2019

Lake Pontchartrain and Vicinity / West Bank and Vicinity GRR Hydrology and Hydraulics – Appendix C

U.S. Army Corps of Engineers,
New Orleans District

Non-Federal Sponsor: Coastal Protection and Restoration Authority Board

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Table of Contents

LAKE PONTCHARTRAIN AND VICINITY (LPV) / WEST BANK AND VICINITY (WBV) – OVERTOPPING AND INTERIOR FLOODING ASSESSMENT

1 GENERAL DESCRIPTION OF WORK

The purpose of this effort is to evaluate overtopping and interior flooding for hurricane and tropical storm surge events for the Greater New Orleans Hurricane and Storm Damage Risk Reduction System (GNO-HSDRRS) for existing and future no-action scenarios. The GNO-HSDRRS is divided into two sub polders which are the Lake Pontchartrain and Vicinity (LPV) and the West Bank and Vicinity (WBV) projects. Additionally, portions of the HSDRRS are co-located with the Mississippi River Levee (MRL). Interior flooding estimates are produced for the 20YR, 50YR, 100YR, 200YR, 500YR and 1000YR surge events for existing conditions (year 2023) and future no-action conditions (year 2073). Three future 2073 conditions are evaluated for low, medium and high relative sea level change RSLC projections.

2 SOFTWARE

HEC-RAS 5.0.6. The latest version of the Hydraulic Engineering Center's (CEIWR-HEC) River Analysis System (HEC-RAS) was used to model the inundation within the polder resulting from surge and wave overtopping events.

MATLAB R2017a. Matlab was used to automate the simulation of hundreds on RAS simulations, extract and plot model results, and run the ERDC surge statistics code.

ESRI ArcMap 10.2.2. GIS software was used to process lidar, levee and floodwall surveys, channel surveys, land coverage rasters.

3 HEC-RAS MODEL DEVELOPMENT

3.1 OBTAIN PREVIOUS OLDER MODELS

In previous studies, each sub-polder was modeled using storage areas, storage area connections, and 1D channels. There was little to no connectivity between sub-polder models, and so it was difficult to model the entire system as a whole. The 1D approach would not be recommended given the latest 2D (two-dimensional) advancements with HEC-RAS. Figure 1 displays an example of an older HEC-RAS 1D geometry for St. Bernard Parish. Information taken from the previous polder models includes the channel cross-sections (bathymetry) and some interior pump-station information.

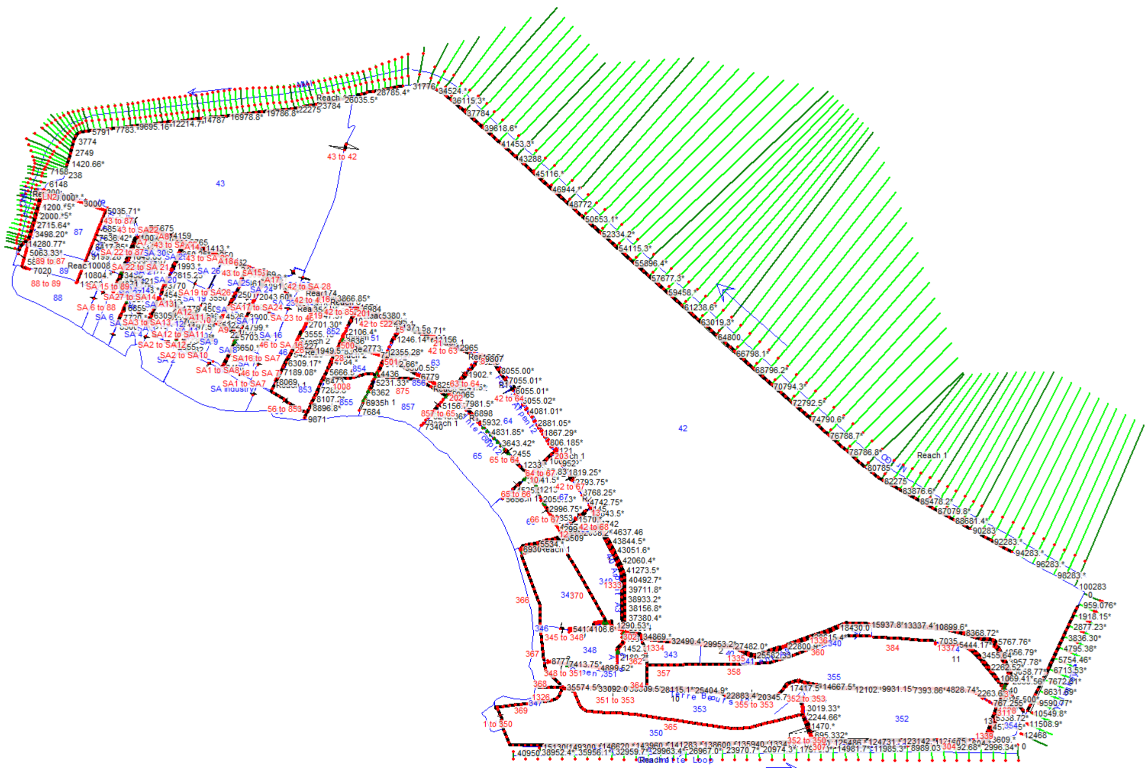

Figure 1. Example of polder model HEC-RAS 1D geometry from post-Katrina Study

3.2 HEC-RAS 2D MODEL DEVELOPMENT

A 2D hydrodynamic model was developed using the latest version of HEC-RAS. The HEC-RAS software has advanced considerably since previous studies of flooding of the polder interior. Given the drastic increase in capability of the newer version of HEC-RAS, an entirely new model geometry was developed using the best available data. Some input data from older models was incorporated into the latest HEC-RAS model.

Separate 2D meshes were created for each sub polder. The LPV includes RAS 2D meshes for St Charles, Orleans and Jefferson Parish east bank, the IHNC Corridor, New Orleans East, and St. Bernard Parish. The WBV includes RAS 2D meshes for Waggaman, Gretna, Belle Chasse and Harvey/Algiers canals. All 2D meshes are connected using storage area connections with weir profiles assigned using the latest available surveys. Figure 2 and Figure 3 display the HEC-RAS 2D computational domain for the entire HSDRRS. Figure 4 and Figure 5 display a zoomed portion of the RAS 2D computational domain in an areas located near Kenner, LA. The nominal mesh resolution is 700ft. The lower mesh resolution facilitates higher computational efficiency.

Figure 6 displays the Manning's n values applied to the HEC-RAS 2D mesh. Table 1 contains the Manning's n values applied to the HEC-RAS 2D mesh. The 2011 National Land Cover Database was used in this modeling effort. More information on this dataset is provided at http://www.mrlc.gov/. Manning's values were assigned to the various land coverage types in a manner consistent with other MVN H&H analyses.

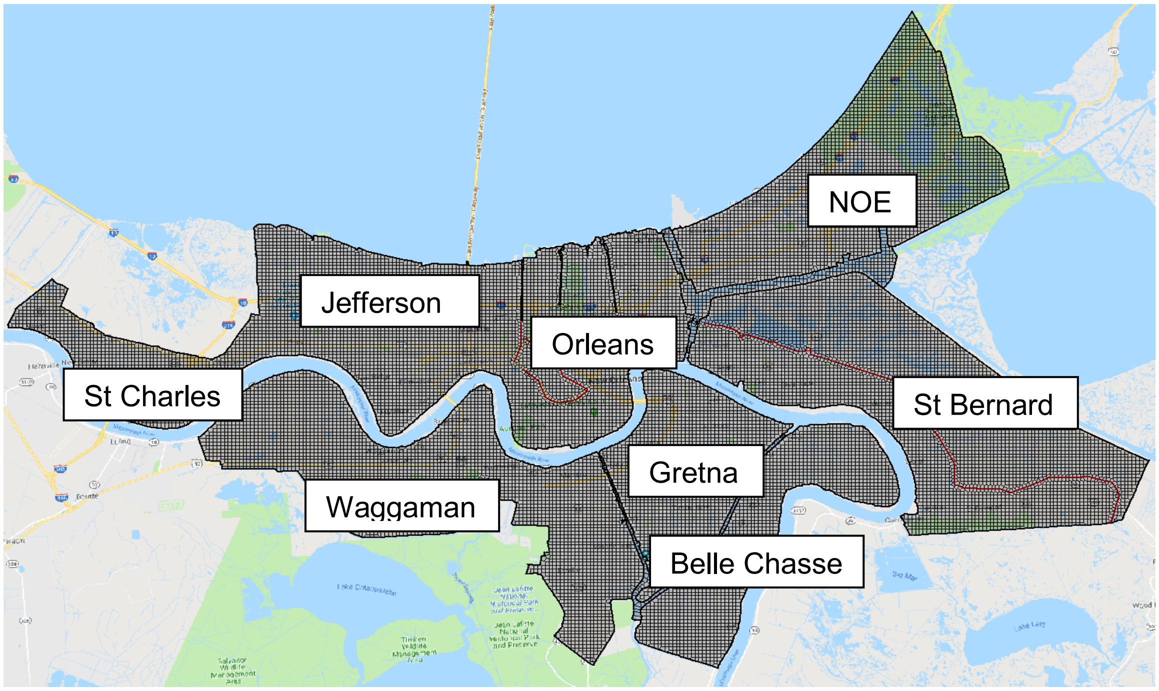

Figure 2. HEC-RAS computational mesh

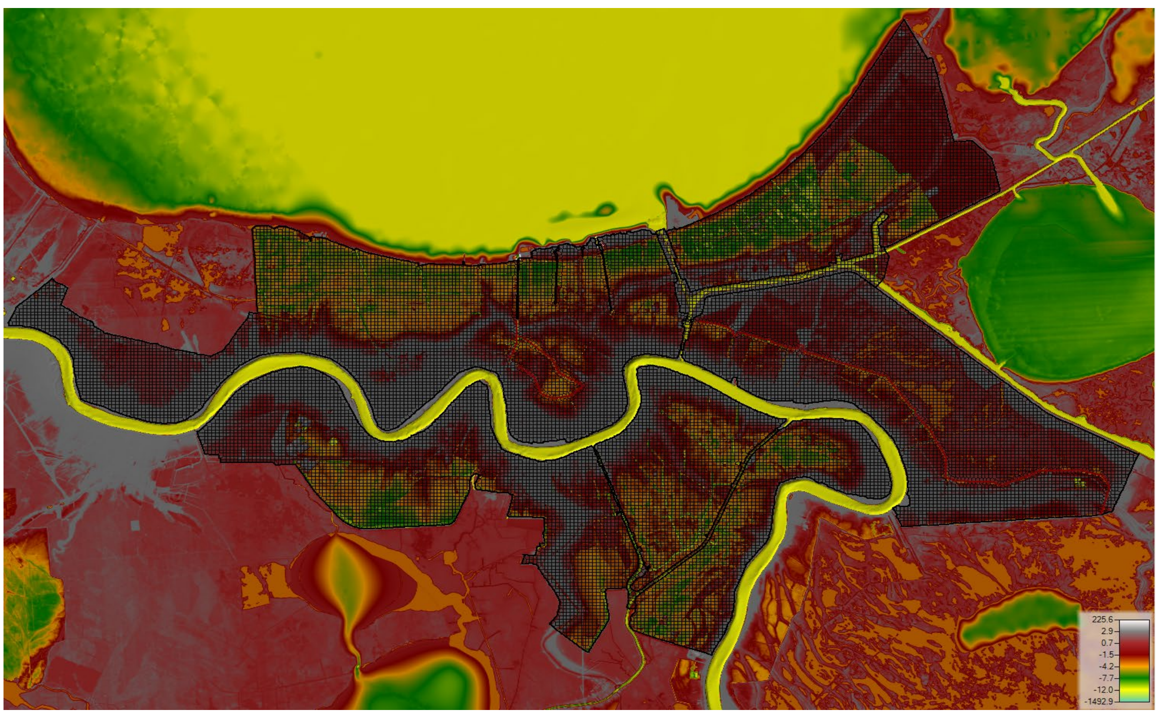

Figure 3. HEC-RAS computational mesh and terrain (ft. NAVD88)

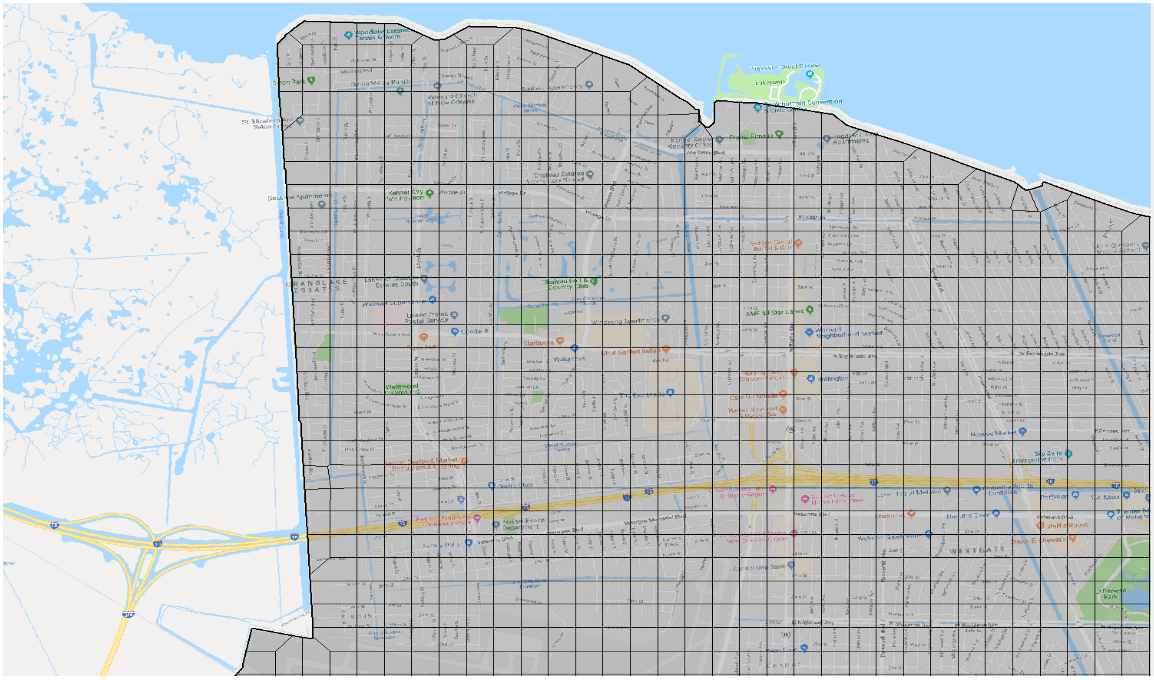

Figure 4. HEC-RAS computational mesh for HSDRRS interior near Kenner, LA

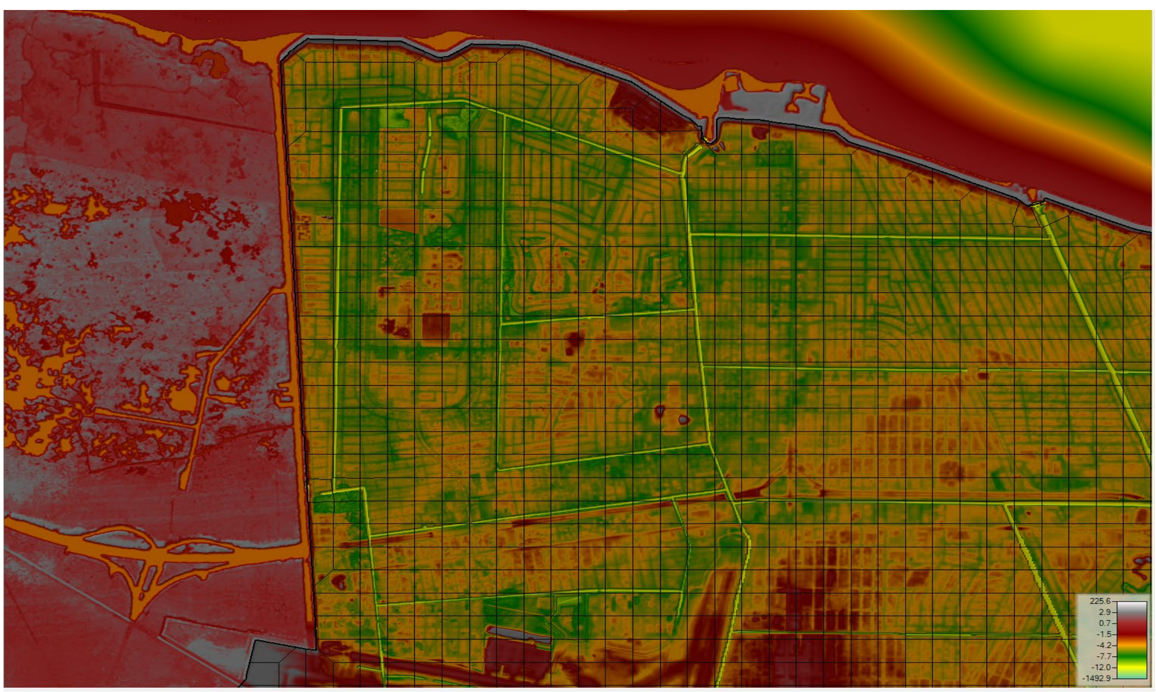

Figure 5. HEC-RAS computational mesh and terrain at HSDRRS interior near Kenner, LA (ft. NAVD88)

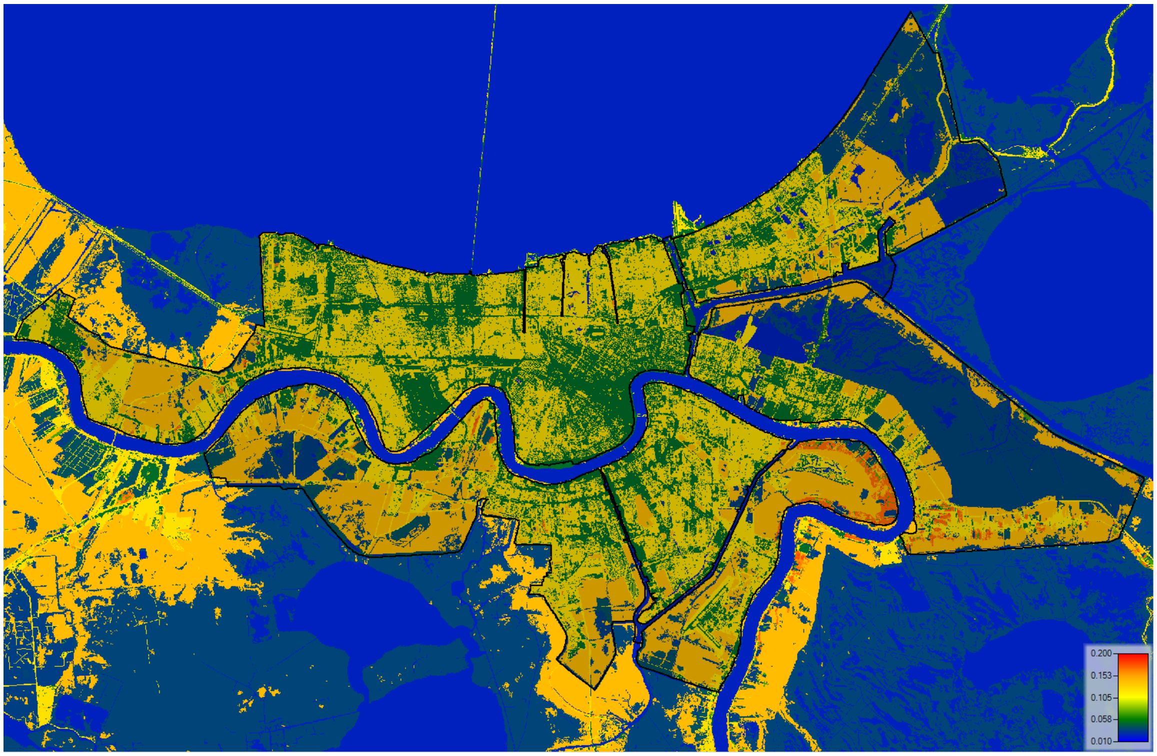

Figure 6. HEC-RAS Manning's n values

value	description	n-value
11	Open Water	0.022
21	Developed, Open Space	0.12
22	Developed, Low Intensity	0.121
23	Developed, Medium Intensity	0.05
24	Developed, High Intensity	0.05
31	Barren Land	0.04
41	Deciduous Forest	0.16
42	Evergreen Forest	0.18
43	Mixed Forest	0.17
52	Shrub/Scrub	0.07
71	Grassland/Herbaceous	0.035
81	Pasture/Hay	0.033
82	Cultivated Crops	0.04
90	Woody Wetlands	0.14
95	Emergent Herbaceous Wetlands	0.035

Table 1. Manning's n values applied to HEC-RAS 2D model

3.3 LEVEE SURVEYS, LIDAR AND CHANNEL BATHYMETRY, PUMPS

The Corps collected comprehensive elevation surveys of all HSDRRS perimeter levees in the fall of 2018. No floodwalls were included in the latest survey. All floodwall elevations were assigned based on the NCC surveys. The perimeter levee and floodwall elevations are not incorporated into the HEC-RAS 2D geometry but are used in overtopping calculations. Elevation profiles for the storage area connections were assigned based on the latest survey information.

RAS Terrain data was obtained from the USGS Northern Gulf Topo-Bathy dataset, which includes high resolution lidar of the HSDRRS interior. More information about USGS dataset can be found here: https://www.usgs.gov/land-resources/eros/coned. Channel bathymetry for all interior drainage canals was extracted from the post-Katrina era RAS1D polder models. Channel bathymetry and lidar were merged into a continuous terrain dataset in RAS Mapper.

Pump information including location and peak capacity was extracted from the Corps pump database located on the EGIS server. The pumps in the model are modeled as 2D connections with outlet rating curves. The rating curve approach ensures the peak capacity of each pump is utilized in the simulations. The pumps are assigned mostly along the perimeter of the mesh and are set to discharge the water out of the system. Some pumps are set to discharge from one 2D area to another, such as those pumping into the IHNC corridor or Harvey and Algiers canals.

3.4 OVERTOPPING FLOW BOUNDARY CONDITIONS AND INITIAL CONDITIONS

Overtopping rates were calculated at all 415 HSDRRS design segments. As part of the design of HSDRRS, the system was divided into 415 design segments. Each segment has unique levee or floodwall geometry and hydraulic boundary conditions including surge elevation, significant wave height, and mean wave periods. The latest version of the design segment shapefile was extracted from EGIS for LPV/WBV as well as the co-located MRL. In total, 415 segments are processed with a series of Matlab scripts that calculate overtopping time-series for 152 synthetic storms.

ADCIRC Hydrographs for all 152 synthetic storms were extracted at each segment using a Matlab script. The ADCIRC dataset used was from the 2017 CPRA master plan. The levee heights and alignments applied in the 2017 CPRA ADCIRC mesh provide an excellent representation of the existing HSDRRS. Peak significant wave heights and wave periods were extracted at each design segment. The wave time-series data was not extracted from the CPRA ADCIRC+SWAN simulations. Instead, the surge elevation time-series were normalized to the peak wave values, producing an approximate wave time-series needed for the overtopping calculations. This assumption is conservative since it assumes the peak wave and surge will be coincidental. This assumption was also made by USACE in the post-Katrina surge hazard analysis. Additional inputs into the overtopping calculations include levee geometry parameters including wave berm elevation, levee slope and crest elevations. Levee and floodwall surveyed elevations were mapped to each of the 415 segment profiles.

Wave overtopping rates for levees were calculated using the equations 5.10 and 5.11 provided in Eurotop overtopping manual (Figure 7). Equation 5.17 was used for floodwalls. These equations represent the “mean-value” estimate of overtopping. More information about the Eurotop formulae can be found here: http://www.overtopping-manual.com/. A specialized Matlab function was written to estimate overtopping for levees or floodwalls and for surge and wave overtopping. If the surge level is less than the crest elevation, wave overtopping formuale are used. If the surge is greater than the crest elevation, the weir equation is combined with the wave overtopping formulae, and the relative freeboard (Rc) value is set to 0. This approach is consistent with the guidance provided in the Eurotop manual. Overtopping rate time-series were calculated at each survey point along each of the 415 design segments. The resulting overtopping rates at each survey point are multiplied by the width between each point, then summed to produce a total flow for each segment. The overtopping time-series at each segment are then summed to the corresponding RAS 2D flow boundary. In total, 81 flow boundary conditions were assigned to the RAS 2D geometry.

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.023}{\sqrt{\tan\alpha}} \gamma_b \cdot \xi_{m-1,0} \cdot \exp[-(2.7 \frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v})^{1.3}] \quad 5.10$$

with a maximum of

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot \exp[-(1.5 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma *})^{1.3}] \quad 5.11$$

Figure 7. Eurotop wave overtopping formulae for levees

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.047 \cdot \exp[-(2.35 \frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta})^{1.3}] \quad 5.17$$

Figure 8. Eurotop wave overtopping formula for vertical wall

Figure 9 displays the levee and floodwall survey elevations for the entire HDSRRS perimeter taken from the fall 2018 levee survey and the NCC floodwall surveys. The LPV-HSDRRS is the continuous perimeter from Bonnet Carre Spillway to Caernarvon Diversion. For example purposes, Figure 9 also displays the peak surge and wave information along each profile for one of the synthetic storms (storm 027). The plot shows how the surge elevation is greater than the crest elevation in certain areas. For this particular storm, surge and wave overtopping occurs in several locations including St Charles Parish on the east bank, New Orleans East, and the co-located MRL. This plot was produced for all 152 synthetic storms.

Table 2 contains the starting water surface elevations assumed in the HEC-RAS modeling for different polders. The starting water surface elevations were assigned based on water surface elevations that were captured in the lidar surface. Initial water levels in the IHNC corridor and Harvey and Algiers canals were assigned based on the closure trigger levels for the IHNC surge barrier and the Western Closure Complex.

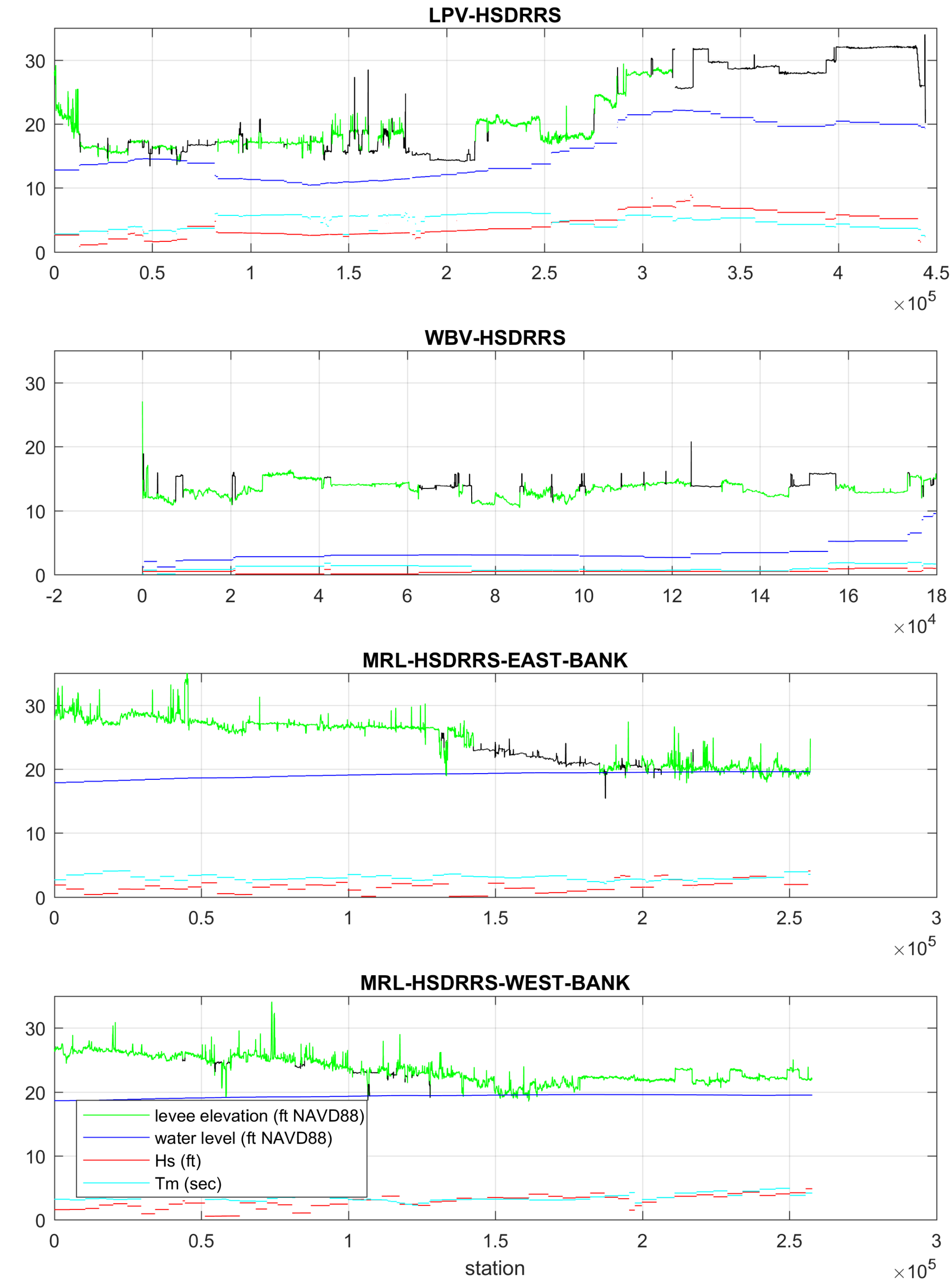

Figure 9. Levee and floodwall elevations, peak surge elevations and waves

Scenario	Starting Water Surface Elevation (ft. NAVD88)
St Charles, Jefferson, Orleans	-13.5
IHNC	3.0
New Orleans East	-15
Saint Bernard	-7
Waggaman	-10.9
Gretna	-10.9
Belle Chasse	-10.9
Harvey and Algiers Canals	2.5

Table 2. Starting water surface elevations in HEC-RAS modeling.

3.5 HEC-RAS 2D SIMULATIONS OF 152 SYNTHETIC STORMS

HEC-RAS simulations were computed for all 152 JPM-OS synthetic storms. The storms cover a range of hypothetical tracks, forward speeds, intensities and sizes. Figure 10 displays the tracks for all 152 synthetic storms compared against a series of historically significant storms. The JPM-OS synthetic storms are basically an extension of the limited observed record. Figure 11 compares the wind-speeds of the synthetic storms compared against the historically significant storms. The synthetic storms are parametrically similar to actual storms in the record. All 152 storms must be simulated in order to estimate storm surge statistics.

As previously described, the overtopping time-series for each storm was applied to the RAS 2D polder model. To accomplish the task of running 152 synthetic storms, a specialized Matlab script was written to automate the process. The Matlab script overwrites and unsteady flow file with overtopping flow time-series for a given storm, then runs the simulation and saves the results. Figure 12 displays the peak water surface elevation produced by synthetic storm 027. The figure shows overtopping in St. Charles Parish and portions of the co-located MRL, consistent with what is shown in Figure 9. The surge of this event at these locations is roughly equivalent to a 500YR return period.

The RAS simulation of one storm crashed. In this case, the overtopping flow rate was too extreme for the software to handle. A 100,000cfs limit was applied to the inflow hydrographs at each flow boundary, which resolved the stability problem.

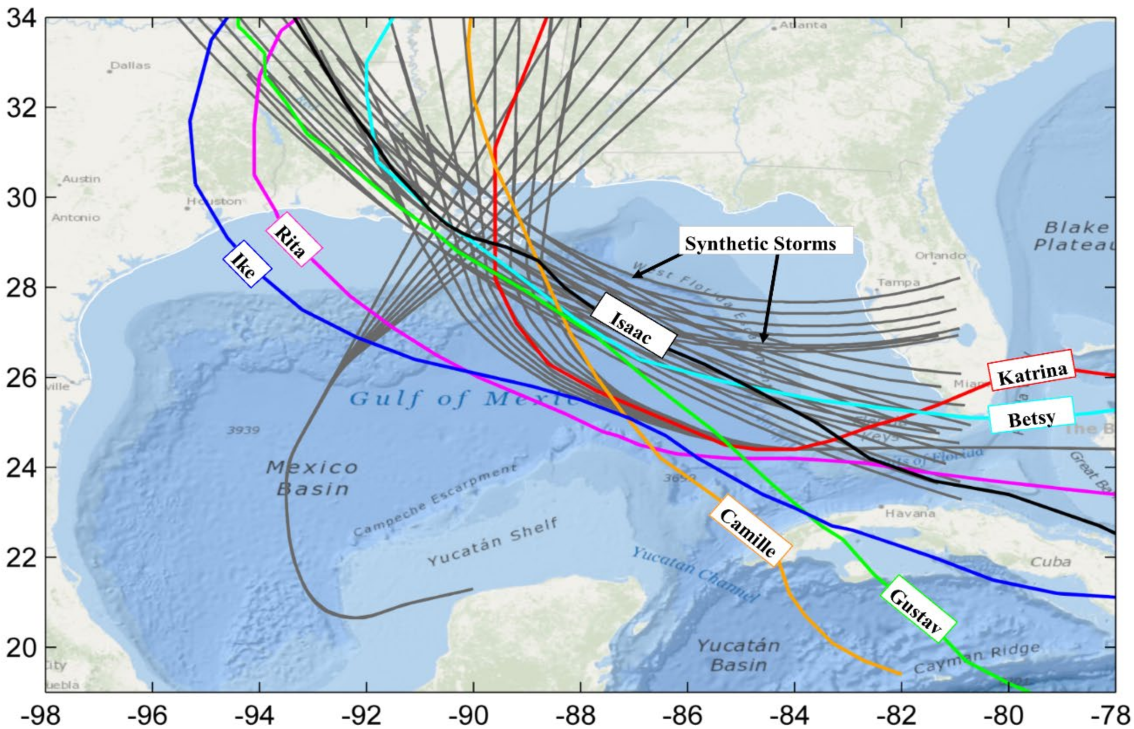

Figure 10. Storm tracks for JPM-OS synthetic events and historical storms of significance

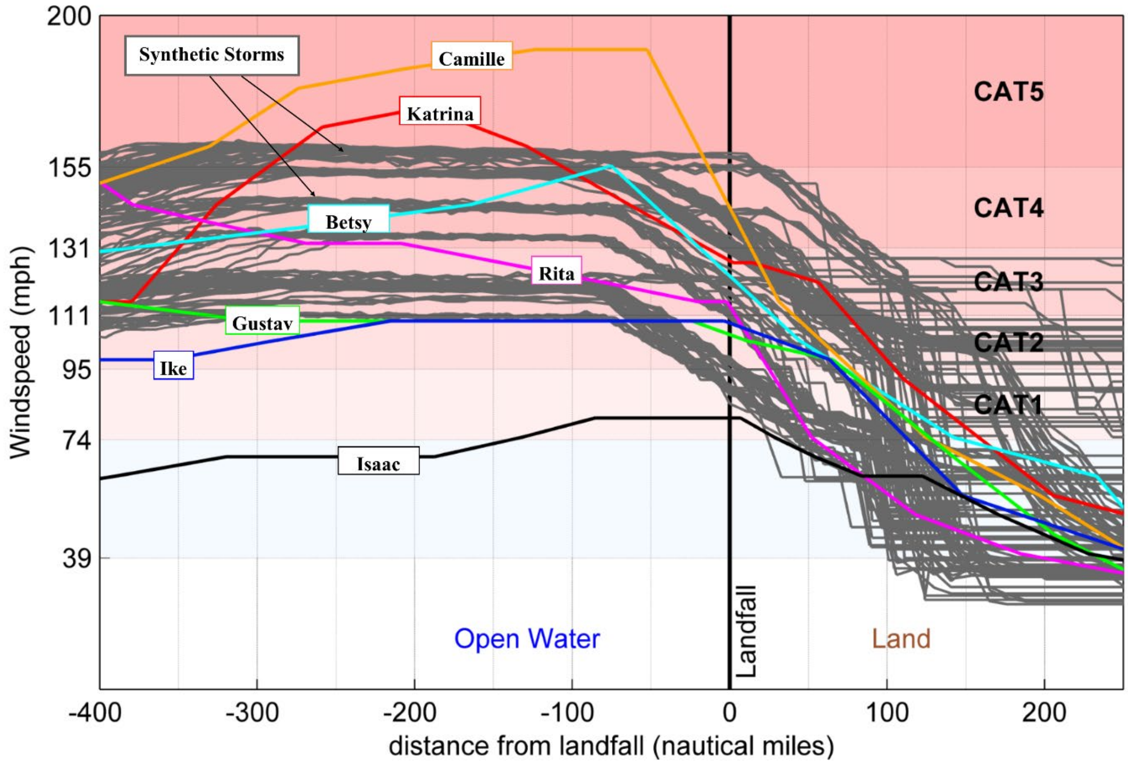

Figure 11. Storm wind-speeds for JPM-OS synthetic events and historical storms of significance

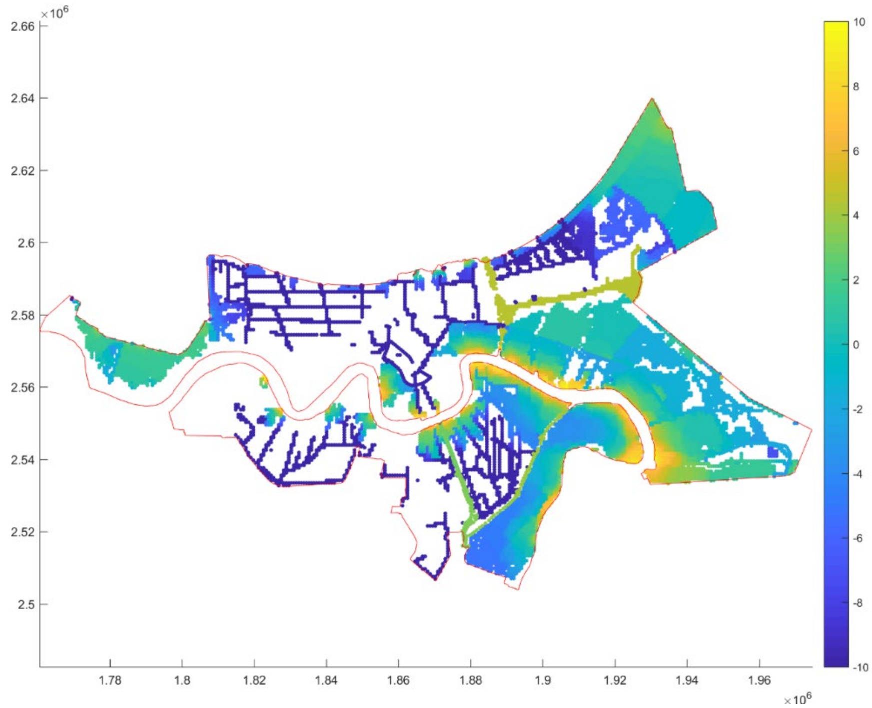

Figure 12. Peak water surface elevation (ft. NAVD88) for synthetic storm 027

3.6 JPM-OS SURGE STATISTICS

Once all 152 synthetic storms were evaluated, surge statistics could be completed using the latest JPM-OS code. The code was supplied by ERDC's Coastal Hydraulics Lab. The code combines the meteorological probability and the peak surge elevation of all 152 storm events to estimate the 20YR, 50YR, 100YR, 200YR, 500YR and 1000YR surge elevations. Figure 13 displays the 100YR water surface profile for existing conditions. The model shows some overtopping in certain areas where there are known low spots relative to the 100YR required design including St. Charles Parish and portions of the co-located MRL. Figure 14 displays the 500YR water surface profile for existing conditions. The 500YR inundation is much more extensive than the 100YR. The water surface profile for each return period was provided to economics. Figure 15 and Figure 16 display the peak depth for the 100YR and 500YR frequencies for the 2023 without-project condition.

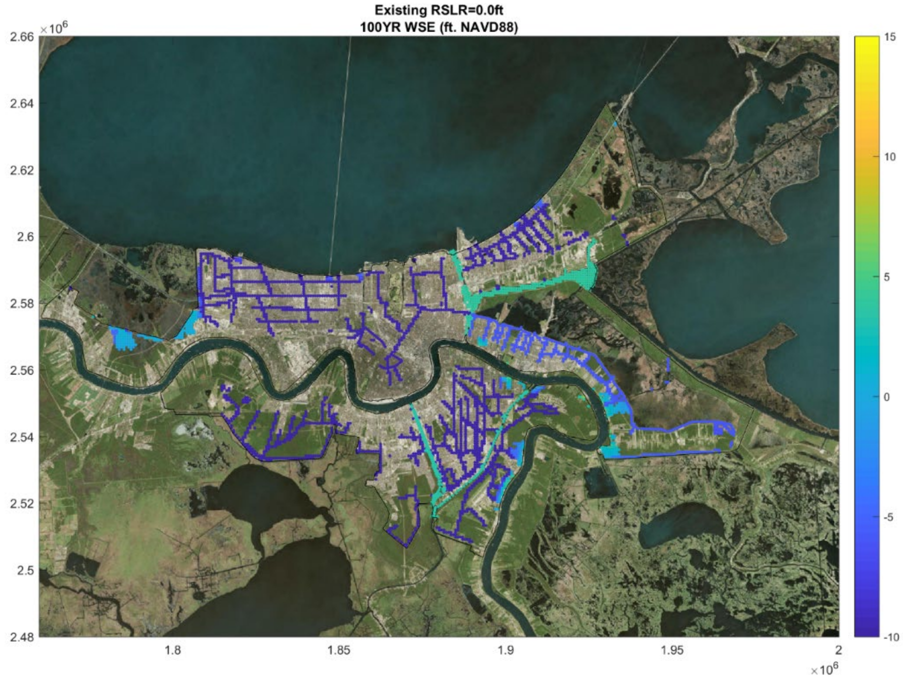

Figure 13. 100 year peak water surface elevation (ft. NAVD88) for existing 2023 conditions

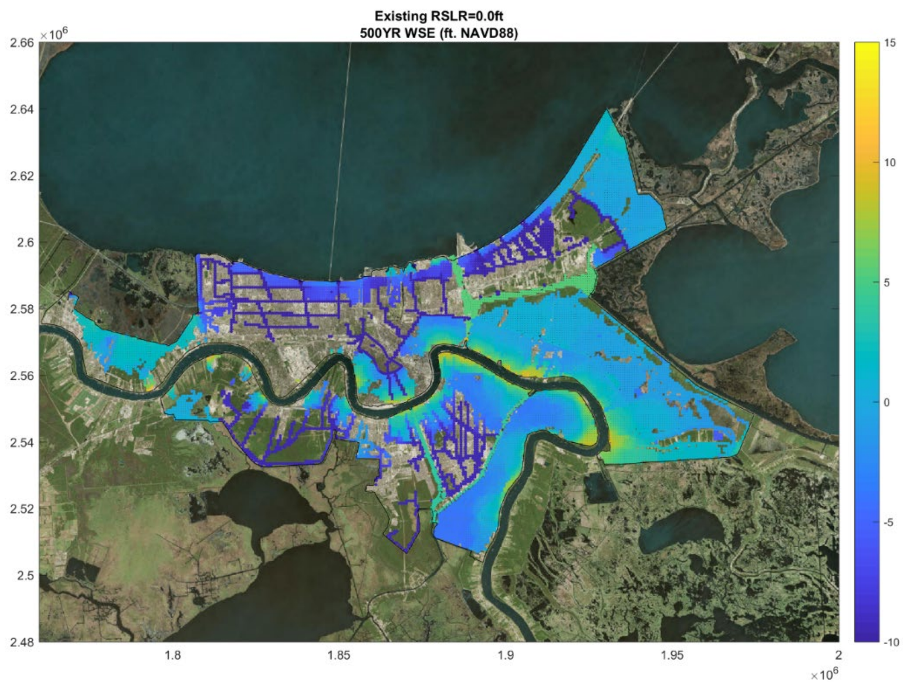

Figure 14. 500 year peak water surface elevation (ft. NAVD88) for existing 2023 conditions

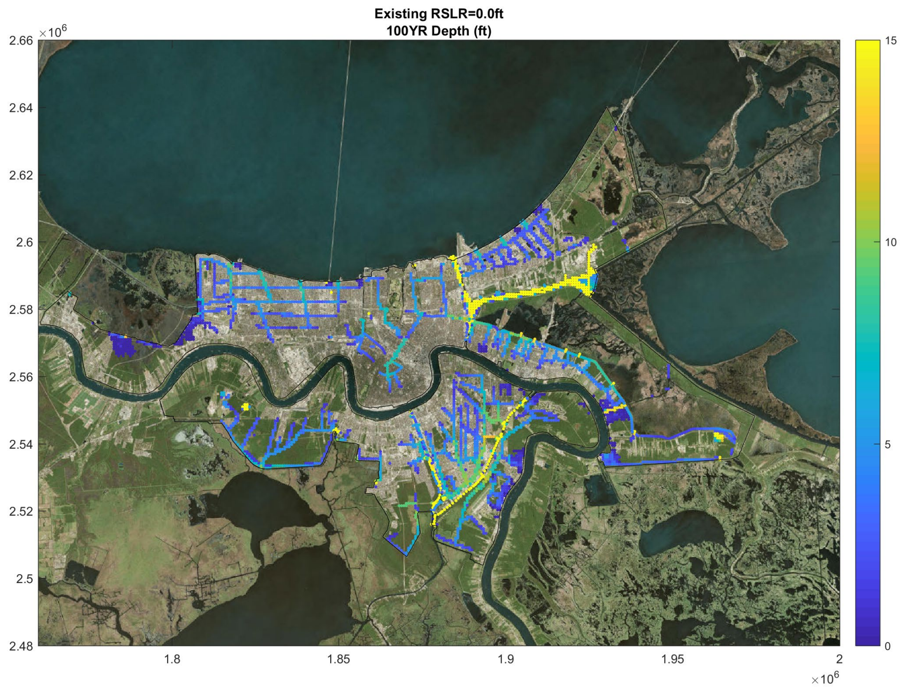

Figure 15. 100 year peak depth (ft.) for existing 2023 conditions

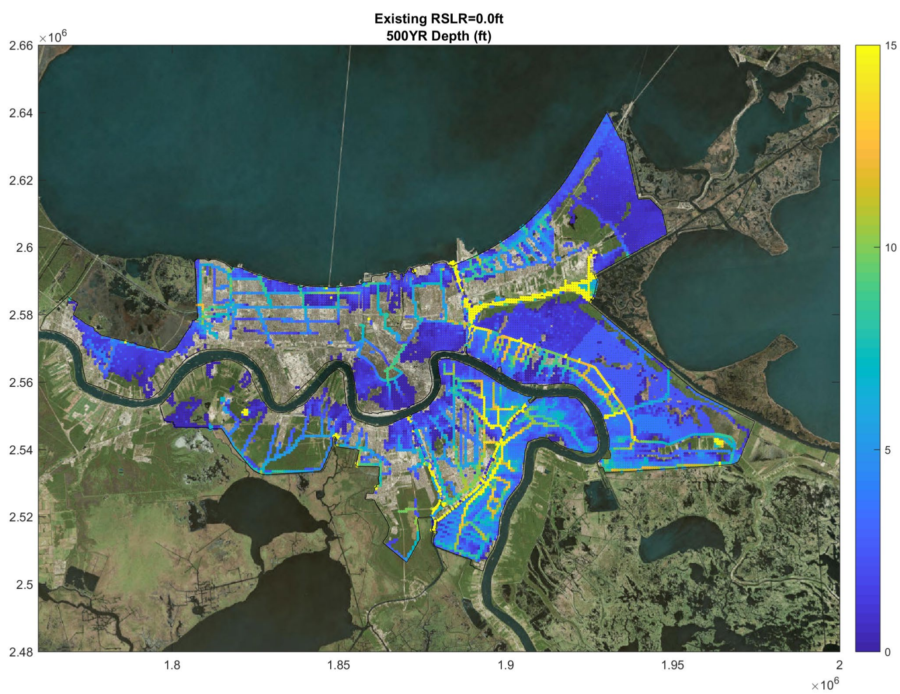

Figure 16. 500 year peak depth (ft.) for existing 2023 conditions

3.7 FUTURE CONDITIONS 2073

The overtopping calculations, RAS simulation and JPM-OS statistics were repeated for the 2073 future no-action condition. Three relative sea level change (RSLC) values were evaluated including 1.3, 1.8 and 3.4 ft. The Corps climate change website was used to determine the three RSLC amounts: http://corpsmapu.usace.army.mil/rccinfo/slc/slcc_nn_calc.html. The average RSLC projections at 7 gages was used. Table 3 contains the RSLC projections at the 7 gages. Figure 17 displays the location of the 7 gages relative to HSDRRS.

Location	Rate of Ground Movement (mm/yr)	Subsidence over 50 Years (ft)	Projected RSLC from 2023 to 2073		
			Low (ft)	Int (ft)	High (Ft)
Lake Pontch West End (85625)	7.11	1.2	1.4	1.9	3.5
Rigolets (85700)	3	0.5	0.7	1.2	2.9
IHNC (76160)	8.77	1.4	1.7	2.2	3.8
Bayou Barataria (82750)	5.3	0.9	1.2	1.6	3.2
IHNC lock (01340)	5.1	0.8	1.1	1.6	3.2
MS River Carrolton (01300)	5.4	0.9	1.2	1.7	3.2
MRGO Shell Beach (85800)	8.5	1.4	1.7	2.2	3.7
average:	6.2	1.0	1.3	1.8	3.4

Table 3. RSLC projections

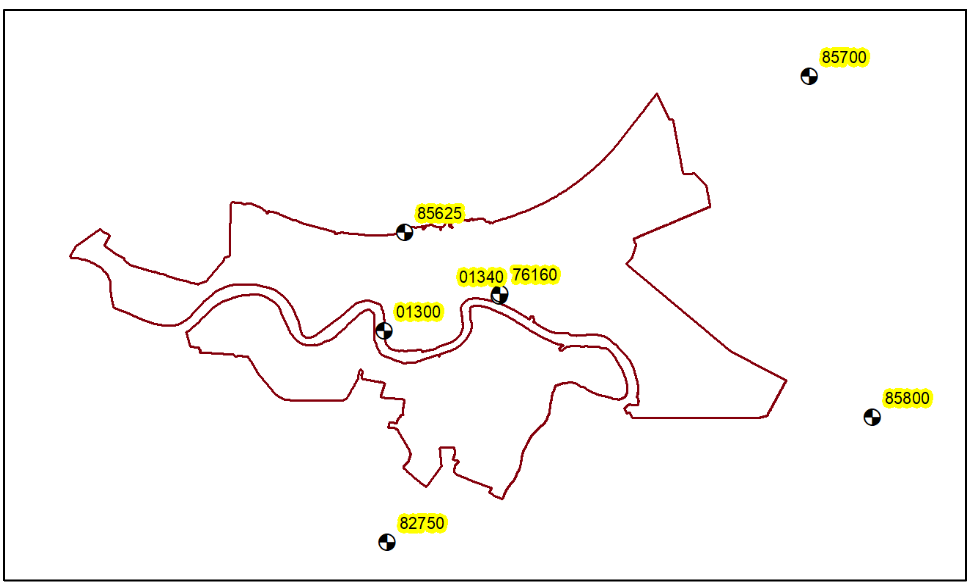

Figure 17. Location of water level gages used to determine RSLC projections

CPRA conducted a full suite of 152 storms for the future condition. The amount of eustatic sea level rise assigned in the ADCIRC simulations was 1.5ft. The grid bathymetry was changed to reflect future conditions. Some portions of the grid were subsided and some accreted, as depicted in Figure 18. The subsidence varies by region, but around HSDRRS the amount was close to -0.5ft. For the purposes of this study, we assume the CPRA future condition runs evaluated a total RSLC of approximately 2.0ft (1.5 eustatic + 0.5ft subsidence).

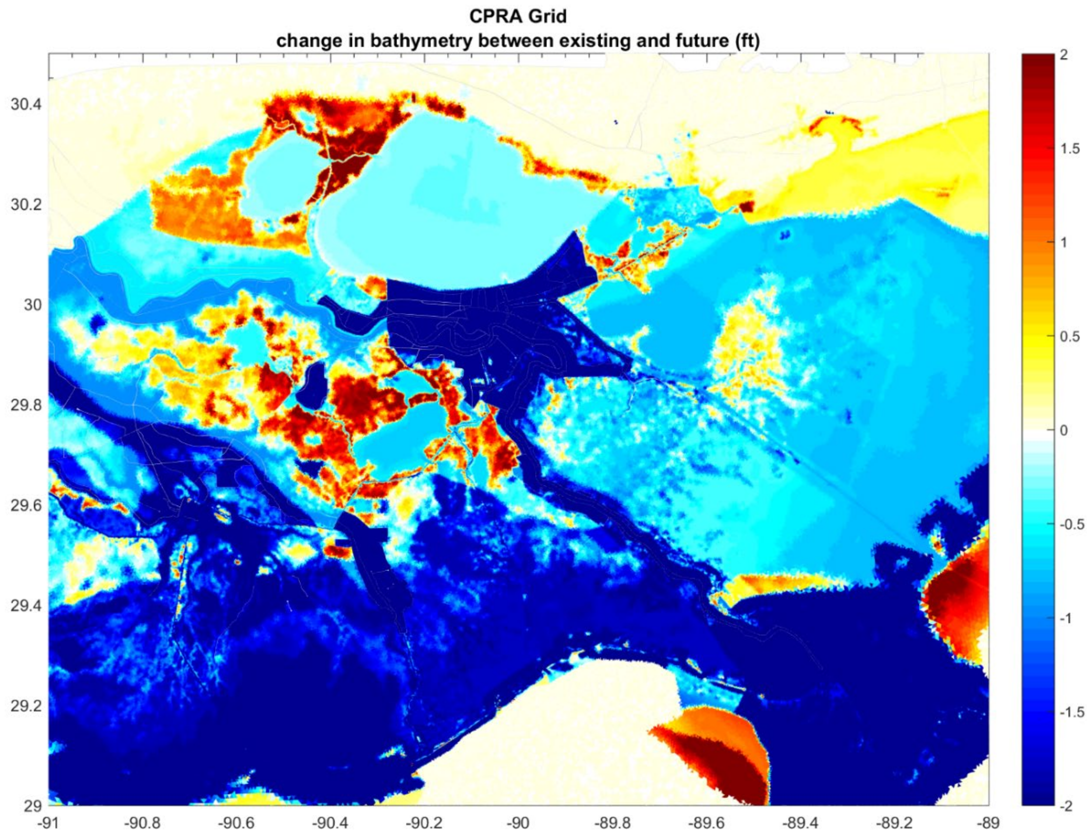

Figure 18. Change in bathymetry from existing to future conditions (S13G60).

Surge and wave time-series for the future condition for the various RSLC conditions (1.3, 1.8 and 3.4) were developed using linear interpolation and extrapolation of the CPRA simulation results. 152 simulations were conducted with 0.0ft and 2.0 ft of RSLC. The confidence level for the interpolated surge and wave results (RSLC= 1.3 and 1.8ft) are higher than the extrapolated case (RSLC=3.4ft). The CPRA simulations provide the best representation of future conditions available due to the incorporation of spatially variable subsidence, land use changes, morphology and updates to bottom friction and canopy coefficients.

Future condition overtopping calculations also factor in levee settlement over the 50 year period of analysis. Levee settlement data was provided by the MVN Geotechnical branch. Levee settlement values vary by location. The worst case settlement projection is 5.4ft, but the average settlement values of all levees is 2.2ft. Figure 19 displays the projected levee settlement values provided by the MVN Geotech branch. No settlement was assumed for the floodwalls.

Figure 19. Projected levee settlement values by 2073. Levees are plotted as green line. Floodwalls are grey lines. No settlement was assumed at floodwalls.

Levee settlement and RSLC result in greater overtopping volumes and more inundation in the HEC-RAS simulations. Figure 20 displays the resulting 100YR water surface elevation for the future no-action scenario assuming intermediate 1.8 ft RSLC. The resulting inundation is much greater in the future no-action scenario. Figure 21 displays the resulting 500YR water surface elevation for the future no-action scenario assuming intermediate 1.8 ft RSLC. All statistical water surfaces were provided to economics for evaluation. Figure 22 and Figure 23 display the 100YR and 500YR depths for 2073 intermediate RSLC conditions for without project conditions.

The modeling of synthetic storms estimates overtopping rate time-series at the IHNC Surge Barrier, Seabrook, and the IHNC lock. Statistical processing of modeled water-levels produces stage frequency data within the closed IHNC basin. Water levels for future no-action intermediate RSLC conditions within the closed IHNC basin were estimated to be 4.0 ft NAVD88 for the 1% event and 6.2ft NAVD88 for the 0.2% event. These water levels do not include the effects of rainfall pumping into the basin, rainfall directly on the basin, wind-setup within the basin, or the uncertainty of water levels. In the past, 90% water levels were assumed. All of these added effects would bump up the expected water level within the basin. It is assumed water levels within the basin would not exceed the previously established safe-water-level of 8ft. If there are problems exceeding the safe-water-level, there are ways to mitigate, aside from raising barriers, such as adding a pump-station, expanding storage by establishing a conduit to the central wetlands, or accepting a higher level of risk within the basin. Another important observation is when Hurricane Gustav produced approximately 12ft NAVDD88 surge within the basin (prior to barrier construction), and the interior floodwalls performed adequately, suggesting a higher safe-water-level may be possible. Since the interior IHNC basin is a sensitive area, it is important to provide a more detailed review the expected interior water levels for with and without project conditions during the next phases of the project.

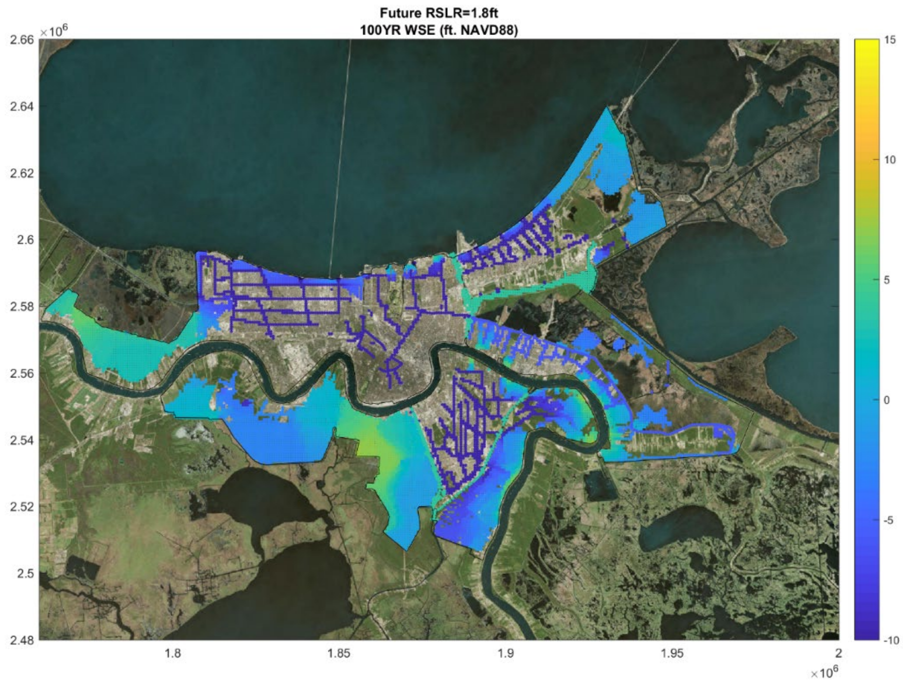

Figure 20. 100 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC conditions – without project

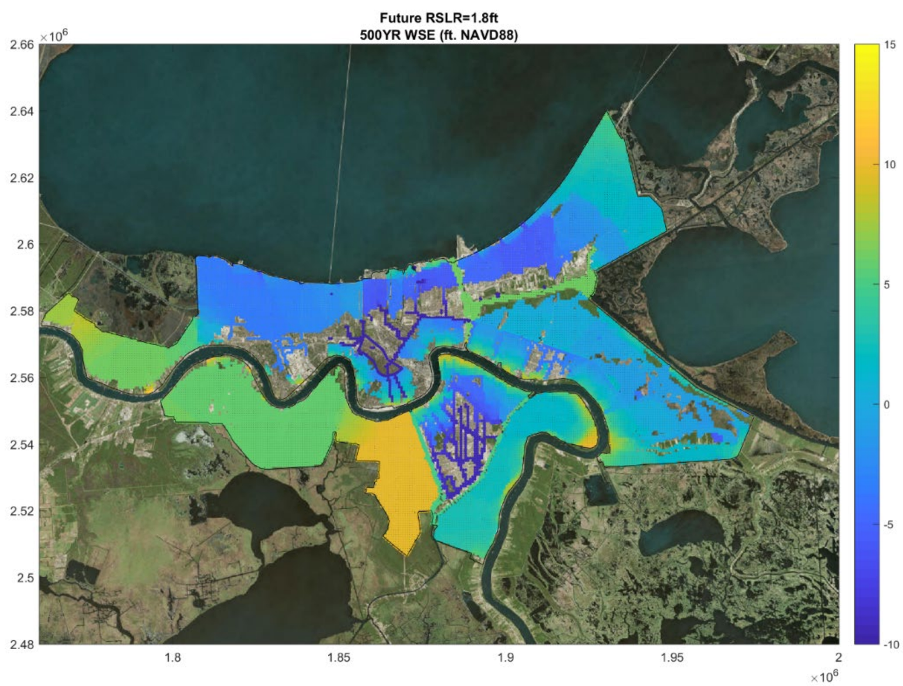

Figure 21.1 500 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC conditions – without project

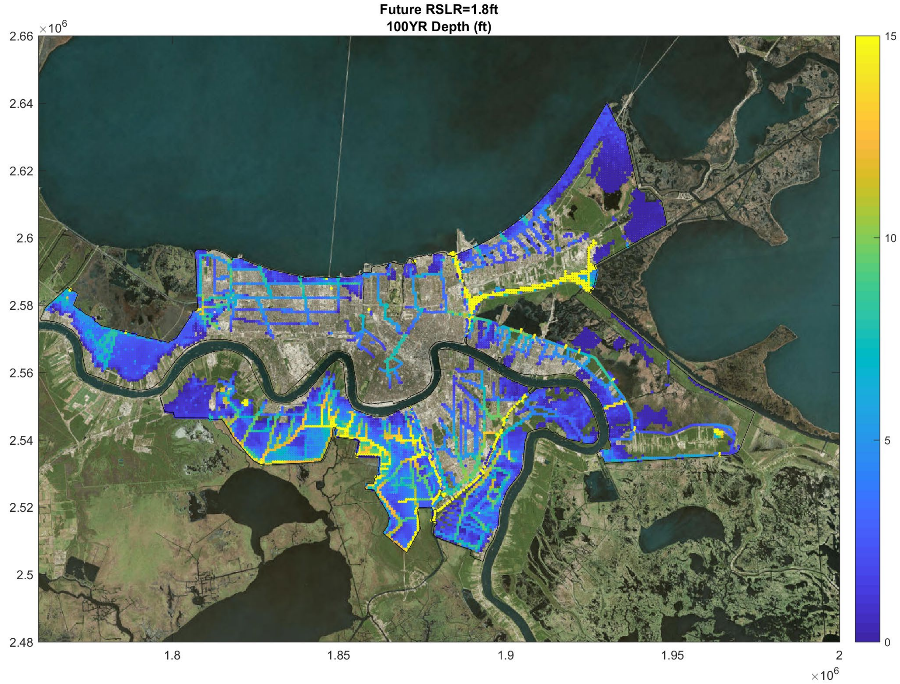

Figure 22. 100 year peak depth (ft) for future 2073 intermediate RSLC conditions – without project

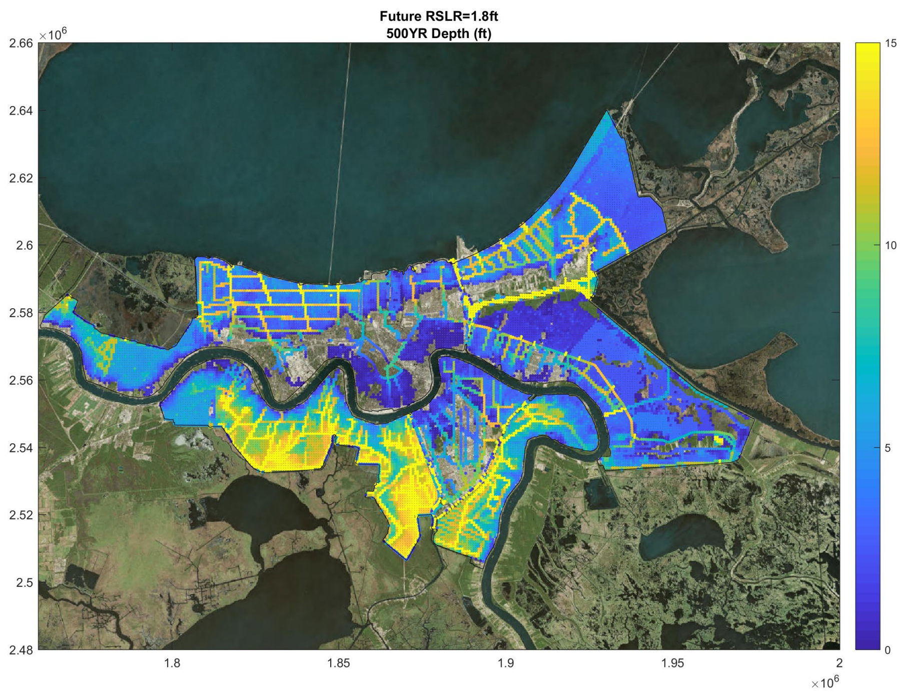

Figure 23. 500 year peak depth (ft) for future 2073 intermediate RSLC conditions – without project

3.8 EXTERIOR SURGE STATISTICS

The CPRA ADCIRC+SWAN simulations were processed with the ERDC JPM-OS statistical code to produce exterior surge and wave statistics for existing and future conditions. Exterior surge and wave statistics are needed to determine the required 100YR design elevations for the 2073 future condition for the intermediate and high RSLC scenarios. The statistical code was run on the CPRA ADCIRC+SWAN results for a small area encompassing HSDRRS. Figure 24 through Figure 26 display the 100YR and 500YR still water level, significant wave height, and mean wave period for existing conditions (RSLC = 0 ft). Figure 27 through Figure 29 display the 100YR and 500YR still water level, significant wave height, and mean wave period for future conditions (RSLC = 2 ft). Surge and wave statistics were linear interpolated and extrapolated for RSLC of 1.8 and 3.4 ft. The extrapolation to RSLC=3.4 ft is more uncertain than the interpolated values for RSLC=1.8ft.

Figure 30 through Figure 32 display comparisons between the older post-Katrina surge and wave statistics and the updated statistics produced for this study. The 100YR/500YR water levels and waves are mostly consistent aside from a few differences. The CPRA ADCIRC+SWAN simulations assigned a flow boundary of 325,000 cfs for the Mississippi River. This value is significantly lower than previous Corps estimates for Mississippi River discharge assigned for surge hazard modeling. In the past, the Corps evaluated a range of discharges and determined that 400,000 cfs gives reasonable surge values in the river and is consistent with more sophisticated statistical analysis of coincident hazards. Due to the lower 325,000 cfs boundary condition for the Mississippi River, a significant discrepancy exists between the older Corps surge statistics in the river and the statistics produced with the CPRA ADCIRC+SWAN simulations. The comparison in Figure 24 shows how 100YR and 500YR water levels are much lower with the updated statistics. The main reason for this discrepancy is the lower antecedent discharge assumed in the CPRA ADCIRC+SWAN simulations, but some of the discrepancy might be attributed to the new ERDC statistical code. Another discrepancy between the new and old statics existing in the mean wave periods on the WBV and portions of the LPV, as shown in Figure 32.

It was decided to adjust the surge statistics in the river to account for a higher 400,000 cfs. This adjustment provided more realistic surge values in the river. The adjustment was based on a regression analysis comparing surge levels between the CPRA ADCIRC+SWAN simulations and the older set of ADCIRC+STWAVE simulations which assumed 400,000cfs. The adjustment increases surge values in the river by approximately 1 to 2.5ft. The adjusted surge levels in the river are shown in Figure 33.

More information concerning the CPRA ADCIRC+SWAN simulations can be found online here:

http://coastal.la.gov/wp-content/uploads/2017/04/Attachment-C3-25.1_FINAL_04.05.2017.pdf

http://coastal.la.gov/our-plan/2012-coastal-masterplan/cmp-appendices/

3.9 MISSISSIPPI RIVER DISCHARGE DURING HURRICANE SEASON

The 400,000 cfs Mississipi River discharge design assumption was checked against observed flow records during hurricane season. Figure 34 displays the entire record of observed daily dicharges for the lower Mississippi River along with the cumulative probability distribution of discharges by month. The plot shows how discharge in the river is, on average, lower than 400,000 during the peak of hurricane season (August/Sept), but there are exceptions. The original HSDRRS analysis processed river discharges from 1976 to 2002 and computed cumulative probability of discharges for each month during hurricane season. Figure 35 displays the cumulative probability of discharge for each month in hurricane season based on data from 1976 to 2002. This data, along with hurricane frequency information was needed to compute surge statistics in the river. Figure 36 displays the cumulative probability of discharge for each month in hurricane season based on data from 1976 to 2019. When the latest data is added and statistics processed, there appears to be a small increase in the expected discharge during hurricane season. For example, the 50% or mean discharge during July was approximately 410,000 cfs with the data from 1976 to 2002. When the data is updated, the mean discharge during July becomes 450,000 cfs. Updating the assumed design discharge from 400,000 to 450,000 might change design water levels by 0.5ft to 1.0ft based on crude approximations.

Another assumption that can change stage-frequency information in the river is observed hurricane frequency by month. In the older HSDRRS analysis, a sample of 14 observed storms provided the hurricane probability by month. Table 4 contains the assumed probabilities of hurricane by month assumed in the original HSDRRS analysis. Since 2005, more storms have impacted New Orleans including Gustav, Ike, Isaac, Karen and Barry. These added storms may change some of the assumptions about hurricane frequency and ultimately impact the stage-frequency calculations in the river. The latest hurricane frequency and river discharge data suggests that the assumptions made concerning hurricane frequency and discharge frequency are still valid for a feasibility level study. However, they have changed enough to warrant a revisit during later design assessments such as the ERDC-CPRA surge hazard analysis, which is currently underway.

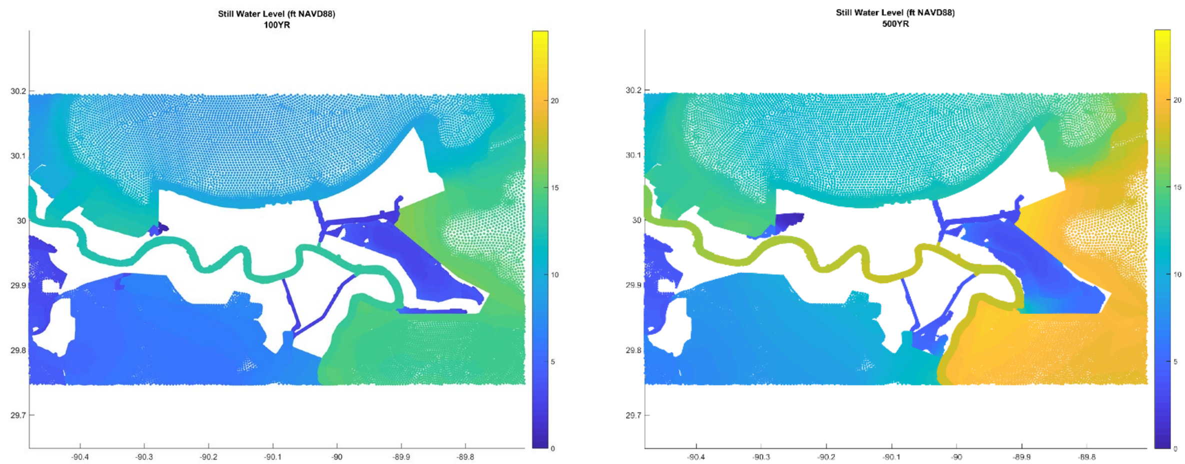

Figure 24. 100YR and 500YR still water levels (ft. NAVD88) for existing conditions (RSLC=0 ft)

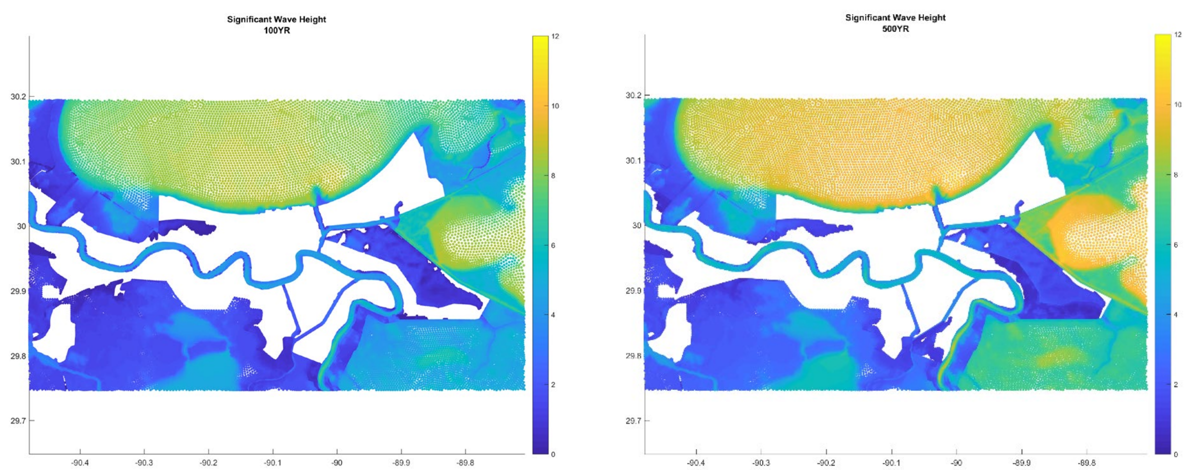

Figure 25. 100YR and 500YR significant wave heights (ft) for existing conditions (RSLC=0 ft)

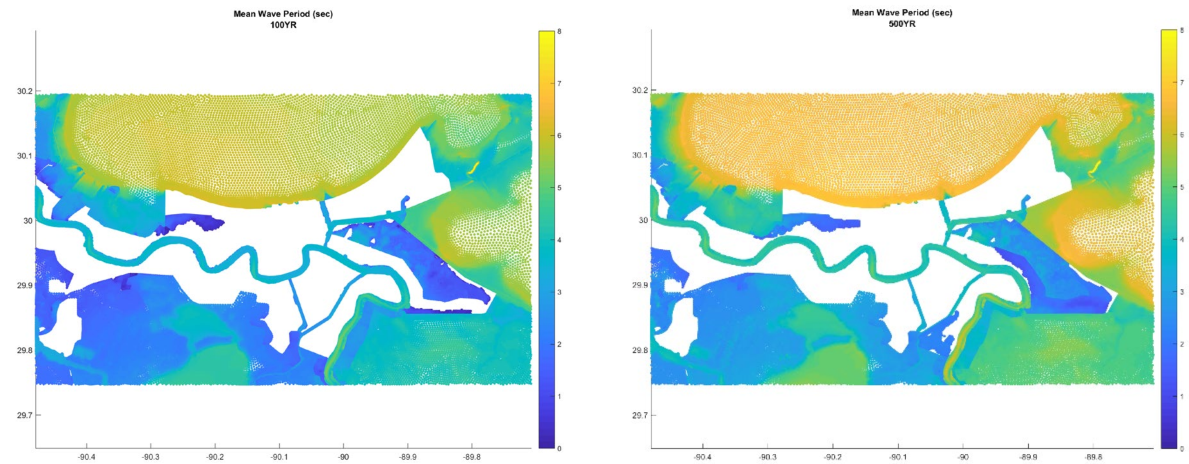

Figure 26. 100YR and 500YR mean wave period (sec) for existing conditions (RSLC=0 ft)

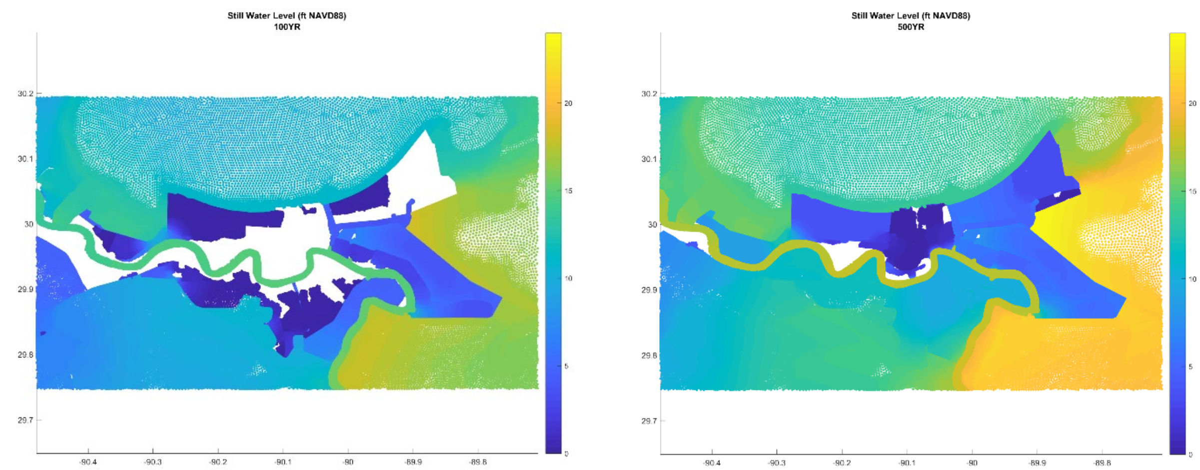

Figure 27. 100YR and 500YR still water levels (ft. NAVD88) for future conditions (RSLC=2 ft)

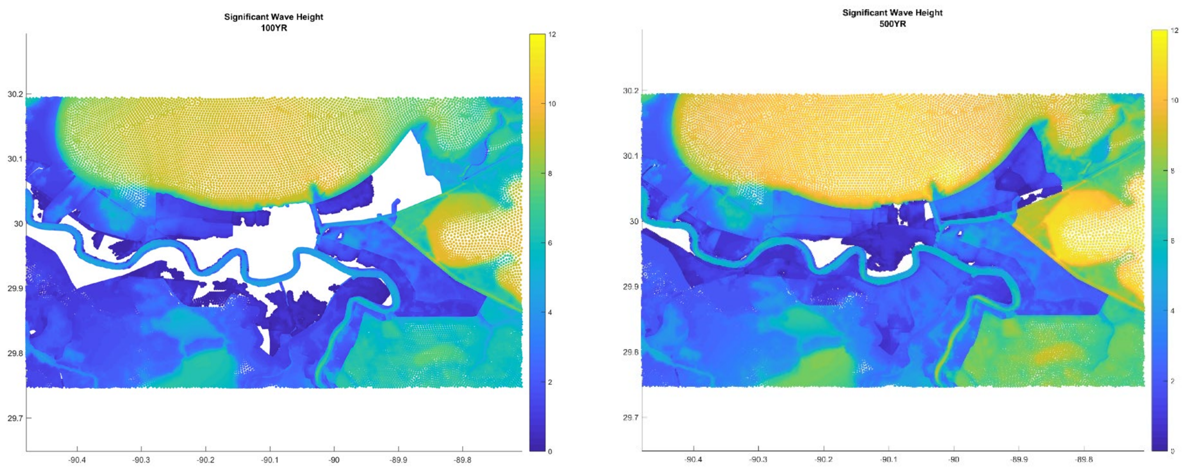

Figure 28. 100YR and 500YR significant wave heights (ft) for future conditions (RSLC=2 ft)

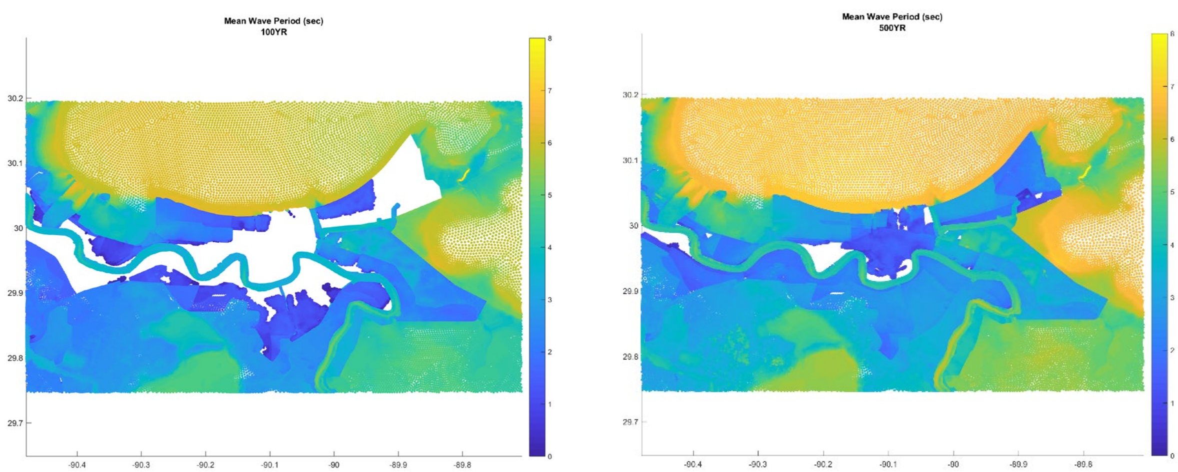

Figure 29. 100YR and 500YR mean wave period (sec) for future conditions (RSLC=2 ft)

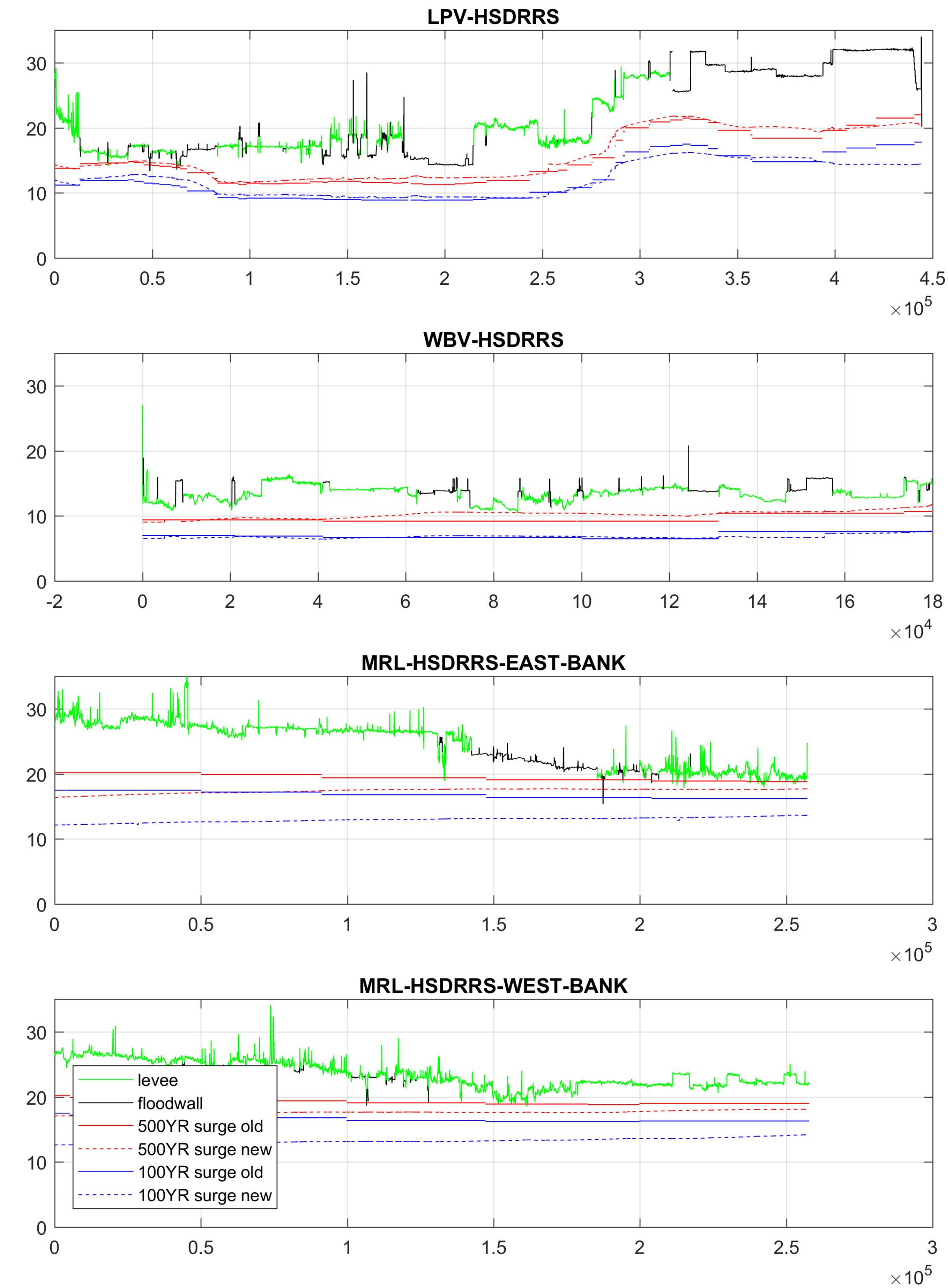

Figure 30. Comparison of new and old 100YR and 500YR still water level statistics

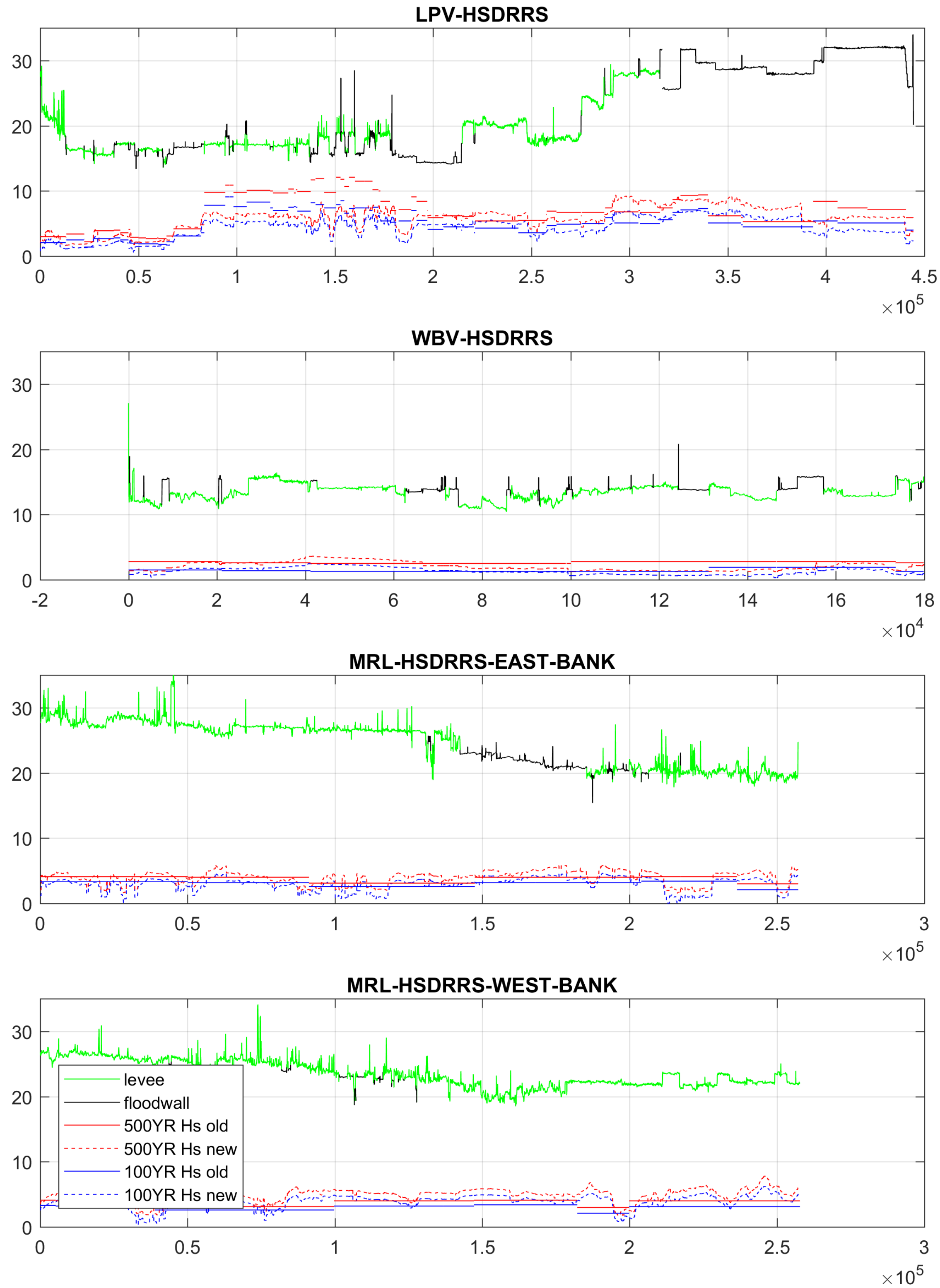

Figure 31. Comparison of new and old 100YR and 500YR significant wave height statistics

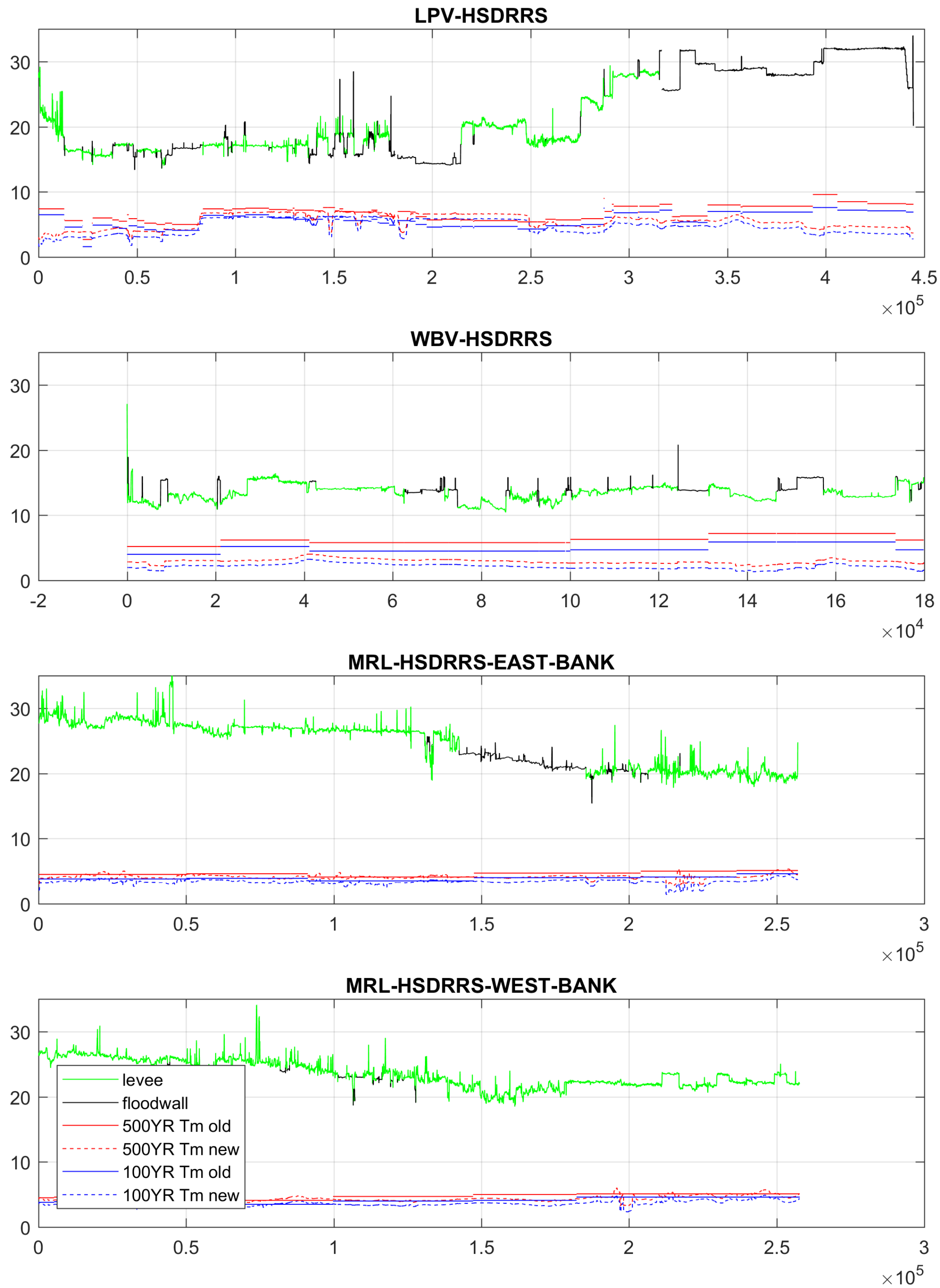

Figure 32. Comparison of new and old 100YR and 500YR mean wave period statistics

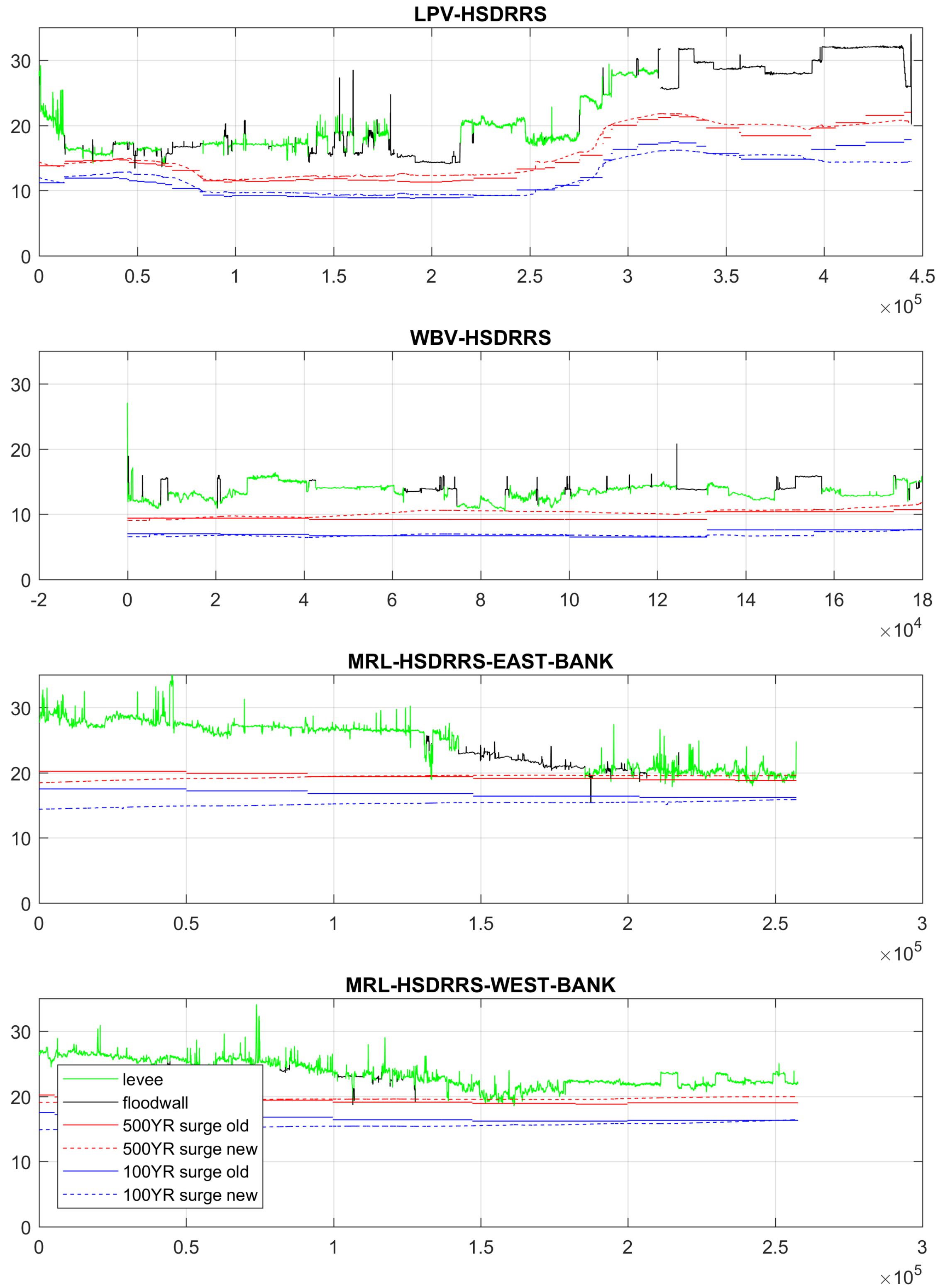

Figure 33. Comparison of new and old 100YR and 500YR still water level statistics with correction applied the Mississippi River surge statistics

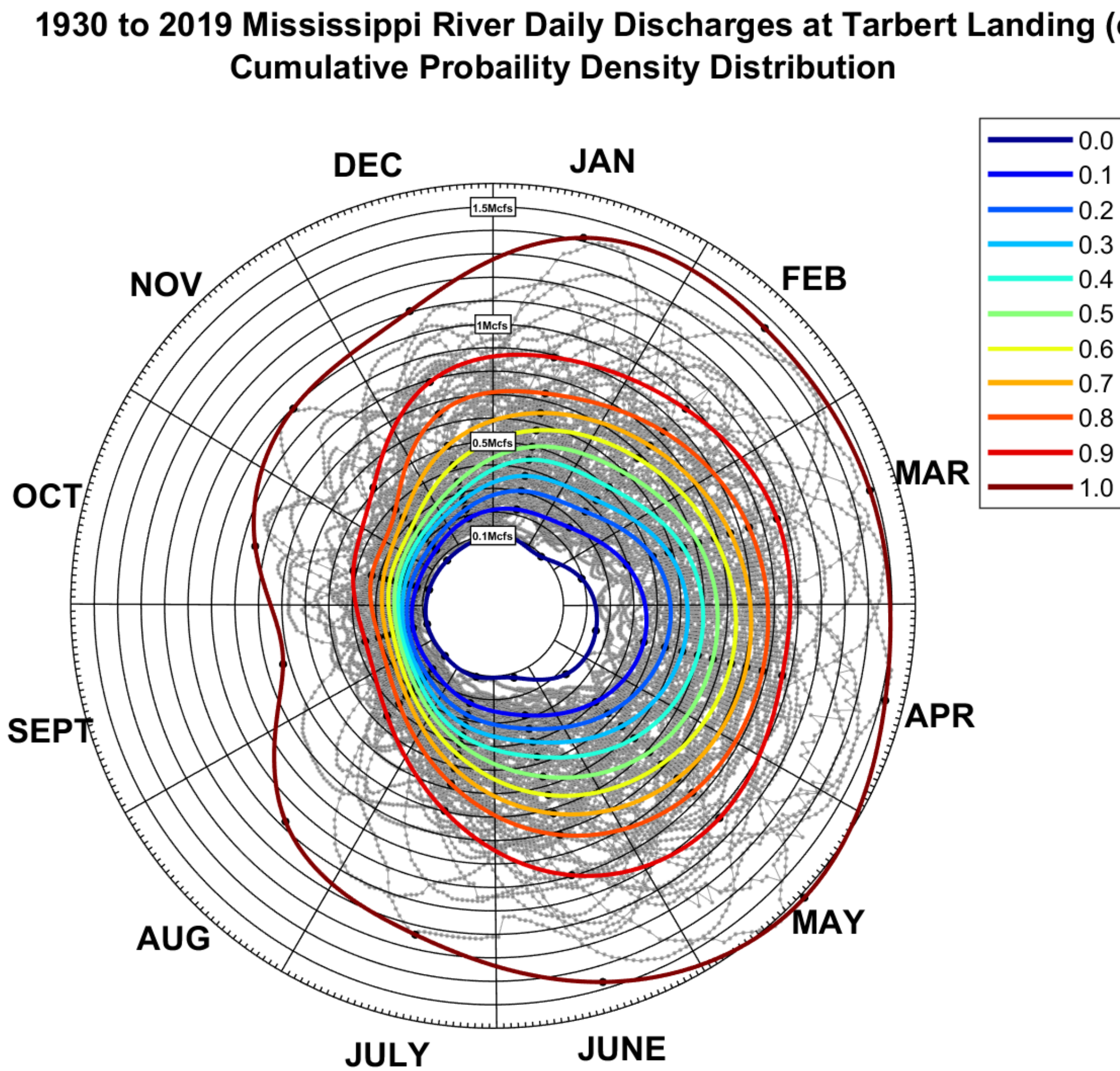

Figure 34. Cumulative probability density distribution of lower Mississippi River discharges and daily discharge observations (1930 to 2019)

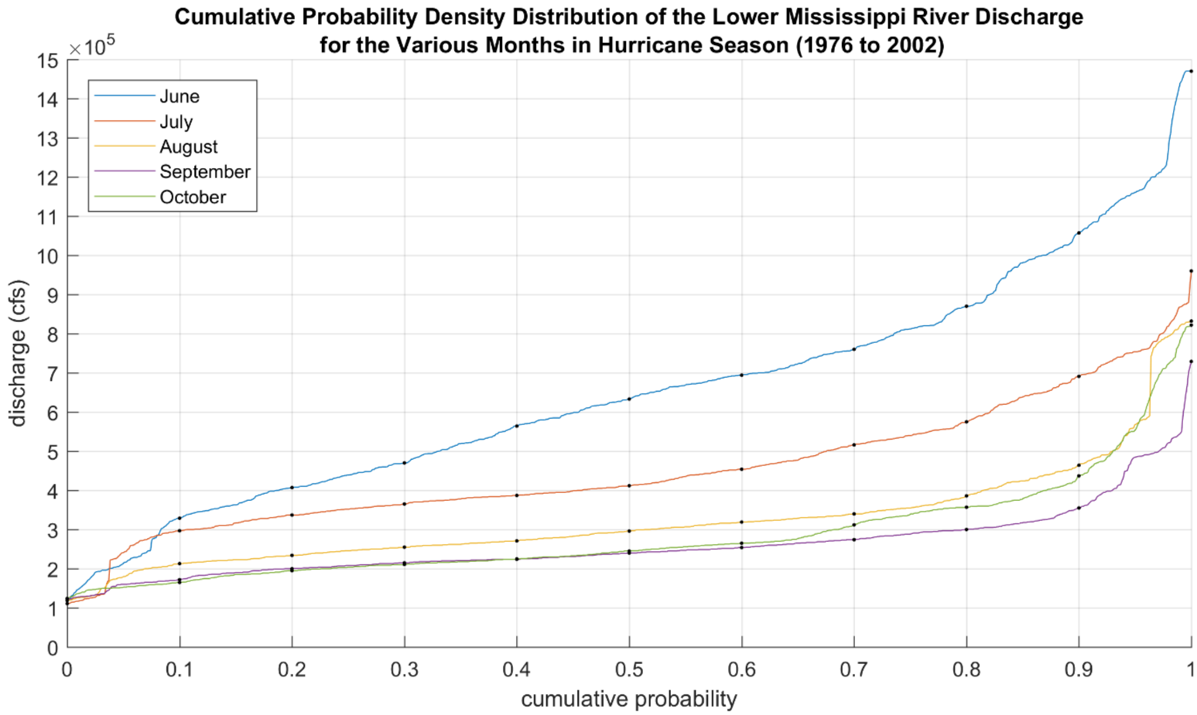

Figure 35. Cumulative probability density distribution of the lower Mississippi river during hurricane season (1976 to 2002 data)

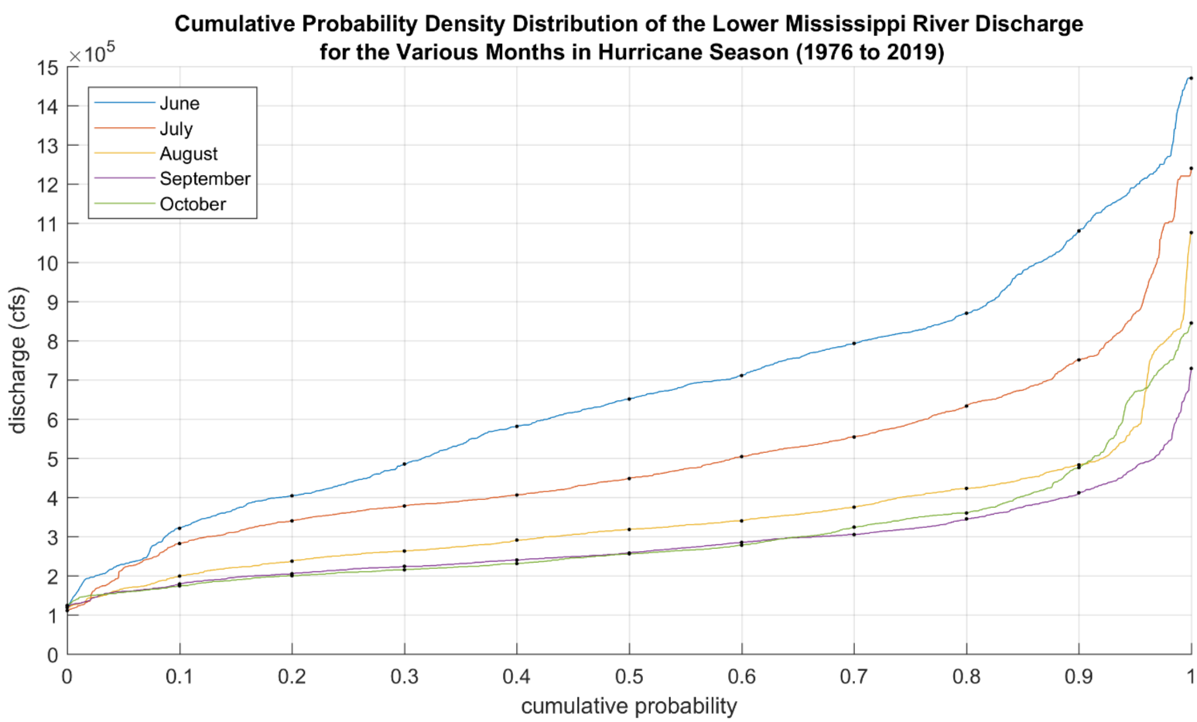

Figure 36. Cumulative probability density distribution of the lower Mississippi river during hurricane season (1976 to 2019 data)

	June	July	August	September	October	Total
No. of hurricanes	1	1	4	6	2	14
p(m)	1/14	1/14	4/14	6/14	2/14	1

Table 4. Probability density of hurricanes in various months based on hurricanes in the New Orleans areas in the period 1941 – 2005.

3.10 FUTURE CONDITIONS 2073 – WITH PROJECT

100YR and 200YR design elevations for the 2073 intermediate RSLC condition were determined using a Monte Carlo based overtopping tool developed with Matlab. The Monte Carlo approach is thoroughly documented in the original HSDRRS Design Elevation Report. Monte Carlo analysis is a statistical method to evaluate the probability distribution of a particular output parameter of concern, given uncertain input parameters. In this case, we are concerned about the overtopping rate of the levee or floodwall section, and we are uncertain about water levels, wave heights and wave periods, also known as the levee design hydraulic boundary conditions. The Monte Carlo analysis creates many different combinations of input parameters (water levels and waves) and estimates overtopping rate for each sample. Some input parameters such as levee elevation and slope are assumed to be constant in each iteration. The overtopping rates are estimated using the empirical Eurotop wave overtopping equations. The final product of the Monte Carlo simulation is a distribution of overtopping rates, including the 50% and 90% non exceedance overtopping rates (q-50 and q-90).

The overtopping formulae used in the Monte Carlo scripts have been updated to use equation 5.10, 5.11 and 5.17 (Figure 7 and Figure 8) from the Eurotop manual. The updated Monte Carlo code output was compared to the example output provided in the design elevation report. The comparison shows the updated overtopping functions do not have a tremendous effect on final required design elevation for the segment evaluated. The original DER provided a required elevation of 16.5ft NAVD88 for segment JL01, while the updated script provided 16.0ft NAVD88. Figure 37 displays an example of the new Monte Carlo output for section JL-01 assuming the same hydraulic boundary conditions applied in the original DER. Figure 38 displays the output from the original code.

1% design elevations were determined for the entire HSDRRS perimeter using an automated version the Monte Carlo based design script. Figure 39 and Figure 40 display the 2073 required 100YR and 200YR levee and floodwall elevations for the intermediate RSLC scenario. The required design elevations should be considered as a rough estimate. Further site-specific analysis might refine the required design elevations. The future 2073 required design elevations were provided to the PDT.

The "cross-over" points are the locations where the MR&T design grade intersects the hurricane design grades for the MRL co-located levees and floodwalls. The location of the cross-over points along the MRL were determined to be river mile 90.5 for the east bank and river mile 95.5 for the west bank for intermediate RSLC projections (RSLC=1.8ft) for the 100YR design. The location of the cross-over points along the MRL were determined to be river mile 92.5 for the east bank and river mile 96.5 for the west bank for intermediate RSLC projections (RSLC=1.8ft) for the 200YR design.

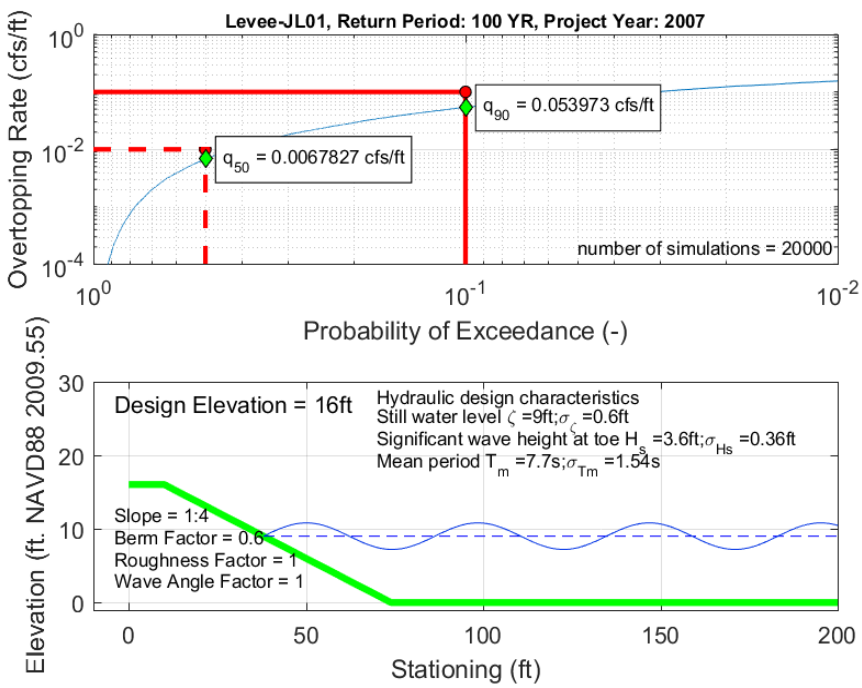

Figure 37. Example of Monte Carlo output for the Jefferson Lakefront levee JL01 from updated code.

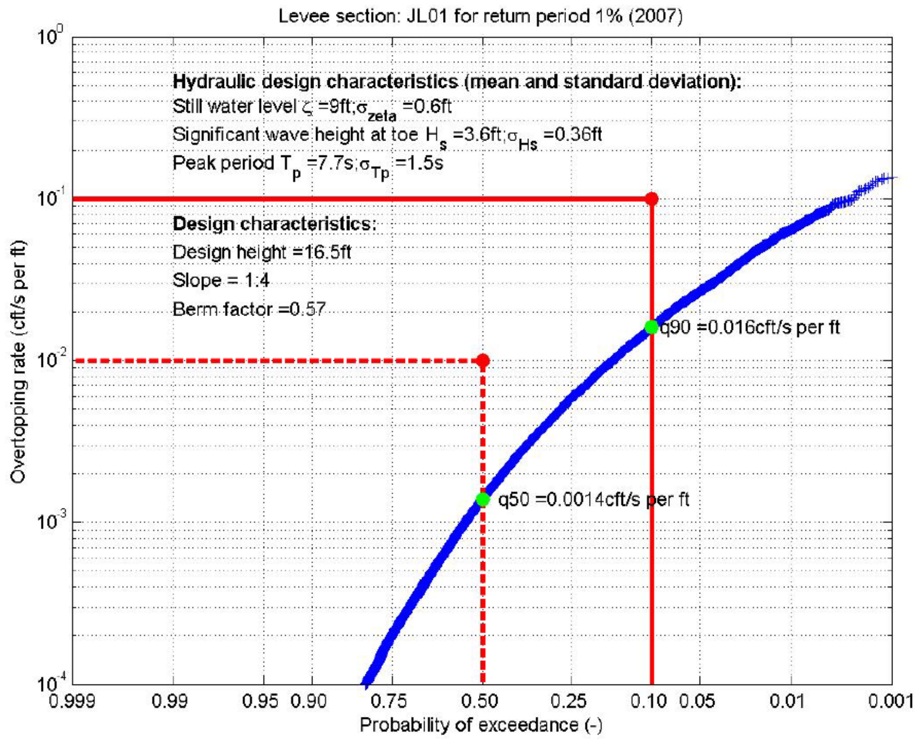

Figure 38. Example of Monte Carlo output for the Jefferson Lakefront levee JL01 from 2007 code.

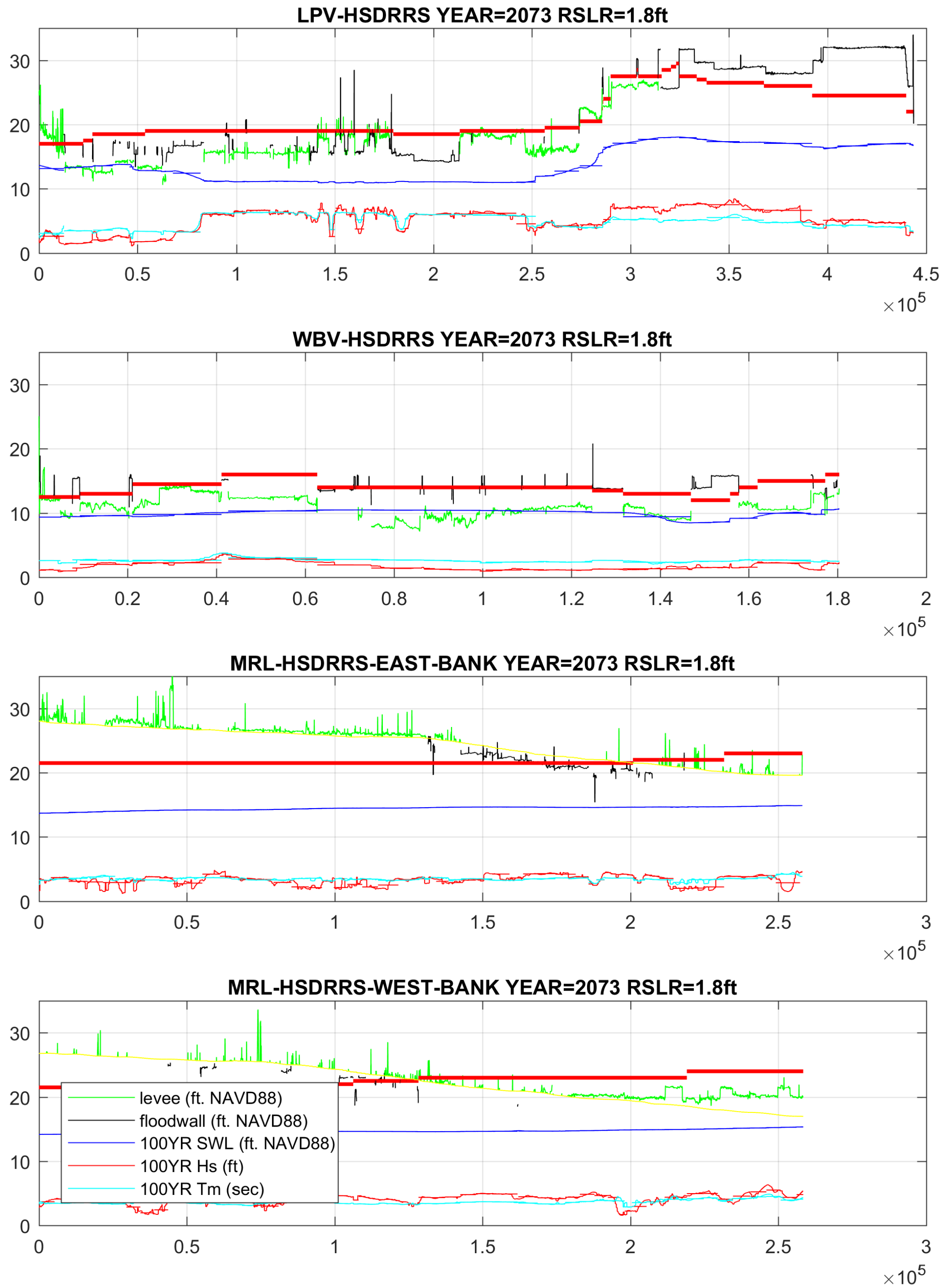

Figure 39. 2073 100YR required design elevations and still water level (SWL), significant wave height (Hs) and mean wave period (Tm) for intermediate RSLC scenario.

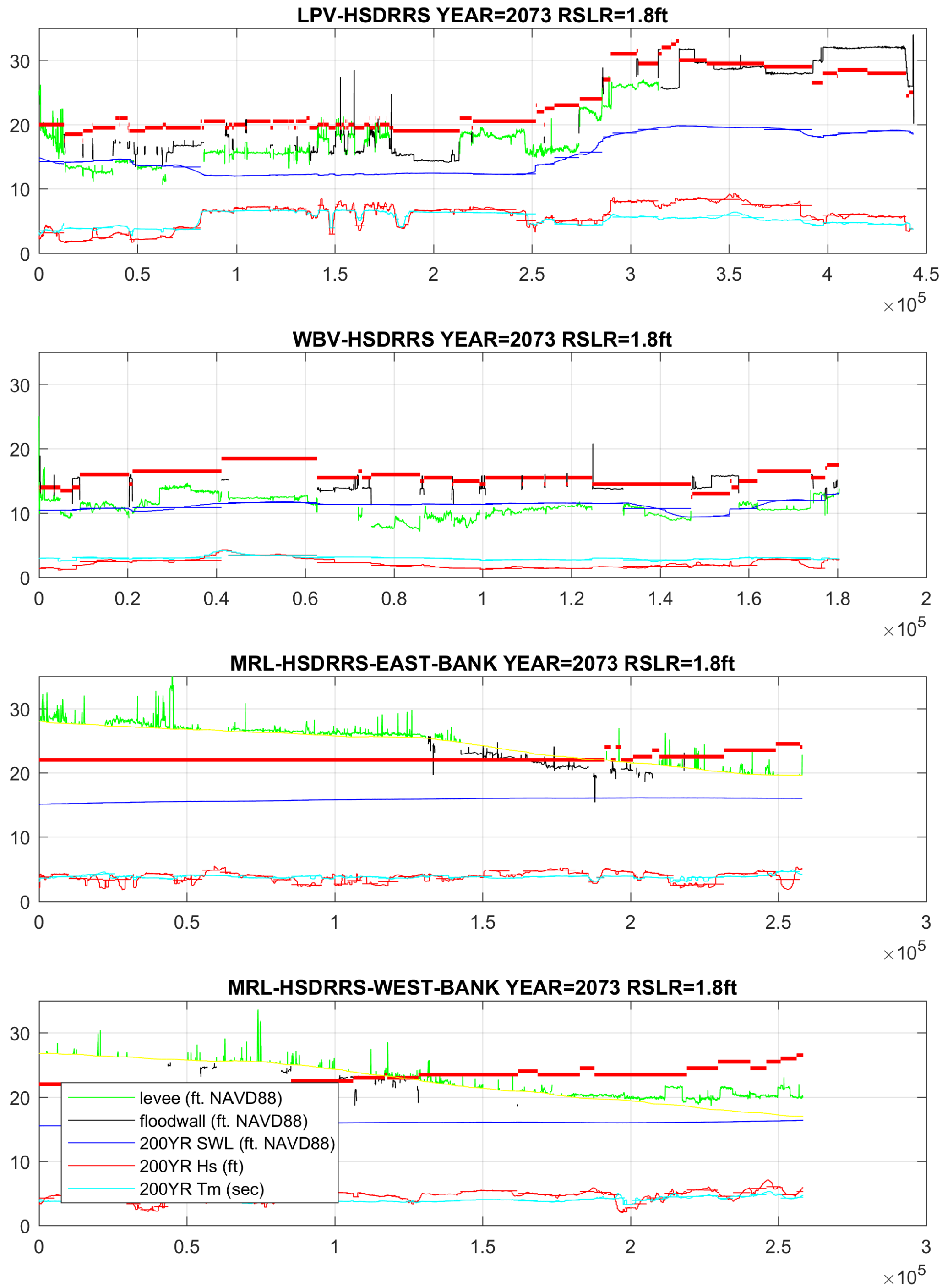

Figure 40. 2073 200YR required design elevations and still water level (SWL), significant wave height (Hs) and mean wave period (Tm) for intermediate RSLC scenario.

The overtopping and RAS simulations were conducted for the with-project condition. Figure 41 displays the resulting 100YR inundation for the future with-project condition for the intermediate RSLC scenario (RSLC=1.8ft). The levee and floodwall lifts delivered with the 2073 100YR system prevent the massive inundation estimated in the without-project condition, as presented in Figure 20. 500YR with-project inundation is presented in Figure 42 for the intermediate RSLC scenario. The 100YR system still allows some inundation within the polder for the 500YR event, but it is significantly less than the without project condition. Figure 43 and Figure 44 display the 100YR and 500YR flood depths for the with project condition (100YR HSDRRS) assuming intermediate RSLC conditions.

Figure 45 and Figure 46 display the 100YR and 500YR inundation for the with-project (200YR HSDRRS) condition for the intermediate RSLC scenario (RSLC=1.8ft).

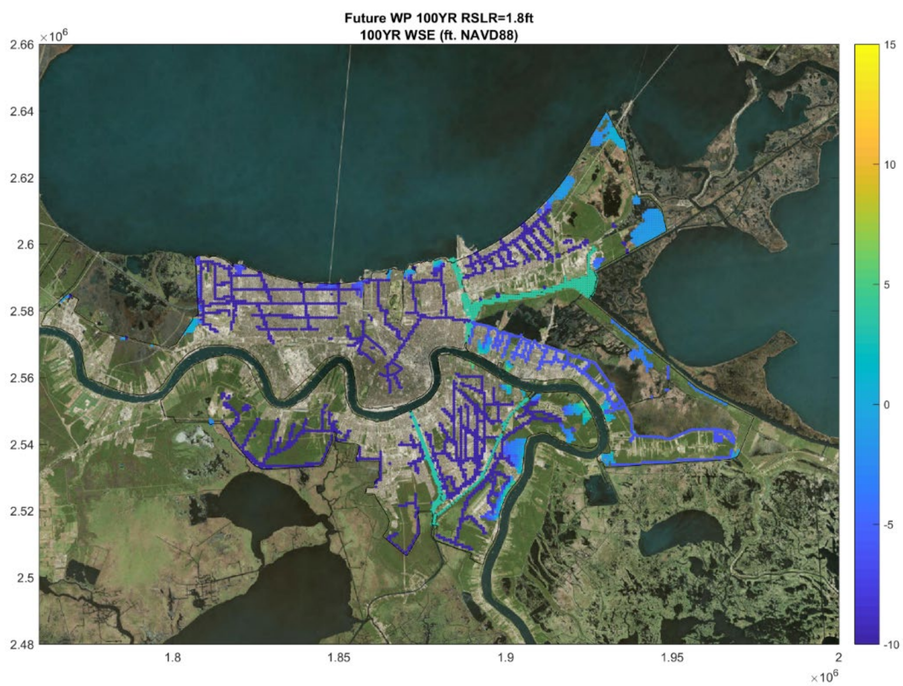

Figure 41. 100 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

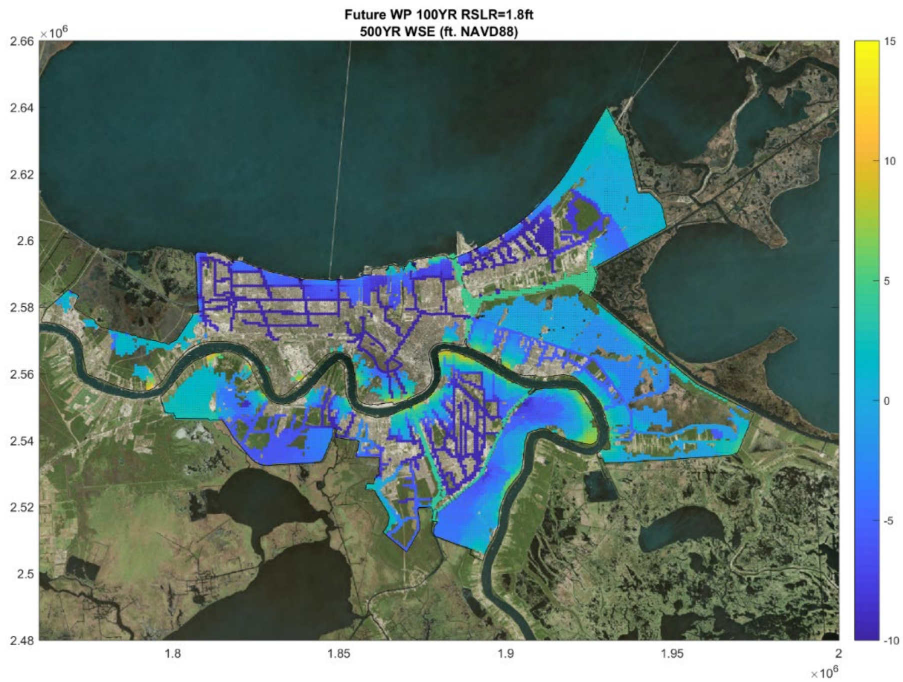

Figure 42. 500 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

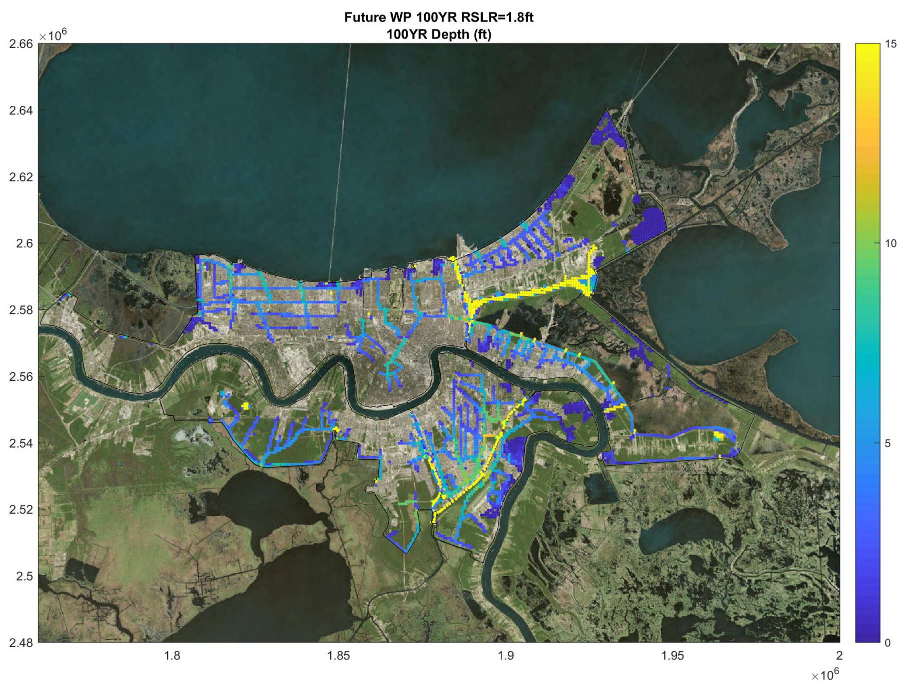

Figure 43. 100 year peak depth (ft.) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

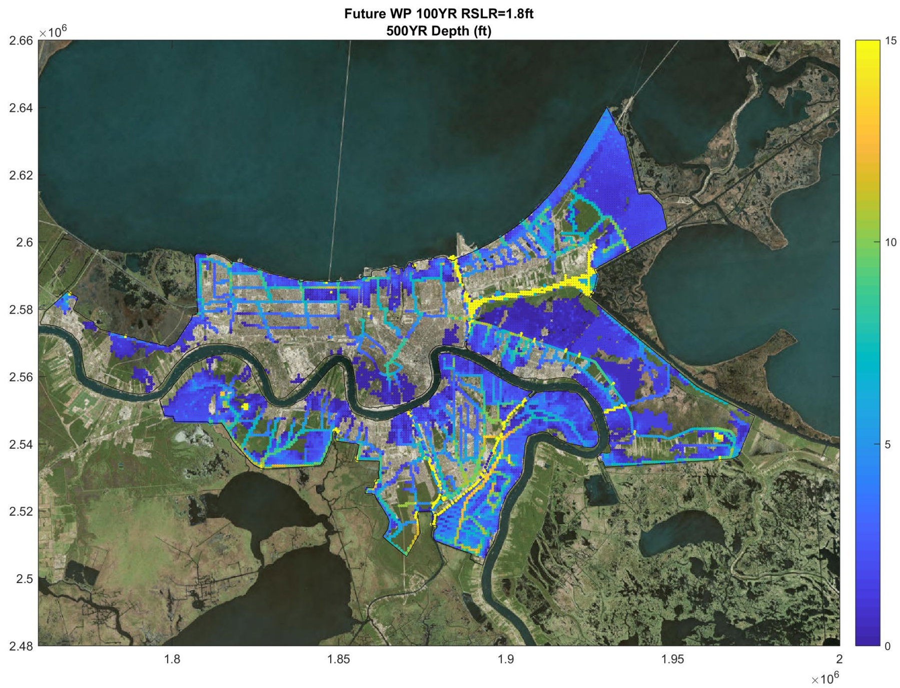

Figure 44. 500 year peak depth (ft.) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

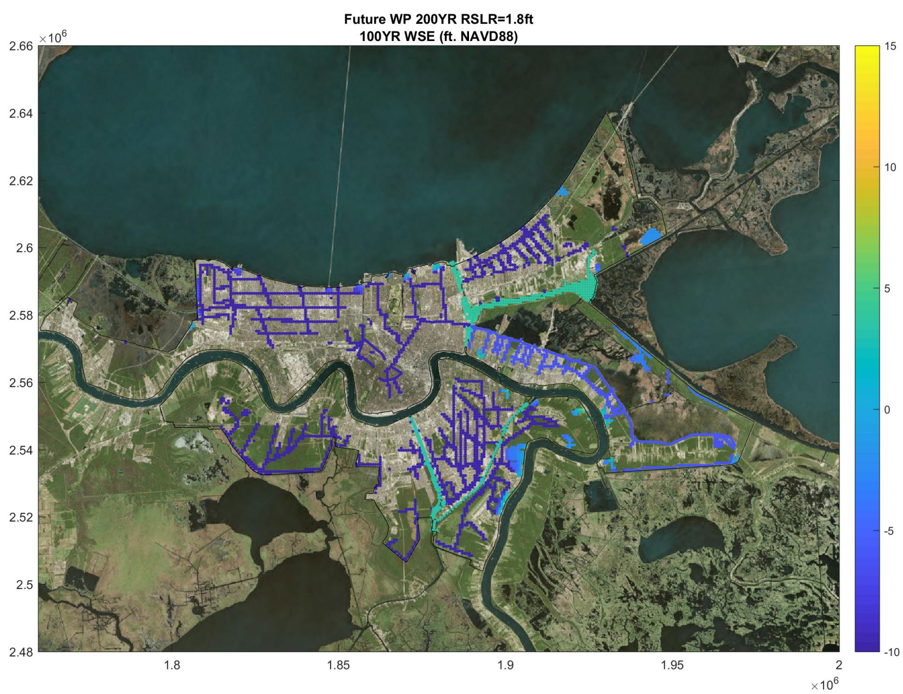

Figure 45. 100 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 200YR HSDRRS

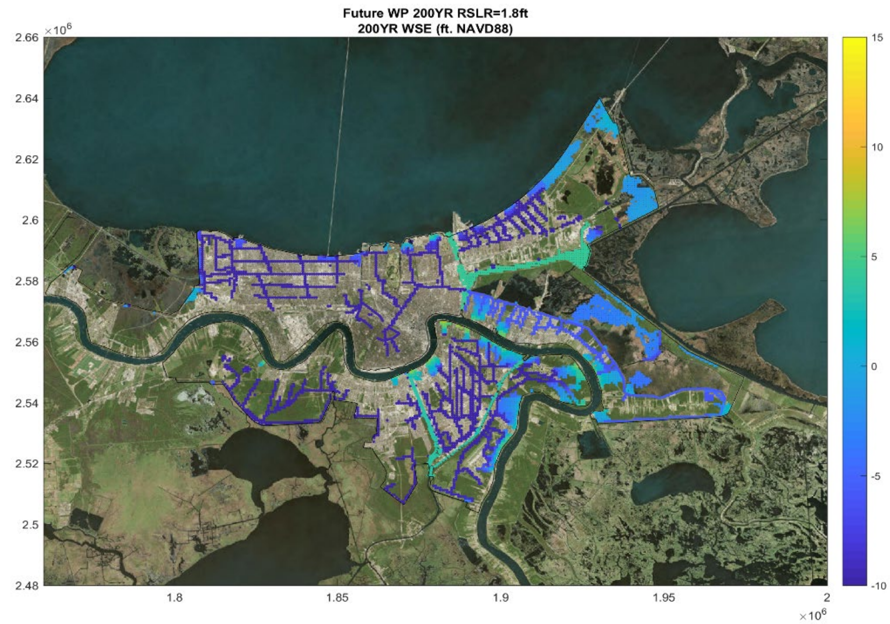

Figure 46. 200 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 200YR HSDRRS

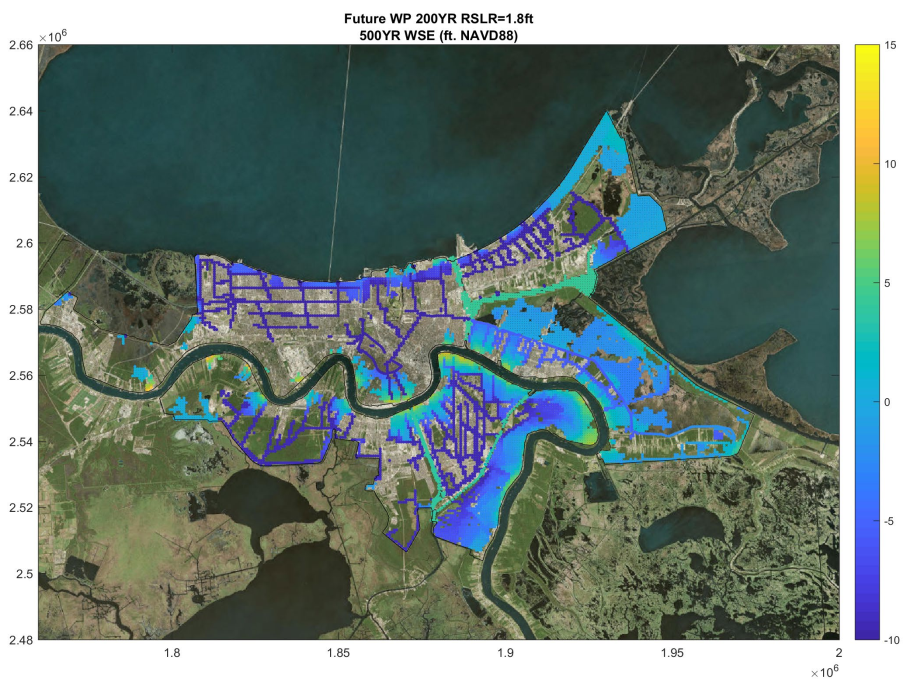

Figure 47. 500 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 200YR HSDRRS

4 ASSUMPTIONS AND LIMITATIONS

HEC-RAS MODEL

- The HEC-RAS polder model used in this analysis was not validated with hurricane Katrina. Katrina would be the only storm available for validation of such a model.
- The pump station flows are controlled by the rating curve. In reality, the flow is governed by the interior and exterior stage and the specific pump-efficiency curve for each station. The modeling also assumes all pump stations will be in operation and achieve full capacity.
- No rainfall time-series are available for the 152 synthetics storms. Rainfall was not included in the HEC-RAS simulations
- Sub-surface drainage features were not accounted for in HEC-RAS geometry. Subsurface drainage would likely have a small effect during large overtopping events.
- No breaching or floodwall failures was accounted for in the HEC-RAS modeling. Breaching would make the inundation potentially much worse for certain storms.

OVERTOPPING CALCS

- The water levels, significant wave heights, and wave periods used in the overtopping calculations are based on the results of the 2017 CPRA surge and wave modeling. An updated surge hazard analysis is currently being developed by CPRA, ERDC and other agencies. New surge and wave estimates might be different than the values developed for this study.
- The wave overtopping calculations are based on the average discharge equations. A more conservative equation could be used. Wave overtopping is a significant component of total overtopping volume for certain storms. For storms with free flow overtopping, wave overtopping is less significant in the total overtopping volume.
- The overtopping calculations and resulting inundation estimates are all 50% or average value deterministic estimates. The uncertainty in water levels was not evaluated in the overtopping and inundation calculations. 90% estimates of inundation will be significantly higher.
- The exterior water levels assumed in the overtopping calculations are not effected by volume lost to overtopping. In reality, there may be a drawdown effect on the exterior once a levee is overtopped. The modeling assumes that any volume lost to the polder interior is replaced by the storm.
- The exterior water levels assumed for the with project overtopping and design calculations are assumed to not be effected by the with project levee lift. In reality, a raised levee will prevent inundation in the interior and amplify exterior water levels. Without additional with and without project ADCIRC simulations, it is impossible to determine this amplification effect.
- The surge and wave time-series assume coincident peaks. In reality, the timing of peak surge and wave may not correspond exactly.
- Wave direction is assumed to be perpendicular at all time-steps.

WATER LEVEL STATISTICS

- Interior water level statistics were computed with the latest JPM-OS code from ERDC. The code was applied as-is with no modification or verification.
- No estimate of uncertainty is provided in the surge statistics. The results of the statistical code are 50% or average value. 90% statistics would be significantly higher.