2021

West Bank & Vicinity GRR
Appendix C – Hydrology and Hydraulics

US Army Corps of Engineers®
New Orleans District

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APPENDIX C – HYDROLOGY AND HYDRAULICS

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 (HSDRRS) for with and without project scenarios. The 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 Levees (MRL) project. Interior flooding estimates are produced for the 20YR, 50YR, 100YR, 200YR, 500YR and 1000YR surge events for existing conditions (year 2023) and future conditions (year 2073). Three future 2073 conditions are evaluated for low, medium and high relative sea level change (RSLC) projections. As described in the study authorization, one project alternatives is evaluated which is 100YR perimeter system. The 100YR HSDRRS ensures the expected overtopping rate at any given levee or floodwall segment is less than 0.1 cfs/ft with 90% confidence less than 0.01cfs/ft with 50% confidence for a 100YR surge and wave event. Interior flood risk varies tremendously by location and a 100YR perimeter system may not guarantee 100YR project performance at every location within the system. Furthermore, the 100YR perimeter system does not reduce the risk associated with rainfall flooding.

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 polders 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 water level statistics code.

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

3 LPV/WBV INTERIOR FLOODING ASSESSMENT

3.1 OVERVIEW

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 impossible to model the entire system properly with a single model. The 1D HEC-RAS modeling 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.

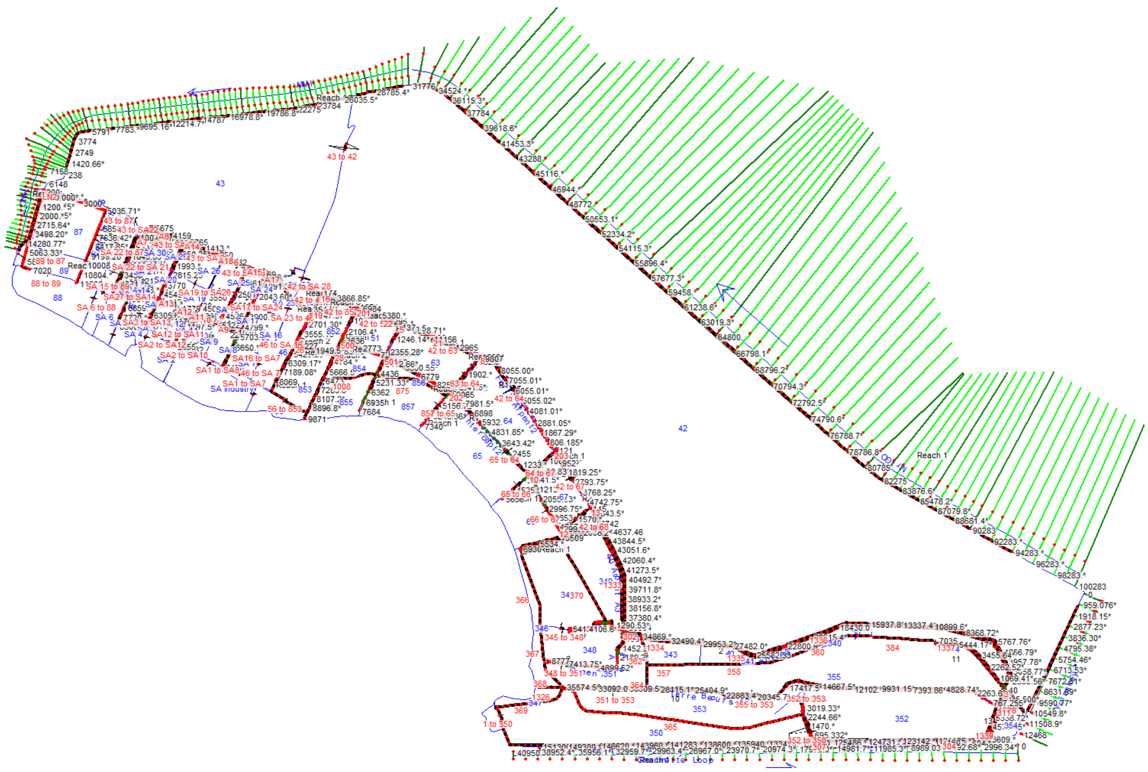

Figure 1. Example of polder model HEC RAS 1D geometry from post-Katrina study

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, including a 1D/2D HEC-RAS model of the Orleans Metro Polder developed by Saint Paul District in 2018.

Separate 2D meshes were created for each sub polder. The LPV includes 2D meshes for St Charles, Orleans and Jefferson Parish east bank, the IHNC Corridor, New Orleans East, and St. Bernard Parish. The WBV includes 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 levee/floodwall 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, while still providing realistic results for large scale overtopping and inundation events.

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.

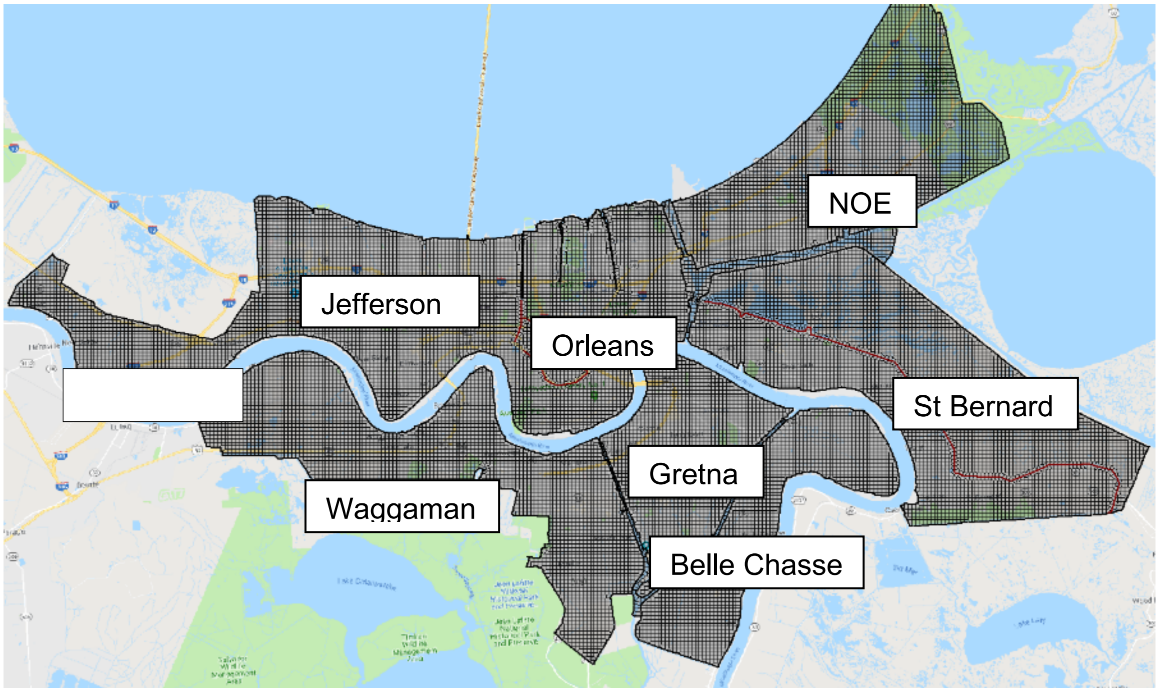

Figure 2. HEC-RAS computational mesh

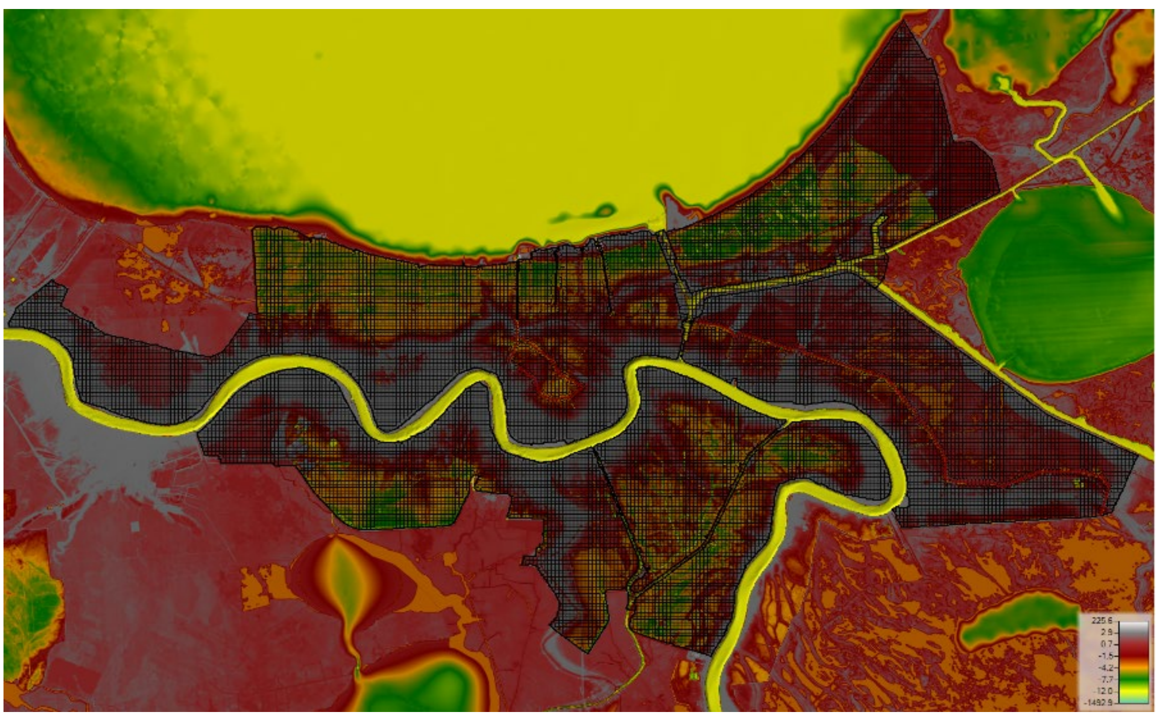

Figure 3. HEC-RAS computational mesh and terrain (ft. NAVD88)

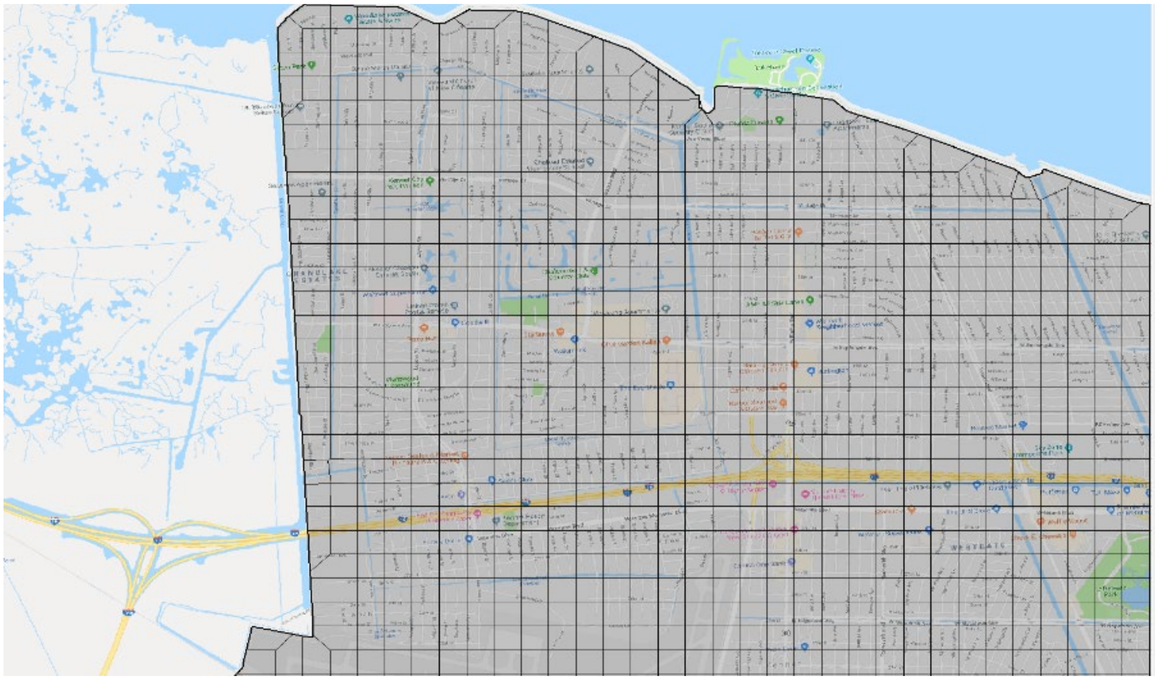

Figure 4. HEC-RAS computational mesh for HSDRRS interior near Kenner, LA

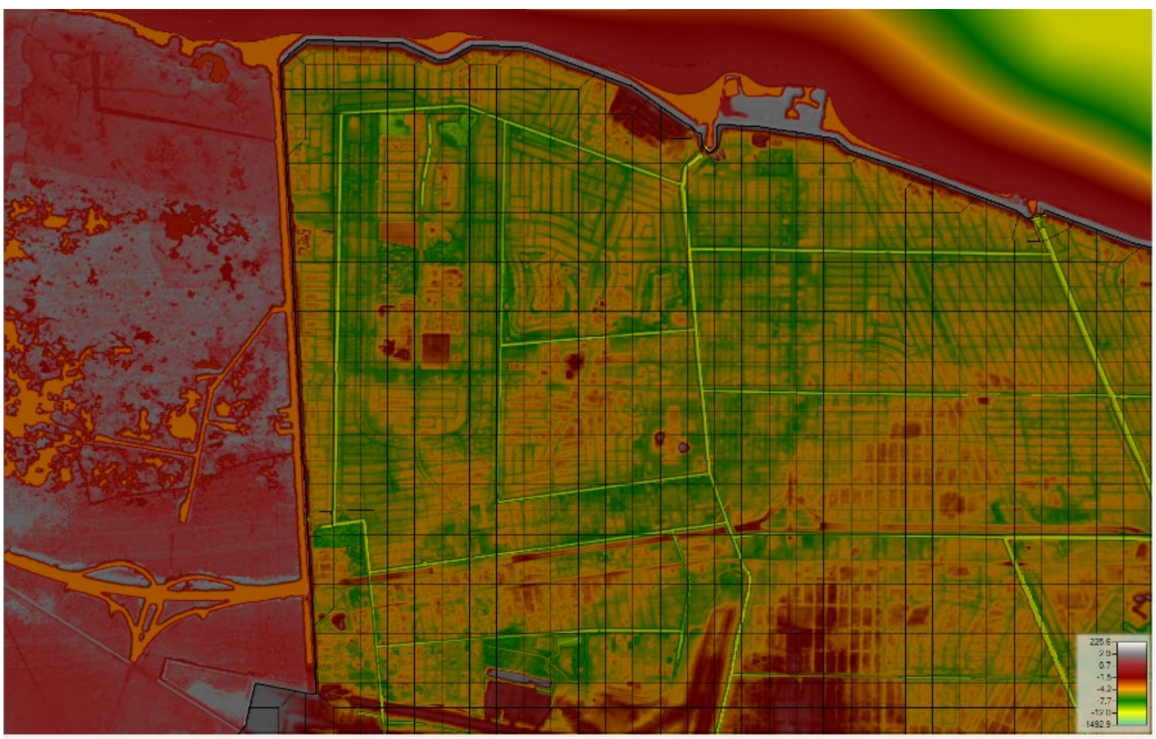

Figure 5. HEC-RAS computational mesh and terrain at HSDRRS interior near Kenner, LA (ft. NAVD88)

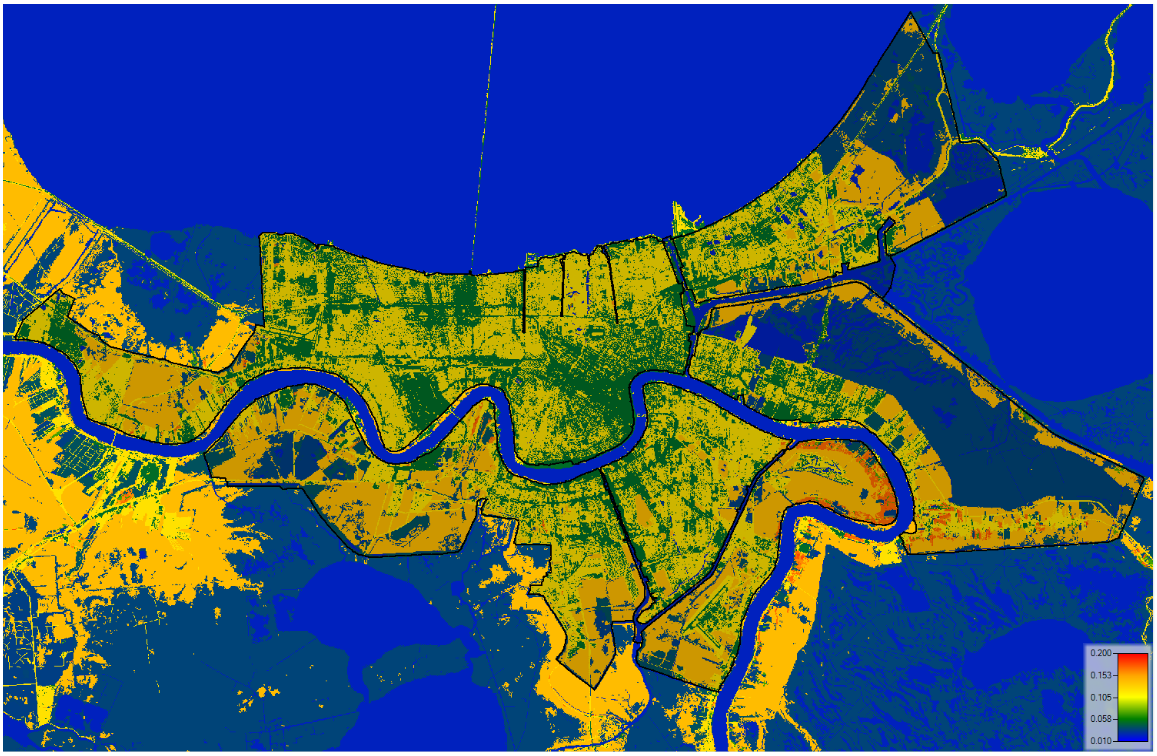

Figure 6. HEC-RAS Manning's n values

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

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

3.3 HEC-RAS MODEL VALIDATION

The HEC-RAS 2D model was validated by simulating hurricane Katrina for the Orleans Metro and Jefferson Parish portion of the model geometry. During Katrina, interior floodwalls along the 17th Street Canal, London Canal and the western side of the IHNC were breached, allowing a tremendous volume to inundate the Jefferson Parish and Orleans Metro polder. Data from a separate HEC-RAS analysis conducted by Saint Paul District (MVP) was utilized in the latest simulation of Hurricane Katrina. The MVP model estimated the inflow into the polder by modeling the breaches using lateral structures with a specified breach width, invert and timing of failure. The MVP model setup produced realistic results of the inundation within the polder. All pump flow time-series, breach locations and widths, breach timing, observed high water marks, rainfall, and other model assumptions were consistent with information from the Interagency Performance Evaluation Taskforce (IPET) report. To validate the HEC-RAS 2D model used in the latest analysis, flows at each breach and pump location were extracted from the MVP model and applied at the boundary of the latest 2D mesh. Given that the boundary conditions are nearly equivalent, the latest simulation produced very similar results to the MVP model. The simulation shows that the latest 2D interior model produces realistic results when accurate inflows/outflows are applied at the model boundary.

Figure 7 displays a map with the Orleans Metro Polder divided into separate polygons. Each polygon contains observed high water mark data used in the validation of the model. Table 2 contains the comparison of observed and modeled high water mark data for the Orleans Metro Polder for Hurricane Katrina. Four separate model runs were compared. The first simulation is an early 1D model developed around the time of IPET using HEC-RAS 3.2. The second simulation is the MVP 1D/2D model developed in 2018 using HEC-RAS 4.0.2. The third simulation is the latest 2D polder model using HEC-RAS 5.0.7. The fourth simulation the latest 2D polder model without rainfall. The comparison of model to measurements shows that all simulations provide realistic water surface elevations and inundation extents. When rainfall is removed from the simulation, the water levels drop a few tenths of a foot in some areas, and drop by roughly 1ft in others. For the Katrina simulation, a single rainfall time-series was applied for the entire 2D mesh. It is unclear how realistic this assumption is given the wide spatial variability of rainfall during hurricanes. Despite totaling approximately 11.5 inches over a 24-hr period, removal of the rainfall does not significantly alter the validation of the model for this particular storm. For other storms, rainfall might be more significant. Figure 8 and Figure 9 display the maximum water surface elevation from the simulation of Hurricane Katrina for the 2018 MVP model and the latest 2020 HEC-RAS model. The comparison of model results shows very similar flood extents and elevations.

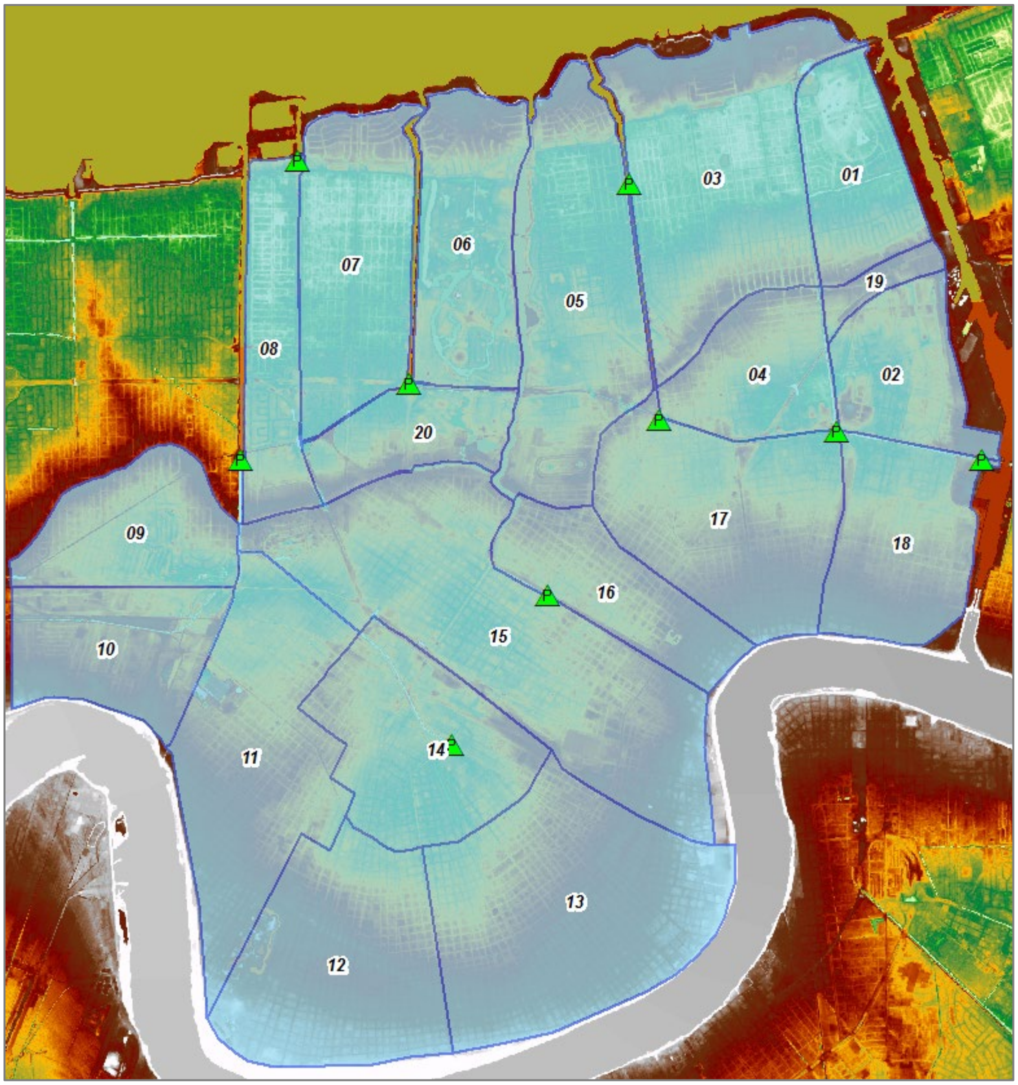

Figure 7. Orleans metro polder divisions used in high water mark comparison

Table 2. Comparison of observed and modeled high water mark data for Hurricane Katrina for Orleans Metro Polder

Storage Area	Measured High Water (ft, NAVD88)			HEC-RAS 3.2 Beta (1D Model) Calculated High Water (ft, NAVD88)	HEC-RAS 5.0.4 (1D/2D Model) Calculated High Water (ft, NAVD88)	HEC-RAS 5.0.7 (2D Model) Calculated High Water (ft, NAVD88)	HEC-RAS 5.0.7 (2D Model Without Rain) Calculated High Water (ft, NAVD88)
	# of HWM	Average	Range				
1	1	2.6	2.6	2.8	2.7	3.3	2.6
2	5	4.7	3.4 – 5.3	4.9	4.3 – 10.9	4.8	4.5
3	3	2.9	2.2 – 3.3	3.0	3.5	3.3	2.6
4	3	3.9	3.8 – 4.0	3.6	3.7 – 4.3	3.9	3.5
5	3	3.1	3.0 – 3.2	3.0	3.4	3.3	3.0
6	2	3.2	3.2 – 3.3	3.5	3.4	3.3	3.0
7	3	3.7	3.6 – 3.8	3.7	3.7	3.3	3.0
8	1	3.8	3.8	3.7	3.7 – 3.8	3.3	3.0
9	2	2.8	2.8	2.7	2.9	3.3	2.2
10	0			2.7	2.9	3.3	2.2
11	4	3.0	2.9 – 3.1	3.0	2.9	3.3	2.2
12	0			3.0	2.9	3.3	2.2
13	6	2.6	2.7 – 2.8	3.0	2.9	3.3	2.3
14	6	2.9	2.8 – 3.0	3.0	2.9	3.3	2.3
15	9	2.8	2.3 – 3.0	3.0	3.0	3.3	2.3
16	1	2.9	2.9	3.0	3.0	3.3	2.7
17	7	3.3	3.0 – 4.0	3.6	3.3 – 4.6	3.5	3.4
18	7	4.7	2.4 – 5.7	4.9	4.6 – 9.5	5.5	5.4
19	0			3.1	3.5 – 4.2	4.5	4.3
20	1	2.5	2.5	3.5	3.4	3	2.9

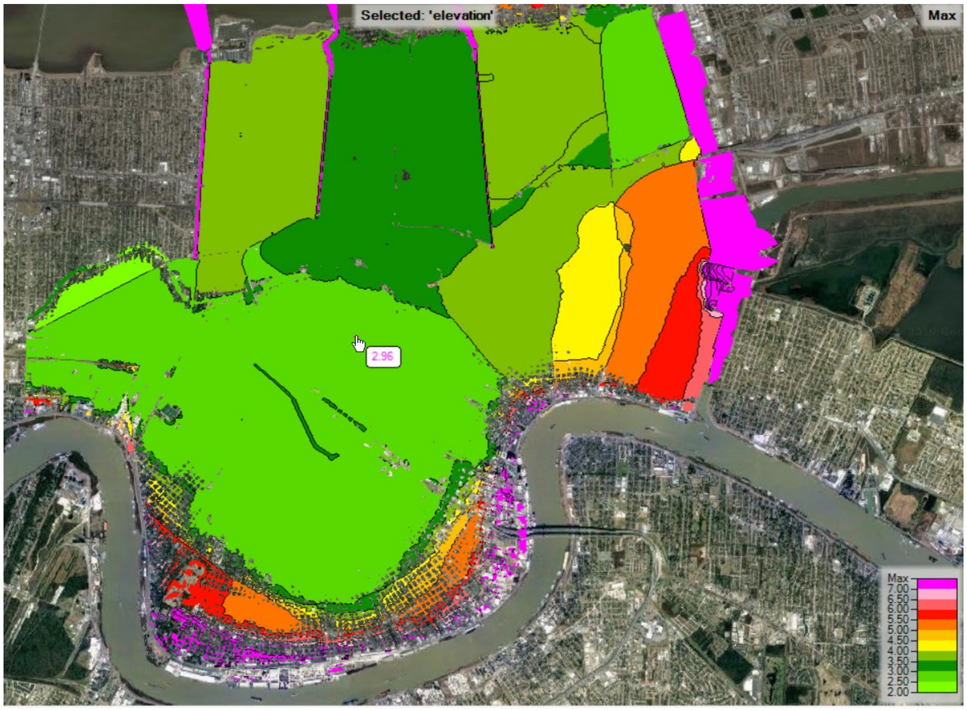

Figure 8. Hurricane Katrina maximum water surface elevation from 2018 MVP HEC-RAS 1D/2D model

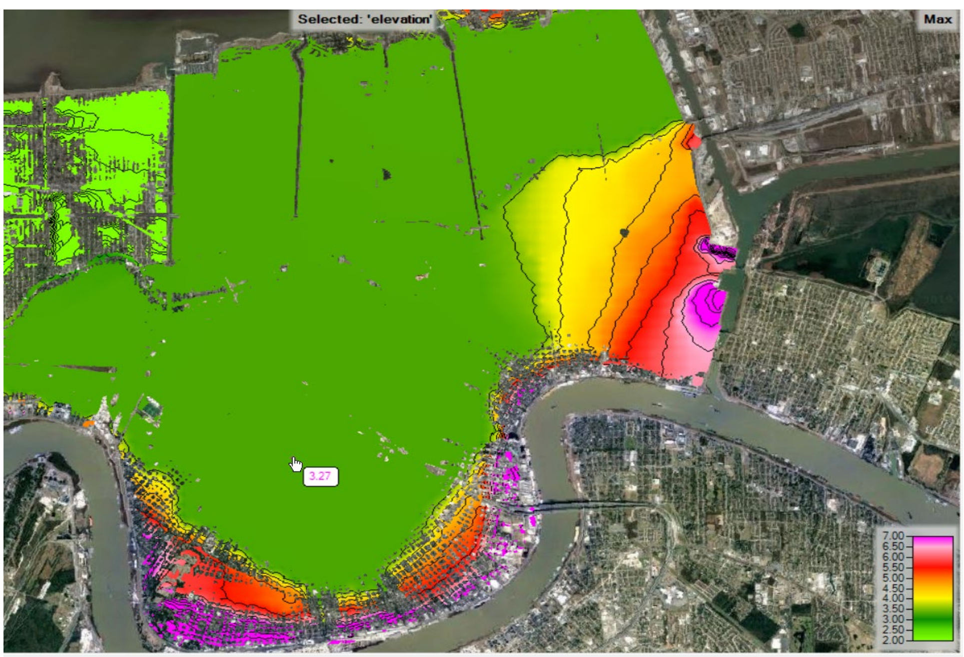

Figure 9. Hurricane Katrina maximum water surface elevation from 2020 MVN HEC-RAS 2D model

3.4 LEVEE SURVEYS, LIDAR, CHANNEL BATHYMETRY, PUMPS

The Corps collected comprehensive elevation surveys of all HSDRRS perimeter levees in 2019. No floodwalls were included in the latest survey. All floodwall elevations were assigned based on the NCC surveys. The perimeter levee and floodwalls are not incorporated into the HEC-RAS 2D geometry, but are instead used in overtopping calculations. Elevation profiles for the storage area connections, which allow polder to polder flow, 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 area 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 into Harvey and Algiers canals. The rating curve approach to modeling pump-stations does not account for decreased pump flow during high head scenarios. The approach taken with the modeling allows somewhat more water to be removed from the system that would occur in reality during a surge overtopping event. Figure 10 displays the locations and total capacities (cfs) of pump-stations within the HSDRRS.

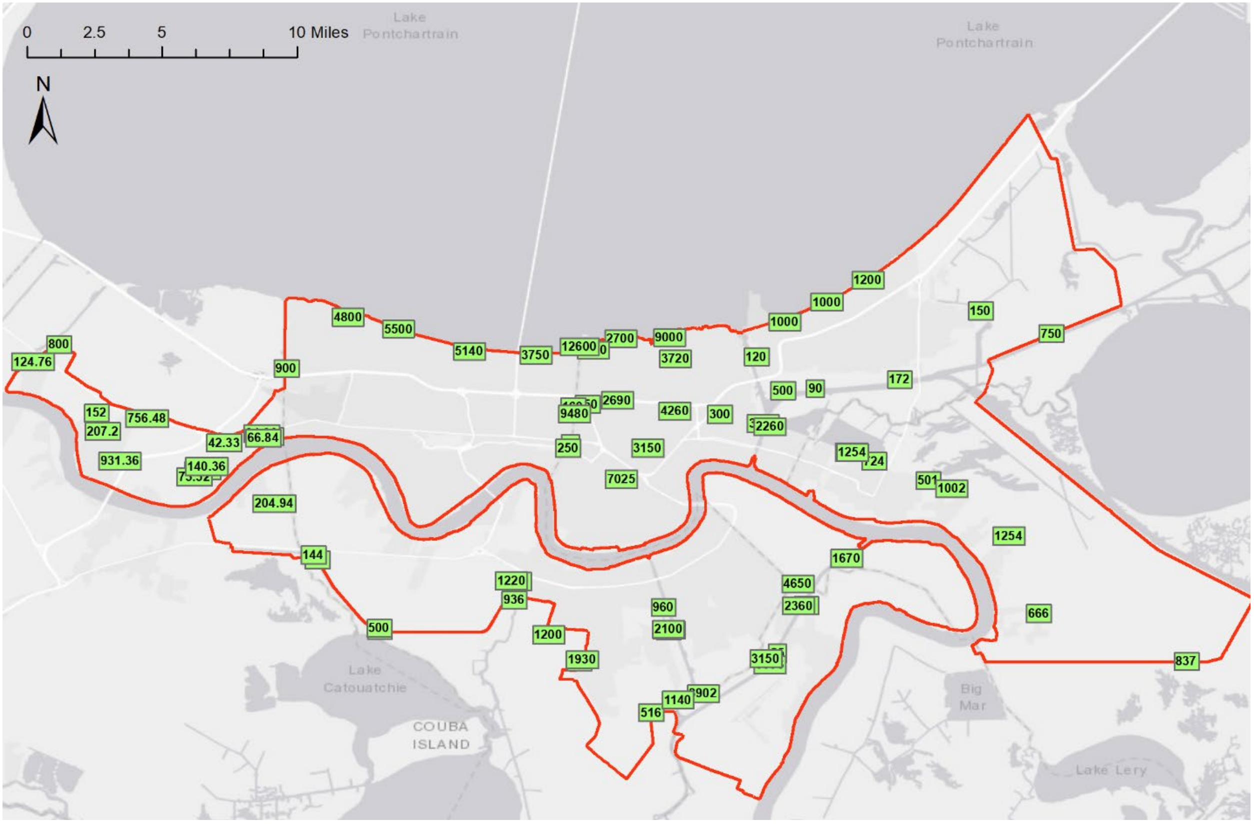

Figure 10. Major pump stations within HSDRRS

3.5 OVERTOPPING FLOW BOUNDARY CONDITIONS AND INITIAL CONDITIONS

Overtopping rates were calculated at the HSDRRS perimeter and applied as boundary conditions for the HEC-RAS 2D model. As part of the design of HSDRRS, the system was divided into 427 hydraulic design segments. Each segment has unique levee or floodwall geometry and hydraulic boundary conditions including still water elevation (swe), significant wave height (Hs) and mean wave period (Tm). The latest version of the design segment shapefile was extracted from EGIS for LPV/WBV as well as the co-located MRL. In total, 427 segments are processed with a series of Matlab scripts that calculate overtopping time-series at each location for all 152 synthetic storms.

ADCIRC Hydrographs for all 152 synthetic storms were extracted at each segment using a Matlab script. The ADCIRC surge hazard dataset used is from the 2017 CPRA master plan. The levee heights and alignments applied in the 2017 CPRA ADCIRC mesh provide a decent representation of the existing 2020 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 427 segment profiles.

Wave overtopping rates for levees were calculated using the equations 5.10 and 5.11 provided in Eurotop overtopping manual (Figure 11). 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 elevation is less than the crest elevation, wave overtopping formuale are used. If the surge elevation is greater than the crest elevation, the Eurotop recommended 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 427 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.

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

with a maximum of

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

Figure 11. Eurotop wave overtopping formulae for levees

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

Figure 12. Eurotop wave overtopping formula for vertical wall

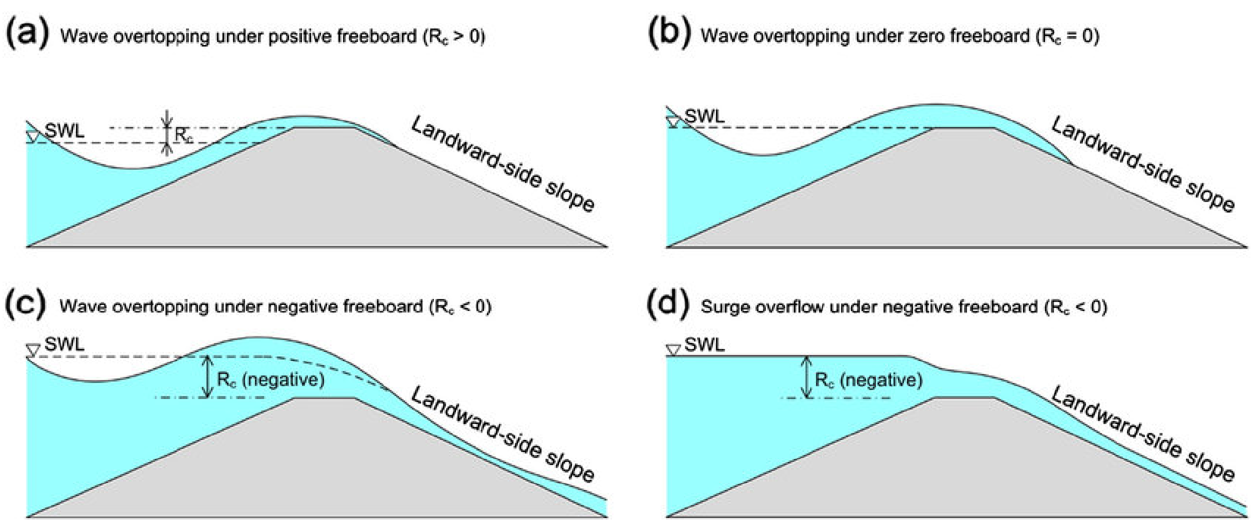

Figure 13 Wave overtopping for positive and negative free-board conditions.

Figure 14. HEC-RAS flow boundary segments (81 total segments

Figure 15 displays the levee and floodwall survey elevations for the entire HDSRRS perimeter taken from the 2019 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 15 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 3 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.

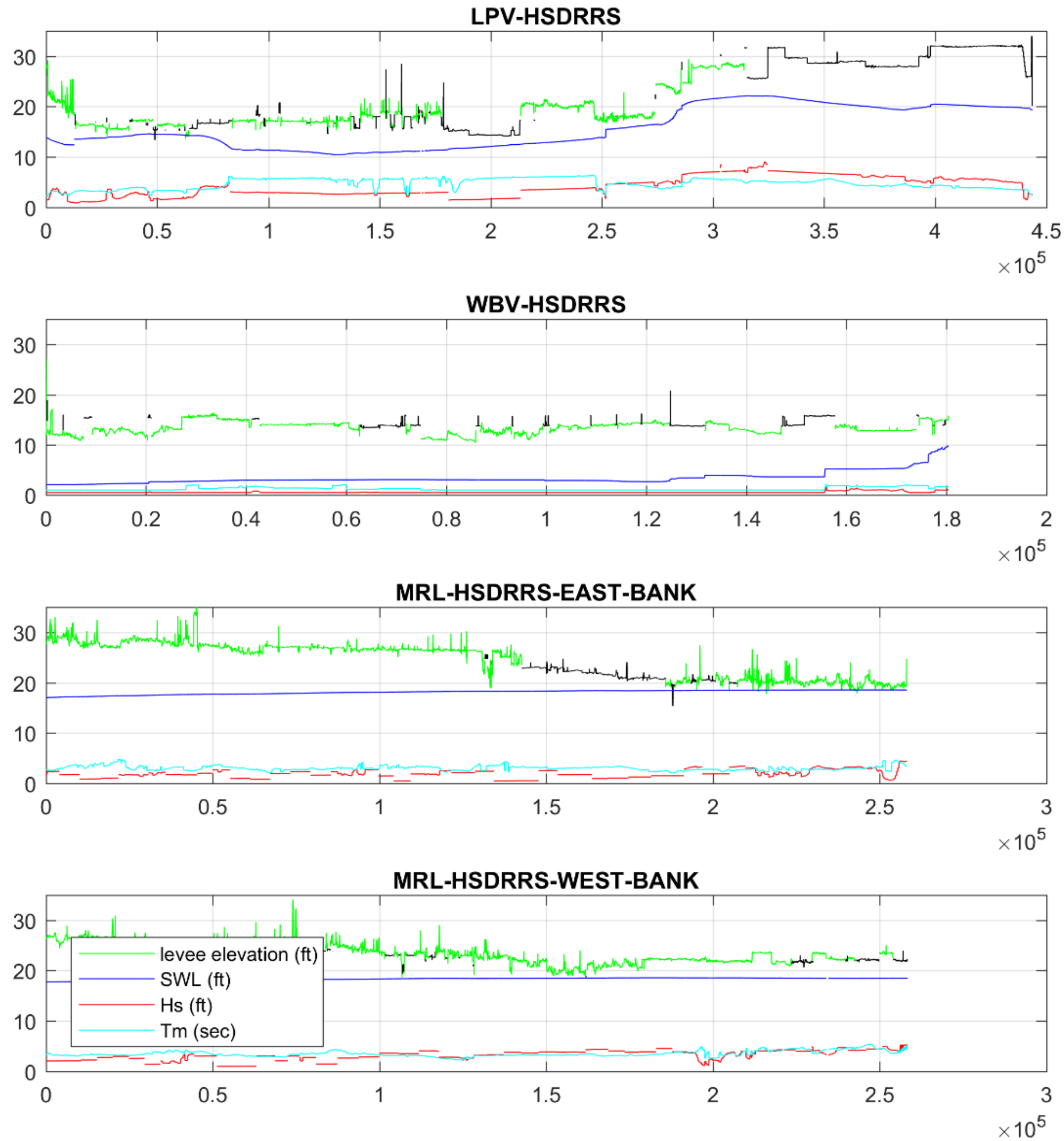

Figure 15. Levee and floodwall elevations, peak surge elevations and waves for synthetic storm 027

Table 3. Starting water surface elevations in HEC-RAS modeling.

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

3.6 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 16 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 17 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 at each boundary segment for a given storm, then runs the simulation and saves the results. Figure 18 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 15. 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.

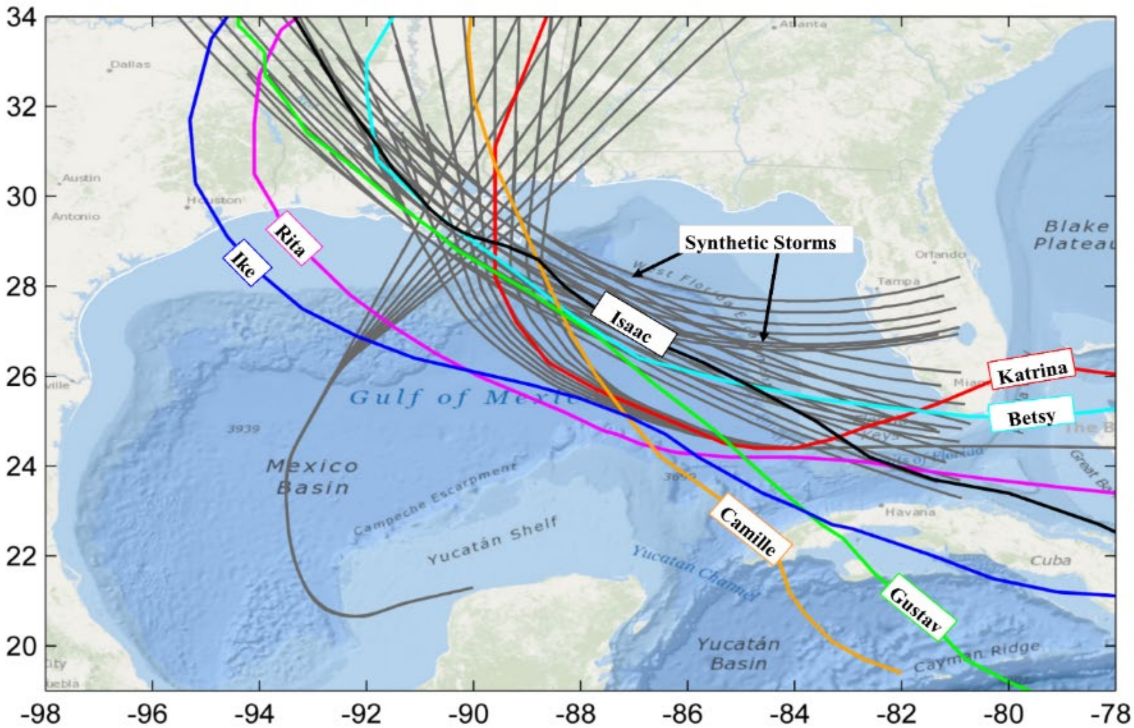

Figure 16. Storm tracks for JPM-OS synthetic events and historical storms of significance

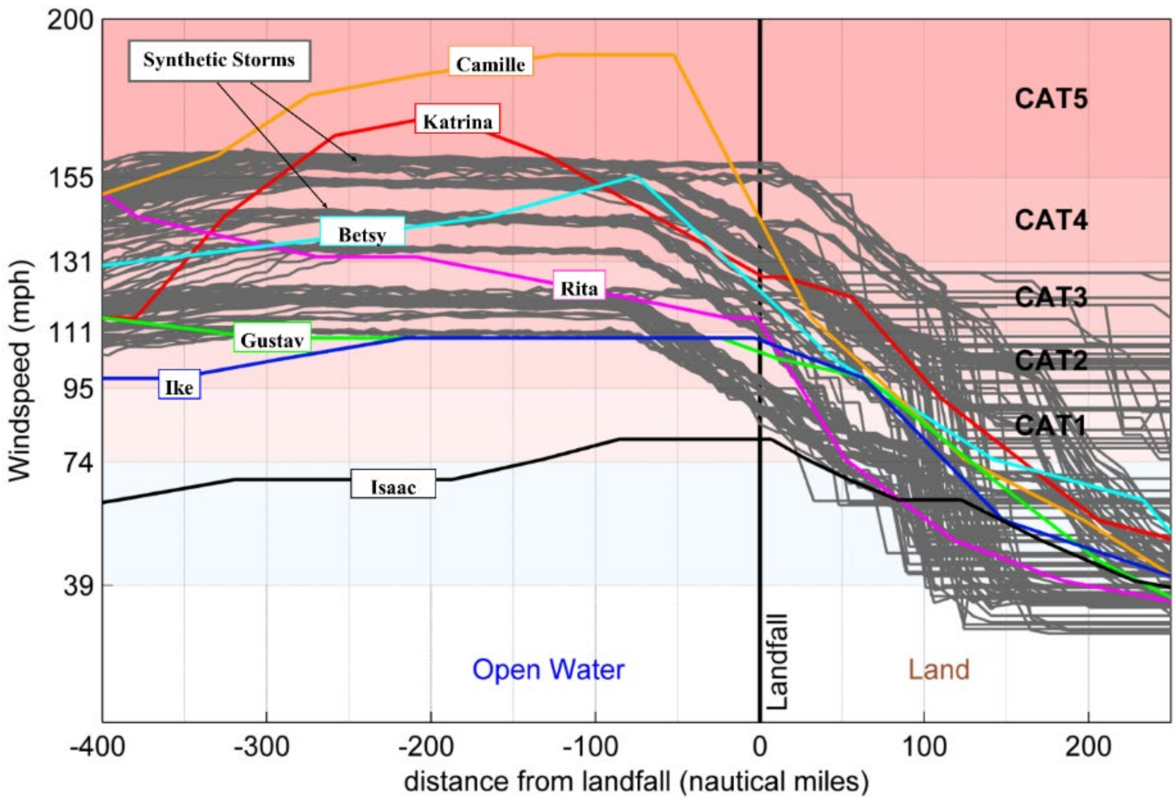

Figure 17. Storm wind-speeds for JPM-OS synthetic events and historical storms of significance

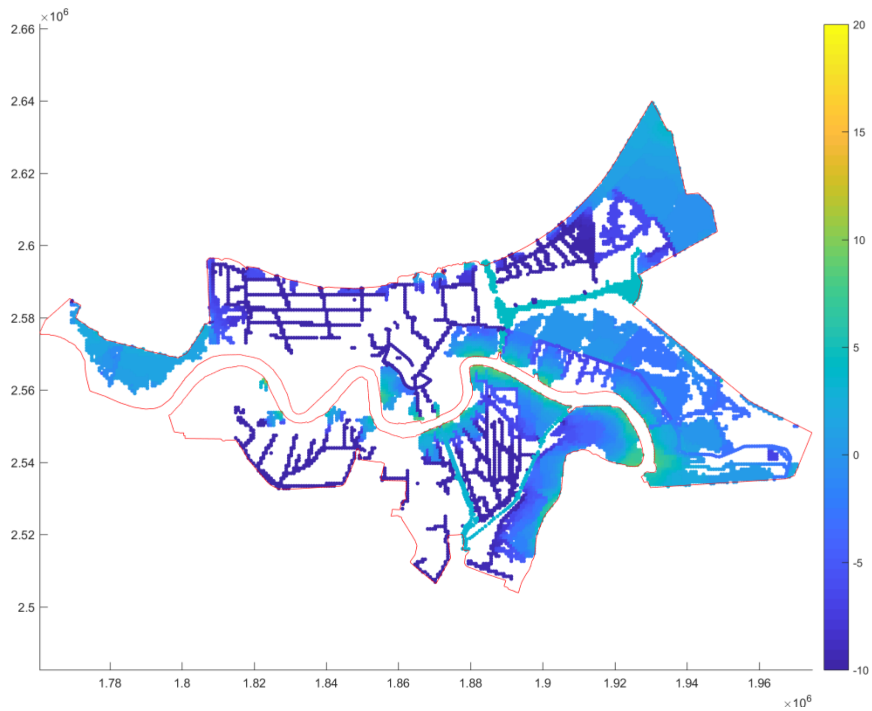

Figure 18. Peak water surface elevation (ft. NAVD88) for synthetic storm 027

3.7 INTERIOR WATER LEVEL STATISTICS

Once all 152 synthetic storms were evaluated, water level statistics could be completed using the latest JPM-OS code. The code was provided 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 19 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 20 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 21 and Figure 22 display the peak depth for the 100YR and 500YR frequencies for the 2023 without-project condition.

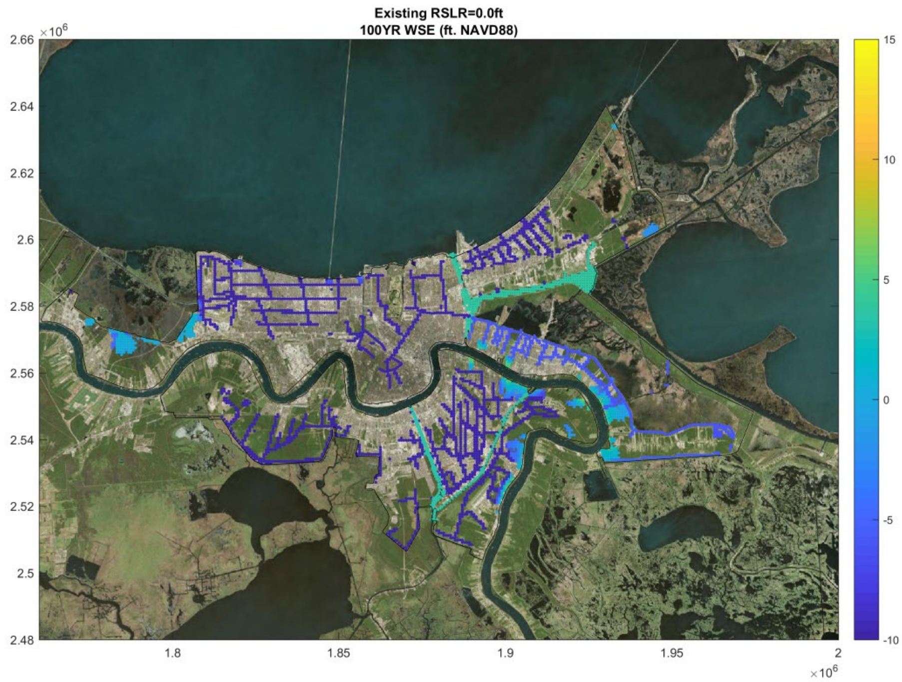

Figure 19. 100 year peak water surface elevation (ft. NAVD88) for existing 2023 conditions

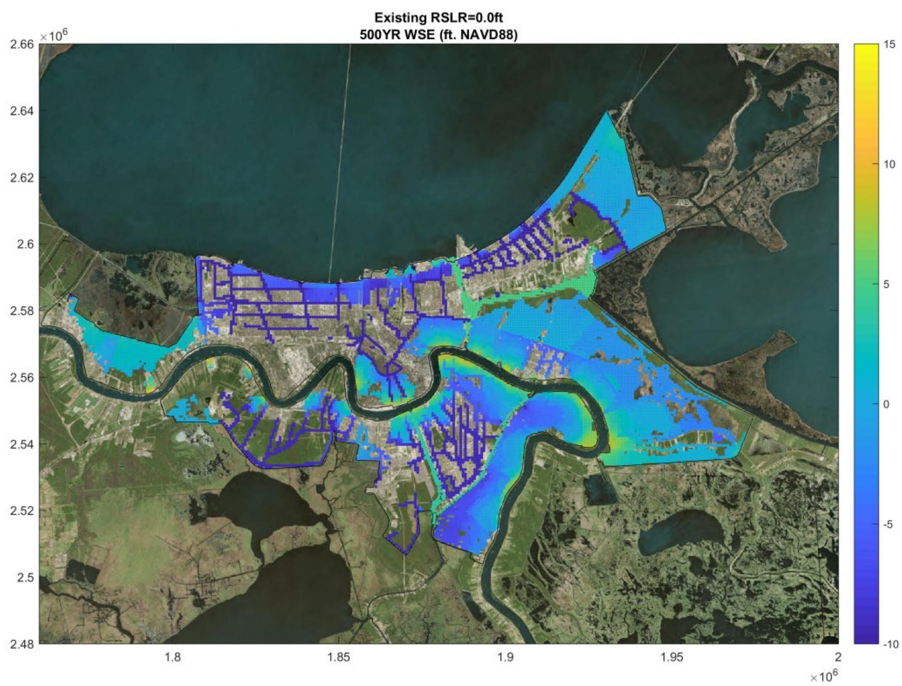

Figure 20. 500 year peak water surface elevation (ft. NAVD88) for existing 2023 conditions

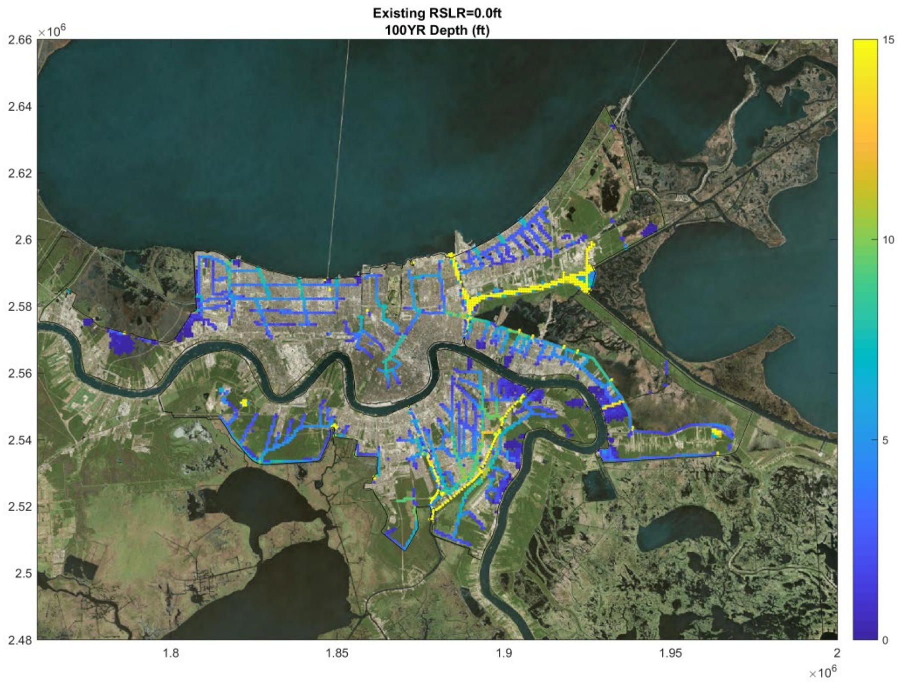

Figure 21. 100 year peak depth (ft.) for existing 2023 conditions

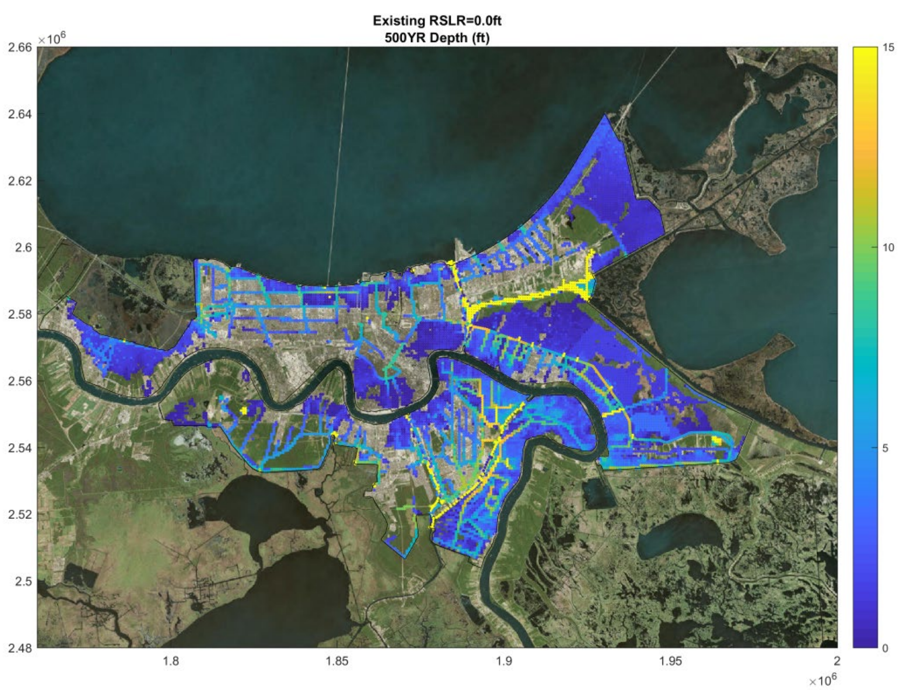

Figure 22. 500 year peak depth (ft.) for existing 2023 conditions

3.8 FUTURE CONDITIONS AND RELATIVE SEA LEVEL CHANGE

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 nearby gages was used for the project evaluation. Table 4 contains the RSLC projections at the 7 gages. Low, intermediate and high RSLC projections assumed for the project evaluation are provided in Table 5 and Figure 23. The plot shows how the project performance of a system designed and built to intermediate RSLC conditions (1.8ft in 2073) would begin to decrease near 2053 for a high RSLC scenario or be extended to 2091 for the low RSLC projection. This uncertainty in project performance has been bracketed in Figure 23. Figure 24 displays the location of the 7 gages relative to HSDRRS.

Table 4. RSLC projections

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

An evaluation was performed to estimate the performance of each project alternative up to and after year 2073, which is the ending year of the design evaluation. Corps policy demands an evaluation of major infrastructure for a time period of 100 years, which would be year 2123. The performance of the project through time depends on the RSLC projection (low/intermediate/high), the initial performance of each project alternative, and an understanding how the exterior stage-frequency changes through time for the various RSLC amounts. Figure 25 displays the estimated project performance through time for 500YR, 200YR and 100YR project alternatives at a portion of HSDRRS near the LPV Lakefront. Figure 26 displays the project performance through time for 500YR, 200YR and 100YR project alternative at a portion of HSDRRS near the WBV West Closure Complex. As described previously, the project begins to lose performance near 2053 for the high RSLC projections. For intermediate RSLC projections, the project begins to lose performance at 2073 (as designed). For low RSLC projections, the project performs adequately to 2091 and then begins to lose performance. A 200YR project alternative designed for intermediate RSLC conditions seems to guarantee 100YR performance up to roughly year 2070 for high RSLR conditions, which should be an added benefit of a 200YR project selection.

The 1% AEP project performance should not change through time for with-project conditions, assuming the project is fully funded, constructed and maintained. The goal of the project, as described in the authorization, is to maintain the project performance to the authorized level which is 1% AEP. As long as the levee lifts are frequent and include some overbuild and are based on the latest available surge hazard data, the project should be able to maintain 1% AEP though time, regardless of what RSLC is realized. The risk to the interior is only increasing if the system is not adaptively managed to keep up with RSLC based of the latest available surge hazard data. For without project conditions or for conditions where the RSLC is higher than intermediate projections and lifts did not accommodate the difference, the project performance would decrease, as shown in Figure 25 and Figure 26.

Table 5. USACE Relative Sea Level Change from 2023 to 2123. Average of 7 gages.

	Low	Int	High
2023	0.0	0.0	0.0
2028	0.1	0.2	0.3
2033	0.3	0.3	0.5
2038	0.4	0.5	0.8
2043	0.5	0.6	1.1
2048	0.6	0.8	1.4
2053	0.8	1.0	1.8
2058	0.9	1.2	2.2
2063	1.0	1.4	2.6
2068	1.2	1.6	3.0
2073	**1.3**	**1.8**	**3.4**
2078	1.4	2.0	3.8
2083	1.5	2.2	4.3
2088	1.7	2.4	4.7
2093	1.8	2.6	5.2
2098	1.9	2.8	5.7
2103	2.1	3.1	6.3
2108	2.2	3.3	6.9
2113	2.3	3.5	7.4
2118	2.4	3.8	8.0
2123	**2.6**	**4.0**	**8.6**

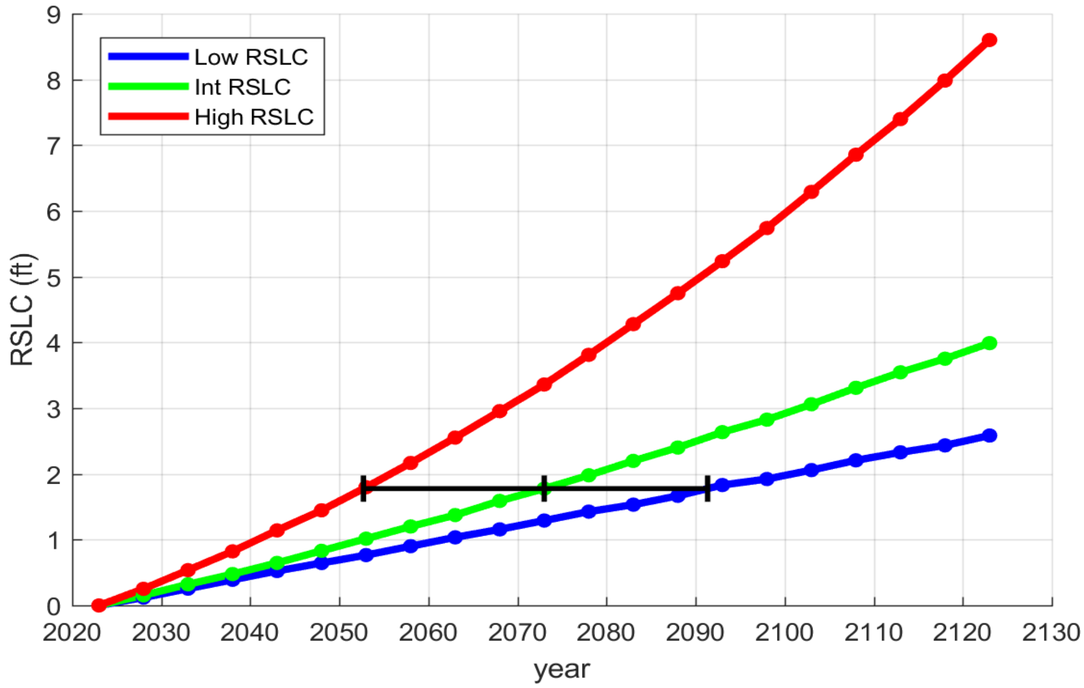

Figure 23. Low, intermediate and high relative sea level change projections from 2023 to 2123

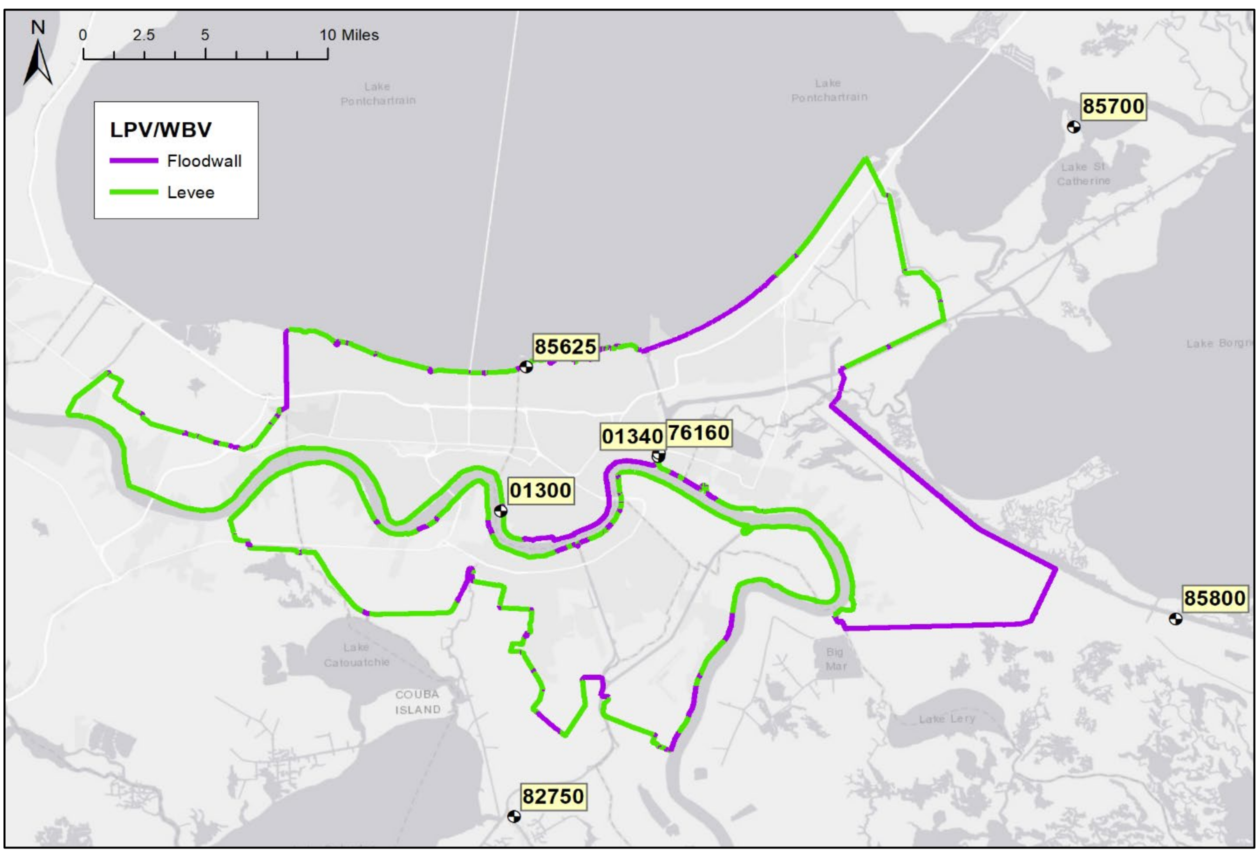

Figure 24. Location of water level gages used to determine RSLC projections

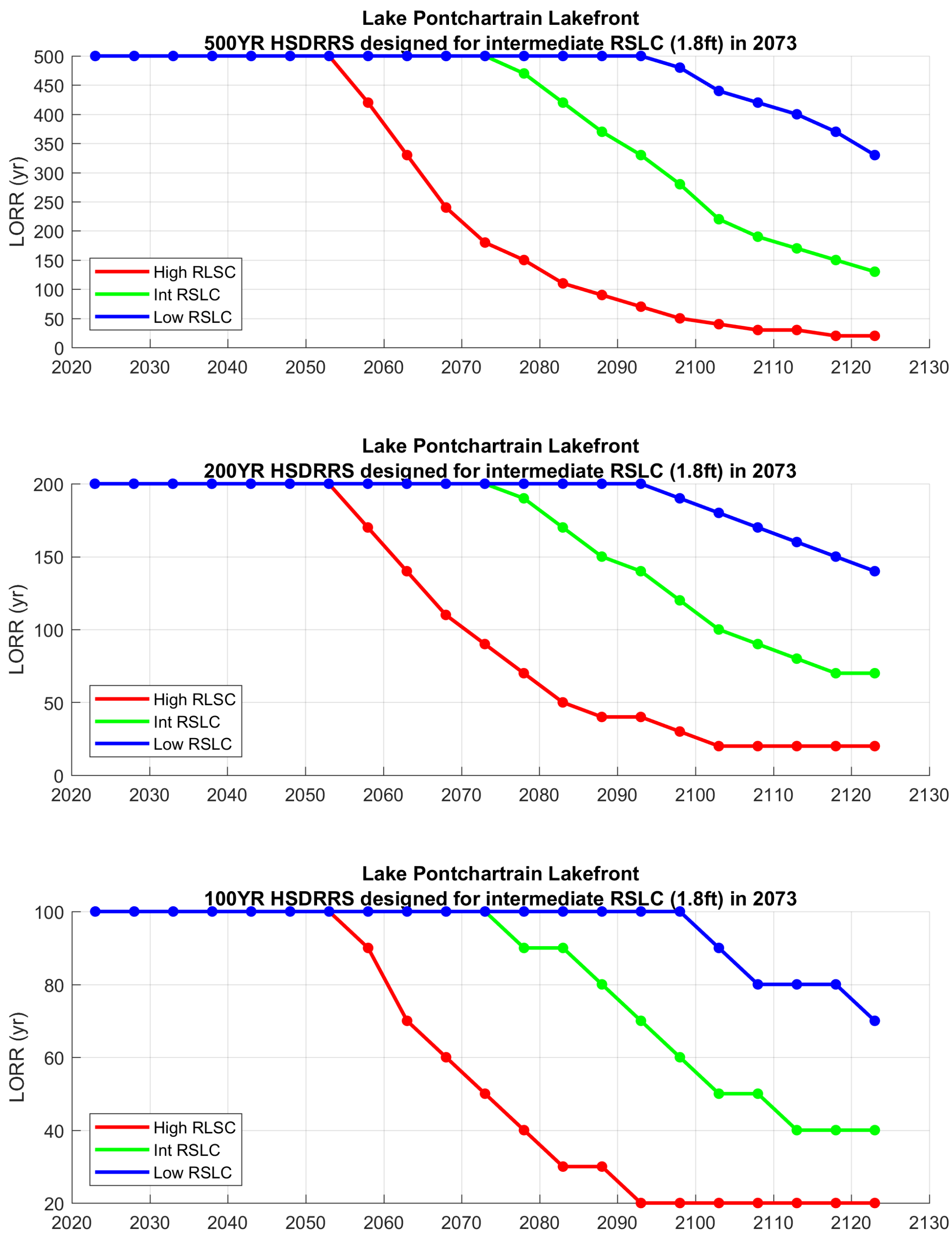

Figure 25. Project performance for 500YR, 200YR and 100YR project alternatives for 3 RSLC projections at a location along the LPV Lakefront

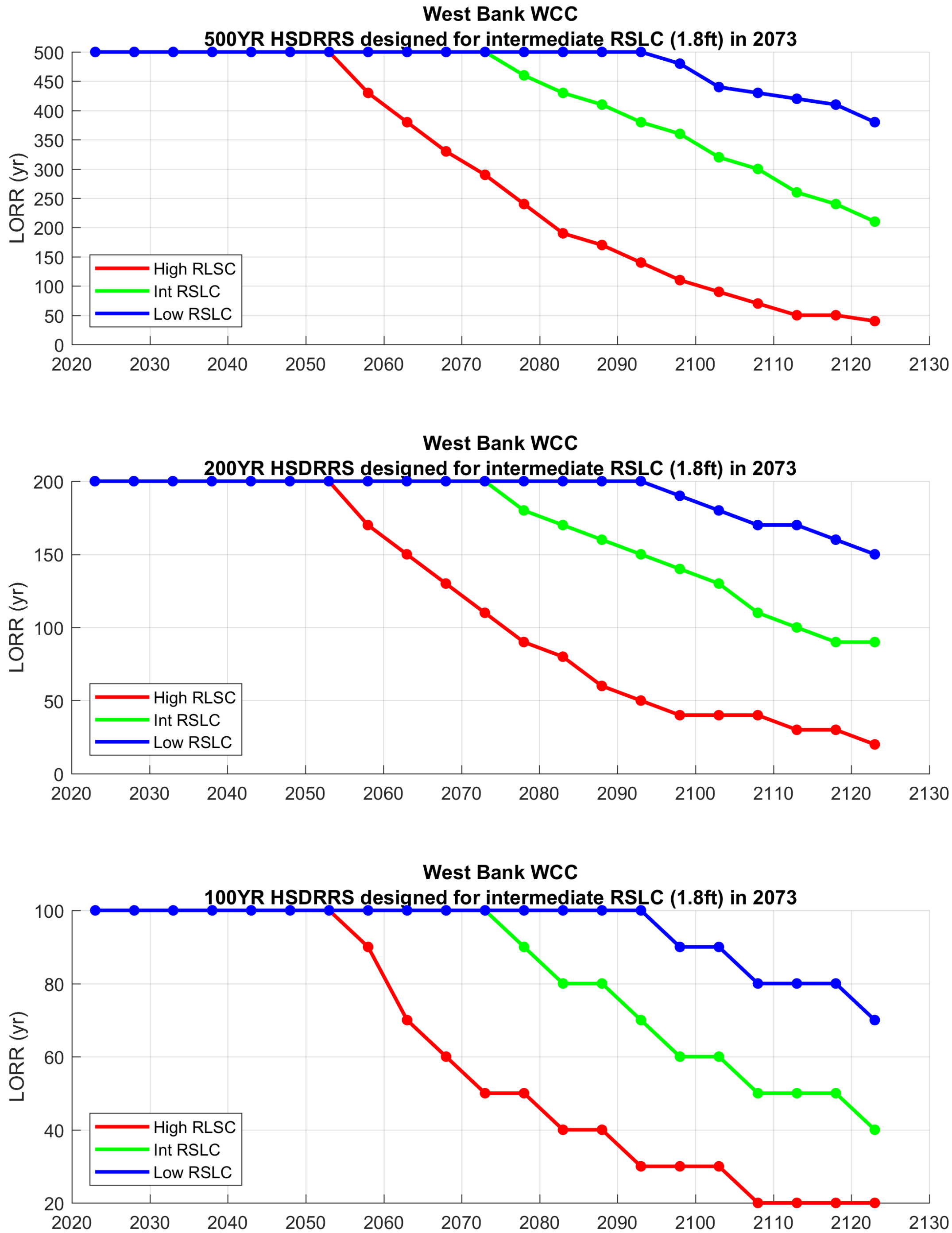

Figure 26. Project performance for 500YR, 200YR and 100YR project alternatives for 3 RSLC projections at a location on the West Bank

The overtopping calculations, RAS simulations and JPM-OS statistics were repeated for the 2073 future without-project condition for low, intermediate and high RSLC. 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 27. The subsidence varies by region, but around HSDRRS the amount was close to -0.5ft. For the purposes of this study, we assumed the CPRA future condition ADCIRC runs evaluated a total RSLC of approximately 2.0ft (1.5 eustatic + 0.5ft subsidence).

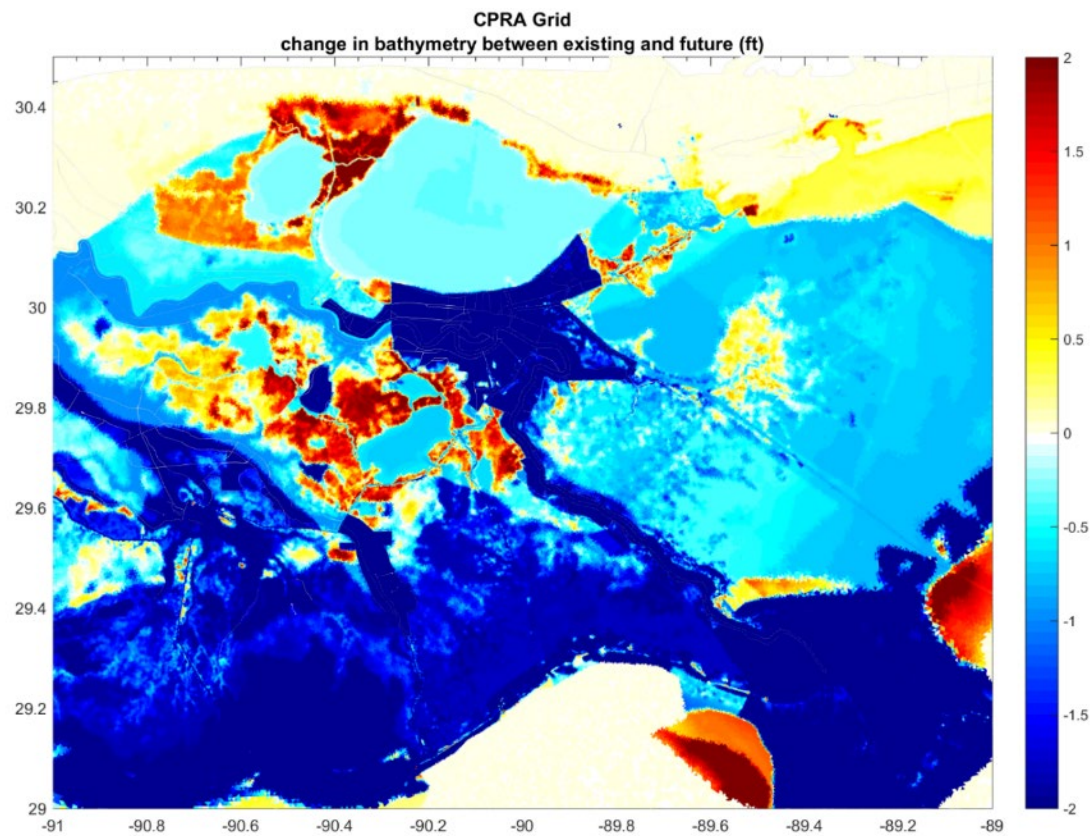

Figure 27. Change in bathymetry from existing to future conditions (CPRA mesh S13G60)

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. CPRA conducted the full suite of 152 simulations 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 adjustments 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 Geotech 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 28 displays the projected levee settlement values provided by the MVN Geotechnical Engineering branch. No settlement was assumed for the floodwalls. The MR&T levees above the cross-over points are assumed to settle 0.5ft by 2073, although if the levee settles below the MR&T authorized grade, then it is assumed to be lifted to

MR&T authorized grade. In other words, the analysis assumes the MR&T levees are always built to at least authorized grade in the 2073 future conditions. Figure 29 displays an example of an existing and future condition levee showing the effects of local settlement, regional subsidence and eustatic sea level rise.

Figure 28. Projected levee settlement values by 2073. Levees are plotted as green line. Floodwalls are grey lines

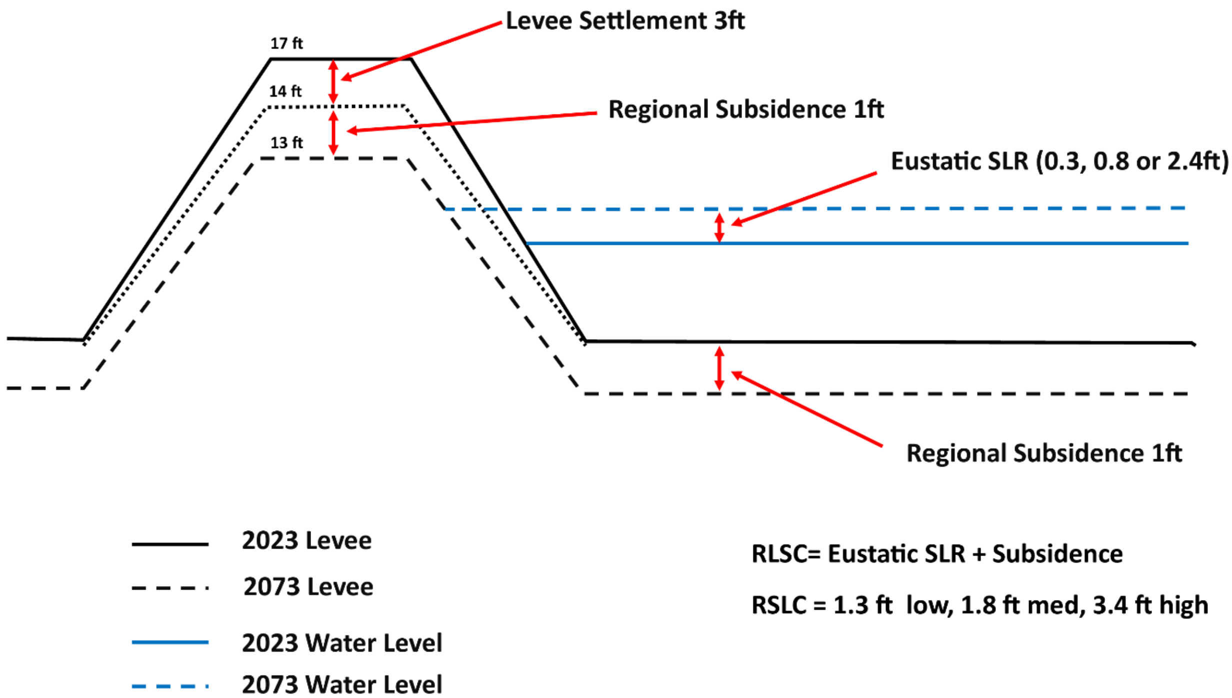

Figure 29. Example of settlement, subsidence and eustatic sea level rise.

3.9 FUTURE CONDITION OVERTOPPING AND INUNDATION

Levee settlement and RSLC result in greater overtopping volumes and more inundation in the HEC-RAS simulations of future without-project conditions. Figure 30 displays the resulting 100YR water surface elevation for the future without-project scenario assuming intermediate 1.8 ft RSLC. The resulting 100YR inundation is much greater in the future without-project scenario. Figure 31 displays the resulting 500YR water surface elevation for the future without-project scenario assuming intermediate 1.8 ft RSLC. All statistical water surfaces were provided to economics for evaluation. Figure 32 and Figure 33 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 existing and future without-project conditions within the closed IHNC basin are provided in Table 6. The table includes raw RAS model output, and water levels accounting for the effects of rainfall, pumping and wind-setup. In the past, 90% water levels were assumed and they are also provided in the table. All of these added effects increase the expected water level within the basin. A previously assumed safe water level within the basin is 8.0ft NAVD88. 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 elevation 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 PED phase of the project.

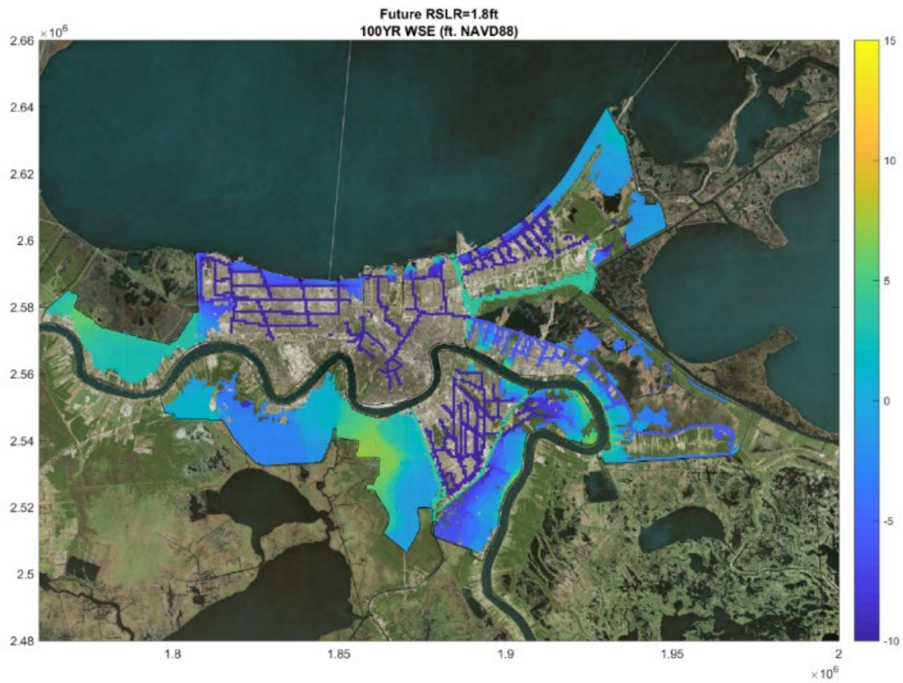

Figure 30. 100 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC conditions – without project

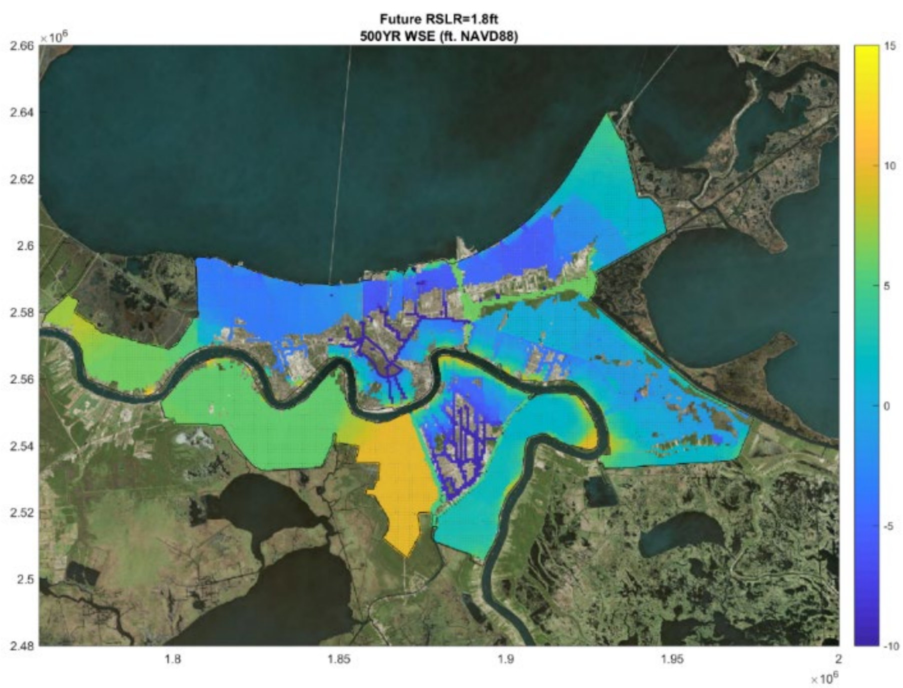

Figure 31. 500 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC conditions – without project

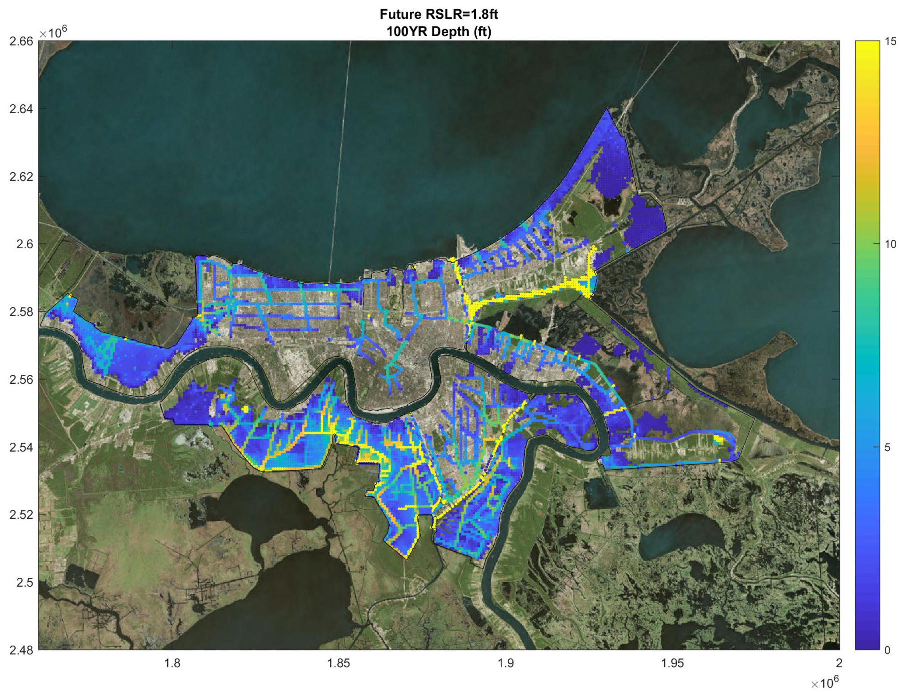

Figure 32. 100 year peak depth (ft) for future 2073 intermediate RSLC conditions – without project

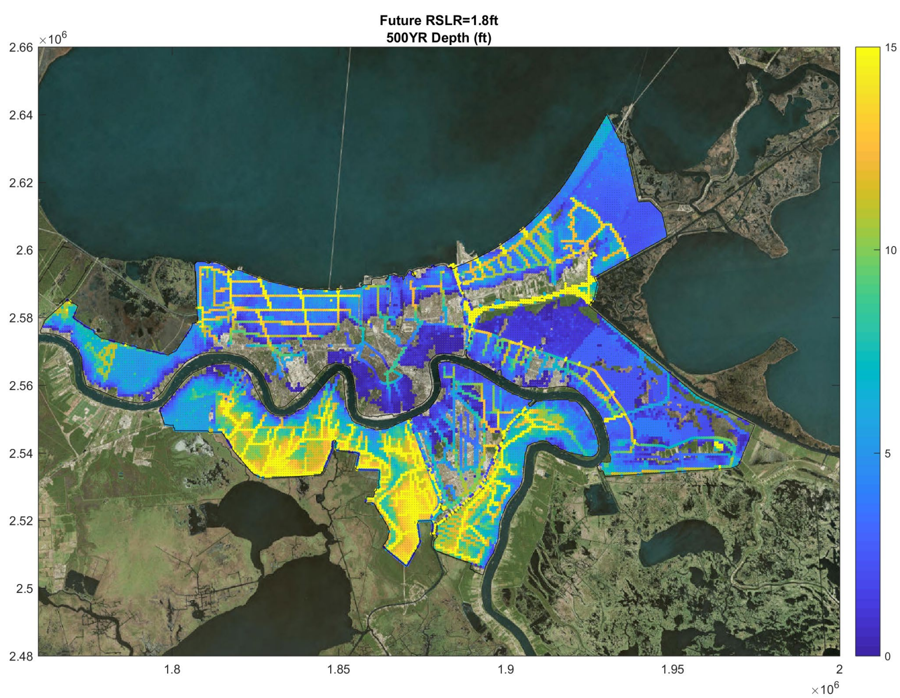

Figure 33. 500 year peak depth (ft) for future 2073 intermediate RSLC conditions – without project

Table 6. IHNC Corridor water level statistics for existing and future conditions

LPV-WBV STUDY RAS MODEL OUTPUT				
Without Rain and Wind Setup (RAW OUTPUT)				
	RSLR= 0.0	**RSLR= 1.3ft**	**RSLR= 1.8ft**	**RSLR= 3.4ft**
0010YR	-999	-999	-999	-999
0020YR	3.05	3.07	3.08	3.16
0050YR	3.61	3.64	3.66	3.87
0100YR	3.93	3.99	4.02	4.58
0200YR	4.25	4.4	4.45	6.12
0500YR	4.92	6.26	6.74	8.85
1000YR	6.24	11.59	11.95	12.61
With Rain and Wind Setup				
0050YR	4.8	4.8	4.9	5.1
0100YR	5.2	5.3	5.3	5.9
0200YR	5.9	6.0	6.1	7.7
0500YR	6.5	7.9	8.3	10.5
90% Water Levels				
50YR	5.8	**5.9**	**5.9**	**6.1**
100YR	6.2	**6.3**	**6.3**	**6.9**
200YR	7.0	**7.0**	**7.1**	**8.7**
500YR	7.7	**9.0**	**9.5**	**11.6**

a. Exterior water level 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 34 through Figure 36 display the 100YR and 500YR still water level, significant wave height and mean wave period for existing conditions (RSLC = 0 ft). Figure 37 through Figure 39 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 40 through Figure 42 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 34 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 42.

It was decided to adjust the surge statistics in the river to account for a higher 400,000 cfs. This adjustment provided surge values in the river that are more consistent with more sophisticated analysis conducted for design of HSDRRS. 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 43.

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/

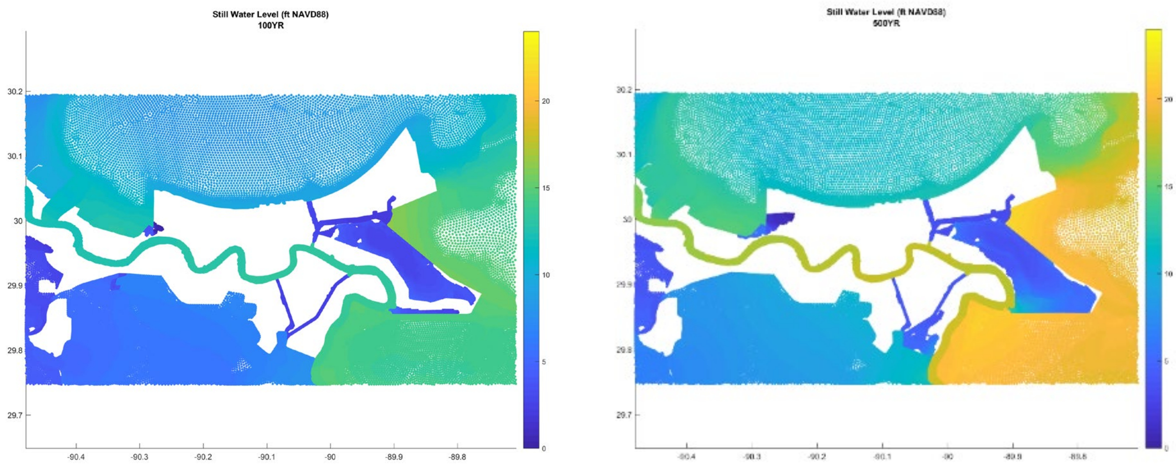

Figure 34. 100YR and 500YR still water levels (ft. NAVD88) for existing conditions (RSLC=0 ft)

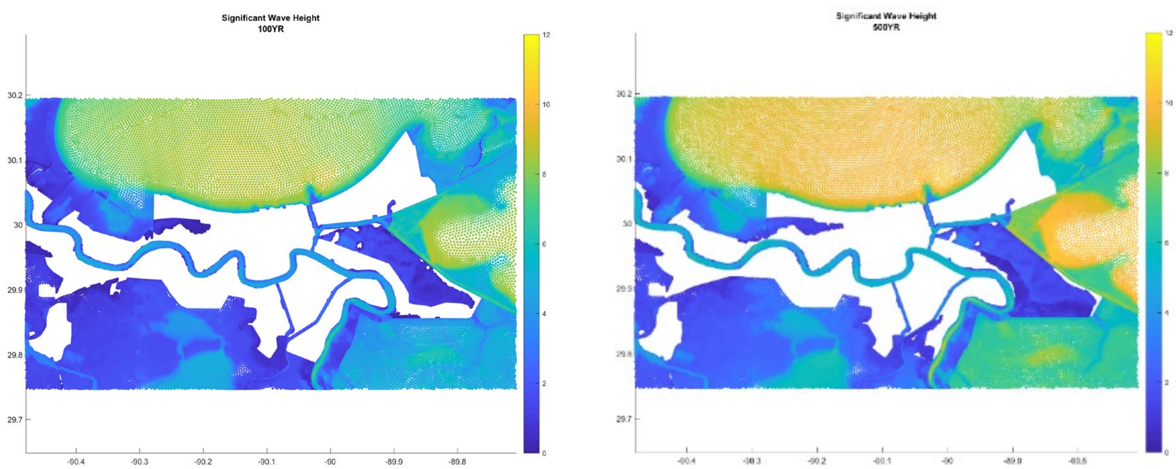

Figure 35. 100YR and 500YR significant wave heights (ft) for existing conditions (RSLC=0 ft)

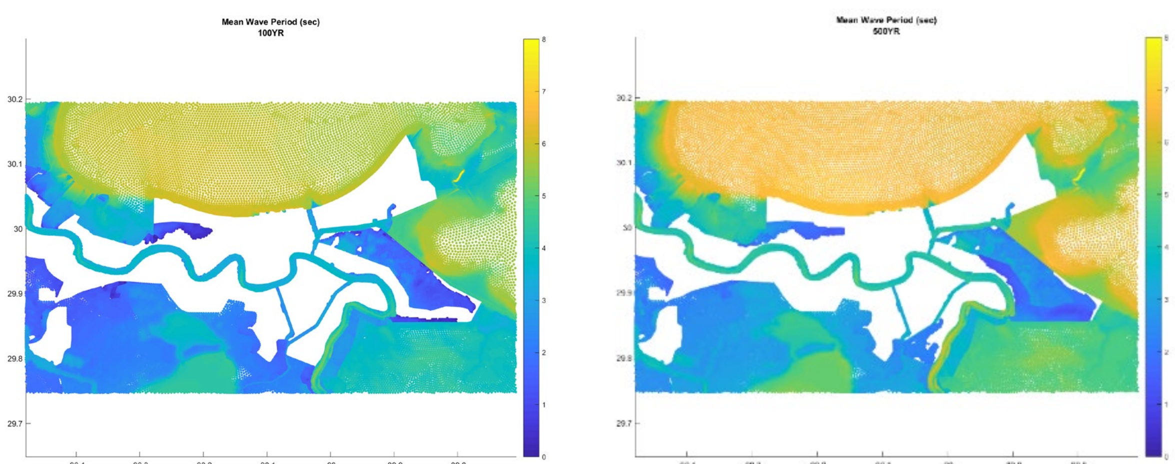

Figure 36. 100YR and 500YR mean wave period (sec) for existing conditions (RSLC=0 ft)

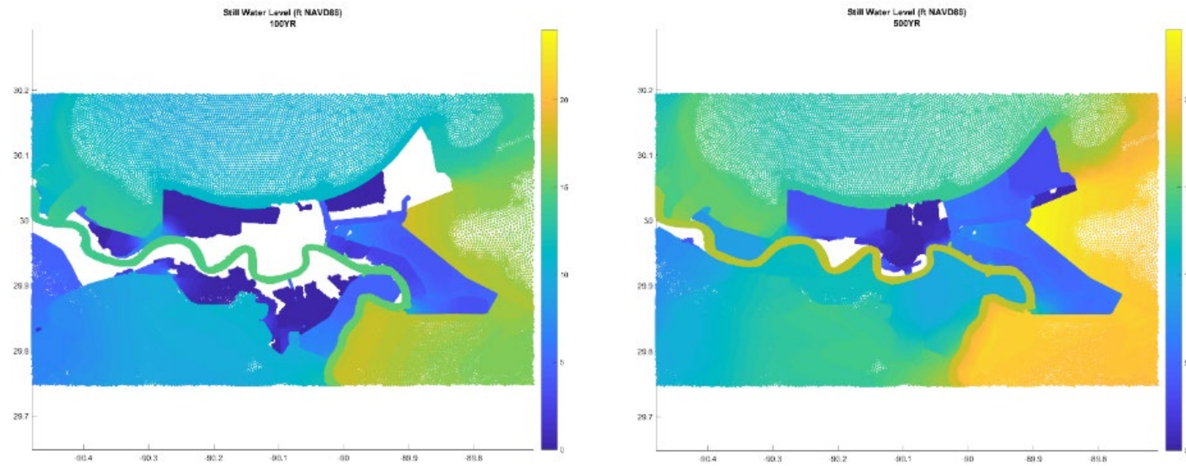

Figure 37. 100YR and 500YR still water levels (ft. NAVD88) for future conditions (RSLC=2 ft)

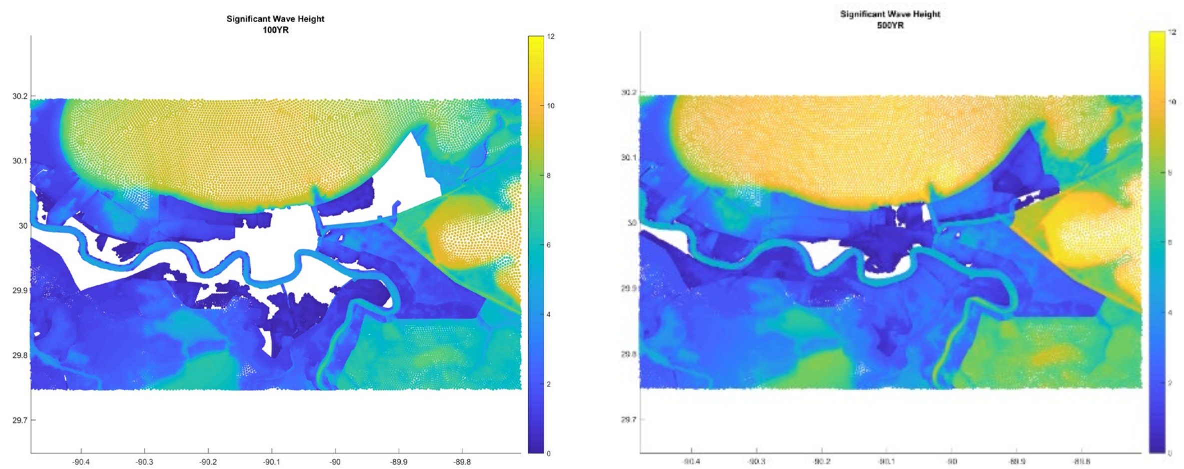

Figure 38. 100YR and 500YR significant wave heights (ft) for future conditions (RSLC=2 ft)

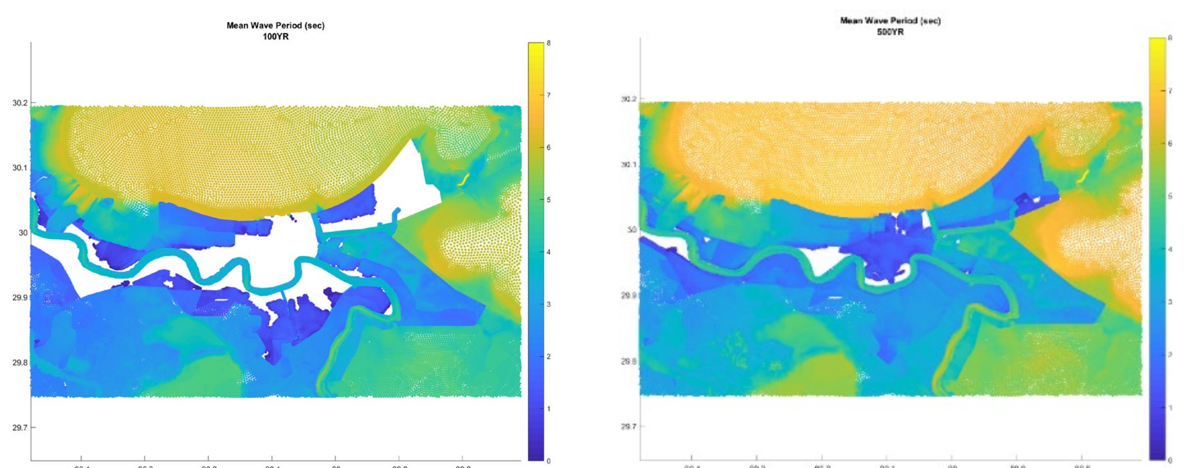

Figure 39. 100YR and 500YR mean wave period (sec) for future conditions (RSLC=2 ft)

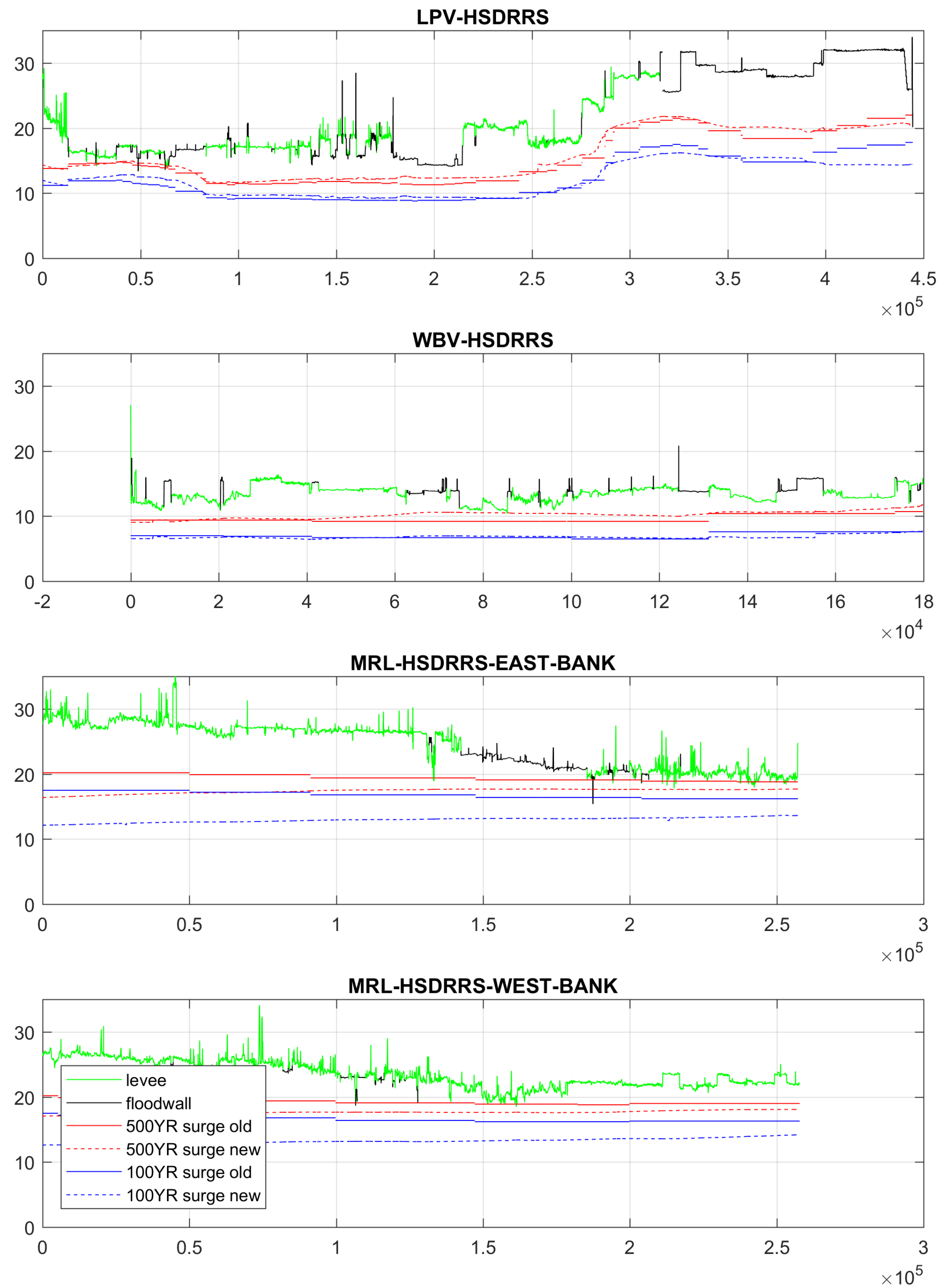

Figure 40. Comparison of new and old 100YR and 500YR still water level statistics (ft. NAVD88)

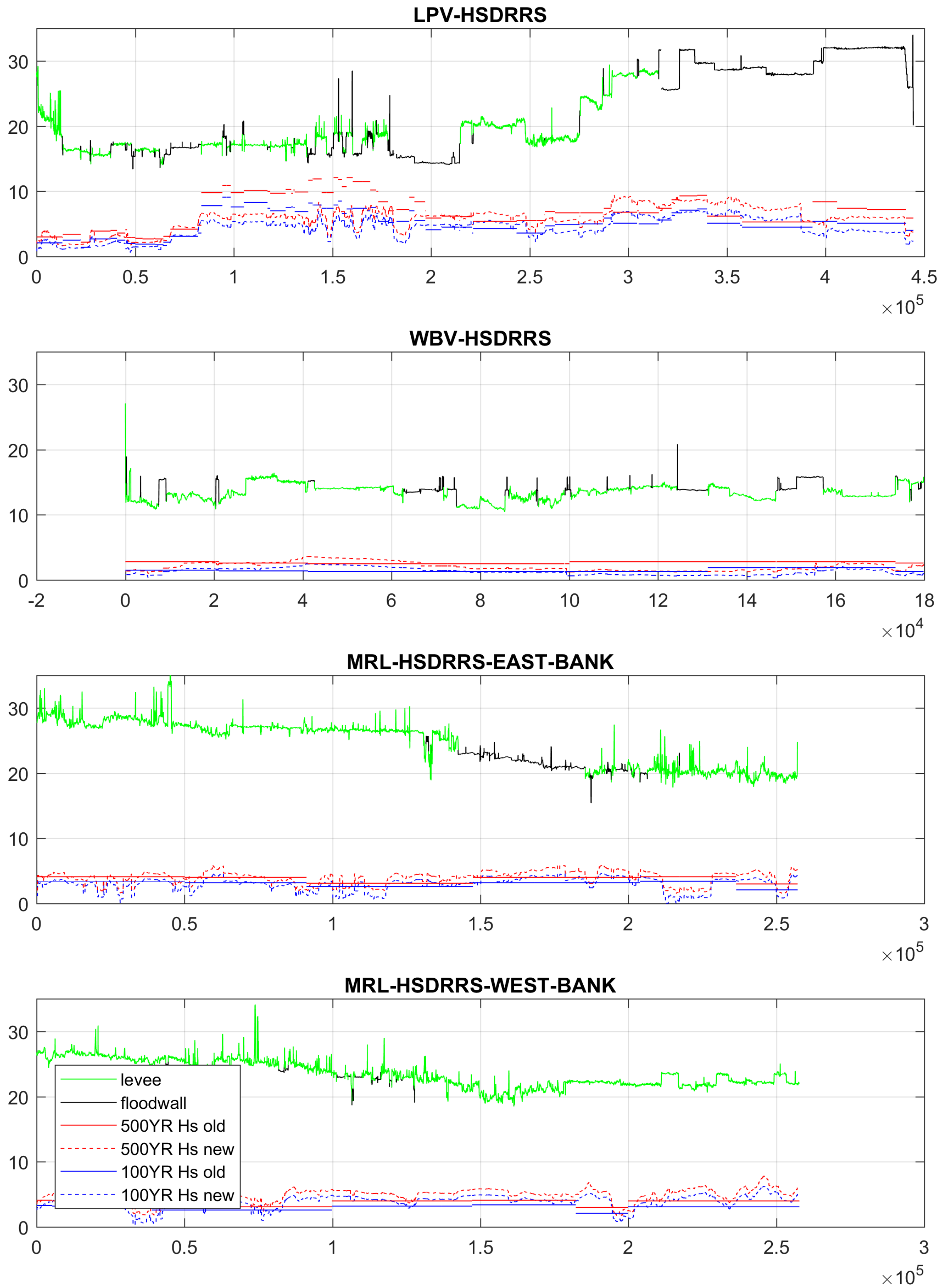

Figure 41. Comparison of new and old 100YR and 500YR significant wave height statistics (ft)

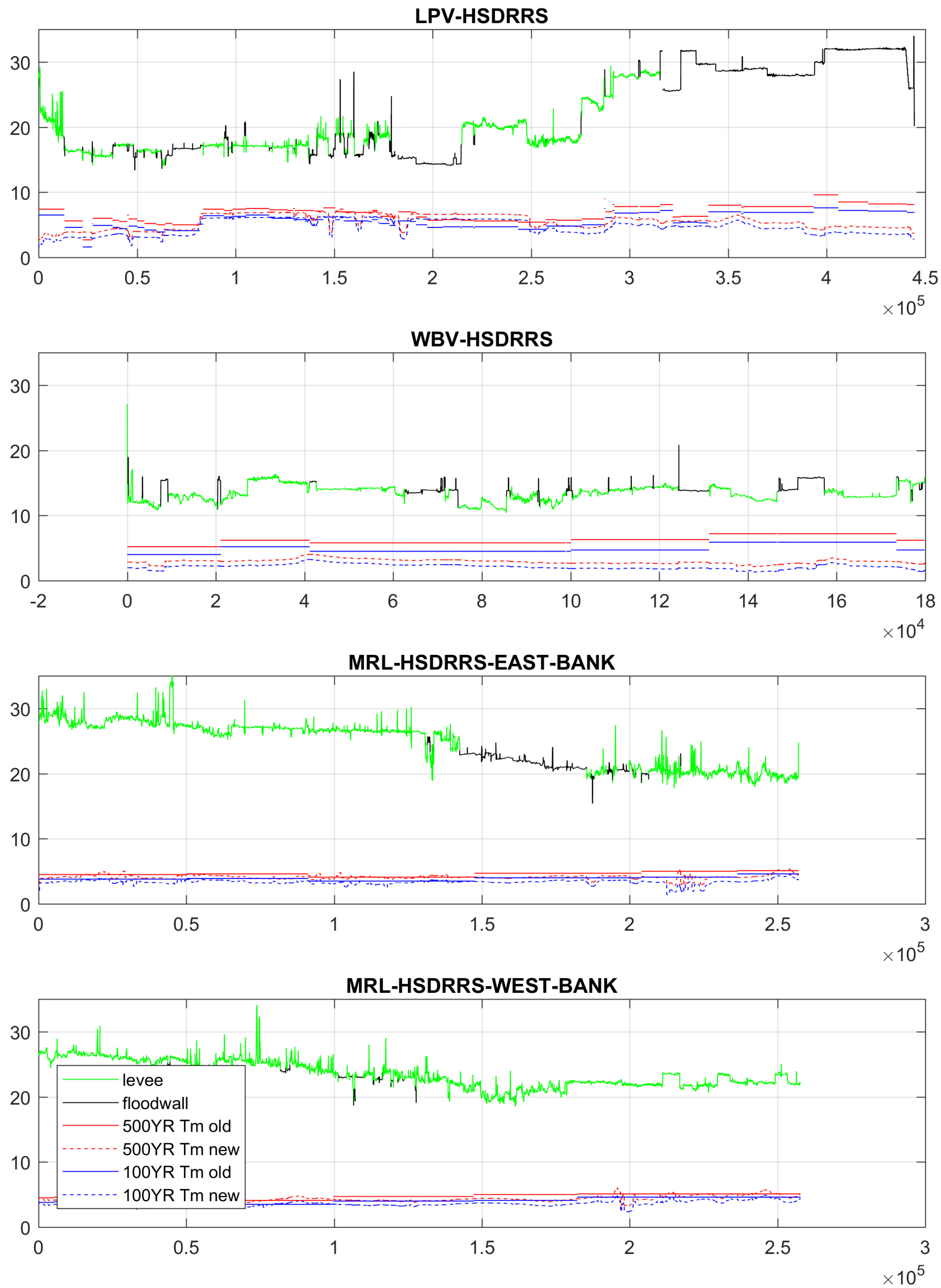

Figure 42. Comparison of new and old 100YR and 500YR mean wave period statistics (sec)

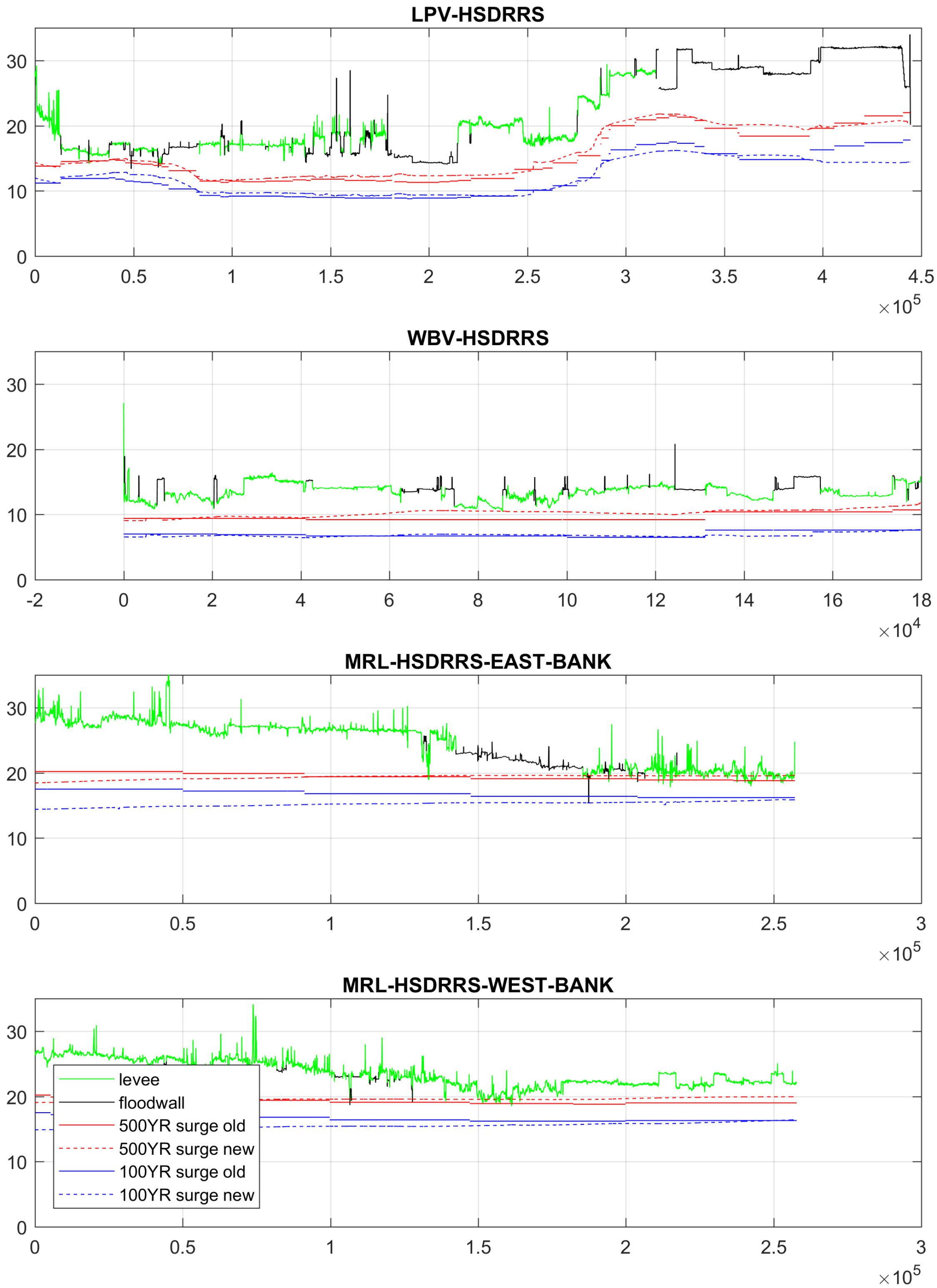

Figure 43. Comparison of new and old 100YR and 500YR still water level statistics with correction applied the Mississippi River surge statistics (ft NAVD88)

3.10 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 44 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 typically 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 45 displays the cumulative probability of discharge for each month in hurricane season based on data from 1976 to 2002. This data, along with storm frequency information was needed to compute surge statistics in the river. Figure 46 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 small increase in the expected discharge during hurricane season. For example, the 50% or mean discharge during July (a month with relatively low storm activity) was approximately 410,000cfs with the data from 1976 to 2002. When the data is updated, the mean discharge during July becomes 450,000cfs. 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 storm frequency by month. In the older HSDRRS analysis, a sample of 14 observed storms impacting the New Orleans area provided the hurricane probability by month. Table 7 contains the storm probabilities by month assumed in the original HSDRRS analysis. Since 2005, more storms have impacted New Orleans including:

Storm	Date
Hurricane Gustav	August 31, 2008
Hurricane Ike	September 13, 2008
Hurricane Ida	November 10, 2009
Tropical Storm Lee	September 4, 2011
Hurricane Isaac	August 29, 2012
Hurricane Nate	October 8, 2017
Hurricane Barry	July 10, 2019

These additional storms occurring since 2005 may change some of the assumptions about storm frequency and ultimately impact the stage-frequency estimates in the river. An additional analysis was performed on NOAA's HURRDAT records. The entire storm dataset was filtered for Category 1 and above. A spatial analysis of storm frequency is provided in Figure 47 for years 1941 to 2005, and in Figure 48 for years 1941 to 2019. When the latest data (2005 to 2019) is added, the storm frequency estimates appear to lower slightly in the Northern Gulf of Mexico.

The latest storm frequency and river discharge data suggests that the assumptions made concerning storm frequency and discharge frequency are still valid for a feasibility level study.

However, the observed discharges have changed enough to warrant a revisit during later design assessments such as the PED phase of this project.

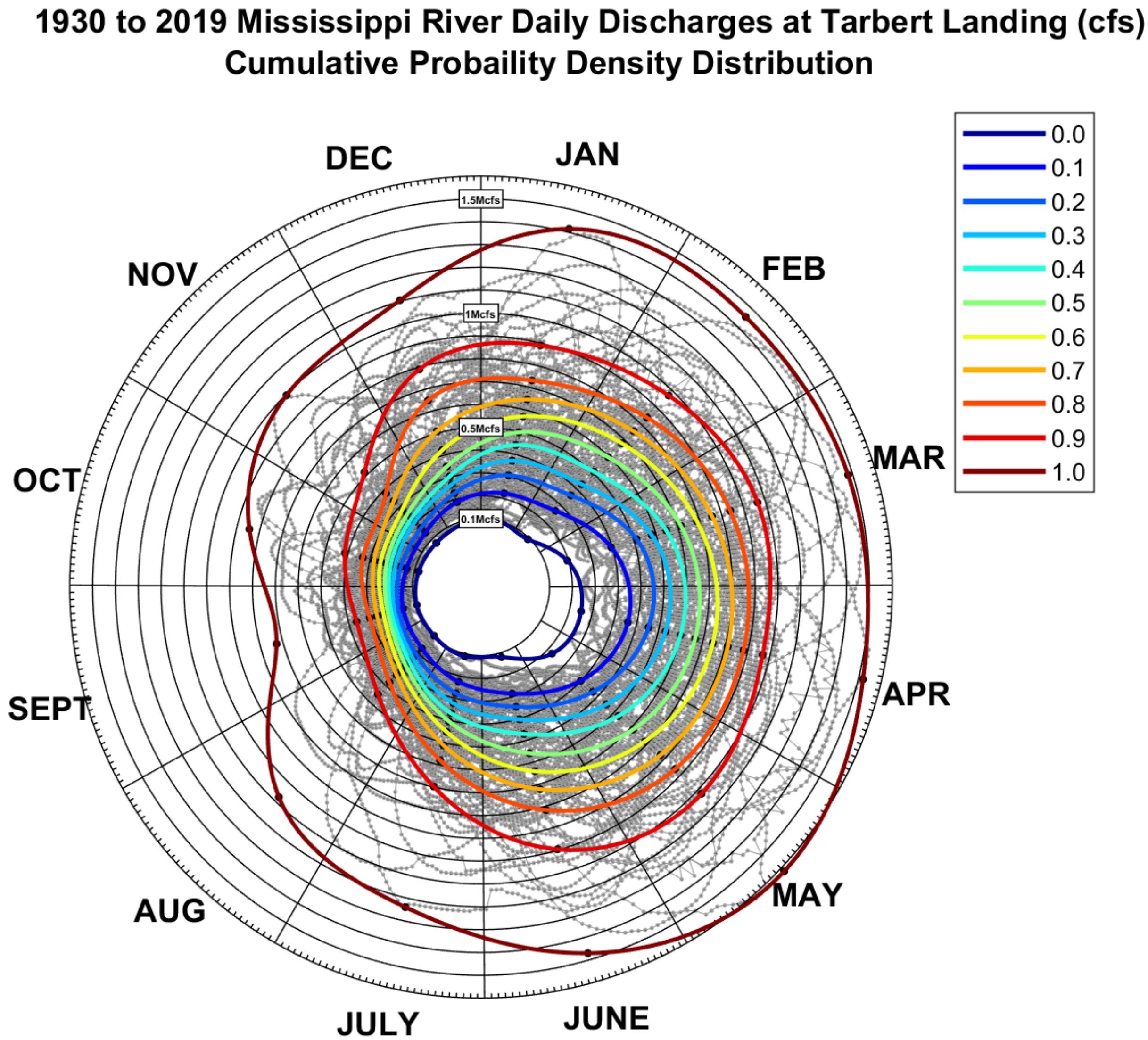

Figure 44. Cumulative probability density distribution of lower Mississippi River discharges and daily discharge observations (1930 to 2019)

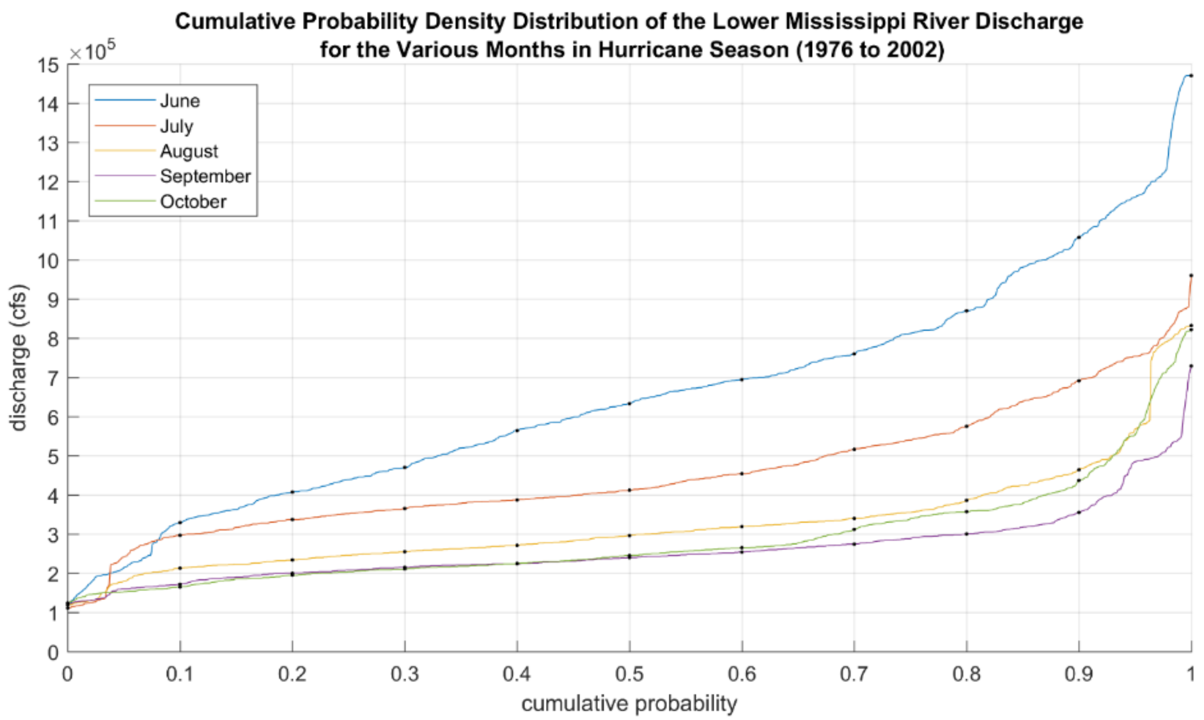

Figure 45. Cumulative probability density distribution of the lower Mississippi river during hurricane season (1976 to 2002 data)

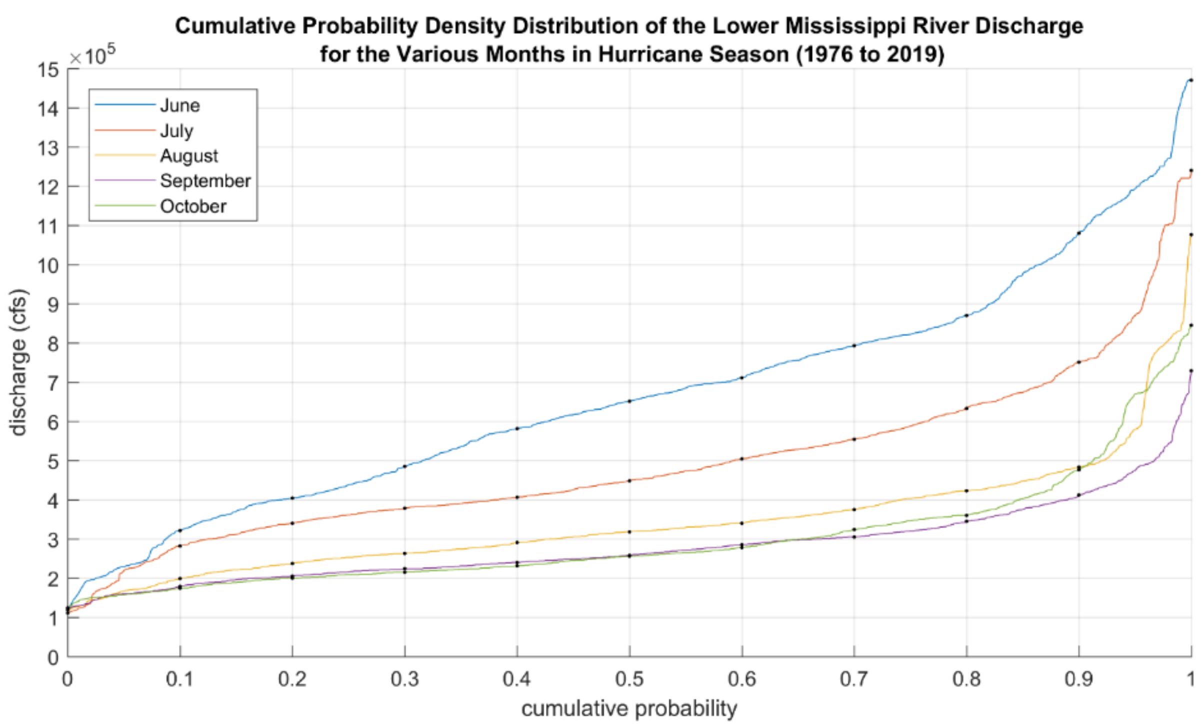

Figure 46. Cumulative probability density distribution of the lower Mississippi river during hurricane season (1976 to 2019 data)

Table 7. Probability density of hurricanes in various months based on hurricanes in the New Orleans areas in the period 1941 – 2005.

	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

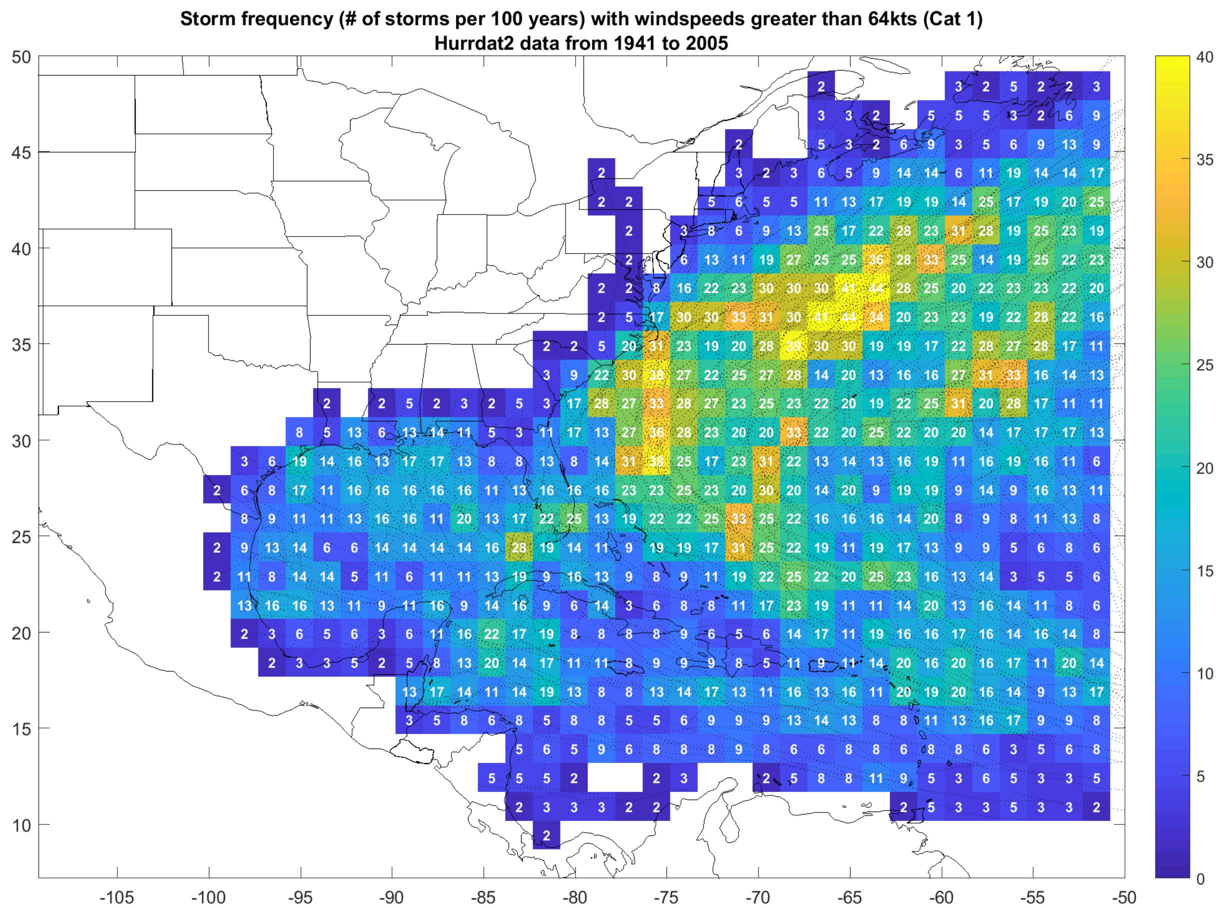

Figure 47. Category 1 and above storm frequency using NOAA HURRDAT filtered for years 1941 to 2005

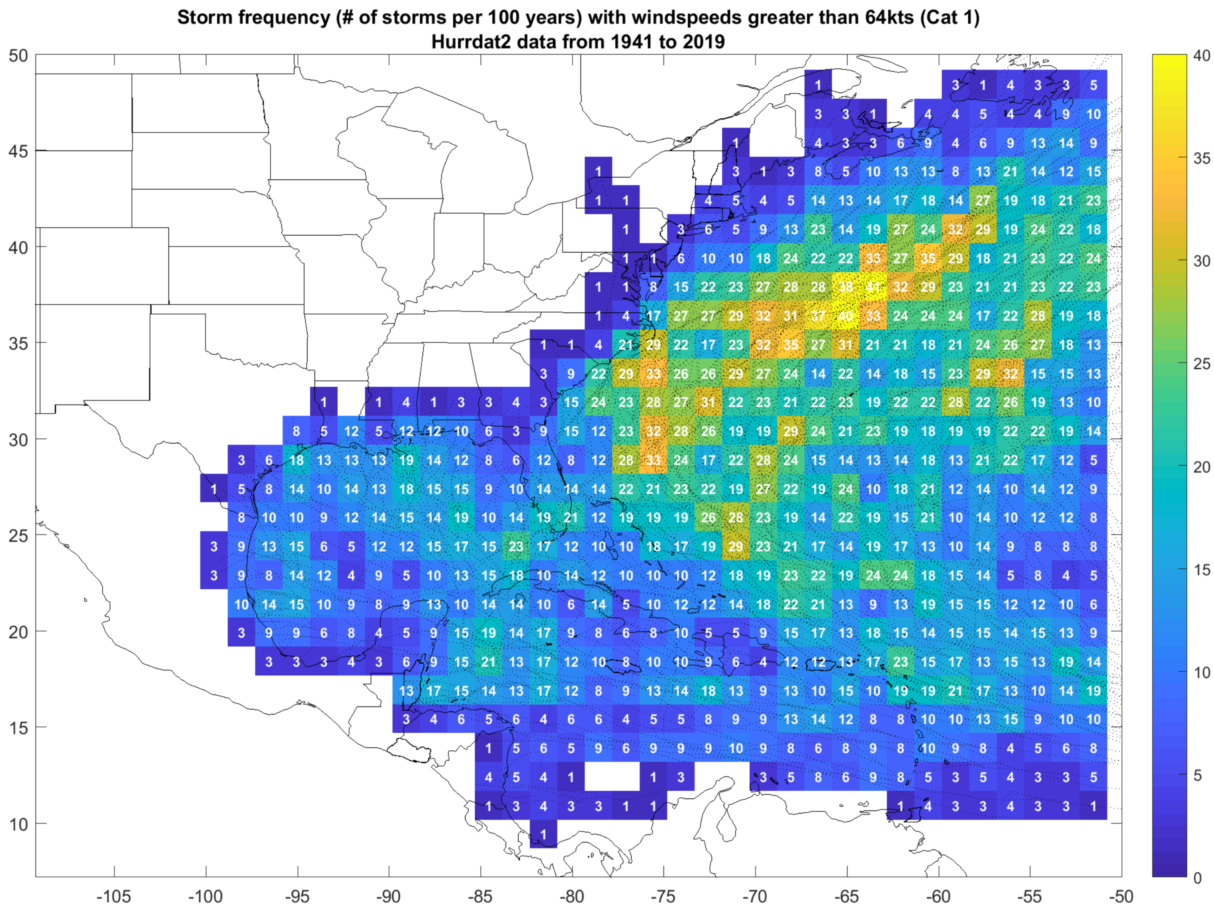

Figure 48. Category 1 and above storm frequency using NOAA HURRDAT filtered for years 1941 to 2019

3.11 FUTURE CONDITIONS 2073 – WITH PROJECT

100YR 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 11 and Figure 12) 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 49 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 50 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 51 displays the 2073 required 100YR 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 resiliency check is simply an extra design constraint which ensure the levee or floodwall elevation is at or above the 0.2% still water elevation at 50% confidence. In some cases, specifically on the WBV, this design check determines the final design elevation of the levee, but it is not the governing factor in deciding final design elevations at all segments of the system. The “resiliency check” is a sanity check on final 1% H&H design elevations. Setting the levee elevation to at least the 0.2% still water ensures some level of risk reduction for events beyond the 1%, but it is does not ensure full resiliency out to 0.2% conditions. True resiliency has more to do with armoring

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 cross-over point is determined by the intersection of the MR&T grade with the 1% hurricane design grade. The hurricane design grade is derived from the estimated water levels and waves for a given future year. With RSLC, waves and surge increase in the river, which in turn drives the required 1% hurricane design elevations higher, which pushes the cross-over further up-stream. The translation of the cross-over does not influence waves and surge, overtopping or inundation, but it is the other way around, waves and surge influence the cross-over. The location of the cross-over points make no difference in the estimated interior flood risk or how it is calculated, nor does it influence design elevations. It is merely the transition from MR&T grade to HSDRRS grade. The cross-over point is more important in determining which project is responsible for funding levee lifts and maintenance. Levees above the cross-over are funded by MR&T, while levee upgrades below are funded by HSDRRS.

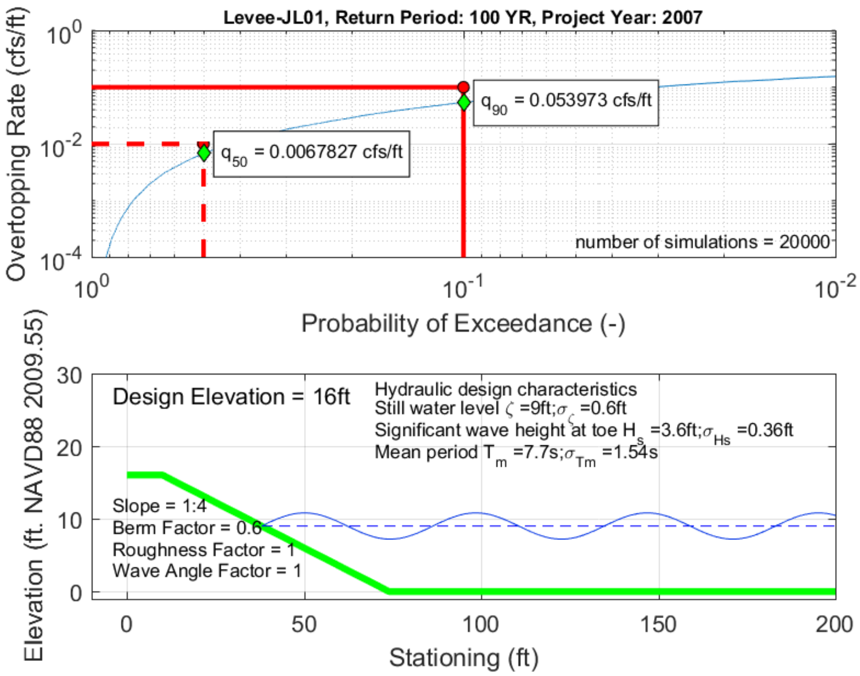

Figure 49. Example of Monte Carlo output for the Jefferson Lakefront levee JL01 from updated code.

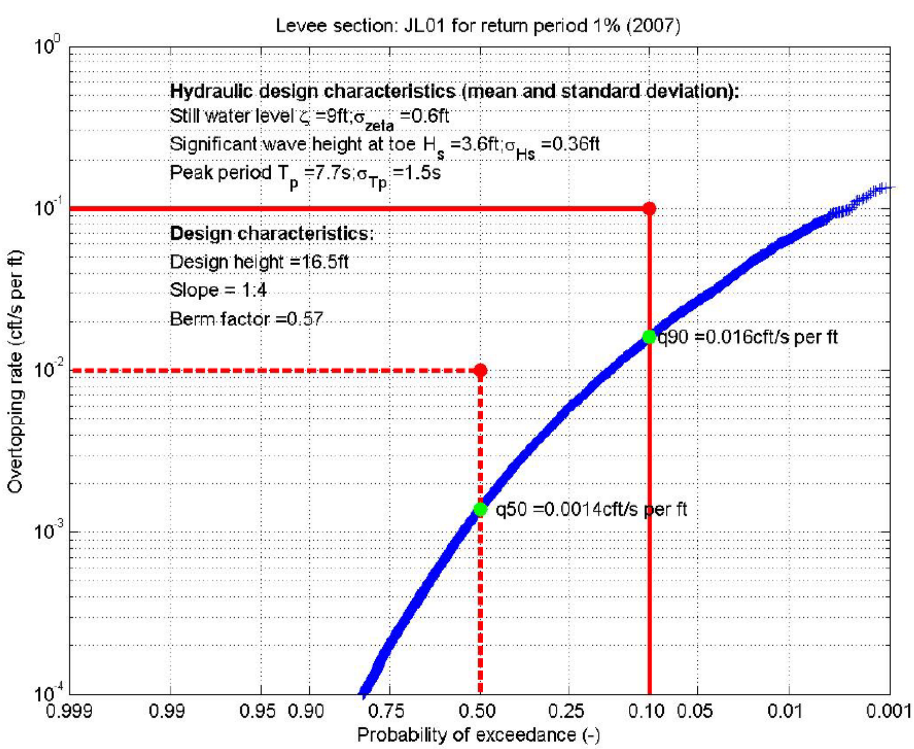

Figure 50. Example of Monte Carlo output for the Jefferson Lakefront levee JL01 from 2007 code.

Design elevations were re-computed at each segment taking into account wave berms and other natural ground elevations on the flood-side of the levee/floodwall. When these features were taken into account, the wave heights were reduced in some areas and this resulted in lower overtopping and lower required design elevations. The lower required design elevations reduced to need to rip out and replace certain expensive floodwalls. I would consider the first round of design elevations as a very rough draft that was updated as the project progressed. The system includes 427 unique design segments, each with their own hydraulic boundary conditions (SWL, Hs, Tp and uncertainty), geometry, and foreshore parameters. The second round of designs involved going to every segment and identifying foreshore elevations. In the Monte Carlo overtopping analysis, these higher foreshore areas reduce wave heights. These features are not captured in ADCIRC+SWAN models, so they must be incorporated into the Monte Carlo overtopping analysis.

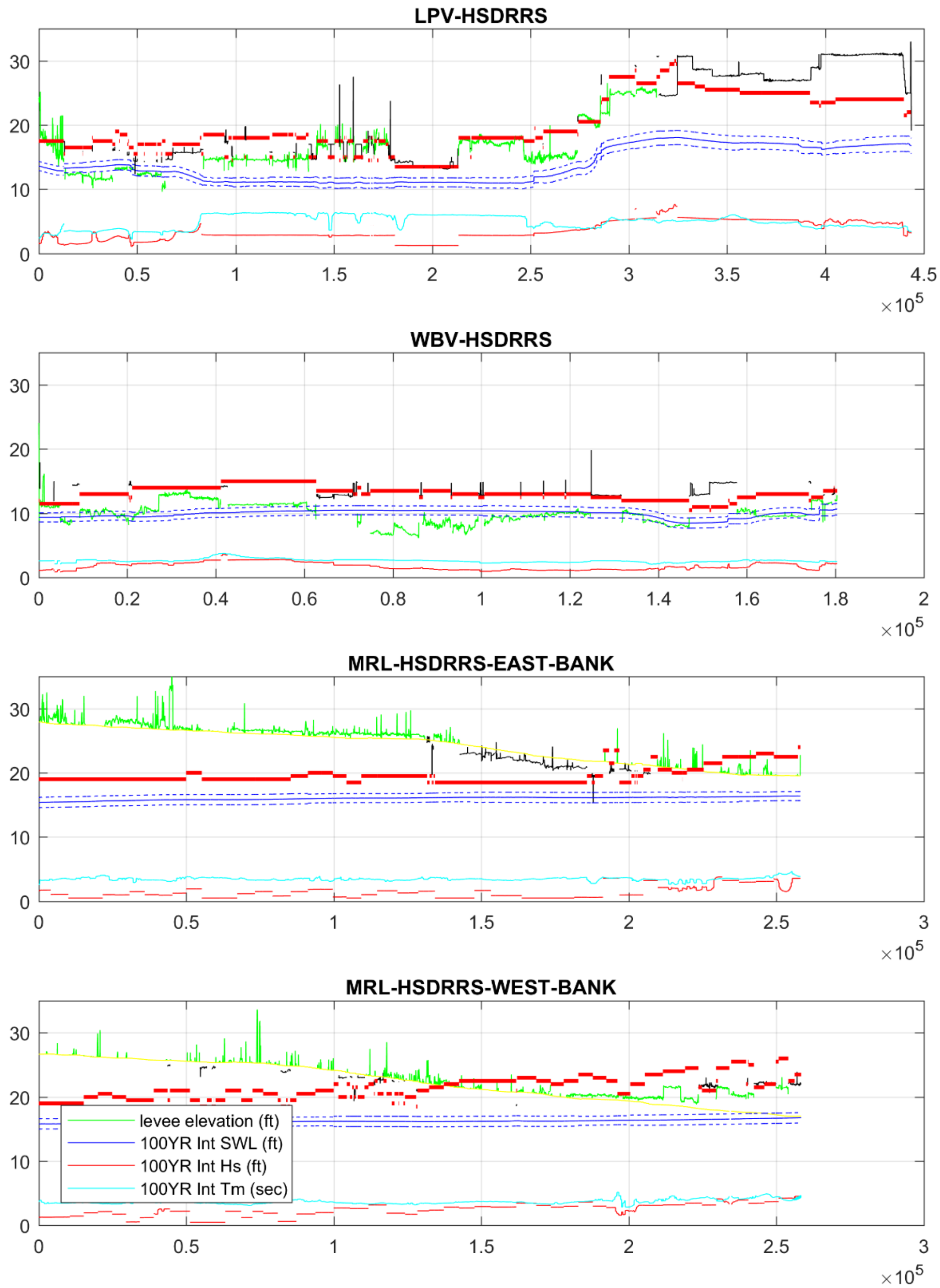

Figure 51. 2073 100YR required design elevations (thick red line) and still water level (SWL), significant wave height (Hs) and mean wave period (Tm) for intermediate RSLC scenario

The overtopping and RAS simulations were conducted for the with-project condition. Figure 53 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 30. 500YR with-project inundation is presented in Figure 54 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 55 and Figure 56 display the 100YR and 500YR flood depths for the with project condition (100YR HSDRRS) assuming intermediate RSLC conditions.

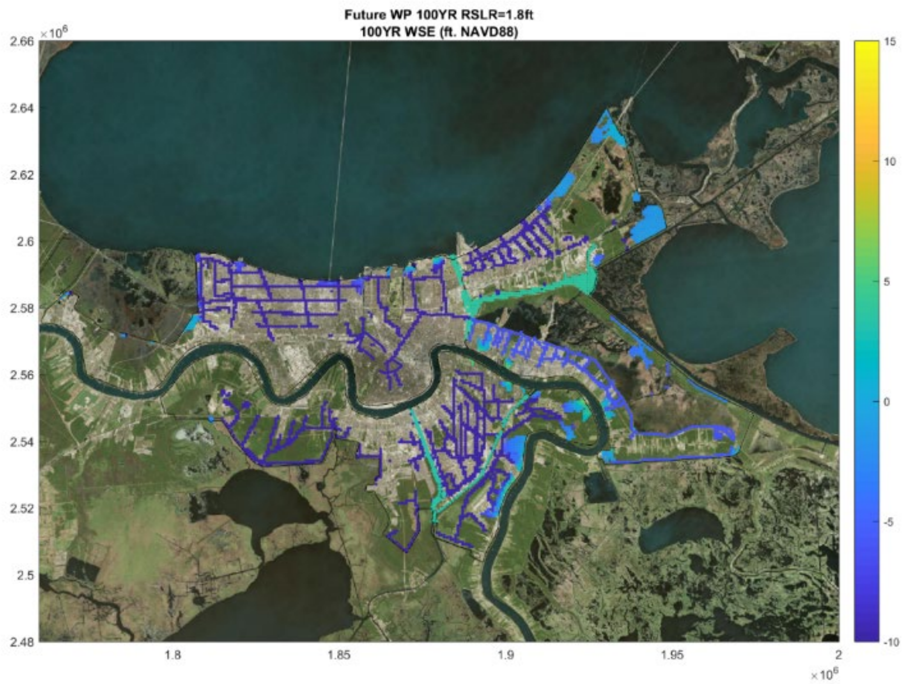

Figure 52. 100 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

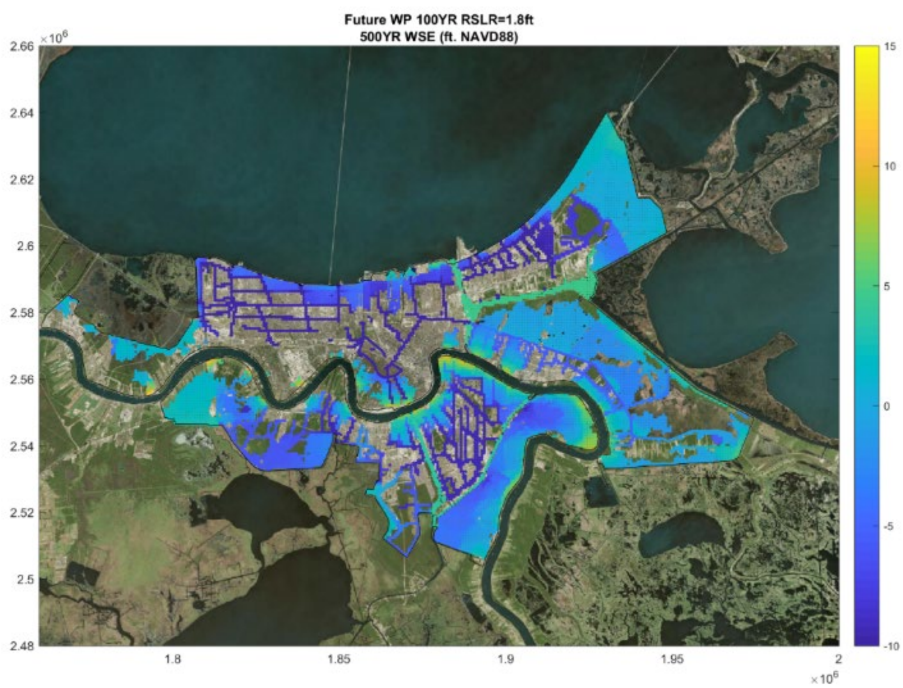

Figure 53. 500 year peak water surface elevation (ft. NAVD88) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

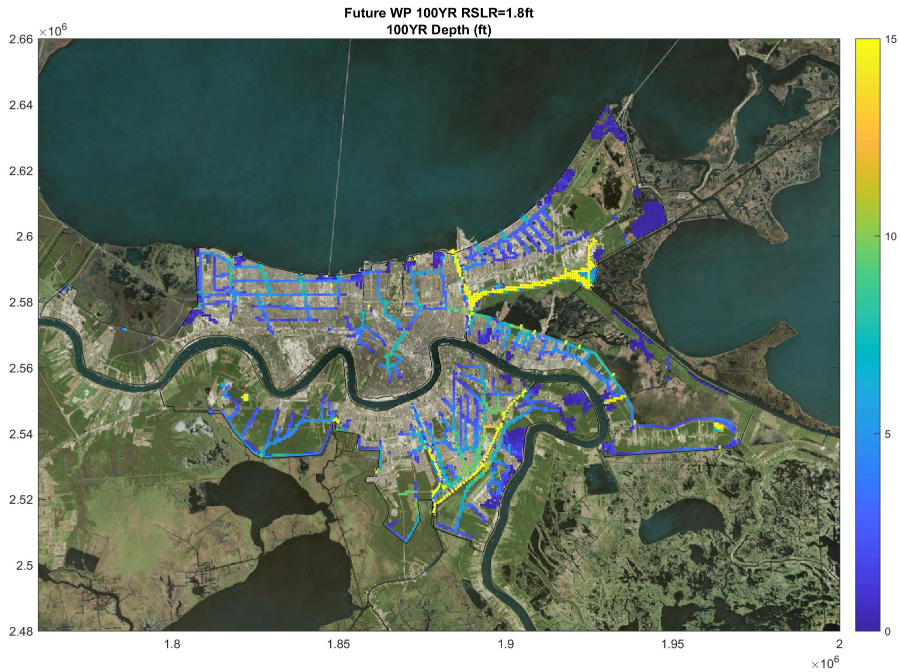

Figure 54. 100 year peak depth (ft.) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

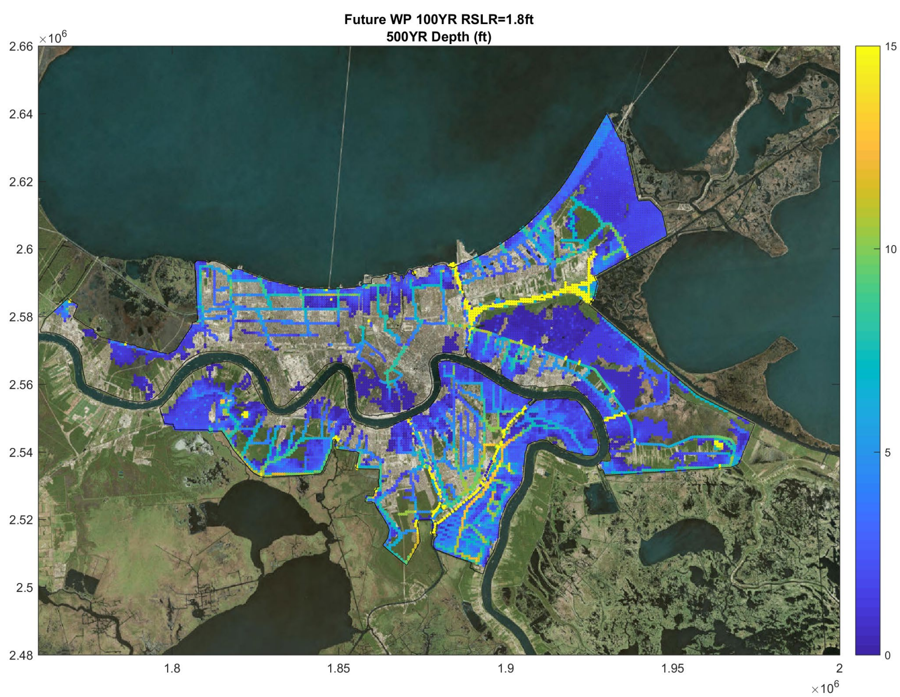

Figure 55. 500 year peak depth (ft.) for future 2073 intermediate RSLC scenario – With 100YR HSDRRS

3.12 PROJECT IMPACTS

ADCIRC simulations of all 152 storms were completed for future condition assuming intermediate RSLR for with and without the 100YR levee system. Future condition without project simulations show tremendous interior inundation as the system no longer meets the 100YR design criteria due to levee settlement and RSLR. When the system is lifted to 100YR future design elevations, the interior flood volumes will be displaced and raise water levels in the exterior. The ADCIRC simulations of with and without project gives an estimate of the induced flooding impacts for 2070 conditions, when the differences in interior/exterior water levels between with and without project are expected to be largest. Figure 60 through Figure 65 display the 50 through 1000YR without project water levels. The without project simulations and resulting statistics show a large volume of water entering the polders around the 100YR and above. Figure 66 through Figure 71 display the 50 through 1000YR with project water levels. The with project simulations show less inundation inside the polder, especially for 100YR conditions. Figure 72 through Figure 77 show the difference in water level between with and without project for the various alternatives. The worst increase in exterior flooding occurs for the 1000YR storm surge. For the extreme return period, the interior water levels in some areas are reduced by 10ft, exterior water levels generally increase less than 0.5ft. The actual water level statistics were passed to the PDT in order to evaluate economics effects and impacts to other projects.

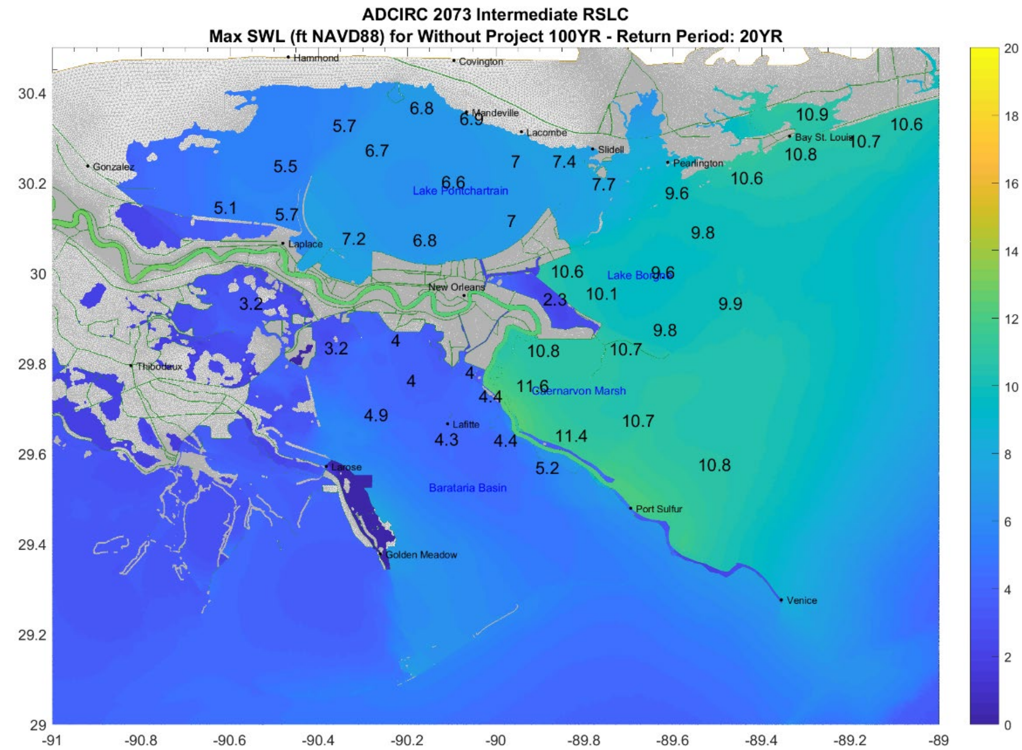

Figure 56. 2070 Without Project 20 year peak water surface elevation (ft. NAVD88)

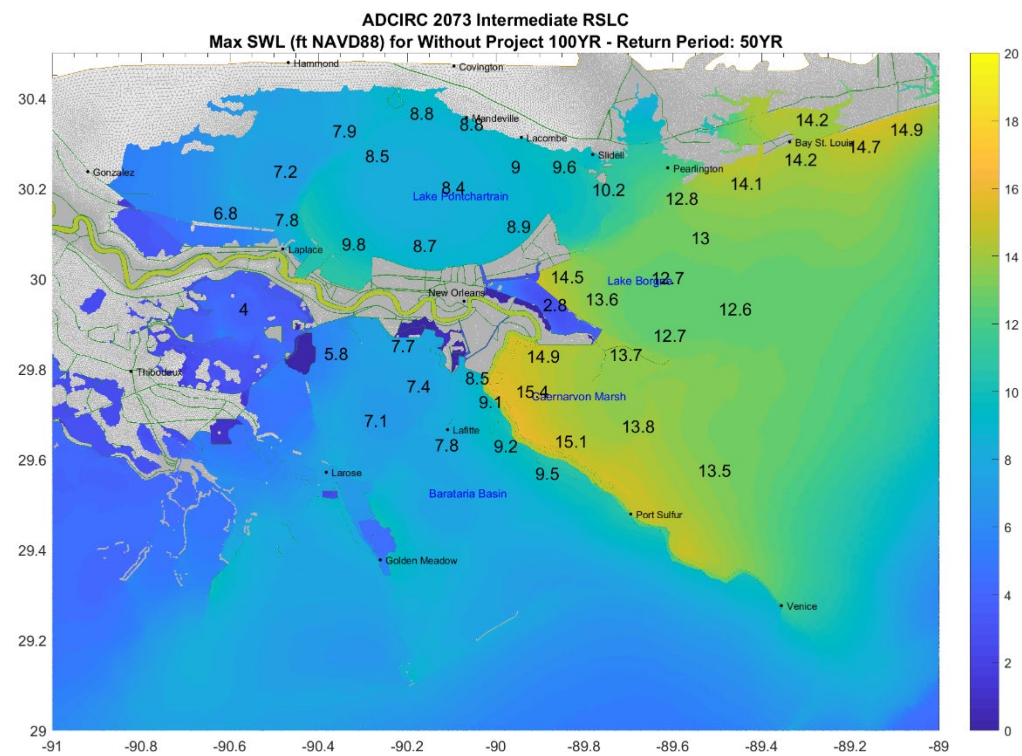

Figure 57. 2070 Without Project 50 year peak water surface elevation (ft. NAVD88)

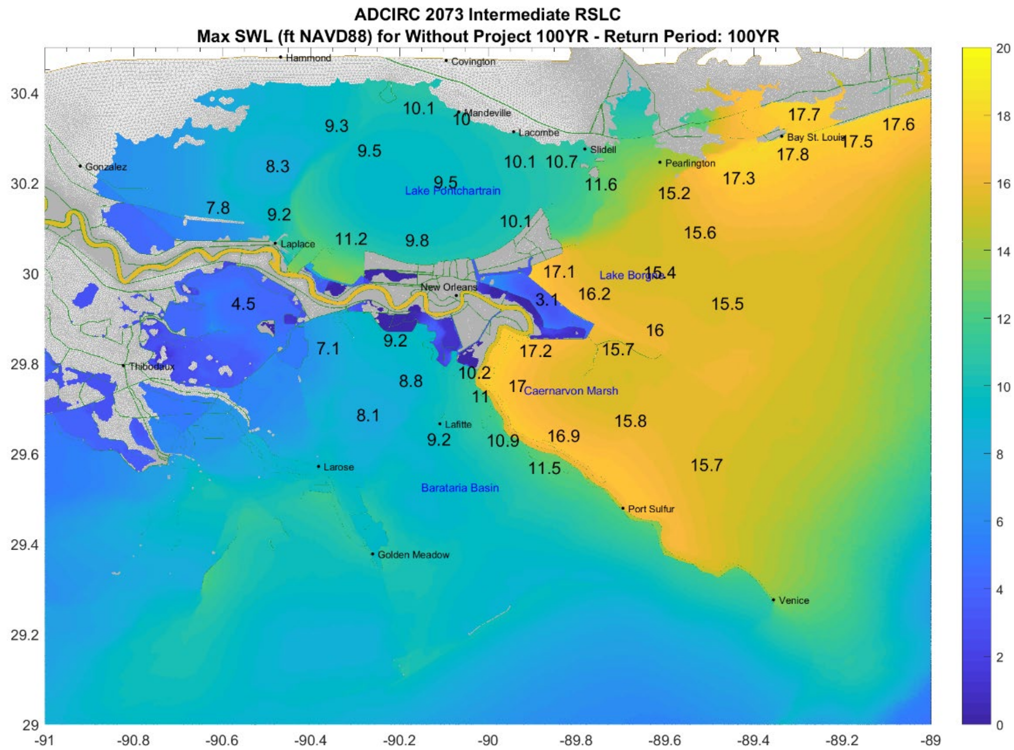

Figure 58. 2070 Without Project 100 year peak water surface elevation (ft. NAVD88)

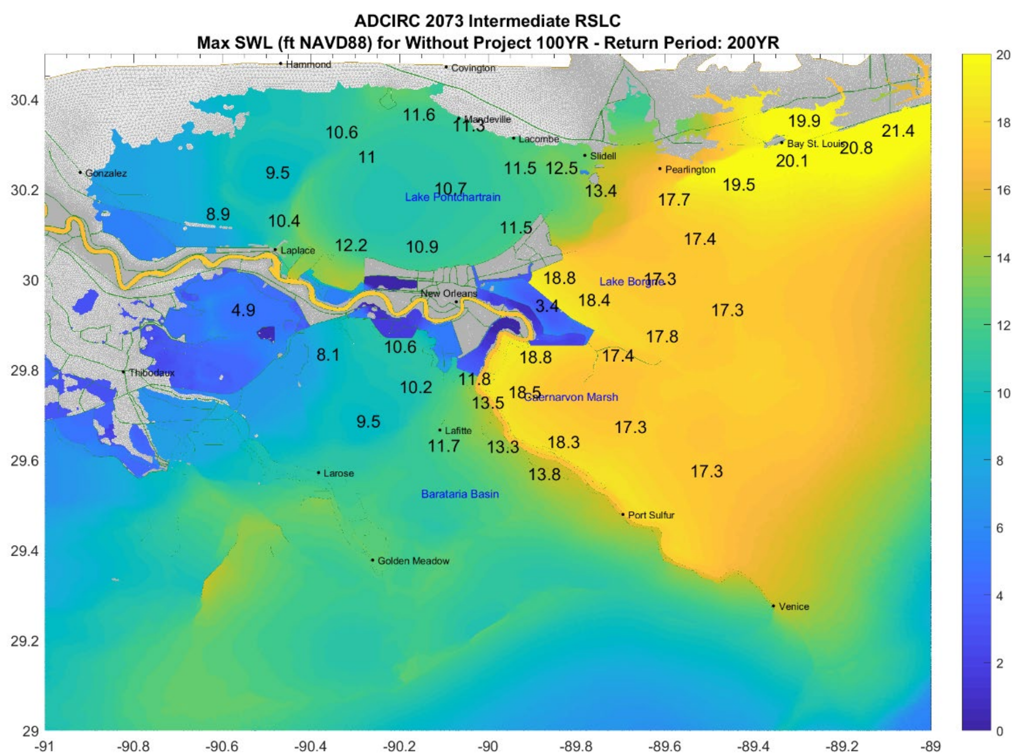

Figure 59. 2070 Without Project 200 year peak water surface elevation (ft. NAVD88)

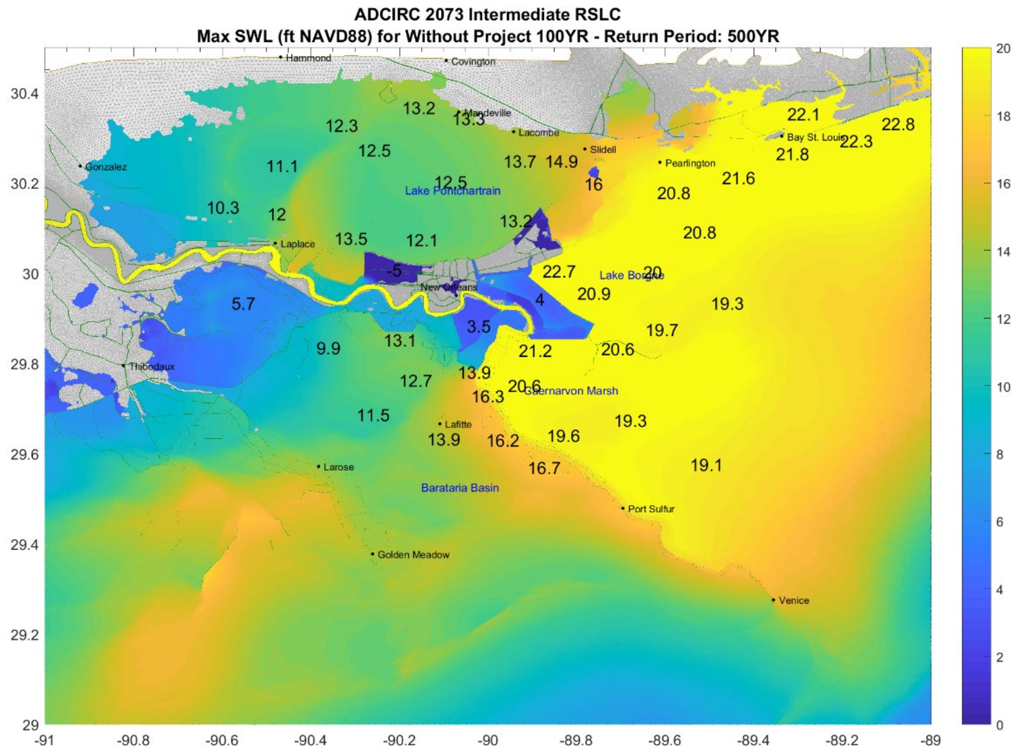

Figure 60. 2070 Without Project 500 year peak water surface elevation (ft. NAVD88)

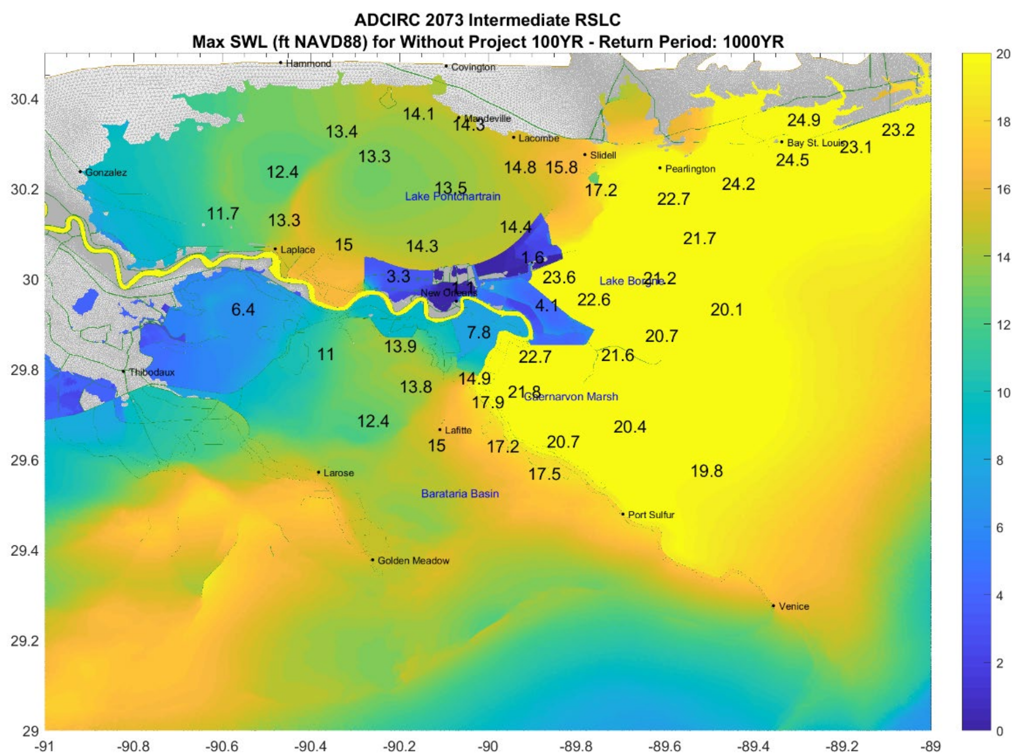

Figure 61. 2070 Without Project 1000 year peak water surface elevation (ft. NAVD88)

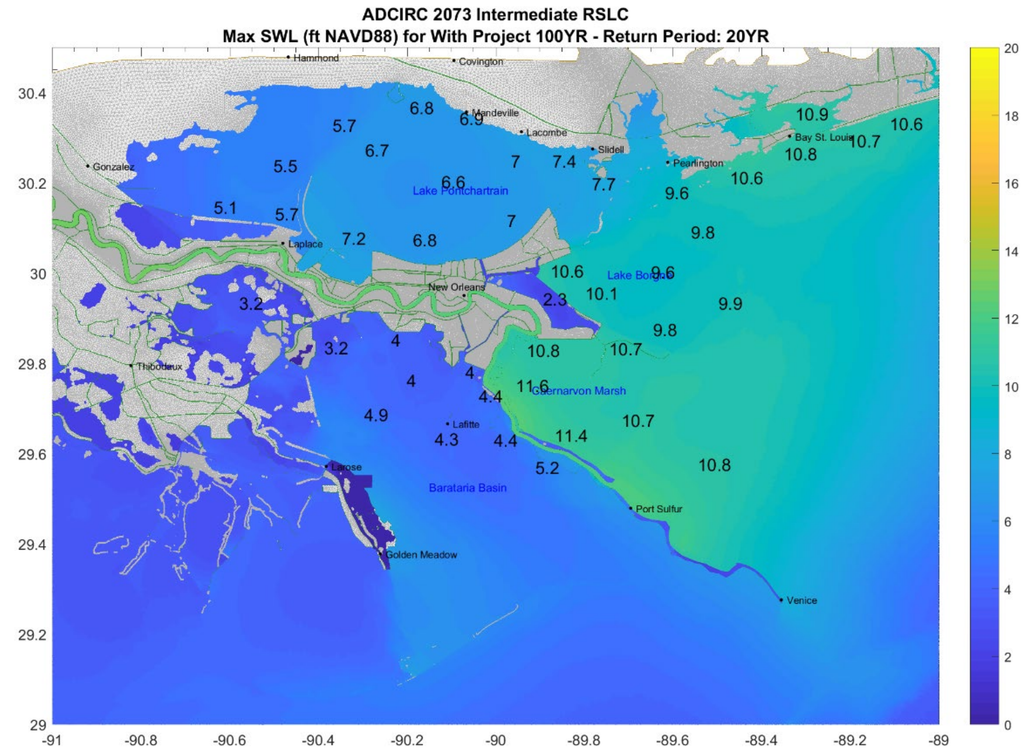

Figure 62. 2070 With Project 20 year peak water surface elevation (ft. NAVD88)

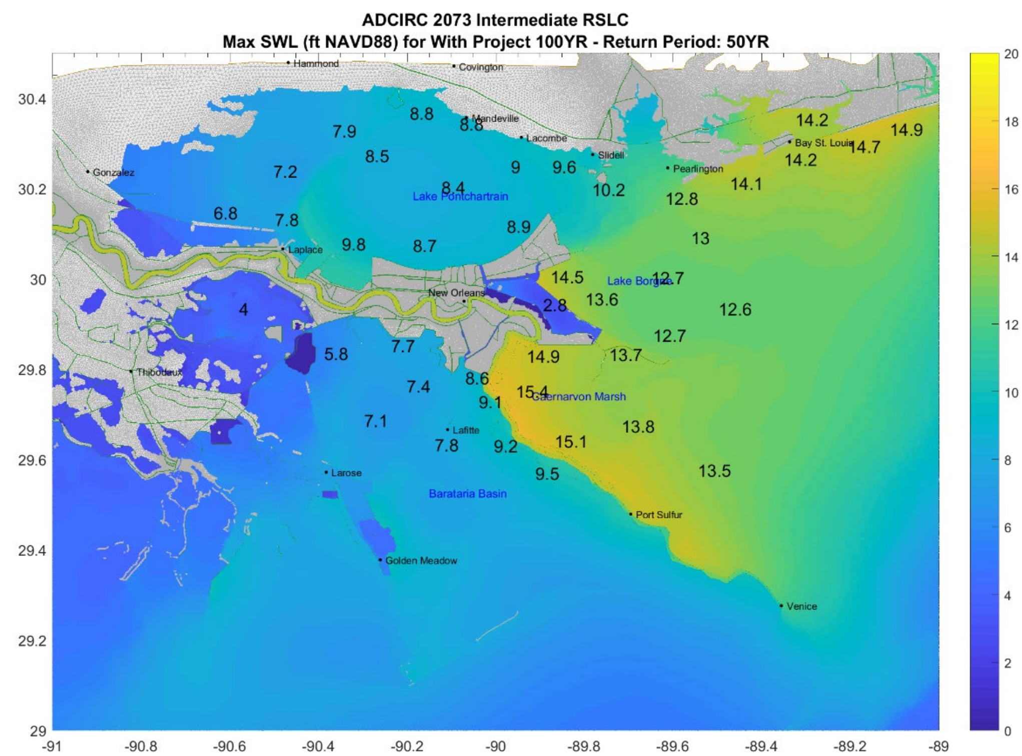

Figure 63. 2070 With Project 50 year peak water surface elevation (ft. NAVD88)

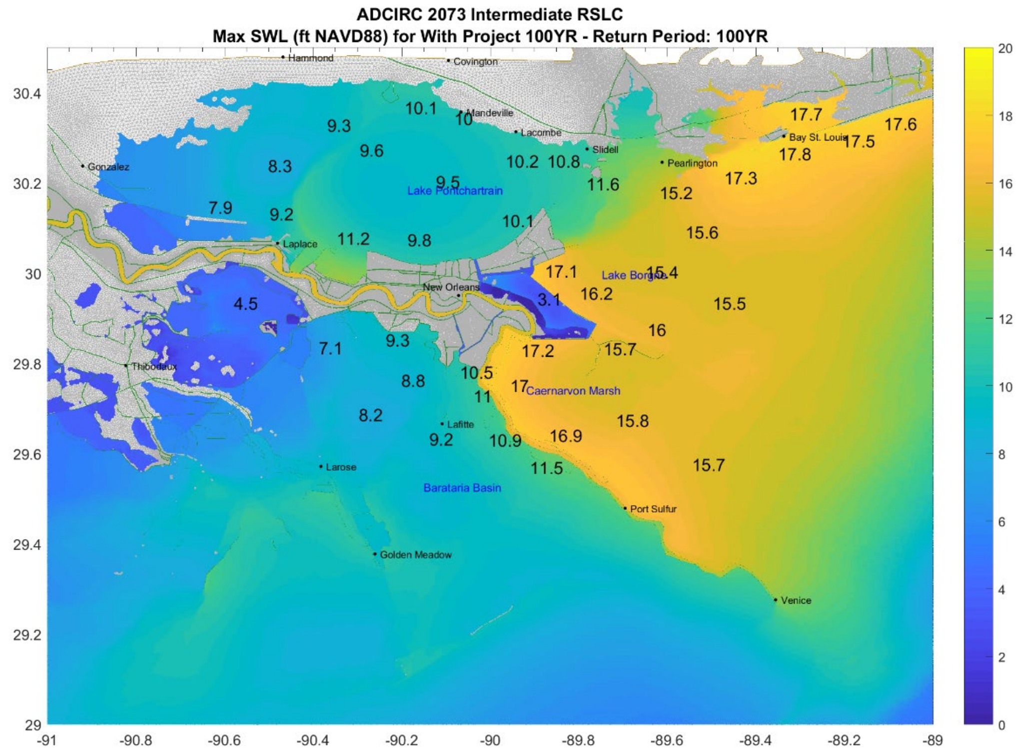

Figure 64. 2070 With Project 100 year peak water surface elevation (ft. NAVD88)

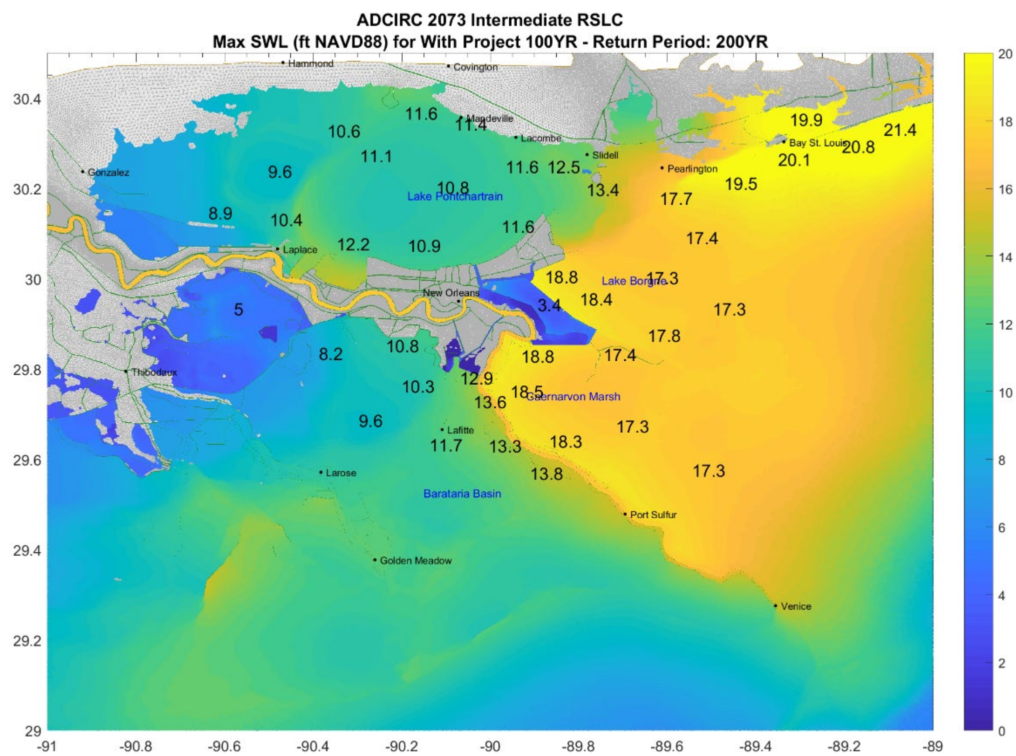

Figure 65. 2070 With Project 200 year peak water surface elevation (ft. NAVD88)

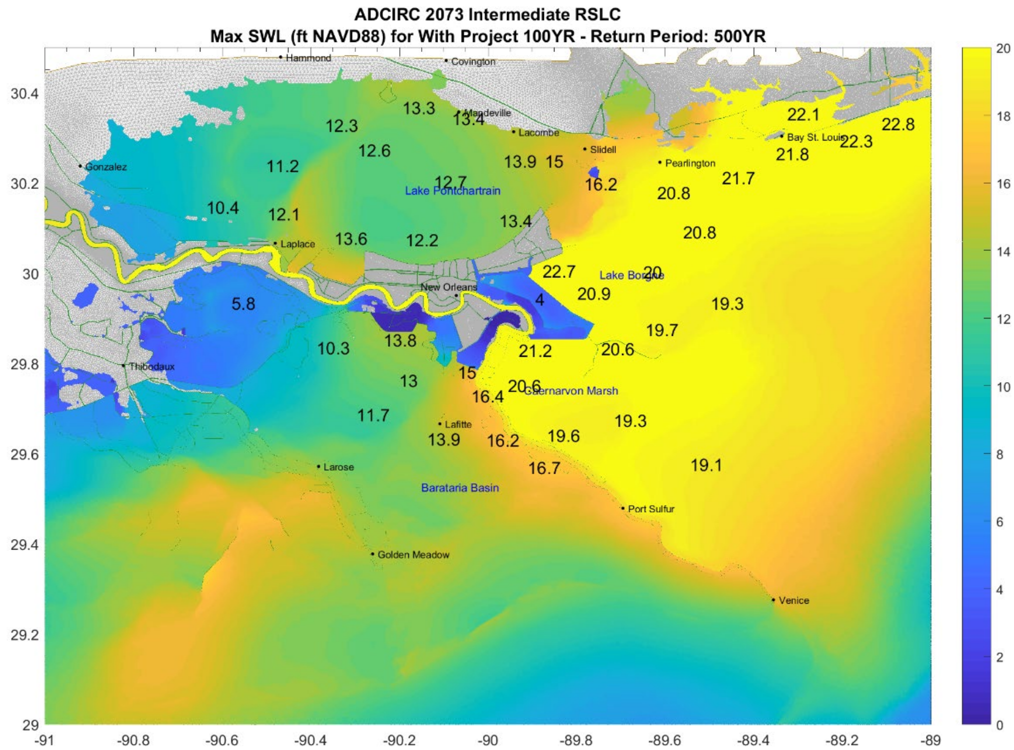

Figure 66. 2070 With Project 500 year peak water surface elevation (ft. NAVD88)

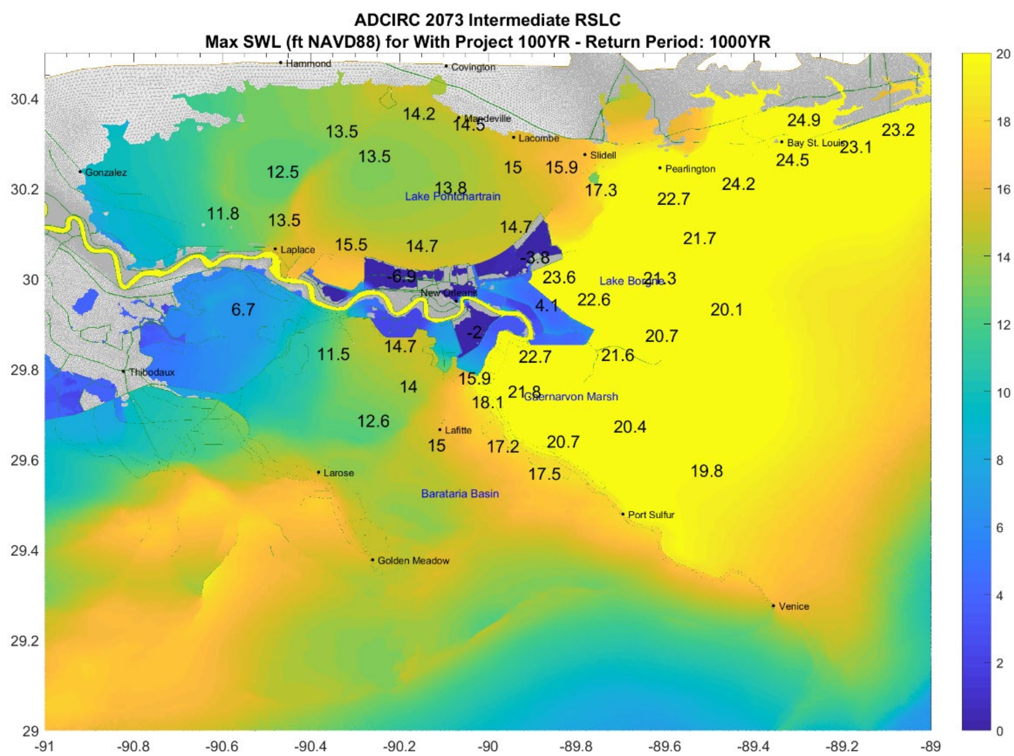

Figure 67. 2070 With Project 1000 year peak water surface elevation (ft. NAVD88)

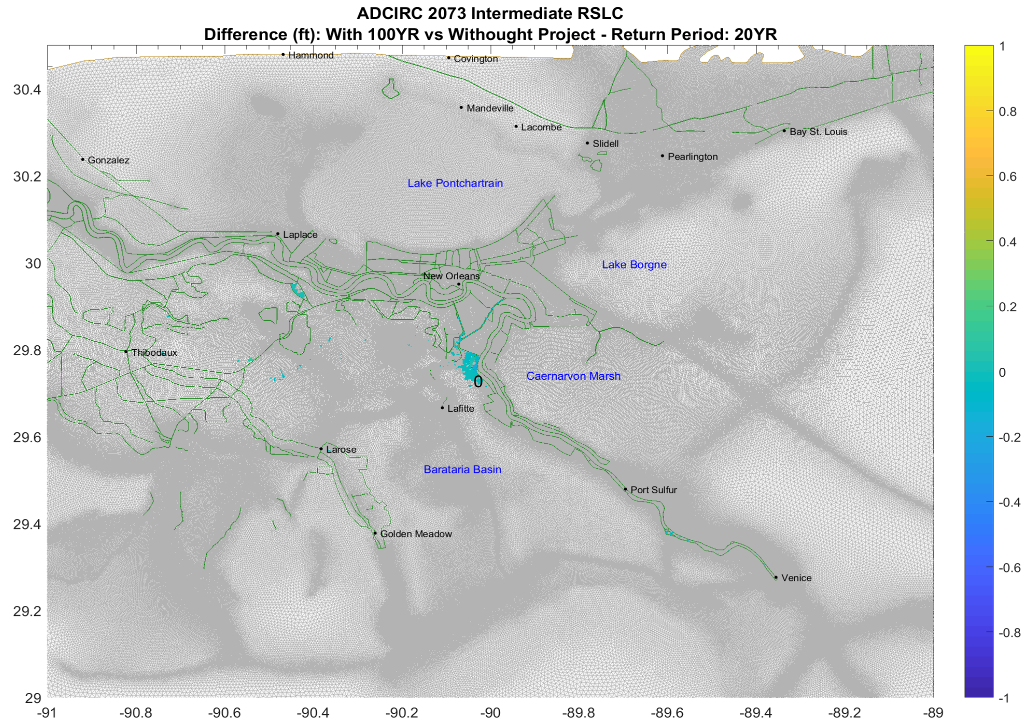

Figure 68. Difference in 20 year maximum water surface elevation between with and without project (ft)

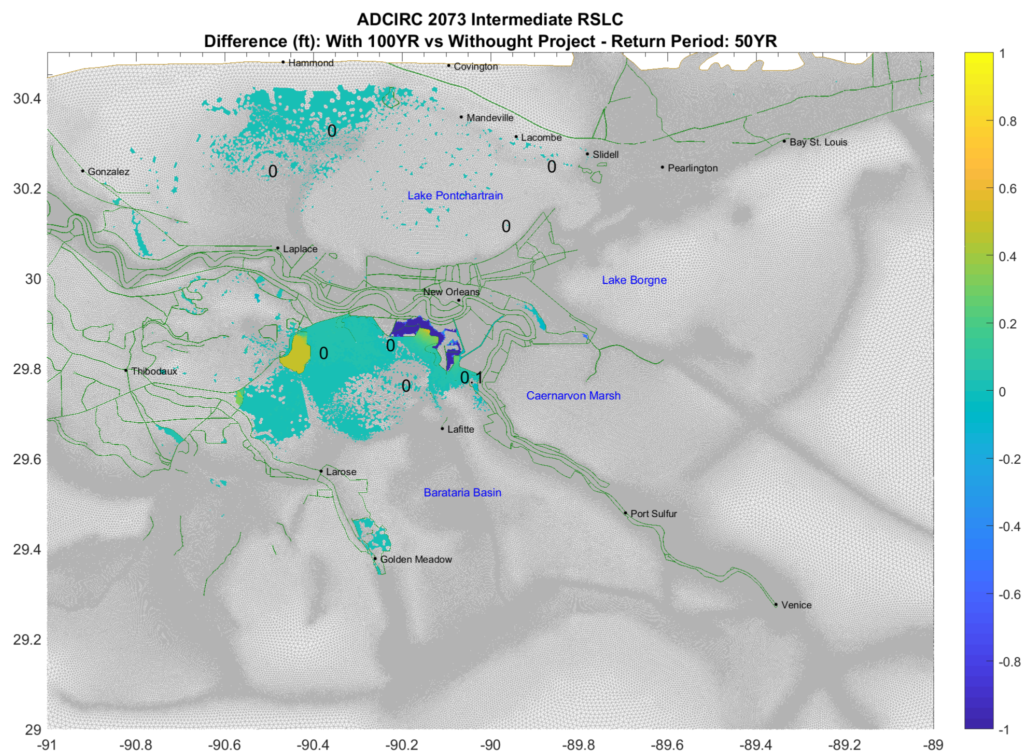

Figure 69. Difference in 50 year maximum water surface elevation between with and without project (ft)

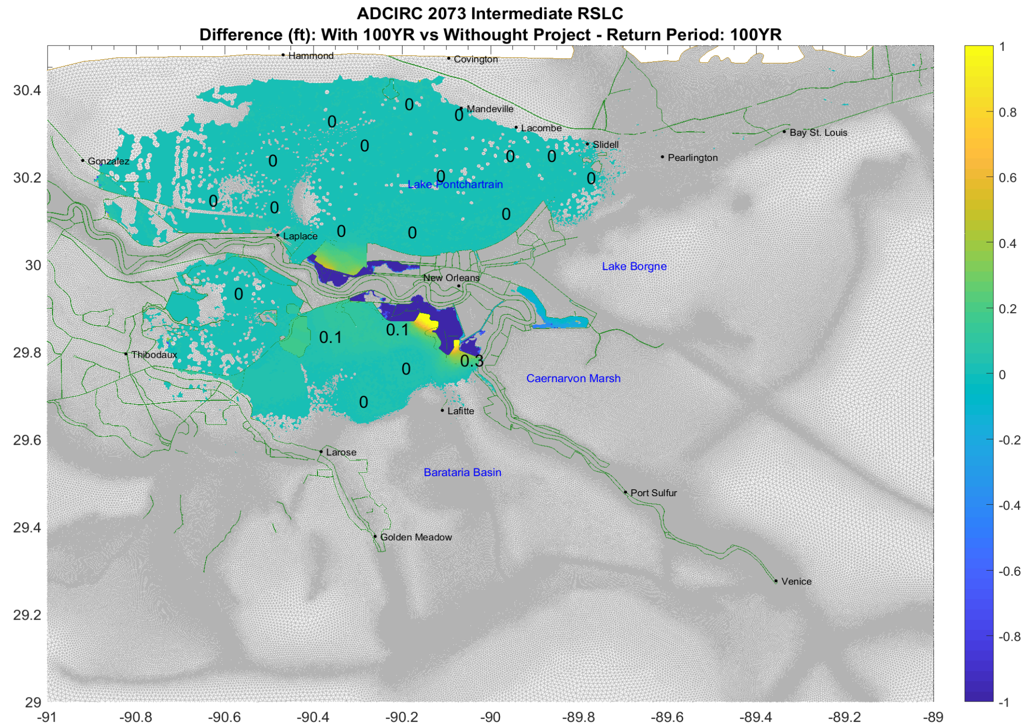

Figure 70. Difference in 100 year maximum water surface elevation between with and without project (ft)

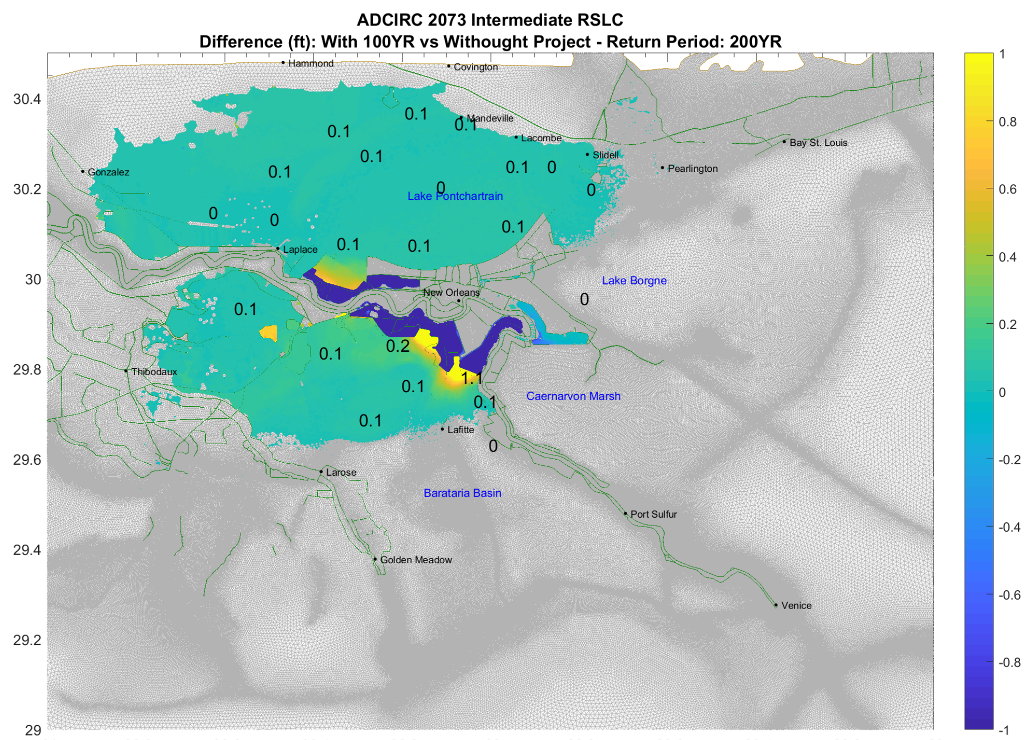

Figure 71. Difference in 200 year maximum water surface elevation between with and without project (ft)

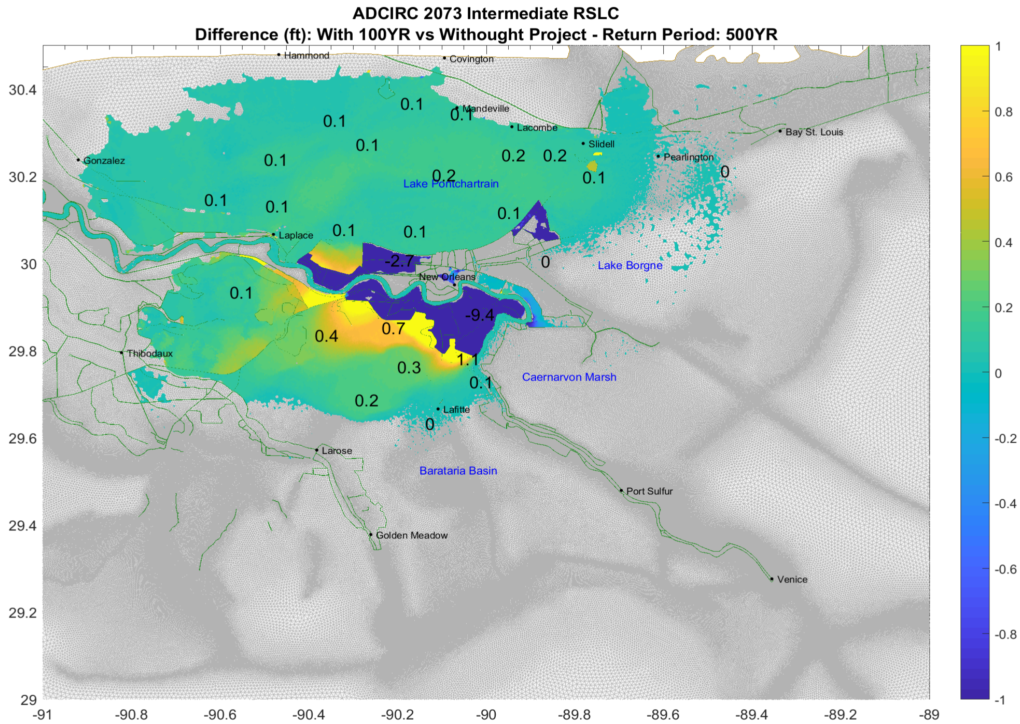

Figure 72. Difference in 500 year maximum water surface elevation between with and without project (ft)

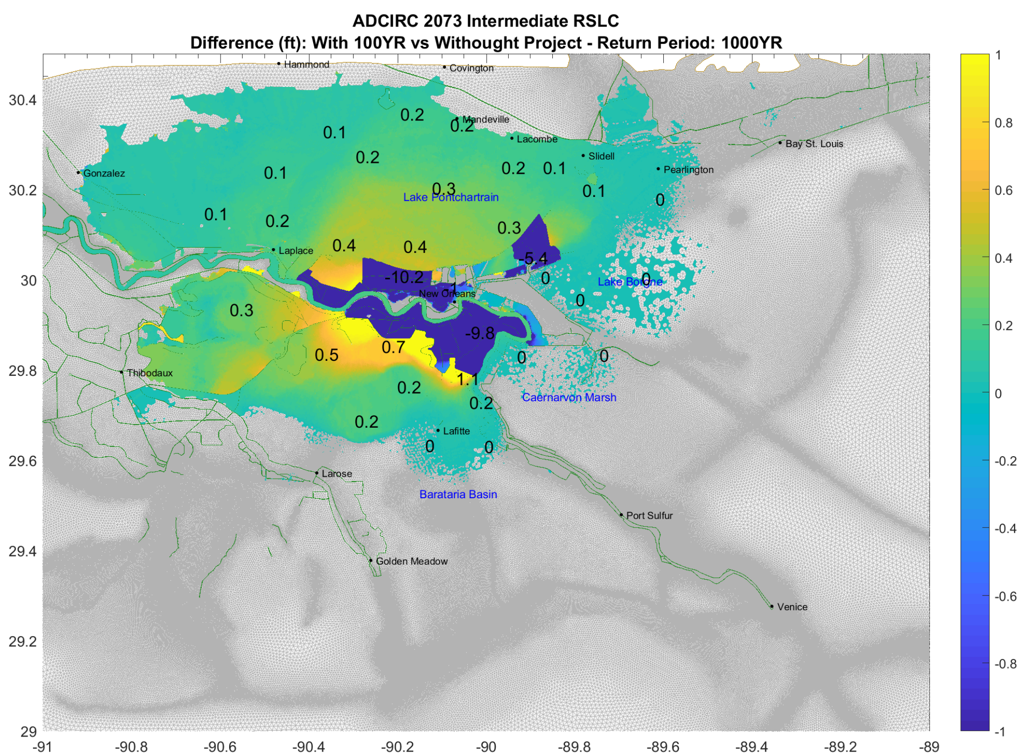

Figure 73. Difference in 1000 year maximum water surface elevation between with and without project (ft)

3.13 CRITICAL INFRASTRUCTURE DISCUSSION

USACE guidance requires that special consideration is given to critical infrastructure within the polder such as hospitals, airports, schools, refineries, other high value facilities. The guidance requires the PDT to evaluate possible solutions to further reduce flood risk for critical infrastructure. In the case of HSDRRS, one potential solution might be compartmentalization by either building a small ring levee or floodwall around certain areas containing critical infrastructure. The PDT did identify certain pieces of critical infrastructure within the floodplain, but could not identify realistic ways to further reduce flood risk in these areas. One of the challenges of a separate ring levee or floodwall around a specific portion of the dense urban area is the lack of real estate and high cost of construction in the urban setting. Furthermore, any area compartmentalized would need its own interior drainage system, such as a small pump station to remove rainfall flooding. The existing sub-surface drainage system for any given compartmentalized area would need to be modified, leading to additional design challenges and project costs. Ultimately, the PDT decided the best way to further reduce risk for critical infrastructure would be raising and armoring the perimeter system.

4 ASSUMPTIONS AND LIMITATIONS

HEC-RAS MODEL

- The HEC-RAS polder model used in this analysis was validated with hurricane Katrina. Katrina would be the only storm available for validation of the interior flood model. Hurricane Betsy may be another storm that could be used for validation but data is limited from the 1960s. Typically, hydraulic models are validated with more than one storms. Without more validation, there is a high degree of uncertainty in interior flooding results.
- The pump station flows in HEC-RAS 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 as well as other operating criteria which are uncertain. 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. The lack of rainfall associated with the synthetic storms is perhaps offset by the neglect of many of the smaller scale subsurface drainage features.
- Sub-surface drainage features were not accounted for in HEC-RAS geometry. Specifically, all catch basins and small culverts in subdivisions that bring water to the large open canals are not modeled. This lack of subsurface drainage features has an effect of raising water levels in neighborhoods since water is forced to flow overland instead of underground in culverts. Subsurface drainage would likely have a small effect during large overtopping events as the culvert volume would quickly be overwhelmed.

LEVEE FRAGILITY

- No breaching or floodwall failures was accounted for in the HEC-RAS modeling. Breaching would make the interior inundation potentially much worse for certain storms.
- The assumption of no breaching may be reasonable assuming most levees are armored with high performance, turf reinforced map (HPTRM) and the backside of floodwalls are armored with splash pads. Some levees are not armored including those above the

cross-over point on the MRL. During extreme overtopping events, such as the 500YR or 1000YR without-project, breaching may have less of an influence since the polder is filling to extreme water levels anyways.

- Breaching and levee fragility would be difficult to incorporate into the existing framework of the HEC-RAS model. The weir equation and Eurotop wave overtopping equations are used to determine volumes entering the system. These equations work well for levees and floodwalls that stay intact. During a breach, the equations no longer apply, since the interior can fill and slow down inflows. In other words, the flow into the system becomes tail-water influenced. Also, the situation becomes even more complex, because if there is a breach, the exterior water level drops, reducing head and reducing flows. Modeling breaching is complicated and would push the limits of the one-way coupling of the current model set-up.
- Fragility curves are likely highly uncertain and require a probabilistic approach to fully evaluate. A probabilistic or Monte Carlo based approach to levee fragility and interior flood risk requires many more simulations, perhaps thousands, which is beyond the current capabilities of the interior model.
- The damages due to breaching are most likely in the higher return periods (500YR to 1000YR). For without project conditions, the model is already showing large areas completely inundated. In this case, the effect of breaching might not change the annual expected damages, since the structures are already underwater. If the study provides a high BCR for the 100YR alternative without including breaching, the BCR would likely become stronger if breaching were included, since that would lead to higher expected damages for without project and future without project conditions. The 100YR system, if authorized and funded, will reduce damages associated with breaching, especially with armoring added to all levee segments. Armoring is an essential component of HSDRRS resiliency and reduces the possibility of breaching.

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 and ERDC. New surge and wave estimates are expected to be different than the values developed for this study. It is entirely possible that the updated water levels could be several feet different, and thus the 100YR required design elevations might shift by a similar amount.
- The wave overtopping calculations for the simulation of synthetic storms 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 evaluated deterministically. The uncertainty in water levels was not evaluated in the overtopping and inundation calculations. For example, 90% confidence estimates of inundation would be significantly higher. A probabilistic approach would be useful to evaluate the uncertainty in exterior water levels, waves and overtopping volumes. Ultimately, the economic modeling of damages in the interior accounting for uncertainty in the water levels.

- 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. This amplification effect was found to be rather small in the ADCIRC simulations of with and without project simulations.
- The surge and wave time-series assume coincident peaks. In reality, the timing of peak surge and wave may not correspond exactly.
- In overtopping calculations for design, wave direction is assumed to be perpendicular to the levee for all Monte Carlo samples.

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, although surge statistics from the post-Katrina study (2007 to 2009) were compared to the latest statistics and found to be comparable.
- No estimate of uncertainty is provided in the interior water level statistics. Instead, to address uncertainty, the economics team assigned a “length of record” in FDA. The results of the ERDC statistical code are 50% or average value. 90% statistics would be significantly higher.