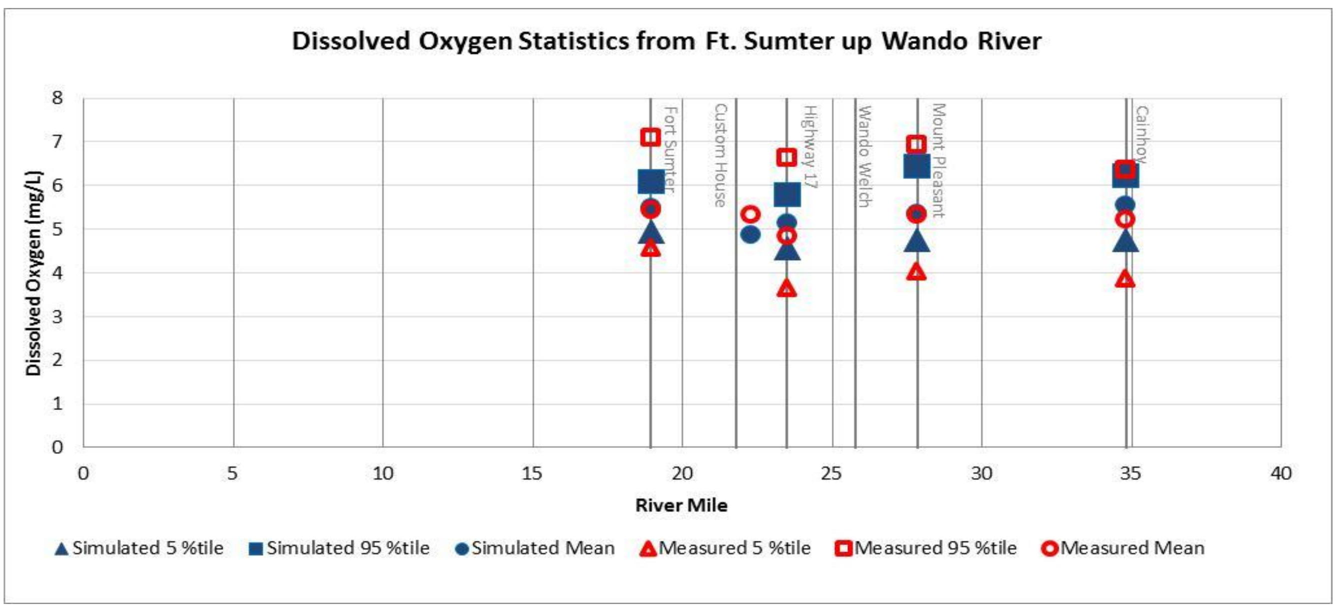


Figure 3.4.28. Longitudinal dissolved oxygen on the Wando River

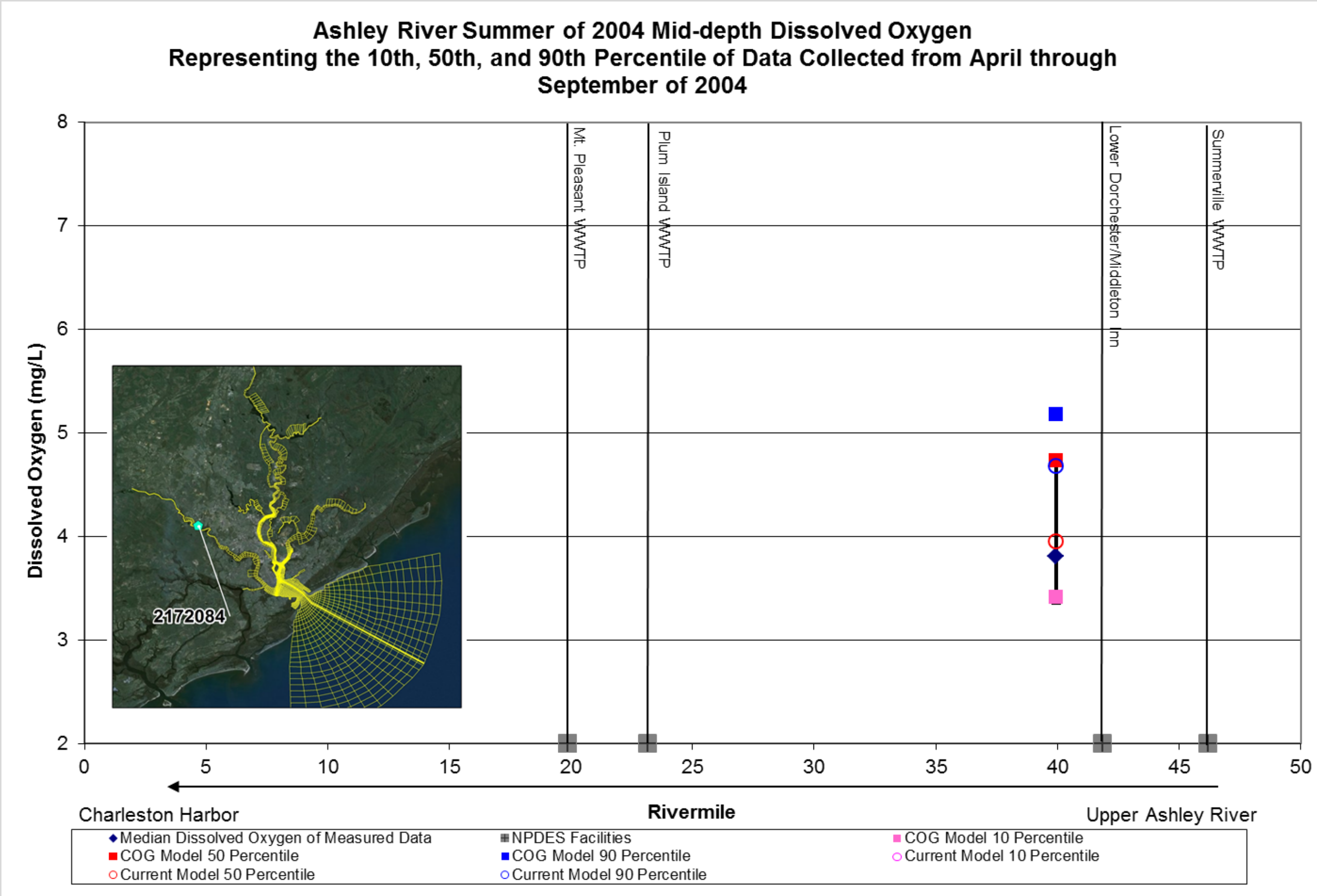


Figure 3.4.29. Longitudinal mid-depth dissolved oxygen on the Ashley River comparing the BCDCOG results with the current model

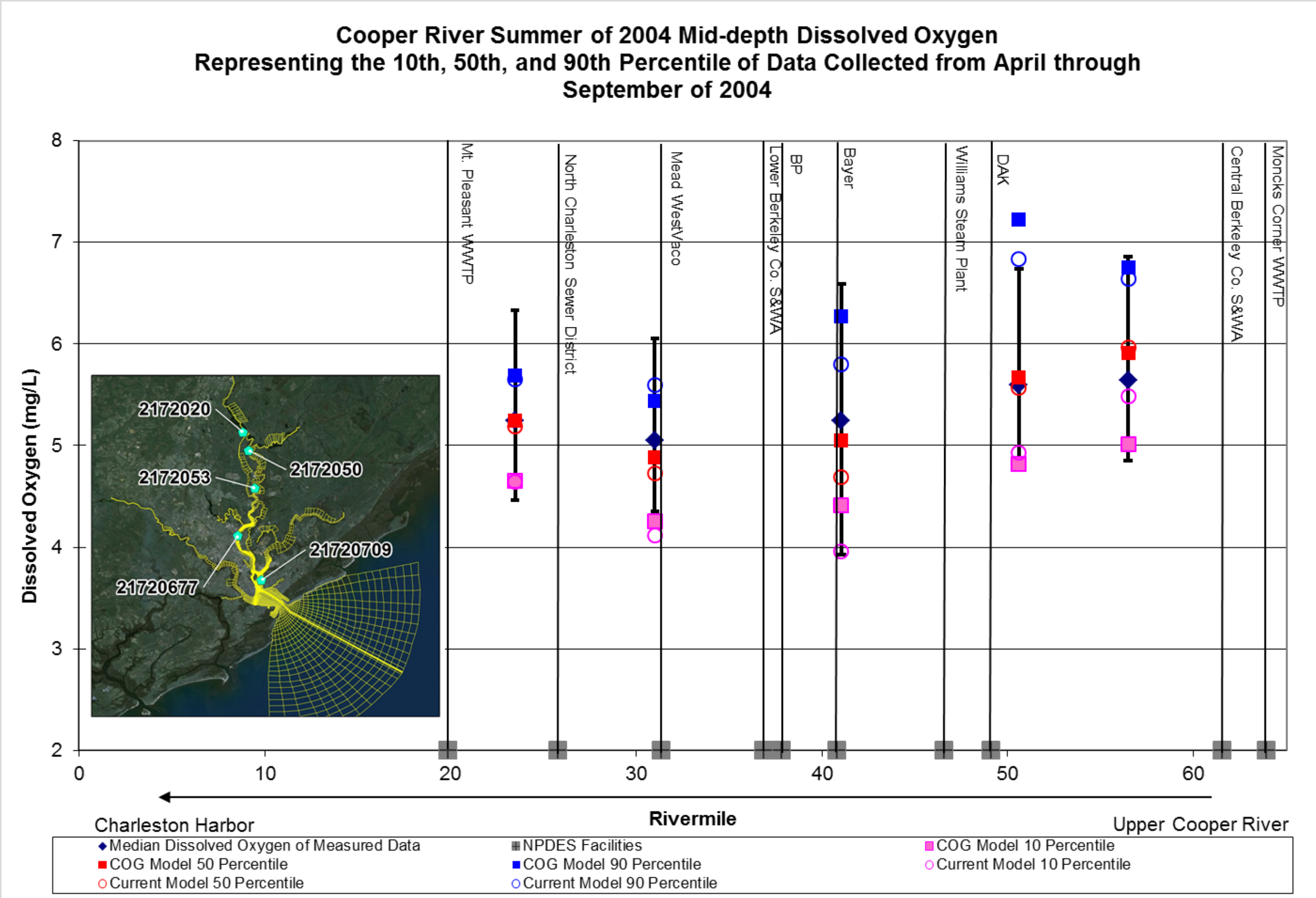


Figure 3.4.30. Longitudinal mid-depth dissolved oxygen on the Cooper River comparing the BCDCOG results with the current model

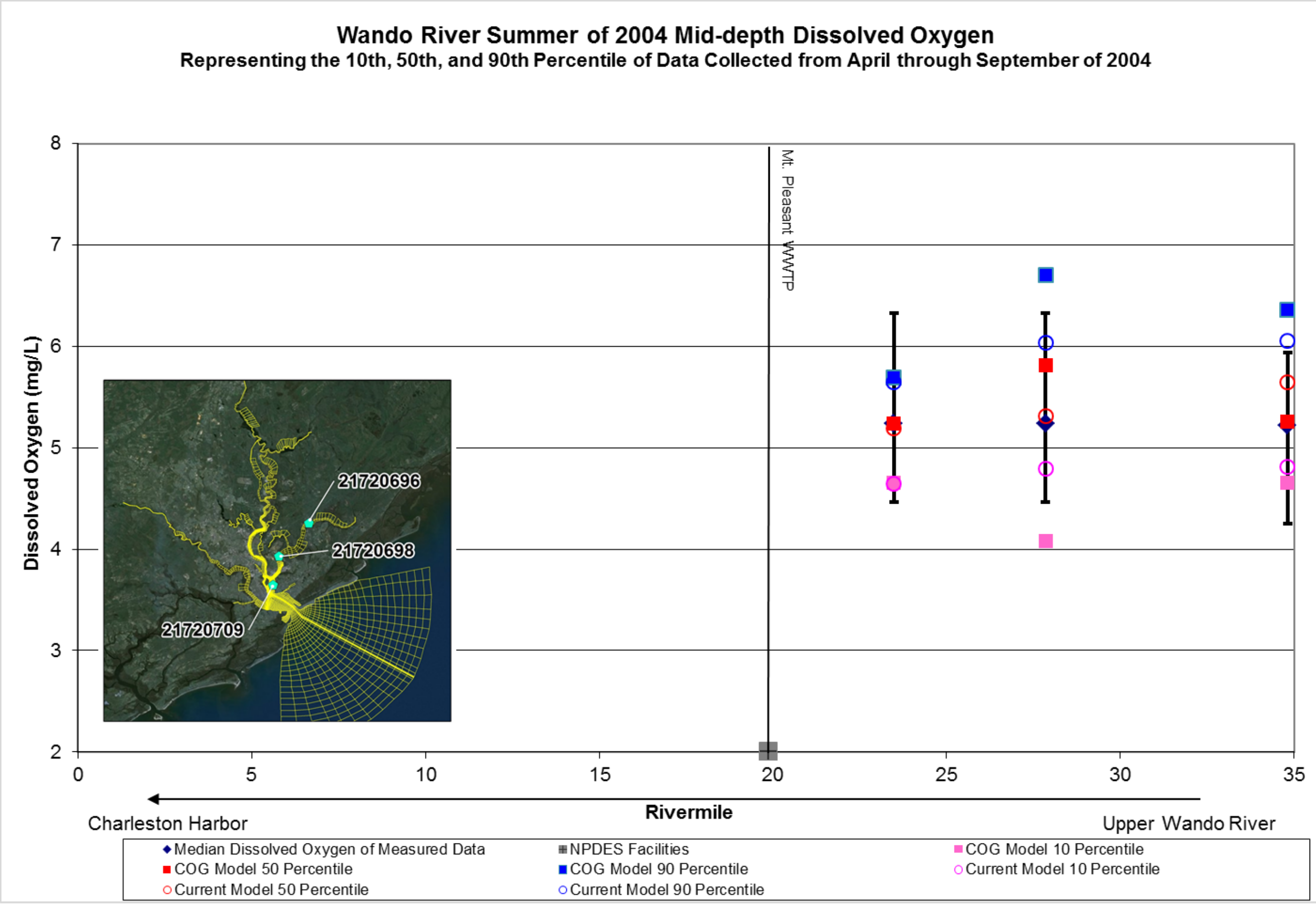


Figure 3.4.31. Longitudinal mid-depth dissolved oxygen on the Wando River comparing the BCDCOG results with the current model

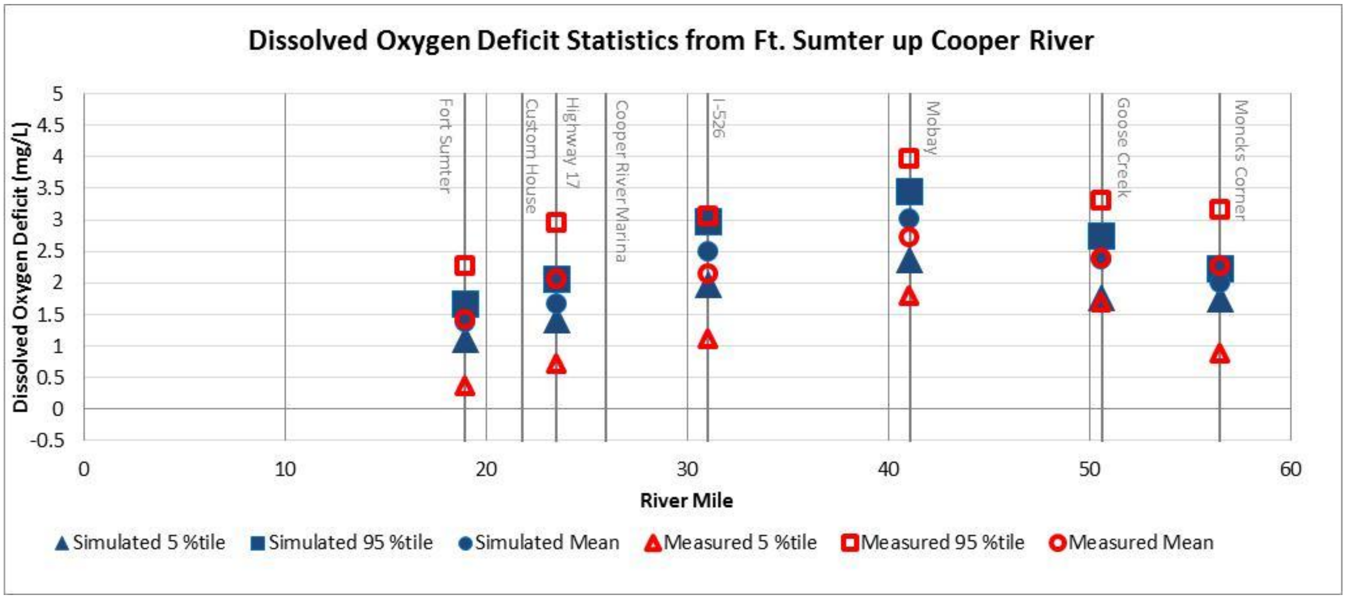


Figure 3.4.32. Longitudinal dissolved oxygen deficit on the Cooper River

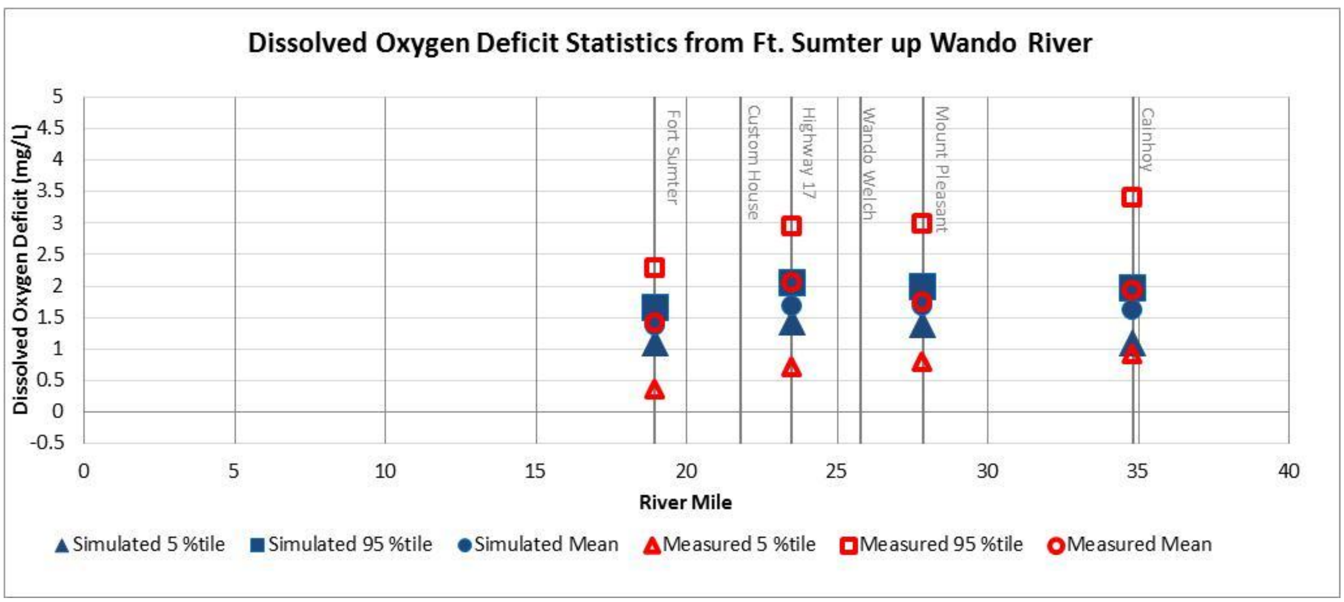


Figure 3.4.33. Longitudinal dissolved oxygen deficit on the Wando River

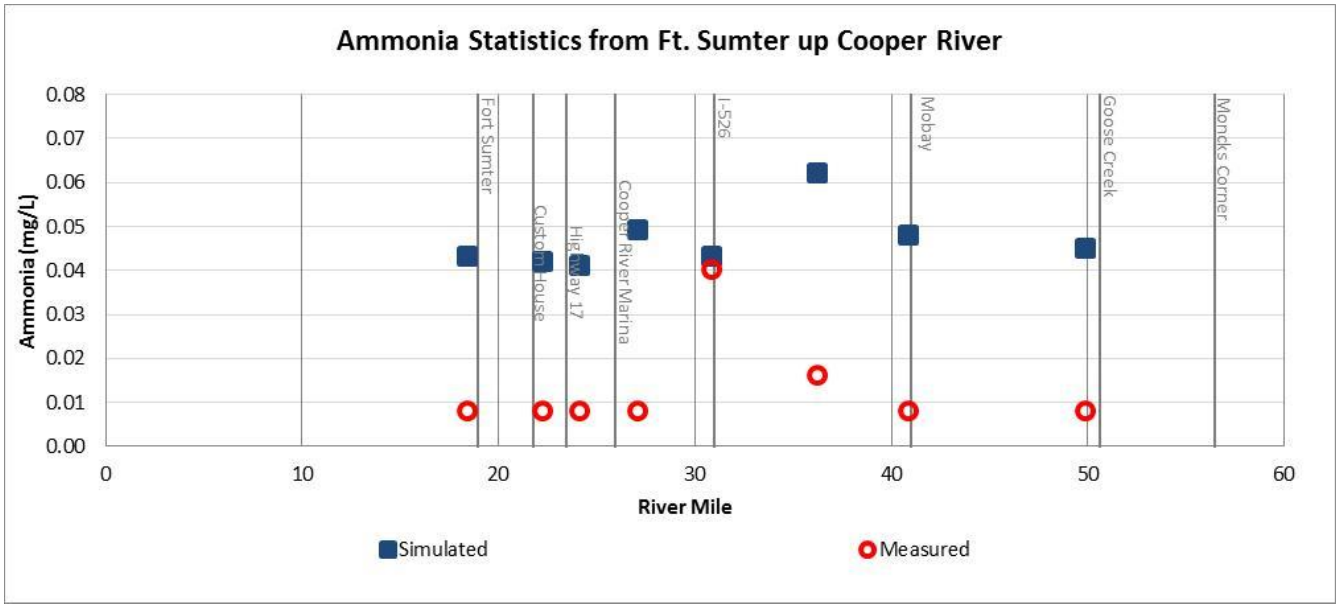


Figure 3.4.34. Longitudinal ammonia on the Cooper River

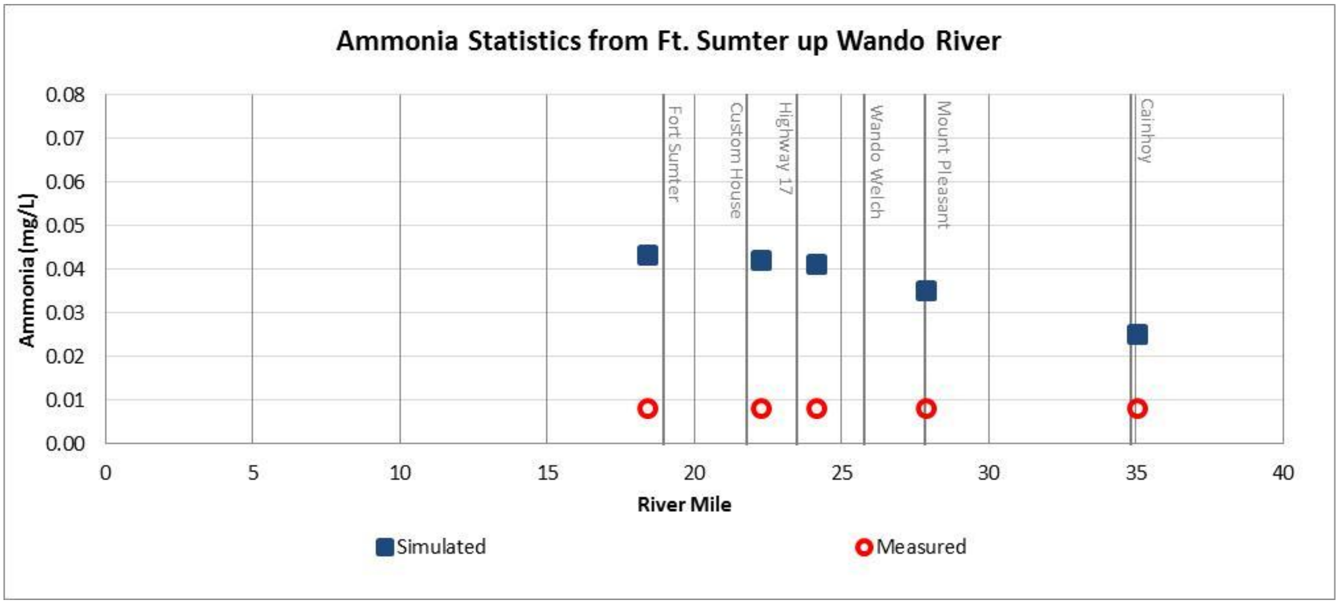


Figure 3.4.35. Longitudinal ammonia on the Wando River

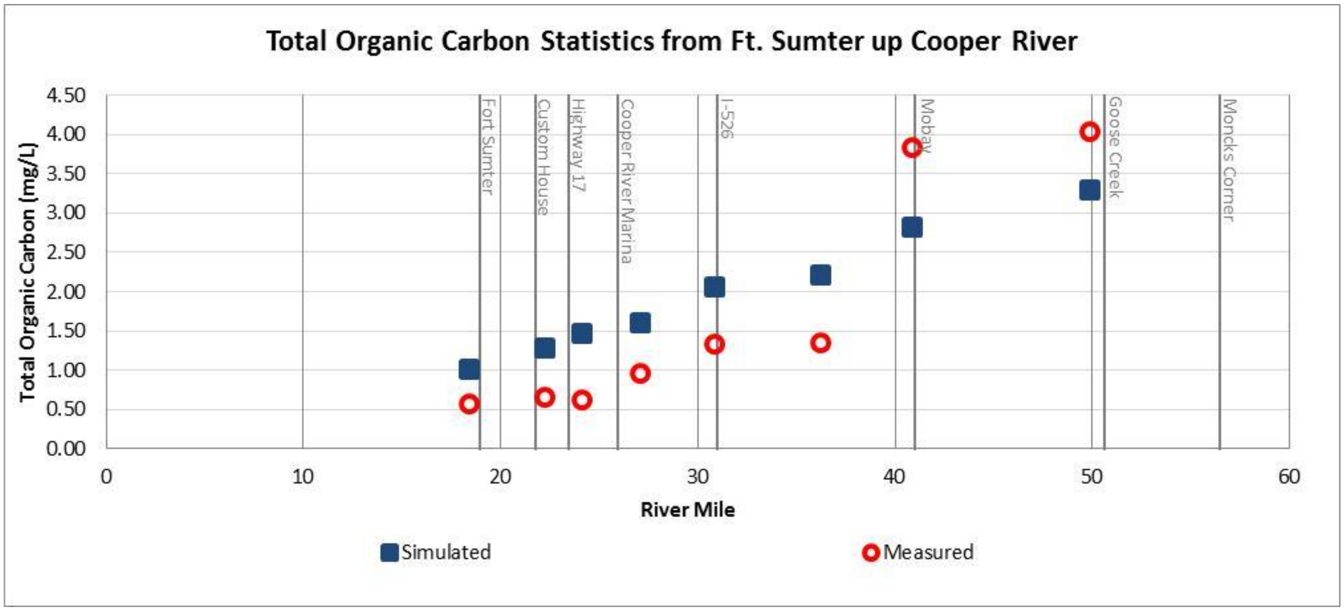


Figure 3.4.36. Longitudinal total organic carbon on the Cooper River

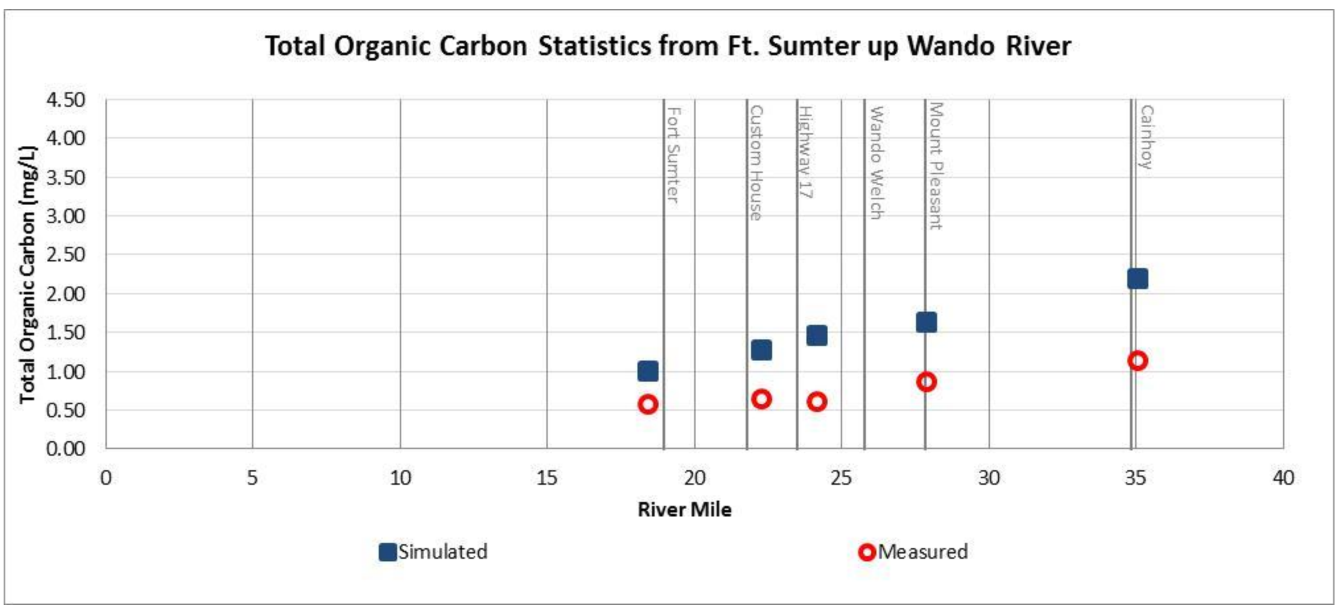


Figure 3.4.37. Longitudinal total organic carbon on the Wando River

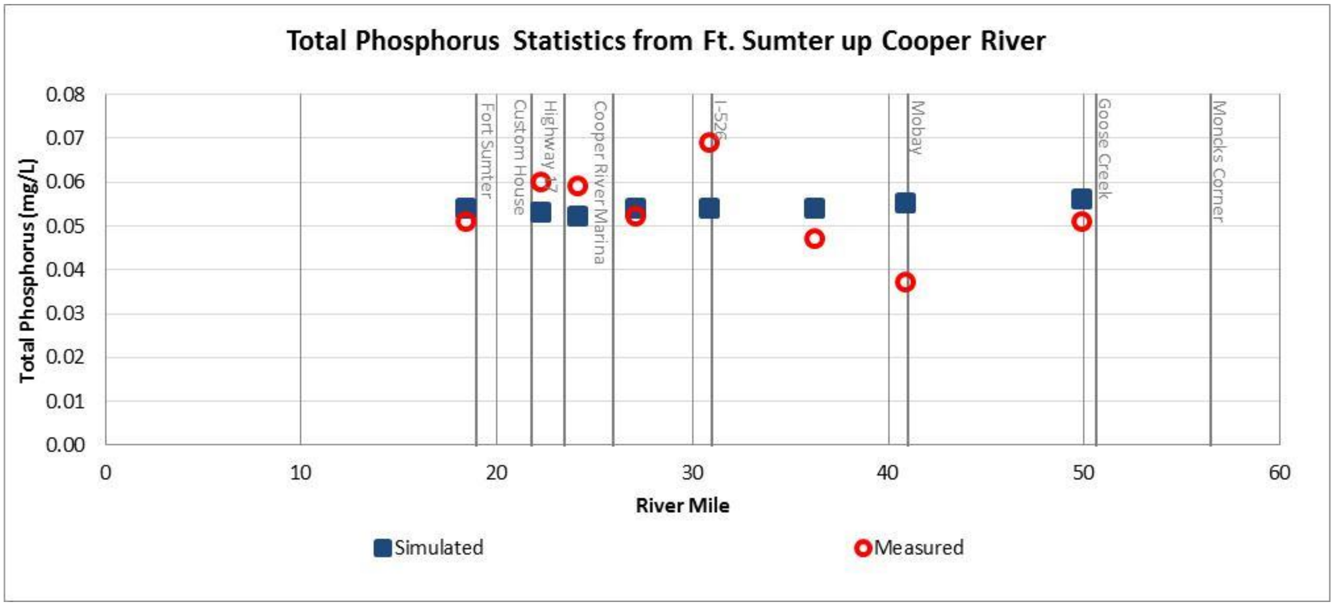


Figure 3.4.38. Longitudinal total phosphorous on the Cooper River

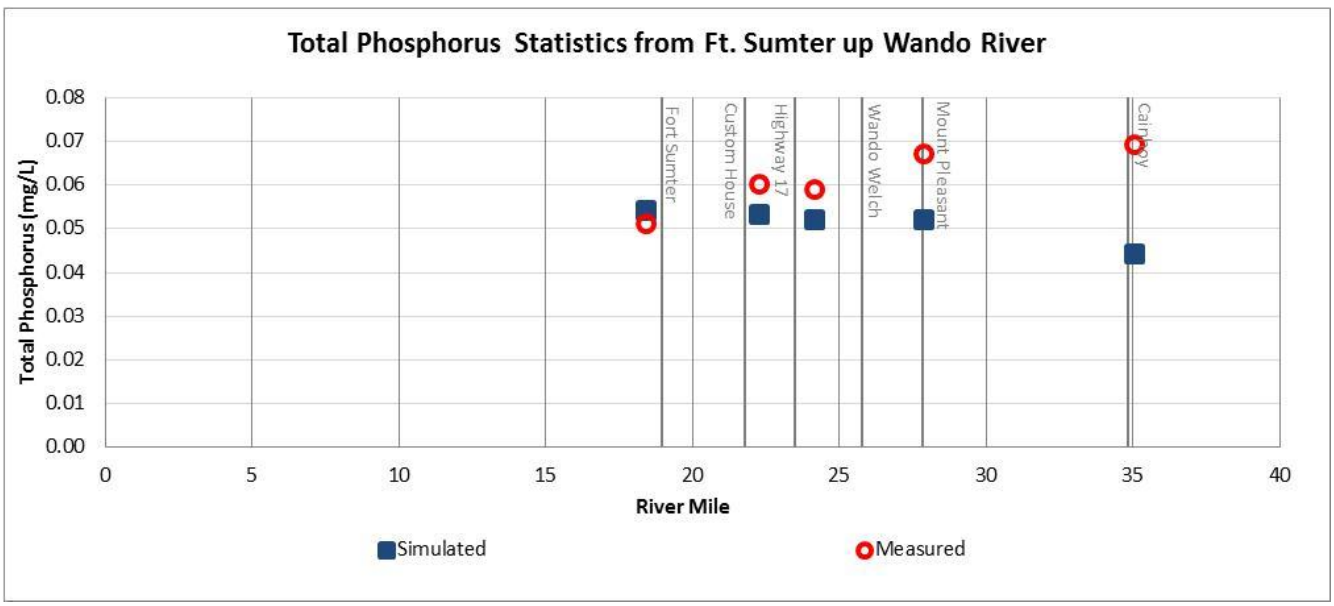


Figure 3.4.39. Longitudinal total phosphorous on the Wando River

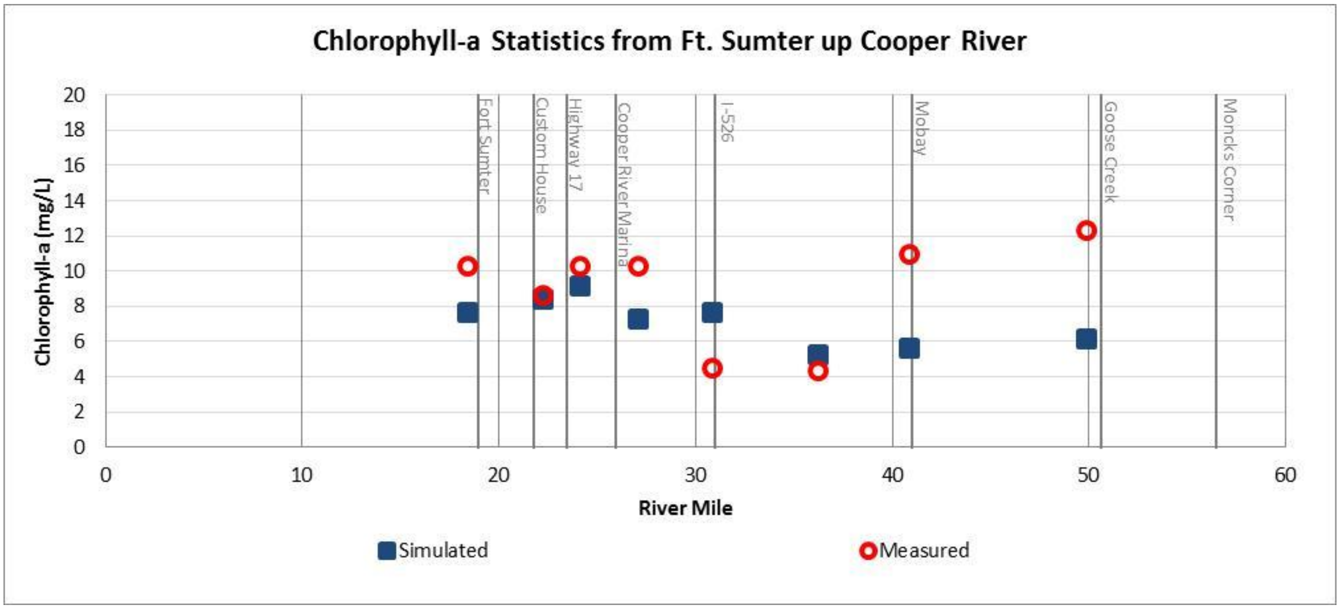


Figure 3.4.40. Longitudinal chlorophyll-*a* on the Cooper River

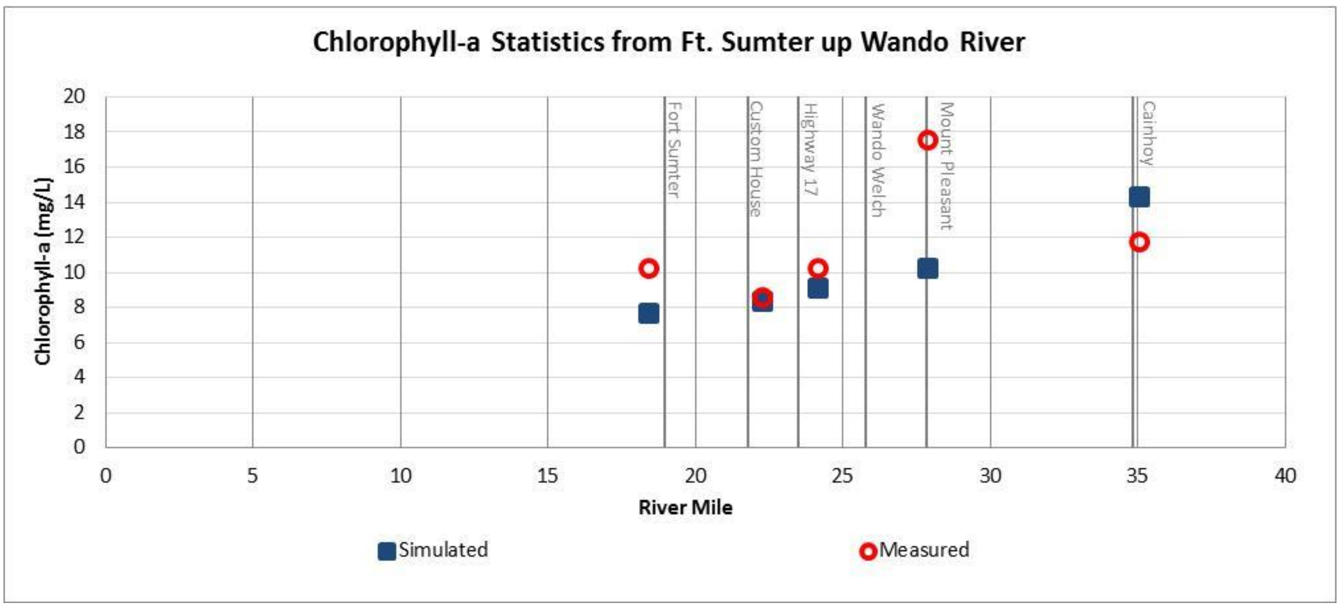


Figure 3.4.41. Longitudinal chlorophyll-*a* on the Wando River

3.4.5 Sensitivity Analysis

This section describes the sensitivity analyses of EFDC for hydrodynamic and water quality parameters during this modeling effort. A sensitivity analysis is the process of varying model input parameters over a reasonable range (range of uncertainty in model parameters) and observing the relative change in model response. The purpose of the sensitivity analyses is to demonstrate the sensitivity of the model simulations to uncertainty in model input data or calibration parameters. For the purpose of this study, hydrodynamic parameters were compared where appropriate. A similar sensitivity analysis was performed as part of the BCDCOG DO TMDL modeling effort where the effect of the parameters below were compared for changes in DO (Tetra Tech 2008). Sensitivity analyses were performed on the following model parameters and boundary inputs:

- Offshore salinity +/- 20%
- Pinopolis Dam flows +/- 20%
- Watershed inflow +/- 20%
- Bottom friction +/- 20%
- Vertical mixing coefficients +/- 20%
- Cohesive sediment critical shear stress for sediment erosion +100% and -50%
- Cohesive sediment fall velocity +100% and -50%
- Noncohesive sediment grain size diameter +100% and -50%

3.4.5.1 Offshore Salinity Concentration

The ocean salinity boundary was increased by 20% and decreased by 20%. The results are summarized by plotting the simulated salinity longitudinally from Fort Sumter to Pinopolis Dam (Figure 3.4.42) and from the harbor up the Wando River (Figure 3.4.43). As would be expected, the results show that the model salinity is highly sensitive to changes in boundary salinity.

3.4..5.2 Pinopolis Dam Flows

The inflows from Pinopolis Dam were increased by 20% and decreased by 20%. The results are summarized by the salinity plots in Figures 3.4.44 and 3.4.45. A 20% increase in flow results in about a 0.0 – 0.6 ppt decrease in the harbor and a 1 – 1.6 ppt decrease along the Cooper River between Highway 17 and Mobay. A 20% decrease in flow causes about a 0.0 – 0.6 ppt increase in the harbor and a 1 – 1.9 ppt increase along the Cooper River between Highway 17 and Mobay. This effect diminishes gradually between Mobay and Goose Creek as the salinity trends towards zero.

The Wando River salinity is also sensitive to the Pinopolis dam inflows. A 20% increase in flow results in about a 1.2 ppt decrease at the Wando Welch Terminal. A 20% decrease in flow results in a 1.3 ppt increase at the Wando Welch Terminal. This effect diminishes gradually toward the upstream end of the Wando River.

3.4.5.3 Watershed Inflows

The watershed inflows were increased by 20% and decreased by 20%. The results are summarized by the salinity plots in Figures 3.4.46 and 3.4.47. The results show no changes in the Cooper River and negligible changes in the most upstream portions of the Wando River.

3.4.5.4 Bottom Friction

Bottom friction (roughness height) was increased by 20% and decreased by 20%. The results are summarized by the mean water level plots in Figures 3.4.48 and 3.4.49. The results indicate that the model simulated mean water levels are insensitive to bottom friction on the Cooper River. Minor changes were simulated in the Wando River upstream of river mile 27.

3.4.5.5 Vertical Diffusion Coefficients

Vertical diffusion coefficients were increased by 20% and decreased by 20%. The results are summarized by the mean water level plots in Figures 3.4.50 and 3.4.51. The results indicate that the model simulated mean water levels are insensitive to changes in vertical diffusion coefficients.

3.4.5.6 Cohesive Sediment Critical Shear Stress for Erosion

The critical shear stress for cohesive sediment erosion, TAUR, was increased by 100% and decreased by 50%. The results are summarized by the chart of sediment deposition rates in the federal navigation channel in Figure 3.4.52. The results indicate that overall the model simulated deposition in the federal navigation channel is sensitive to this variable.

3.4.5.7 Cohesive Sediment Fall Velocity

The cohesive sediment fall velocity, WSEDO, was increased by 100% and decreased by 50%. The results are summarized by the chart of sediment deposition rates in the federal navigation channel in Figure 3.4.53. The results indicate that the model is less sensitive to this variable in the lower reaches. The model is more sensitive to this variable in the turning basin areas where the highest deposition rates of cohesive sediments occur.

3.4.5.8 Cohesive Sediment Fall Velocity

The noncohesive sediment grain size, SEDDIA, was increased by 100% and decreased by 50%. The results are summarized by the chart of sediment deposition rates in the federal navigation channel in Figure 3.4.54. The results indicate that the model is very sensitive to this variable.

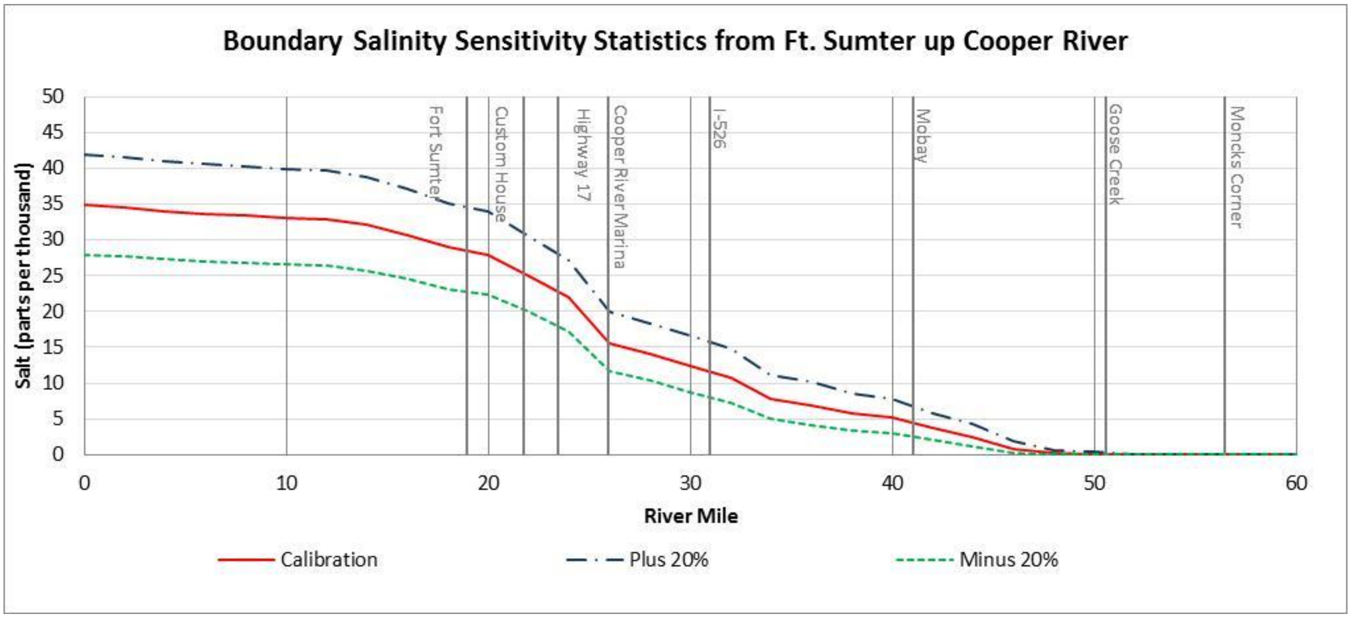


Figure 3.4.42. Offshore boundary salinity sensitivity analysis ±20% plotted longitudinally from Fort Sumter to Pinopolis Dam

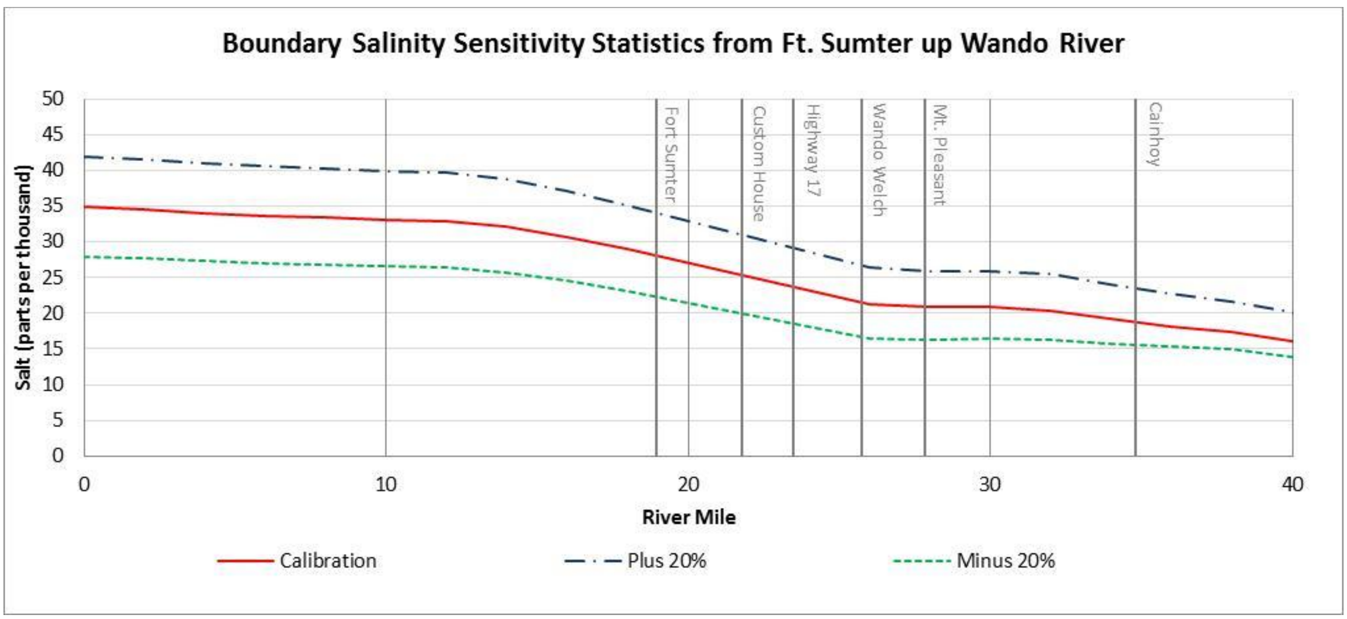


Figure3.4.43. Offshore boundary salinity sensitivity analysis ±20% plotted longitudinally from Charleston Harbor up Wando River

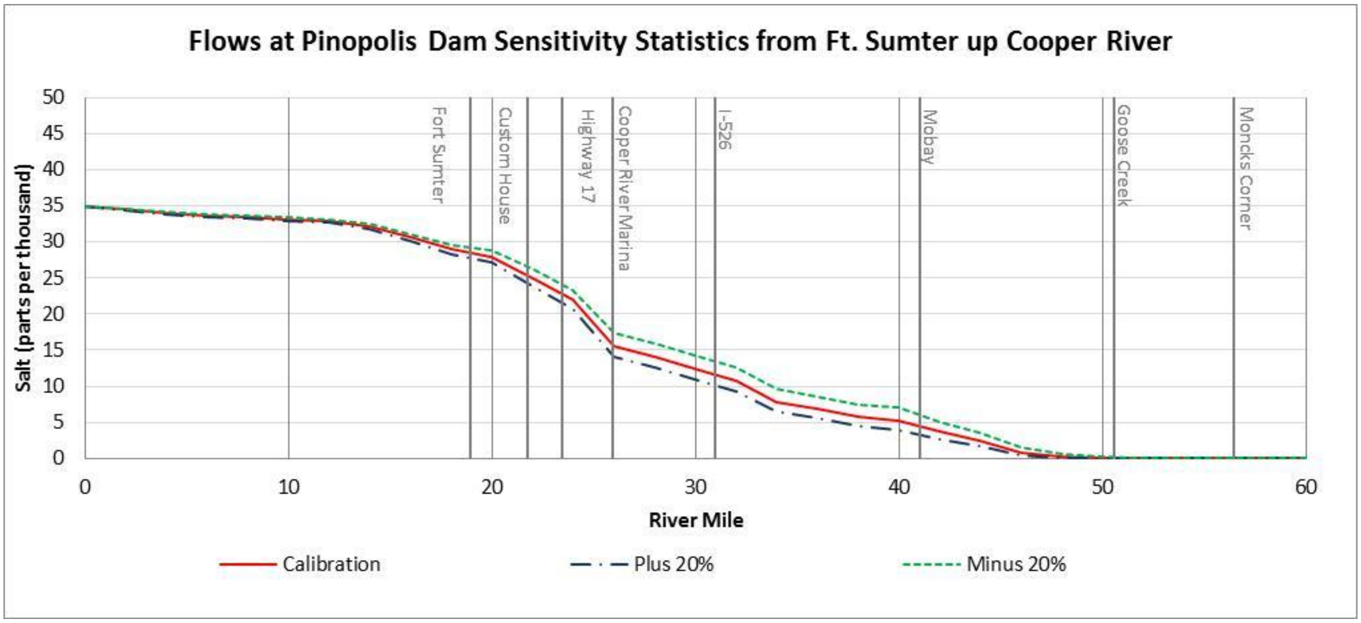


Figure 3.4.44. Flows at Pinopolis Dam sensitivity analysis ±20% plotted longitudinally from Fort Sumter to Pinopolis Dam

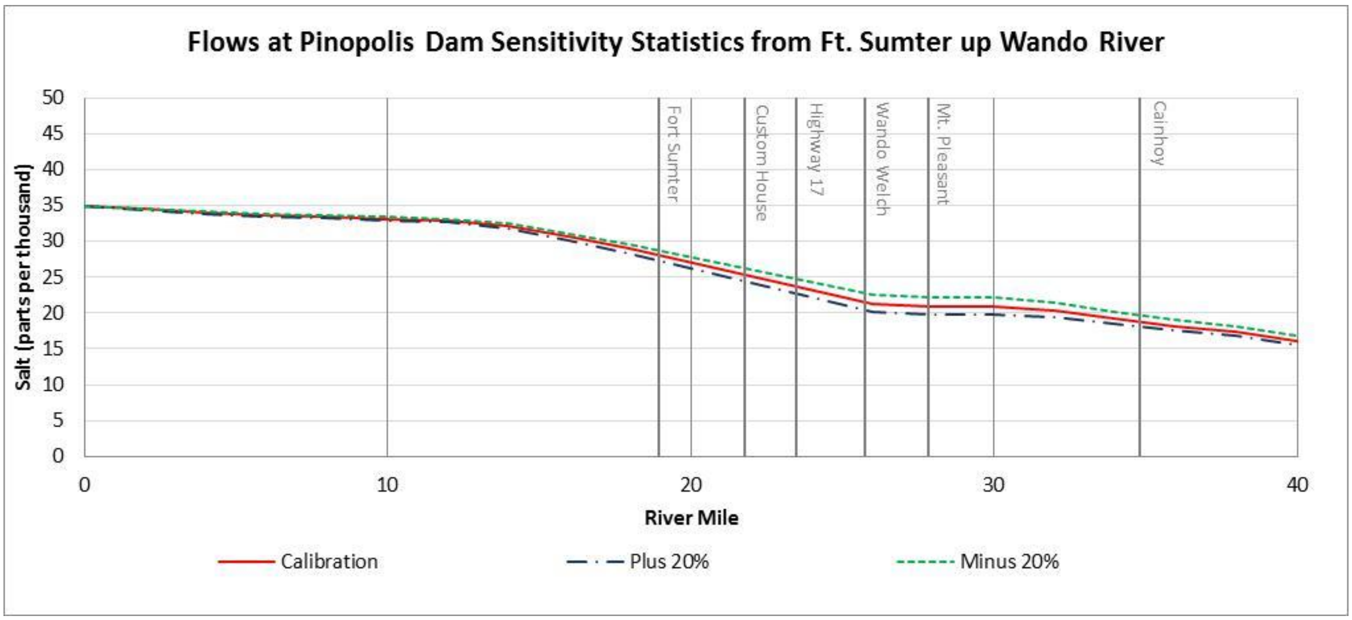


Figure 3.4.45. Flows at Pinopolis Dam sensitivity analysis ±20% plotted longitudinally from Charleston Harbor up Wando River

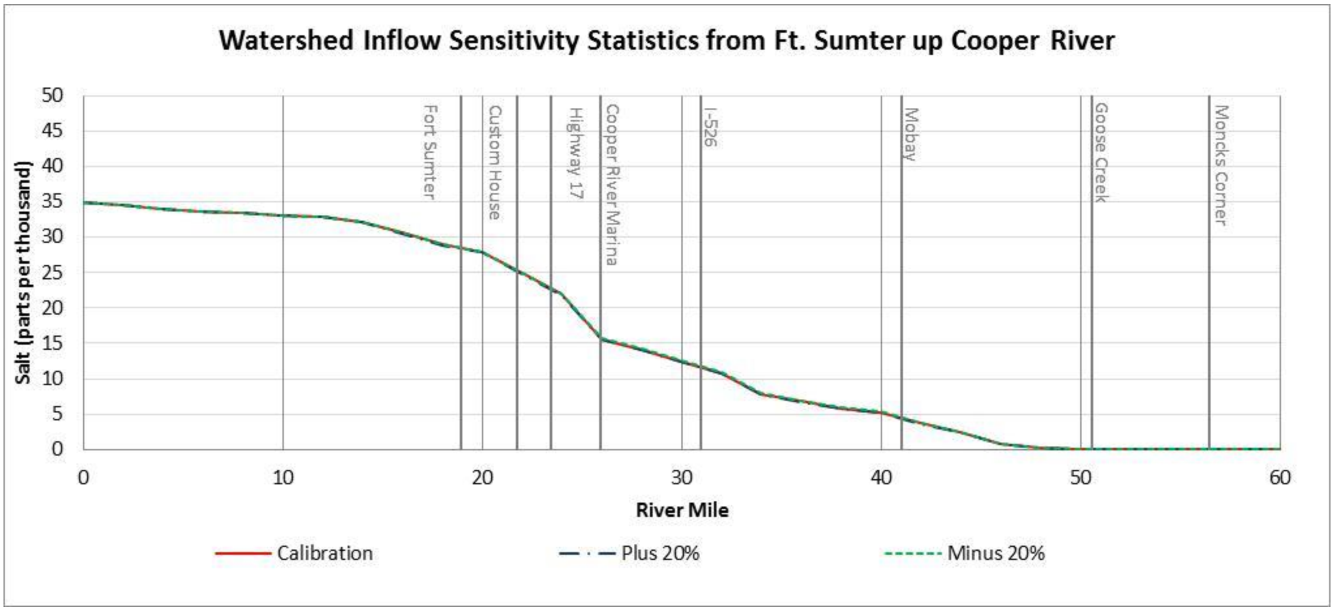


Figure 3.4.46. Watershed inflow sensitivity analysis ±20% plotted longitudinally from Fort Sumter to Pinopolis Dam

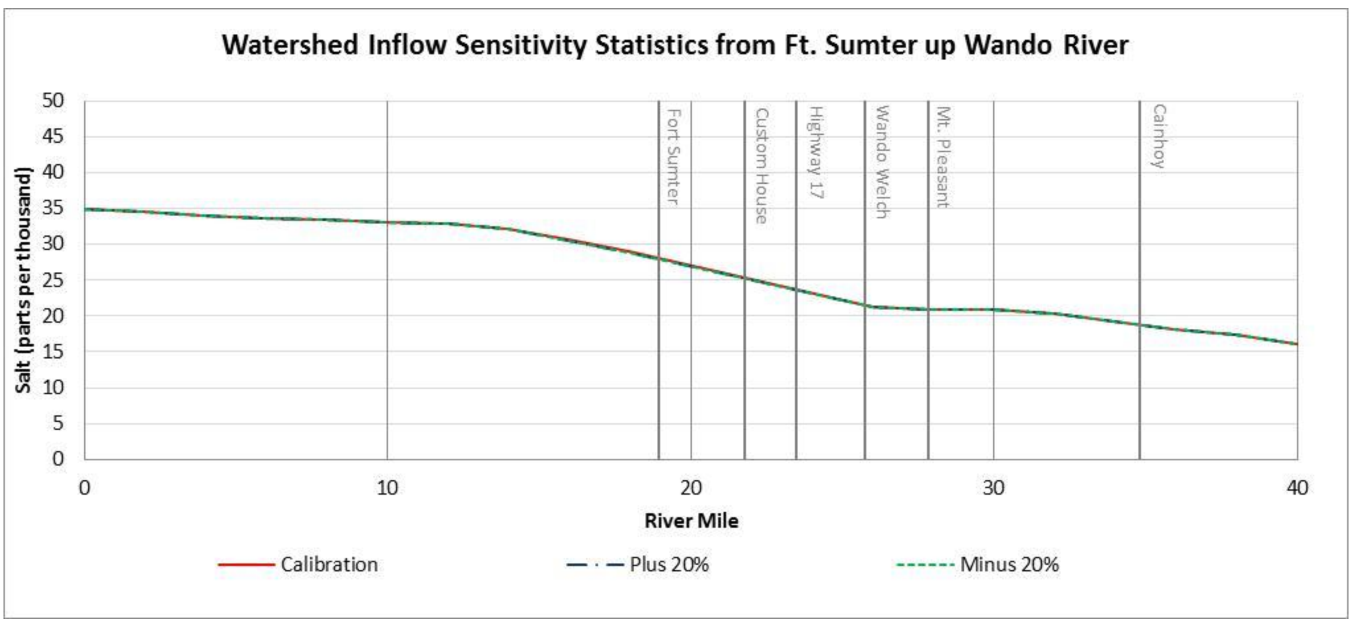


Figure 3.4.47. Watershed inflow sensitivity analysis ±20% plotted longitudinally from Charleston Harbor up Wando River

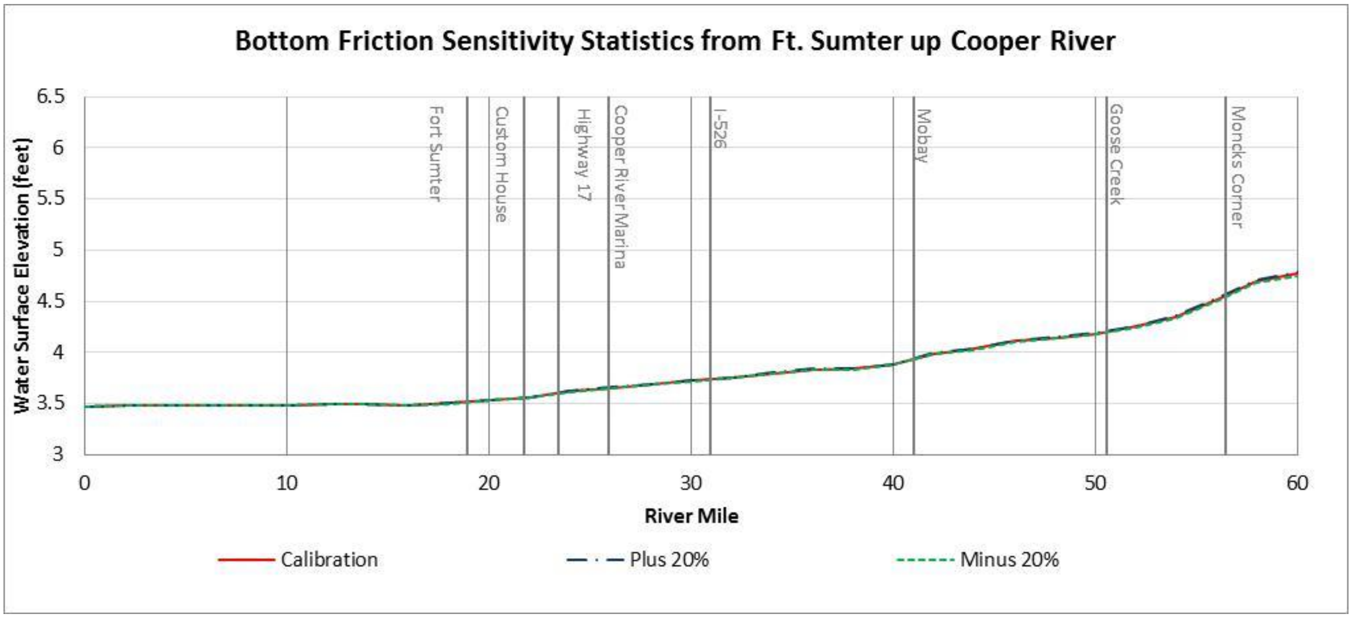


Figure 3.4.48. Bottom friction sensitivity analysis ±20% plotted longitudinally from Fort Sumter to Pinopolis Dam

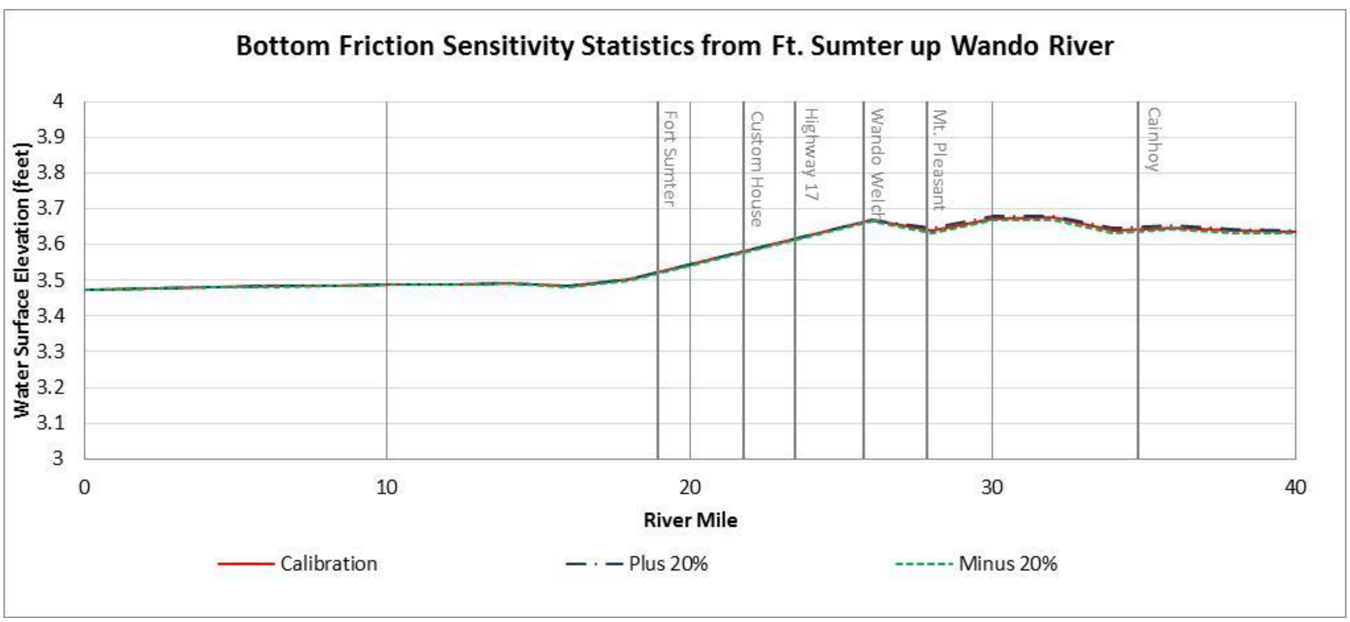


Figure 3.4.49. Bottom friction sensitivity analysis ±20% plotted longitudinally from Charleston Harbor up Wando River

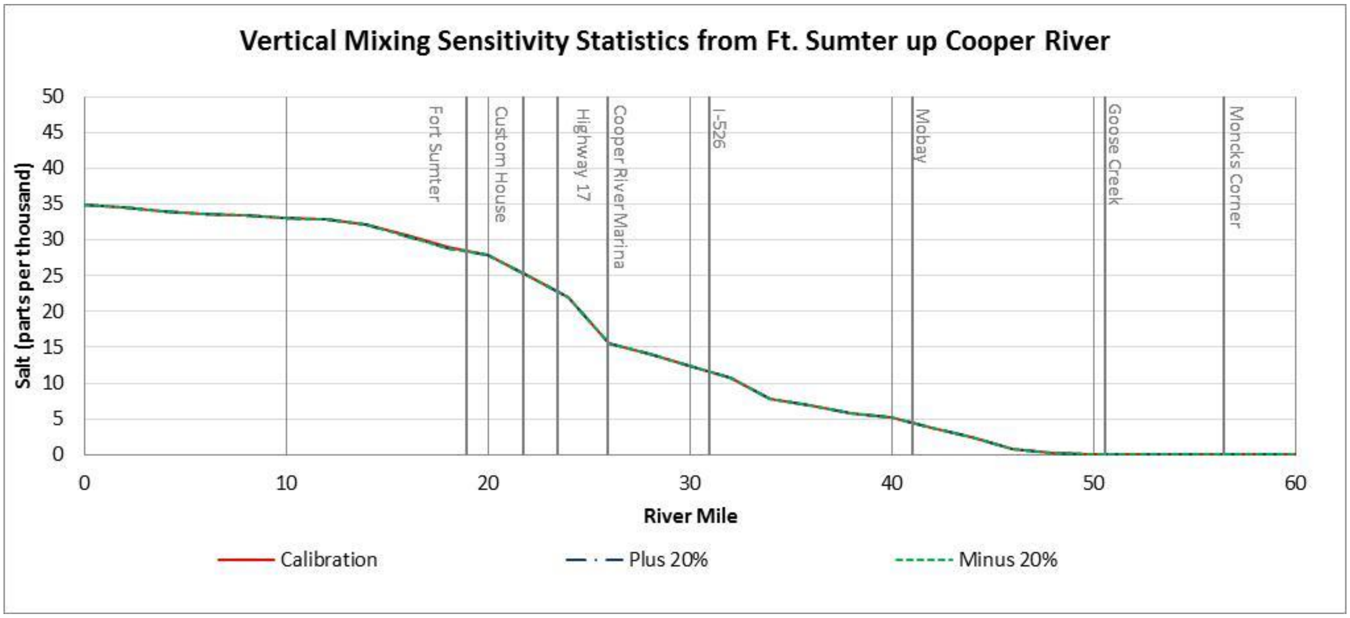


Figure 3.4.50 Vertical mixing sensitivity analysis ±50% plotted longitudinally from Fort Sumter to Pinopolis Dam

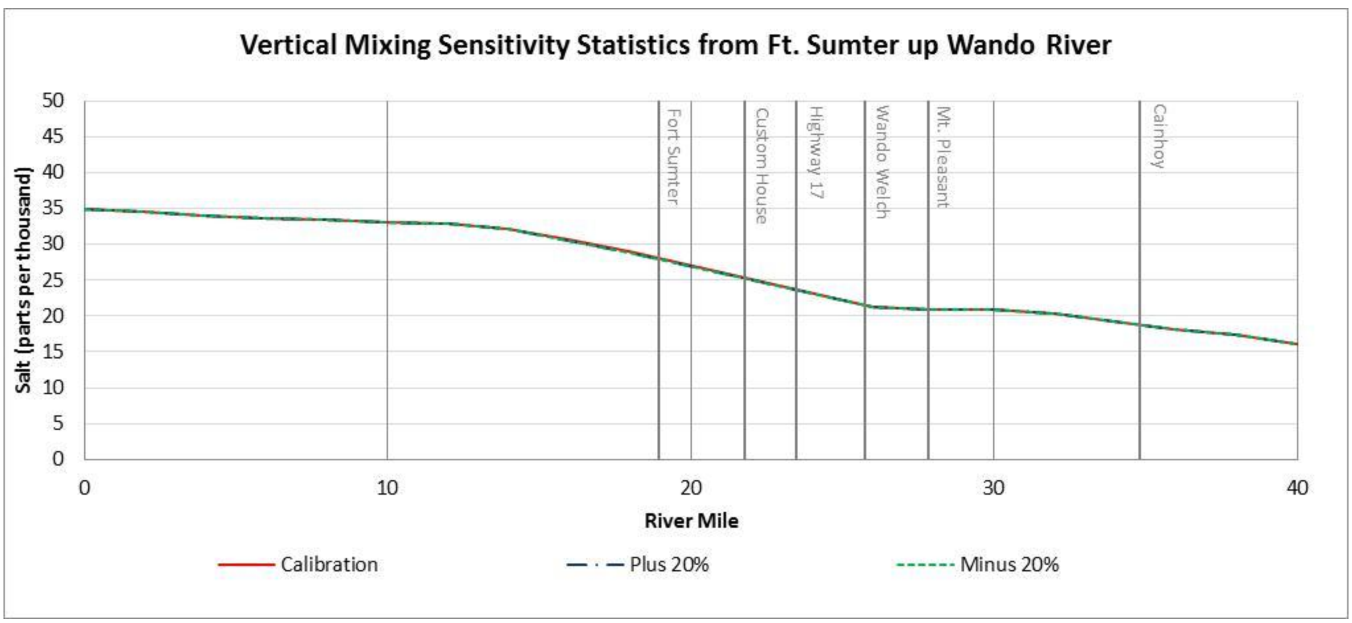


Figure 3.4.51. Vertical mixing sensitivity analysis ±50% plotted longitudinally from Charleston Harbor up Wando River

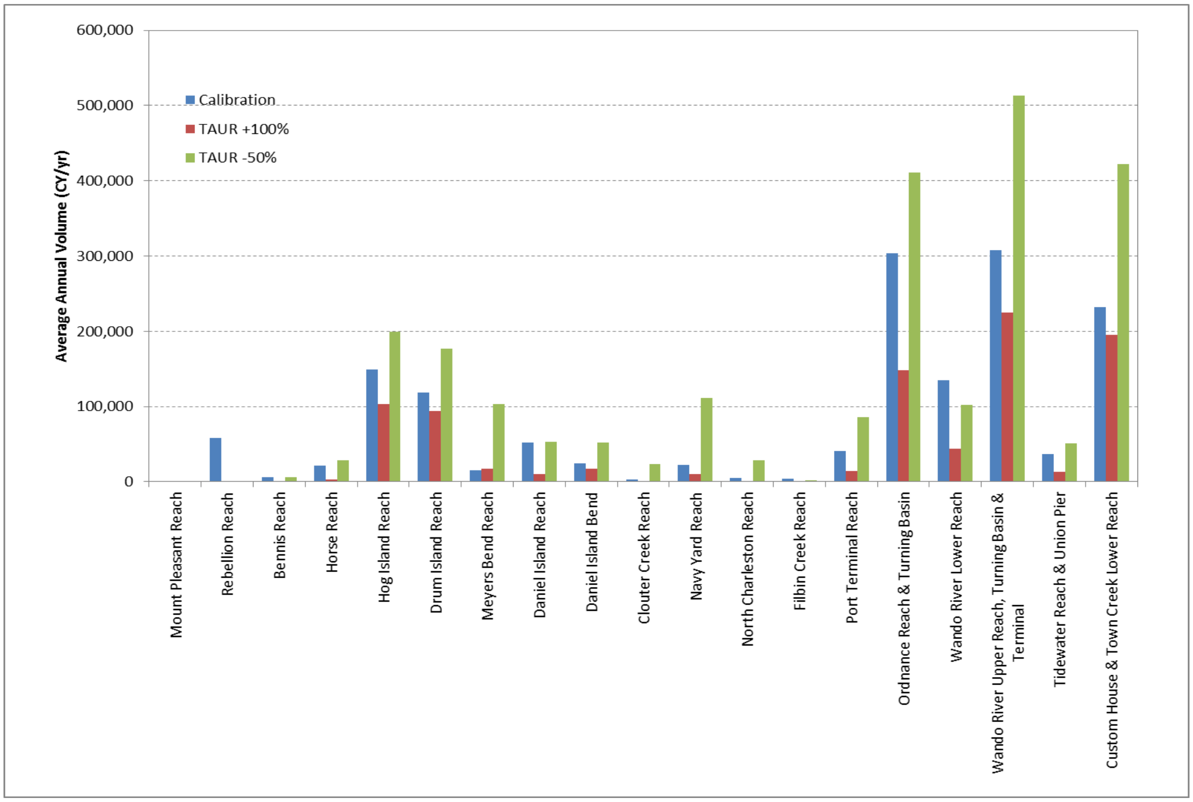


Figure 3.4.52. Sensitivity of predicted deposition rates in the federal navigation channel to +100% and -50% variations in critical shear stress for cohesive sediment erosion, TAUR

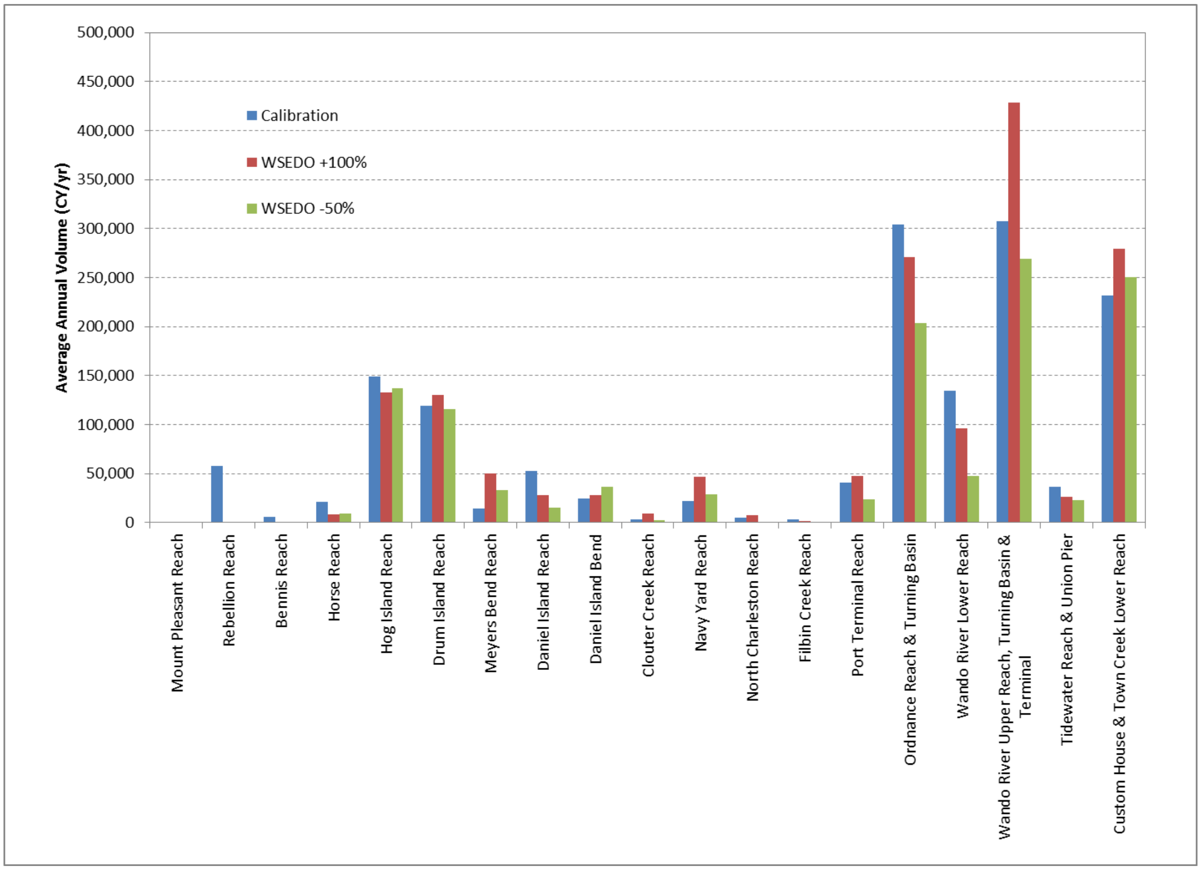


Figure 3.4.53. Sensitivity of predicted deposition rates in the federal navigation channel to +100% and -50% variations in settling velocity for cohesive sediment, WSEDO

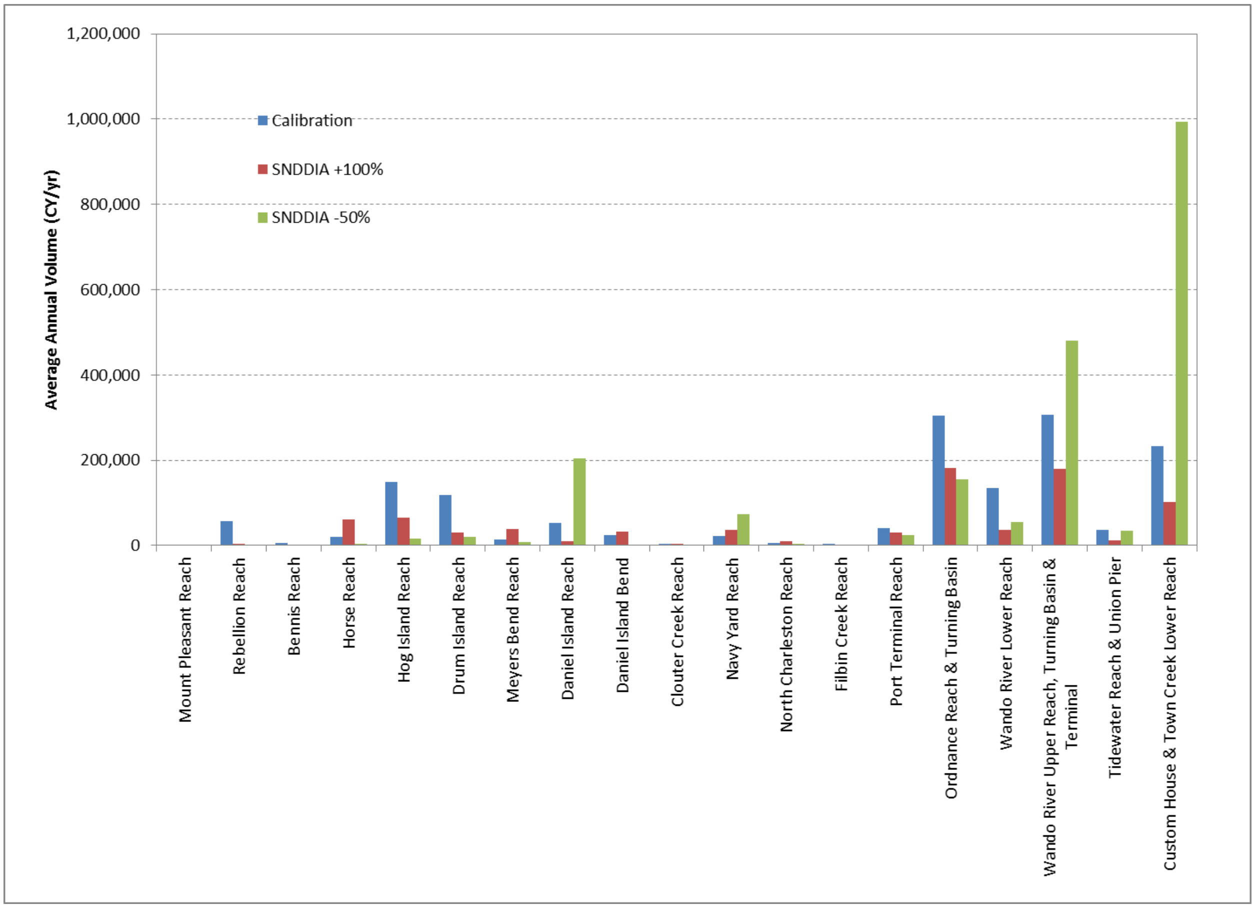


Figure 3.4.54. Sensitivity of predicted deposition rates in the federal navigation channel to +100% and -50% variations in grain size diameter for noncohesive sediment, SEDDIA

3.4.6 Post Processor

The Water Assessment and Management Support (WAMS-Charleston Harbor) is a post-processing tool developed to work with EFDC generated BMD files that contain hydrodynamic and water quality simulation output. The tool links to a suite of FORTRAN based executable files and performs analysis and processing based on input from the user. The GUI has four major blocks: Scenario Control, Model Selector, WAMS Modules, and Auxiliary modules.

The current version of WAMS- Charleston Harbor has eight independent modules: Fish Habitat, Wetlands, Segment WQ, Percentiles Distribution, Scenarios Subtractor, Vertical Profile, Polaris ETD, and Fish Preprocessor. Fish Preprocessor, Fish Habitat Segment WQ, Percentiles Distribution and Scenarios Subtractor were used for this study.

Fish Preprocessor is a module of the WAMS-Charleston. It converts the binary EFDC output (BMD format) into the set of ASCII files for all General Parameters variables: Dissolved Oxygen, Salinity, Temperature, Velocity, Shoaling rate and Depth. One time conversion for the whole simulation period accelerates significantly the subsequent calculation and analysis of Habitat Suitability Indexes (HSI) for different species of fish.

Fish Habitat analyzes the EFDC hydrodynamics and water quality simulation outputs in BMD format and generates the following information for a user selected simulation period:

1. BMD files with snapshots of the user selected percentiles distributions of General Parameters: DO, Salinity, Temperature, Velocity, Shoaling rate and Depth.
2. BMD files with snapshots of distributions of indexes of habitat suitability (HSI) for Striped bass, Shortnose sturgeon, Southern flounder, Blueback herring, Red drum, Atlantic sturgeon and Oysters; and their life stages.
3. CSV files with the calculated areas of Suitable Habitat, SH indexes of the aforementioned fishes and their life stages in each horizontal cell of the model domain.

Segment-WQ analyzes the EFDC Water Quality output in the BMD format and generates segment-averaged and time-averaged information for user selected simulation periods. The user has an option of selecting the following type of time-averaging: No Averaging, Moving Averaging and/or Simple Averaging. The user has an option of selecting horizontal shape and vertical boundaries of segments (aggregation of cells that are bounded by horizontal shape and selected vertical layers). The Segment-WQ module performs 2 primary calculations:

1. Temporal-spatial averaging of time series of model variables in cells of a computational grid
2. Calculations of percentiles distributions and averages of the model's variables in a user selected spatial segments

Percentiles Distribution analyzes the EFDC simulation outputs and generates the percentile distributions for a user's selected state variables of the EFDC hydrodynamic or water quality model output file. The Percentiles module performs primary calculations of an user's selected state variables' percentiles distribution in cells of analyzed lake's computational domain

Scenario Subtractor compares WAMS generated BMD and CSV files for a user's selected simulation scenarios and calculates new BMD and CSV files that contain Delta of compared state variables.

3.5 Study Scenarios

The calibrated model was used to evaluate changes to salinity, water temperature, shoaling, and DO that would be expected from various harbor deepening alternatives, including the existing and future without conditions. The modeling for alternative conditions were based on the maximum widening as discussed in Section 2.1.2 and shown on Figures 3.5.1.a., 3.5.1.b., and 3.5.1.c.

Boundary conditions were defined by (1) meteorological conditions; (2) watershed conditions; (3) point source conditions; (4) offshore boundary conditions; and (5) upstream boundary conditions. These model inputs were generally related and defined by evaluating freshwater stream flows. However, because point source conditions are controlled by independent permitted dischargers, they tend to be more variable and unrelated to other boundary conditions. Therefore, the boundaries for point sources were based on the reported discharge rates in water year (WY) 2012. The offshore boundary conditions for WY 2012 were also used for consistency.

Selecting the appropriate boundary conditions must consider how the model will be applied to evaluate project impacts. The model will be used to provide input to Habitat Suitability Index (HSI) models for several species. These HSI are informed by average and low flow conditions for several simulation periods, including monthly, seasonal, annual, and specific spawning and life stage periods (e.g., egg development, larval development, and growing season). The most periods listed for evaluation are at least one month in duration (although explicit periods were not initially available for the life stage periods). Therefore, the appropriate minimum time-averaging period to consider when determining *typical* and *drought low-flow* conditions was one month. That is, daily minimum flows were not considered in selecting the appropriate low-flow period.

The management of the flows from the Pinopolis Dam minimizes, to a large extent, the effects of drought conditions on the flows into the Cooper River. As a result, flow observed below the Pinopolis Dam was not used as the sole indicator of drought effects for the entire Charleston Harbor estuary system. Although the Pinopolis Dam flow is the primary variable controlling the salinity concentrations and residence times in the Cooper River, the Ashley River and Wando River are primarily influenced by the local watershed inflows. Therefore, an analysis was done to assess the other freshwater flow contributions to the system, particularly the Wando River inflows where the proposed project could cause changes to the hydrodynamics, sedimentation rates, or water quality (in contrast, the proposed project is not expected to cause significant changes in the Ashley River).

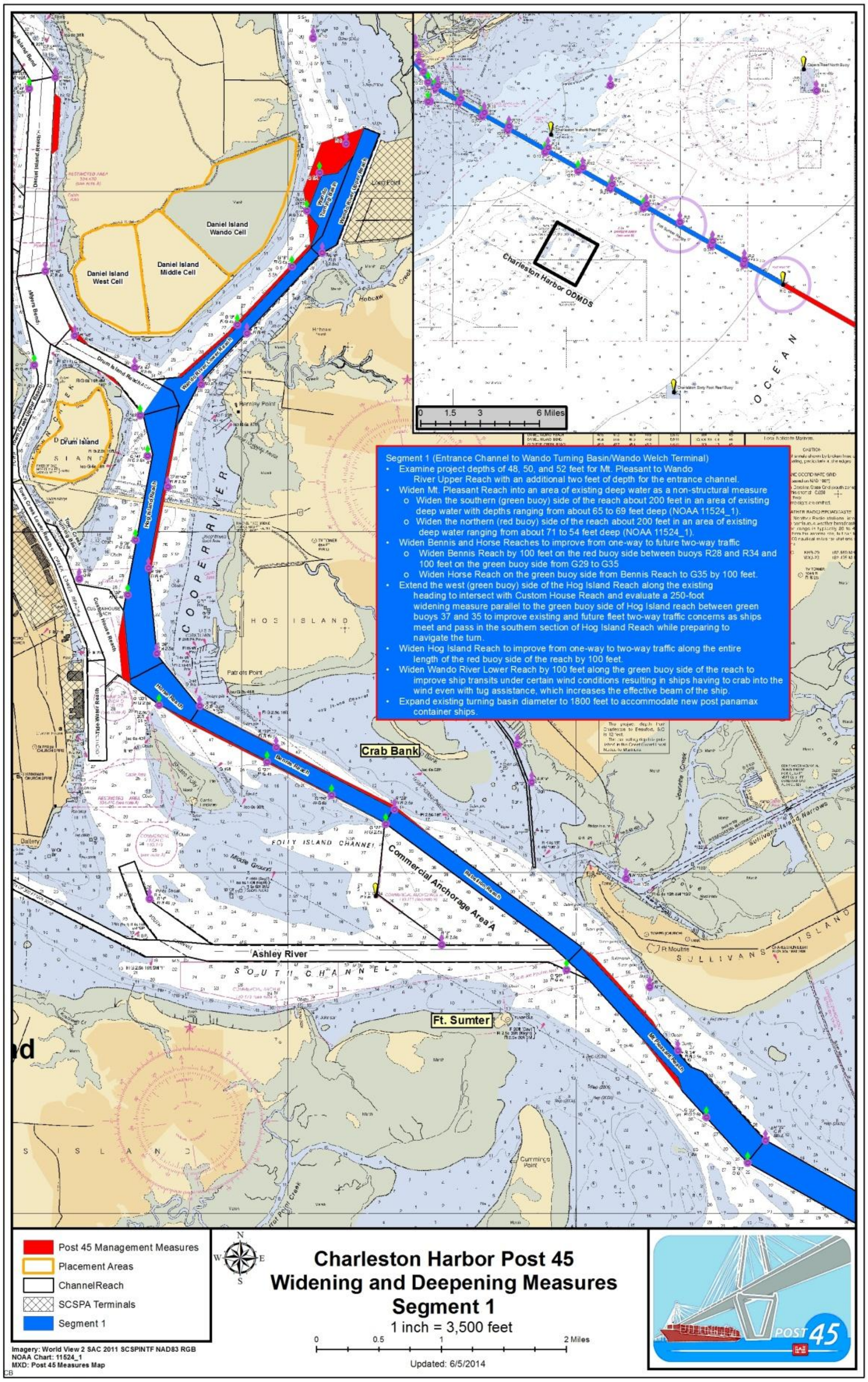


Figure 3.5.1.a. Segment 1 Channel Width Alternatives for Charleston Harbor

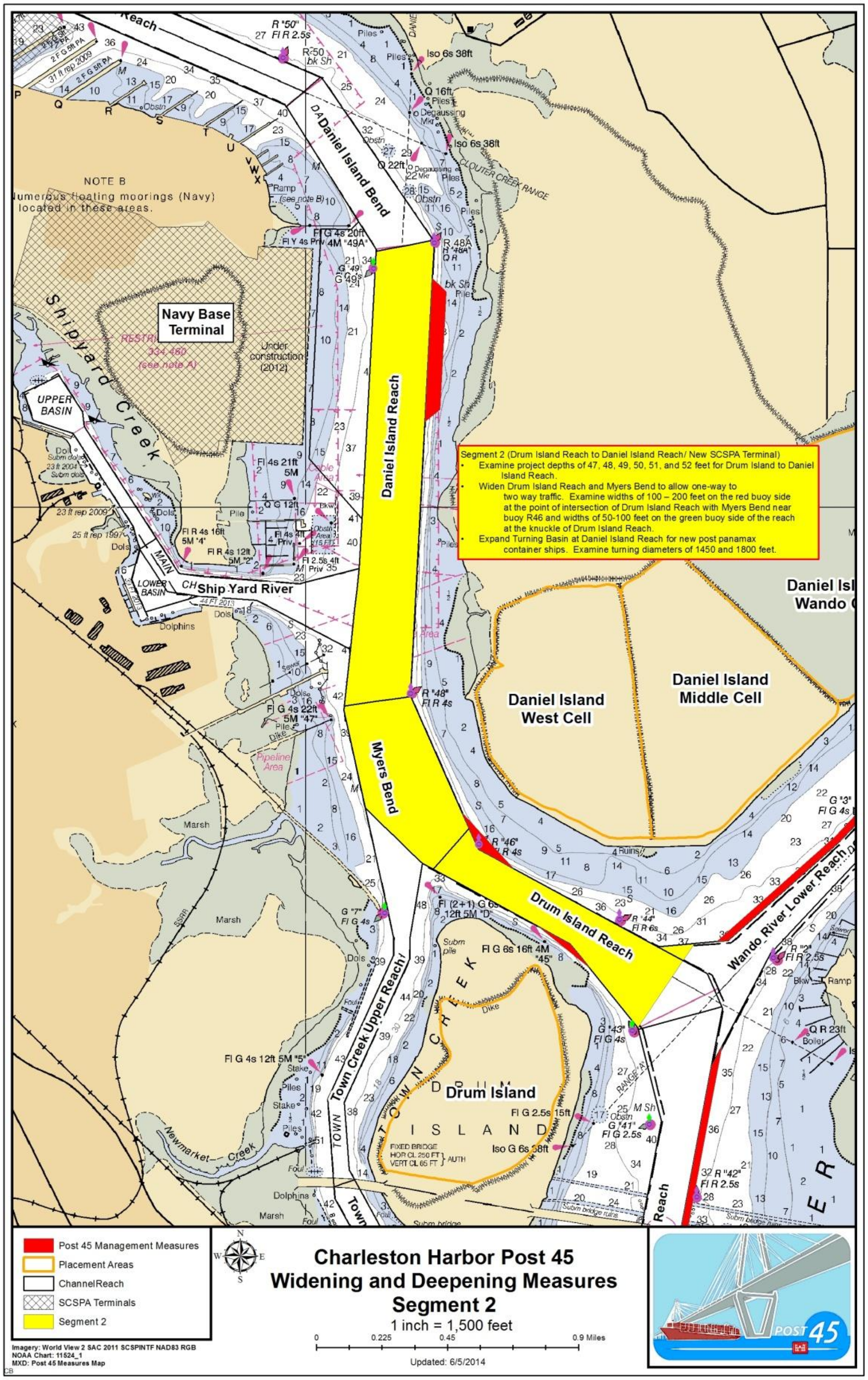


Figure 3.5.1.b. Segment 2 Channel Width Alternatives for Charleston Harbor

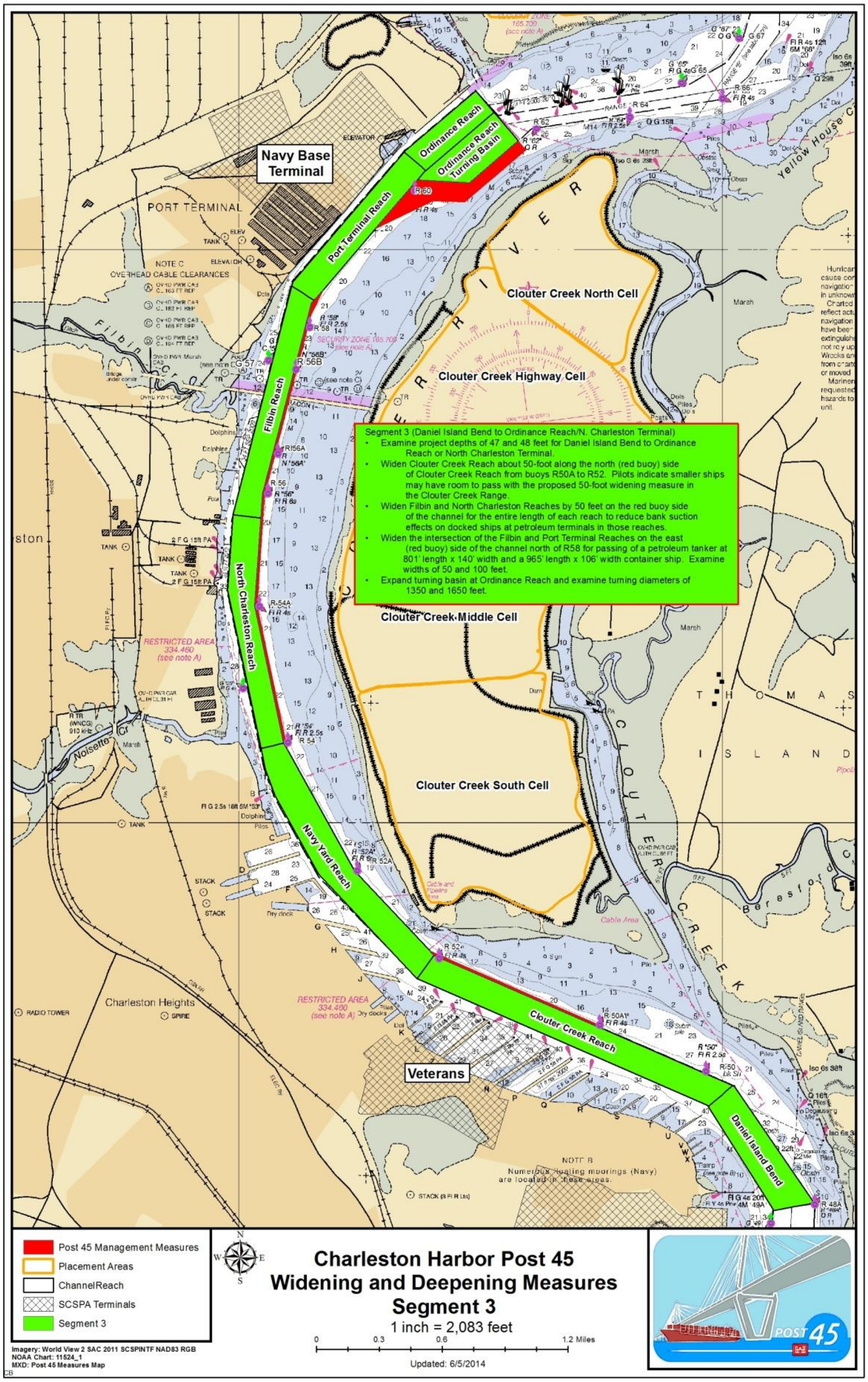


Figure 3.5.1.c. Segment 3 Channel Width Alternatives for Charleston Harbor

An analysis of low-flow conditions and typical annual conditions was performed to determine the typical and low-flow periods. The analyses established a simulation period for the WY 2008 (October 1, 2007 through September 30, 2008) that would encompass two Pinopolis Dam low-flow periods during the 2007–2008 severe drought conditions. This period, WY 2008, was selected to define low-flow conditions. WY 2006 was determined to be reasonably representative of typical annual and seasonal conditions, and was used for the typical conditions simulation. Both the typical and low-flow periods were extended through October (Table 3.5.1). The 30-day moving average flows from Pinopolis Dam are illustrated in Figure 3.5.2. Corresponding meteorological, watershed, offshore, and upstream boundary conditions during these periods were recommended to represent these conditions. As mentioned previously, the NPDES permitted point source discharges were simulated using the most recent DMRs. Finally, the 2004 critical conditions identified by SCDHEC and EPA were used in the TMDL simulation (March 1 through October 31, 2004).

The set of model simulations include each of the 8 model scenarios and three model conditions (Low-Flow, Typical, and TMDL). The boundary time periods for each of these simulations are listed in Table 3.5.1. The simulations include a 2-month spin-up period that proceeds the simulation period. Based on the model calibration results, the model stabilizes relatively quickly, within 2 weeks of the model startup. Therefore, a 2-month spin-up period is more than adequate for the application scenarios.

Table 3.5.1. Time Period of Boundary for each modeled condition

	Modeled Condition[1]		
Boundary	**Low-Flow**	**Typical**	**TMDL**
Meteorological conditions	Oct. 2007 - Oct. 2008	Oct. 2005 - Oct. 2006	Mar. 2004 – Feb. 2005
Watershed conditions	Oct. 2007 - Oct. 2008	Oct. 2005 - Oct. 2006	Mar. 2004 - Feb. 2005
Point source conditions (DMRs)	*Oct. 2011 - Oct. 2012*	*Oct. 2011 - Oct. 2012*	*Mar. 2012 - Feb. 2013*[2]
Offshore boundary conditions (salt, temp, and tide)	*Oct. 2011 - Oct. 2012*	*Oct. 2011 - Oct. 2012*	*Mar. 2012 - Feb. 2013*[2]
Upstream boundary conditions	Oct. 2007 - Oct. 2008	Oct. 2005 - Oct. 2006	Mar. 2004 - Feb. 2005

[1] A two month spin-up period was added to each condition using the two-months prior to the start date listed here.

[2] The point source and offshore boundary conditions for the TMDL model condition are seasonally consistent with the critical period defined by the TMDL to ensure seasonal variability is captured by the boundary conditions.

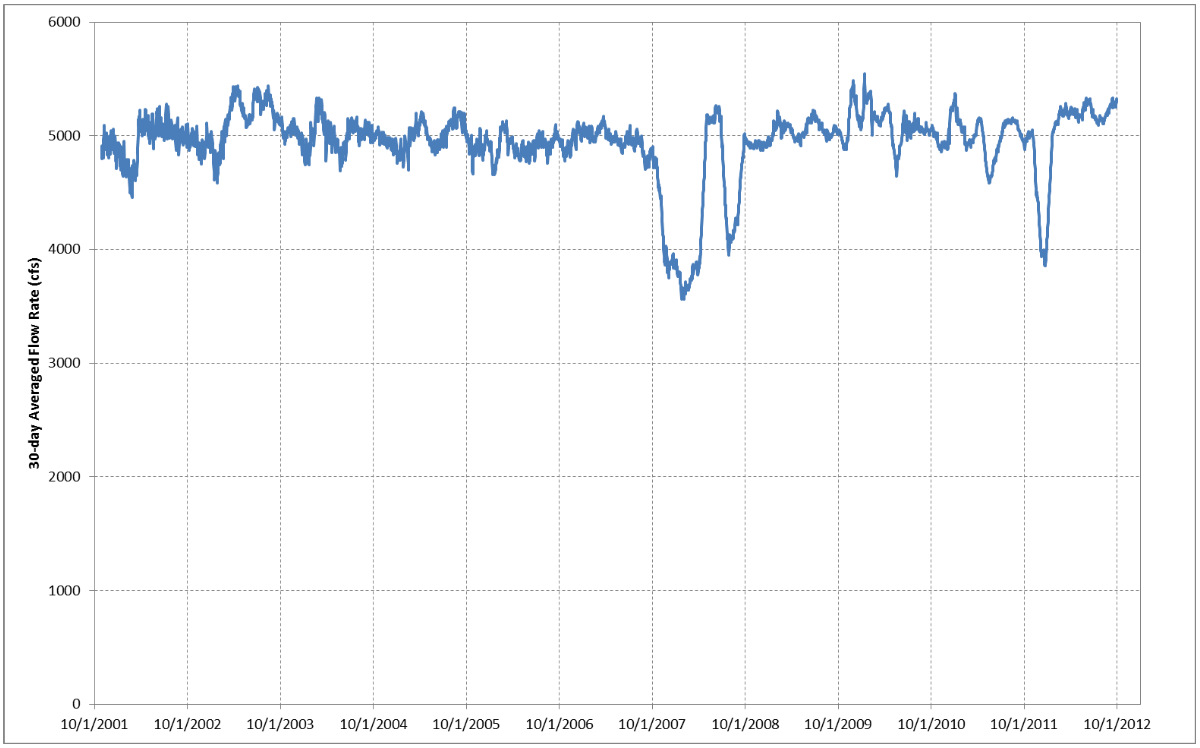


Figure 3.5.2. 30-day Moving Average Flows from Pinopolis Dam

As previously described, the model calibration grid uses bathymetric survey data to define the bottom elevation for each grid cell. The calibrated model was used to evaluate changes to salinity, water temperature, shoaling, and DO that would be expected from various harbor deepening alternatives.

The performance and function of the navigation channel is not highly sensitive to the rate of sea level change. Therefore, according to ER 1110-2-8162, paragraph 6.(d).(1), this study will work within a single scenario, “most-likely” or historic (low) rate, and identify the preferred alternative under that scenario. The preferred alternative’s impacts is then be evaluated under the other scenarios to determine its overall potential impacts.

The depths presented in each scenario include an additional 2-foot over depth and 2-foot advanced maintenance dredging below the design depths. Areas of advance maintenance greater than 2 ft have greater advance maintenance because they fill in quickly. Therefore it was assumed that additional advance maintenance was not the condition to model. The access channel area for the new marine container terminal (NBT) at the former Navy base is the same depth as the adjacent Daniel Island Reach for all scenarios.

1. **Existing Condition**: The existing condition channel widths are the existing channel template. Design depth is 47 feet MLLW in the entrance channel, 45 feet MLLW from Mount Pleasant Range throughout the Charleston Harbor, Wando

River, and Cooper River. Access channel for new marine container terminal is at design depth of 45 feet MLLW. Areas not presently maintained by the USACE remain at the most recently surveyed depth (as used in the model calibration), including the Ashley River, Anchorage Basin A, Shipyard Creek, and Upper Town Creek.

2. **Future Without:** Channel widths are the existing channel template. Design depth is 47 feet MLLW in the entrance channel, 45 feet MLLW from Mount Pleasant Range throughout the Charleston Harbor, Wando River, and Cooper River. Assumes sea level change (SLC) is 0.57 feet above existing level as determined by USACE guidance (ER 1110-2-8162) for low or historic rate.
3. **Alternative 48-47:** Channel widths are maximum widenings, transitions, bend easings and turning basin enlargements. Design depth is 50 feet MLLW in the entrance channel, 48 feet MLLW from Mount Pleasant range to Wando River up to Wando terminal (includes turning basin) and to Cooper River at proposed CNCMCT (includes turning basin). This includes the widener area in Customs House Reach, but the remainder of Customs House Reach, as well as Tidewater Reach, Town Creek Turning Basin and Lower Town Creek Reach remain at the Existing Condition design depths. Design depth is 47 feet MLLW from Daniel Island bend to Ordnance reach (includes turning basin). Assume SLC is 0.57 feet above existing level.
4. **Alternative 50-48:** Channel widths are maximum widenings, transitions, bend easings and turning basin enlargements. Design depth is 52 feet MLLW in the entrance channel, 50 feet MLLW from Mount Pleasant range to Wando River up to Wando terminal (includes turning basin) and to Cooper River at proposed CNCMCT (includes turning basin). Design depth is 48 feet MLLW from Daniel Island bend to Ordnance reach (includes turning basin). Assume SLC is 0.57 feet above existing level.

5. **Alternative 48-48:** Channel widths are maximum widenings, transitions, bend easings and turning basin enlargements. Design depth is 50 feet MLLW in the entrance channel, 48 feet MLLW from Mount Pleasant range to Wando River up to Wando terminal (includes turning basin) and to Cooper River at proposed CNCMCT (includes turning basin). Design depth is 48 feet MLLW from Daniel Island bend to Ordnance reach (includes turning basin). Assume SLC is 0.57 feet above existing level.

6. **Alternative 50-47:** Channel widths are maximum widenings, transitions, bend easings and turning basin enlargements. Design depth is 52 feet MLLW in the entrance channel, 50 feet MLLW from Mount Pleasant range to Wando River up to Wando terminal (includes turning basin) and to Cooper River at proposed CNCMCT (includes turning basin). Design depth is 47 feet MLLW from Daniel Island bend to Ordnance reach (includes turning basin). Assume SLC is 0.57 feet above existing level.

7. **Alternative 52-48 (Recommended Plan) :** Channel widths are maximum widenings, transitions, bend easings and turning basin enlargements. Design depth is 54 feet MLLW in the entrance channel, 52 feet MLLW from Mount Pleasant range to Wando River up to Wando terminal (includes turning basin) and to Cooper River at proposed CNCMCT (includes turning basin). Design depth is 48 feet MLLW from Daniel Island bend to Ordnance reach (includes turning basin). Assume SLC is 0.57 feet above existing level.

8. **Alternative 52-47:** Channel widths are maximum widenings, transitions, bend easings and turning basin enlargements. Design depth is 54 feet MLLW in the entrance channel, 52 feet MLLW from Mount Pleasant range to Wando River up to Wando terminal (includes turning basin) and to Cooper River at proposed CNCMCT (includes turning basin). Design depth is 47 feet MLLW from Daniel Island bend to Ordnance reach (includes turning basin). Assume SLC is 0.57 feet above existing level.

It should be noted that the model grids remain static throughout the simulations at the elevations described above. That is, the bottom elevations in the model do not change over time during the simulations. In reality, many parts of the federal navigation channel remain at these bottom elevations for only a short period of time following maintenance dredging. In areas that experience shoaling, the channel immediately begins to fill back in following maintenance dredging. Therefore, this analysis provides a conservative estimate of potential project alternative effects by assuming the channel stays at the initial depth through the simulation.

3.6 Existing Conditions

The model calibration grid was modified to create the Existing Condition model grid by assigning a depth of 51 feet MLLW in the entrance channel, and 49 feet MLLW from Mount Pleasant Range throughout the federal navigation channel in Charleston Harbor, Wando River, and Cooper River. All depths assigned to the federal navigation channel include the design depth of 47 feet for the entrance channel and 45 feet throughout the navigation channel plus 2 feet overdredge and 2 feet advance maintenance.

The purpose of the existing condition was to provide a basis for comparison of the relative changes due to sea level change with the future without condition. Comparison to Future Without conditions is discussed in Section 3.7. Additionally, existing condition was used as a comparison for the water quality impacts to dissolved oxygen, per the request of SCDHEC, as further discussed in Section 3.8.5.

3.7 Future Without Conditions

Channel widths are the existing channel template with an assumed sea level change (SLC) of 0.57 feet above existing level as determined by USACE guidance (ER 1110-2-8162).

3.7.1 Water Levels

The primary concerns regarding potential changes in water levels are related to high water levels, because these affect both marsh habitat inundation frequency, as well as flooding of low lying coastal areas. High tide water levels also affect the alert system designed to prevent salinity intrusion to the Bushy Park Reservoir. One of the USACE criteria for increased flow releases from Pinopolis includes observed tidal water levels at the Customs House gage to determine if an emergency release of fresh water from the Pinopolis Dam is necessary to avoid salinity intrusion to the Bushy Park Reservoir. As demonstrated in Section 1.4.5, the water level at the Customs House gage is the controlling threshold for most alerts that trigger increased discharges from the Pinopolis Dam (this trigger is referred to as a tidal alert).

The increase in water surface elevation of the future without project condition as compared to the existing condition is approximately equivalent to the sea level change as discussed in Section 2.2. Contraction dikes are not overtopped. As part of normal maintenance, disposal areas would be assessed for erosion and toe protection needs. Low-lying and marsh areas will be impacted and waterfront property owners will need to assess their own risk and adapt.

It should be noted that it cannot be concluded how many more tidal alerts would be expected from sea level under this analysis. The computational method for determining a tidal alert is based on a tide factor which is formula involving the mean tide level and the tide range of the preceding 24-hour period. The number of tidal alerts is expected to rise with sea level but it cannot be concluded by exactly how much with this analysis. Impacts on marsh habitat are discussed in the Environmental Appendices of this report.

Table 3.7.1.1 Water Surface Elevations Percentiles

USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC				USGS 02172069.8 Wando River above Mount Pleasant		
	Existing	FWO			Existing	FWO
1st percentile	2.51	3.14		**1st percentile**	-0.25	0.27
10th percentile	3.22	3.76		**10th percentile**	0.65	1.18
50th percentile	4.16	4.68		**50th percentile**	3.27	3.82
90th percentile	5.04	5.56		**90th percentile**	5.89	6.48
99th percentile	5.63	6.18		**99th percentile**	6.95	7.57
USGS 02172050 Cooper River near Goose Creek, SC				USGS 02172070.9 Cooper River at Hwy 17		
	Existing	FWO			Existing	FWO
1st percentile	1.19	1.83		**1st percentile**	-0.44	0.11
10th percentile	1.96	2.54		**10th percentile**	0.53	1.08
50th percentile	3.82	4.35		**50th percentile**	3.3	3.85
90th percentile	5.54	6.07		**90th percentile**	5.79	6.37
99th percentile	6.33	6.87		**99th percentile**	6.81	7.39
USGS 02172053 Cooper River at Mobay				USGS 02172086.9 Ashley River near North Charleston		
	Existing	FWO			Existing	FWO
1st percentile	0.25	0.81		**1st percentile**	-0.71	-0.16
10th percentile	1.1	1.69		**10th percentile**	0.32	0.88
50th percentile	3.58	4.11		**50th percentile**	3.43	3.99
90th percentile	5.85	6.42		**90th percentile**	6.01	6.58
99th percentile	6.74	7.28		**99th percentile**	7.05	7.63
USGS 02172067.7 Cooper River at I-526				USGS 02172100 Charleston Harbor at Fort Sumter		
	Existing	FWO			Existing	FWO
1st percentile	-0.33	0.22		**1st percentile**	-0.48	0.08
10th percentile	0.61	1.19		**10th percentile**	0.48	1.04
50th percentile	3.49	4.02		**50th percentile**	3.17	3.74
90th percentile	5.97	6.54		**90th percentile**	5.67	6.25
99th percentile	7	7.59		**99th percentile**	6.7	7.27

3.7.2 Currents

The 95th percentile depth-averaged simulated current speeds based on the typical flow regime for the Future Without scenario are shown in Figure 3.7.1 The current speeds are over 3 feet/second in the federal navigation channel near the harbor entrance and in the narrow bends of the upper Cooper River upstream from the federal navigation channel.

Figure 3.7.2 shows the change in 95th percentile depth-averaged simulated current speeds based on a typical flow regime from the Existing Conditions to the Future Without scenario. This shows that the 50-year SLC causes small changes in current speeds in the estuary (changes mostly less than 0.1 feet/second and some increases in current speed on the order of 0.1 to 0.2 feet/second in bends of the Cooper River upstream from the federal navigation channel). No impacts due to changes in current are expected. Impacts to ship maneuverability will be assessed during ship simulation in PED phase.

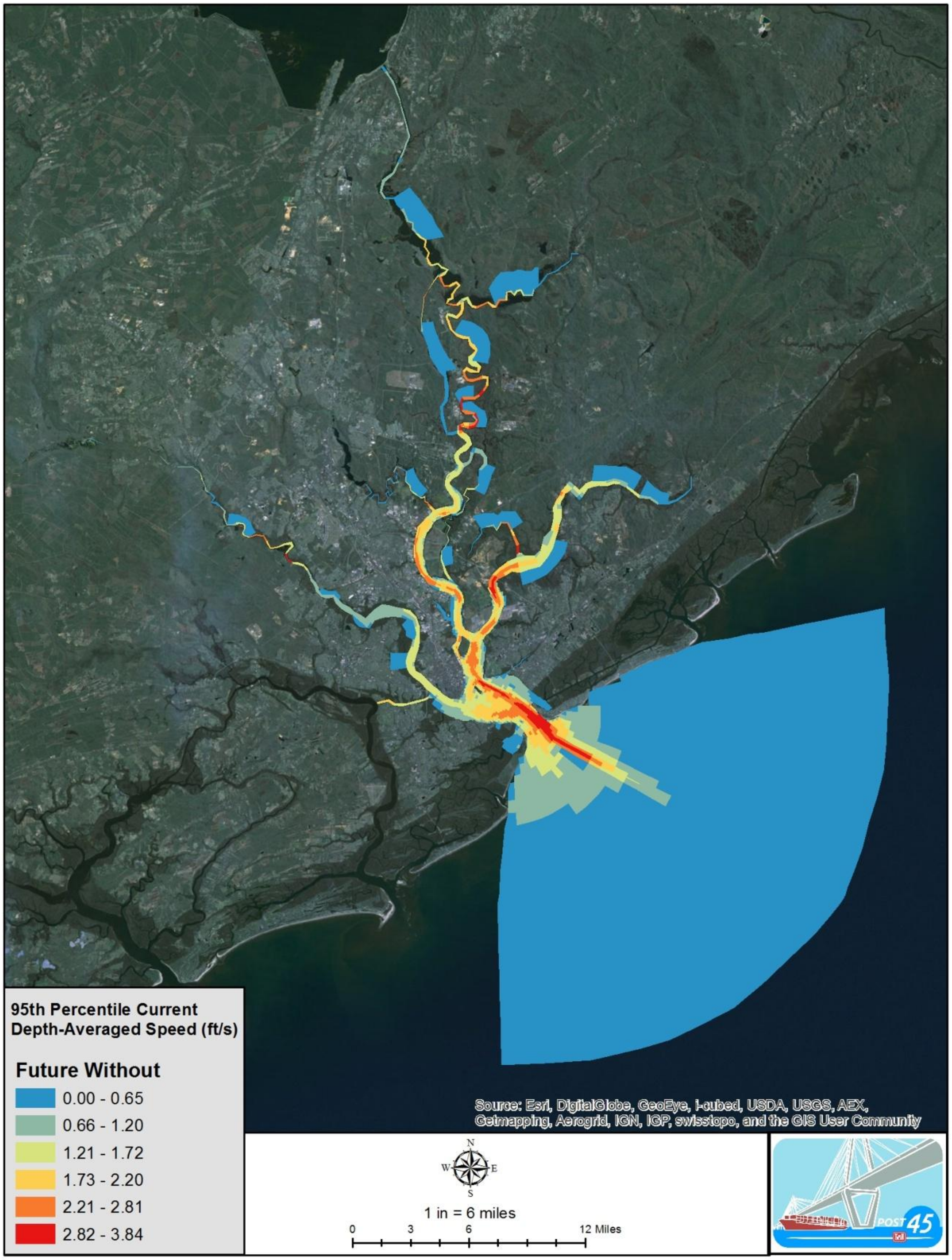


Figure 3.7.1 Future Without 95th percentile Depth-Averaged Current Speed

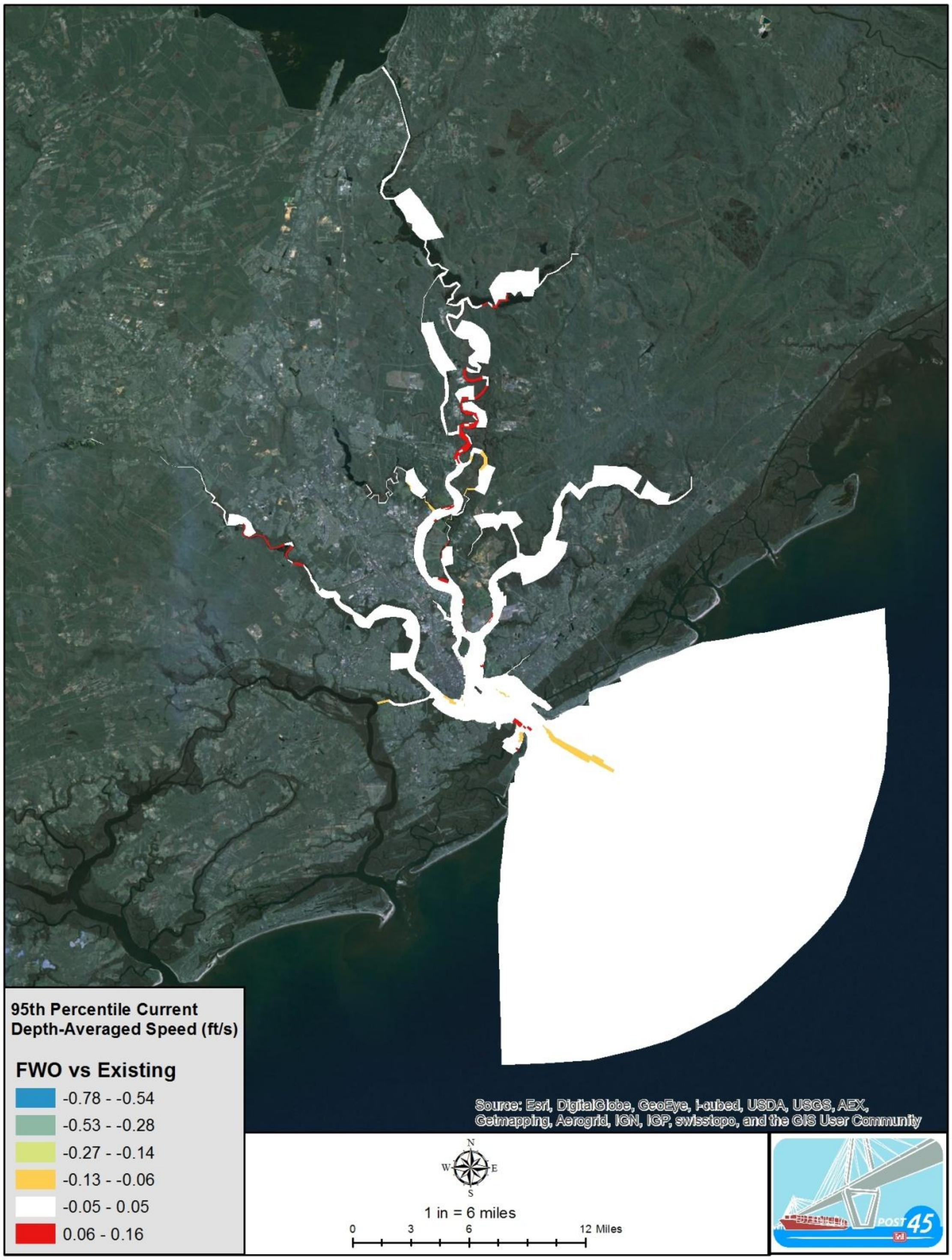


Figure 3.7.2 Future Without Change in 95th percentile Depth-averaged Current Speed from Existing Condition

3.7.3 Shoaling Rates

The sediment transport module of EFDC was used to assess potential changes to SSC in the water column and deposition rates in the federal navigation channel. As discussed in the model calibration Section 3.4 of this report, the model should not be used to simulate sediment transport over long time periods. Instead, the best approach to modeling potential changes in suspended sediment concentrations and federal navigation channel shoaling rates is to simulate the resuspension and deposition of sediments over a one-month period that includes typical spring and neap tidal conditions. This approach is suitable to identify the patterns of SSC and deposition in the federal navigation channel that are indicative of long term trends. Therefore, the results from the first month of the typical conditions simulation were used for the analysis. Also, similar to the model calibration simulations, for each scenario the model was first used to simulate a 14-day spring tide condition spin-up period, after which the sediment bed conditions were saved and used as the initial conditions for each scenario simulation.

The simulated deposition rates for the Future Without conditions are shown in Figure 3.7.3. The total inner harbor shoaling for the Future Without scenario is computed to be slightly less (4 percent less) than the Existing Conditions (Table 3.7.3.1). Since the existing condition model generated deposition rates are based on a typical flow regime of the year 2005 and the historic average is based on an average of the dredging records of different flow regimes from 2005 through 2012, the existing rate generated by the model does not equal the historic average maintenance dredging. Therefore, the impact analysis of alternatives does not rely on the model predicted absolute sedimentation rates. Instead, the relative change in sedimentation rates computed by the model is used in conjunction with observed long-term dredging rates.

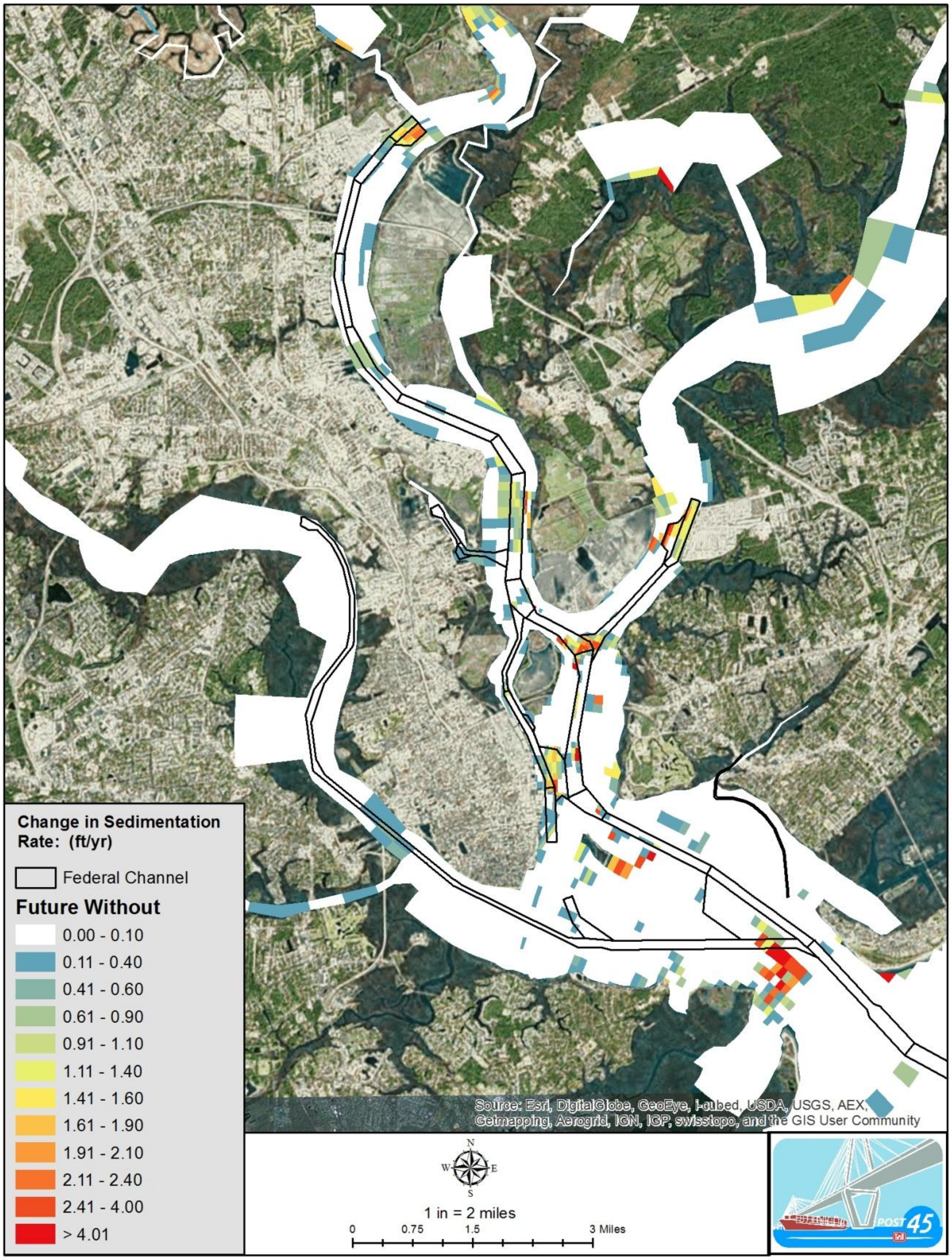


Figure 3.7.3. Future Without Simulated Sedimentation rates

Table 3.7.3.1 Simulated Shoaling Rates Existing vs Future Without

			Existing Conditions	Future Without Project
Reach	**Average Maintenance Dredging 2004-2012 (CY/yr)**		**Simulated Deposition Rate (CY/yr)**	**Simulated Deposition Rate (CY/yr)**
Mount Pleasant Reach	0		0	0
Rebellion Reach	0		0	0
Bennis Reach	0		10020	8323
Horse Reach	0		15662	16925
Hog Island Reach	117444		127662	114235
Drum Island Reach	91897		102239	108750
Meyers Bend Reach	23686		33237	29933
Daniel Island Reach	175287		113728	110225
Daniel Island Bend	10497		0	0
Clouter Creek Reach	0		3928	2618
Navy Yard Reach	28726		42739	42841
North Charleston Reach	0		5005	3175
Filbin Creek Reach	6504		0	0
Port Terminal Reach	4436		19176	18599
Ordnance Reach	144535		53711	51258
Ordnance Reach Turning Basin	327444		100075	100657
Wando River Lower Reach	58177		52346	53179
Wando River Upper Reach & Terminal	93457		116166	108771
Wando River Turning Basin	85515		138832	126883
Tidewater Reach & Union Pier	21762		34596	34296
Custom House Reach	51353		119131	118203
Town Creek Lower Reach	212216		39071	37953
	1452935		1127324	1086824

3.7.4 Salinity

Two primary concerns regarding salinity effects are: (1) changes to marsh vegetation caused by changes in the salinity regime; and (2) Salinity alerts that would require increased freshwater releases from Pinopolis Dam to prevent any salinity from reaching the inlet to the Back River (also known as the Bushy Park Reservoir). The Back River is an important freshwater supply source.

Salinity effects were evaluated for two flow conditions: typical and low-flow conditions. The model results are summarized by calculation of percentiles at selected USGS gage locations. Contour plots of annual average surface salinity concentrations are used to indicate the impacts to marsh vegetation. This is discussed in the environmental appendices.

For typical flow conditions, simulated water column averaged salinity values at selected USGS gage locations are listed in Table 3.7.4.1 for each scenario Existing Conditions, Future Without. Future Without salinity average annual condition is shown in Figure 3.7.4. The change in annual average water column averaged salinity from the Existing Conditions to the Future Without scenario is shown in Figure 3.7.5. Note that figures with salinity change do not show salinity changes in the offshore region. The model is not calibrated to predict changes in this area, and the predicted changes are small (i.e., less than 1 percent change in salinity).

This typical condition flow regime may change in the future because of the Bushy Park Reservoir salinity intrusion alert system implemented by the USACE (discussed in Section 1.4 of this appendix). Sea level change is expected to cause an increased frequency of tidal alerts (triggered by Customs House water levels) and possibly also result in an increase in salinity alerts (triggered by USGS gage specific conductivity readings). Because the percentile calculated by the post-processor is based on all time series output, it captures sequential alert levels and includes them in the computation. However, in reality, once an alert level is reached and required discharges are implemented, salinity levels should decrease so the next time step does not reach alert levels. Only one alert level is counted each day as an alert, unless the level increases to the next higher level. The EFDC model is not a reactive model and cannot capture the sequence of alert level having been reached and subsequent required discharge. Thus the model cannot be used to predict the increase in number of alerts. However, changes in percentiles may give an idea of the possibility of increased alerts, though not the quantity.

The model indicated there was not an increase in alert levels at USGS 02172020 West Branch Cooper River at Pimlico gage, whose alert criteria is 180 micro mhos/cm or approx .08 ppt. Historic records support that the typical year should not incur any alerts at the Pimlico gage and modeling indicates sea level change will not alter this.

The USGS 02172050 Cooper River near Goose Creek, whose alert criteria is 1550 micro mhos/cm or 0.76 ppt, indicted a greater than 98 percentile but less than 99 percentile of alert levels would occur under a typical year. Historical records indicate that only 1 occurred in the 2005-2006 and other typical years indicate only 2 or 3 alerts. The increase in alert levels for a typical year for the future without conditions indicated approximately a 94 percentile of alert levels. While this number does not reflect the actual number of alerts for the reason discussed in the previous paragraph, it does indicate at potential for increased salinity alerts. However, it is critical to remember salinity alerts may be prevented by the increase in tidal alerts due to sea level. Tidal alerts are intended to prevent a salinity alert. Tidal alert responses are generally performed within the required weekly flow releases and do not result in additional weekly average flows.   If there was a significant increase in alerts, USACE would evaluate the relocation of the intake to Bushy Park Reservoir farther upstream and reassessment of the alert system.

Table 3.7.4.1 Depth-Averaged Average Annual Salinity percentiles for Typical Conditions

USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC					USGS 02172069.8 Wando River above Mount Pleasant		
Salinity	Existing	FWO				Existing	FWO
1st percentile	0.05	0.05			**1st percentile**	19.1	19.48
10th percentile	0.05	0.05			**10th percentile**	20.14	20.49
50th percentile	0.05	0.05			**50th percentile**	21.79	22.13
90th percentile	0.05	0.05			**90th percentile**	24.3	24.63
99th percentile	0.05	0.05			**99th percentile**	25.81	26.11
USGS 02172050 Cooper River near Goose Creek, SC					USGS 02172070.9 Cooper River at Hwy 17		
	Existing	FWO				Existing	FWO
1st percentile	0.05	0.05			**1st percentile**	20.58	20.97
10th percentile	0.05	0.05			**10th percentile**	22.23	22.62
50th percentile	0.05	0.05			**50th percentile**	25.36	25.73
90th percentile	0.09	0.35			**90th percentile**	28.2	28.47
99th percentile	1.08	2.05			**99th percentile**	29.87	30
USGS 02172053 Cooper River at Mobay					USGS 02172086.9 Ashley River near North Charleston		
	Existing	FWO				Existing	FWO
1st percentile	0.06	0.12			**1st percentile**	11.41	12.37
10th percentile	0.89	1.76			**10th percentile**	13.94	14.74
50th percentile	6.1	6.92			**50th percentile**	16.59	17.35
90th percentile	10.3	10.91			**90th percentile**	18.37	19.11
99th percentile	12.67	13.2			**99th percentile**	19.41	20.15
USGS 02172067.7 Cooper River at I-526					USGS 02172100 Charleston Harbor at Fort Sumter		
	Existing	FWO				Existing	FWO
1st percentile	9.63	10.27			**1st percentile**	20.03	20.38
10th percentile	11.66	12.29			**10th percentile**	22.77	23.15
50th percentile	15.07	15.65			**50th percentile**	27.86	28.04
90th percentile	18.88	19.38			**90th percentile**	31.23	31.34
99th percentile	20.9	21.38			**99th percentile**	32.8	32.87

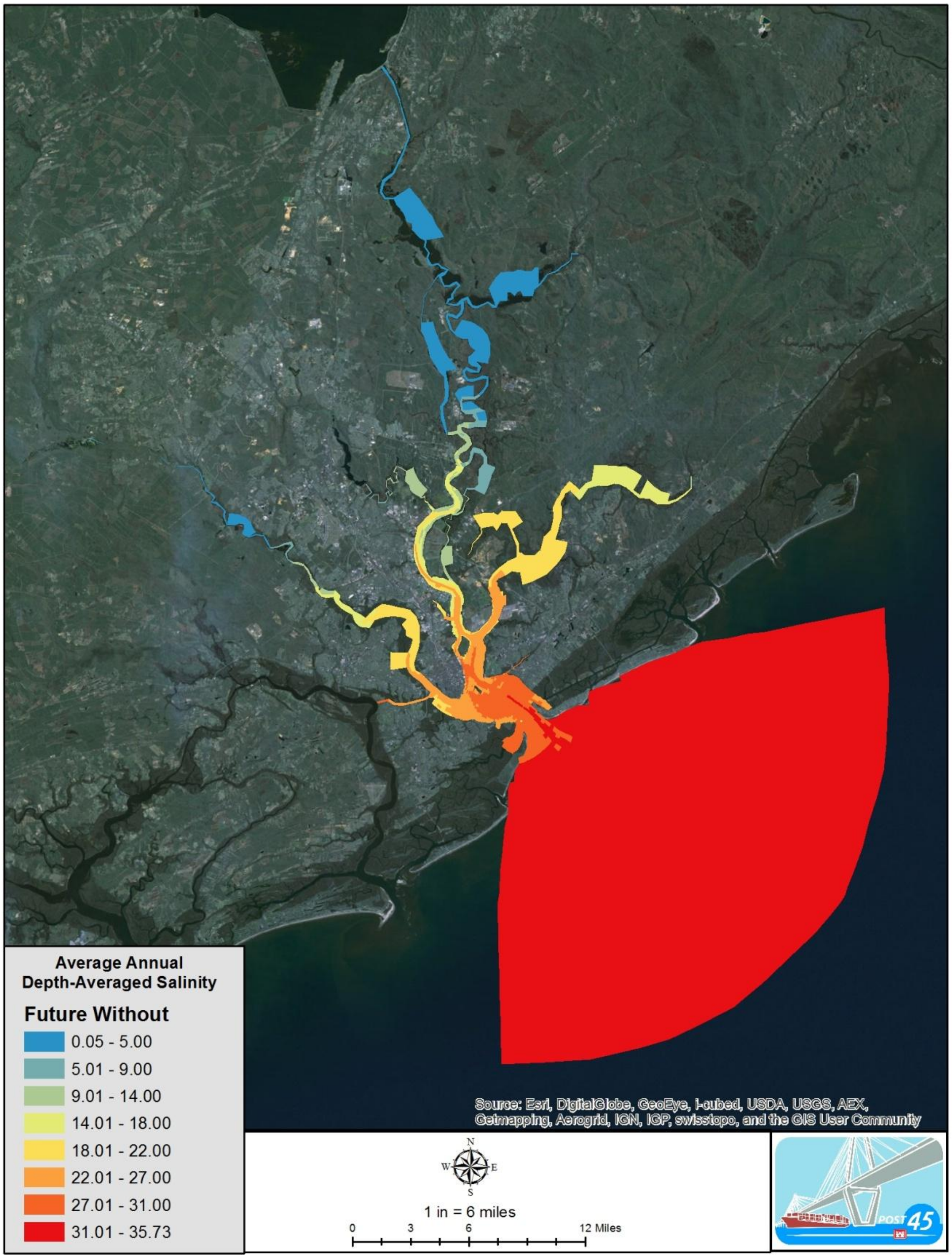


Figure 3.7.4 . Future Without in annual average surface salinity – typical flow conditions

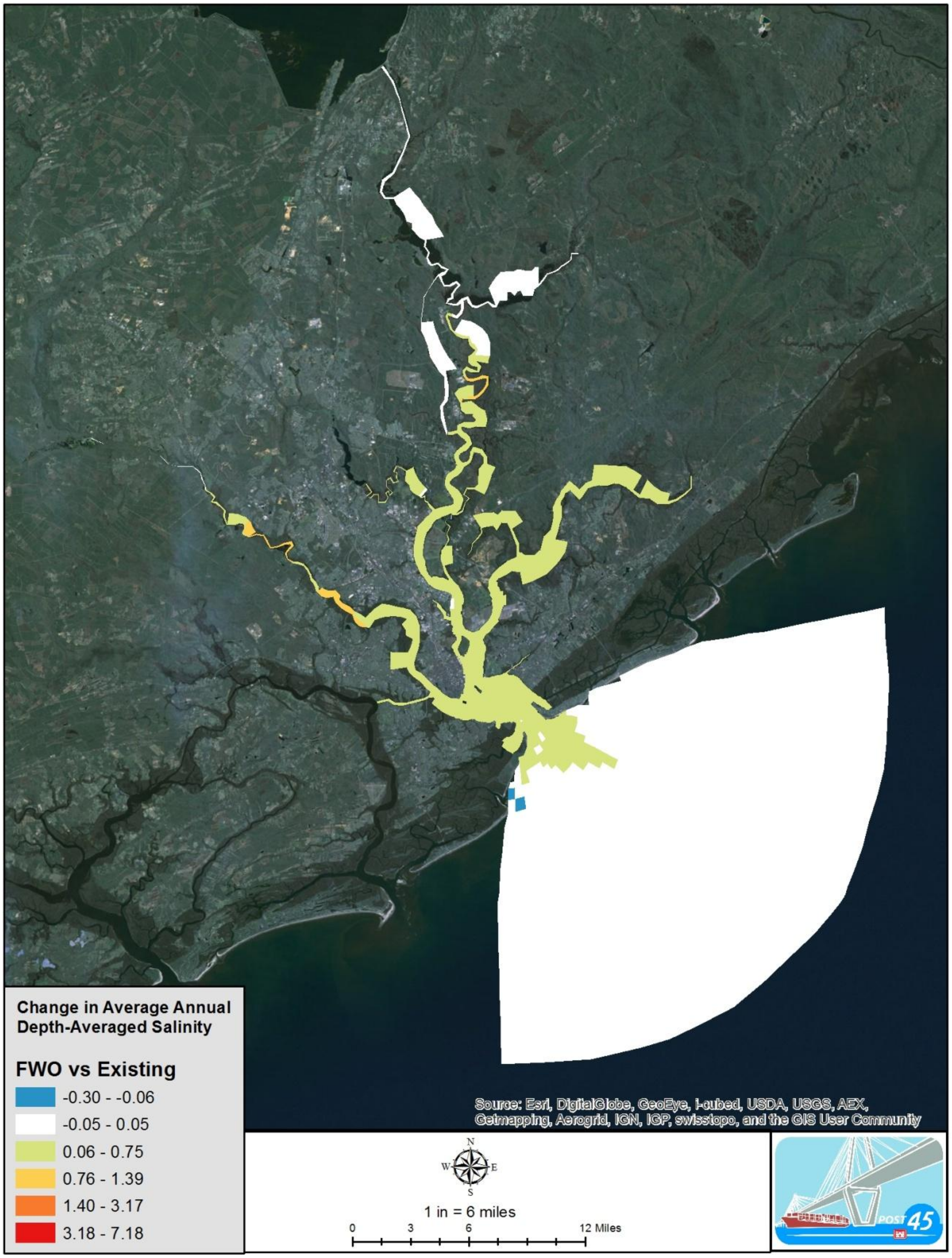


Figure 3.7.5 . Future Without change in annual average surface salinity from Existing Conditions – typical flow conditions

3.8 Alternative Analysis

3.8.1 Water Levels

In the harbor, the Ashley River and the Wando River, changes between the project alternatives and the Future Without scenarios in Tables 3.8.1.1 are very small (0.07 feet or less increase in 99th percentile water level). Therefore, the project alternatives are expected to cause a small increase in high tide water levels along the upper Cooper River. The increase does not change any of the impacts over the without condition alternative.

In regard to the Bushy Park Reservoir salinity intrusion alert system, it is expected that SLC will increase the frequency of alerts triggered by the Customs House water levels (i.e., tidal alerts). However, the project alternatives are not expected to increase the frequency of tidal alerts.

Table 3.8.1.1 Changes in Water Surface Elevations

Percentile		USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC					
WSE	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	3.14	3.15	3.15	3.15	3.15	3.14	3.14
10th percentile	3.76	3.76	3.76	3.77	3.77	3.77	3.77
50th percentile	4.68	4.7	4.7	4.7	4.7	4.69	4.69
90th percentile	5.56	5.58	5.58	5.58	5.57	5.57	5.57
99th percentile	6.18	6.19	6.19	6.18	6.18	6.18	6.18
		USGS 02172050 Cooper River near Goose Creek, SC					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	1.83	1.79	1.79	1.8	1.8	1.8	1.8
10th percentile	2.54	2.52	2.52	2.53	2.53	2.54	2.54
50th percentile	4.35	4.36	4.36	4.36	4.36	4.36	4.36
90th percentile	6.07	6.1	6.1	6.1	6.1	6.09	6.08
99th percentile	6.87	6.9	6.91	6.9	6.89	6.89	6.88
		USGS 02172053 Cooper River at Mobay					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	0.81	0.73	0.73	0.74	0.75	0.77	0.77
10th percentile	1.69	1.65	1.65	1.65	1.65	1.66	1.66
50th percentile	4.11	4.12	4.12	4.11	4.11	4.11	4.11
90th percentile	6.42	6.45	6.44	6.44	6.43	6.43	6.43
99th percentile	7.28	7.34	7.34	7.33	7.32	7.32	7.32
		USGS 02172067.7 Cooper River at I-526					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	0.22	0.12	0.13	0.15	0.15	0.16	0.17
10th percentile	1.19	1.05	1.12	1.13	1.13	1.15	1.15
50th percentile	4.02	4.01	4.01	4.01	4.01	4.02	4.02
90th percentile	6.54	6.59	6.59	6.58	6.58	6.58	6.57
99th percentile	7.59	7.66	7.65	7.64	7.64	7.63	7.63

		USGS 02172069.8 Wando River above Mount Pleasant					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	0.27	0.19	0.19	0.2	0.2	0.21	0.21
10th percentile	1.18	1.11	1.12	1.12	1.12	1.14	1.14
50th percentile	3.82	3.8	3.8	3.81	3.81	3.81	3.81
90th percentile	6.48	6.5	6.5	6.5	6.5	6.49	6.49
99th percentile	7.57	7.61	7.6	7.6	7.6	7.59	7.59
		USGS 02172070.9 Cooper River at Hwy 17					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	0.11	0.03	0.03	0.05	0.05	0.07	0.07
10th percentile	1.08	1.03	1.03	1.04	1.04	1.05	1.05
50th percentile	3.85	3.86	3.86	3.86	3.86	3.85	3.85
90th percentile	6.37	6.39	6.39	6.39	6.39	6.38	6.38
99th percentile	7.39	7.42	7.42	7.41	7.41	7.41	7.41
		USGS 02172086.9 Ashley River near North Charleston					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	-0.16	-0.23	-0.22	-0.22	-0.21	-0.2	-0.2
10th percentile	0.88	0.84	0.84	0.85	0.85	0.86	0.86
50th percentile	3.99	3.99	4	3.99	3.99	3.99	3.99
90th percentile	6.58	6.6	6.6	6.59	6.59	6.59	6.59
99th percentile	7.63	7.64	7.65	7.64	7.64	7.64	7.64
		USGS 02172100 Charleston Harbor at Fort Sumter					
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	0.08	0.03	0.03	0.04	0.04	0.06	0.06
10th percentile	1.04	1.01	1.01	1.02	1.02	1.03	1.03
50th percentile	3.74	3.73	3.73	3.73	3.73	3.74	3.74
90th percentile	6.25	6.27	6.27	6.26	6.26	6.26	6.26
99th percentile	7.27	7.29	7.29	7.29	7.29	7.28	7.28

3.8.2 Currents

Changes in 95th percentile depth-averaged simulated current speeds caused by the Alternatives scenario, resulted in very small increases in current speeds in the lower harbor (maximum increases on the order of 0.1 to 0.2 feet/second). Some areas in the navigation channel show reduction in current speed as a result of the channel deepening and widening, with the largest reductions occurring the turning basin expansion areas. Figure 3.8.2.1 shows the FWO conditions. Figure 3.8.2.2 shows the 50-48 compared to FWO and Figure 3.8.2.3 shows the 52-48 compared to FWO. No impacts due to changes in current are expected. Impacts to ship maneuverability will be assessed during ship simulation in PED phase.

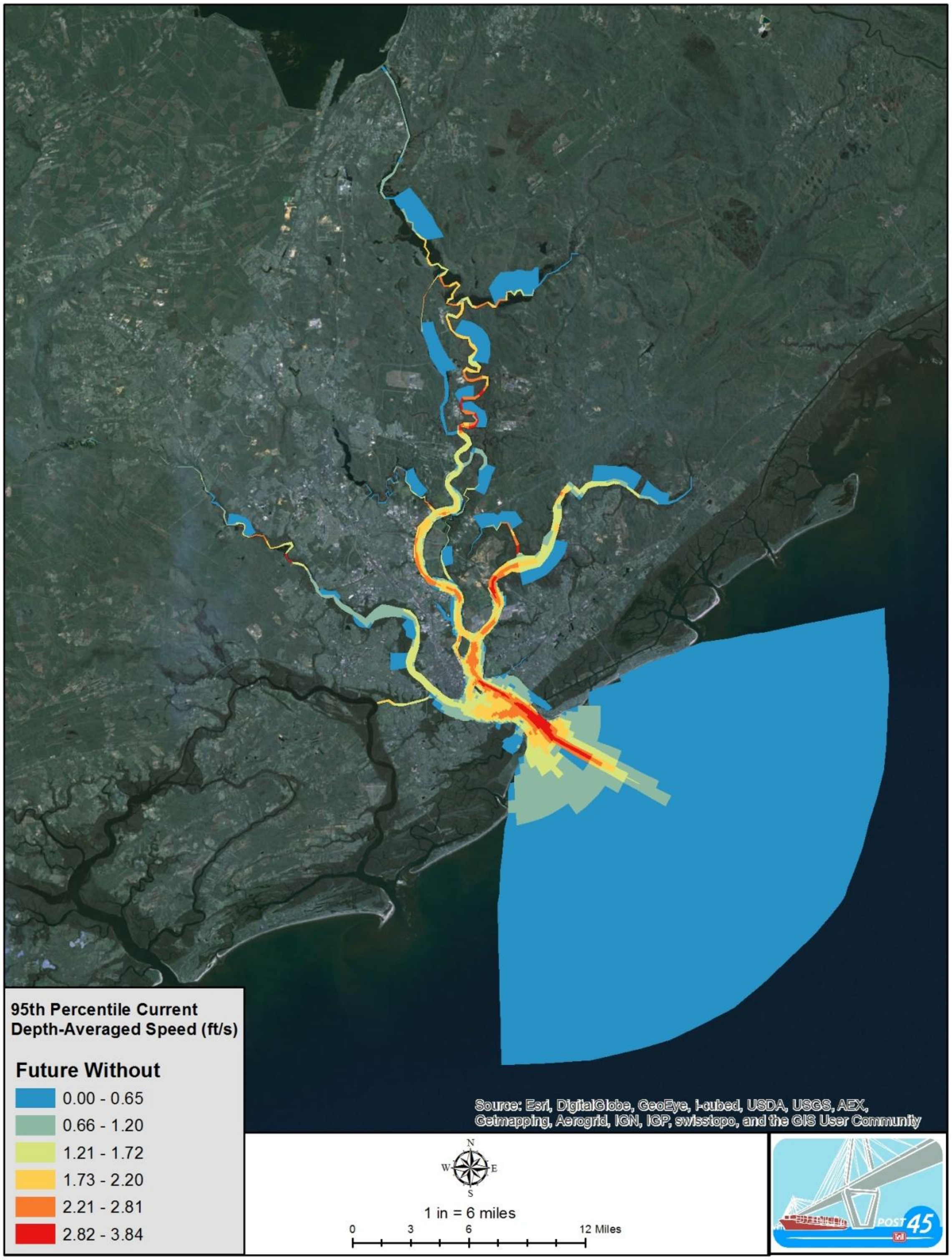


Figure 3.8.2.1 95th percentile Depth-Averaged Current Speed FWO condition

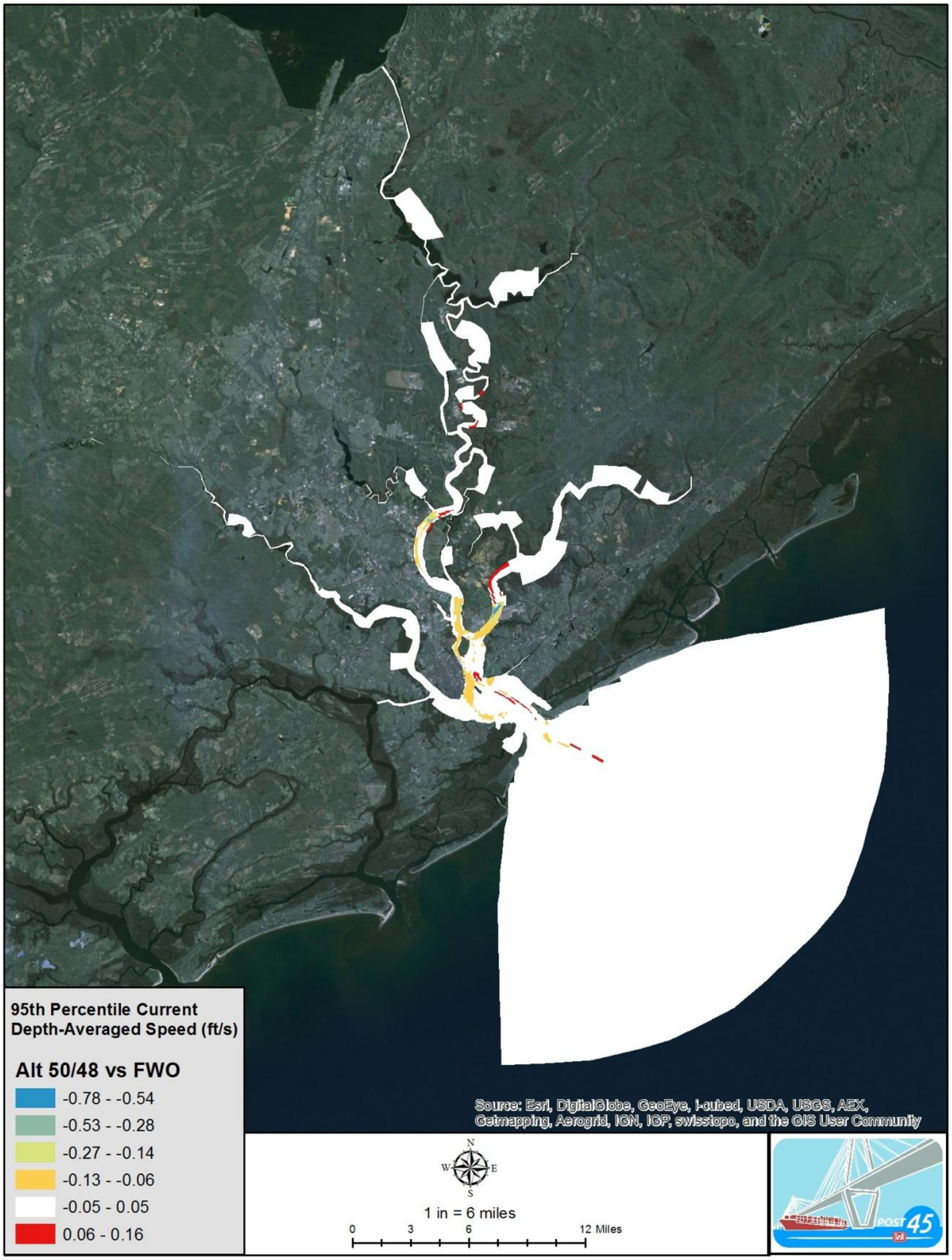


Figure 3.8.2.2 95th percentile Depth-Averaged Current Speed Alternative 50-48 vs FWO

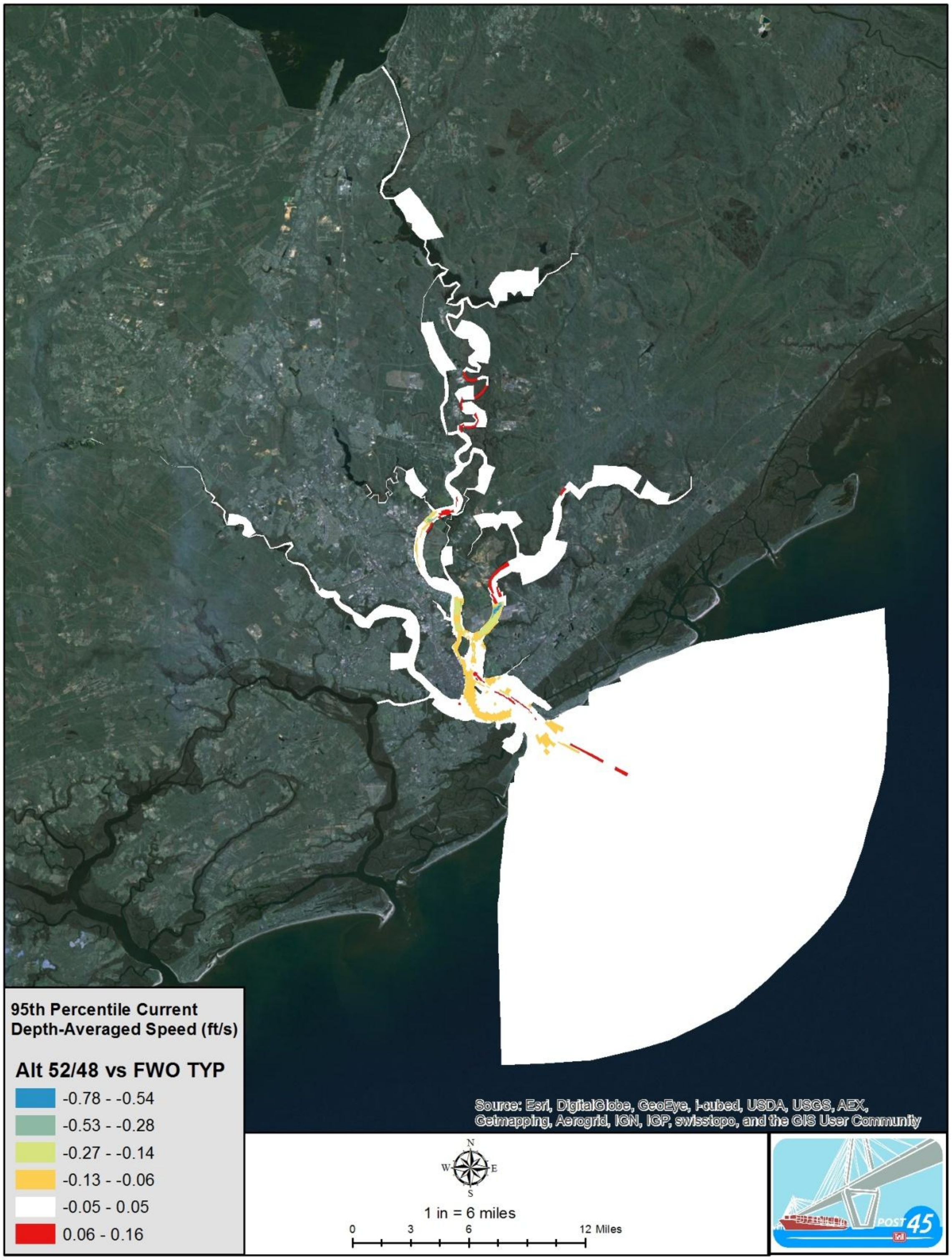


Figure 3.8.2.3 95th percentile Depth-Averaged Current Speed Alternative 52-48 vs FWO

3.8.3 Shoaling Rates:

The sediment transport module of EFDC was used to assess potential changes to suspended sediment concentration (SSC) in the water column and deposition rates in the federal shipping channel (federal navigation channel). The total shoaling rates in each reach of the inner harbor federal navigation channel were calculated for each deepening scenario.

Although the sedimentation rates computed are absolute values, the impact analysis does not rely on the model predicted absolute sedimentation rates (See Section 3.7). Instead, the relative change in sedimentation rates computed by the model is used in conjunction with observed long-term dredging records. For this purpose, the project effects are presented as shoaling rate indices in Table 3.8.3.1. The shoaling rate indices provide the relative change in sedimentation rate as compared to the Future Without scenario (which is assigned a shoaling rate index of 1.0). The relative change in cy/yr from the Future Without condition is shown for alternatives 50-48 and 52-48 in Figures 3.8.3.1 and 3.8.3.2

A theoretical scenario of 52-48 with no widening and another existing condition depth with maximum widening were done to estimate how much shoaling increase was due to deepening versus how much was done for widening. The estimate of 9% increase in shoaling for a deepening with no widening scenario closely approximates was actually occurred with the last deepening project that had minimal widening of reaches. The majority if the increase in shoaling is due to the widenings. This will be evaluated during ship simulation in PED phase.

Future dredging quantities for each scenario were estimated using the shoaling rate indices multiplied by historic dredging rates. Table 3.8.3.2 demonstrates that the modeling indicates increase in shoaling volumes for each reach for each deepening alternative assuming maximum widening alternative. The wideners are only a 9 percent increase in surface area. However, shoaling is not equally distributed through the system and changes in hydrodynamics may redistribute depositional locations. The greatest increase in sedimentation caused by the project alternatives will occur in the Wando River Upper Reach, Turning Basin and Terminal. The next largest predicted increase in sedimentation occurs in the Ordnance Reach & Turning Basin. Sedimentation is also predicted to increase along Hog Island Reach, Drum Island Reach, Meyers Bend Reach and Daniel Island Reach. Note there are four areas in which the model estimated shoaling that historical records do not support. These are Rebellion Reach, Bennis Reach, Clouter Creek Reach and North Charleston reach. For these reaches the model predicted sedimentation rates was used in estimating dredging quantities. It is estimated that the Wando TB increases 89 % over its existing size, Ordnance TB increase 76% over its existing size. Thus the majority of increased shoaling is due to the increase in footprint.

Evaluation of shoaling rates based on footprint of area remained fairly consistent with existing shoaling rates. Ordnance and Hog Island increases in shoaling rate are almost directly proportional to the increase in widening. However, evaluation of the rate of shoaling per square foot of area did indicate an increase in the Wando turning basin reach. An assessment of whether advance maintenance locations would change based on rate of shoaling concluded that the existing advance maintenance was justified and the only area affected was Wando Turning basin, which already is an area of advance maintenance. It is recommended that the Corps monitor the shoaling rate after construction to determine if further advance maintenance is needed.

Due to the increased shoaling, several combinations of contraction dikes were considered to determine if dredging needs could be decreased. Placement of contraction dikes around the Ordnance turning basin did not result in an overall improvement in shoaling, as the material was merely transported farther downstream, and dredging would result in deposition within the same disposal area. Several options in the Wando river and around the Wando turning basin were considered. Overall reductions were estimated and migration of shoaling into the lower harbor may also result in a reduction in upland disposal area placement, potentially lengthening the life of the disposal area. However, until ship simulation is done in PED phase, it is not known if the contraction dikes would be warranted, located in the same place or be of the same length. Therefore, contraction dike reductions will be included in PED phase, after ship simulation determines the final widening and turning basin measures that are to be constructed.

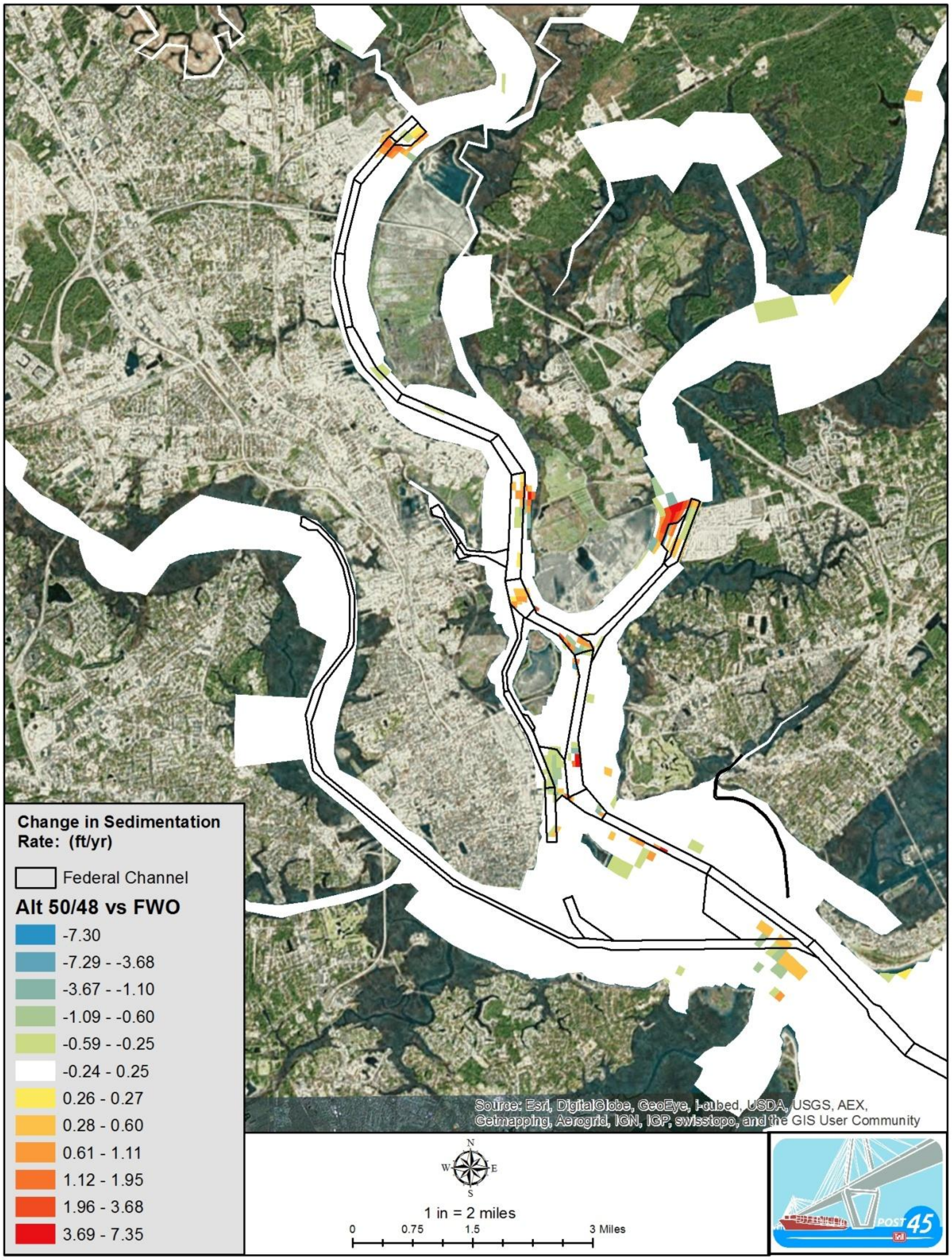


Figure 3.8.3.1 Alternative 50-48 vs FWO – Change in Sedimentation

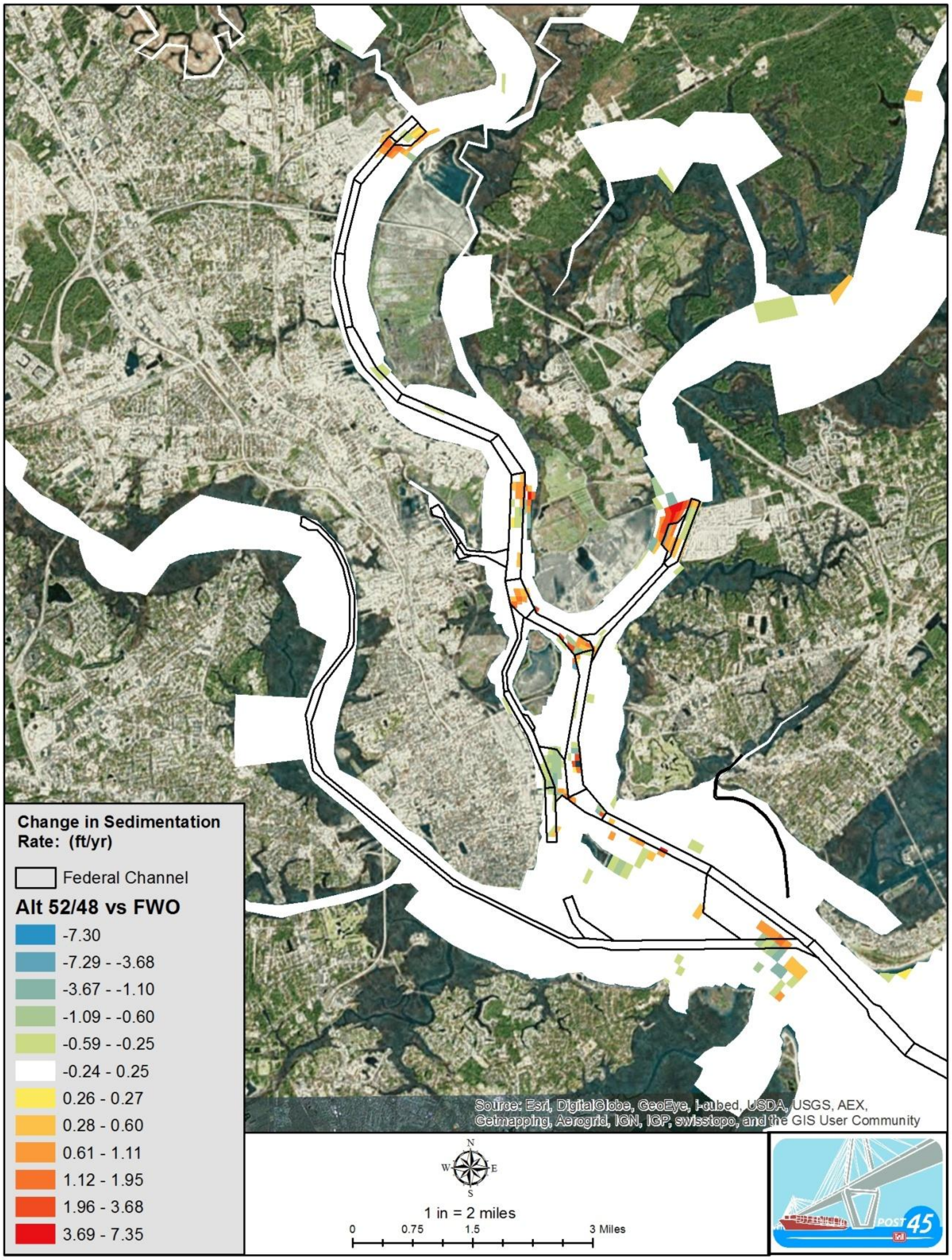


Figure 3.8.3.2 Alternative 52-48 vs FWO – Change in Sedimentation

Table 3.8.3.1 Shoaling Rate Indices

Shoaling Rate Indices	Future Without Project	Alt 48-47	Alt 50-48	Alt 48-48	Alt 52-47	Alt 50-47	Alt 52-48	wide-no deep	52-48 no wide
Mount Pleasant Reach	1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Rebellion Reach	1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Bennis Reach	1.0	2.96	4.10	2.91	4.52	4.10	4.48	0.30	4.90
Horse Reach	1.0	0.71	0.74	0.71	0.95	0.72	0.95	0.43	1.07
Hog Island Reach	1.0	1.36	1.44	1.38	1.51	1.42	1.53	1.05	1.22
Drum Island Reach	1.0	1.24	1.29	1.24	1.41	1.29	1.43	1.19	0.99
Meyers Bend Reach	1.0	1.93	2.09	1.92	2.33	2.10	2.33	1.79	1.42
Daniel Island Reach	1.0	1.28	1.25	1.19	1.41	1.36	1.32	1.28	0.92
Daniel Island Bend	1.0	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Clouter Creek Reach	1.0	14.23*	12.70*	12.64*	13.83*	14.03*	12.79*	16.21*	0.50
Navy Yard Reach	1.0	0.79	0.78	0.83	0.74	0.79	0.75	0.87	0.93
North Charleston Reach	1.0	1.35	1.28	1.49	1.31	1.31	1.61	1.46	1.78
Filbin Creek Reach	1.0	1.74	1.67	1.65	1.76	1.74	1.65	1.91	1.00
Port Terminal Reach	1.0	3.31	3.30	3.27	3.42	3.46	3.29	3.42	1.26
Ordnance Reach	1.0	1.16	1.14	1.14	1.16	1.14	1.15	1.19	0.93
Ordnance Reach Turning Basin	1.0	1.62	1.62	1.62	1.63	1.62	1.63	1.62	1.00
Wando River Lower Reach	1.0	1.09	1.16	1.09	1.20	1.16	1.20	1.02	0.99
Wando River Upper Reach & Terminal	1.0	1.02	1.06	1.02	1.09	1.06	1.09	0.93	1.06
Wando River Turning Basin	1.0	2.98	3.07	2.98	3.08	3.07	3.08	2.84	1.10
Tidewater Reach & Union Pier	1.0	1.21	1.07	1.18	0.93	1.06	0.92	1.43	1.00
Custom House Reach	1.0	0.99	0.81	0.99	0.66	0.81	0.66	1.21	1.11
Town Creek Lower Reach	1.0	1.42	1.27	1.43	1.10	1.25	1.11	1.64	0.93

- Historical dredging records indicate that Clouter creek reach does not have any annual shoaling, while the EFDC model did generate a small shoaling rate there (less than 2% of total shoaling) . The widening measure in the model overestimates the increase that would occur. This is concluded to be a result of the small widener cell when adjacent cells are much wider. The actual model generated quantity was used for cost estimating to be conservative.

Table 3.8.3.2 Shoaling Quantities

	Average Maintenance Dredging 2004-2012 (CY/yr)	Alternative 48-47	Alternative 50-48	Alternative 48-48	Alternative 52-47	Alternative 50-47	Alternative 52-48
Mount Pleasant Reach	0	0	0	0	0	0	0
Rebellion Reach	0	0	0	0	615	0	923
Bennis Reach	0	24664*	34138*	24259*	37634*	34138*	37264*
Horse Reach	0	12012*	12457*	12099*	16035*	12210*	16035*
Hog Island Reach	117444	159205	169094	162086	177854	167265	179838
Drum Island Reach	91897	114075	118305	113966	129927	118106	131287
Meyers Bend Reach	23686	45613	49538	45560	55280	49648	55119
Daniel Island Reach	175287	223665	218978	209287	246705	237817	231652
Daniel Island Bend	10497	10497	10497	10497	10497	10497	10497
Clouter Creek Reach	0	37251*	33243*	33102*	36201*	36741*	33501*
Navy Yard Reach	28726	22761	22271	23751	21339	22581	21520
North Charleston Reach	0	4272*	4075*	4734*	4156*	4156*	5104*
Filbin Creek Reach	6504	11307	10883	10742	11448	11307	10742
Port Terminal Reach	4436	14697	14632	14516	15178	15361	14581
Ordnance Reach	144535	167635	165254	164922	167423	165438	166433
Ordnance Reach Turning Basin	327444	528996	530448	530742	532577	529658	532713
Wando River Lower Reach	58177	63222	67723	63367	69811	67748	69984
Wando River Upper Reach & Terminal	93457	95501	98954	95645	102133	98954	101985
Wando River Turning Basin	85515	254717	262293	255216	263097	262434	263097
Tidewater Reach & Union Pier	21762	26243	23250	25759	20179	23171	20021
Custom House Reach	51353	50690	41808	50777	33895	41733	34047
Town Creek Lower Reach	212216	301796	268535	303270	233456	265071	235123
	Total 1452935	2168816	2156377	2154296	2185440	2174036	2171467
	Note * actual model generated value was used as historical dredging does not occur in this area.						

EFDC modeling did not include the entrance channel in computations for shoaling and all coastal modeling was delegated to PED phase on the SMART planning (3-3-3) method. Dredging records since last deepening estimated 407,000 cy annually dredged from the entrance channel. The annual dredging prior to the last deepening was 328,400 cy annually. Therefore, the Final Feasibility Report for Charleston Harbor, Charleston SC dated February 1996 which estimated an increase of annual maintenance of 16,000 cy per foot of depth dredged (80,000 for a increase of 5 feet in depth) is assumed to be a reasonable estimate for dredging increases in the entrance channel.

3.8.4 Salinity

The proposed channel deepening will increase the salinity concentrations in the estuary. Two primary concerns regarding this potential effect on salinity are: (1) changes to marsh vegetation caused by changes in the salinity regime; and (2) the need for increased freshwater releases from Pinopolis Dam to prevent any salinity from reaching the inlet to the Back River (also known as the Bushy Park Reservoir). The Back River is an important freshwater supply source.

Salinity effects were evaluated for two flow conditions: typical and low-flow conditions. Changes to marsh vegetation are based on surface salinity at low flow conditions and this is discussed in the environmental appendices of the report. The model results for typical conditions are summarized by calculation of percentiles at selected USGS gage locations (Table 3.8.4.1.)

The model indicated there was not an increase in alert levels at USGS 02172020 West Branch Cooper River at Pimlico gage, whose alert criteria is 180 micro mhos/cm or approx .08 ppt. Historic records and the Future without condition simulation support that the typical year should not incur any alerts at the Pimlico gage and modeling indicates the project will not alter this.

The USGS 02172050 Cooper River near Goose Creek, whose alert criteria is 1550 micro mhos/cm or .76 ppt, indicted an increase of alert levels would occur under a typical year for all project alternatives compared to the future without condition. However, it is not a reactive model. Without a reactive model analysis that includes flow releases due to a tidal alert, as well as salinity alert flow responses, there is no way to determine if or how much increase in salinity alerts would occur due to the project.

The majority of increase releases would likely be due to sea level rise and tidal alerts, not salinity migration. The contract agreement for flows are based on flow rates that the Corps desires in order to reduce shoaling in the harbor. Santee Cooper would actually prefer to release through Pinopolis into the Cooper River due to more power production, but USACE limit the flows per contractual agreement. Tidal alerts are usually done within the existing contract requirements and are expected to do so. Salinity alerts are usually in years of drought where the natural watershed flows are reduced. The additional flow releases from Pinopolis offset the loss of watershed runoff flows. So when alerts require more flows from Pinopolis, the associated cost would be shoaling of the harbor not payment to SCPSA. These alerts are of short duration and not expected to affect annual shoaling rates, as they have not thus far. The Corps will continue the salinity alert monitoring and the protection of Bushy Park Reservoir.

Table 3.8.4.1 Depth-Averaged Salinity Percentiles for Typical Flow Conditions

USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC							
Salinity	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile							
10th percentile							
50th percentile							
90th percentile							
99th percentile	0.05	0.05	0.05	0.05	0.05	0.05	0.05
USGS 02172050 Cooper River near Goose Creek, SC							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile							
10th percentile							
50th percentile	0.05	0.05	0.05	0.05	0.05	0.05	0.05
90th percentile	0.35	0.6	0.57	0.54	0.52	0.49	0.46
99th percentile	2.05	2.58	2.55	2.44	2.47	2.32	2.25
USGS 02172053 Cooper River at Mobay							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	0.12	0.19	0.19	0.17	0.16	0.15	0.15
10th percentile	1.76	2.23	2.22	2.16	2.11	2.04	1.99
50th percentile	6.92	7.81	7.71	7.62	7.54	7.44	7.34
90th percentile	10.91	11.89	11.81	11.66	11.59	11.44	11.35
99th percentile	13.2	14.09	13.97	13.86	13.8	13.62	13.54
USGS 02172067.7 Cooper River at I-526							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	10.27	11.3	11.19	11.06	10.99	10.89	10.78
10th percentile	12.29	33.15	13.29	13.15	13.06	12.92	12.84
50th percentile	15.65	16.9	16.84	16.66	16.6	16.41	16.35
90th percentile	19.38	20.78	20.77	20.49	20.47	20.18	20.17
99th percentile	21.38	22.68	22.67	22.37	22.38	22.07	22.07
USGS 02172069.8 Wando River above Mount Pleasant							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	19.48	21.86	21.84	21.49	21.47	21.12	21.1
10th percentile	20.49	22.7	22.69	22.34	22.33	21.97	21.95
50th percentile	22.13	24.13	24.11	23.78	23.76	23.43	23.4
90th percentile	24.63	25.87	25.87	25.59	25.58	25.31	25.3
99th percentile	26.11	27.16	27.14	26.91	26.91	26.66	26.64
USGS 02172070.9 Cooper River at Hwy 17							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	20.97	21.85	21.85	21.66	21.66	21.47	21.44
10th percentile	22.62	23.39	23.38	23.21	23.19	23.01	23
50th percentile	25.73	26.23	26.24	26.07	26.08	25.92	25.91
90th percentile	28.47	28.77	28.78	28.68	28.69	28.59	28.58
99th percentile	30	30.2	30.19	30.12	30.12	30.05	30.05

Table 3.8.4.1 continued Depth-Averaged Salinity Percentiles for Typical Flow Conditions							
USGS 02172086.9 Ashley River near North Charleston							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	12.37	12.75	12.69	12.62	12.69	12.59	12.6
10th percentile	14.74	15.17	15.16	15.09	15.1	15	14.99
50th percentile	17.35	17.77	17.76	17.66	17.67	17.58	17.57
90th percentile	19.11	19.54	19.56	19.43	19.45	19.32	19.34
99th percentile	20.15	20.64	20.65	20.51	20.55	20.43	20.42
USGS 02172100 Charleston Harbor at Fort Sumter							
	FWO	52-48	52-47	50-48	50-47	48-48	48-47
1st percentile	20.38	21.05	21.03	20.89	20.89	20.77	20.77
10th percentile	23.15	23.7	23.67	23.54	23.55	23.44	23.42
50th percentile	28.04	28.28	28.26	28.2	28.23	28.17	28.17
90th percentile	31.34	31.42	31.41	31.41	31.41	31.39	31.38
99th percentile	32.87	32.9	32.87	32.88	32.87	32.87	32.86

Graphical representation change in average annual depth averaged salinity for of the 50-48 alternative versus Future Without is shown in Figure 3.8.4.1 and change in average annual depth averaged salinity for of the 52-48 alternative versus Future Without is shown in Figure 3.8.4.2.

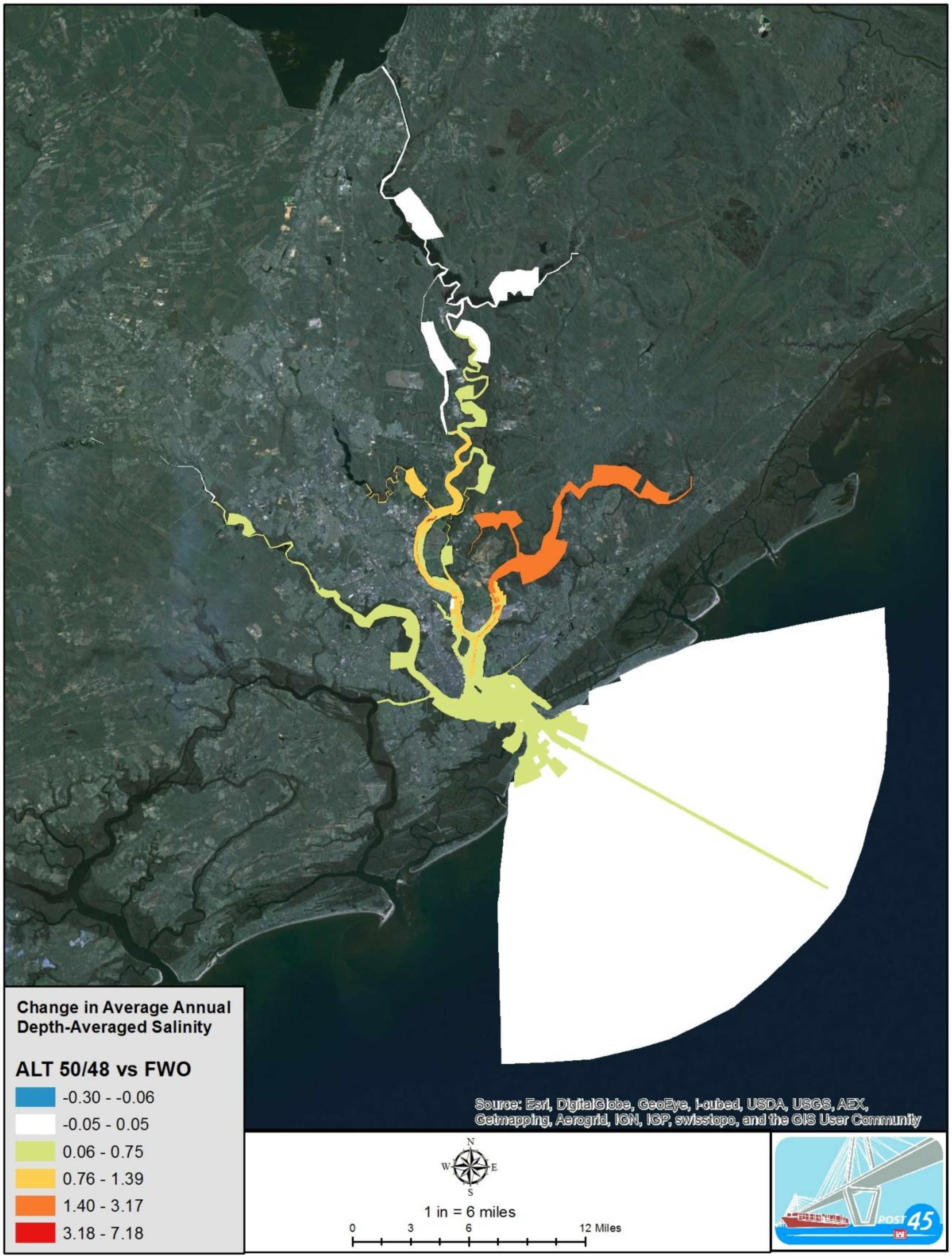


Figure 3.8.4.1 Change in Average Annual Depth Averaged Salinity for 50-48 –FWO

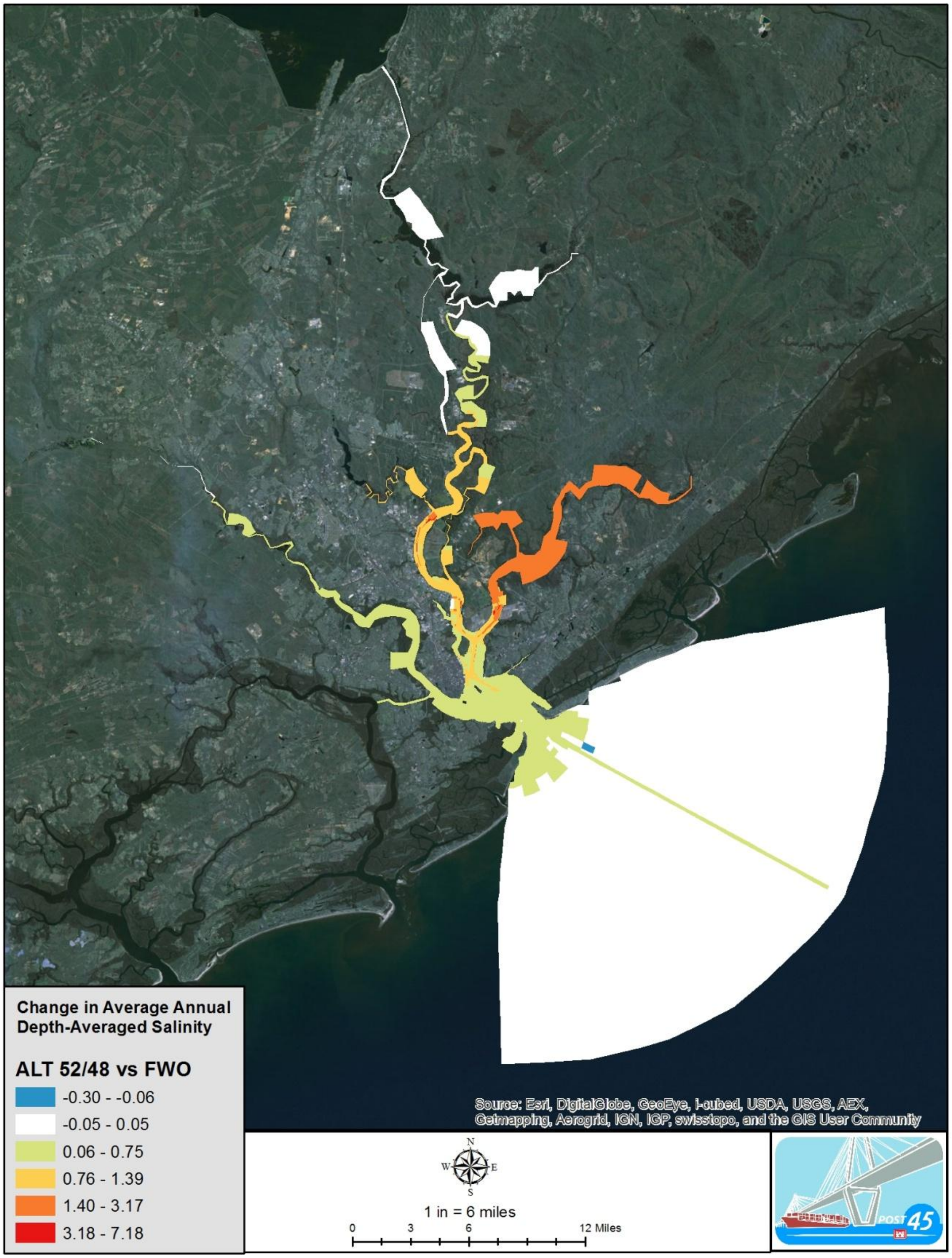


Figure3.8.4.2 Change in Average Annual Depth Averaged Salinity for 52-48-FWO

3.8.5 Dissolved Oxygen

Adequate DO concentration in the water column is necessary to support a healthy aquatic ecosystem. Therefore, the potential project effects on DO are one of the primary environmental impact concerns.

Portions of the Charleston Harbor do not meet the South Carolina water quality standard for DO. Therefore, in accordance with Section 303(d) of the Clean Water Act, a TMDL for the estuary has been established to limit discharges of oxygen demanding substances. As explained in the TMDL, the waters in and around Charleston Harbor are considered to be both naturally low in DO and further impacted by wastewater dischargers. Natural factors such as organic loading and reduced oxygen levels from wetlands and marshes and estuarine dynamics in the mixing zone where freshwater and saltwater come together can create naturally low DO conditions (Cantrell 2013).

In South Carolina, waters that do not meet numeric criteria for DO due to natural conditions are covered by anti-degradation requirements in S.C. R.61-68, Section D.4 as follows:

> *4. Certain natural conditions may cause a depression of dissolved oxygen in surface waters while existing and classified uses are still maintained. The Department shall allow a dissolved oxygen depression in these naturally low dissolved oxygen waterbodies as prescribed below pursuant to the Act, Section 48-1-83, et seq., 1976 Code of Laws:*
>
> *a. For purposes of section D. of this regulation, the term "naturally low dissolved oxygen waterbody" is a waterbody that, between and including the months of March and October, has naturally low dissolved oxygen levels at some time and for which limits during those months shall be set based on a critical condition analysis. The term does not include the months of November through February unless low dissolved oxygen levels are known to exist during those months in the waterbody. For a naturally low dissolved oxygen waterbody, the quality of the surface waters shall not be cumulatively lowered more than 0.1 mg/l for dissolved oxygen from point sources and other activities; or*
>
> *b. Where natural conditions alone create dissolved oxygen concentrations less than 110 percent of the applicable water quality standard established for that waterbody, the minimum acceptable concentration is 90 percent of the natural condition. Under these circumstances, an anthropogenic dissolved oxygen depression greater than 0.1 mg/l shall not be allowed unless it is demonstrated that resident aquatic species shall not be adversely affected pursuant to Section*

48-1-83. The Department may modify permit conditions to require appropriate instream biological monitoring.

c. The dissolved oxygen concentrations shall not be cumulatively lowered more than the deficit described above utilizing a daily average unless it can be demonstrated that resident aquatic species shall not be adversely affected by an alternate averaging period.

These provisions allow a lowering of DO of no more than 0.1 mg/L. In practice, a lowering of 0.1499 mg/L is allowable, as evidenced by the allowed DO depression in the Charleston Harbor TMDL (Cantrell 2013).

The dissolved oxygen concentrations in the CHS are controlled by many factors. The processes through which a navigation channel expansion may affect dissolved oxygen include:

- changes in reaeration, as a result of changes in water depth or current speed;
- changes in dissolved oxygen saturation concentration, as a result of changes in salinity or temperature;
- changes in residence time of oxygen demanding substances, as a result of changes in the tributary hydrodynamics.

The EFDC water quality model includes all of the above major processes that affect the DO concentrations in the CHS. Therefore, the model will provide reasonable estimates of potential changes in DO concentration resulting from the project alternatives.

Model-predicted DO concentrations require appropriate spatial and time averaging when interpreting the results. The best use of the model is in evaluating large scale aggregate changes to the system caused by the project, and the model is less accurate for predicting changes on a cell-by-cell basis. Therefore, it is appropriate to average groups of grid cells (horizontally and vertically) when interpreting model results. Figure 3.8.5.1 presents the horizontal segments used to spatially average DO concentrations. This is the same segmentation as that used by DHEC for determining the DO TMDL for the estuary (Cantrell 2013).

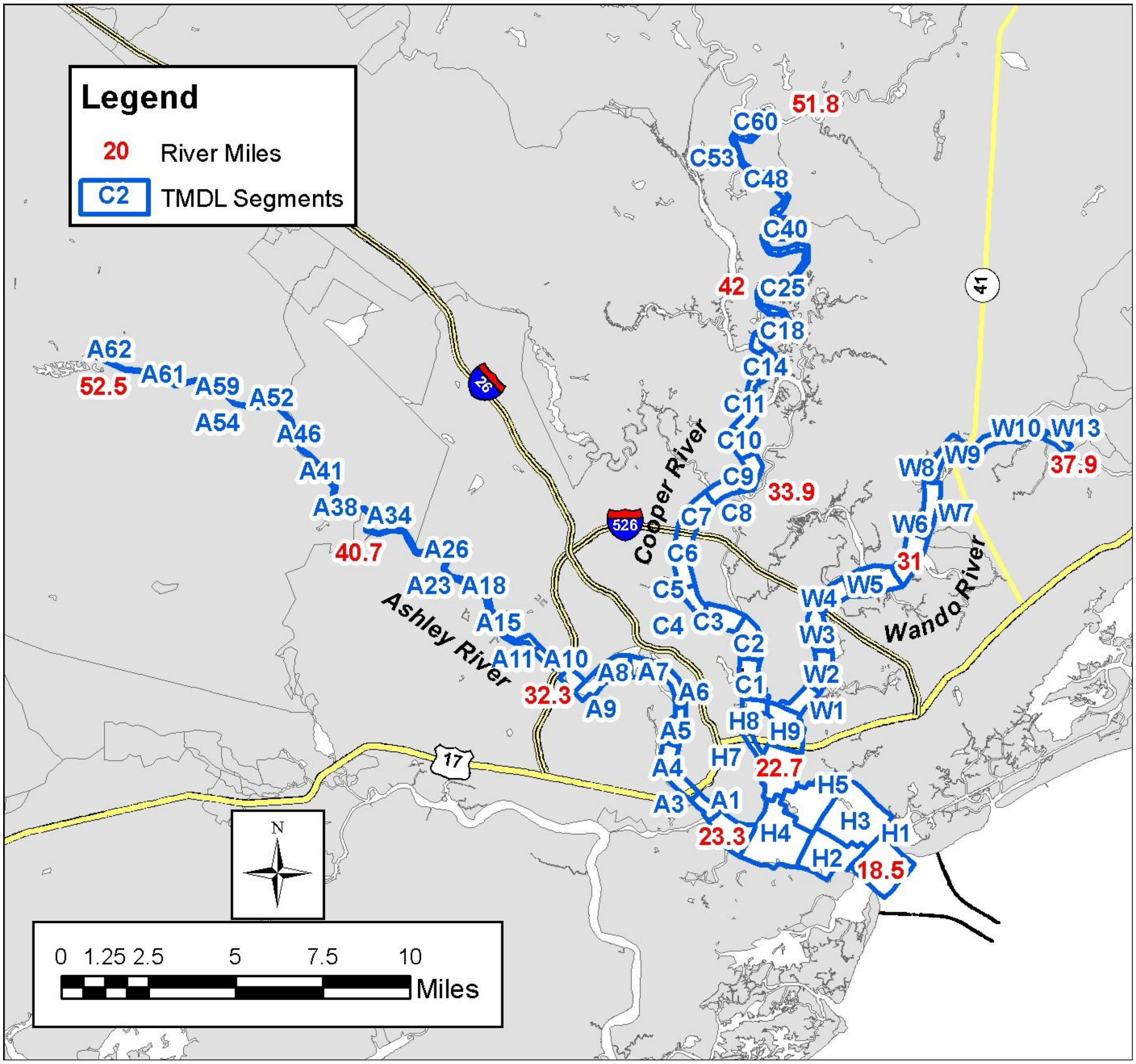


Figure 3.8.5.1 Volume Averaging Segments

3.8.5.1 Proposed Project Impacts on Dissolved Oxygen

Through its use of an Environmental Fluid Dynamics Code (EFDC) model, modified from the original EFDC model used for the Charleston Harbor DO Total Maximum Daily Load (TMDL) analysis, the Charleston District has determined that all of the Post 45 alternatives will have the effect of reducing dissolved oxygen within the Charleston Harbor system. The Post 45 project effects on oxygen also have a cumulative effect in addition to the DO effects already caused by point-source pollution discharges in the harbor. Determining whether the direct, indirect or cumulative effect is significant enough to require compensatory mitigation requires careful consideration of the methodology used to evaluate the Post 45 alternatives in conjunction with the existing point-source pollution discharges into the Charleston Harbor system. The dissolved oxygen impacts and impact on the TMDL were performed in consultation and with approval of SCDHEC and EPA.

The assumptions used to establish the existing DO TMDL for Charleston Harbor are overly conservative for use in a realistic Post 45 project cumulative impact analysis. The DO TMDL was calculated based on the assumption that all of the NPDES discharges are constantly and simultaneously discharging at the maximum permitted load. Although the methodology used by DHEC is common for the purposes of establishing the waste load allocation (WLA), it does not mean that it is an accurate assumption for the purposes of the Post 45 cumulative impact analysis, which should be evaluated based on more realistic model assumptions. This assumption does not recognize the time-varying nature of the individual point-source discharge loading rates, which is particularly important for a system with multiple point-source dischargers. In general, point-source discharges tend to have a wide range of discharge rates that occur over time. For example, as shown in Figure 3.8.5.1.1, the daily ultimate oxygen demand (UOD) pollutant loading rate from the largest discharger into to the harbor (Kapstone) varies by more than an order of magnitude over time. Figure 3.8.5.1.2 shows the cumulative probability distribution curve for the same measured UOD loading rate data shown in Figure 3.8.5.1.1. As shown by this figure, the median pollutant load (i.e., the load not exceeded 50 percent of the time) is much smaller than the peak load (e.g., the 99th percentile load, which is the load not exceeded 99 percent of the time). Similar to the discharge shown in these figures, each of the other discharges in the estuary has a range of possible discharge rates and associated probability distribution. The probability of all dischargers being at the maximum load at the same point in time is extremely small, and it is even less likely that these discharges would be sustained at that constant maximum permitted load over the entire TMDL analysis time period (March through October). Although DHEC used the conservative assumption of constant discharge rates for the purposes of establishing the Waste Load Allocation for the TMDL, this analysis for the Post 45 project uses improved methods that provide a more accurate approach to characterize the point-source discharges. Specifically, in order to incorporate the time-varying nature of the point-source discharges, this analysis uses time-varying discharge loading rates input to the TMDL model that are based on measured daily discharge data collected by the existing dischargers. The use of time-varying loading rates has been used elsewhere for discharge permitting, including: the Savannah Harbor dissolved oxygen TMDL (USEPA 2010; HydroQual 2010), and the NPDES permitting for the MeadWestvaco Pulp and Paper Mill in Covington, Virginia (USEPA 2010). In addition, the EPA’s Technical Support Document for Toxics (USEPA

1991) describes the use of dynamic modeling of time-varying discharges to calculate probability distributions for statistically-based permit limits.

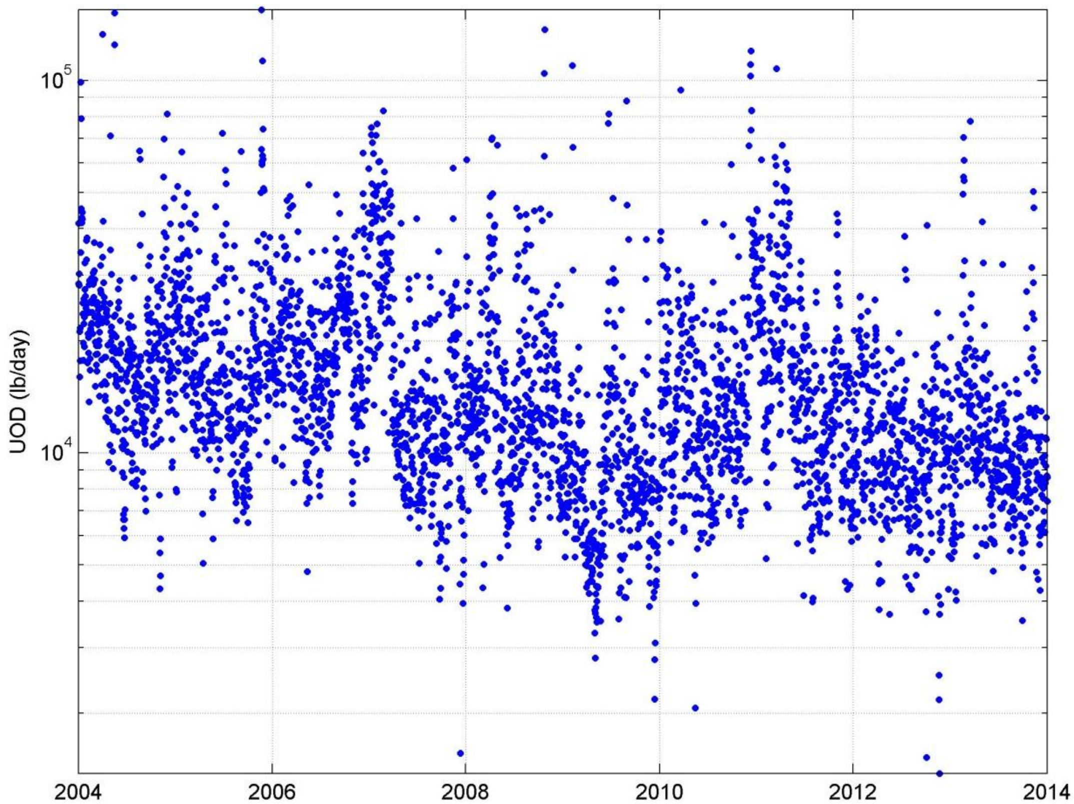


Figure 3.8.5.1.1. Example time series of daily UOD loads from the largest discharger (Kapstone)

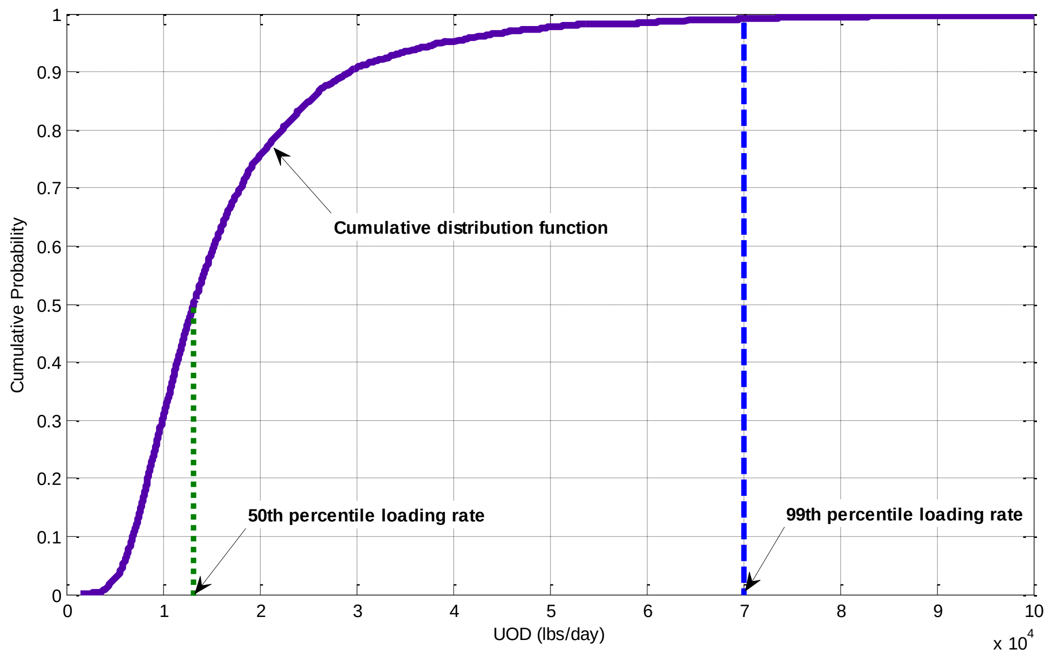


Figure 3.8.5.1.2. Example cumulative distribution function for daily UOD loads from the largest discharger (Kapstone)

The District coordinated with SCDHEC and EPA to use an alternate methodology for evaluating the cumulative effect of the Post 45 alternatives and the existing point-source pollution discharges. This methodology, presented below, incorporates an assessment of the variability of the point-source discharges in order to provide a more accurate evaluation than the methodology used to establish the existing TMDL for the harbor. MG Associates was contracted by the South Carolina Ports Authority to complete an analysis of the cumulative dissolved oxygen (DO) impacts to the Charleston Harbor estuary resulting from both the point-source pollution discharges into the estuary and the proposed Post 45 Project navigation channel expansion in support of the Environmental Impact Statement (EIS) for the project being completed by the US Army Corps of Engineers (USACE) Charleston District.

3.8.5.2 Variable Load Cumulative Impact Methodology

The study methodology includes the following steps:

(1) **Analysis of Historic Point Source Effluent Data** – Historic daily measured flow, 5-day biochemical oxygen demand (BOD5) and ammonia nitrogen data were obtained for each of the major point-source discharges in the estuary. These data were analyzed to characterize the discharge flow and pollutant load frequency distributions for each of these discharges.

(2) **Generation of Long-Term Daily Loads** – In order to evaluate a wide range of potential loading combinations in the estuary, the daily discharge flow and pollutant load frequency distributions were used to synthesize a 50-year time series of loading rates for each discharge. For each discharge, the loading rate time series was then multiplied by a scaling factor so that the 99^{th} percentile of the monthly-averaged ultimate oxygen demand (UOD) was equal to the monthly permit limit allocated in the TMDL. The resulting time series of loading rates incorporates daily variations consistent with the measured data while representing the maximum loading rate given by the TMDL wasteload allocation.

(3) **Evaluation of Point-Source and Cumulative Dissolved Oxygen Impacts** – The previously calibrated TMDL EFDC model was used to estimate the DO impacts from the point-source discharges. This was accomplished by modeling DO concentrations during 50 one-year periods. These 50 scenarios use the model inputs for the 2004 hydrologic conditions used for the TMDL model, but

the point source loads for each scenario were replaced by each of the 50-years of synthesized daily point-source loads (from Step 2 above). The results from the modeled DO impacts were then added to the Post 45 project DO impacts for the purposes of assessing the cumulative project impacts. The Post 45 impacts were based on the 52-48 Alternative, which represents the maximum deepening and widening alternative under consideration for the EIS.

In regard to step 3 above, it should be noted that a single representative hydrological year was used for the analysis. Long-term hydrological conditions (e.g., a 50-year record) were not modeled because the hydrological conditions and assimilative capacity in the Cooper River are largely controlled by freshwater releases from the Pinopolis Dam at the upstream end of the Cooper River. As explained by Cantrell (2013):

> *Assimilative capacity in the Charleston Harbor System is, on the whole, relatively stable with low variability from year to year. It is governed to a large degree by tightly controlled flow releases at Pinopolis Dam, which have both minimum and maximum flow constraints, and regular tidal cycle forcing from the ocean, although the use of actual water surface elevations can introduce irregular meteorological effects. Rainfall over the watershed varies annually, but rainfall patterns do not appreciably alter flow, dilution, or assimilative capacity conditions on the lower Cooper River. High flows diverted from the Santee River that were previously sent down the Cooper River and into Charleston Harbor were routed back to the Santee River beginning in the 1980s.*
>
> *The Ashley River, particularly the upper Ashley, is influenced by local rainfall patterns. Sustained periods of dry weather reduce inflows from Cypress Swamp and other sources allowing salt water to intrude up the Ashley River creating poor flushing conditions for continuous wastewater discharges. During wet conditions, salt water and effluent are pushed downstream and out of the system.*

DHEC used a 7-year (2000-2006) EFDC model simulation to evaluate year-to-year variability in hydrological conditions and the resulting effects on flushing and DO impacts from point-source discharges in the estuary (Cantrell 2013). The analysis found dilution in the Ashley River to be variable year to year, and 2004 is a year with relatively low flushing that is suitable as critical conditions for the TMDL evaluation. The analysis also found that the predicted DO impacts in the Cooper River showed a low degree of variability from year to year. Based on the low variability of the DO impacts in the Cooper River and considering 2004 to be an appropriate critical year for the Ashley River based on dilution results, the calibration year 2004 was selected by DHEC for critical conditions to develop the TMDL. This cumulative impact analysis uses the same representative hydrological year of 2004 based on the analysis of hydrological conditions completed by DHEC for the TMDL study.

It should also be noted that the implementation of time-variable point-source loads in the model was limited to discharges on the Cooper River and in the harbor. The two discharges on the Ashley River (the Town of Summerville wastewater treatment plant [WWTP] and the Dorchester County WWTP) were kept at the constant discharge rates used for the 2013 TMDL report, because these two discharges do not affect the DO in the critical segments of the Cooper and Wando Rivers. Furthermore, the Post 45 project alone will cause only very small DO impacts on the Ashley River (which are less than 0.01 mg/L in the critical segment of the river), and use of time-variable loading for these two discharges is not necessary in order to demonstrate that cumulative impacts on the Ashley River will not exceed the allowable 0.1 mg/L DO impact criterion.

3.8.5.3 Point Source Discharge Loading Rates

This section describes the daily point source monitoring data obtained for each point-source discharge. This section also describes the frequency distributions for each of these discharges and the synthesis of long-term 50-year loading rate time series.

Available daily discharge monitoring data for the past 10 years was requested from the major dischargers listed in Table 3.8.5.3.1 that are permitted under the National Pollutant Discharge Elimination System (NPDES). This table summarizes the major point source discharges included in the TMDL study completed previously by DHEC (Cantrell 2013), and it shows the relative contribution of each discharge to the 90th percentile decrease in DO in the critical segments of the Cooper and Wando Rivers calculated by the 2013 TMDL. The locations of these discharges are shown in Figure 3.8.5.3.1. As discussed in Section 3.8.5.2 and shown by Table 3.8.5.3.1, the discharges on the Ashley River (the Town of Summerville WWTP and the Dorchester County WWTP) do not affect the DO in the critical segments of the Cooper and Wando Rivers. The daily point-source loading rates from these two discharges were not evaluated in this study. Additionally, there is currently no discharge from CPW’s permitted Daniel Island outfall. Given the lack of recent data from this discharge, and given that it is only a small permitted loading rate, the CPW Daniel Island discharge was represented as a constant loading rate for this cumulative impacts analysis.

Table 3.8.5.3.1. Major NPDES Point Source Discharges in the TMDL Model

Discharge Name	NPDES Permit No.	UOD[1] (lb/day)	% of DO impact in critical Cooper River Segment[1]	% of DO impact in critical Wando River Segment[1]
Summerville	SC0037541	2,745	0%	0%
DCPW/Lower Dorchester	SC0038822	2,365	0%	0%
Moncks Corner	SC0021598	5,730	8%	4%
BCWSA/Central Berkeley	SC0039764	3,788	7%	3%
DAK Americas & Dupont	SC0026506 & SC0048950	2,466	3%	2%
Sun Chemical	SC0003441	7,625	12%	9%
BP Amoco	SC0028584	4,736	4%	4%
BCWSA/Lower Berkeley	SC0046060	8,846	9%	10%
KapStone	SC0001759	40,959	28%	25%
CPW/Daniel Island	SC0047074	403	0%	0%
NCSD/Felix Davis	SC0024783	29,090	22%	29%
CPW/Plum Island	SC0021229	24,612	4%	7%
Mount Pleasant - CS & RR	SC0040771	11,415	4%	7%

1. UOD loading rates and percent DO impact values are from the 2013 TMDL report (Cantrell 2013) based on constant discharges at maximum permitted rates.

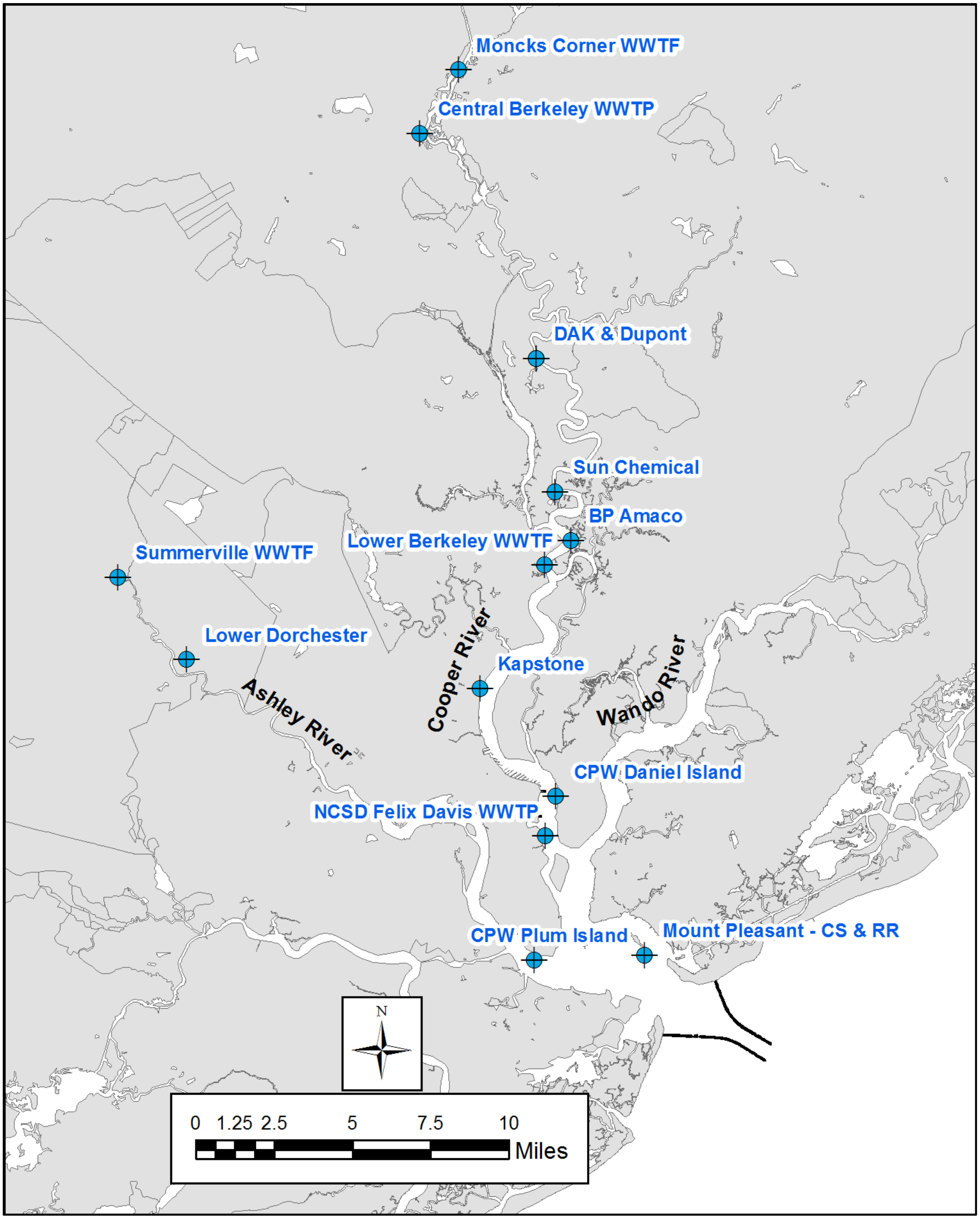


Figure 3.8.5.3.1 Location of Major NPDES Discharges included in TMDL Model

3.8.5.3.1 Measured Daily Discharge Data

Measured daily discharge data are plotted in Figures 3.8.5.3.2 through 3.8.5.3.12 for the 11 discharges analyzed for this evaluation. Each figure includes plots of flow, BOD_5 and ammonia nitrogen. All ammonia nitrogen is referred to as NH4 this report, because the EFDC model uses the NH4 variable to represent the grouped quantity of NH_3 (dissolved ammonia gas) plus NH_4^+ (ammonium ion). In all plots, the BOD values are from five-day BOD tests. For the subset of dischargers that are required by permit to monitor carbonaceous BOD (CBOD), that variable is used instead of BOD. Also, note that daily data are not available for all variables. Although daily flow rate data are available for all discharges, BOD and NH4 data were measured less frequently (between 1 and 5 days per week).

Some of the historic discharge data are no longer representative of discharge because of plant improvements or improvements in process controls. Where older data is not used in this evaluation to characterize future discharge loads, the data are plotted as red symbols (e.g., in Figures 3.8.5.3.2, 3.8.5.3.8, and 3.8.5.3.11). In 2008, improved aeration equipment was installed at the Moncks Corner WWTP which resulted in higher effluent DO and lower effluent BOD concentrations, and therefore data prior to June 1, 2008 are not used (Figure 3.8.5.3.2). A new Central Berkeley WWTP became operational in 2013, and only a short record of data is available for this discharge (Figure 3.8.5.3.3). The Lower Berkeley WWTP implemented better process controls for period after Sept 1, 2009, and the data prior to this date is not used (Figure 3.8.5.3.8). The North Charleston Sewer District (NCSD) Felix Davis WWTP completed construction of plant improvements in 2009, and therefore only data after June 1, 2009 are used (Figure3.8.5.3.11).

The mean and standard deviation of the measured discharge data used for this analysis are summarized in Table 3.8.5.3.2. This table also notes which discharges measured CBOD data.

Table 3.8.5.3.2. Statistics for measured discharge data

Discharge Name	Flow (MGD)		BOD_5 (mg/L)		Ammonia-N (mg/L)	
	Mean	**Std. Dev.**	**Mean**	**Std. Dev.**	**Mean**	**Std. Dev.**
Moncks Corner	1.02	0.35	16.09	6.09	15.72	6.81
BCWSA/Central Berkeley	0.21	0.06	6.62	2.83	6.35	6.51
DAK Americas[1]	0.95	0.14	8.51	4.02	0.56	0.85
Dupont	0.29	0.11	1.15	2.37	1.36	0.63
Sun Chemical[1]	2.21	0.46	4.32	5.50	2.21	5.23
BP Amoco	4.38	0.99	4.27	2.61	0.70	1.27
BCWSA/Lower Berkeley[1]	8.71	3.46	3.02	1.47	1.00	2.00
KapStone	25.24	2.14	9.84	7.21	0.65	0.99
NCSD/Felix Davis	16.15	4.68	4.08	3.27	0.73	2.65
CPW/Plum Island	22.50	5.14	6.53	5.42	2.49	3.01
Mount Pleasant - CS & RR[1]	6.80	0.76	4.36	2.17	2.59	3.10

1. The BOD_5 value for these discharges represents $CBOD_5$ concentration.

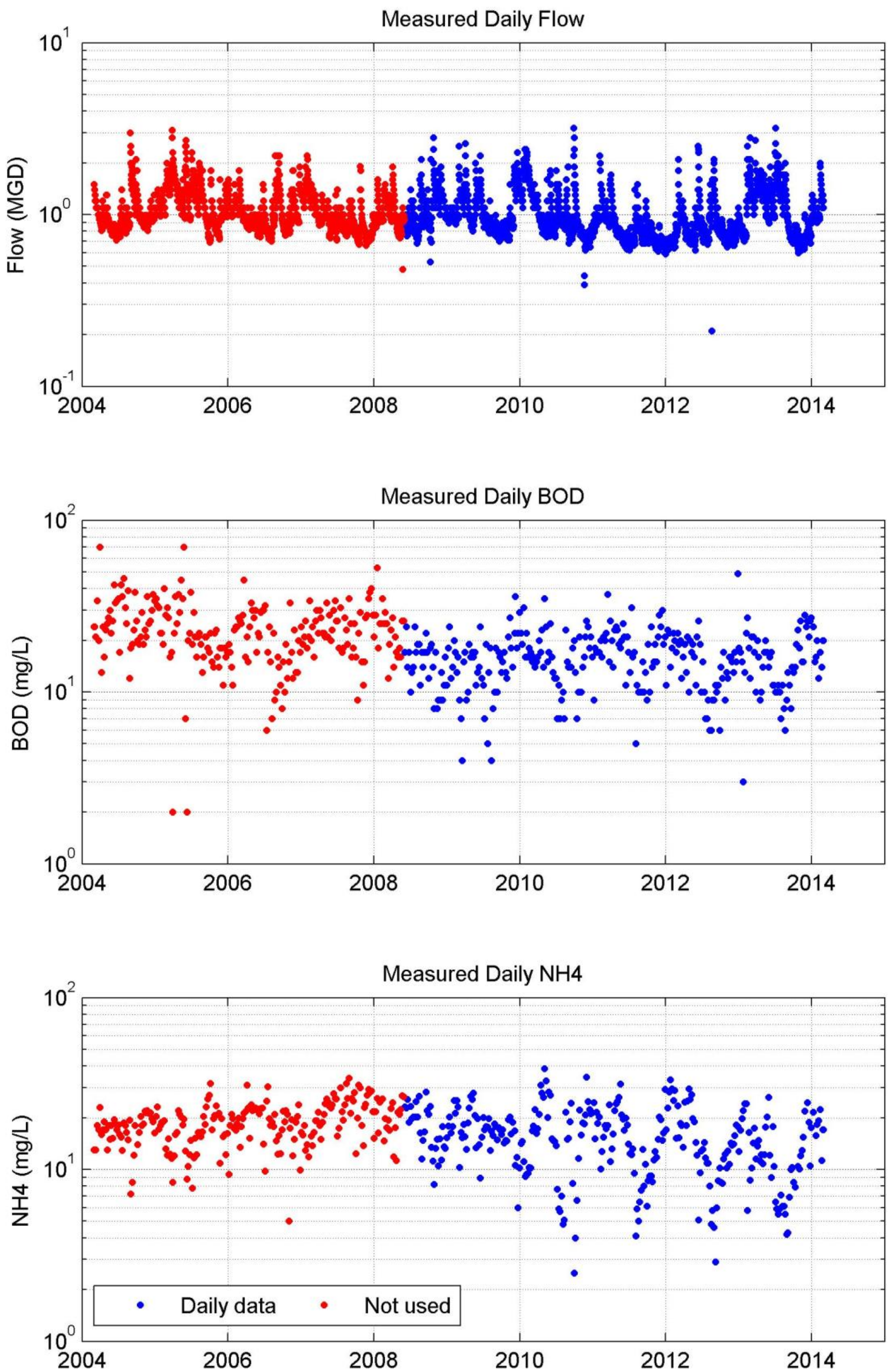


Figure 3.8.5.3.2. Moncks Corner measured discharge data

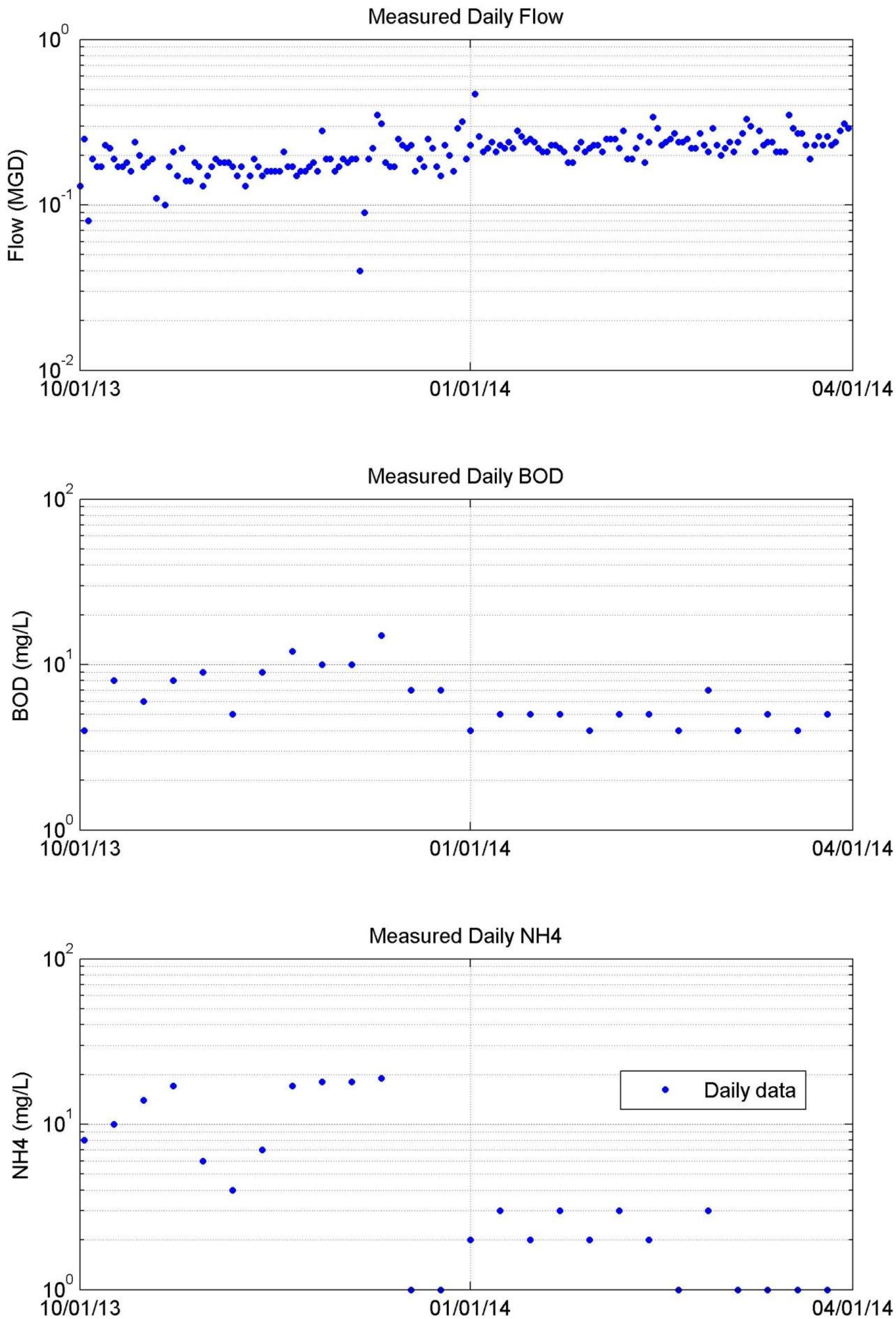


Figure 3.8.5.3.3 Central Berkeley measured discharge data

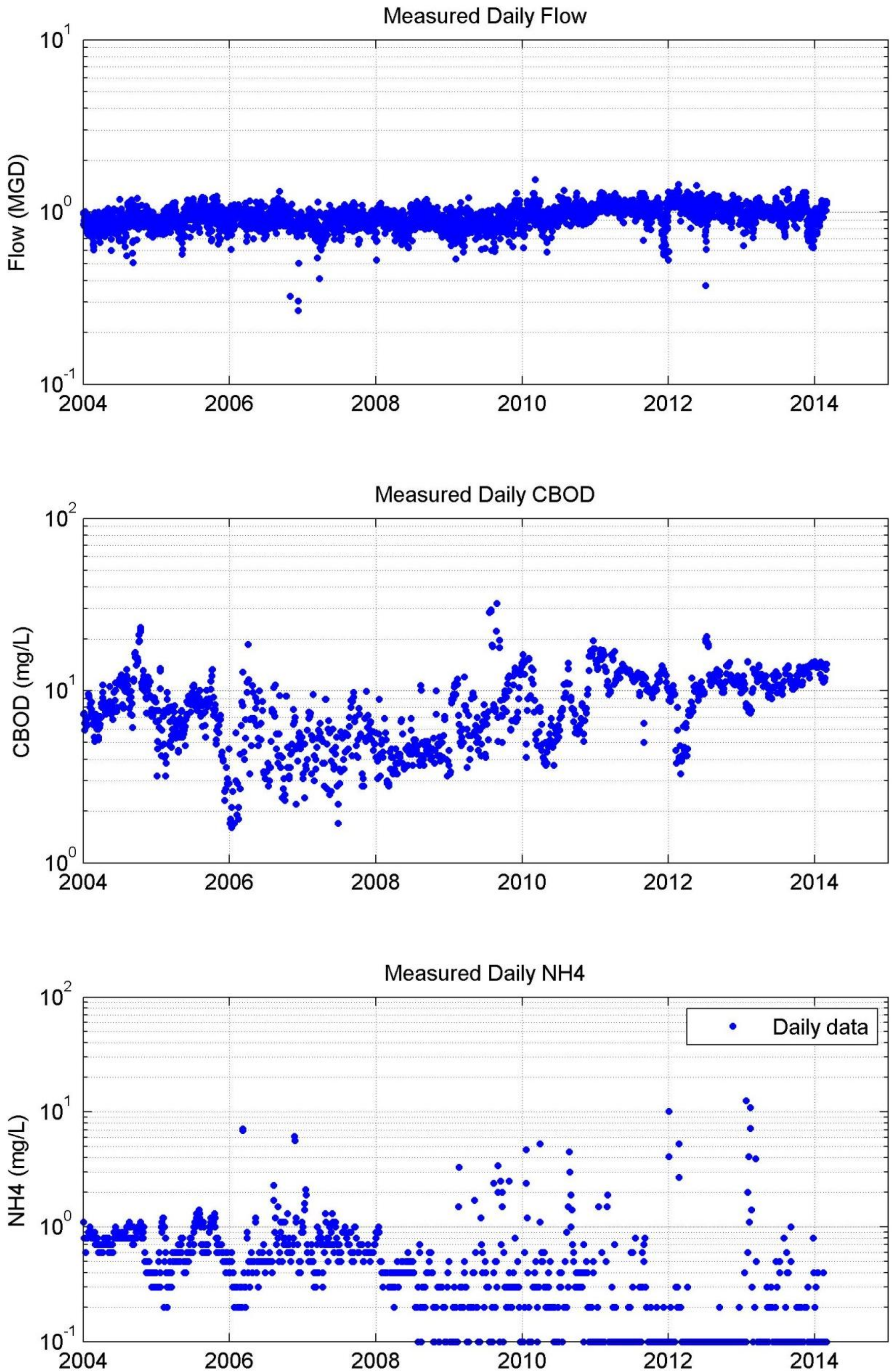


Figure 3.8.5.3.4 DAK measured discharge data

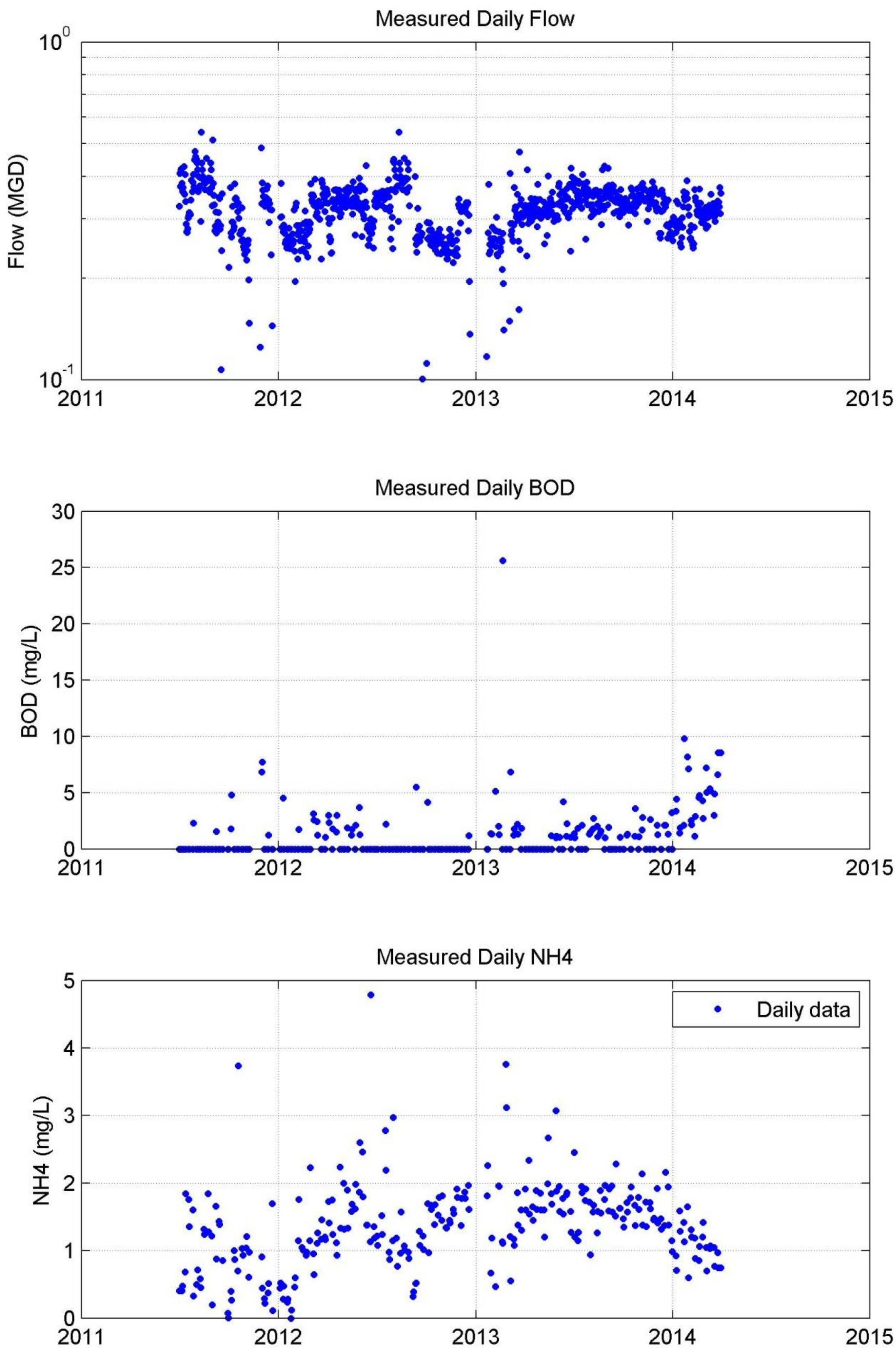


Figure 3.8.5.3.5 Dupont measured discharge data

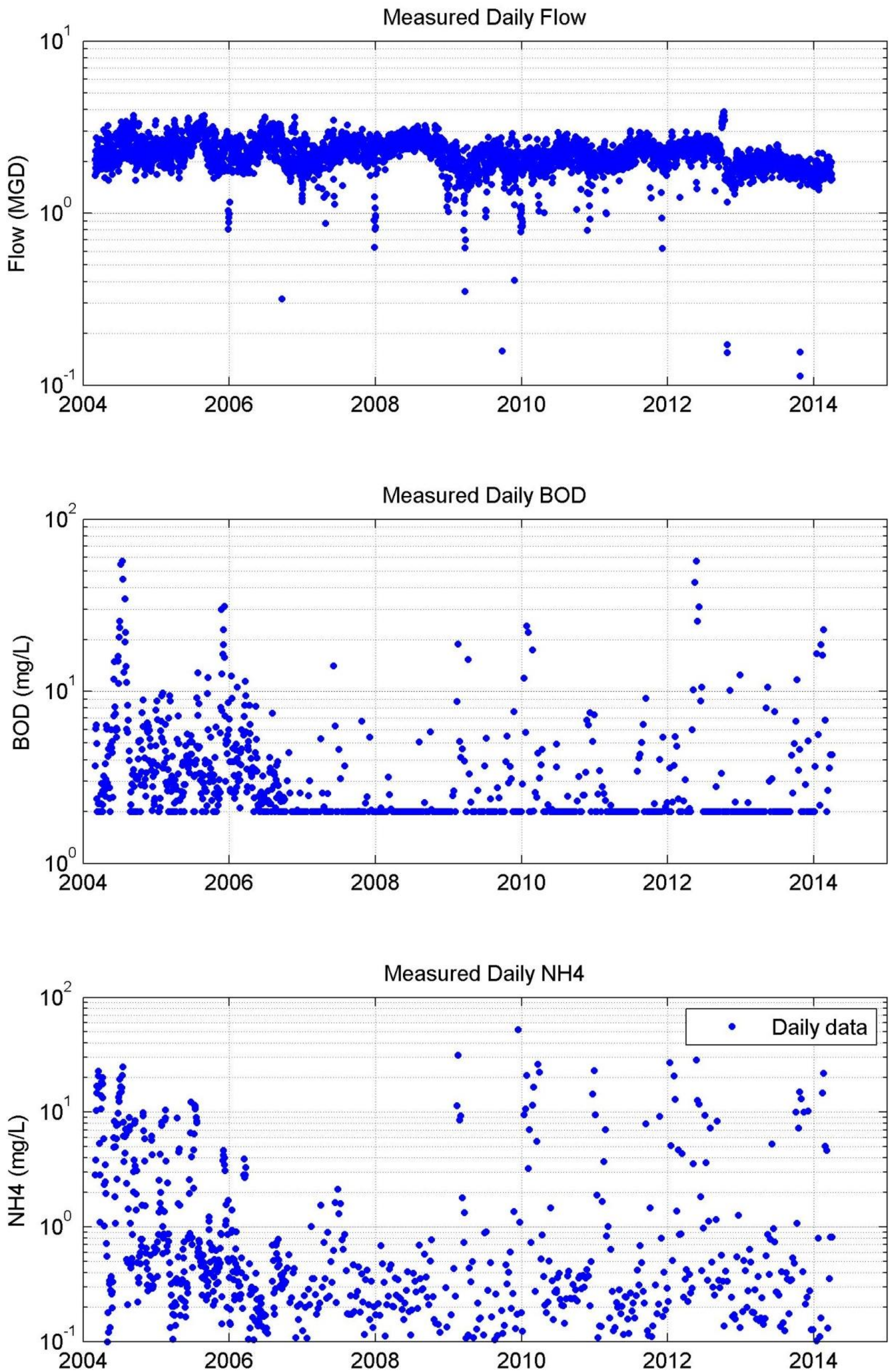


Figure 3.8.5.3.6 Sun Chemical Measured Discharge Data

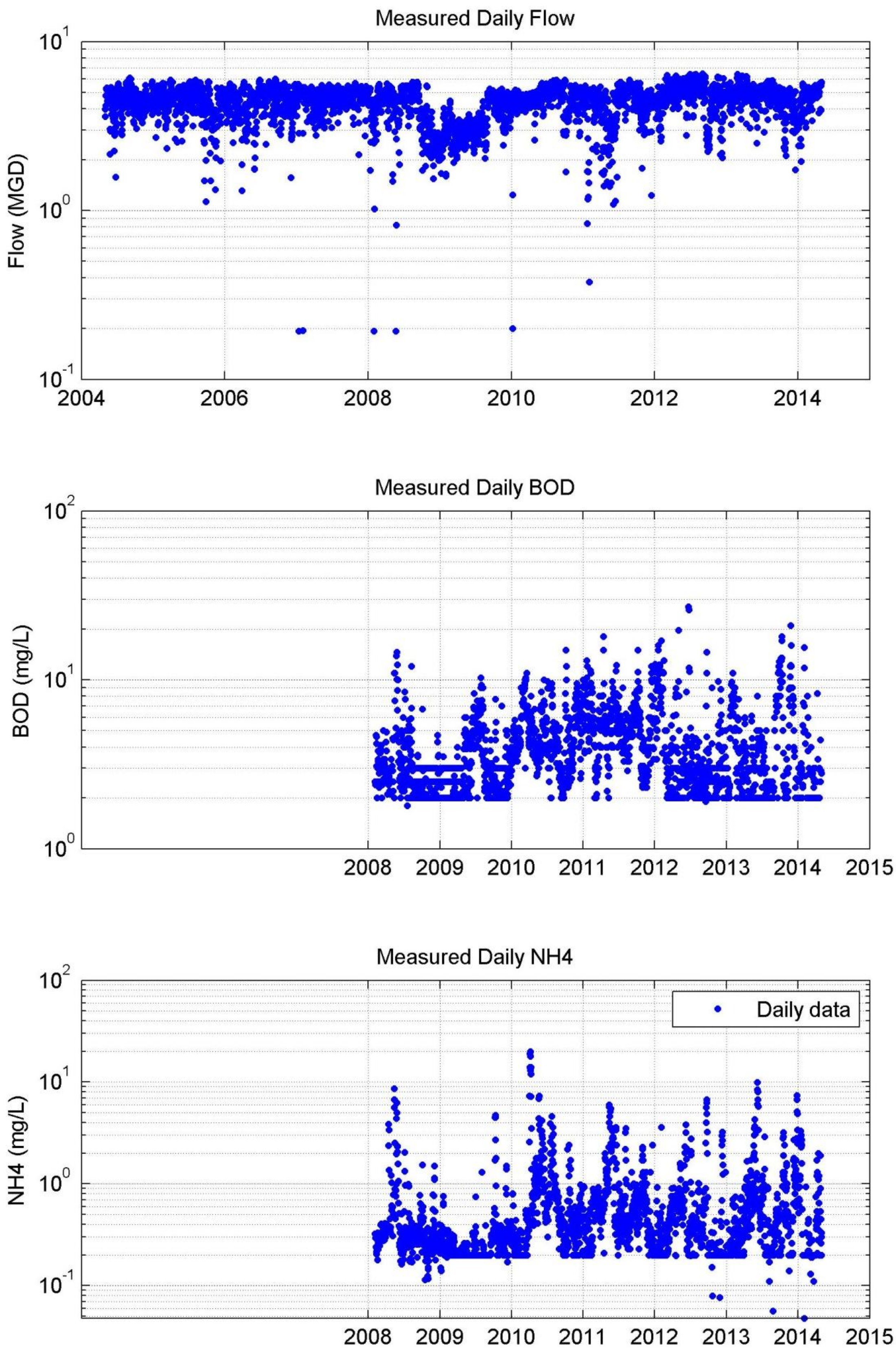


Figure 3.8.5.3.7 BP Amoco measured discharge data

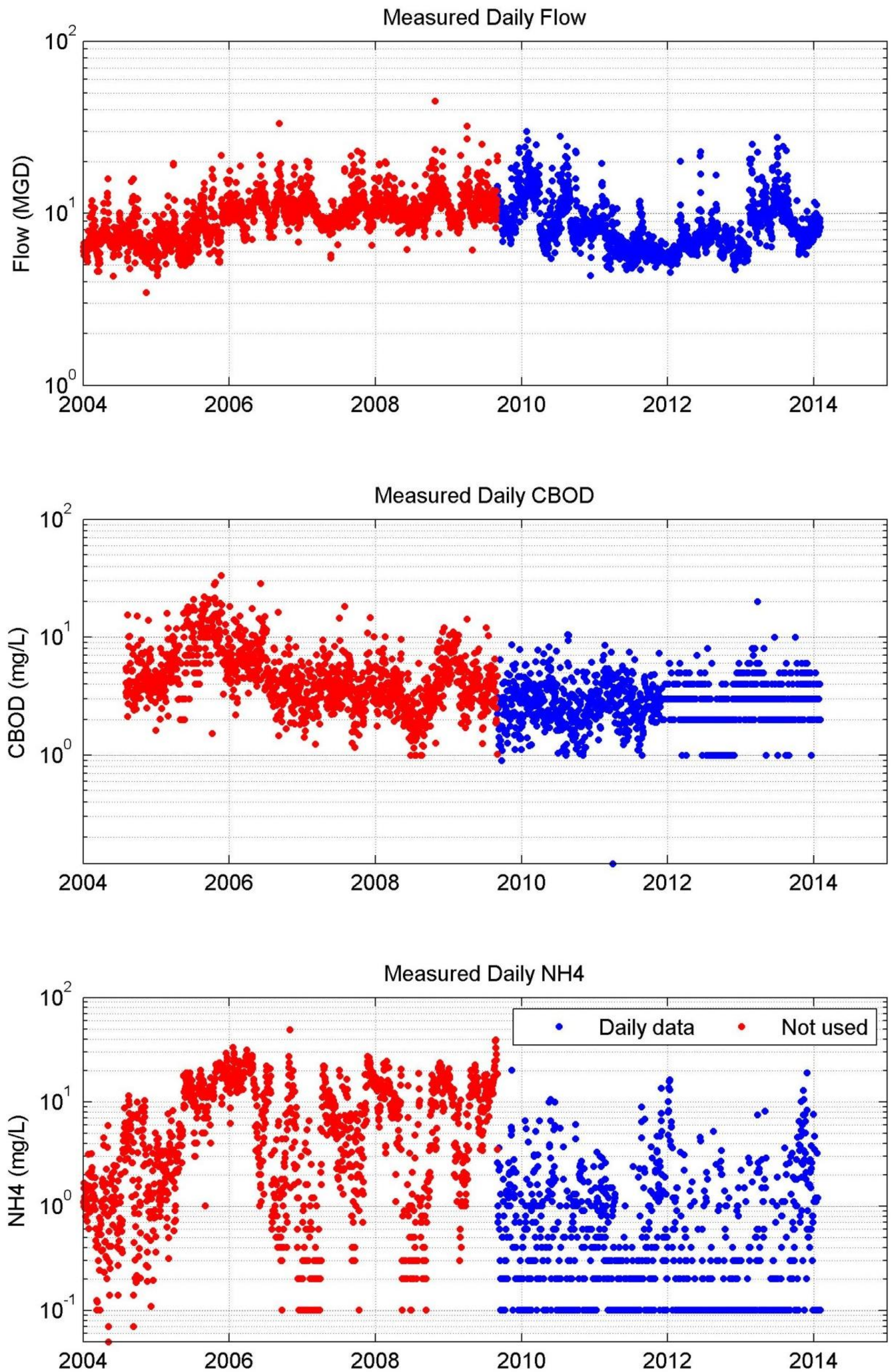


Figure 3.8.5.3.8 Lower Berkeley measured discharge data

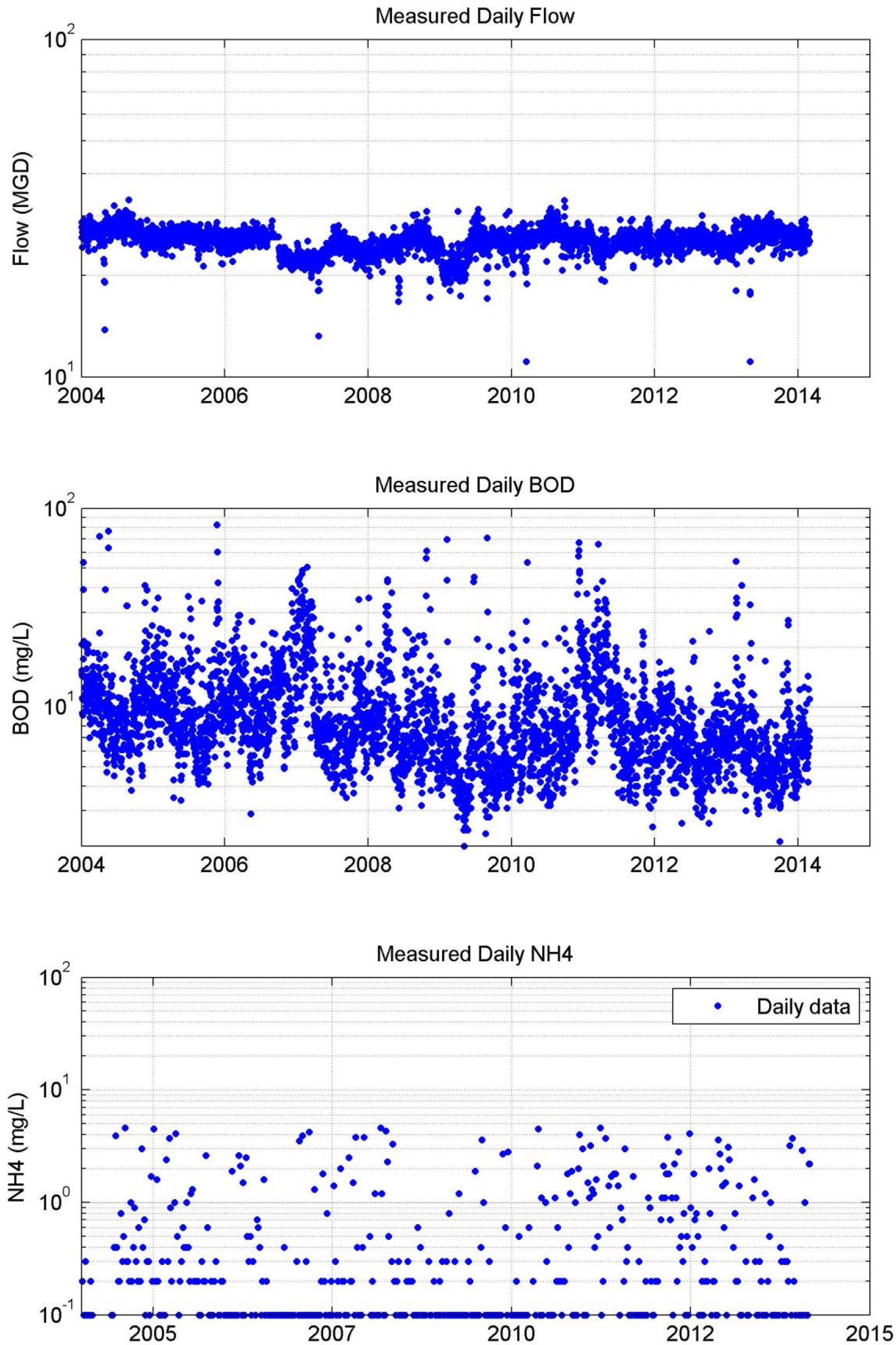


Figure 3.8.5.3.9 Kapstone measured discharge data

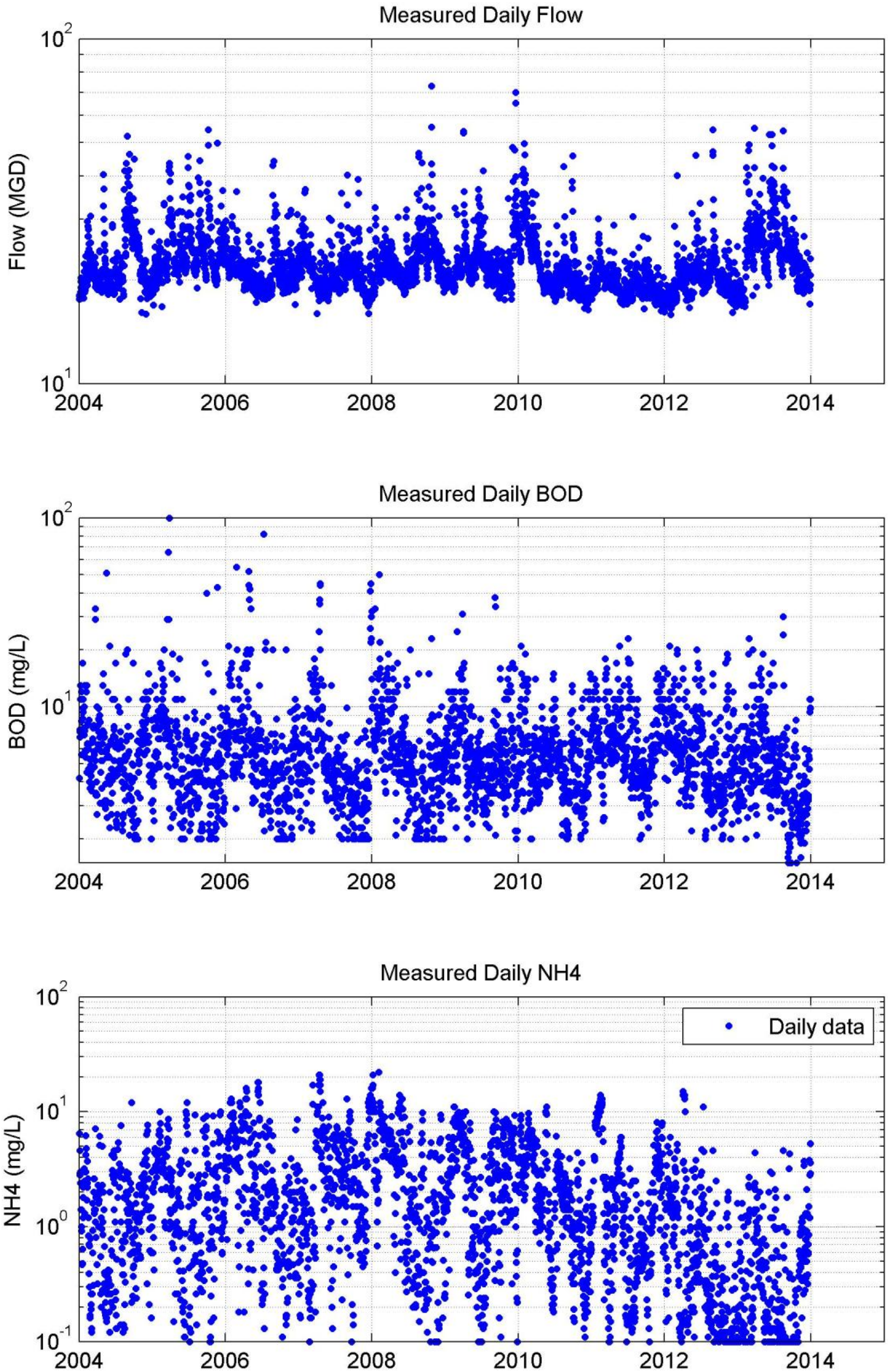


Figure 3.8.5.3.10 CPW Plum Island measured discharge data

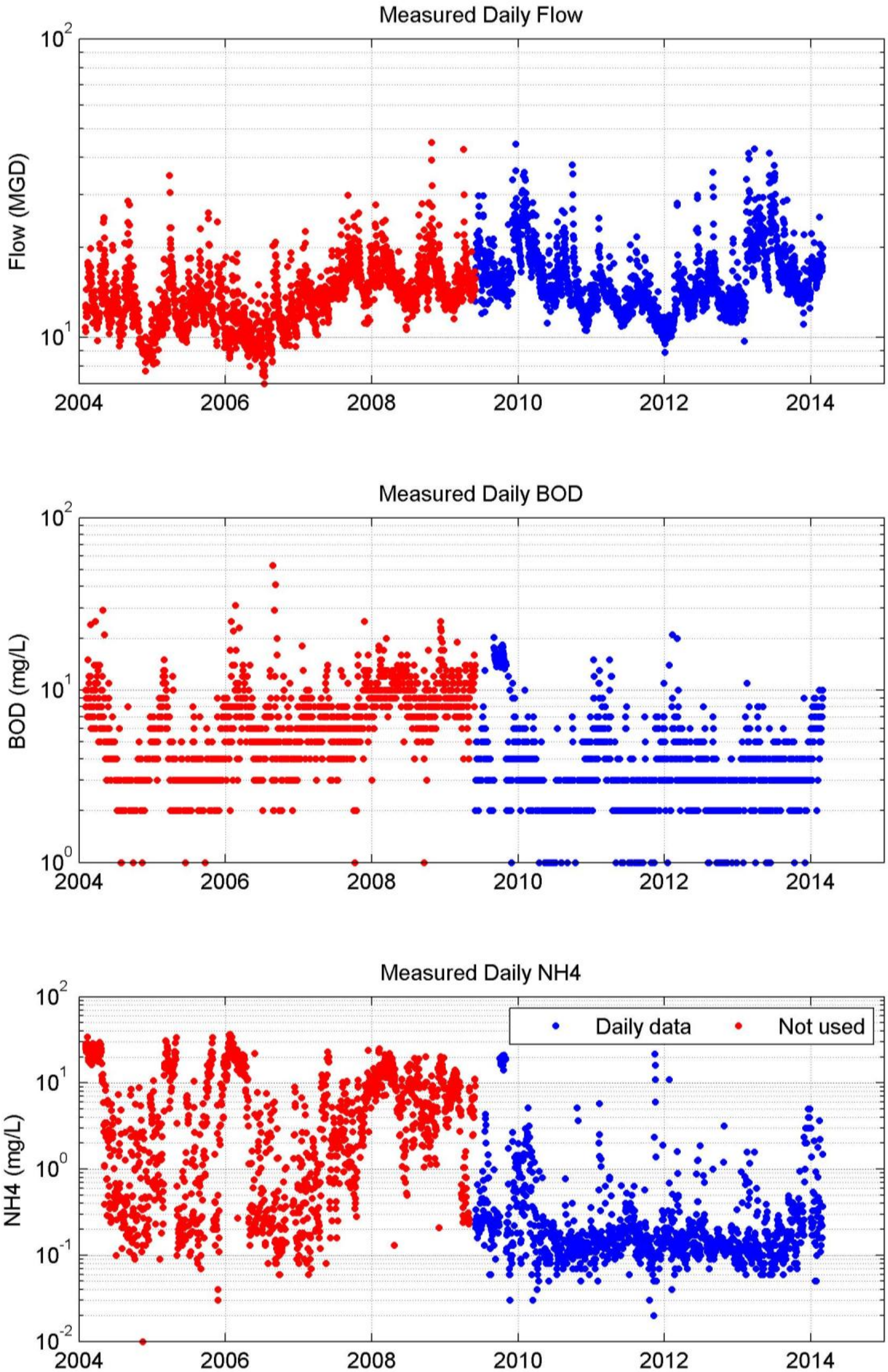


Figure 3.8.5.3.11 NCSD measured discharge data

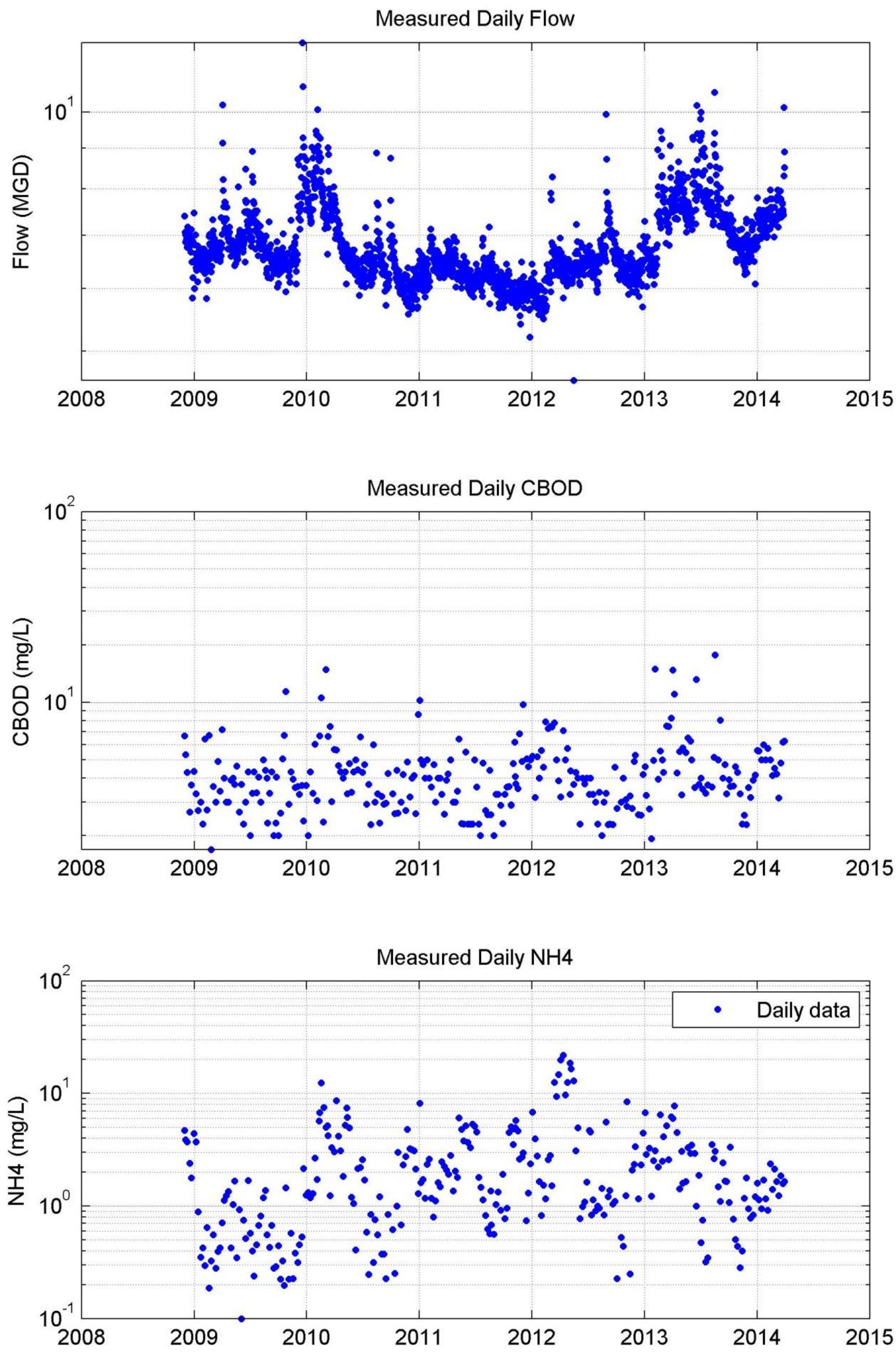


Figure 3.8.5.3.12 Mount Pleasant CS & RR measured discharge data

3.8.5.3.2 Synthesized 50-year Daily Loading Rates

Randomized long-term 50-year time-series of daily discharge loads were created based on the measured daily discharge data. This was accomplished using the following steps:

1. The BOD and NH4 data were interpolated to create daily time series. As mentioned previously, although daily flow rate data are available for all discharges, BOD and NH4 data were measured less frequently (between 1 and 5 days per week).

2. The ultimate nitrogenous and carbonaceous BOD (NBODu and CBODu) concentrations were calculated, because the EFDC model input organic carbon loads were derived from CBODu, and both CBODu and NBODu were required to evaluate the discharge UOD. The UOD was calculated in order to compare the measured and synthesized frequency distributions of UOD loading, and the UOD was also used to determine the scaling factor to set each discharge at a level consistent with the loading rate allocated by the TMDL (discussed further in step 6 below). These concentrations were calculated using the equations below:

$$NBODu = NH4 \times 4.57$$

$$CBODu = BOD\ (or\ CBOD) \times f\ ratio$$

$$UOD = CBODu + NBODu$$

   where the f-ratio is the ratio of the 5-day BOD value to the ultimate BOD value based on long-term BOD laboratory tests completed for the TMDL study (described by Tetra Tech and Jordan, Jones, and Goulding [2008]). The f-ratios used are listed in Table 3.8.5.3.3. The f-ratios are the same as those used for the TMDL, with the exception of Central Berkeley, which is assumed to have the same f-ratio as Lower Berkeley following the new plant construction (the old Central Berkeley discharge had an f-ratio of 1.5 in the TMDL). CBOD concentrations were used for calculation of CBODu when available. Most discharges measured only the BOD concentrations, in which case the BOD concentration was used to calculate the CBODu. The use of the BOD concentrations for calculation of CBODu results in a conservative analysis (that is, it will over-estimate the impact of the discharge on DO), because the NBOD load is double-counted (nitrogenous oxygen demand is included in the BOD measurement as well as the ammonia nitrogen measurement).

3. The natural logarithms of the flow, NBODu concentration and CBODu concentration were calculated, and probability distributions were fit to these data

for each of the 11 discharges. A normal distribution fit was initially attempted for each of the data sets (a normal distribution fit to the log of the data provides a log-normal distribution fit). However, most often, a better distribution fit for each data set was found by testing different probability distribution types and selecting one with the best fit. Distribution types fit to the log data include: normal, Burr, t location-scale, generalized extreme value, logistic and kernal distributions.

Table 3.8.6.3.3 . Discharge f-ratios, labile and refractory organic carbon fractions, and DO concentrations

Discharge Name	f-ratio	Refractory fraction	Labile fraction	DO conc. (mg/L)
Moncks Corner	4.11	0.57	0.43	5
BCWSA/Central Berkeley	3.8	0.5	0.5	5
DAK Americas	6.99	0.8	0.2	4
Dupont	6.99	0.8	0.2	4
Sun Chemical	4.03	0.55	0.45	4
BP Amoco	6.5	0.78	0.22	4
BCWSA/Lower Berkeley	3.8	0.5	0.5	5
KapStone	8.7	0.87	0.13	4
NCSD/Felix Davis	4.43	0.61	0.39	5
CPW/Plum Island	3.18	0.39	0.61	5
Mount Pleasant - CS & RR	3.18	0.39	0.61	1

50-year time-series of flow, NBODu concentration, and CBODu concentration were created using random numbers generated from the probability distribution for each discharge variable. The generated time series also include a 1-day auto-correlation based on coefficients calculated from the measured data. This is necessary to ensure that the generated daily loading values are appropriately correlated to the previous day’s value and that the tendency of the discharge to remain at the same magnitude from day-to-day is properly shown by the generated loads. Measured data from 2004-2014 were used where data was available.

4. The 50-year time series of NBODu and CBODu loading rates for each discharge were calculated, and these loading rates were then converted to EFDC model input variables. The model input variables include refractory particulate organic carbon (RPOC), dissolved organic carbon (DOC), and NH4. The DOC variable

is used by the model to represent the labile fraction of the organic carbon. In addition, the ultimate oxygen demand (UOD) was calculated in order to compare the measured and synthesized frequency distributions of UOD loading. These variables were calculated as follows:

$$RPOC = \frac{CBODu * f_{\ refractory}}{2.67}$$

$$DOC = \frac{CBODu * f_{\ labile}}{2.67}$$

$$NH4 = \frac{NBODu}{4.57}$$

$$UOD = CBODu + NBODu$$

where $f_{refractory}$ and f_{labile} are the refractory and labile fractions of the organic carbon for each discharge based on long-term BOD laboratory tests completed for the TMDL study (described by Tetra Tech and Jordan, Jones, and Goulding. [2008]). The refractory and labile fractions used are listed in Table 3.8.5.3.3.

5. The synthesized loading rates were multiplied by a scaling factor so that the UOD from the synthesized loading rates is equal to the wasteload allocation UOD for each discharge in the TMDL. For this analysis, the 99th percentile monthly-averaged UOD is used as a value that is equivalent to the monthly-averaged WLA loads given by the TMDL. The 99th percentile monthly-averaged UOD is a value with a probability of exceedance of 1-in-100 each month, and UOD is expected to exceed this value once every 100 months (8.3 years), on average. The scaling factors were calculated as the ratio of the TMDL UOD to the 99th percentile of the monthly-averaged UOD for synthesized time series. The calculated scaling factors for the time-varying discharges are listed in Table 3.8.5.3.4.

6. The daily DO loads associated with each discharge were calculated based on the minimum permitted DO concentration for each discharge (listed in Table 3.8.5.3.3) times the daily flow rate. The daily flow rates used for this DO calculation were adjusted by a scaling factor such that the 99th percentile monthly-averaged flow rates for the time series are equal to the monthly-averaged flows used in the TMDL study. However, a minimum scaling factor of one was used, so that flow rates are not less than that indicated by the measured data (this affected only discharges without permit limits for flow rate). The scaling factors used for the flow and DO load calculations are listed in Table 3.8.5.3.5.

Table 3.8.5.3.4. Discharge 99th percentile time-varying UOD, TMDL UOD, and scaling factor

Discharge	99th Percentile Monthly-Ave. UOD Load (lb/day)	TMDL UOD (lb/d)	Scaling Factor (Ratio of TMDL to 99th Monthly-Ave. UOD)
Moncks Corner	2,209	5,730	2.59
BCWSA/Central Berkeley	55	3,788	68.81
DAK Americas	1,274	1,653	1.30
Dupont	431	814	1.89
Sun Chemical	2,100	7,625	3.63
BP Amoco	2,050	4,736	2.31
BCWSA/Lower Berkeley	2,357	8,846	3.75
Kapstone	35,917	40,959	1.14
NCSD/Felix Davis	9,135	29,090	3.18
CPW/Plum Island	14,742	24,612	1.67
Mount Pleasant- CS & RR	4,045	11,415	2.82
TOTAL	74,315	139,267	1.87

Because there was insufficient data to characterize the BOD and NH4 concentrations for the Central Berkeley WWTP, and the available measured data reflect a period of transition from the old lagoon plant to the new treatment plant, it was assumed that the distributions for these variables in the effluent from the new plant will be similar to those for the Lower Berkeley WWTP over the long-term. Therefore, the CBODu and NBODu concentration distribution curves from the Lower Berkeley WWTP were used to generate the long-term records for the Central Berkeley WWTP.

Table 3.8.5.3.5 Discharge 99th percentile time-varying flow, TMDL flow, and scaling factor

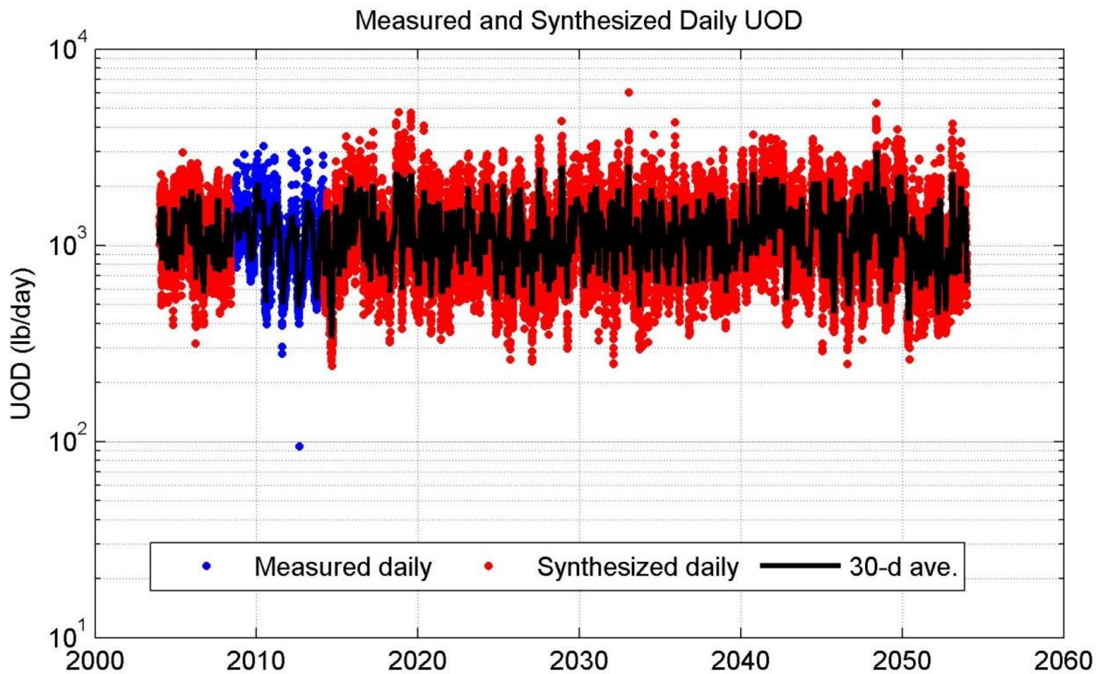
Discharge	99th Percentile Monthly-Ave. Flow (MGD)	TMDL Flow (MGD)	Flow Scaling Factor
Moncks Corner	1.68	3.20	1.91
BCWSA/Central Berkeley	0.30	6.00	20.36
DAK Americas & Dupont	2.73	1.32	1.00
Sun Chemical	2.82	4.50	1.60
BP Amoco	5.14	4.19	1.00
BCWSA/Lower Berkeley	14.8	22.50	1.52
Kapstone	27.8	25.60	1.00
NCSD/Felix Davis	25.8	34.00	1.32
CPW/Plum Island	29.4	54.00	1.83
Mount Pleasant- CS & RR	8.02	9.70	1.21
TOTAL	118.53	165.01	1.39

Statistics describing measured and synthesized point-source discharge UOD loads are summarized in Table 3.8.5.3.6 (note that these values do not include the scaling factors to adjust to the loads to the TMDL allocations). The 90th percentile UOD loading rates are of particular interest because they represent the upper range of discharge loading rates, and DHEC's implementation of the anti-degradation rule is focused on the 90th percentile decrease in DO caused by the discharges (the "delta DO"). Overall, the mean total synthesized UOD loading rate is only about one percent higher than measured loading rate, and 90th percentile synthesized UOD loading rate is only about two percent higher than the modified measured loading rate. Therefore, overall, the mean and 90th percentile loading rates to the system are well-represented by the synthesized long-term daily loading rate time series.

Figures 3.8.5.3.13 through 3.8.5.3.23 show the 50-year time series of UOD load rates for each discharger. These are the loading rates prior to being multiplied by the scaling factor. The blue data points indicate the measured daily discharge data used, and the red data points show the synthesized daily time series data. Black markers show the 30-day average of the daily loading rate data. These figures also include comparisons of the measured and synthesized UOD cumulative distribution functions (CDFs). In general, the synthesized CDFs are in good agreement with the measured CDFs. One exception is the lower half of the distribution for Dupont (Figure 3.8.5.3.16), where the synthesized data over-estimate the loading rates. This is caused by the high percentage of concentration data that were reported at the laboratory method detection limit, which results in a frequency distribution that is not smooth. The synthesized data instead are characterized by a smooth distribution function. This difference between the observed and synthesized distributions for the Dupont discharge is acceptable because it is conservative (i.e., it over-estimates the effect on DO) and the discharge is small enough that this difference does not have a significant effect on the calculated DO in the critical segment of the river (the Dupont discharge represents a small fraction [1 percent] of the permitted point-source UOD discharged into the harbor). It should also be noted that the Central Berkeley WWTP discharge is not compared to a measured CDF because the Lower Berkeley WWTP were used to characterize the BOD and NH4 concentrations for this discharge. The Central Berkeley WWTP also represents a small fraction (3 percent) of the permitted point-source UOD loading into the harbor, and therefore any uncertainty associated with this assumption does not have a significant effect on the calculated DO in the critical segment of the river.

Table3.8.5.3.6. Measured and synthesized discharge UOD load statistics

		UOD (lb/day)		
Discharge Name	**Data set**	**Mean**	**90th per.**	**1-day auto-corr.**
Moncks Corner	Measured	1,136	1,634	0.90
	Synthesized	1,158	1,878	0.91
BCWSA/Central Berkeley	Measured	-	-	-
	Synthesized	28	47	0.55
DAK Americas	Measured	495	818	0.95
	Synthesized	498	845	0.95
Dupont	Measured	35	72	0.84
	Synthesized	59	141	0.76
Sun Chemical	Measured	476	954	0.97
	Synthesized	422	760	0.91
BP Amoco	Measured	1,077	1,958	0.81
	Synthesized	1,076	1,882	0.75
BCWSA/Lower Berkeley	Measured	1,203	2,116	0.62
	Synthesized	1,160	2,038	0.58
KapStone	Measured	16,708	29,398	0.65
	Synthesized	16,740	30,440	0.67
NCSD/Felix Davis	Measured	2,844	5,539	0.87
	Synthesized	3,085	5,685	0.71
CPW/Plum Island	Measured	5,881	10,657	0.61
	Synthesized	5,971	11,018	0.76
Mount Pleasant - CS & RR	Measured	1,475	2,587	0.94
	Synthesized	1,445	2,415	0.92
Total	Measured	31,331	55,732	-
	Synthesized	31,614	57,102	-



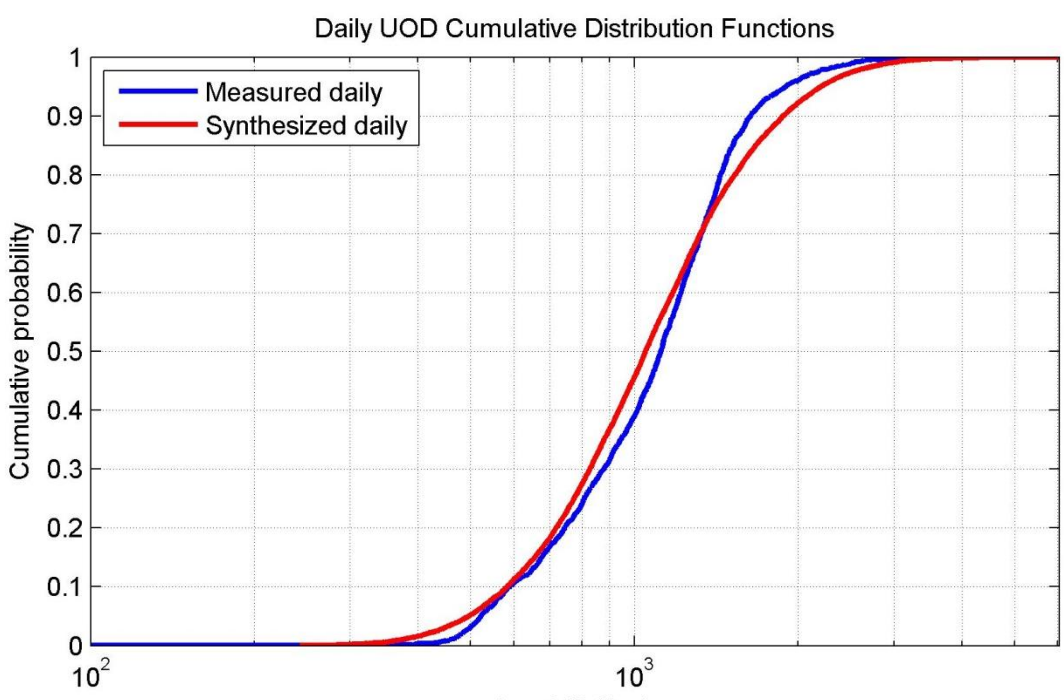
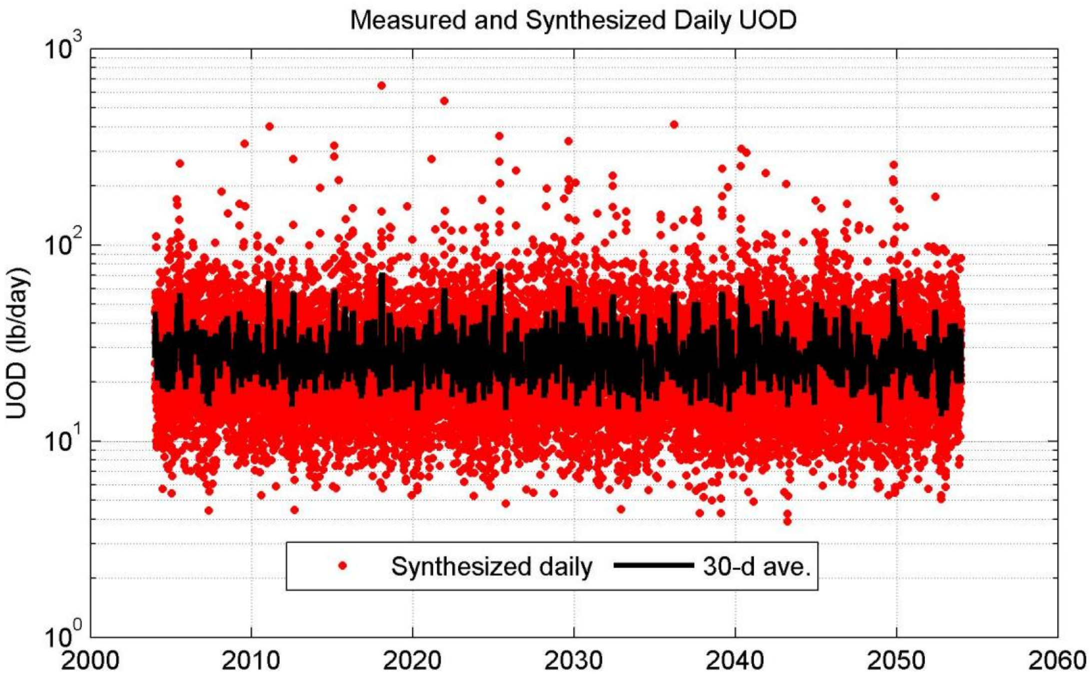


Figure 3.8.5.3.13. Moncks Corner 50-year UOD discharge time series and CDFs



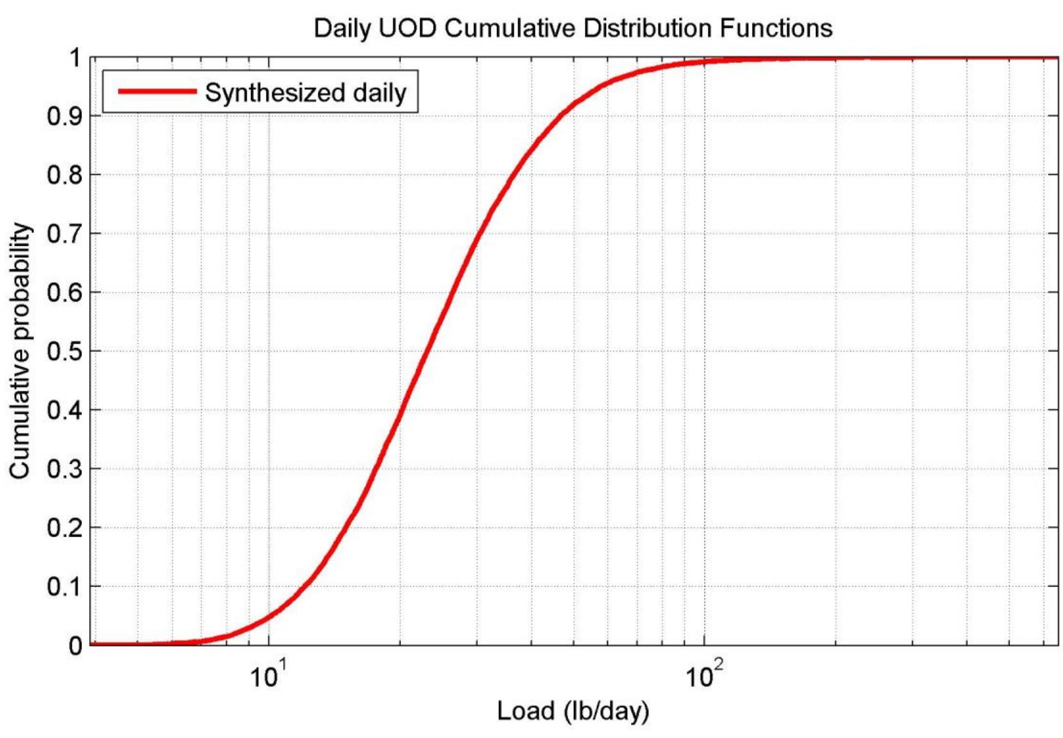
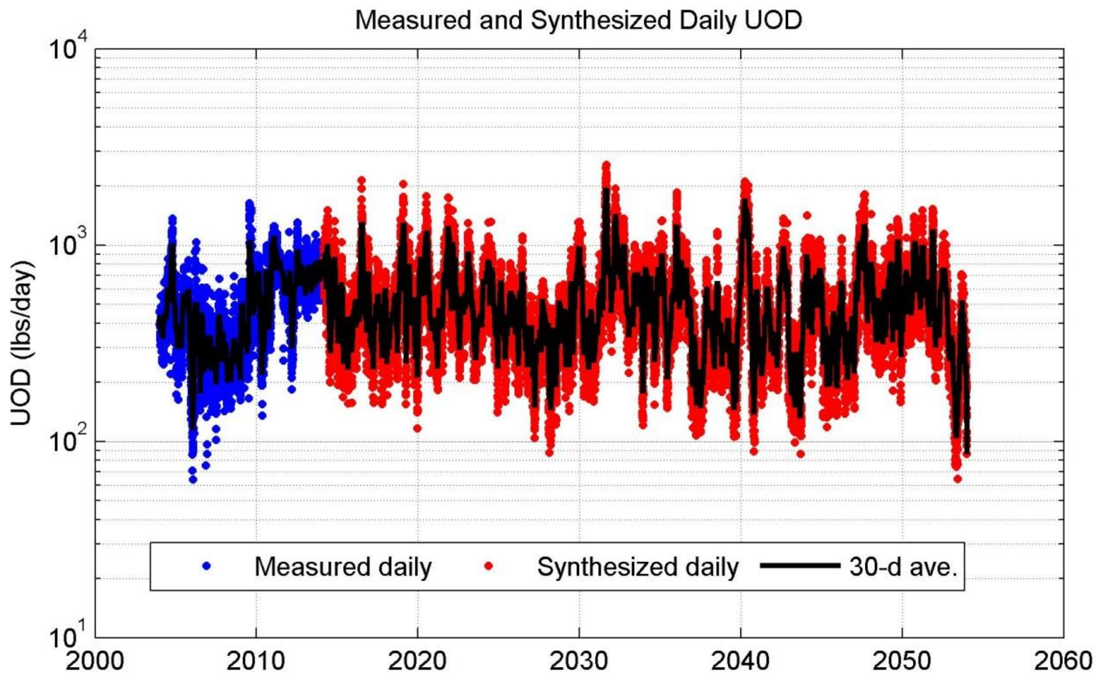


Figure 3.8.5.3.14. Central Berkeley 50-year UOD discharge time series and CDFs



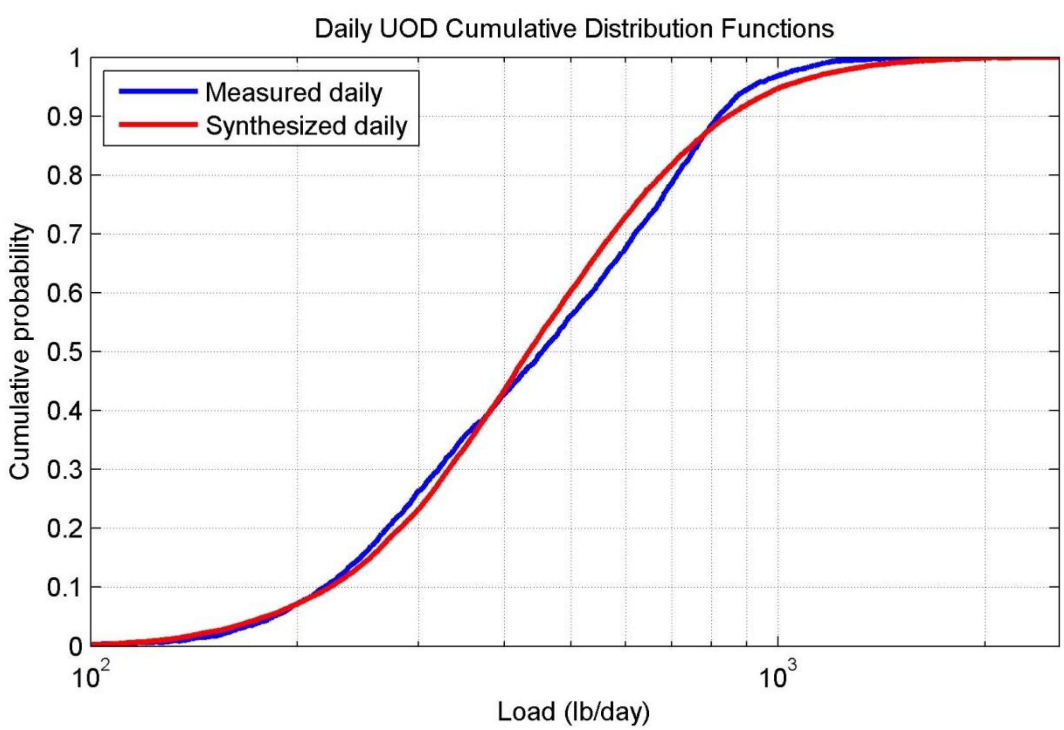
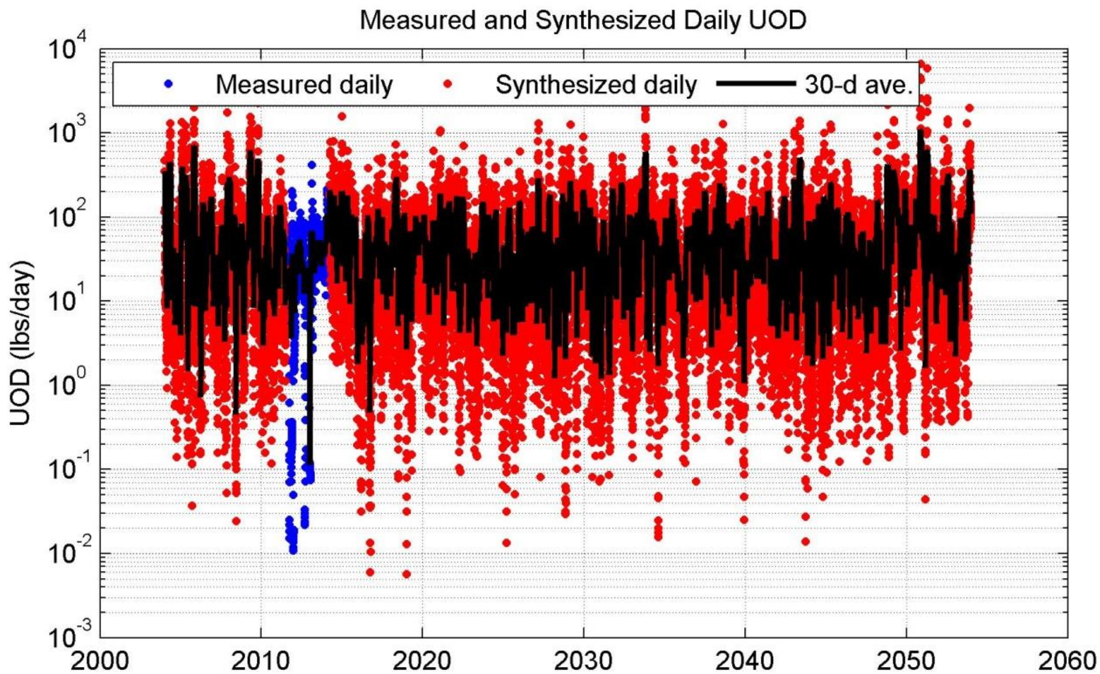


Figure 3.8.5.3.15. DAK 50-year UOD discharge time series and CDFs



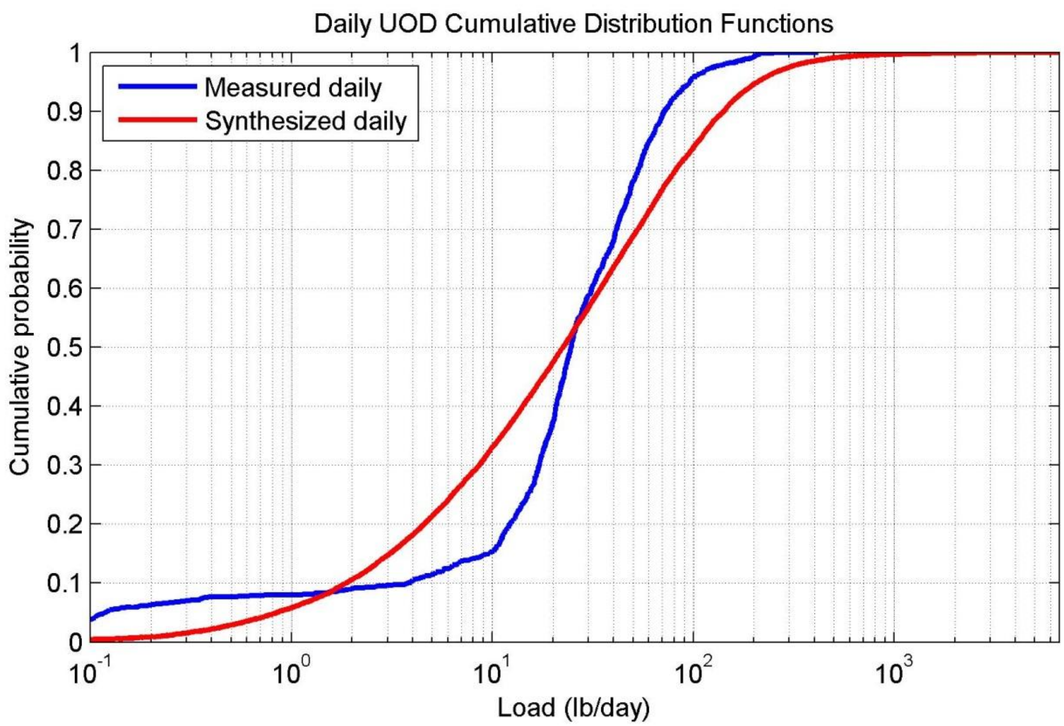
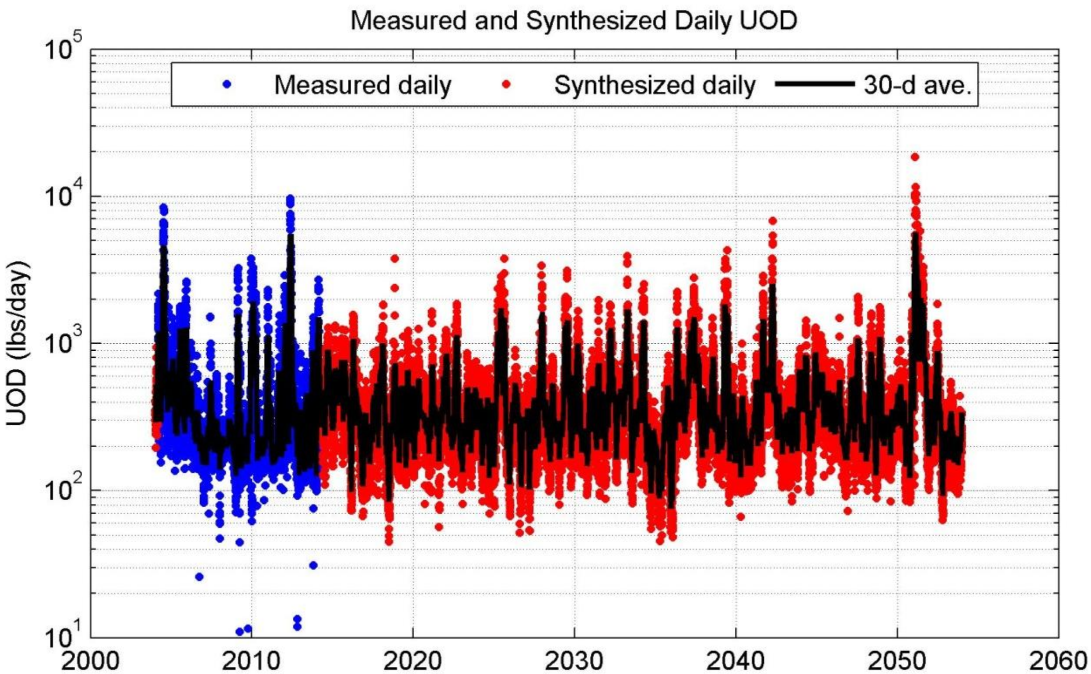


Figure 3.8.5.3.16. Dupont 50-year UOD discharge time series and CDFs



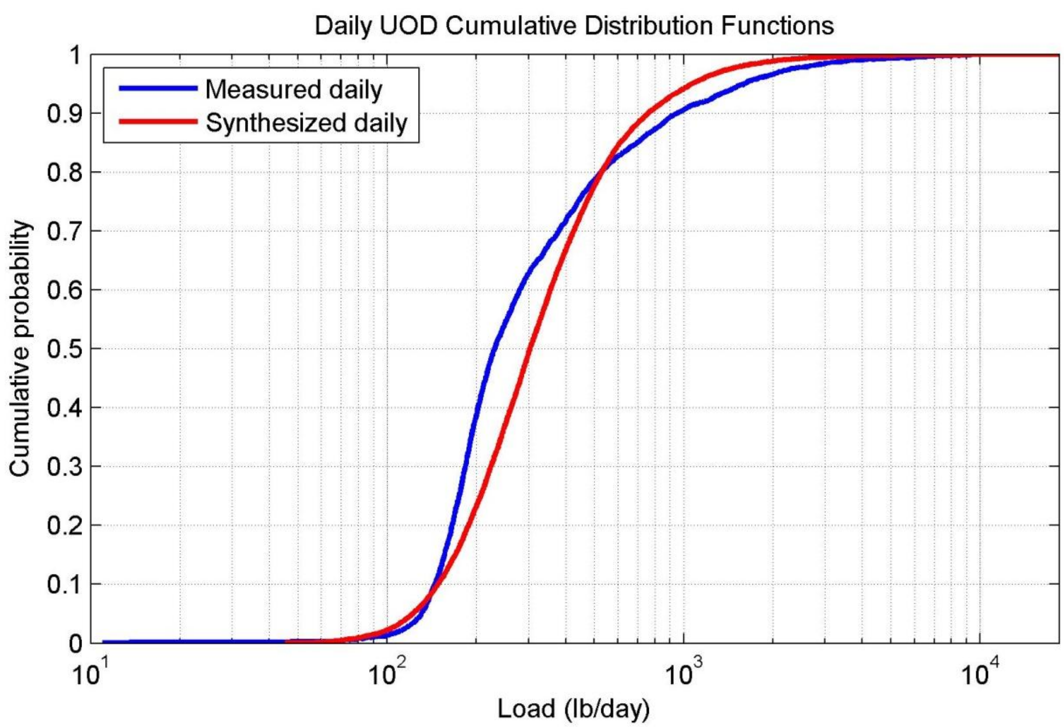
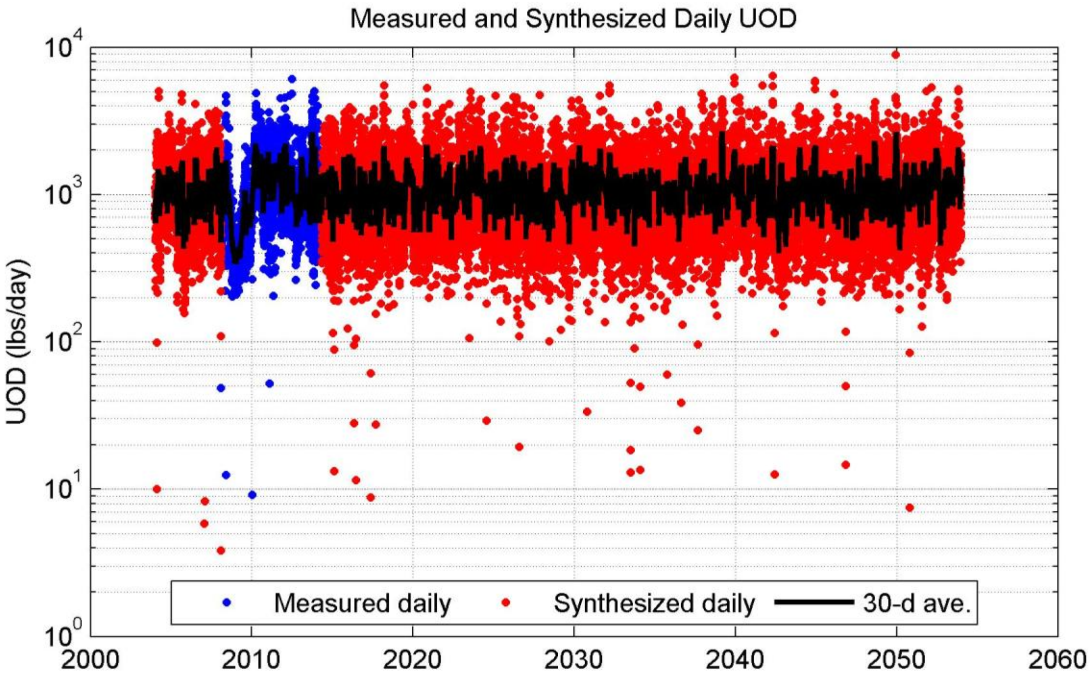


Figure 3.8.5.3.17. Sun Chemical 50-year UOD discharge time series and CDFs



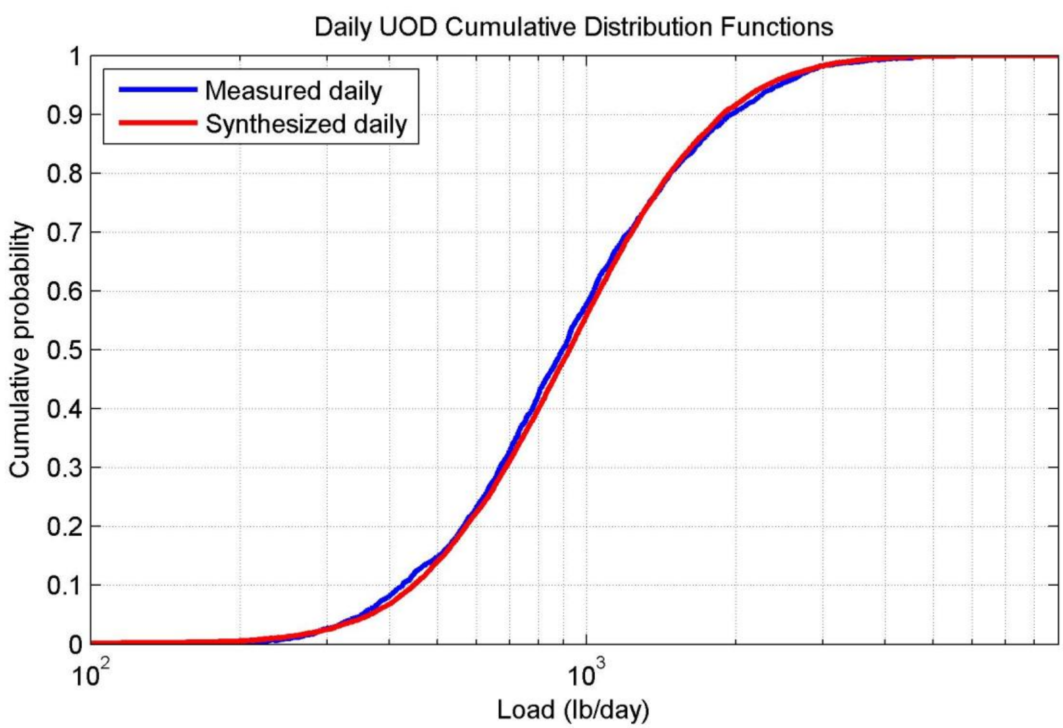


Figure 3.8.5.3.18. BP Amoco 50-year UOD discharge time series and CDFs

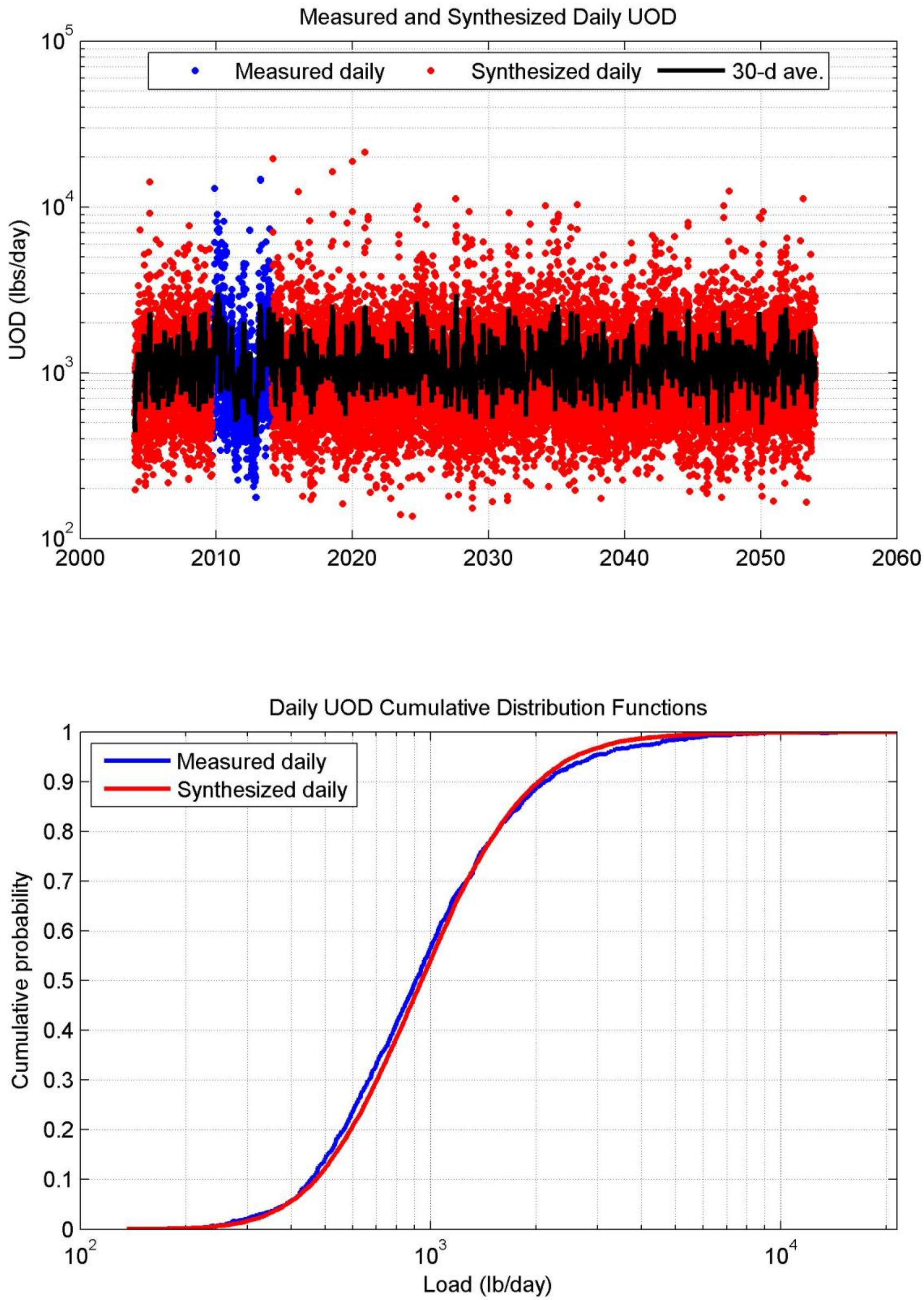


Figure 3.8.5.3.19. Lower Berkeley 50-year UOD discharge time series and CDFs

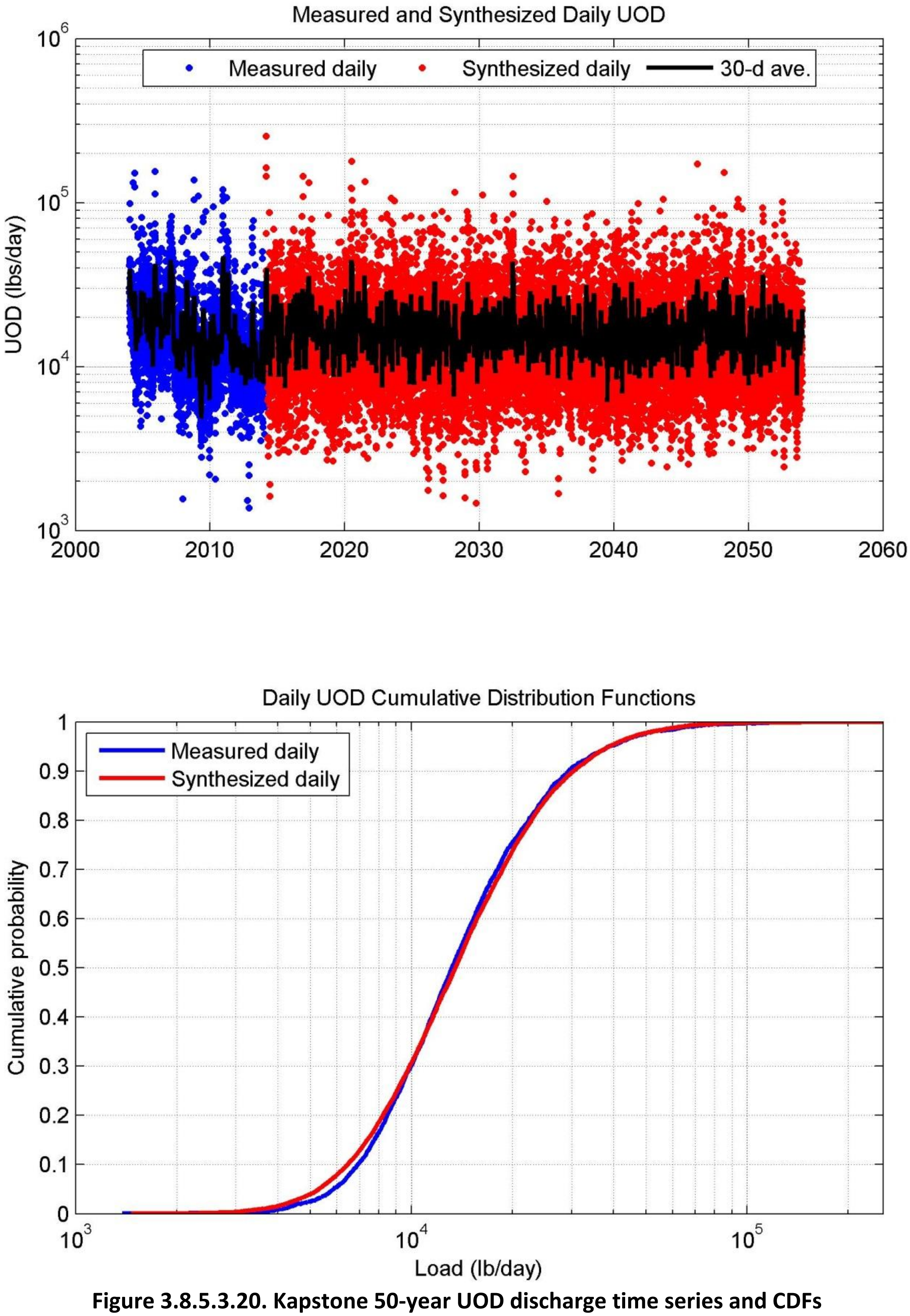
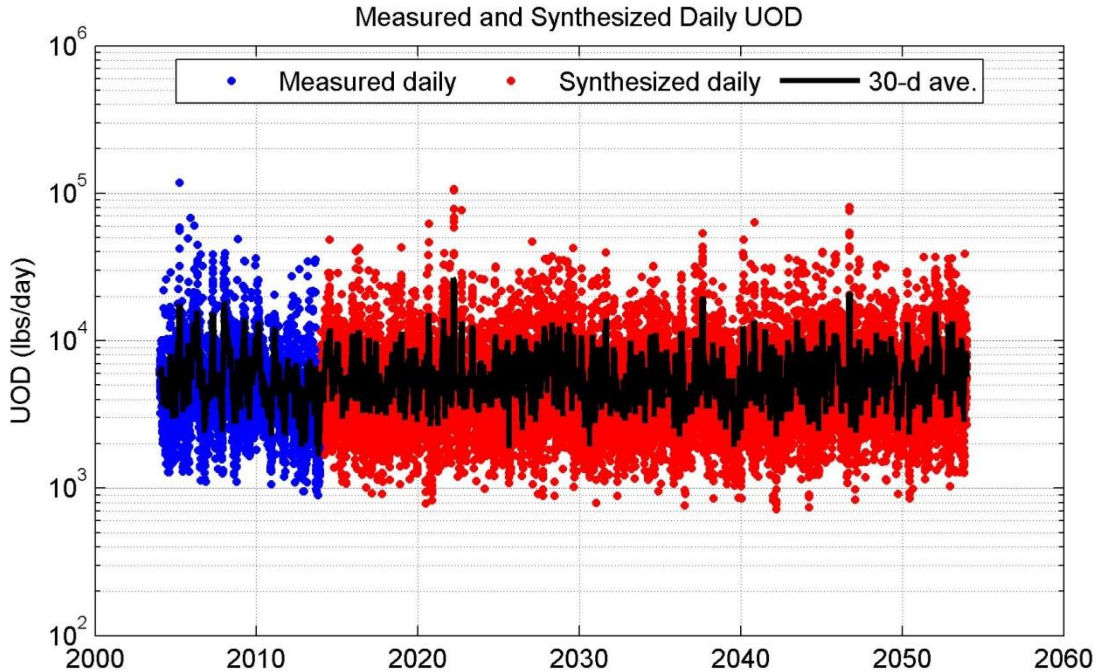


Figure 3.8.5.3.20. Kapstone 50-year UOD discharge time series and CDFs



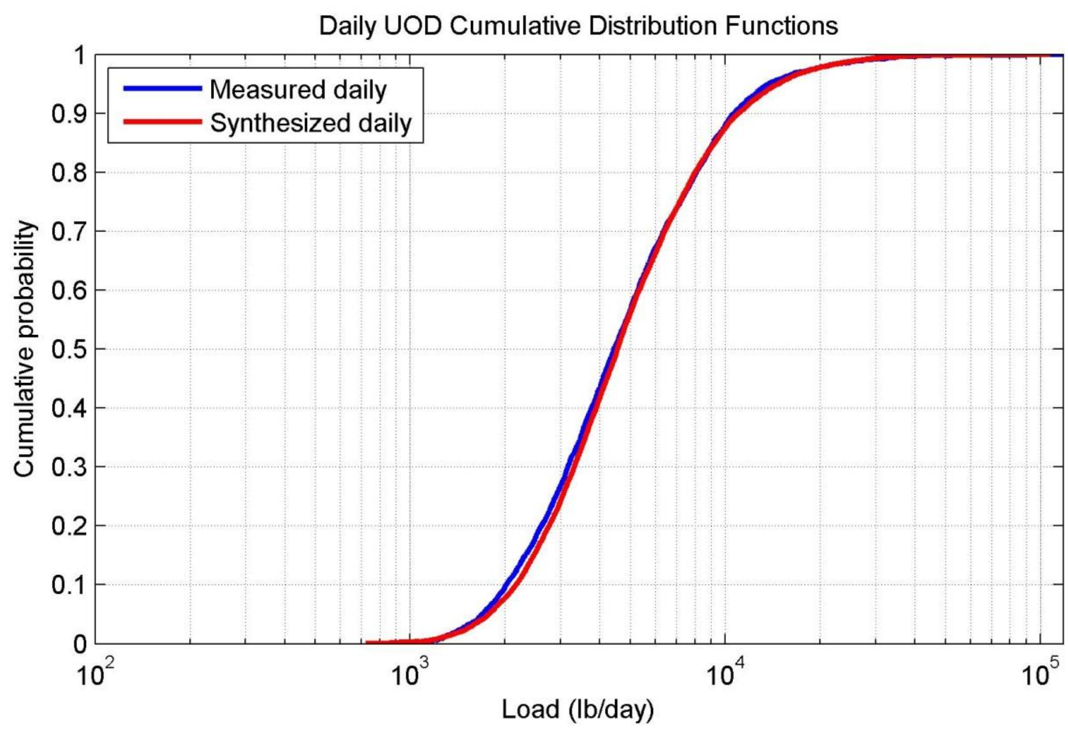


Figure 3.8.5.3.21. CPW Plum Island 50-year UOD discharge time series and CDFs

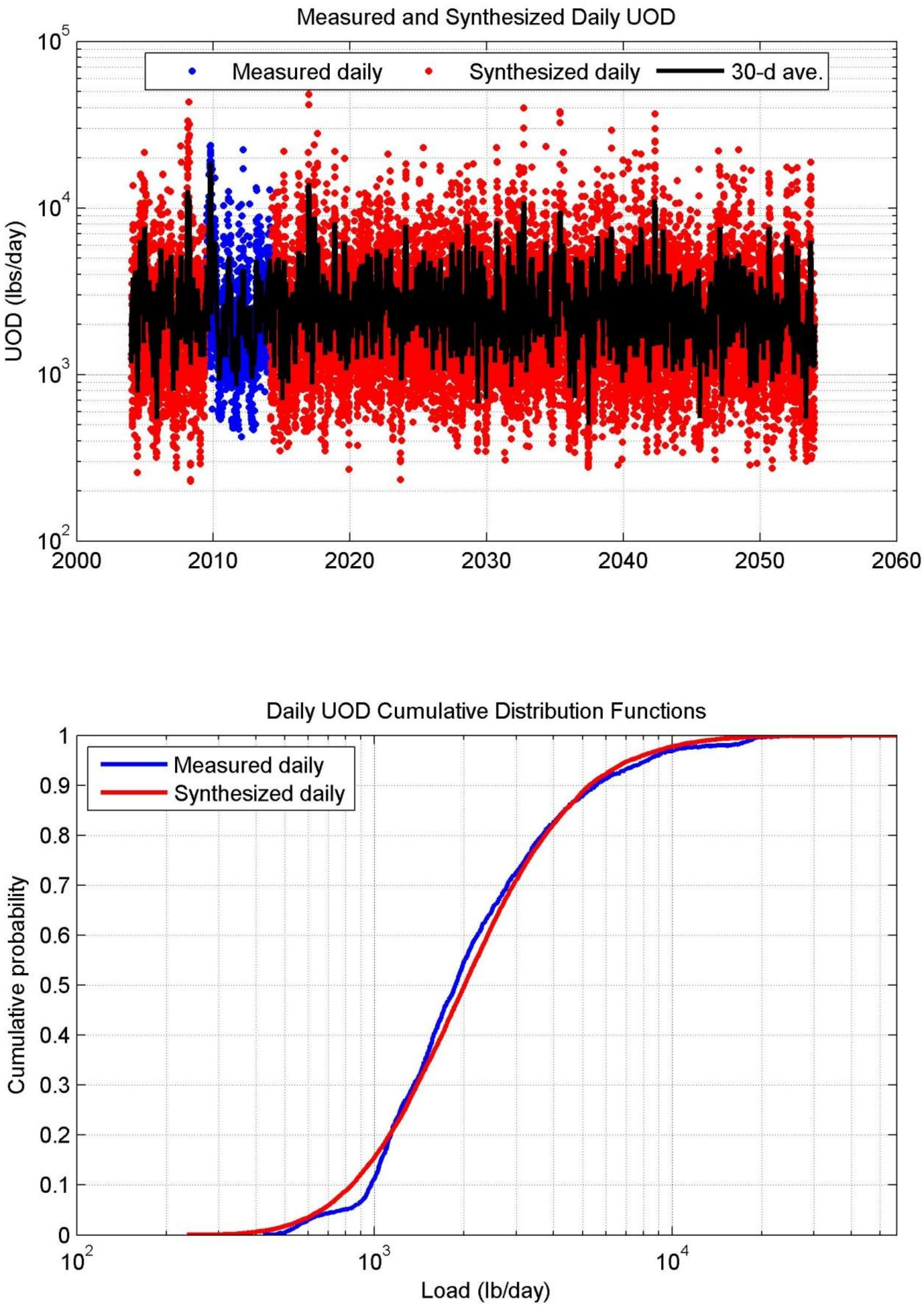
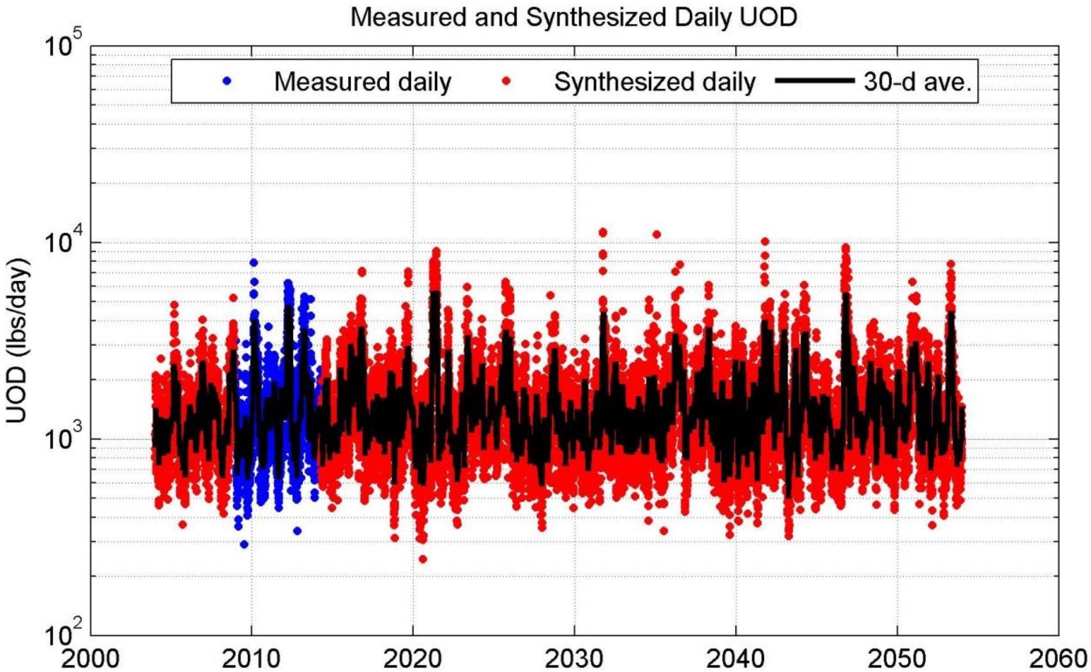


Figure 3.8.5.3.22 NCSD 50-year UOD discharge time series and CDFs



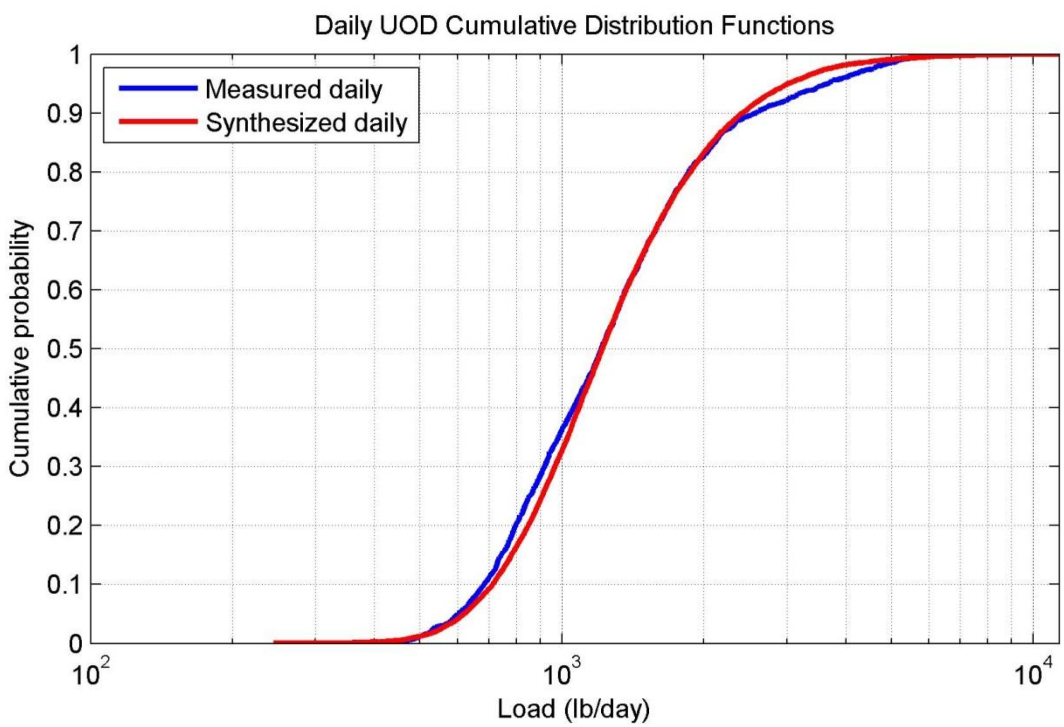


Figure 3.8.5.3.23. Mount Pleasant CS & RR 50-year UOD discharge time series and CDFs

3.8.5.4 Modeled Dissolved Oxygen Impacts

The previously calibrated TMDL EFDC model was used to estimate the DO impacts from the time-variable point-source discharges. This was accomplished by modeling DO concentrations during 50 one-year periods. A single 50-year simulation was not used primarily because of computational time constraints. The 50 one-year scenarios used the model inputs for the 2004 hydrologic conditions from the TMDL model (as discussed in Section 3.8.5.2), but the point source loads for each scenario were replaced by each of the 50 one-year periods of synthesized daily point-source loads (described in Section 3.8.5.3.2). The results from the modeled DO impacts were then added to the Post 45 project DO impacts for the purposes of assessing the cumulative project impacts.

3.8.5.4.1 Model Setup

The EFDC model was previously set up for the TMDL study to model both a loading scenario and a no-load scenario for simulating changes in DO concentration resulting from NPDES point-source discharge loads. For this study, the TMDL loading scenario was modified by revising the input mass-loading rate file (WQPSL.INP) to use the time-varying daily loading rate time series for the major NPDES dischargers listed in Table 3.8.5.3.2. The synthesized long-term 50-year loading rate time series was divided into 50 one-year loading rate files. These 50 one-year model runs were then executed for the time period from January 1 through October 31, which includes a model spin-up period from January 1 through February 29 prior the March 1 through October 31 TMDL evaluation period.

In addition to the 50 loading scenario model runs, a no-load scenario was modeled. This no-load specified zero loading rates for the point-source discharges (with the exception of the Summerville and Dorchester WWTP discharges, as explained in Section 3.8.5.2 of this report).

3.8.5.4.2 Cumulative Impacts Assessment

The model results were post-processed to achieve the same time and spatial averaging of DO concentrations as the TMDL model study. The concentrations were time-averaged as daily DO concentrations for the TMDL assessment period extending from March 1 through October 31. The concentrations were also volume-averaged within the same river segments as those used for the TMDL model study (Figure 3.8.5.1). These volume-averaged segments include averaging vertically through the water column as well as horizontal averaging over the segment areas.

The change in DO concentration (the delta DO) caused by the point-source discharges was calculated by subtracting the time series of daily-averaged, segment-averaged DO concentrations for the loading scenarios (all 50 one-year scenarios) from the no-load scenario. The 90th percentile of this delta DO for all 50 years was then calculated for each segment.

To calculate the cumulative impacts from the time-varying point-source loads plus the Post 45 project impacts, the time-series of daily delta DO for the point-source loading was added to the daily delta DO for the Post 45 project, resulting in a time-series of daily cumulative delta DO values. The 90th percentile of this time series of daily cumulative delta DO was calculated as the cumulative impact.

The results are shown in Figures 3.8.5.4.1 and 3.8.5.4.2, which show the cumulative delta DO concentrations along the Cooper and Wando Rivers, respectively. The x-axis of these plots extends from the harbor entrance (Fort Sumter) up the harbor and along the channel of each river. The approximate river mile locations are also shown on Figure 3.8.5.1. These results are also tabulated in Tables 3.8.5.4.1 and 3.8.5.4.2.

As shown by Figure 3.8.5.4.1 and Table 3.8.5.4.1, the maximum cumulative delta DO in the Cooper River is -0.1419 mg/L, which occurs at river mile 14. This maximum Cooper River cumulative impact occurs in Segment C7, which is just above interstate I-526. The time series of the daily-averaged delta DO for this segment is shown in Figure 3.8.5.4.3.

As shown by Table 3.8.5.4.1, the maximum delta DO in the Cooper River caused by the point-source discharges is -0.0773 mg/L, which occurs in segment C10. By comparison, the TMDL study (Cantrell 2013) found that constant discharges at the maximum permitted loading rates results in a maximum delta DO of -0.146 mg/L in segment C11. Therefore, the maximum predicted delta DO impact from the constant loading rates is approximately 1.9 times the value resulting from time-varying loading rates.

As shown by Figure 3.8.5.4.2 and Table 3.8.5.4.2, the maximum cumulative delta DO in the Wando River is -0.1439 mg/L, which occurs at river mile 8.1. This maximum Wando River cumulative impact occurs in Segment W2, which is just above the confluence of the Wando and Cooper Rivers. The time series of the daily-averaged delta DO for this segment is shown in Figure 3.8.5.4.4.

As discussed in Section 3.8.5.2, the cumulative impacts to the Ashley River were not evaluated with the time-varying loading model, because these two discharges do not affect DO in the critical segments of the Cooper and Wando Rivers. TMDL model was used to evaluate cumulative impacts from constant loading from the discharges, and the results indicate that the cumulative impacts in the Ashley River will not exceed the allowable 0.1 mg/L impact to DO (which is an allowable 0.1499 mg/L, in practice), even with the assumption of constant point-source discharges at the maximum permitted loading rates. The delta DO values along the Ashley River calculated using the TMDL model results are shown in Figure 3.8.5.4.5. As shown in this figure, the maximum cumulative delta DO is -0.1152 mg/L, which is less than the allowable impact to DO.

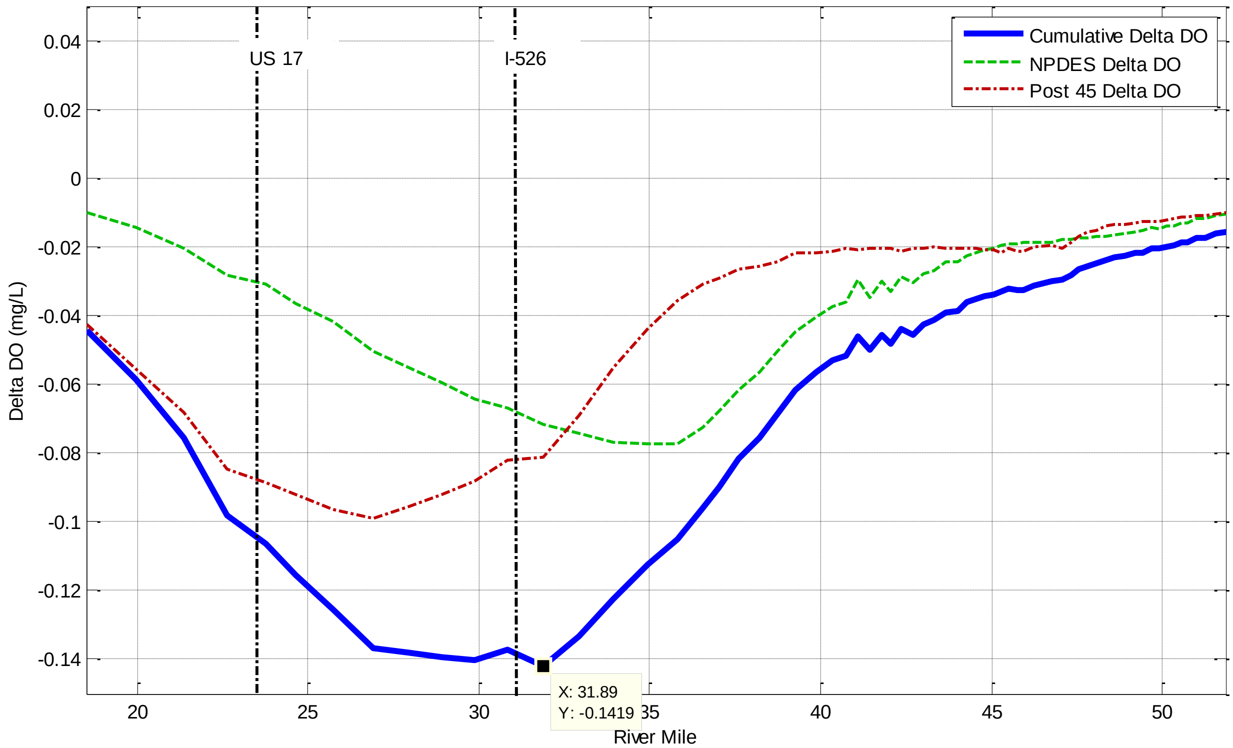


Figure 3.8.5.4.1 Longitudinal plot of 90th percentile delta DO along Cooper River

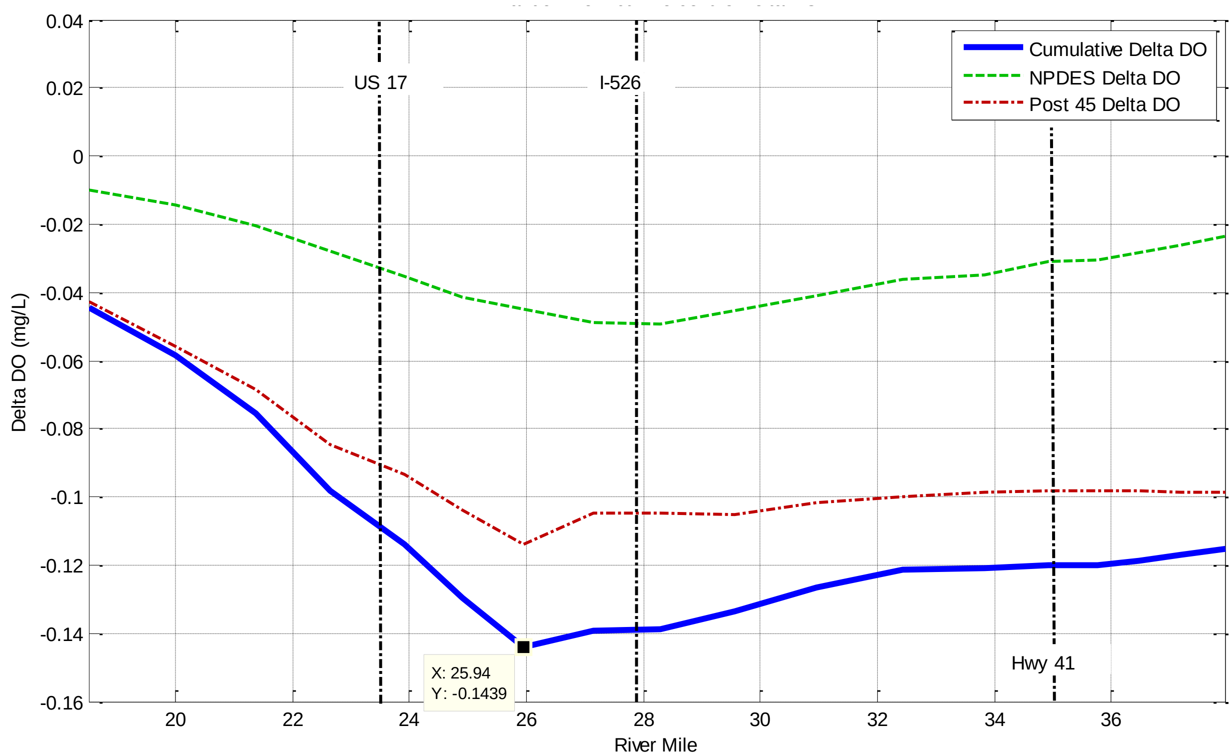


Figure 3.8.5.4.2 Longitudinal plot of 90th percentile delta DO along Wando River

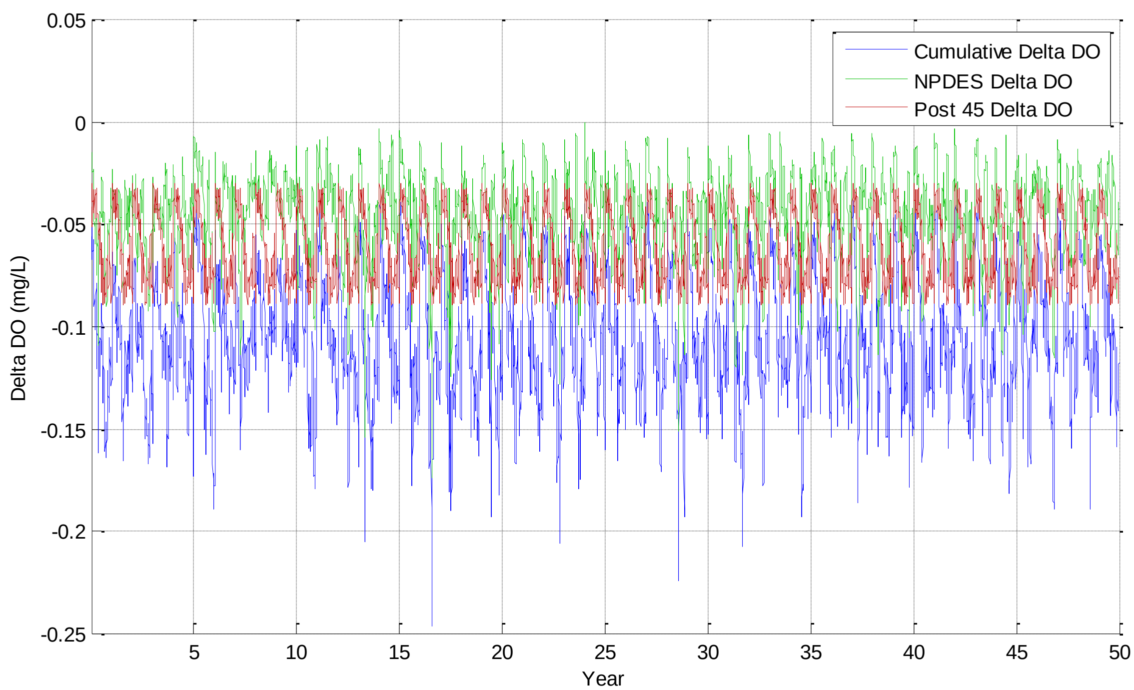


Figure 3.8.5.4.3 Time series of delta DO in Cooper River Segment C7

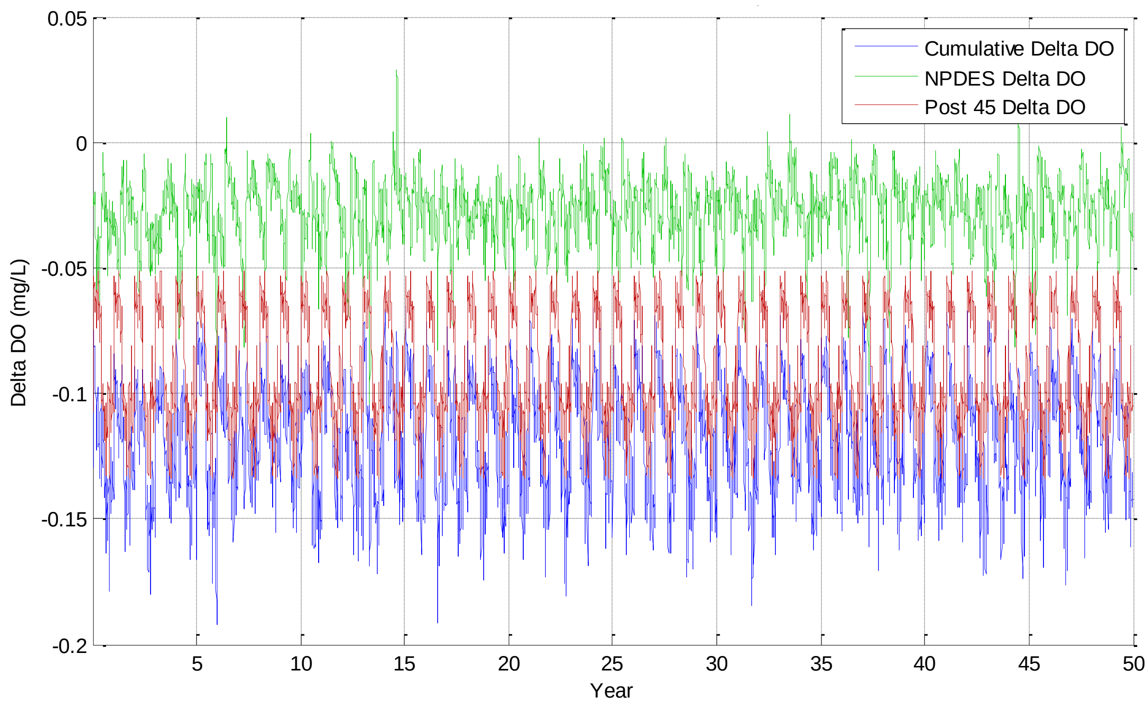


Figure 3.8.5.4.4 Time series of delta DO in Wando River Segment W2

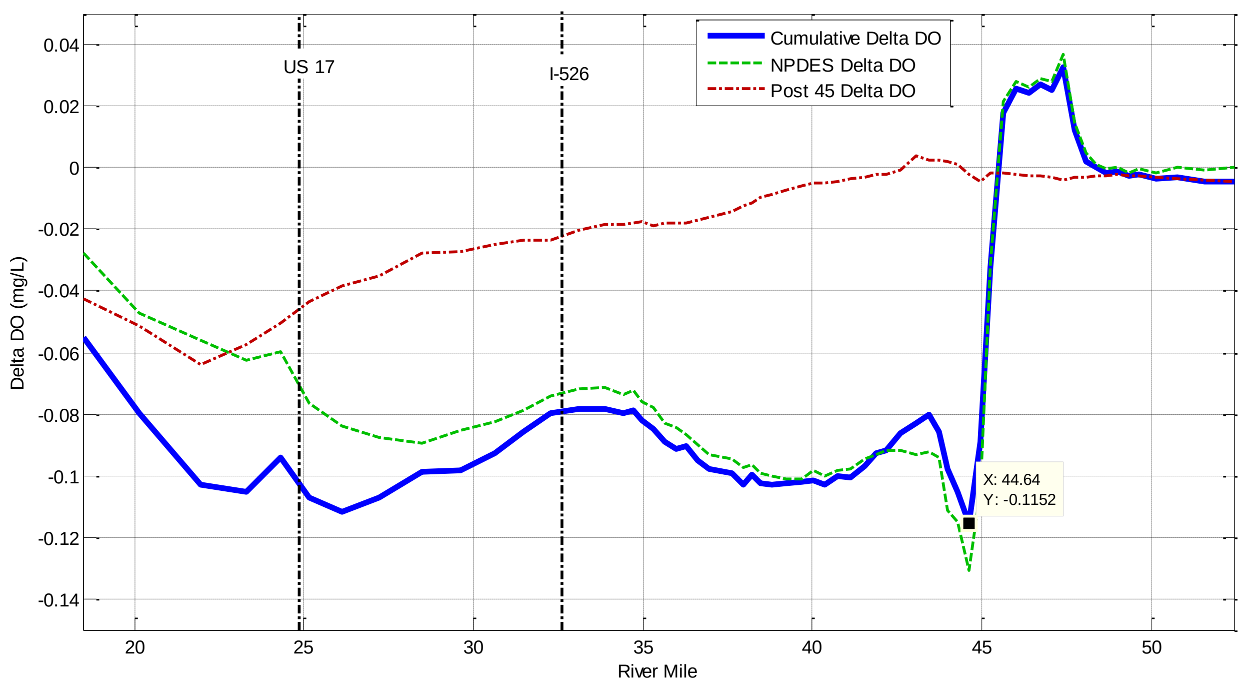


Figure 3.8.5.4.5 Longitudinal plot of 90th percentile delta DO along Ashley River

Table 3.8.5.4.1 Cooper River 90th percentile delta DO from point-source discharges, Post 45 project impacts and cumulative impacts

Segment	River Miles	NPDES Delta DO (mg/L)	Post45 Delta DO (mg/L)	Cumulative Delta DO (mg/L)
H1	18.5	-0.0099	-0.0428	-0.0445
H3	20.0	-0.0143	-0.0557	-0.0586
H5	21.4	-0.0206	-0.0684	-0.0754
H6	22.7	-0.0281	-0.0848	-0.0983
H7	23.8	-0.0308	-0.0885	-0.1062
H8	24.6	-0.0366	-0.0919	-0.1154
C1	25.7	-0.0416	-0.0965	-0.1255
C2	26.9	-0.0505	-0.0991	-0.1367
C3	28.0	-0.0552	-0.0955	-0.1379
C4	28.9	-0.0597	-0.0920	-0.1392
C5	29.9	-0.0644	-0.0880	-0.1402
C6	30.8	-0.0671	-0.0821	-0.1374
C7	31.9	-0.0715	-0.0811	-0.1419
C8	32.9	-0.0742	-0.0693	-0.1333
C9	33.9	-0.0771	-0.0553	-0.1223
C10	34.9	-0.0773	-0.0438	-0.1125
C11	35.8	-0.0772	-0.0356	-0.1050
C12	36.5	-0.0726	-0.0307	-0.0960
C13	37.0	-0.0679	-0.0291	-0.0899
C14	37.6	-0.0616	-0.0266	-0.0816
C15	38.2	-0.0564	-0.0256	-0.0754
C16	38.7	-0.0507	-0.0242	-0.0687
C17	39.3	-0.0450	-0.0220	-0.0615
C18	39.8	-0.0405	-0.0216	-0.0565
C19	40.4	-0.0376	-0.0212	-0.0532
C20	40.8	-0.0361	-0.0206	-0.0516
C21	41.1	-0.0298	-0.0211	-0.0460
C22	41.5	-0.0348	-0.0206	-0.0502
C23	41.8	-0.0301	-0.0203	-0.0456
C24	42.0	-0.0332	-0.0207	-0.0483
C25	42.4	-0.0286	-0.0213	-0.0439
C26	42.7	-0.0305	-0.0206	-0.0456
C27	43.0	-0.0277	-0.0207	-0.0425
C28	43.3	-0.0271	-0.0201	-0.0415

Table 3.8.5.4.2 Wando River 90th percentile delta DO from point-source discharges, Post 45 project impacts and cumulative impacts

Segment	River Miles	NPDES Delta DO (mg/L)	Post45 Delta DO (mg/L)	Cumulative Delta DO (mg/L)
H1	18.5	-0.0099	-0.0428	-0.0445
H3	20.0	-0.0143	-0.0557	-0.0586
H5	21.4	-0.0206	-0.0684	-0.0754
H6	22.7	-0.0281	-0.0848	-0.0983
H9	23.9	-0.0352	-0.0936	-0.1141
W1	24.9	-0.0413	-0.1041	-0.1298
W2	25.9	-0.0451	-0.1138	-0.1439
W3	27.1	-0.0487	-0.1048	-0.1392
W4	28.3	-0.0493	-0.1047	-0.1390
W5	29.6	-0.0455	-0.1050	-0.1335
W6	31.0	-0.0408	-0.1019	-0.1266
W7	32.4	-0.0363	-0.0999	-0.1215
W8	33.8	-0.0348	-0.0987	-0.1207
W9	34.9	-0.0311	-0.0984	-0.1199
W10	35.8	-0.0304	-0.0984	-0.1202
W11	36.5	-0.0284	-0.0984	-0.1187
W12	37.2	-0.0260	-0.0988	-0.1170
W13	37.9	-0.0235	-0.0986	-0.1151

3.8.5.5 Conclusions

The project will not cause a need to change the TMDL. The existing TMDL has no capacity for additional loading; however the loading that is ACTUALLY occurring is well below the TMDL allotment.

Analysis of the cumulative dissolved oxygen DO impacts resulting from both the point-source pollution discharges into the estuary and the proposed Post 45 Project navigation channel expansion indicates that the Post 45 project will not cause cumulative DO impacts greater than the 0.1 mg/L allowed by DHEC's anti-degradation rule. Although the greatest cumulative impacts are estimated to be 0.14 mg/L, this is less than the 0.1499 mg/L allowed in practice. As a result, mitigation for DO impacts should not be required to offset project impacts in order to comply with the anti-degradation rule.

4.0 Sea Level Change Rate Evaluation

Initial runs of EFDC were done using the low (historic) rate of sea level change, for comparative purposes. Per ER 1110-2-8162, after selection of the TSP, EFDC was to be rerun with intermediate and high rates for TSP and future without conditions. As stated previous in Section 2.2, using the IWR online Sea-Level Change calculator and spreadsheet the trend at Charleston is estimated to be 2.94 mm/yr. Estimating construction completion of 2021, and a 50 year project life, starting with 2012 (thus estimate the increase in 59 years) - the “low” rate of change is 0.57 feet, the “intermediate” is 1.08 feet and the “high” is 2.74 feet.

NED was selected as 50-48 alternative, however, the Locally Preferred Plan (LPP) is 52-48, and thus the recommended plan is 52-48. Intermediate and high sea level changes were computed for the future without project, and both the 50-48 and the 52-48 alternatives.

4.1 Water Levels

Comparison of water levels between alternative depths to future without project conditions, using both the intermediate sea level (ISL) and the high sea level (HSL) rates, show little difference due to the project.

Under ISL the contraction dikes are not overtopped, but they are under HSL. Corps will assess the need for modification as part of normal operations and maintenance actions. As part of normal maintenance of disposal areas, erosion and toe protection would be evaluated as needed. Low-lying and marsh areas will be impacted and waterfront property owners will need to assess their own risk and adapt.

River changes between the project alternatives and the Future Without scenarios in Tables 4.1.1 are very small (0.06 feet or less increase in 99th percentile water level). The increase does not change any of the impacts over the without condition alternative. It is expected that more tidal alerts would occur with higher sea level changes.

Table 4.1.1 Changes in Water Surface Elevation

Percentile			USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC						
WSE	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	3.14	3.63	5.22	3.15	3.63	5.22	3.15	3.63	5.22
10th percentile	3.76	4.25	5.84	3.77	4.25	5.85	3.76	4.26	5.85
50th percentile	4.68	5.16	6.75	4.7	5.17	6.76	4.7	5.18	6.76
90th percentile	5.56	6.04	7.65	5.58	6.05	7.67	5.58	6.05	7.67
99th percentile	6.18	6.67	8.29	6.18	6.69	8.31	6.19	6.69	8.32
			USGS 02172050 Cooper River near Goose Creek, SC						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	1.83	2.35	4.07	1.8	2.33	4.04	1.79	2.33	4.04
10th percentile	2.54	3.08	4.71	2.53	3.06	4.69	2.52	3.06	4.69
50th percentile	4.35	4.83	6.48	4.36	4.85	6.49	4.36	4.85	6.49
90th percentile	6.07	6.55	8.17	6.1	6.59	8.2	6.1	6.59	8.21
99th percentile	6.87	7.36	8.95	6.9	7.38	8.98	6.9	7.39	8.98
			USGS 02172053 Cooper River at Mobay						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	0.81	1.39	3.16	0.74	1.33	3.11	0.73	1.32	3.11
10th percentile	1.69	2.25	3.94	1.65	2.21	3.93	1.65	2.2	3.92
50th percentile	4.11	4.58	6.19	4.11	4.58	6.19	4.12	4.58	6.18
90th percentile	6.42	6.92	8.55	6.44	6.94	8.57	6.45	6.95	8.59
99th percentile	7.28	7.79	9.44	7.33	7.82	9.47	7.34	7.84	9.49
			USGS 02172067.7 Cooper River at I-526						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	0.22	0.73	2.35	0.15	0.66	2.28	0.12	0.64	2.28
10th percentile	1.19	1.69	3.29	1.13	1.63	3.26	1.05	1.61	3.25
50th percentile	4.02	4.5	6.1	4.01	4.49	6.11	4.01	4.5	6.12
90th percentile	6.54	7.06	8.74	6.58	7.09	8.77	6.59	7.1	8.77
99th percentile	7.59	8.11	9.82	7.64	8.17	9.85	7.66	8.17	9.86

Table 4.1.1 (continued) Changes in Water Surface Elevation

			USGS 02172069.8 Wando River above Mt. Pleasant						
WSE	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	0.27	0.75	2.29	0.2	0.68	2.22	0.19	0.67	2.21
10th percentile	1.18	1.66	3.21	1.12	1.6	3.17	1.11	1.59	3.16
50th percentile	3.82	4.31	5.94	3.81	4.3	5.94	3.8	4.3	5.94
90th percentile	6.48	7.01	8.73	6.5	7.02	8.74	6.5	7.03	8.74
99th percentile	7.57	8.11	9.88	7.6	8.14	9.9	7.61	8.15	9.9
			USGS 02172070.9 Cooper River at Hwy 17						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	0.11	0.59	2.21	0.05	0.54	2.18	0.03	0.53	2.18
10th percentile	1.08	1.58	3.17	1.04	1.54	3.15	1.03	1.52	3.14
50th percentile	3.85	4.35	5.98	3.86	4.36	3.15	3.86	4.36	5.98
90th percentile	6.37	6.88	8.57	6.39	6.9	8.59	6.39	6.91	8.59
99th percentile	7.39	7.92	9.62	7.41	7.94	9.64	7.42	7.95	9.64
			USGS 02172086.9 Ashley River near North Charleston						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	-0.16	0.34	1.94	-0.22	0.29	1.9	-0.23	0.27	1.89
10th percentile	0.88	1.38	3.03	0.85	1.34	3	0.84	1.34	2.99
50th percentile	3.99	4.5	6.11	3.99	4.49	6.1	3.99	4.49	6.09
90th percentile	6.58	7.09	8.77	6.59	7.1	8.78	6.6	7.11	8.78
99th percentile	7.63	8.14	9.82	7.64	8.16	9.83	7.64	8.16	9.84
			USGS 02172100 Charleston Harbor at Fort Sumter						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	0.08	0.57	2.22	0.04	0.55	2.2	0.03	0.54	2.19
10th percentile	1.04	1.54	3.18	1.02	1.52	3.17	1.01	1.51	3.16
50th percentile	3.74	4.24	5.91	3.73	4.24	5.91	3.73	4.24	5.91
90th percentile	6.25	6.77	8.44	6.26	6.78	8.45	6.27	6.79	8.46
99th percentile	7.27	7.79	9.47	7.29	7.8	9.48	7.29	7.81	9.48

4.2 Currents

Changes in 95th percentile depth-averaged simulated current speeds caused by the sea level changes, resulted in very small increases in current speeds in the lower harbor (maximum increases on the order of 0.13 feet/second for ISL to 0.12 feet/second for HSL). No impacts due to changes in current are expected. Impacts to ship maneuverability will be assessed during ship simulation in PED phase. For intermediate sea level change, Figure 4.2.1 shows the FWO conditions, while Figure 4.2.2 shows the 50-48 compared to FWO and Figure 4.2.3 shows the 52-48 compared to FWO. For high sea level change, Figure 4.2.4 shows the FWO conditions, while Figure 4.2.5 shows the 50-48 compared to FWO and Figure 4.2.6 shows the 52-48 compared to FWO.

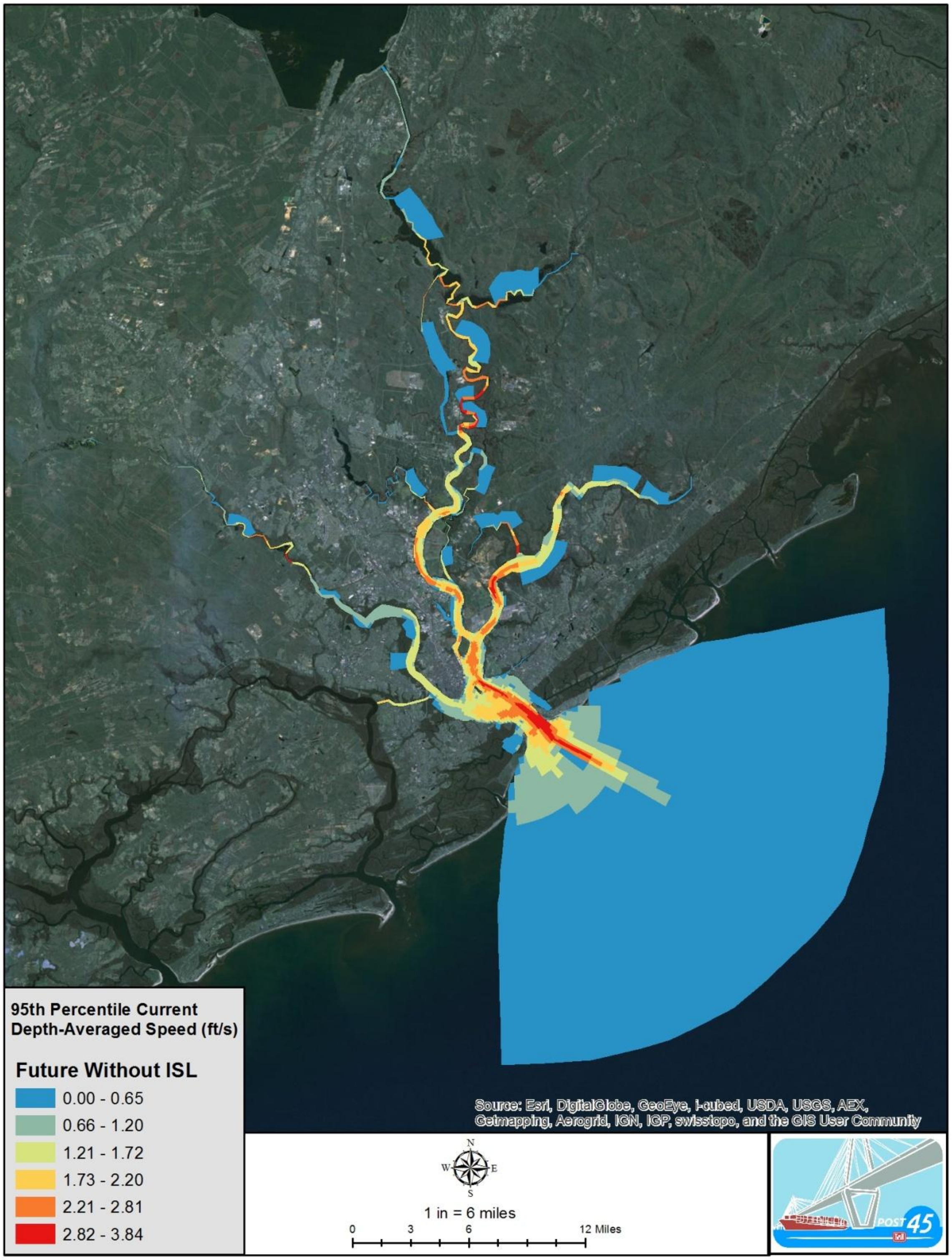


Figure 4.2.1 Intermediate Sea Level Change Currents FWO conditions

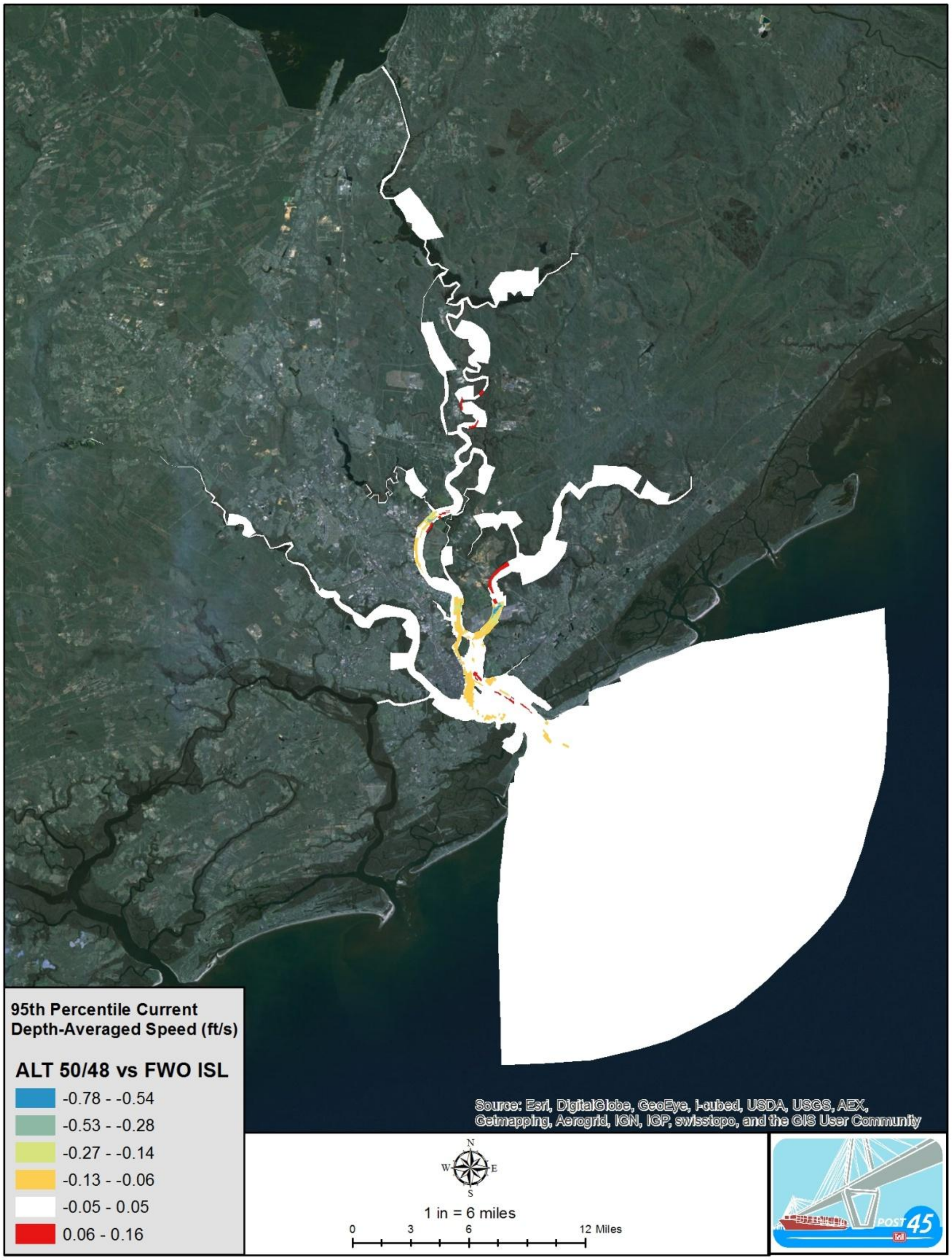


Figure 4.2.2 Intermediate Sea Level Change Currents 50-48 compared to FWO

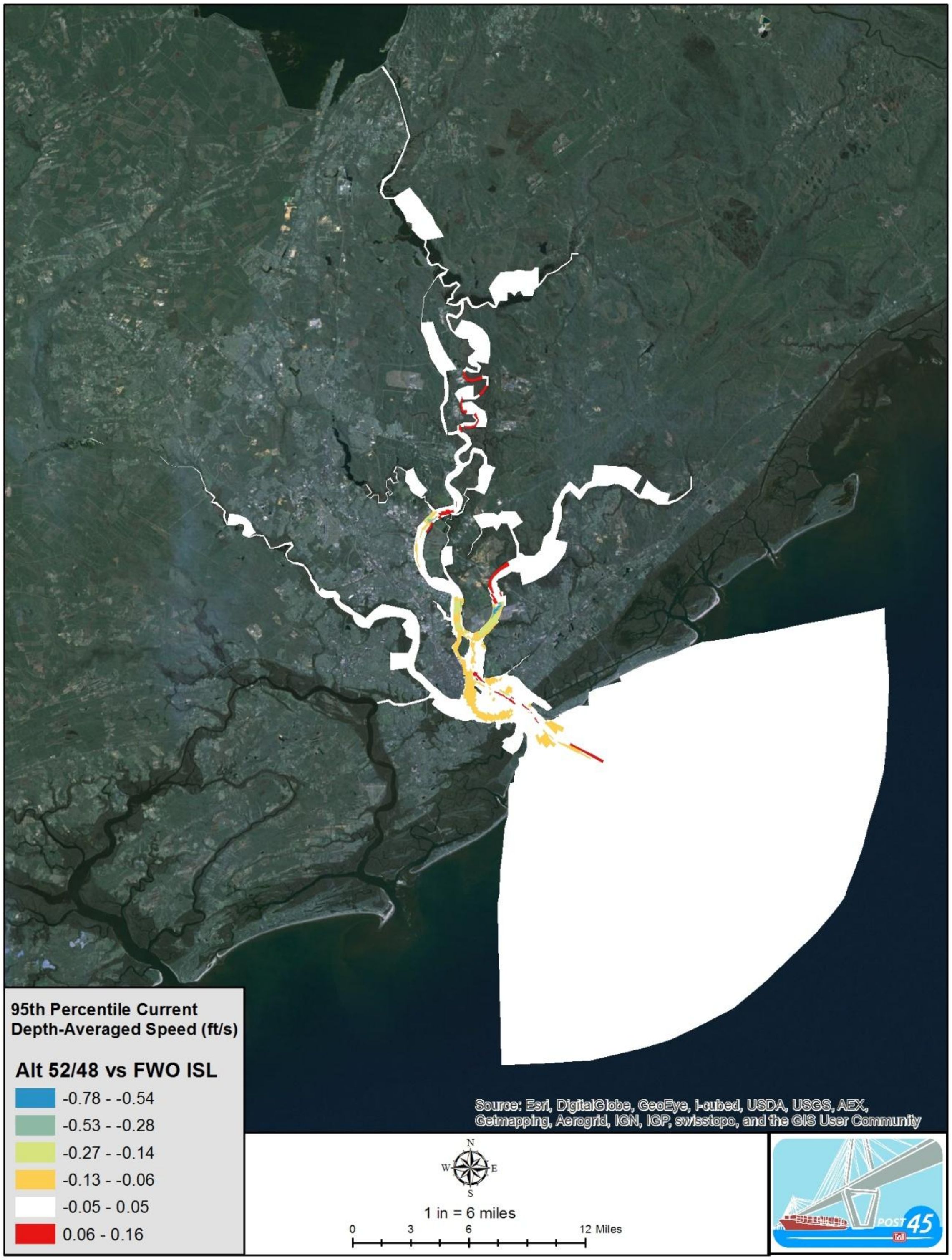


Figure 4.2.3 Intermediate Sea Level Change Currents 52-48 compared to FWO.

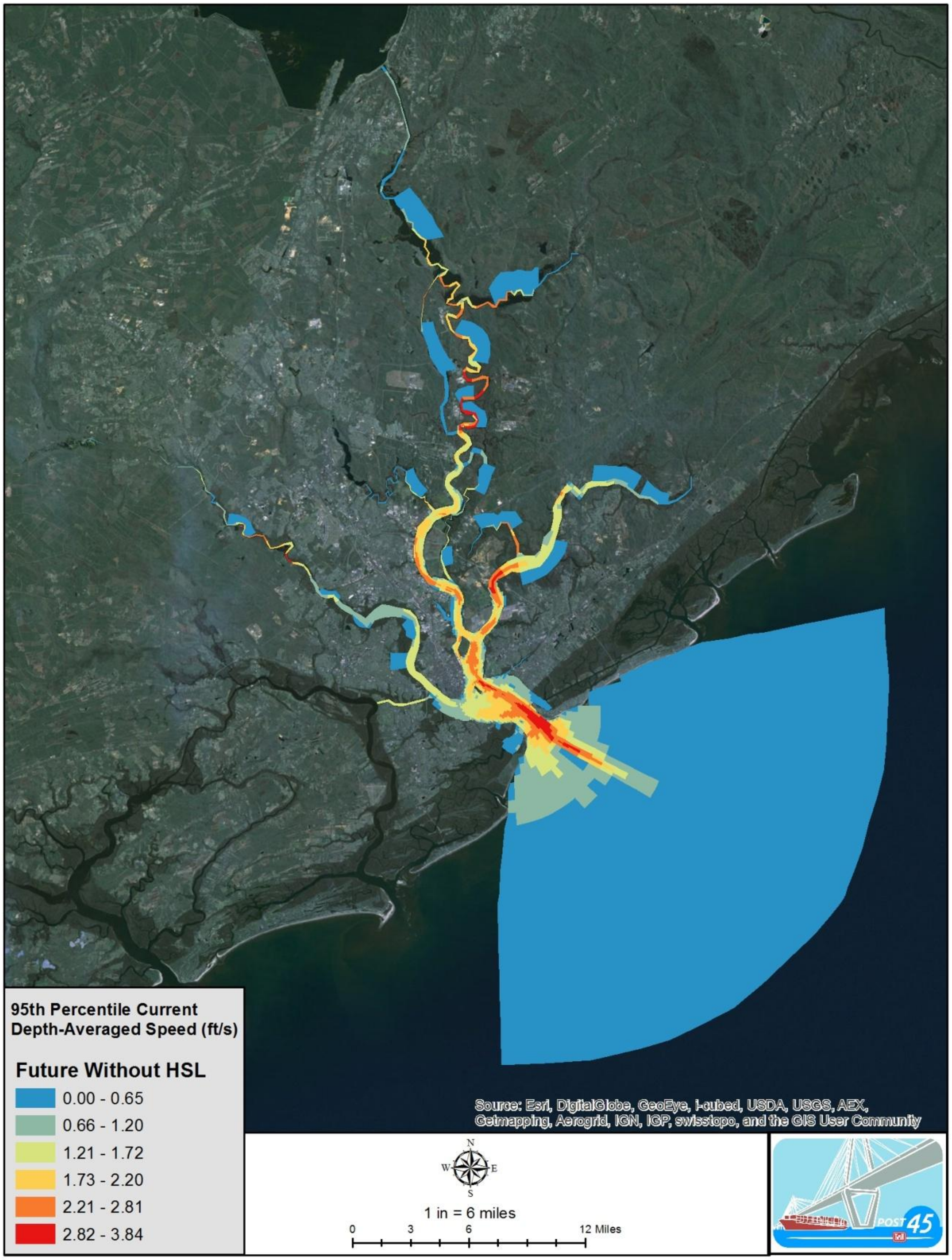


Figure 4.2.4 High Sea Level Change Currents FWO Conditions

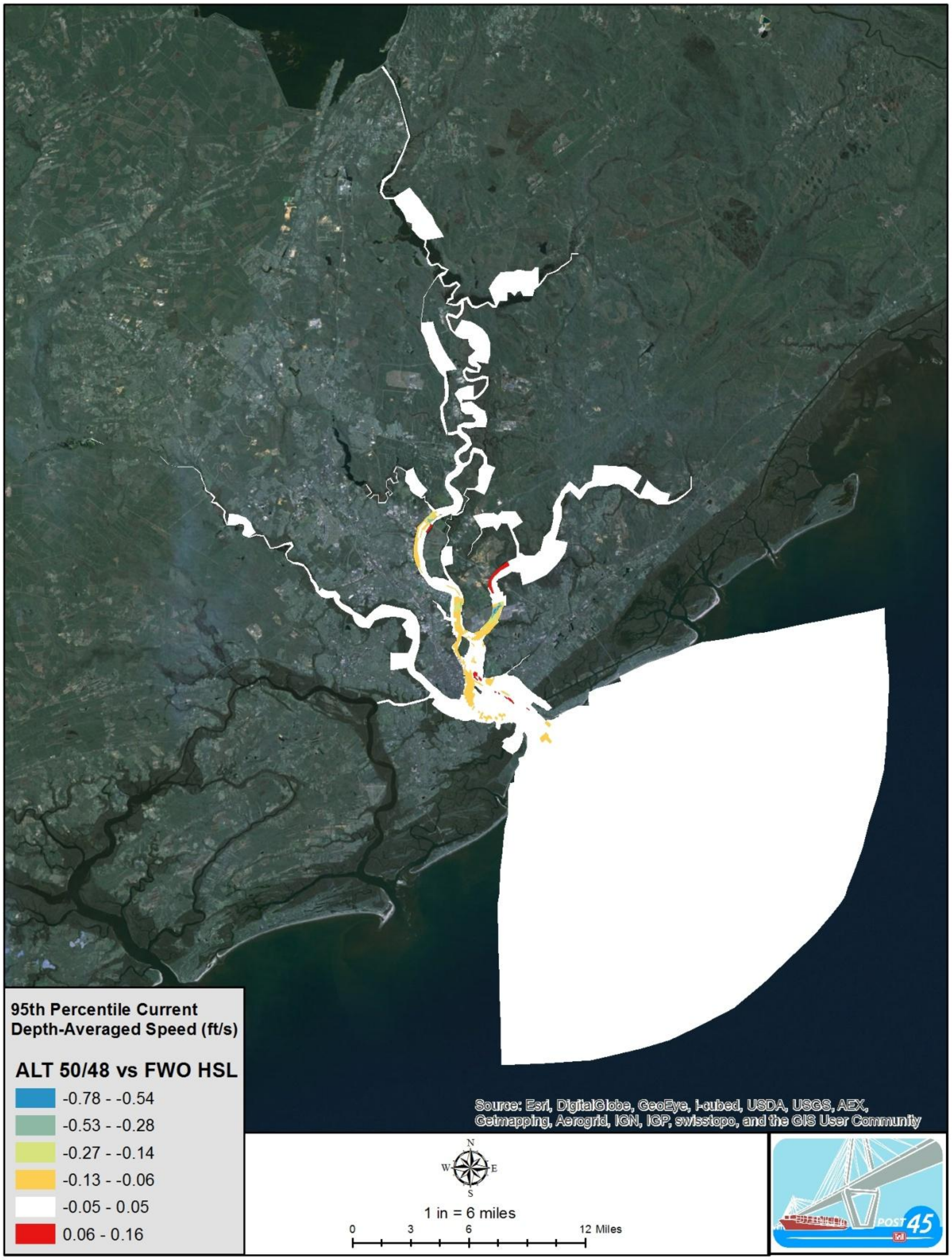


Figure 4.2.5 High Sea Level Change Currents 50-48 compared to FWO

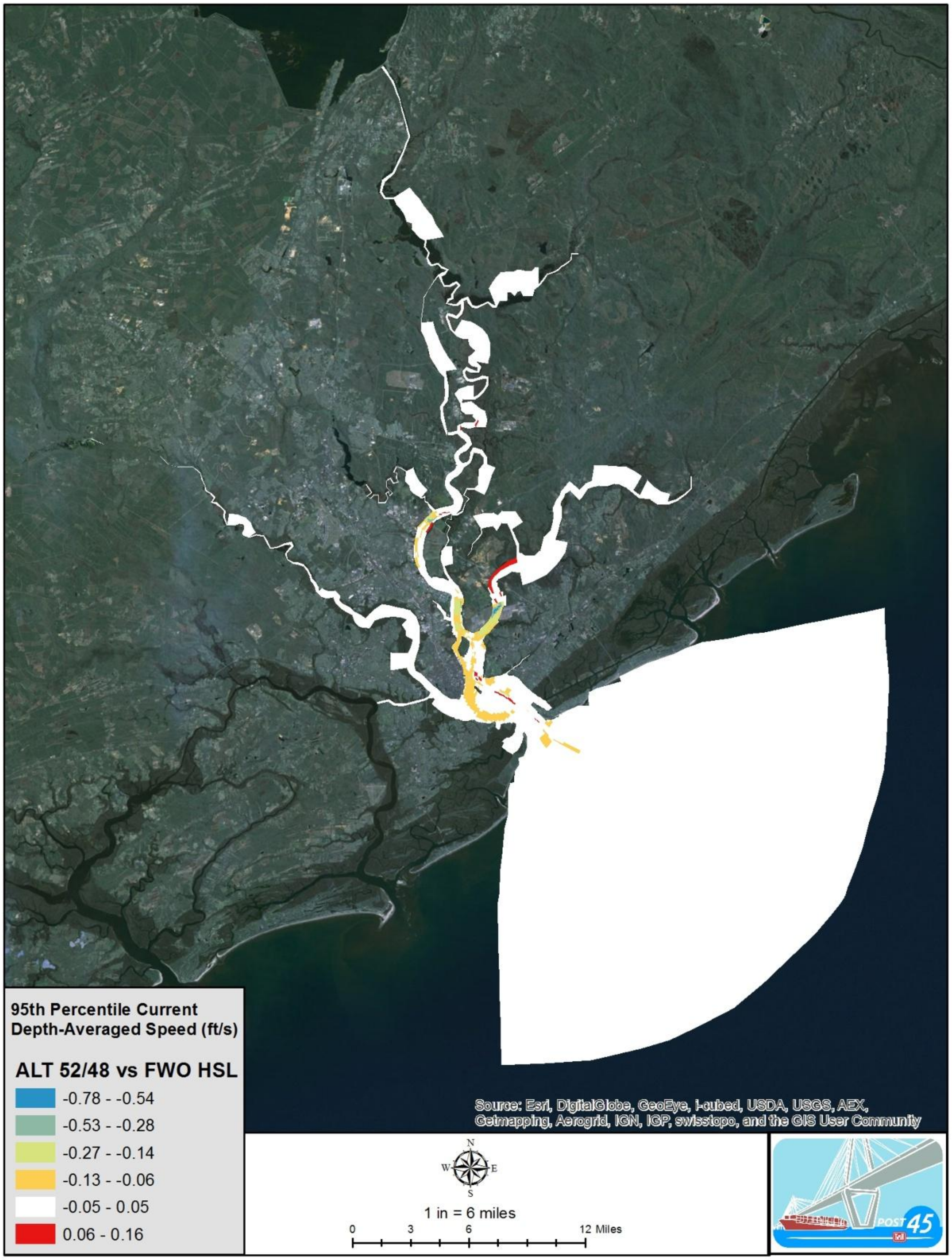


Figure 4.2.6 High Sea Level Change Currents 52-48 compared to FWO.

4.3 Shoaling Rates

Shoaling rates decrease with increasing sea level change. The change in sedimentation caused by change in sea level probably mostly caused by: change in current velocities (caused by change in depth), and change in salinity intrusion (which changes the location of the turbidity maximum in the estuary). In the model, there is no change in sediment source rates.

The reduction in shoaling between sea level changes is about 3.5% for ISL to historic (low) and 16% for HSL to low rate for without project conditions. The reduction in shoaling between sea level changes is about 4.5% for ISL to low and 18% for HSL to low rate for 50-48 project conditions. The reduction in shoaling between sea level changes is about 5% for ISL to low and 18% for HSL to low rate for 52-58 project conditions.

Shoaling rates within the sea level change remained approximately the same ratios of increase per alternative. Potentially, high rates of sea level change may indicate a reduction in maintenance costs over the low rate of sea level change.

For ISL, Figure 4.3.1 shows the FWO conditions, while Figure 4.3.2 shows the 50-48 compared to FWO and Figure 4.3.3 shows the 52-48 compared to FWO. For HSL, Figure 4.3.4 shows the FWO conditions, while Figure 4.3.5 shows the 50-48 compared to FWO and Figure 4.3.6 shows the 52-48 compared to FWO.

Table 4.3.1 Shoaling Quantities

Shoaling Rate Indices	Avg Maintenance Dredging 2004-2012 (CY/yr) FWO	FWO ISL	FWO HSL	5048	5048 ISL	5048 HSL	52-48	5248 ISL	5248 HSL
Mount Pleasant Reach	0	0	815	0	0	0	0	0	0
Rebellion Reach	0	0	0	0	0	0	923	2143	2898
Bennis Reach	0	3468*	3337*	34138*	27255*	10955*	37264*	24314*	19527*
Horse Reach	0	19320*	27387*	12457*	22135*	22827*	16035*	26352*	23443*
Hog Island Reach	117444	98160	67759	169094	159583	120350	179838	168942	128833
Drum Island Reach	91897	96986	103995	118305	129638	147624	131287	142161	159714
Meyers Bend Reach	23686	20193	10756	49538	46898	30590	55119	51926	38173
Daniel Island Reach	175287	169355	154016	218978	214994	197048	231652	224269	198050
Daniel Island Bend	10497	10497	10497	10497	10497	10497	10497	10497	10497
Clouter Creek Reach	0	1309*	0*	33243*	33759*	34227*	33501*	33763*	35126*
Navy Yard Reach	28726	29335	22445	22271	24441	21744	21520	22529	20324
North Charleston Reach	0	3175*	2540*	4075*	4191*	3138*	5104*	4191*	3138*
Filbin Creek Reach	6504	6504	6504	10883	10035	8199	10742	10035	8199
Port Terminal Reach	4436	5140	6661	14632	15132	15432	14581	15017	15316
Ordnance Reach	144535	141689	127008	165254	146684	118944	166433	145424	118639
Ordnance Reach Turn Basin	327444	291829	239695	530448	485936	405266	532713	486683	401246
Wando River Lower Reach	58177	61848	64294	67723	69325	68281	69984	72129	71229
Wando River Upper Reach & Terminal	93457	88175	75048	98954	96431	77475	101985	98875	81163
Wando River Turning Basin	85515	78607	65853	262293	235790	188815	263097	237261	186586
Tidewater Reach & Union Pier	21762	21429	18266	23250	22766	18515	20021	20042	16126
Custom House Reach	51353	51422	48407	41808	43797	43361	34047	36015	36372
Town Creek Lower Reach	212216	202375	161527	268535	259581	227935	235123	226753	199013
TOTAL INNER HARBOR	1452935	1400815	1216810	2156377	2058867	1771224	2171467	2059319	1773610

Note * actual model generated value was used as historical dredging does not occur in this area.

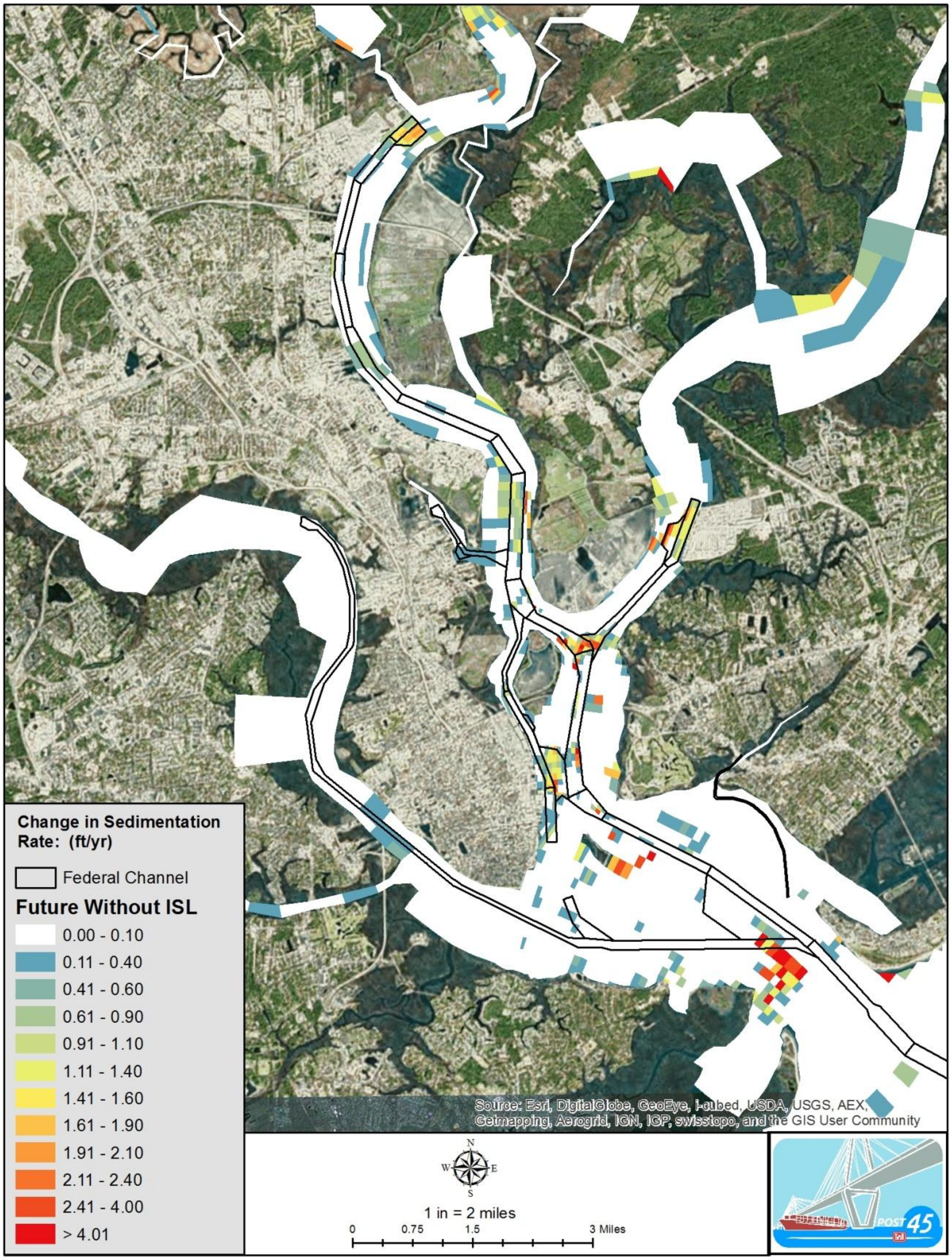


Figure 4.3.1 Intermediate Sea Level Change Shoaling FWO Conditions

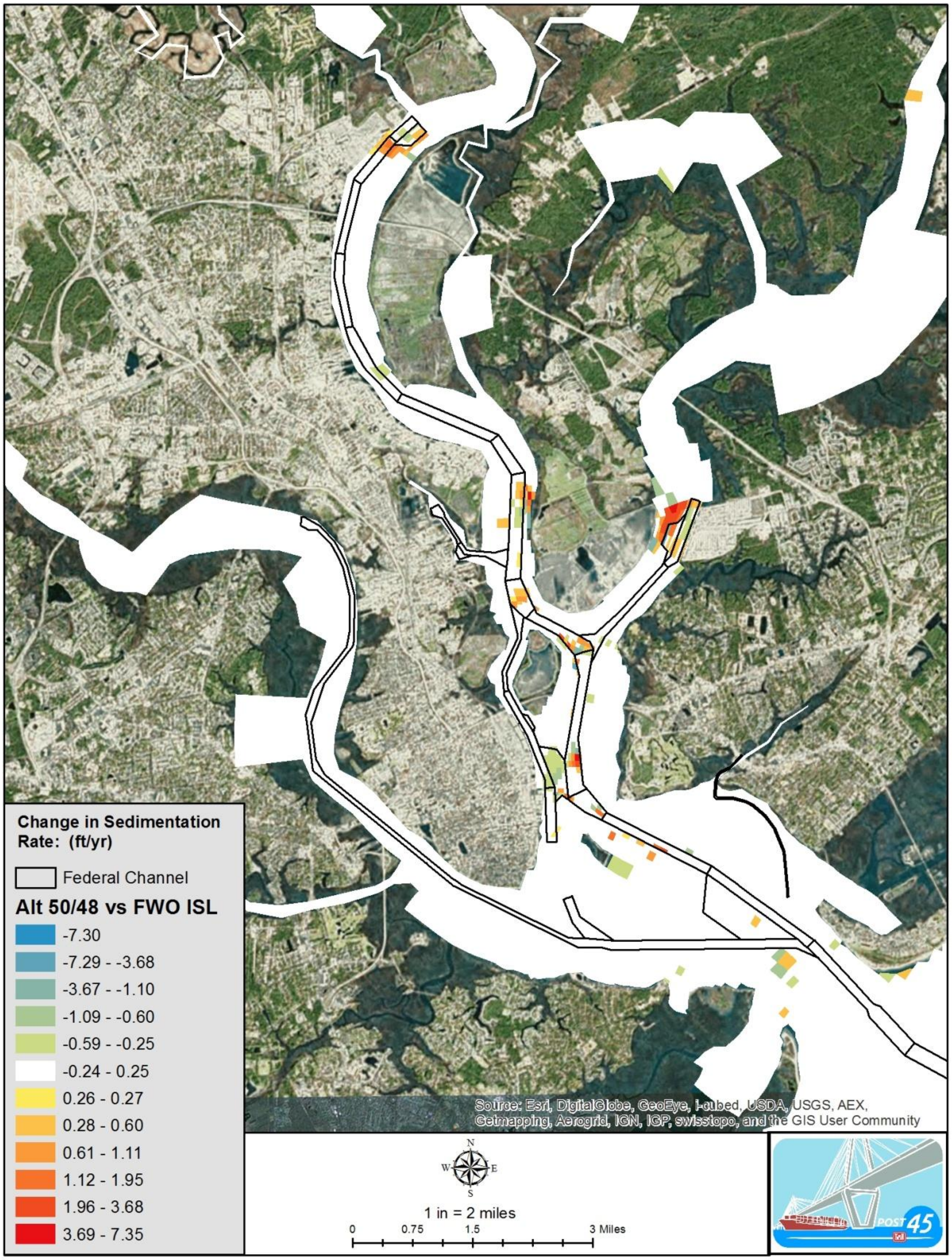


Figure 4.3.2 Intermediate Sea Level Change Shoaling 50-48 compared to FWO

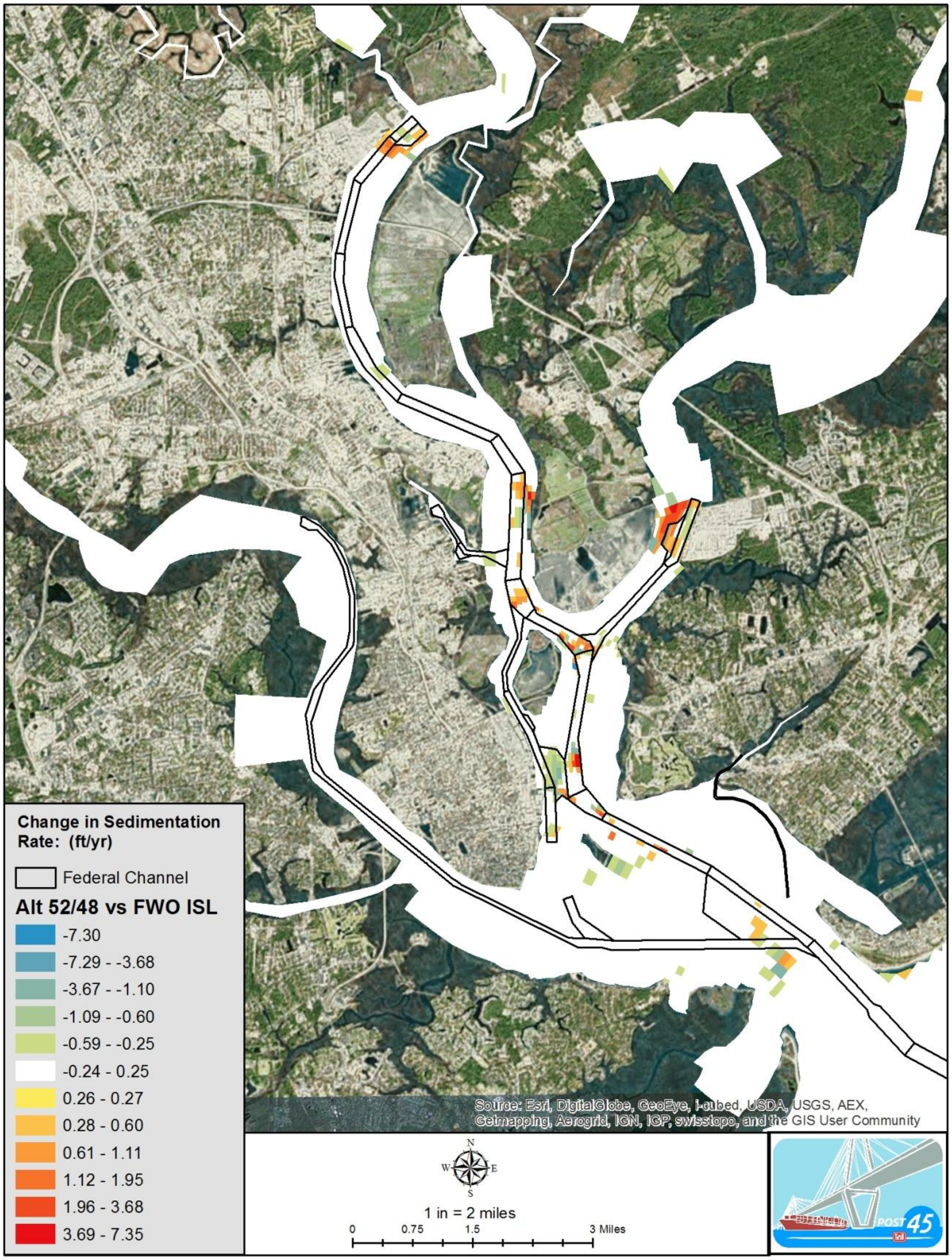


Figure 4.3.3 Intermediate Sea Level Change Shoaling 52-48 compared to FWO.

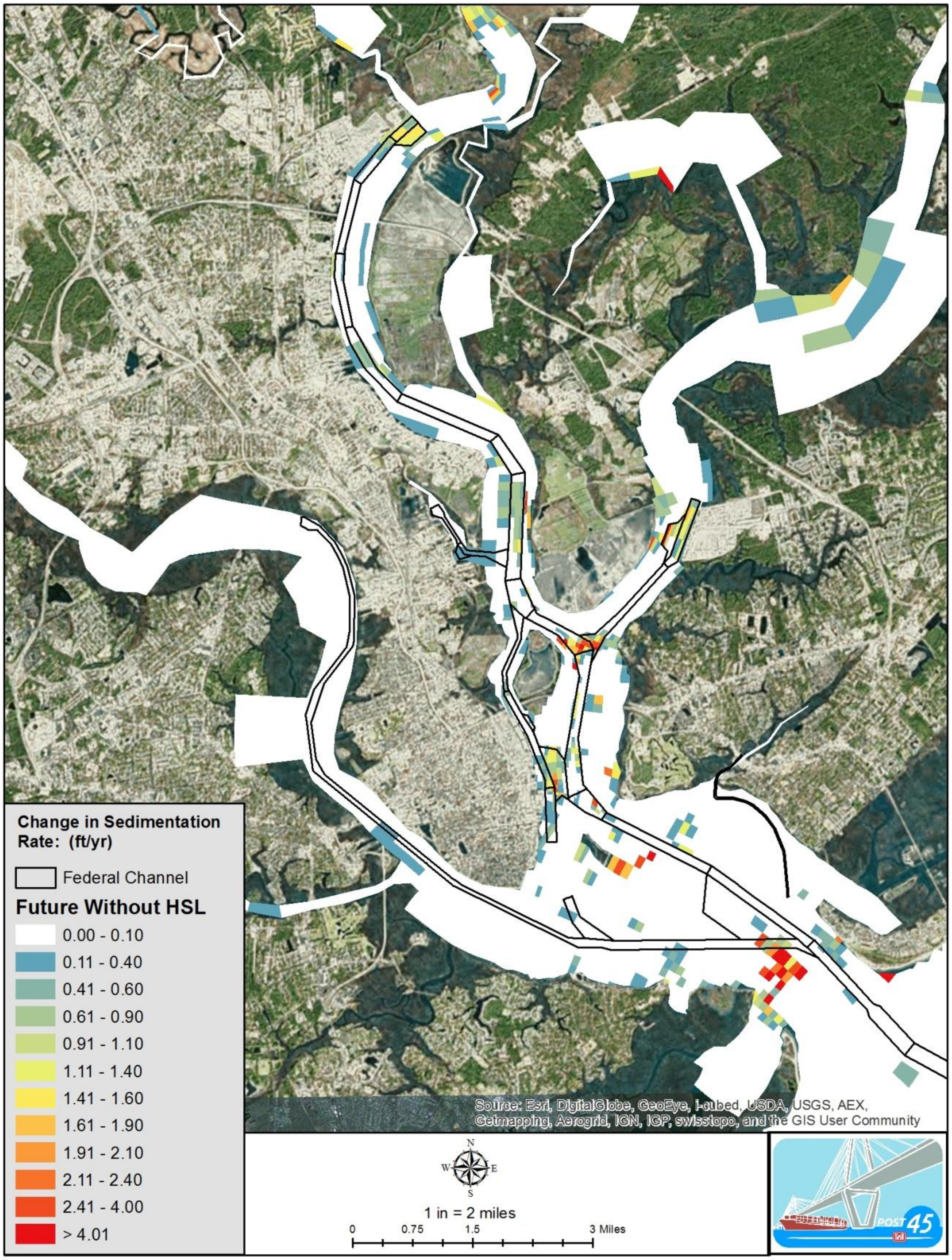


Figure 4.3.4 High Sea Level Change Shoaling FWO conditions,

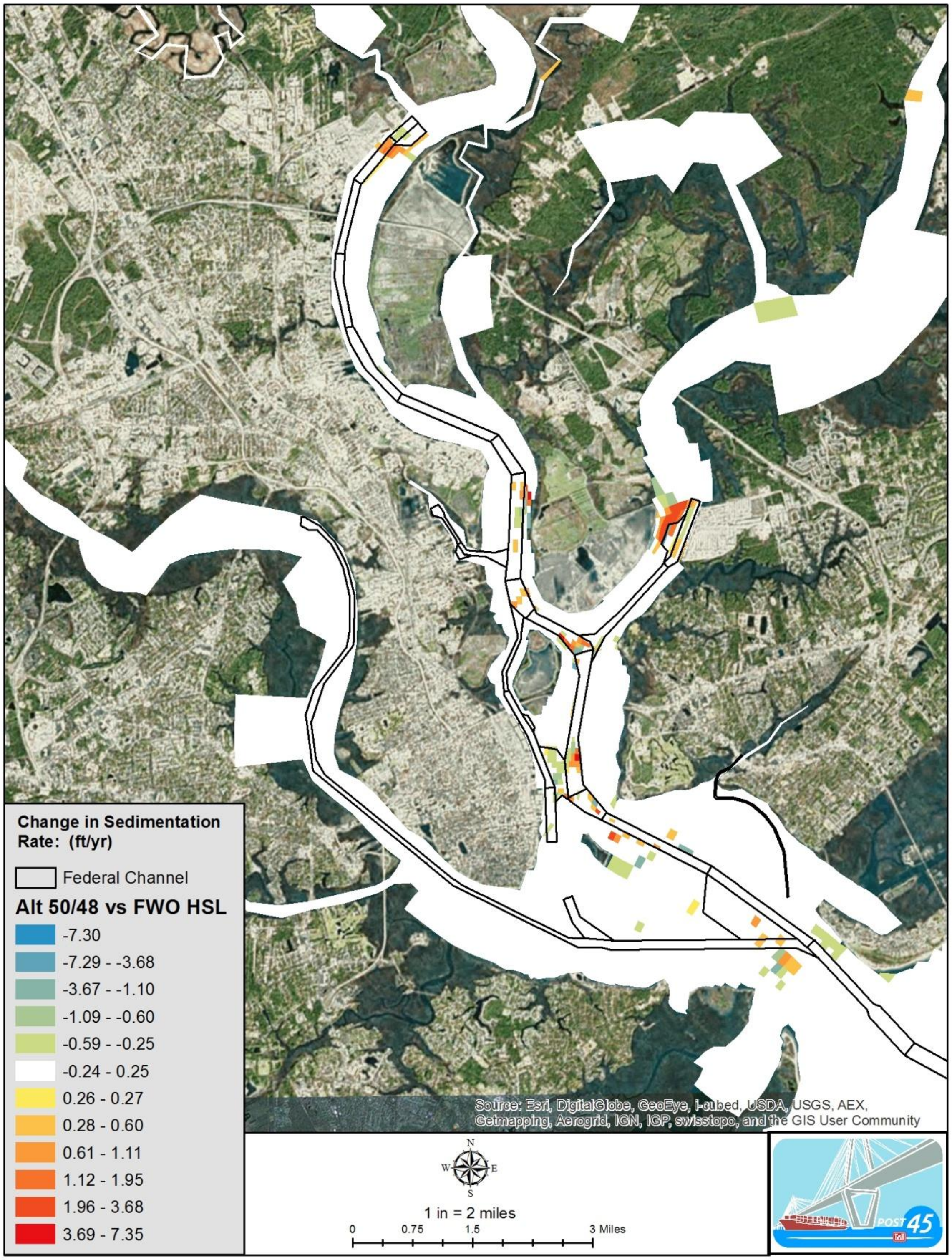


Figure 4.3.5 High Sea Level Change Shoaling 50-48 compared to FWO

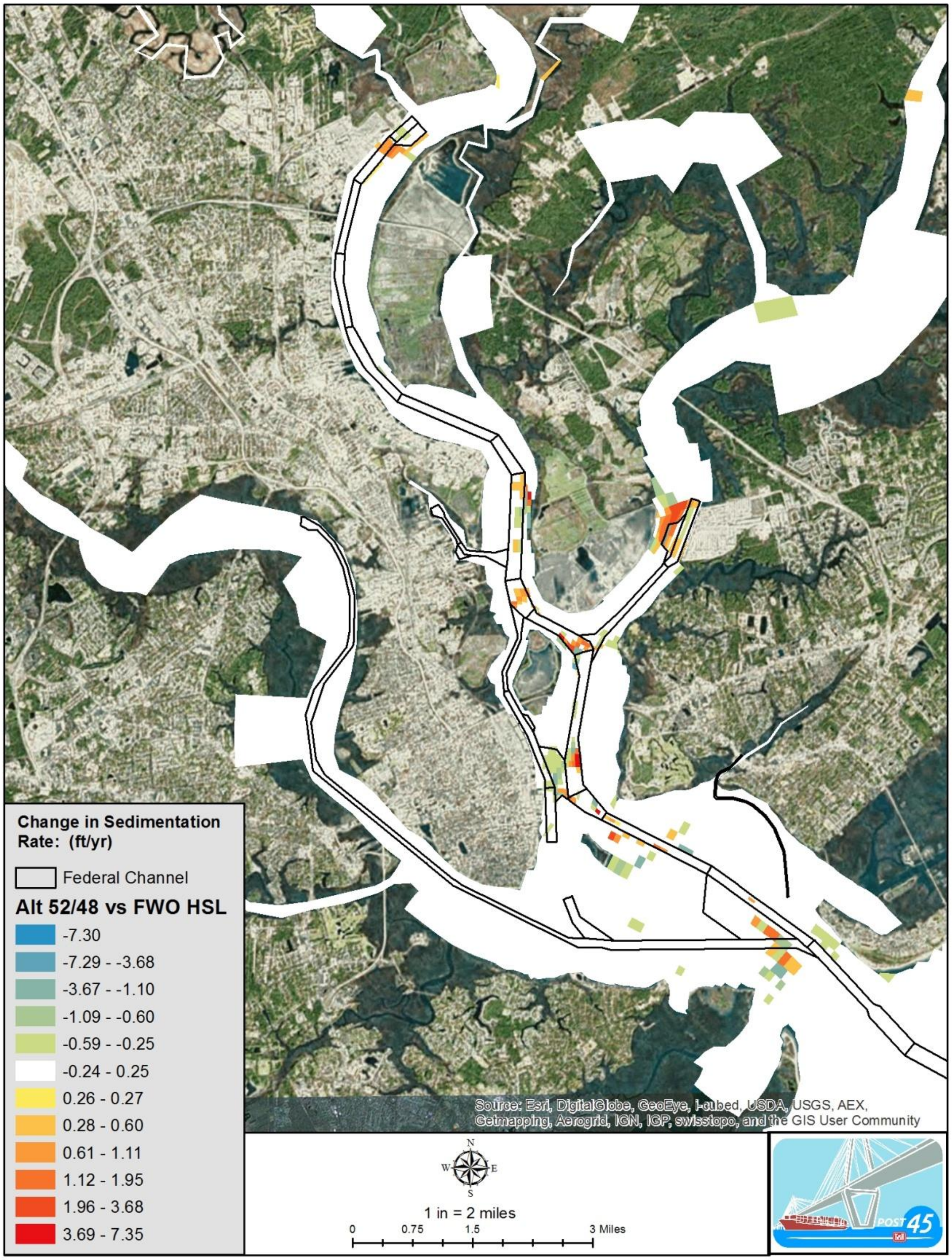


Figure 4.3.6 High Sea Level Change Shoaling 52-48 compared to FWO.

4.4 Salinity

For typical flow conditions, simulated water-column-averaged salinity values at selected USGS gage locations are listed in Table 4.4.1 for Future Without, 50-48 and 52-48 under historic, intermediate (ISL) and high (HSL) sea level changes. Future without project under ISL is shown in Figure 4.4.1 and for HSL in Figure 4.4.4. The change in annual average water column averaged salinity from the Future Without scenario to 50-48 alternative shown in Figure 4.4.2 for ISL and Figure 4.4.5 for high sea level. The change in annual average water column averaged salinity from the Future Without scenario to 52-48 alternative shown in Figure 4.4.3 for ISL and Figure 4.4.6 for high sea level. Note that figures with salinity change do not show salinity changes in the offshore region. The model is not calibrated to predict changes in this area, and the predicted changes are small (i.e., less than 1 percent change in salinity).

Comparison of percentiles indicate the changes in salinity at the intermediate and high sea level change are less in the lower harbor where salinity is already high. As expected, progression up the rivers is more dramatic at the high sea level rate change than the intermediate sea level change when compared to the historic rate. Future-Without project condition indicates the water at Goose Creek would become more brackish for both ISL and HSL but more severely under the HSL. Mobay is already brackish under existing conditions and both sea level change and proposed project could make it more brackish or even saltwater. However, recall the salinity alert system and tidal alerts would be expected to offset these impacts.

The results indicate the 50- year intermediate sea level change rate will increase salinity throughout most of the estuary and significant effects will extend upriver past the Mobay gage affecting the 1st percentile, indicating a change from brackish to saltwater. Impacts at Goose Creek are affected around the 90th percentile.

The results indicate the 50- year high sea level change rate will increase salinity throughout most of the estuary and significant effects will extend upriver past the Goose Creek gage affecting the 50th percentile. Impacts at Pimlico are only affected within the 99th percentile.

Higher sea level change projections will have higher impacts for a with-project condition than a without project condition. However, recall the model is not reactive and does not capture the releases that would accompany an alert. Therefore, the salinity alert system and tidal alerts would be expected to offset these impacts. Tidal alert responses are generally performed within the required weekly flow releases and do not result in additional weekly average flows.

If it appeared that the salinity alert system was no longer meeting the agreements to Bushy Park or became cost prohibitive, USACE would evaluate the relocation of the intake to Bushy Park Reservoir farther upstream and reassessment of the alert system.

Table 4.4.1 Salinity Percentiles due to Sea Level Change Rates

Percentile			USGS 02172020 West Branch Cooper River at Pimlico near Moncks Corner, SC						
Salinity	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile									
10th percentile									
50th percentile									
90th percentile			0.05			0.05			0.05
99th percentile	0.05	0.05	0.08	0.05	0.05	0.09	0.05	0.05	0.1
			USGS 02172050 Cooper River near Goose Creek, SC						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile									
10th percentile			0.05						0.05
50th percentile	0.05	0.05	0.12	0.05	0.05	0.17	0.05	0.05	0.19
90th percentile	0.35	0.83	2.83	0.54	1.11	3.17	0.6	1.19	3.29
99th percentile	2.05	2.93	5.48	2.44	3.32	5.88	2.58	3.48	6
			USGS 02172053 Cooper River at Mobay						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	0.12	0.25	1.76	0.17	0.43	2.02	0.19	0.46	2.1
10th percentile	1.76	2.47	4.3	2.16	2.86	4.61	2.23	2.99	4.72
50th percentile	6.92	7.57	9.58	7.62	8.22	10.16	7.81	8.41	10.32
90th percentile	10.91	11.5	13.44	11.66	12.17	13.93	11.89	12.37	14.12
99th percentile	13.2	13.69	15.59	13.86	14.34	16.1	14.09	14.56	16.28
			USGS 02172067.7 Cooper River at I-526						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	10.27	10.92	13.07	11.06	11.73	13.96	11.3	11.92	13.96
10th percentile	12.29	12.89	14.94	13.15	13.73	15.66	13.38	13.94	15.85
50th percentile	15.65	16.17	17.81	16.66	17.13	18.58	16.9	17.35	18.78
90th percentile	19.38	19.81	21.15	20.49	20.87	22.01	20.78	21.14	22.24
99th percentile	21.38	21.78	22.93	22.37	22.7	23.68	22.68	22.99	23.91
			USGS 02172069.8 Wando River above Mount Pleasant						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	19.48	19.86	21.13	21.49	21.82	22.78	21.86	22.16	23.09
10th percentile	20.49	20.79	21.92	22.34	22.58	23.48	22.7	22.93	23.77
50th percentile	22.13	22.46	23.54	23.78	24.02	24.88	24.13	24.35	25.17
90th percentile	24.63	24.92	25.94	25.59	25.85	26.75	25.87	26.13	26.99
99th percentile	26.11	26.36	27.26	26.91	27.13	27.98	27.16	27.36	28.18

Table 4.4.1 (continued) Salinity Changes due to Sea Level Change Rates

				USGS 02172070.9 Cooper River at Hwy 17					
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	20.97	21.31	22.68	21.66	21.97	23.16	21.85	22.18	23.31
10th percentile	22.62	22.96	24.22	23.21	23.53	24.66	23.39	23.71	24.79
50th percentile	25.73	26.07	27.11	26.07	26.38	24.66	26.23	26.53	27.48
90th percentile	28.47	28.7	29.48	28.68	28.92	29.63	28.77	29	29.7
99th percentile	30	30.12	30.6	30.12	30.24	30.72	30.2	30.32	30.8
			USGS 02172086.9 Ashley River near North Charleston						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	12.37	13.09	15.5	12.62	13.45	15.79	12.75	13.53	16.01
10th percentile	14.74	15.49	17.72	15.09	15.81	17.97	15.17	15.88	18.11
50th percentile	17.35	17.98	20	17.66	18.3	20.3	17.77	18.42	20.39
90th percentile	19.11	19.74	21.9	19.43	20.09	22.18	19.54	20.19	22.26
99th percentile	20.15	20.86	23.02	20.51	21.21	23.3	20.64	21.33	23.37
			USGS 02172100 Charleston Harbor at Fort Sumter						
	FWO	FWO ISL	FWO HSL	50-48	50-48 ISL	50-48 HSL	52-48	52-48 ISL	52-48 HSL
1st percentile	20.38	20.86	21.79	20.89	21.22	22.13	21.05	21.36	22.23
10th percentile	23.15	23.47	24.49	23.54	23.84	24.77	23.7	23.93	24.86
50th percentile	28.04	28.26	28.79	28.2	28.39	28.91	28.28	28.43	28.93
90th percentile	31.34	31.46	31.87	31.41	31.5	31.88	31.42	31.52	31.9
99th percentile	32.87	32.95	33.1	32.88	32.92	33.19	32.9	32.93	33.22

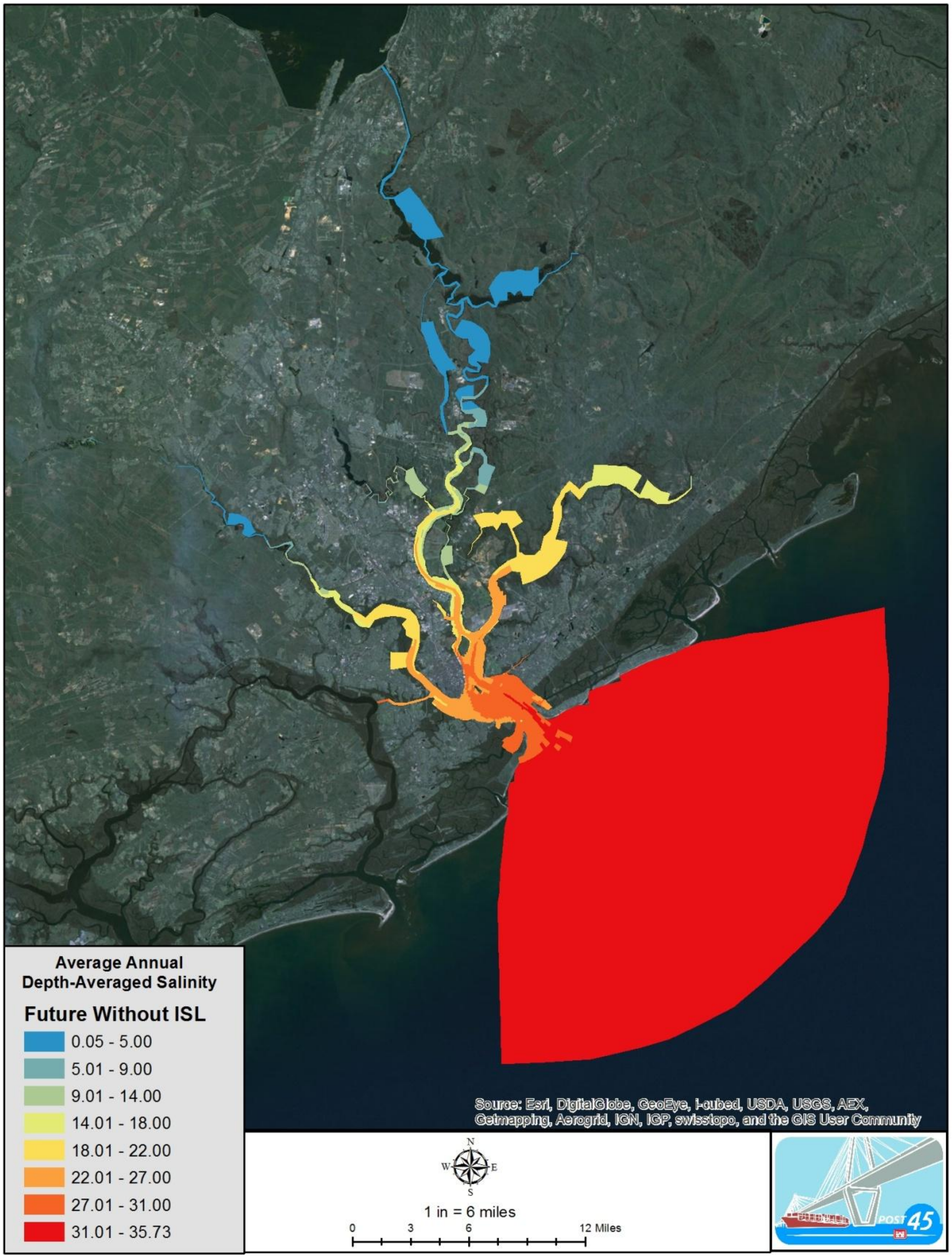


Figure 4.4.1 Intermediate Sea Level Change Salinity FWO conditions

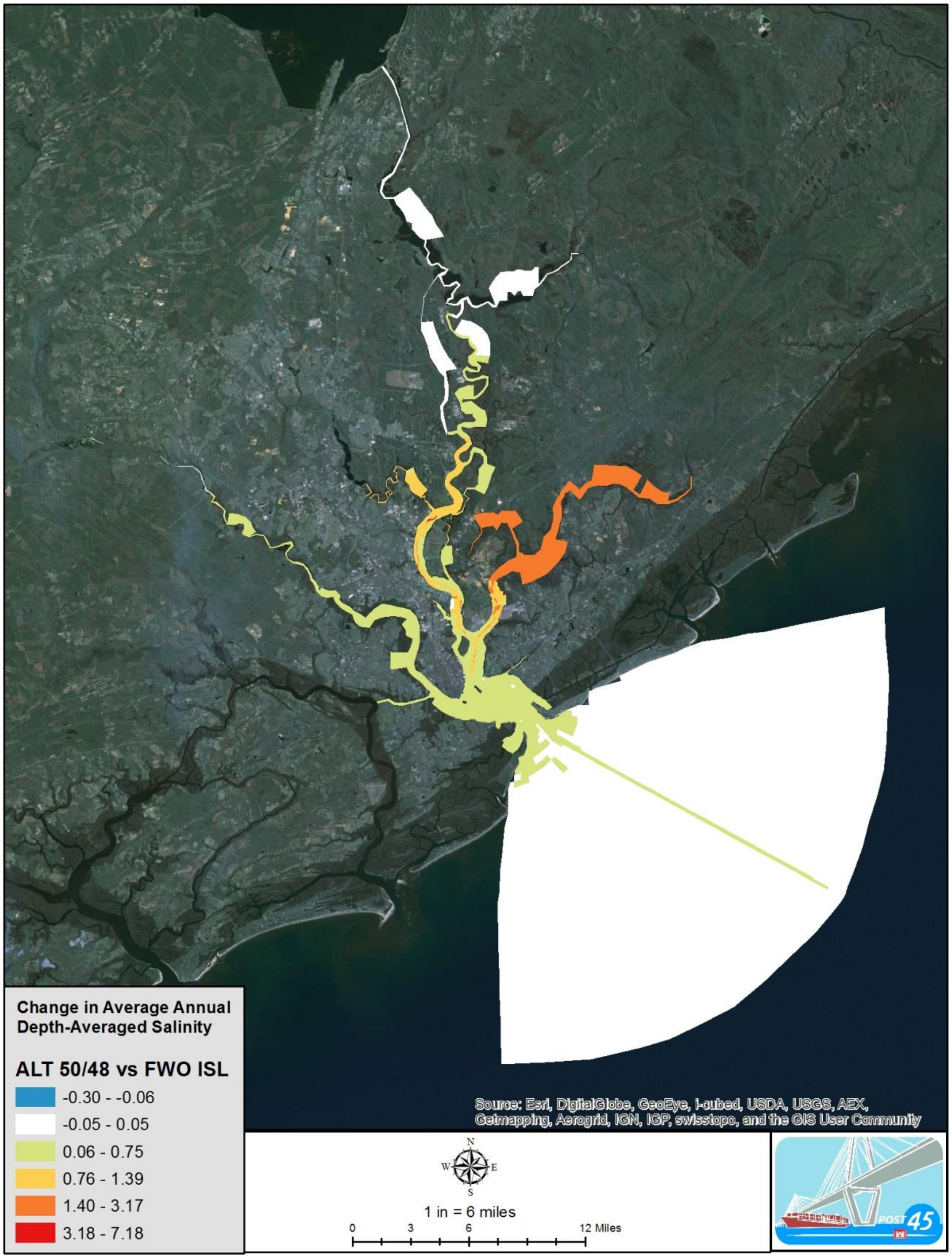


Figure 4.4.2 Intermediate Sea Level Change Salinity 50-48 compared to FWO

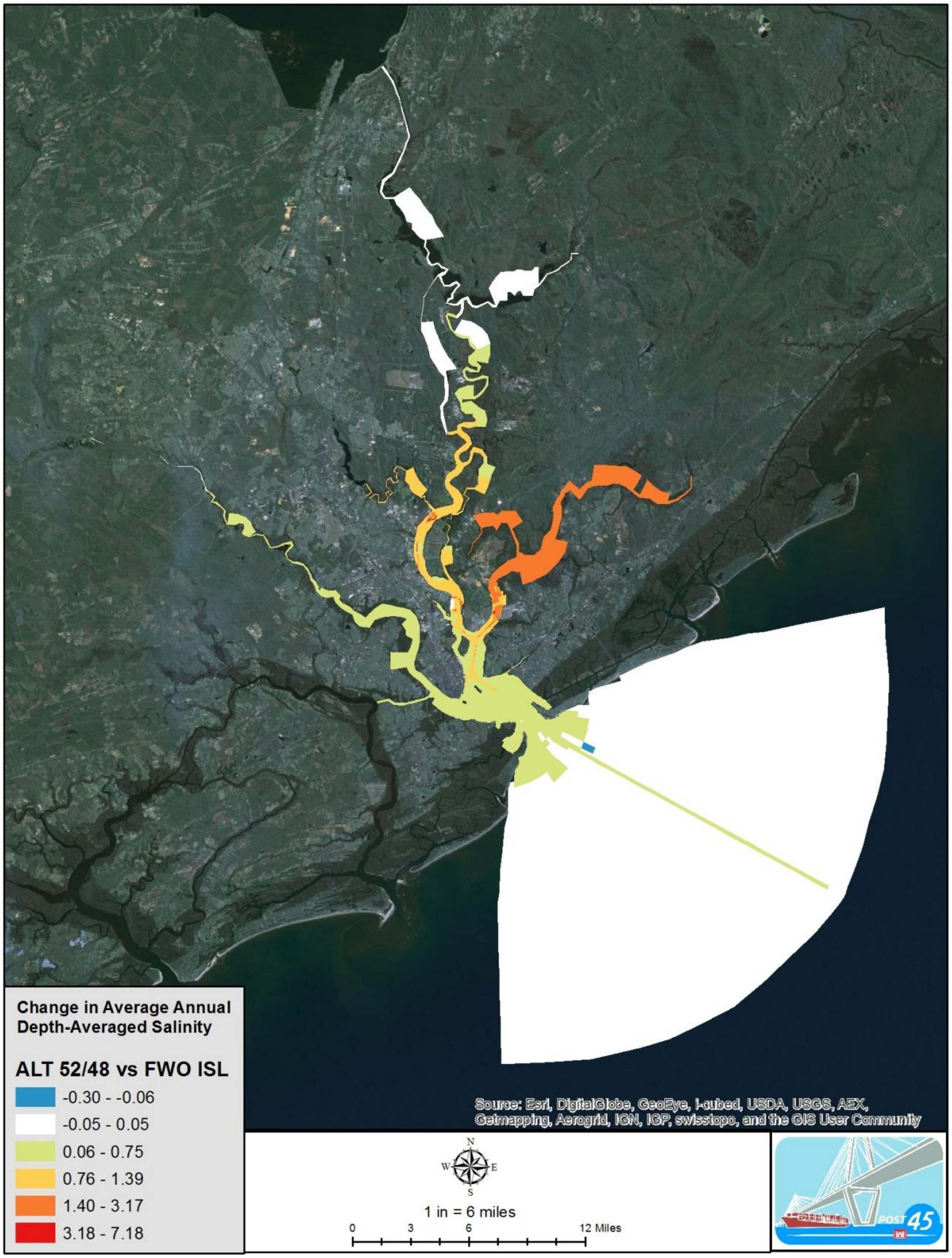


Figure 4.4.3 Intermediate Sea Level Change Salinity 50-48 compared to FWO.

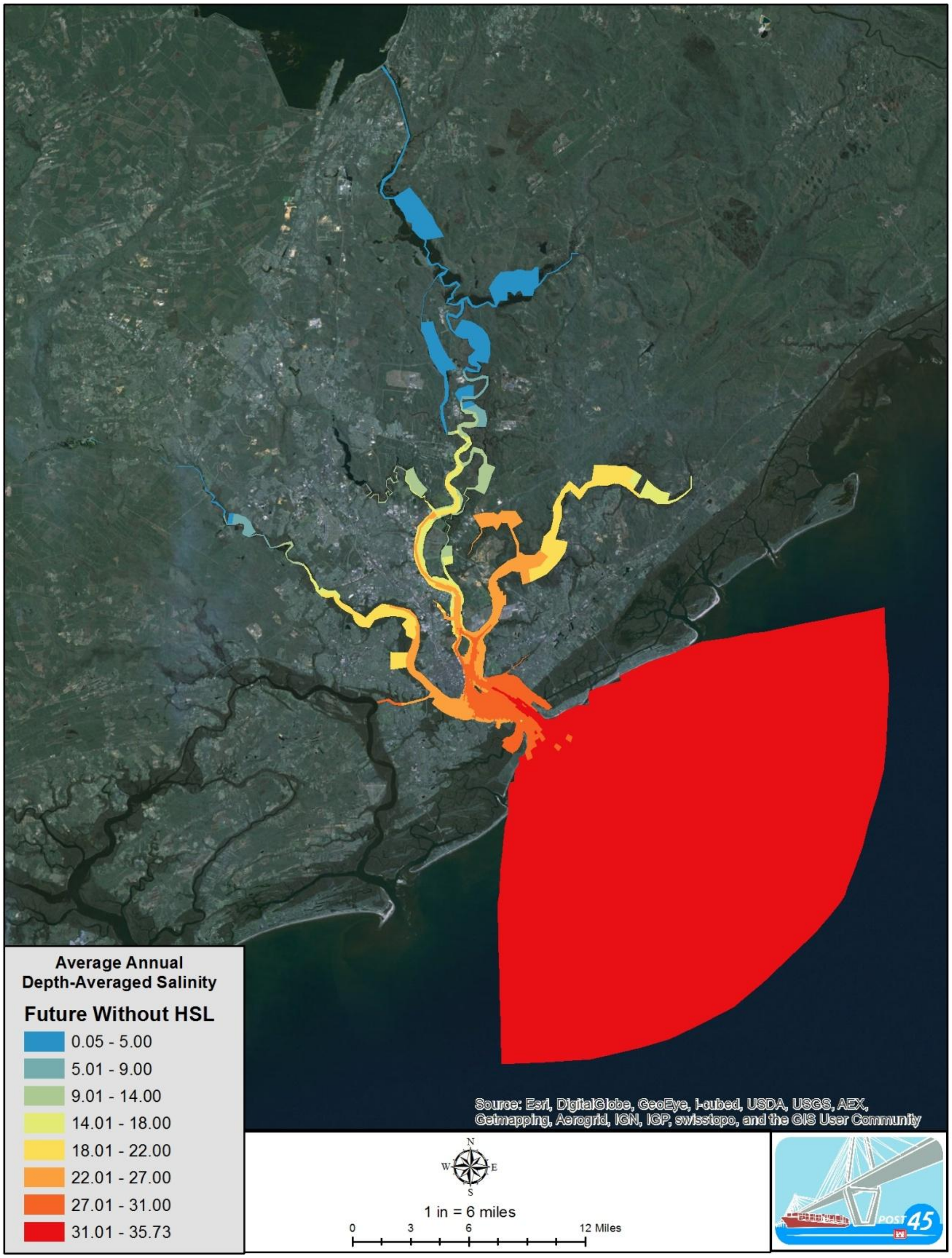


Figure 4.4.4 High Sea Level Change Salinity FWO conditions

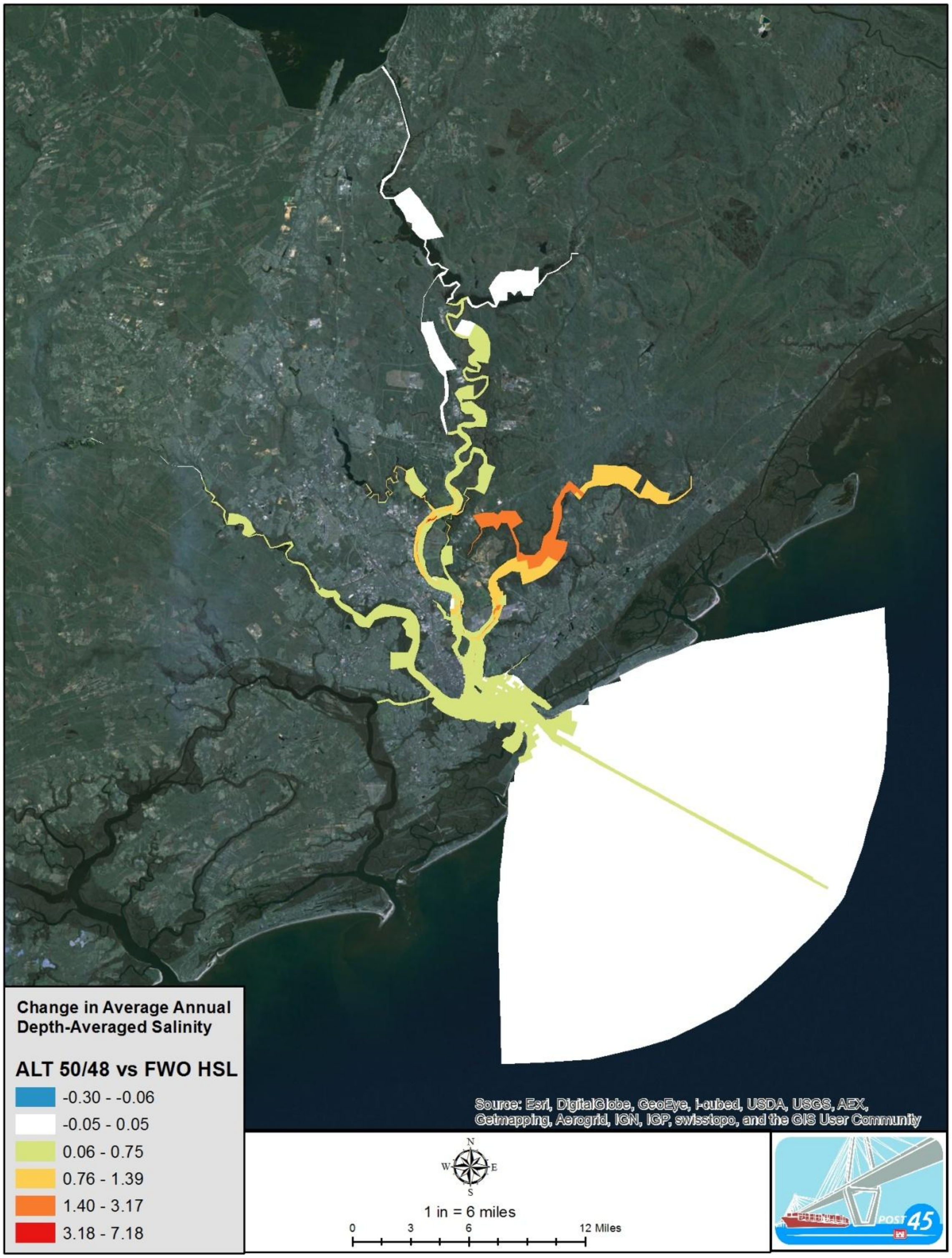


Figure 4.4.5 High Sea Level Change Salinity 50-48 compared to FWO

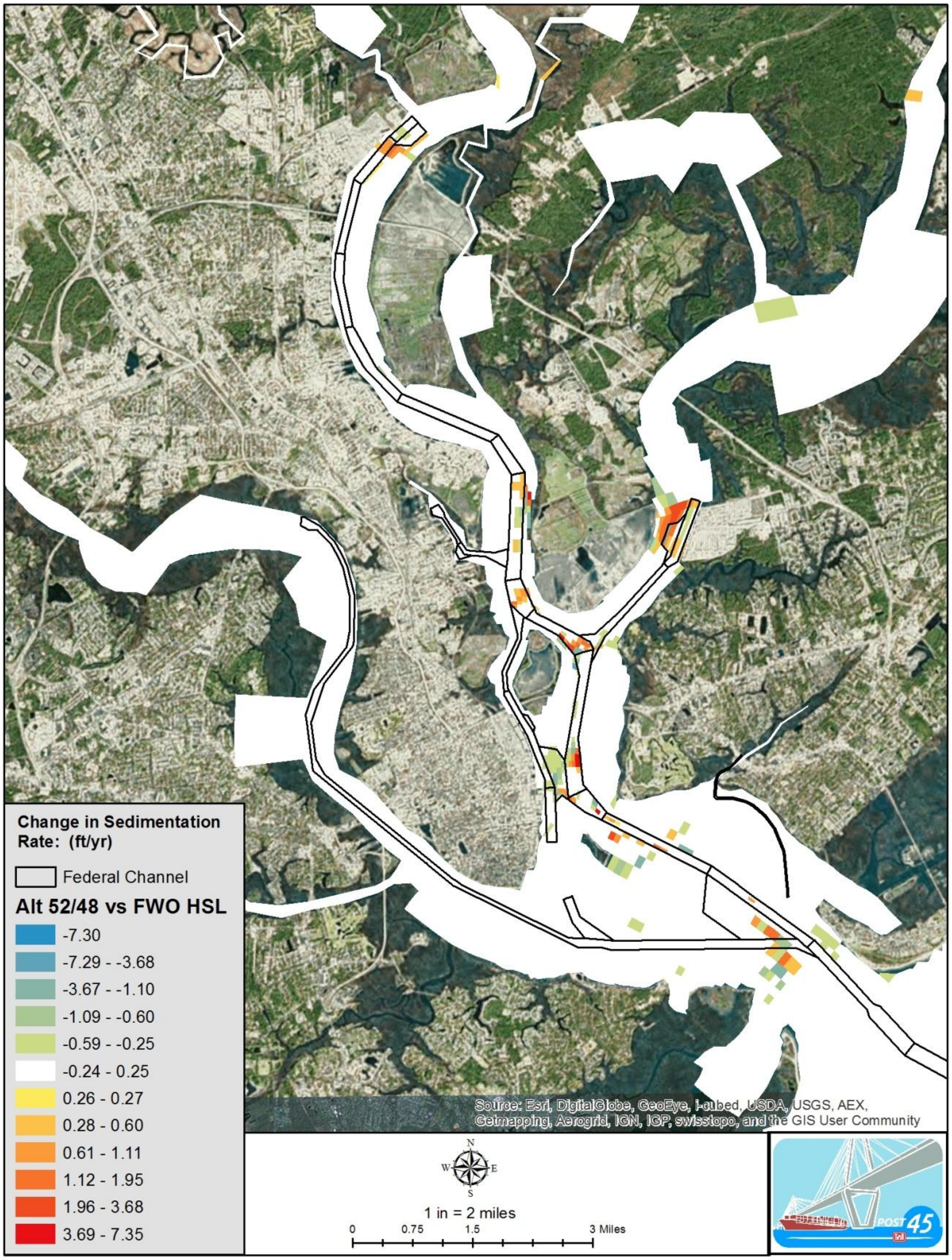


Figure 4.4.6 High Sea Level Change Salinity 52-48 compared to FWO.

5.0 Charleston Harbor Post 45 Feasibility Streamlined Climate Vulnerability Assessment

As part of the sea level change impact assessment, the District consulted with the Climate Preparedness and Resiliency Community of Practice Subject Matter Experts (SMEs), who prepared the tables and figures in this section. As stated previously in Section 2.2, using the online Sea-Level Change calculator (http://www.corpsclimate.us/ccaceslcurves.cfm), the sea level change trend at Charleston Harbor is estimated to be 2.94 mm/yr (0.116 inch/yr) based on the data from NOAA tide gauge #8665530 (period of record 1899-present). Estimating construction completion of 2021, and a 50 year project life, starting with 2012 (thus estimate the increase in 59 years) - the “low” rate of change is 0.57 feet, the “intermediate” is 1.08 feet and the “high” is 2.74 feet. At 2100, the changes are 1.12, 2.15 and 5.44 feet, respectively, for the low, intermediate, and high rate scenarios.

The assessment also included use of the NOAA Digital Coast tool (Marcy et al 2011) to visualize the projected regional impacts of sea level rise within the study area. These impacts are predicted for the future without (no action) project condition.

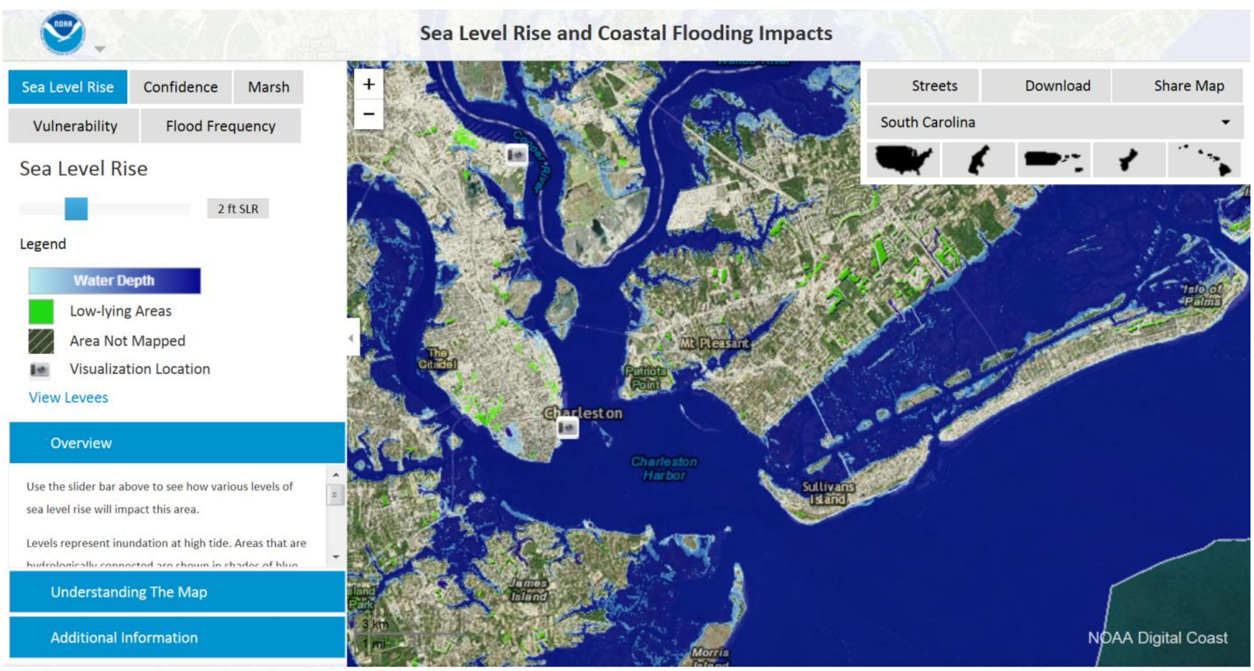


Figure 5.1 Plan View, Sea Level Change Map - 2 feet RSLR +MHHW

http://coast.noaa.gov/digitalcoast/tools/slr

5.1. Climate Preparedness and Resilience Register (CPRR) – Performance Impact Register

Assessment of climate impacts is facilitated by the use of impact or risk register tables. These tables identify separable elements of the navigation project whose function is necessary to the overall system function of the project. Lifecycle performance for the individual elements identified in these tables is assessed over the planning horizon of 100 years as prescribed by ER 1110-2-8159.

Individual elements for the Charleston Harbor project are;

- Dredged Material Containment Dikes
- Channel Contraction Dikes
- Coastal Structures (Jetties)
- Channel Depth

Additional elements for the Charleston Harbor project, which would impact performance or have environmental impacts, are:

- Bridge Crossings and low chords
- Wetland Mitigation Features

The critical project elements of the Charleston Harbor system are indicated in Figure 5.1.2. They include the containment dikes of the upland disposal areas, the contraction dikes located on the Cooper River near the new Navy Base Terminal, and the jetties at the entrance of the harbor. Additional elements of concern include the adjacent wetlands, bridge low chords and channel depths. Each of these project elements was assessed for robustness to expected changes in sea level by considering the critical performance elevation that acts as a threshold between full performance and decreased levels of performance, and the total water level components that control performance. These include whether the high or low extreme water level is the metric of interest, the critical tidal datum, the appropriate range of event frequencies, the dynamic component of interest, and when performance might be expected to be impacted for the various sea level scenarios.

The performance impact register identifies the water level or levels which control or impact performance, including dynamic components. The elevation at which performance is impacted is identified as a critical or controlling elevation. Its total water level signature, i.e., frequency of occurrence, and type, extreme/tidal are also identified in the performance impact register.

Relative Sea Level Change (RSLR) is applied to the total water level component by linear superposition for each of the three sea level change scenarios. Where the projected total water level intersects the critical or controlling elevation is the point where performance is impacted. This point will have a future time value associated with the total water level projection, and the associated future time is identified in the table.

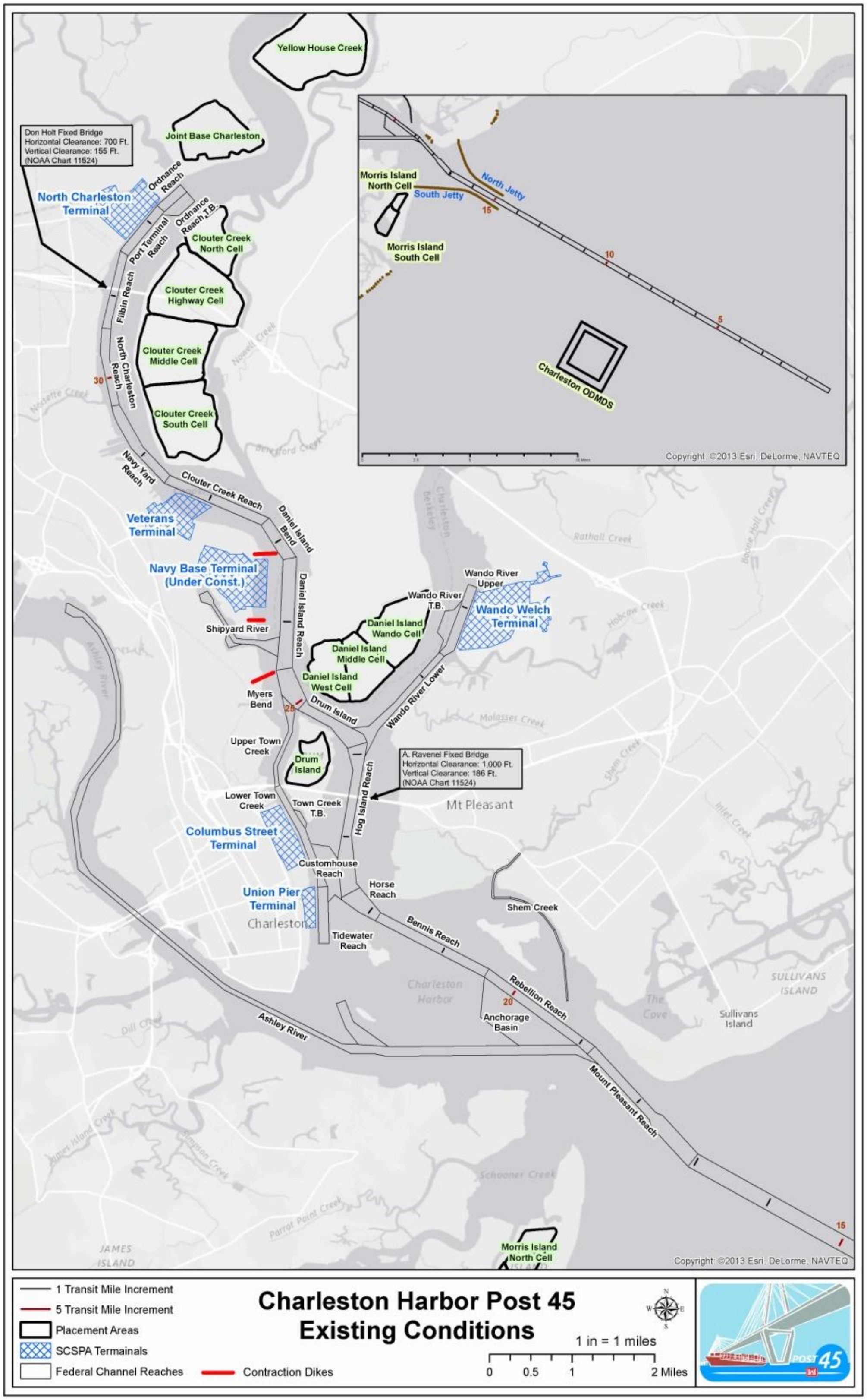


Figure 5.1.2 Map of Critical Project Elements, Relative Sea Level Rise

5.2. Tidal Datum.

A basic element of the climate preparedness and resilience register for any coastal project is the determination of tidal datum. Information from the benchmark sheet for NOAA tide gauge 8665530, located near the Customs House in Charleston SC, is shown in Table 5.1.2. Statistical data is provided in Figure 5.2.1 and 5.2.2.

Table 5.2.1. Tidal Datum NOAA Tide Gauge 8665530, Charleston SC near the Customs House (gage zero – MLLW) 7.7

Elevations of tidal datums referred to Mean Lower Low Water (MLLW),			
		meters	feet
HIGHEST OBSERVED WATER LEVEL (09/21/1989)		3.817	12.52
MEAN HIGHER HIGH WATER	MHHW	1.757	5.76
MEAN HIGH WATER	MHW	1.648	5.41
North American Vertical Datum	NAVD88	0.957	3.14
MEAN SEA LEVEL	MSL	0.891	2.92
MEAN TIDE LEVEL	MTL	0.853	2.80
MEAN LOW WATER	MLW	0.057	0.19
MEAN LOWER LOW WATER	MLLW	0	0.00
LOWEST OBSERVED WATER LEVEL (03/13/1993)		-1.245	-4.08
Based on North American Vertical Datum (NAVD88)			

Version of Data :	05/01/2014
ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.27 (ft)
MSL:	-0.22 (ft)
MLW:	-2.95 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
EWL Type:	NOAA GEV (NAVD88)
100 Yr:	6.90 (ft)
50 Yr:	6.32 (ft)
20 Yr:	5.67 (ft)
10 Yr:	5.26 (ft)
5 Yr:	4.90 (ft)
2 Yr:	4.47 (ft)
Yearly:	3.95 (ft)
Monthly:	NaN (ft)
From:	8/01/1900
To:	8/01/2013
Years of Record:	113

Print Gauge Datums & EWL

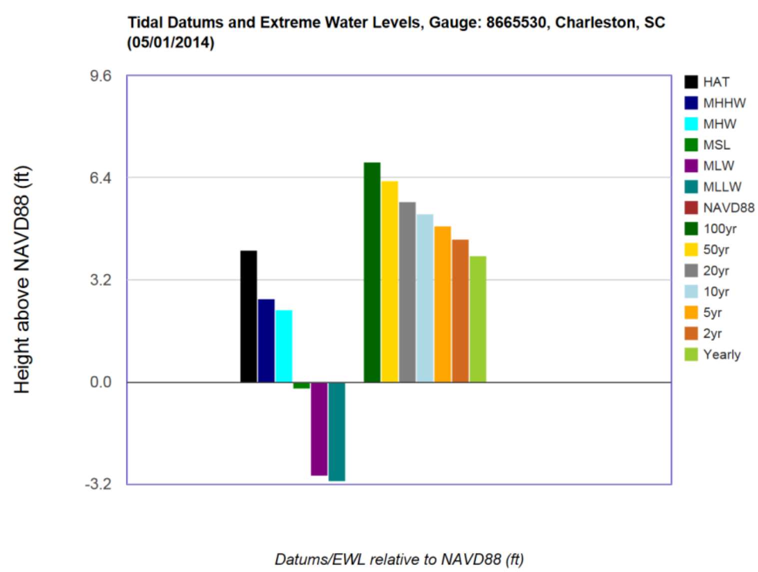


Figure 5.2.1 - Total Water Levels (NAVD 88) – Extreme High Water Levels Annual Exceedance Probability (AEP)

Version of Data :	05/01/2014
ID:	8665530
Reference Datum:	NAVD88
Name:	Charleston, SC
HAT:	4.12 (ft)
MHHW:	2.62 (ft)
MHW:	2.27 (ft)
MSL:	-0.22 (ft)
MLW:	-2.95 (ft)
MLLW:	-3.14 (ft)
NAVD88:	0.00 (ft)
EWL Type:	NOAA GEV (NAVD88)
100 Yr:	-6.69 (ft)
50 Yr:	-6.47 (ft)
20 Yr:	-6.16 (ft)
10 Yr:	-5.91 (ft)
5 Yr:	-5.65 (ft)
2 Yr:	-5.25 (ft)
Yearly:	-4.53 (ft)
Monthly:	NaN (ft)
From:	8/01/1900
To:	8/01/2013
Years of Record:	113

Print Gauge Datums & EWL

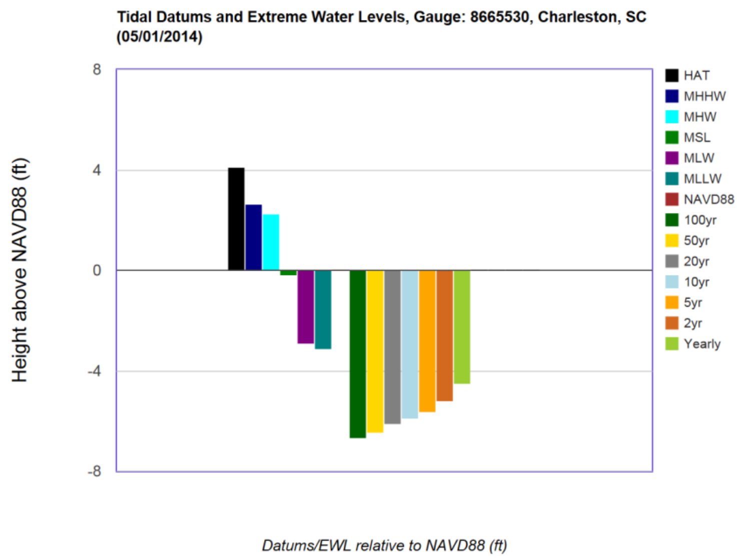


Figure 5.2.2 -Total Water Levels (NAVD 88) – Extreme Low Water Levels Annual Exceedance Probability (AEP)

5.3 Climate Preparedness and Resilience Register (CPRR) - Dredged Material Containment Dikes

Dredged material containment dikes are expected to perform under cases of extreme high water at MHHW for all frequencies of events, and should include the dynamic wave component. Based on the 1% annual exceedance probability still-water level, only one of the containment dike is expected to impacted, Yellow House Creek, and that is for the high scenario by 2095 (Table 5.3.1, Figure 5.3.1). This dike is robust to the low and intermediate scenarios. Adaptation measures developed now for the dike at Yellow House Creek can be put in place when appropriate as determined by monitoring sea level change. The other containment dikes are robust to sea level change in this analysis. The ability to raise the containment dikes of the upland confined disposal areas will mitigate the risk of overtopping. As part of normal maintenance of disposal areas, erosion and toe protection would be evaluated as needed.

Table 5.3.1 - Dredged material containment dike performance impact register.

Project Feature	Critical Perfomance Elev (NAVD88)	Controlling Performance TWL Extreme (High/Low)	Tide	Frequency (High, Low, Med)	High	Impacts Intermediate	Low
disposal area containment dikes (NAVD 88)							
Clouter South	28	High	MHHW	all			
Clouter Middle	20	High	MHHW	all			
Clouter Highway	16	High	MHHW	all			
Clouter North	20	High	MHHW	all			
Joint Base Charleston	22	High	MHHW	all			
Yellow House Creek	12	High	MHHW	all	2095 1% AEP		
Drum Island	25	High	MHHW	all			
Daniel Island West	28	High	MHHW	all			
Daniel Island Middle	27	High	MHHW	all			
Daniel Island Wando	25	High	MHHW	all			

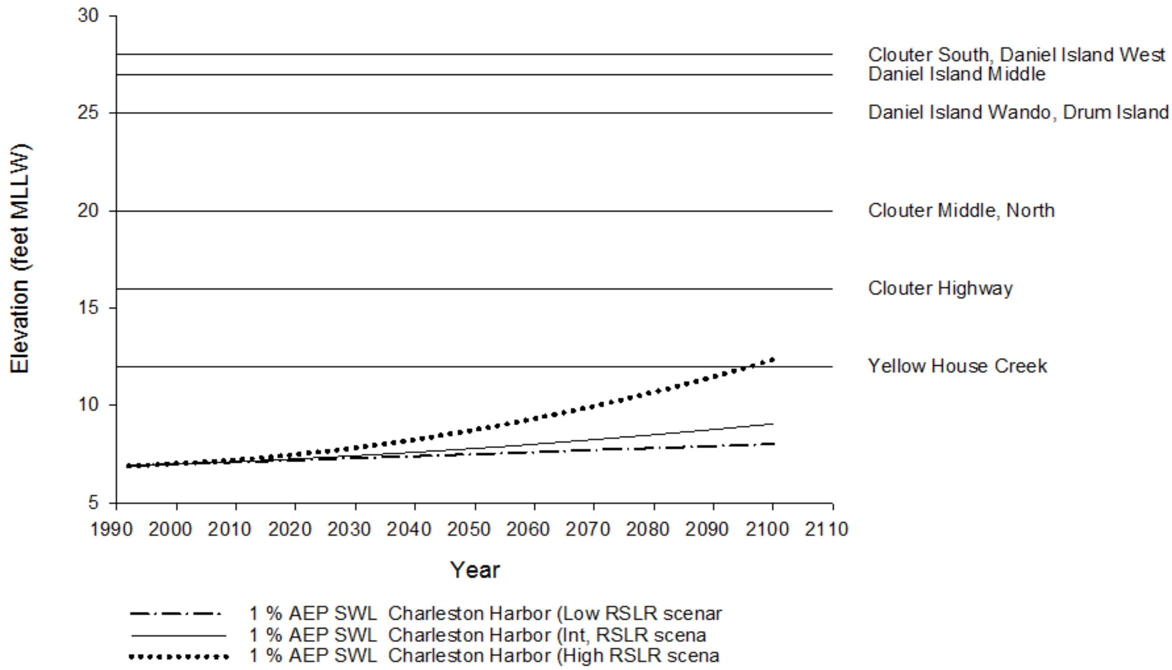


Figure 5.3.1 Critical performance thresholds for dredged material containment dikes. 1 % (High) AEP water level used to assess impact potential.

5.4 Climate Preparedness and Resilience Register (CPRR) Channel Contraction Dikes

Three contraction dikes were constructed in the Cooper River in 1959 to ameliorate shoaling. Contraction dikes are special cases of training dikes designed to focus flows in certain areas. Focusing the flow keeps the sediment in suspension and reduces shoaling. Two of the contraction dikes were constructed on the east and west banks just south of the Shipyard River entrance. The third contraction dike is located along the west side of Daniel Island Bend just upstream of Daniel Island reach and the new terminal. As part of the 1998 authorized project, the contraction dike on the east side was removed and relocated to the west side upstream of Shipyard River entrance, and the other two contraction dikes were rehabilitated to elevation 9.0 MLW. These dikes have been previously studied by ERDC and were shown to reduce shoaling in Daniel Island Reach by 50%. The contraction dikes are expected to perform at MHHW, so this is used as the metric of interest (Table 5.4.1, Figure 5.4.1). The contraction dikes are only at risk of overtopping after 2075 for the high rate of sea level rise. Adaptation measures developed now for the dike can be put in place when appropriate as determined by monitoring sea level change. Charleston District will assess the need for modification as part of normal operations and maintenance actions.

Table 5.4.1 - Channel contraction dike performance impact register.

Project Feature	Critical Perfomance Elev (ft MLLW)	Controlling Performance TWL Extreme (High/Low)	Tide	Frequency (High, Low, Med)	Impacts by SLC Scenario Impacts High	Intermediate	Low
river engineering (MLLW) channel contraction dikes	9		MHHW		2072	>2100	>2100

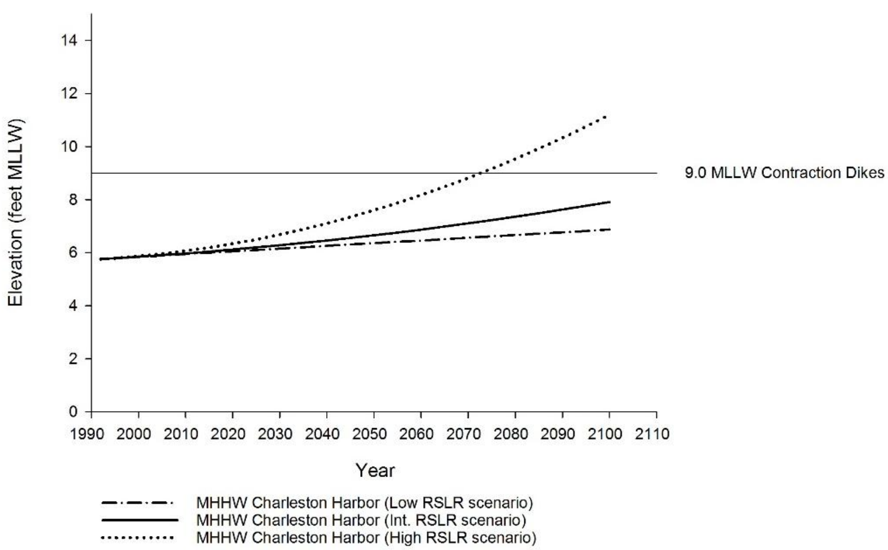


Figure5.4.1 - Critical performance thresholds for channel contraction dikes, Impact assessment based on current and future MHHW.

5.5 Climate Preparedness and Resilience Register (CPRR) - Coastal Structures (Jetties)

The jetties were constructed in the late 1800's and completed around 1895. The southern jetty springs from Morris Island and the northern jetty from Sullivan's Island, both curving toward each other with the convex side toward the channel (Fig 5.5.1). At a point about 9,000 ft. from Sullivan's Island and 14,000 ft from Morris Island the jetties straighten to a parallel about 2900 feet apart. The jetties channel the ebb tidal flow, flushing the channel. The problem of training the ebb tide without impeding the flood tide was solved by leaving the inner half of each jetty nearest land in the convex section, below the water surface acting as a weir. This allows the tide to come into the harbor. In the ebb tide, the curved portion channel the bottom current, while the straight half of the jetties (the parallel portions) channel the water trapped between them. These portions of the jetties were built to a higher elevation, so that for the last quarter of their length they rise above sea level. The intention of the design was that an exact amount of power would be exerted by the tide to keep the channel clear.

Figure 5.5.1 Charleston Harbor, 1968 showing weir-jetty structure and entrance channel.

Design literature shows the emergent portion of the jetties was constructed at around elevation +12 ft MLW (1895), which equates to approximately 11ft MLLW today. There has been significant subsidence in the last 100 years as well as an increase in RSLR. Subsidence of the jetties was documented by the Repair, Evaluation, Maintenance, and Rehabilitation Research Program in 1985 (USACE 1985). The relative position of both jetty reaches is shown in Figure 5.5.1. Documented changes in jetty elevation (USACE, 1985) are shown in Figure 5.5.2. The subsidence rates at the jetties effectively increase the RSLR rate to the range of 5 to 25 mm/yr.

The jetties are of varying elevations as described above. At high tide, portions of the emergent section of the north jetty are overtopped and have been for several years (earliest notation of submergence was on the 1999 NOAA Navigation chart 11521). Despite the overtopping, there has not been an increase in shoaling over that time period. Records indicate only 48,300 cubic yards (cy) have been dredged within the area since 1999, whereas, from 1992 to 1999, 102,000 cy were dredged. The last harbor deepening occurred 1999-2004. Since 1986, O&M dredging quantities for the reaches between the jetties total approximately 172,000 cubic yards (6200 cy/year average).

The Charleston Harbor entrance jetties reduce shoaling is based on their ability to flush the channel on an ebb tide, which it will continue to do with increased sea level. The jetties also block sediment transport from the littoral drift from the north. This capability is more a function of porosity of the jetty, rather than its height. Historical dredging records indicate that the porosity of the jetty is sufficient to function as intended. Many factors could affect dredging requirements in the future, including RSLR.

Jetties at an entrance channel can also provide some improvement to maneuverability. In these cases, performance impacts to maneuverability would be expected to occur at or above MHHW. Local pilots report no significant issues with surface cross currents due to jetty submergence (Charleston Branch Pilots Association, 2015). They do note that cross currents do not flow entirely across the inlet along the emergent portions of the jetty because the south jetty remains visible at the current time, except for the most extreme high tides (approximately the 50% AEP water level. Since there is roughly 2 foot elevation differential between the north and south jetties, with the north jetty average elevation currently estimated at MHHW, open water conditions across the entrance channel will not occur frequently for some time yet. The rate of subsidence at the jetties will be the controlling factor as to when this condition might occur. Further, it is not known how storm and wave climate offshore will interact with subsiding jetties as the relative tidal envelope of RSLR shifts upward in the future.

Table 5.5.1 - Jetty performance impact register.

Project Feature	Critical Perfomance Elev (ft MLLW)	Controlling Performance TWL Extreme (High/Low)	Tide	Frequency (High, Low, Med)	High	Impacts Intermediate	Low
coastal structures (MLLW)							
North Jetty	5.76		MHHW	>99.9 %	2015	2015	2015
South Jetty	7.76	High		> 50%	2015	2015	2015

The North and South jetties have approximately a 2 foot difference, and the elevations are estimated. Currently, both jetties are submerged at elevations above 7.76 feet MLLW (4.62 feet NAVD88), corresponding to roughly a 50% AEP water level.

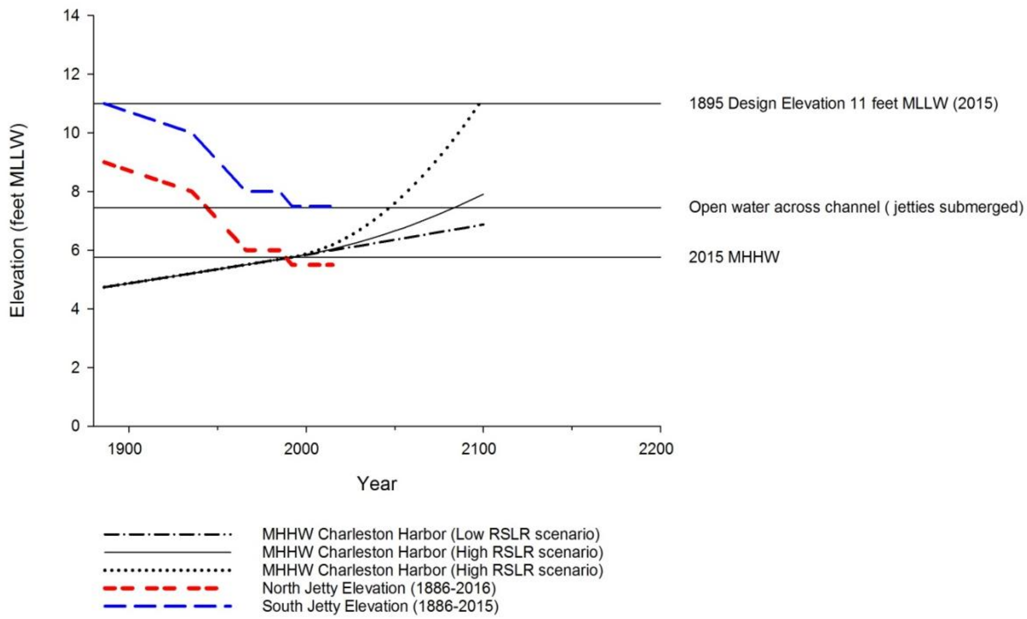


Figure 5.5.2 Critical Performance Impact analysis for jetties, showing historical elevation of jetties versus MHHW (jetty information from USACE 1985).

Figure 5.5.3 shows the existing condition of the entrance channel within the jetties and cross- sections across the jetties and entrance channel. Figure 5.5.4 shows the proposed project condition of the entrance channel within the jetties and cross-section across the jetties and entrance channel. The cross- sections are shown in an exaggerated 10V:1H scale, as well as, a 1V:1H scale in order to demonstrate how far away the existing channel toes or side slopes are from the jetties. The 1V:1H scale also demonstrates the nominal change the proposed project has on the system.

The District will continue to coordinate with subject matter experts (SME's) on the likelihood/probability of potential future jetty submergence adversely impacting commercial navigation. If the SMEs and District conclude there is sufficient risk and consequence, analysis could include determination of the jetty elevations at the current time, evaluation of estimated maneuverability performance at both low and high tide levels expected in the future, and identification of alternative adaptation measures, should they be required over the planning horizon.

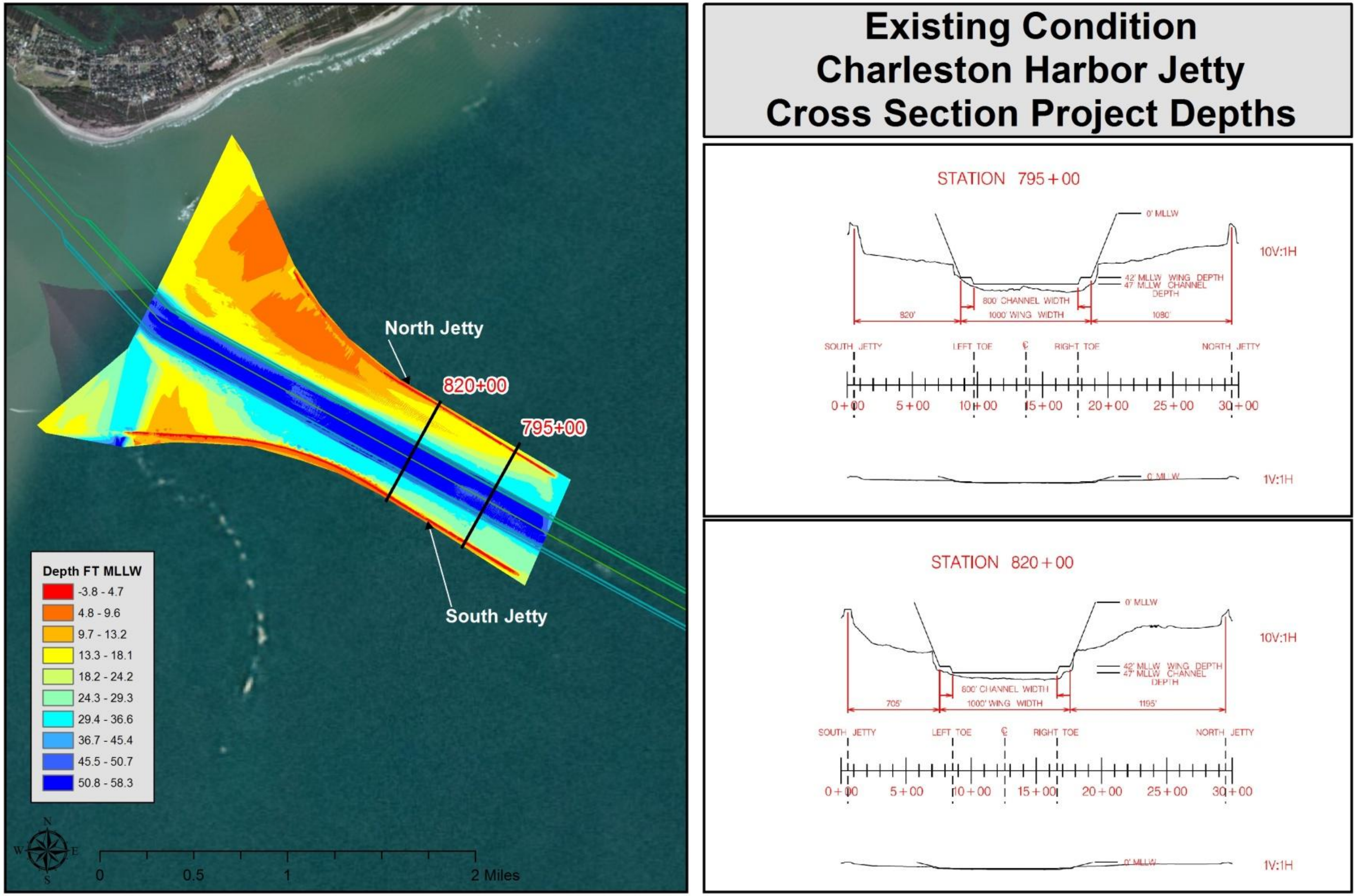


Figure 5.5.3 Existing Condition Charleston Harbor Jetty Cross-Section Project Depths

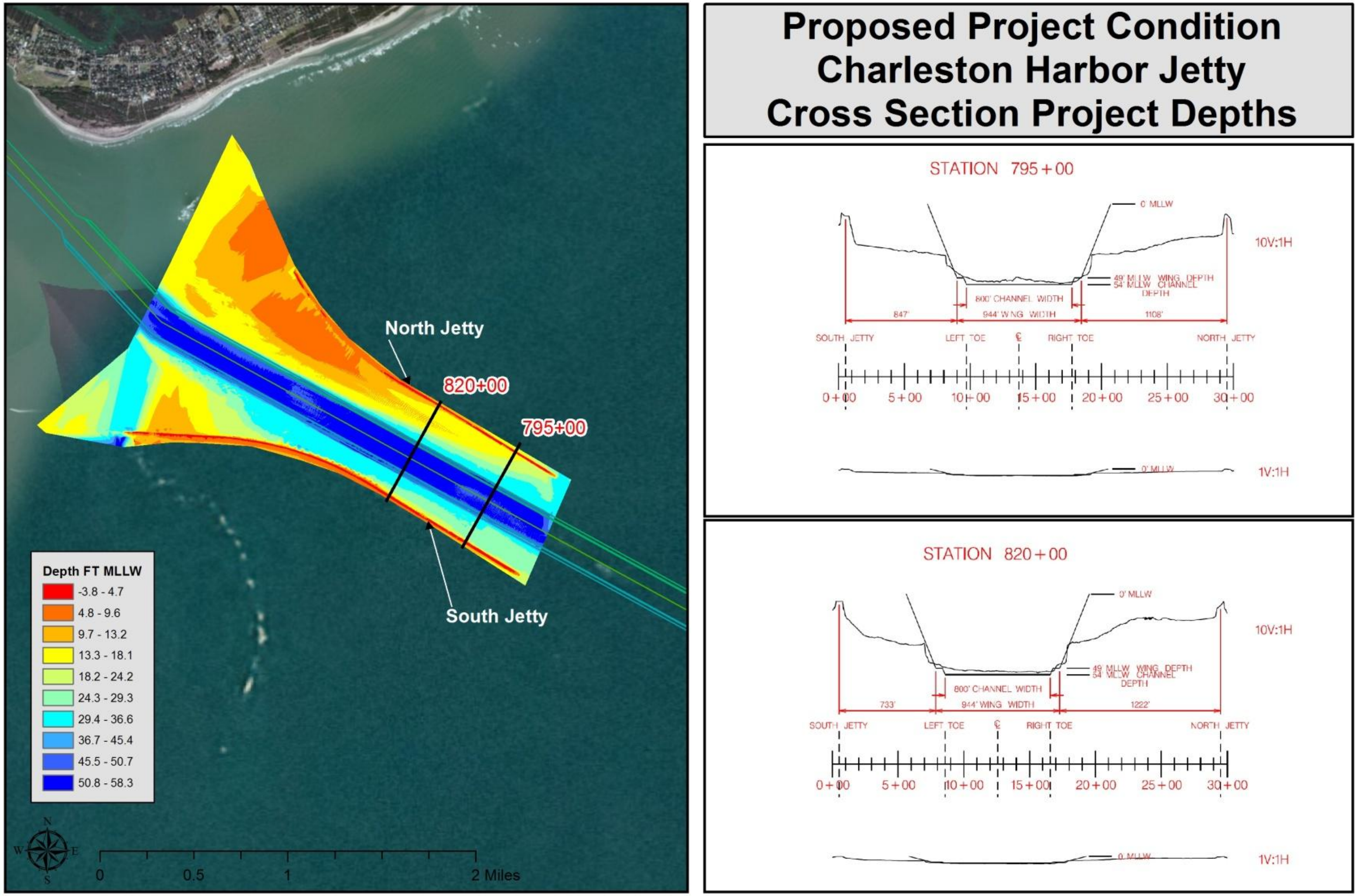


Figure 5.5.4 Proposed Project Condition Charleston Harbor Jetty Cross-Section Project Depths

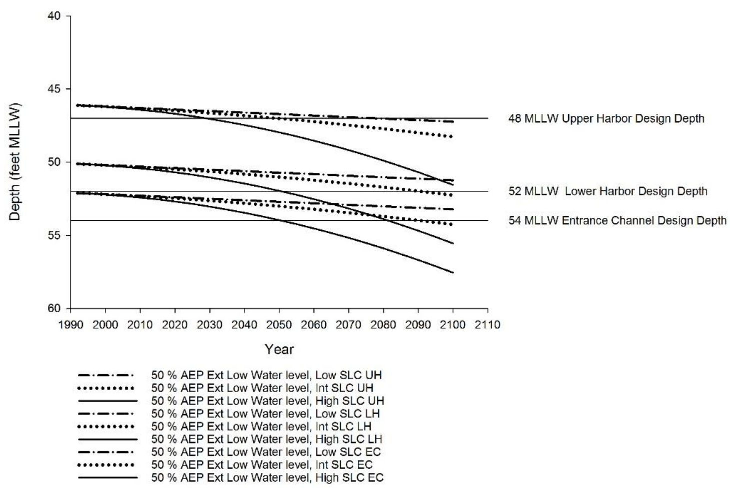
5.6 Climate Preparedness and Resilience Register (CPRR) - Channel Depth

Under present conditions, navigation of large container ships is dependent upon tide, and the controlling performance water level is MHHW. Under low water, channel design depth may not be adequate at all times, potentially resulting in brief closures to navigation. Monitoring will required to provide advice to shipping. This risk should decrease over time as RSLR increases. In general terms, any increase in sea level will increase the reliability of the navigation channel by increasing relative depth under all tidal and extreme low water conditions. The proposed project will improve the reliabilty of the federal channel under all sea level rates, such that they are no longer tide restricted.

Two AEP extreme low water levels were evaluated for impacts to channel depth, 50% and 1 % events (Table 5.6.1). The evaluation shows that now and in the future, the reliability of the channel for navigation will continue to be impacted by extreme low water events, with impacts receding more rapidly for the higher frequency extremes, such as the 50% AEP under the range of SLC scenarios. Exposure to rare or lower frequency AEPs such as the 1% will persist through the end of the century under low and intermediate RSLR, and to 2060-2076 under a high RSLR scenario. As RSLR increases, performance impacts associated with channel depth should decrease (Figure 5.6.1).

Table 5.6.1- Channel depth performance impact register

Project Feature	Critical Perfomance Elev (ft MLLW)	Controlling Performance TWL Extreme (High/Low)	Tide	Frequency (High, Low, Med)	Impacts by SLC Scenario High	Intermediate	Low
navigation channel (MLLW)							
Channel Depth operational (lower harbor)	-52	Low	MLLW	50%	2015-2050	2015-2090	2015-2100
Channel Depth operational (upper harbor)	-48	Low	MLLW	50%	2015-2030	2015-2050	2015-2075
Channel Depth operational (entrance ch - jetties)	-54	Low	MLLW	50%	2015-2050	2015-2090	2015-2100
Channel Depth operational (lower harbor)	-52	Low	MLLW	1%	2015-2075	2015-2100	2015-2100
Channel Depth operational (upper harbor)	-48	Low	MLLW	1%	2015-2062	2015-2100	2015-2100
Channel Depth operational (entrance ch - jetties)	-54	Low	MLLW	1%	2015-2076	2015-2100	2015-2100



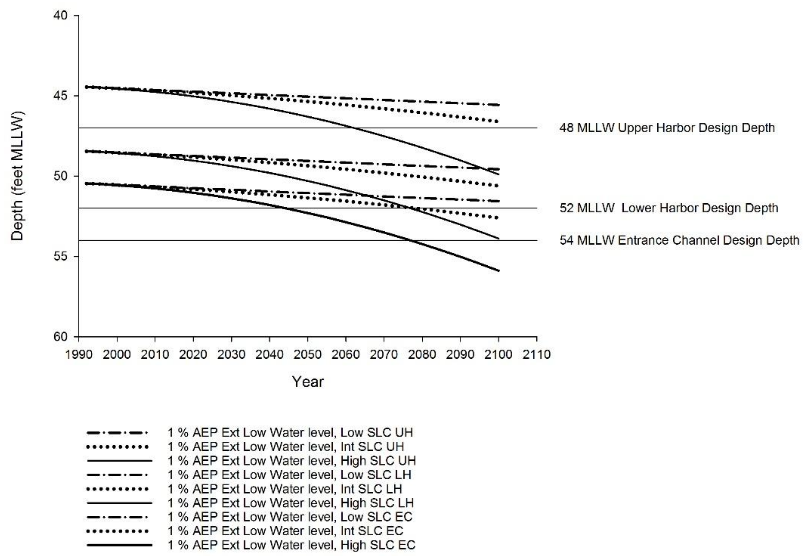


Figure 5.6.1 Evaluation of Channel Depth Design against 50% and 1% AEP extreme low water levels.

5.7 Climate Preparedness and Resilience Register (CPRR) - Critical Infrastructure, Bridge Crossings

Bridges over the federal channel are not impacted by sea level rise, because low chords are well above the navigation channel (Table 5.7.1).

Table 5.7.1 Critical Infrastructure performance impact register

Project Feature	Critical Perfomance Elev (NAVD88)	Controlling Performance TWL Extreme (High/Low)	Tide	Frequency (High, Low, Med)	Impacts by SLC Scenario High	Intermediate	Low
Bridge Crossings (NAVD 88)							
Ravenel bridge (low chord)	186	High	MHHW	all	none	none	none
Don Holt (low chord)	155	High	MHHW	all	none	none	none

5.8 Climate Preparedness and Resilience Register (CPRR) – Wetland Mitigation

Section 2.3 of the Environmental Section describes in more detail how the impacts of the proposed project affect wetlands mitigation. In order to determine the project impact under different sea level rise scenarios, impacts were calculated under four different sea level rise and project scenarios:

1. Impacts at the time of construction based on 10 years of sea level change from the model base year (year 2022)
2. 50 years of historic (low) RSLR.
3. 50 years of intermediate RSLR.
4. 50 years of high RSLR.

Each scenario was anticipated to result in a different baseline isopleth (e.g., location in the river) for the without project condition, and therefore the modeled project impacts were anticipated to be variable and dependent on the location in the river where the baseline isopleth occurs. The first four scenarios were then averaged to determine the average amount of impacts anticipated to occur based on variable baseline conditions. It should be noted that it is important to use the 2022 scenario in the determination because it represents the impacts anticipated to occur at the time of construction and helps bracket the results. This analysis includes the use of a model predicting salinity movement in a tidal system to help project impacts. For details on the analysis and the results, please see Appendix L. The results do not indicate any project sensitivity to the rate of sea level change.

Since wetland mitgation acres may be obtained outside the immediate project area, the project will be able to meet its mitigation requirement regardless of sea level rise impacts to the present mitigation plan.

5.9 Climate Preparedness and Resilience Register (CPRR) – Recommendation

Based on this CPR CoP review, the Charleston Harbor project is relatively robust to changing sea levels in the future. Uncertaninties associated with the stability of the entrance channel jetties indicate that there is potential for required adaptation in the future. Based on available evidence, the probability is about 5% that the jetties may require restoration. Therefore, the potential for this adaptation should be included as a line item in the cost and schedule risk analysis, with order of magnitude estimates from similar East Coastal and Gulf Coast jetty restorations.

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Attachment A-2 2004 Hydrodynamic Model Validation Plots <http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_2_2004_Hydrodynamic_Model_Validation_Plots.pdf>

Attachment A-3 Suspended Sediment Concentration Plots <http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_3_Suspended_Sediment_Concentration_Plots.pdf>

Attachment A-4 Water Quality Validation Plots <http://www.sac.usace.army.mil/Portals/43/docs/civilworks/post45/Attachment_A_4_Water_Quality_Validation_Plots.pdf

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