3.0 PROPOSED CONDITIONS

3.1 Modeled Features

3.1.1 Master Plan Components

The Master Plan for West River Memorial Park, prepared by Vollmer Associates in 1999, presents the City of New Haven's vision for the future of the park. The plan includes many components, as shown in Figure 3.1-1.

The in-stream components of the Master Plan were investigated directly as part of this study. These features have been proposed for improving water circulation, tidal flushing, and recreation. There is currently no opportunity for water circulation in the upper portion of the reflecting pool. Additionally, water flow in the lower portion of the West River channel is limited due to the choke point at the southern connection of the channel with the dredged reflecting pool. Several in-channel modifications included in the Master Plan address those concerns.

All modifications in the plan that have been included in the proposed conditions models are described below, starting at Orange Avenue and proceeding counterclockwise. Note that any dimensions provided below represent an initial interpretation of the features, with the understanding that they are subject to change as design progresses.

Enhance Ponded Area

The existing ponded area in the southeastern portion of the park will remain connected to the West River under the Master Plan. The proposed conditions for the pond include some reshaping, *Phragmites* removal, and could include planting of its edges. That work would improve habitat and water quality in the pond as well as enhancing aesthetics and recreational opportunities. The modeled pond reflected the extent of excavation depicted in the Master Plan.

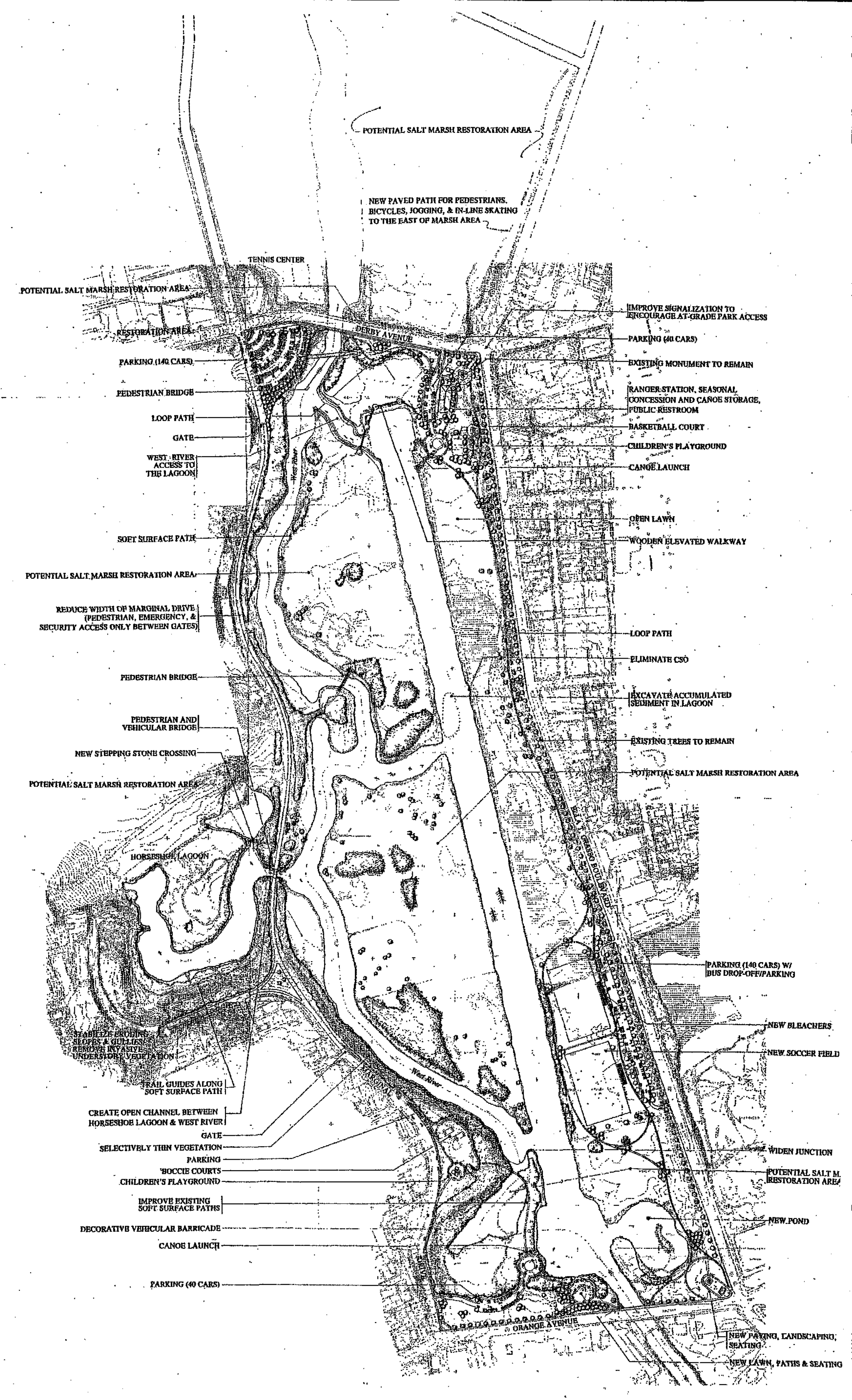
Salt Marsh Restoration South of Soccer Fields

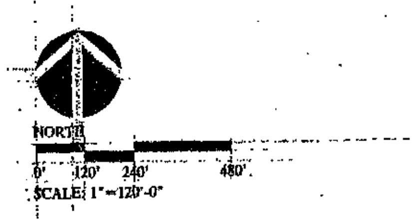
Salt marsh restoration will be difficult to achieve due to the relatively low salinity expected from opening one or more tide gates (see Section 2.3). However, there is an opportunity for tidal marsh restoration (see Section 1.4.3) here, in the existing *Phragmites*-dominated area located to the south of the existing soccer fields and north of the pond discussed above. Marsh restoration work in this location would be well suited to work in conjunction with the proposed pond improvements. Much of the area is above 4.0 feet NGVD, so excavation of the surface would be required to achieve tidal flooding. Hydraulic modeling investigated the extent of tidal flooding of this area under open tide gate scenarios flooding the existing marsh surface as well as an excavated surface.

Future design efforts should include a planned transition to the soccer fields. Under any excavation scenarios, the quality of the spoils and their suitability for other uses/disposal would need to be determined. This area could be regulated as inland and/or tidal wetlands. Additionally, the area below the high tide line (elevation 5.1 feet NGVD - see Chapter 5) would be regulated under the DEP OLISP Structures, Dredging and Fill Program.

Salt Marsh Restoration North of Soccer Fields

As described above, salt marsh restoration will be difficult to achieve in this area. However, this *Phragmites*-dominated area located between the soccer fields and the developed upland area on the Ella Grasso Boulevard side of the Park and is a prime candidate for tidal marsh restoration. Much of the area is low, between 2.7 and 4.0 feet





MASTER PLAN FEATURES

WEST RIVER MEMORIAL PARK
Conceptual Master Plan

Prepared for:
City of New Haven, CT
City of West Haven, CT

Prepared by:
Vollmer Associates
November, 1998

NGVD, and construction access will be much easier here than in the interior salt marsh restoration areas identified on the Master Plan. Due to the lower elevation of this area, some tidal restoration can be achieved by opening one or two tide gates. Excavating the higher portions of the marsh can provide additional restored area. Regulatory issues would be similar to the restoration area described above.

Dredge Sediment Bar

An existing sand bar is located in the reflecting pool downstream of the Legion Avenue combined sewer overflow and storm drain discharge locations. The bar appears to be comprised largely of road sand deposited during storm events. It is currently an obstacle to canoe passage and limits the flushing of the reflecting pool. It is a good area for wildlife, particularly birds. Excavation of the sediment bar, as shown on the Master Plan, was included in the model of proposed conditions in the park.

The sediment quality requires investigation as part of future design efforts in order to determine appropriate reuse and/or disposal. Contamination of the sediments could increase the cost of dredging by an order of magnitude.

Eliminate CSO

The City Water Pollution Control Authority is currently engaged in design of a holding tank system intended to relieve the frequency of overflows from the combined sewer system. Three existing combined sewer overflows (CSOs) discharge into the West River in the Park. The Legion Avenue CSO is one of the more troublesome, discharging into the reflecting pool on a frequent basis. Compounding the difficulty, the overflow from the Legion Avenue CSO stagnates in the reflecting pool, as there is little to no flushing upstream of the discharge location. Elimination of the CSO would certainly improve water quality in the Park in every respect. The raised berm over the CSO provides convenient access to the reflecting pool from the Boulevard. It can fulfill the same function for the restored salt marsh on either side. Although MMI did not address this option directly, the amount of flushing upstream of the Legion Avenue CSO was qualitatively evaluated under proposed conditions. The potential for flooding of the sewer system was also investigated.

Salt Marsh Restoration At Head Of Reflecting Pool

As with the other salt marsh restoration targets described above, this *Phragmites*-dominated area located between the head of the reflecting pool and Derby Avenue is a good candidate area for tidal marsh restoration but may not be able to realize the full benefits of salt marsh restoration. Removal of the Phragmites would improve the view of the Park from the road, an objective supported by the local community. The area is already somewhat low, allowing for some tidal flooding with the opening of one or two tide gates. Regrading would encourage additional flooding. Both restoration options were investigated by MMI as part of this study. Regulatory issues would be similar to the other salt marsh restoration targets discussed above.

Cross-Channel to Reflecting Pool

This concept will undeniably benefit the water quality in the reflecting pool. However, the diversion of water from the West River will be of concern to environmental regulators. The base elevation of the cross-connection has been set above the base elevation of the riverbed, allowing the channel invert to act as an overflow weir from the river to the reflecting pool. This approach will improve conditions in the dredged channel without having a significant negative ecological impact in the river. The modeled cross-channel was envisioned as a tidal creek. It is approximately 25 feet wide and trapezoidal in shape, with the channel invert ranging from -1.0 feet to -1.5 feet NGVD.

Salt Marsh Restoration across from Tennis Center

This large area dominated by *Phragmites* is located across the West River from the Volvo Tennis Center. It is bounded by Derby Avenue to the south, Ella Grasso Boulevard to the east, Chapel Street to the north, and the West River to the west. As with the previously described *Phragmites*-dominated areas, a full salt marsh might not be able to be achieved here, although tidal marsh restoration is possible and would be beneficial. The elevation of this area would limit the amount of flooding, although tidal marsh restoration could be achieved through excavation of the area. The extent of inundation in this area was investigated under existing and excavated scenarios. Given the large amount of area here, there is a great potential for a crafted landscape with tidal creeks providing water to the interior of the land area.

Salt Marsh Restoration on 'Northern Peninsula'

There is good potential for tidal marsh restoration at the lower elevations of the peninsula (2.0 to 4.0 feet NGVD). Previous discussions of salt marsh restoration potential and regulatory issues apply here as well. The Master Plan is not clear on the extent of the proposed restoration in this location. Some interested members of the local community believe that the existing plant communities on the southern portion of the peninsula have developed into interesting habitat and are worthy of preservation. Their idea – to mix tidal marsh restoration in the *Phragmites* dominated areas (approximately one-third of the peninsula) with preservation of existing field and forest – is an interesting alternative.

The extent of flooding of the existing marsh surface was evaluated as part of this study. An excavation alternative was not considered for this location for the following reasons: 1) expressed interest in preserving the existing vegetation; 2) potential difficulty in gaining construction access.

Bridge to Horseshoe Lagoon

Replacement of the existing culvert to Horseshoe Lagoon with a bridge capable of carrying vehicular traffic will both enable canoeists to travel from the West River to the Lagoon as well as provide increased tidal fluctuation in the lagoon. A 15-foot bridge crossing was modeled. The channel bed below the bridge was modeled as trapezoidal in shape, with an invert elevation of -3.5 feet NGVD. Those dimensions were selected to enable canoe passage and tidal flushing. They are subject to change with design of the bridge.

Salt Marsh Restoration At Horseshoe Lagoon

The Master Plan is not clear regarding the location of this restoration. The elevation of the land in this area is relatively high, limiting the opportunity for tidal marsh (in addition to the previously described difficulties with salt marsh restoration). However, the increase in tidal flow into the freshwater lagoon will initiate a transition to a brackish environment characterized by a different plant community. Regulatory agencies are likely to view this change positively.

Salt Marsh Restoration on 'South Island'

This area is only about one-quarter *Phragmites* now, with the largest block located on the eastern side of the island. Given the relatively low elevation of the island, there is potential for tidal marsh restoration here. There is also a danger, however, of *Phragmites* invasion of higher elevation areas. Disturbance of the island should be kept to a minimum to limit that potential invasion. The extent of tidal flooding of the existing marsh surface has been evaluated in the modeling effort. Excavation of the island marsh was not considered due to the complexities of construction access and the relatively low benefit of that complex construction given the lack of current dominance by *Phragmites*.

Widen Southern Confluence of West River and Reflecting Pool

The existing southern connection between the West River and the dredged channel is constricted, limiting flow in the natural channel and directing much of the river's flow through the lower portion of the dredged channel. Expanding this connection would help balance the flow. The new connection is approximately 100 feet wide as modeled, with a 30-foot wide channel invert of -5.0 feet NGVD. For comparison, the existing connection is approximately 60 feet wide, with a 30-foot wide channel bottom of -2.5 feet NGVD.

Salt Marsh Restoration in Southwestern Section of the Park

This low-lying area is located immediately south of the abrupt 90-degree turn of the West River channel, where the West River turns to meet the dredged channel and tide gates. This area is likely the channel bed and floodplain before dredging and installation of the tide gates. Water is frequently ponded here and the marsh is dominated by *Phragmites*. It is an excellent target for restoration. The extent of tidal flooding of the existing marsh surface has been evaluated in the modeling effort.

3.1.2 Tide Gate Modifications

Modifications of the existing tide gate have been proposed to increase the amount of salt-water flooding in the upstream marsh and provide the potential for salt marsh restoration. While tide gate modification can help with those goals, it can also lead to damage of existing infrastructure as well as existing plant communities that are intolerant of salt-water or of increased flooding. This study closely examined the impacts of opening one or more tide gates on the existing upland land uses and infrastructure.

Specifically, this analysis considered the removal of one, two, and three tide gate flaps of the 12 flaps currently in place. There are many options available for tide gate modification that will increase salt water flooding in the marsh upstream, as reviewed in

Appendix C. This analysis considered only those options that rendered a given opening in the tide gate structure either fully open or fully closed.

Given the low salinity of the West River, the option of opening only the lower portion of a tide gate flap warrants future study. This option has the potential of increasing the salinity of the water in the upstream marsh since it would allow only the lower, more saline water through the tide gate structure. Therefore, this option might provide an increased chance of success for salt marsh restoration over a more traditional tide gate modification. There are no known cases of implementation of a tide gate modification of this type.

3.1.3 Marsh Excavation

Excavation of the low-lying areas upstream of the tide gates has also been proposed as a potential method of achieving tidal marsh restoration. This would involve the excavation of the marsh to approximately the elevation of mean high water. The modeling effort targeted the following areas dominated by *Phragmites* and accessible to construction vehicles:

- between the pond and the soccer fields;
- between the soccer fields and the monument area;
- at the head of the reflecting pool; and
- across from the Volvo Tennis Center.

The additional areas targeted for restoration in the Master Plan were not evaluated for excavation. The northern peninsula and southern island were not targeted for excavation since they are largely dominated by vegetation other than *Phragmites*. Members of the local community have expressed an interest in preserving the vegetation at those locations, particularly the northern peninsula. Additionally, construction vehicle access to the island would be complex due to the required river crossing. Construction access to the peninsula would also be somewhat difficult, as it would require crossing marshy areas and potentially require the placement of fill to create an access road. The relatively complex - and correspondingly expensive - access required to excavate those areas does not appear to be warranted given the relatively low dominance by *Phragmites*. If the *Phragmites* should invade those areas more aggressively, future consideration could be given to excavation.

The Horseshoe Lagoon area targeted for salt marsh restoration was not considered for excavation due to its relatively high elevations and dominance by upland vegetation. The *Phragmites* dominated area in the southwestern portion of the park was not considered for excavation due to its already low elevation enabling tidal flooding.

3.2 Plan Features Targeted for Upcoming Construction

3.2.1 New Soccer Fields, Bleachers, and Parking

The City of New Haven is highly committed to improvements of the existing soccer fields located in the south of the park adjacent to Ella Grasso Boulevard. These playing fields were created in the 1920s by the placement of fill material. In the past, citizens of New Haven and West Haven have enjoyed both soccer and baseball there. Today, the fields are undergoing renovation to correct drainage problems due to poor pitch and soil compaction. Plans also call for improvements to site access and additional parking.

The design plans being pursued by the City of New Haven for the West River Memorial Park soccer fields show the fields to be sloped at 1.5% pitch evenly across both the soccer fields and the intermediate buffer area. The plans also call for improved facilities including bleachers, a parking area, and access from Ella Grasso Boulevard (Route 10).

This area is above elevation 5.0 NGVD and is well suited to recreational improvements. The limit of DEP's regulatory jurisdiction under the Office of Long Island Sound Programs (OLISP), defined as the annual high tide line predicted in the absence of the tide gates, was determined by MMI as part of the modeling effort (see Chapter 5). The grading involved with the currently proposed soccer fields improvements are above that regulatory line and well above any normal water surface elevations expected as part of a restoration effort. The modifications, therefore, would not affect water surface elevations, and so they were not directly evaluated with the model.

3.2.2 Playground Improvements

An existing dilapidated playground is located near the Derby Avenue (Route 34)/Ella Grasso Boulevard (Route 10) intersection of the park. The playground will be replaced with a new component playground system on an approximately 50-foot diameter rubberized asphalt safety surface. This playground will provide improved recreational opportunities for children at a local school and in the neighborhood. Little to no excavation is expected. Some minor grading may be required to provide a surface grade of less than two percent. Vegetation at this site consists largely of maintained lawn and gravel driveway, with staggered hardwood trees. This work does not appear to be subject to regulatory review, although it is not clear whether local site plan or wetland review is required.

Section 4

4.0 BACKGROUND HYDROLOGY AND HYDRAULICS

4.1 Overview

The water levels in New Haven Harbor and the West River estuary are influenced primarily by the tides in Long Island Sound, and to a lesser degree by freshwater runoff. Long Island Sound has a semi-diurnal cycle with two high tides and two low tides in each lunar day of 24 hours and 25 minutes. The tides are caused by the gravitational pull of the moon and sun. Tidal magnitude is based upon the relative position of the moon and sun, coupled with barometric pressure and wind.

The tides have a critical influence upon coastal areas because they affect navigation, flooding, aquatic habitat, sedimentation, wetlands, and water quality. The plant and animal species in coastal waters must therefore be able to tolerate variable water depths, currents, and salinity. The lower portion of the West River receives saltwater inflow during high tide periods and drain during low tide. The river segment above Orange Avenue is connected to Long Island Sound via a unidirectional tide gate.

The U. S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA) operates a network of 50 tide gauging reference stations along the east coast of North and South America. These stations are used to measure and predict tide levels at the designated stations and at intermittent subordinate stations. The NOAA reference station closest to the West River area is located in Bridgeport, Connecticut. The closest subordinate station is located at the entrance to New Haven Harbor in New Haven, Connecticut.

NOAA provides interpolation factors that enable prediction of tides at the entrance to New Haven Harbor, based upon conditions at the Bridgeport station. High and low water at New Haven Harbor occur nine and 14 minutes before corresponding events in Bridgeport. The predicted high and low water at New Haven Harbor is 0.92 times the value in Bridgeport. The mean tide range at New Haven Harbor is predicted by NOAA to be 6.2 feet, with a spring range of 7.1 feet.

Mean sea level is the average elevation of the sea at a particular location, based upon hourly measurements observed over a specific 19-year cycle called the National Tidal Datum Epoch. It is recomputed every 19 years to reflect long-term changes in sea level. The mean tide level is the midpoint between mean high water and mean low water. It is usually on the order of two tenths of a foot higher than mean sea level.

Sea levels vary on a seasonal as well as a long-term basis. Mean sea level, mean high water, and mean low water all tend to rise during the summer and early autumn and decline during the winter. This is attributed to water temperatures changing the volume of seawater and the inclination of the earth relative to the sun.

It is important to understand that mean sea level is not equal to 0.0 on the National Geodetic Vertical Datum (NGVD) of 1929 since NGVD is based on an average of sea levels measured in coastal areas around the country. Therefore, local areas may be above or below the average sea level computed for the country. In addition, sea level rise and local land subsidence or tectonic uplift has altered the local sea levels since 1929. The mean sea level and other tide elevations at any specific point along the coast vary depending on the location and are predicted based upon data from the tide gauges.

A spring tide occurs at the new and full moons, when the gravitational force of the moon and sun act together. The spring tide range is typically from 0.5 to 1.8 feet higher than the mean tide range. The highest spring tides occur near the Spring and Autumn equinoxes.

Table 4.1-1: Tidal Term Definitions

Term	*Description*
Statistical Storm	SS - The level reached during the maximum storm surge expected in a specified number of years, for example 5-year, 25-year, or 100-year.
Yearly Storm	YS - The 19-year average level of the maximum storm surge in each one-year period.
Mean High Water, Spring	MHWS - The 19-year average height of high water occurring on spring tides.
Mean Higher High Water	MHHW - The 19-year average of higher high tides.
Higher High Water	HHW - The higher of the two high tides in a lunar day.
Mean High Water	MHW - The 19-year average height of high water.
Mean Sea Level	MSL - The 19-year average of hourly water height.
Mean Tide Line	MTL - The plane midway between MHW and MLW (usually within a few hundredths of a foot of MSL).
Mean Low Water	MLW - The 19-year average height of low water.
Lower Low Water	LLW - The lower of the two low tides in a lunar day.
Mean Lower Low Water	MLLW - The 19-year average of lower low tides.
Mean Low Water, Spring	MLWS - The 19-year average height of low water occurring on spring tides.

The two high tides and low tides in each lunar day usually do not have equal magnitudes. Thus, there is a higher high tide and a lower high tide in each lunar day as well as a higher low tide and lower low tide. The difference is generally less than one foot.

The tides in Long Island Sound and its tributary rivers do not occur simultaneously. The rising tide is a progressive wave that moves westward up Long Island Sound from Stonington to Greenwich. It reaches its peak in Greenwich over two hours after its peak in Stonington. Similarly, it takes time for the high tide to move inland along tidal rivers. Thus, the tide cycle of a river usually lags behind that of its inlet. The lag depends upon length, depth, width, and roughness of the particular channel.

Tidal rivers and coastal areas are subject not only to variations in the tide but also to change in the level of the sea itself. The mean sea level in Long Island Sound is estimated to be rising at an average rate of 0.1 inch per year, a rate that is increasing attributed to global warming. This rising trend has been recorded at gauge stations around the world.

4.2 New Haven Harbor Tides

4.2.1 Flood Tides

Coastal salt marshes are subject to abnormally high flood tides in addition to the daily ebb and flow of the sea. In Connecticut, most flood tides are caused by near shore maritime storms including hurricanes, tropical storms, and nor'easters. These events create a storm surge that temporarily raises the sea level due to a combination of low barometric pressure and on-shore winds.

Long Island Sound is particularly sensitive to storms that move northwards up the eastern seaboard and across Long Island. With counterclockwise circular winds, the leading edge of northwards moving coastal storms produce easterly winds that force water into Long Island Sound creating storm surge. The storm surge of a 100-year frequency event is estimated to be six to eight feet above the ambient stillwater level, depending upon location. The magnitude of the storm tide is influenced by its timing relative to the normal astronomical tide. The worst combination is when the storm surge occurs simultaneously with a normal high tide or spring tide.

The greatest tide levels in this century were produced by the September 1938 hurricane and Hurricane Carol on August 31, 1954. The 1938 hurricane occurred several hours prior to high tide with a seven-foot surge above the predicted tides. The resulting water level at New Haven was recorded at elevation 9.8, with 15-foot waves. Hurricane Carol in 1954 coincided with high tide, producing a stillwater level at elevation 9.7 and a storm surge of five to seven feet. Plots of the hurricane tide cycles at Bridgeport show that the 1938 hurricane was preceded by a normal low tide, while the 1954 event had a irregular curve due to its off peak timing.

The U.S. Army Corps of Engineers has used historic tide elevation data and computer models to predict the elevation of rare flood tides on Long Island Sound, corrected for sea level rise. The following data is from the publication "Tidal Flood Profiles New England Coastline" published in 1988.

Table 4.2-1: ACOE New Haven Harbor Flood Tide Elevations

Tide Frequency	*Stillwater Elevation, NGVD*
100 Year	10.6
50 Year	9.9
10 Year	8.6
1 Year	5.4

4.2.2 Non-Storm Tides

The NOAA mean tide level at the entrance to New Haven Harbor, after conversion to National Geodetic Vertical Datum, is at elevation 0.58 feet NGVD. Based upon the NOAA mean tide range of 6.2 feet, the mean high water (MHW) would fall at elevation 3.68 feet NGVD and the mean low water (MLW) would fall at elevation -2.52 feet NGVD.

The U.S. Army Corps of Engineers has also published mean tide data for Long Island Sound based upon the 1978 Tidal Epoch. This reference date is used because mean sea level varies throughout the world. The mean tide level at the entrance to New Haven Harbor, based upon the Army Corps of Engineers 1978 Tidal Flood Survey, is at elevation 0.7 feet NGVD. MHW is at elevation 3.8 feet NGVD and MLW is at elevation -2.3 feet NGVD.

The following table compares NOAA and ACOE tidal data:

Table 4.2-2: Published New Haven Harbor Tide Data (USACOE, NOAA) (NGVD 1929)

Tidal Condition	*Army Corps of Engineers*	*NOAA*
Mean Spring High Water	4.2	4.2
Mean High Water	3.8	3.68
Mean Tide	0.6	0.58
Mean Low Water	-2.5	-2.52
Tidal Range (Feet)	6.3	6.20

Spring tides represent the highest tides that occur on a frequent basis and generally represent the upper potential elevation limit of salt marsh vegetation. The predicted mean spring tide for Long Island Sound has a range of 7.1 feet with corresponding high water at elevation 4.2 feet NGVD. This compares well with field-measured data during spring tides.

4.2.3 Gauging

In addition to the use of published predicted tide data, MMI has previously recorded tide levels around New Haven as part of previous studies of Morris Creek, New Haven, and Old Field Creek, West Haven. The data found that the open harbor tides at Lighthouse Point Park are very similar to Morris Creek, below its tide gate.

Tide data has been previously recorded in New Haven Harbor that provides local tide information. The mean low water, mean tide level, and mean high water was determined at three points using 30-day recording tide gauges.

During the period June 17 through July 19, 1994, Milone & MacBroom, Inc. and Ocean Surveys, Inc. performed a tide data acquisition program around the harbor, including New

Haven, West Haven, and East Haven, Connecticut. MMI designed the program and performed the benchmark surveys. OSI provided eight Sea Data TDR-3A in-situ recording tide gauges and one Stevens Type A water level recorder.

Continuous water level measurements were acquired at three stations in the Cove River, at three stations in Old Field Creek, at two stations in Morris Creek, and at the mouth of New Haven Harbor at Lighthouse Point. The Sea Data in site recording tide gauges were installed at Lighthouse Point and are described hereafter.

The Sea Data TDR-3A is a subsurface pressure-sensing unit which measures the hydrostatic head above the sensor. The Sea Data TDR-3A tide gauges were set to record tidal height at 15-minute intervals. Tidal bench marks (TBMs) were established above each tide-monitoring site. The TBMs were marked with a nail or drill hole and survey paint. Milone & MacBroom, Inc. conducted a level survey and provided the elevation data for each of the TBMs.

During data processing, the tidal elevations were computed by OSI from the recorded water level measurements; corrected for instrument calibration, water temperature, and barometric pressure; and referenced to the NGVD datum.

The tide data collected in New Haven Harbor at Lighthouse Point exhibited unequal diurnal tides, which compare well to the NOAA predicted tides. The maximum water level recorded was 5.64 feet NGVD datum at 01:00 hours June 25, 1994 during a storm. The minimum water level recorded was -3.36 feet NGVD 05:45 hours June 23, 1994. The mean sea level for this data set was 0.903 feet. This data are summarized in the following table:

Table 4.2-3: Lighthouse Point Tide Data June 18-July 19, 1994 (NGVD 1929)

	Lighthouse Point
Highest tide	5.636
Mean high water	4.078
Mean sea level	0.903
Mean low water	-2.278
Lowest tide	-3.360
Mean tide range	6.365

Data on spring and storm tides have been manually recorded and assembled to determine the DEP regulatory boundary elevation upstream of the tide gate.

4.3 West River Tide Gates

4.3.1 Description

Tidal flow in the upper West River is regulated by a tide gate structure located approximately 1.1 miles upstream of Long Island Sound and 100 feet downstream of Orange Avenue.

The tide gate structures was constructed in 1919 by the Connecticut Agricultural Station to help control mosquitoes by limiting the flow and ponding of salt water on the tidal marshes. It was part of a larger program that included the excavation of small "mosquito ditches" to drain the marsh surface all along the Connecticut shoreline. Similar tide gates in New Haven are located on the Mill River and Morris Creek.

Many of the tide gates and salt marsh drainage systems in Connecticut are a result of an intensive mosquito control program which began in 1912 and subsequent Public Acts of 1915 (Chapter 264) and 1917 (Chapter 402).

The existing tide gate structure includes concrete abutments at each riverbank, mid-channel concrete piers, and a concrete deck that spans the river, supported by the abutments and piers. The downstream side of the structure has a vertical concrete bulkhead extending from the channel bed to the concrete deck.

There are 12 rectangular openings covered by large timber flap gates that are top hinged to the bulkhead above the openings. The flap gates are normally suspended in a vertical position due to their weight. Each tide gate is 7.6 feet high by approximately 5.0 feet wide. The total orifice area is 456 square feet. The channel invert at this location is approximately minus 2.2 feet NGVD. The top of bulkhead elevation is 9.0 NGVD.

The existing tide gate is owned and maintained by the City of New Haven. It is generally in good condition but shows some wear and deterioration. The concrete abutments and piers have shallow concrete wear and spalling on the exposed surfaces near and below the water line. The concrete deck and steel handrails on top of the structure are in good condition. One timber flap gate has been replaced with sheet metal.

At high tide, the top of the flap gates and hinges are slightly above water. At low tide, most of the gates are exposed. This high tailwater condition even at low tide reduces the downstream flood flow capacity.

Inspection of the flap gates reveals that there are small gaps around the sides and tops of most of the flap gates. Thus, they do not have a tight seal and salt-water leaks upstream at high tide. The tide gate leakage contributes to the water surface rise measured in the upstream channel at high tide and to its salinity.

4.3.2 Hydraulics

The magnitude of frequency of occurrence of the riverine flood and tidal flood for which tide gates are designed is usually based upon the extent of potential damages. The size and number of individual tide gates within a structure is based upon the required hydraulic capacity. It is common practice to design flood control structures to have adequate capacity for flows with a recurrence interval of 100 years. The tide gates must be able to discharge riverine flood flows downstream against the normal tides without causing excessive upstream water levels. During storm tides, this is not always possible.

The top elevation of the tide gate bulkhead or embankment must be high enough to block the upstream passage of tidal floods of the selected size and frequency. In Connecticut, this generally requires a crest at elevation 10 to 12 feet NGVD. The elevation at the bottom of the tide gate establishes the lowest level to which the river can drain at low tide. If it is above the streambed, the channel will retain water at low tide and may tend to accumulate sediment.

A hydraulic analysis of tide gate structures is necessary to determine the required capacity for passing low flows without creating excessive water elevations. The instantaneous flow rate through tide gates can be determined with the orifice formula and an appropriate discharge coefficient. However, the overall capacity and resulting water elevations vary with the tides, runoff hydrographs, storage capacity of the upstream marsh, and time.

In the initial analysis, the gates are treated as rectangular orifices with a discharge rate determined by the following equation:

$Q=CA(2GH)^{0.5}$, where

"C" is a coefficient of the theoretical discharge;

"A" is the area of the orifice;

"G" is the gravity constant; and

"H" is the head.

The maximum discharge rates occur at low tide and with high upstream levels. The above equation can be rewritten for the West River tide gates, with appropriate values as follows:

$$Q = (0.6)(38)(64.4H)^{0.5} = 182 \text{ cfs per gate, at } H = 1 \text{ ft}$$

During normal periods with a low head of say one foot, the total discharge rate is 2,195 cubic feet per second (cfs). During periods with a high head differential, such as at four feet, the discharge rate is 365 cfs per gate, with a total potential of 4,390 cfs for all 12 gates open. Kenny measured flows of 300 cfs with one gate open and almost 600 cfs with two gates open, close to the theoretical flow rates. The actual discharge will be influenced by debris, sediment, and gate performances.

4.3.3 Leakage

The existing tide gate has loose fitting timber flaps that fit over orifices in the bulkhead. As with many similar tide gates along the Connecticut coast, there is appreciable leakage of salt water moving upstream through the various joints during high tide.

The leakage is insufficient to cause upstream floods, but it does create a limited upstream tide range within the channel. This fluctuation supports limited tidal marsh between the tide gate and Derby Avenue. The amount of leakage varies depending on the following: the presence of sediment or debris, which can hold the gates partially open; tide levels; and the condition of the timbers, which are periodically maintained.

The upstream rate of leakage was measured by William Kenny in 1993 using a flow meter. The results revealed leakage rates of 30 cfs during the high tide period.

4.4 West River Tide Gauging

William Kenny and Paul Barten, then of the Yale School of Forestry & Environmental Studies, collected water levels during a 1992 study of the West River Memorial Park. (See the discussion in the Data Gathering section of Chapter 2 for more information regarding the Kenny and Barten study.) The following table summarizes the tidal data collected as part of that study:

Table 4.4-1: West River Tide Data June – October 1992

	Water Elevations		
Condition	***MHW***	***MLW***	***Range, Ft.***
Gates Closed	0.45	-0.75	1.20
One Gate Open	2.00	-0.8	2.80
Two Gates Open	2.20	-0.9	3.1

The Kenny and Barten tide data for the period June 27 to October 7, 1992 is the best available information on the marsh. With the gates closed, marsh high tide levels ranged from –0.5 to 2.0±, depending on rainfall and runoff, with a mean of +0.45 NGVD. The experiment with one tide gate open resulted in high tides of 1.5 to 2.5 NGVD, except in dry weather. Water levels with two gates open were only slightly higher but were open primarily in dry weather.

The average tidal lag downstream of the gate was 19 minutes after spring high tides in the harbor, indicating limited restriction over the 1.1 miles. Kenny found the tide range on the downstream side of the gate to be only 0.6 feet less than the harbor predictions, with an average spring high tide at elevation 5.1 NGVD.

Milone & MacBroom Inc. conducted additional gauging at the West River tide gates to develop a relationship between tides at the gate versus tide levels in the harbor. Using data obtained by MMI at the tide gate, and corresponding high tides at the United Illuminating harbor gauge, MMI computed a regression relation between the two. This allowed the use of long-term data from the harbor to be used in forecasting tide levels at the downstream side of the West River tide gate. The data was highly correlated, with an R^2 of 0.9927. The relation is noted below:

Tide gate WSEL = 0.9564 (harbor WSEL) + 0.5713, where
"WSEL" is Water Surface Elevation

The following table presents the conversion between the tidal data determined for the New Haven Harbor to tide levels at the West River tide gates:

Table 4.4-2: Tide Data Conversion from New Haven Harbor to Tide Gates

	Harbor Tide Data (NGVD)	***At Tide Gate (NGVD)***
Mean annual high tide	5.4	5.7
Mean spring high water	4.2	4.6
Mean high water	3.8	4.2
Mean tide level	0.6	1.1
Mean low water	-2.5	-1.8

4.5 West River Freshwater Hydrology

The amount of freshwater inflow to a tidal marsh has a direct impact upon its water levels and salinity. The inflow is a function of the watershed's area, precipitation, land use, soil cover, slope, length, and storage characteristics.

The West River watershed has a drainage area of 35 square miles, including portions of New Haven, West Haven, Woodbridge, Hamden, Bethany, and Prospect. The watershed is about 13 miles long by up to four miles wide, with moderate to steep slopes. Much of the upper watershed is rural land that includes public water supply watersheds, draining to five reservoirs used by the South Central Connecticut Regional Water Authority. The lower watershed includes suburban and urban areas with high impervious cover.

The peak rates of flood runoff in the West River watershed have been previously determined as part of other studies by the City, Soil Conservation Service (SCS), and FEMA. The SCS flood control study is based upon the use of the TR-20 "Computer Program for Project Formulation Hydrology." The results are tabulated below from both the SCS. The City study, entitled "West River Flood Control Study," was prepared by Flaherty-Giavara Associates (FGA) in 1978.

Table 4.5-1: West River Peak Rates of Runoff (City, SCS)

	Peak Rate of Runoff (cfs)	
Design Point Location	***Reference 1 City of New Haven***	***Reference 2 Soil Conservation Service***
Whalley Avenue	7,800	7,662
Blake Street	4,350	4,389
Valley Street	4,200	4,342
East Ramsdell Street	3,580	3,518
Pond Lily		3,003

Much of the upper watershed is owned by the Regional Water Authority (RWA). The flow rates used presume that the large watershed to the Regional Water Authority reservoirs remains in a rural condition. The peak rate of flow can be expected to increase if the RWA lands are developed intensely (FGA 1976). The West River watershed experienced severe flooding in June 1982, with extensive damage between Whalley Avenue and Ramsdell Street, including the loss of the Blake Street bridge.

The FEMA Flood Insurance Study of New Haven (1980) provides data on West River's peak flow rates that are used for regulatory purposes, as noted in the following table:

Table 4.5-2: West River Regulatory (FEMA) Peak Flow Rates

Frequency, Years	*Peak Discharge, CFS*
10	2,750
50	4,000
100	4,800
500	6,300

The rate and volume of runoff in this watershed are influenced by water supply reservoirs and withdrawals. Water storage reservoirs include Lakes Dawson, Watrous, Bethany and Chamberlain, plus Glen Lake. The Regional Water Authority has a treatment plant on Lake Dawson and withdraws water for potable use. This has negligible impact on peak flows but may influence low flows in dry weather.

The southeastern part of the watershed is an urban area drained by Wintergreen, Farm, and Wilmont Brooks. Farm Brook has a SCS PL-566 flood control detention area.

Section 5

5.0 HYDRODYNAMIC COMPUTER MODELING

The West River channel, tide gate, and marshes were analyzed with a hydrodynamic computer model to assess hydraulic conditions under existing conditions and various proposed conditions. The model used for analysis is described in this chapter.

5.1 Theory

The West River was studied with the Surface Water Modeling System (SMS), which is a graphical user interface for a number of numerical models. The RMA2 numerical model was primarily used for this study.

RMA2 is a modeling system used for studying two-dimensional hydraulics, sedimentation, and transport in rivers, reservoirs, bays, and estuaries. Existing and proposed geometry can be analyzed to determine the impact of various project designs on velocity and flow patterns in a water body. The model is also capable of simulating structures and islands. It has been applied by the Army Corps and others in many tidal applications including the Norwalk Mill Pond, Columbia River, Cape Fear River, Atachafalaya Bay, Norfolk Harbor, Chesapeake Bay, New York Harbor, Kings Bay, Terrebonne Marshes, Charleston Harbor, Portsmouth Harbor, and Corpus Christi Bay, to name a few.

RMA2 is entitled "Two-Dimensional Model of Open Channel Flows." It was first applied by the U.S. Army Corps of Engineers (ACOE) in 1973. RMA2 is a finite element solution of the Reynolds form of the Navior-Stokes equations for turbulent flows. Friction is calculated with Manning's equation, and eddy viscosity coefficients are used to define turbulence characteristics. A velocity form of the basic equation is used, with side boundaries treated as parallel or static (zero flow). The model automatically recognizes dry elements and corrects the mesh accordingly. Boundary conditions may be water-surface elevations, velocities, or discharges, and may occur inside the mesh as well as along the edges.

RMA2 computes water-surface elevations and horizontal velocity components for subcritical, free surface flow in two-dimensional fields. The model is capable of analyzing both steady state and dynamic, time dependent problems. The program has been applied to calculate flow distribution around islands, through and around structures (bridges, culverts, etc.), flow at river junctions, flows over floodplains, tidal marsh exchange, and general flow patterns in rivers, reservoirs, and estuaries.

The model operates using separate geometry and boundary condition files. The geometry input file defines a system of elements and nodes, which are defined with respect to their elevation and spatial location in the water body (x-, y-, and z- coordinates). Material types, such as water, wetland, jetties, outfall structures, and the like, are also defined using the system of elements and nodes.

The boundary condition file is used to set hydraulic head and flow conditions at inlet and outlet structures. It is also used to assign Manning's roughness coefficients and viscosity coefficients by material type. Model convergence parameters are specified in the boundary condition file as well.

The two-dimensional hydrodynamic model has been extensively tested and revised to replicate existing topography and hydrography. However, it does have several minor limitations due to the inherit nature of the finite element mathematical equations. The model reaches successful solutions for high runoff and high tides, low runoff and high tides, and some low tide conditions. It does not always reach a solution for low flows at low tide, and very high runoff flows at low tide.

In general, the following limitations must be considered when using hydrodynamic models:

1. All hydrodynamic models must have a continuous mesh at all time steps. This means that the model must not have depressions that would become isolated at low tide. In order to avoid fatal errors and nonconvergence, minor changes in topographic elevations have been made to prevent "puddles" from forming. Even still, the model has difficulty in representing low tide conditions when most of the marsh is dry.

2. The hydrodynamic model is based upon the conservation of energy and momentum equations. These equations are invalid for some high velocity supercritical flow conditions. The mathematical model occasionally fails to reach a solution at a combination of high runoff and low tides due to supercritical flow.

The above limitations prevent the model from reaching solutions for some tide or flow rates.

5.2 Input

RMA2 was applied to the West River from Whalley Avenue to the tide gates to evaluate water levels, velocities, and flushing under existing and proposed conditions. The following discussion covers the types of data used to create the models for evaluation.

5.2.1 Mesh

The foundation of the model is a network of grid cells, or mesh, developed to reflect the topographic features of the study area. Grid cells are defined to represent changes in bathymetry/topography, slope, and form of the marsh area and principal channels.

Elevation data and material types are assigned to the mesh cells, as described in the following text.

The following table describes the mesh created for the existing conditions model. The meshes used in the proposed conditions models vary slightly from the one presented below, due to the refinements required to model proposed conditions.

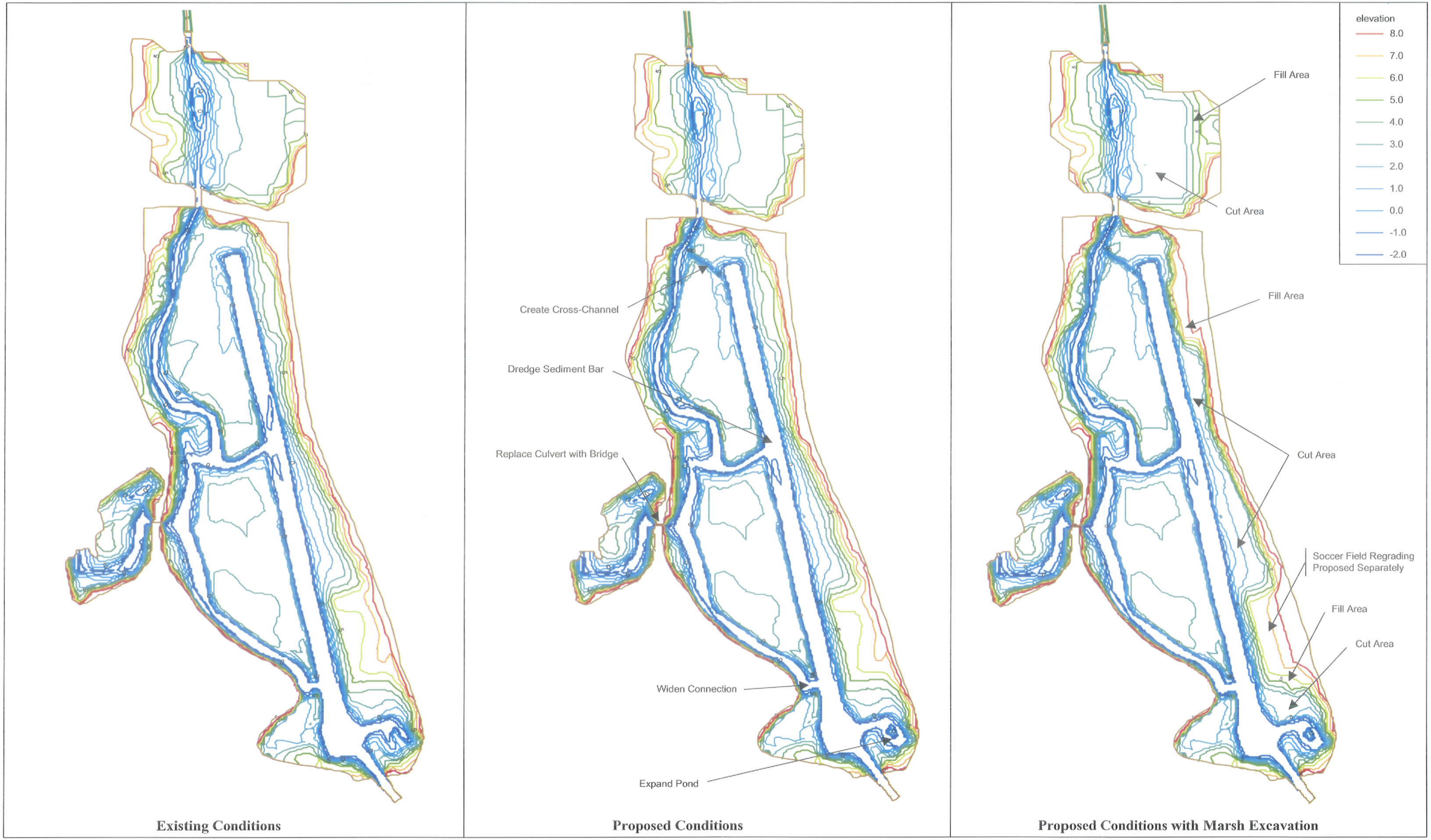
Table 5.2-1: Properties of Existing Conditions Mesh

Total Number of Elements	4,069
Number of Triangular Elements	1,512
Number of Quadrilateral Elements	2,544
Number of Linear Elements	12
Number of Nodes	10,991
Minimum Elevation (ft NGVD)	-8.45
Maximum Elevation (ft NGVD)	16.70
Front Width	404
Number of Boundary Controls	3

5.2.2 Topography

Topographic data assigned to the existing conditions mesh was based primarily on field surveys by Bill Kenny and Paul Barten in 1992 (Kenny and Barten 1992) and by MMI in September 2001. The field surveys were supplemented with City aerial mapping (at a scale of 1 inch to 40 feet and with one-foot contours) and field inspections. The data were used to create a digital elevation model (DEM) in AutoCAD. A dense network of points interpolated from the DEM was exported to the SMS model. SMS interpolated elevations from those points for eight locations on each grid cell. The geometry for the proposed conditions models was based on the existing conditions geometry, with modifications as needed. A detailed description of the proposed modifications is included in the following chapter. The following figures show the mesh geometry used in the existing and proposed conditions models.

Figure 5.2-1: West River Memorial Park SMS Hydraulic Model - Mesh Geometry



Existing Conditions

Proposed Conditions

Proposed Conditions with Marsh Excavation

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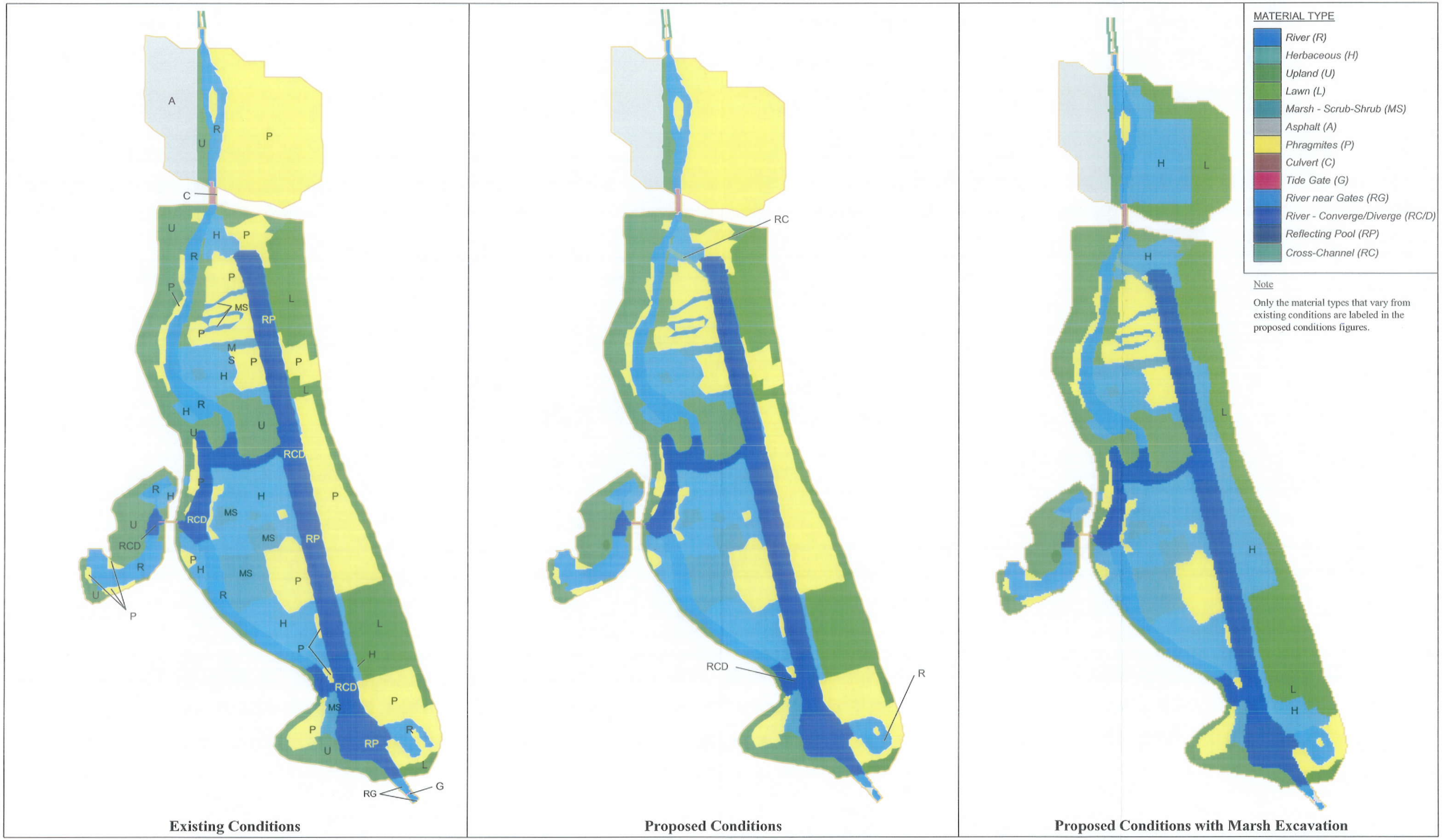
5.2.3 Material Types

RMA2 uses material types to enable variation in hydraulic properties of different land covers. For example, water flow through a clean, straight river channel will encounter much less resistance than water flow across a vegetated field. That resistance to flow can be expressed in the friction coefficient and eddy viscosity, where lower values tend to reflect less resistance. Typically, the friction coefficient is interpreted as physical resistance to flow, while the eddy viscosity can be interpreted as the amount of energy loss to turbulence.

A material type was assigned to each grid cell in the mesh, based on a digitized version of the vegetation features map created by Orson, Brown, and Sharp (see the discussion on Vegetation in Chapter 2). That map was imported into SMS. Material properties were assigned to mesh cells according to the vegetation community represented on the digitized map. As with the topography information, the material types of the proposed conditions models were based on the existing conditions model with modification as required. The following figures show the existing and proposed conditions meshes with the assigned material types.

Figure 5.2-2: West River Memorial Park SMS Hydraulic Model - Mesh Material Types



Existing Conditions

Proposed Conditions

Proposed Conditions with Marsh Excavation

Note

Only the material types that vary from existing conditions are labeled in the proposed conditions figures.

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References provide by SMS were consulted in determining typical values for the friction coefficient and eddy viscosity for each material type. The following table presents the values for all the material types used:

Table 5.2-2: Material Types Used in Existing and Proposed Conditions Models

Material Type	*Friction Coefficient*	*Eddy Viscosity*
River	0.03	80
Reflecting Pool	0.015	80
River - Converge/Diverge	0.025	80
River Near Gates	0.015	80
River 1-D	0.025	80
Culvert	0.018	80
Cross-Channel[1]	0.04	100
Tide Gates	varies[2]	varies[2]
Phragmites	0.04	100
Herbaceous	0.07	100
Shrub marsh	0.08	100
Upland	0.065	100
Lawn	0.04	100
Asphalt	0.02	100

Notes:
[1] Applies to proposed conditions models only
[2] See following discussion

Friction and viscosity values for the tide gates vary according the whether the majority of the gate flaps are open or closed. Under incoming tide conditions, most of the flaps are closed. The only openings allowing water through the gate structure are those that are forced open and so, are open completely providing high but normal resistance to flow. Under outgoing tide conditions, however, most of the flaps are partially opened by the water elevation differential. Under that scenario, the water must force its way past the tide gate flaps, creating conditions of extremely resistant flow and requiring correspondingly higher friction coefficient values. The values for the tide gates were determined in the calibration process (calibration discussion is provided in the following section). The tide gates friction and viscosity values used in the different models are presented in the following table. Note that the friction coefficient varies by a factor of ten between the incoming and outgoing tide scenarios.

Table 5.2-3: Material Values for Tide Gates

ModelConditions	*Friction Coefficient*	*Eddy Viscosity*
1 Open Tide Gate Flap: Incoming Tide	0.06	120
1 Open Tide Gate Flap: Outgoing Tide	0.6	200
2 Open Tide Gate Flaps: Incoming Tide	0.065	120
2 Open Tide Gate Flaps: Outgoing Tide	0.45	200

5.2.4 Boundary Conditions

The boundary conditions are set at locations where water flows either into or out of the model. The modeler defines an open boundary as either a known water surface elevation or a known flow rate, either of which may vary over time. The open boundaries to the West River model include the following: inflow of fresh water from the West River; inflow of fresh water at the Horseshoe Lagoon; and in/outflow at the tide gates. The east and west sides of the system were assumed to be closed boundaries that exclude cross flow.

The fresh water inflows to the model were defined as constant flow rates. It was presumed that any open tide gate flaps would be closed under storm conditions. As a result, the model was run only for non-storm flows. The inflow flow rates were determined using estimates based on watershed size. The estimated flow rates were compared with the known flow rates presented in Chapter 3 and were found reasonable.

The tidal boundary conditions were defined in the calibration models using the water levels downstream of the tide gates collected by Kenny and Barten during spring tides (Kenny and Barten 1992). The tidal boundary conditions in the proposed conditions models under spring tide conditions were defined as the average tidal cycle of the six spring tide cycles measured by Kenny and Barten.

The tidal boundary condition under the annual flood was determined using the following procedure:

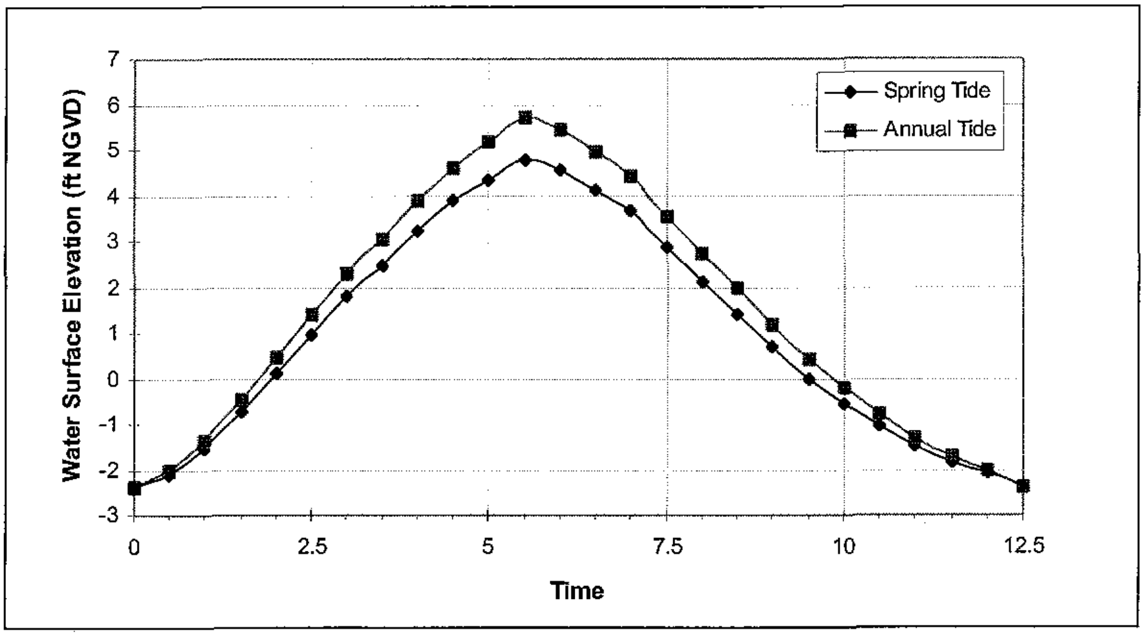
1. The peak of the annual tide for the New Haven Harbor was determined to be 5.4 feet NGVD, from the publication by the U.S. Army Corps of Engineers entitled "Tidal Flood Profiles - New England Coastline," published in 1988.

2. The annual tide peak downstream of the West River tide gates was determined to be 5.7 feet NGVD, using the conversion presented in Section 4.4.

3. The average spring tide cycle measured by Kenny and Barten was converted to an annual tide. The low tide elevation for the annual tide was set equal to the low spring tide elevation. The high tide was set to the elevation determined in #2. The remainder of the points were adjusted to keep the same relationship between points in the cycle.

The following figure shows the spring tide and annual tide boundary conditions:

Figure 5.2-3: Tidal Boundary Conditions



5.2.5 Model Runs

A steady state (static) model with constant flow rates was initially prepared to test the network shape and hydraulic convergence. Dynamic models with varying water surface elevations at the tide gates were then developed and used to evaluate tidal flow conditions. The dynamic models used for evaluation of tidal marsh restoration alternatives were run through a full tidal cycle, beginning and ending at low tide, to "spin up" or set the initial conditions for a model run. As the tidal conditions changed during the spin-up process, the grid cells representing tide gates were enabled and disabled, as needed, to represent the opening and closing of tide gates. Correspondingly, the material properties of the tide gates were also changed as the modeled tide gates opened and closed (see discussion in previous section).