Modeling sediment transport in the York River: final report

Jerome P.-Y. Maa

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Final Report

Modeling Sediment Transport In the York River

Prepared for

Chesapeake Bay Program
Environmental Protection Agency
Desired Outcome ID #: RA-4,
Understanding Nutrient and Sediment Delivery Process below the Fall line

by

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Summary

Numerical simulation on the formation of an Estuarine Turbidity Maximum (ETM) is a difficult task because of the incomplete knowledge on cohesive sediment transport. For example, it is difficult to well simulate the erosion process when the bed erosion properties are only poorly known. It is also hard to simulate the settling process when the bulk density and size of suspended sediment flocs are changing with the ambient environments.

Despite of the difficulties, some processes can be parametricalized, even not perfectly, to make the simulation results close to observed. In this project, we demonstrated the possibility of using a constant erosion rate and the assumption that erosion only occurs at tidal acceleration phases to simulate the formation of turbidity maximums in the York River.

Eight slack water surveys were carried out first during 2001 and 2002 to develop a background truth. Measurements at about 25 stations along the York River System, which is about 120 km long, were conducted to collect salinity and Total Suspended Solid (TSS) profiles. This information is used to developed “snapshots” for calibrating and comparing with the results from a numerical simulation using HEM-3D and the new erosion model.

Measurements of the settling velocity using a currently available technique, i.e., the “Owen Tube method” with tape water and sediment samples collected from the York River were also carried out. Because the Owen Tube is not a commercial product, we have exhausted all the available resource in order to successfully manufacture it. For this reason, we only have enough resource to find the lower limit of settling velocity for the York River System. We have to leave the effects of other two factors (turbulence and salinity) unanswered from our field work. It is expected that salinity will increase the settling velocity. This statement was confirmed through the numerical experiments carried out in this study. With the erosion and settling velocity limited to a range obtained from the field experimental results, the ETM and an occasionally observed secondary turbidity maximum were all satisfactory simulated.

With the current available measurement technique and the data available, it is not
recommended to include a sub-module for simulating cohesive sediment flocculation process. The huge computing time required for adding this sub-module also prohibits this development. Most important, the possible benefit of including the flocculation process, and thus, a second or more size class for cohesive sediment would not be significant, and thus, not justified.

Assuming a rough bottom boundary for granular sediment bed and converting the near-bed suspended sediment concentration for granular sediment to erosion rate, the computer codes developed for cohesive sediment transport can also be used to simulate the suspended sediment transport for non-cohesive sediments. This is another development achieved in this project.

For numerical simulation, it is important to have an accurate bathymetric grid to work with. For this reason, extra effort has been devoted to produce a new curve-linear, orthogonal grid with high bathymetric resolution. Although there is a low resolution bathymetric grid available for use, it is worth to document that the performance of this new high resolution grid is well worth the effort.

This study produces a satisfactory numerical model for suspended sediment transport which address the questions related to estuarine turbidity maximum(s). More efforts, however, are needed to include consolidation process for cohesive sediments and bed load transport for non-cohesive sediments in order to address questions related to estuarial morphology.
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CHAPTER 1. INTRODUCTION

Suspended sediment is a key factor affecting water quality in the Chesapeake Bay and its tributaries. Indeed, turbidity is the chief culprit in the degradation of estuarine ecologies by damping light penetration, reducing the thickness of the eutrophication zone, and in turn, limiting primary productivity. Extreme turbidity threatens fish habitats by clogging gill structures and interfering with respiration. Suspended sediment that settles out of high freshwater runoff may bury clams and oysters and impact the settling site of planktonic larvae. Furthermore, suspended sediment enriched with contaminants (e.g., heavy metals, insecticides, petroleum by-products, and some radio-nuclides) is a pollutant. Therefore, the ability to predict sediment transport and turbidity maximum (TBM) in estuaries is an important step toward mitigation of water quality problems. Unfortunately, a well established sediment transport model is not existed yet, and thus, under great demand.

During 1996 and 1997, we have conducted a series of slack water surveys along the York River with a project name “Contaminants And Sediment Transport (CAST).” This study revealed the general pictures of salinity and sediment distributions in the York River and found two possible TBMs. As expected, the primary TBM was found near the end of salinity intrusion, upstream of the York River. A secondary TBM, however, was also measured sometimes in the middle of the York River. Using a three-dimensional numerical model (HEM-3D), Lin and Kuo (2001) has revealed a potential mechanism of formation for the secondary TBM.

HEM-3D is a fully verified hydrodynamic and eutrophication model with limited capabilities in sediment transport modeling (Park et al., 1995). HEM-3D has been used in the York River (Sisson et al., 1997, Shen et al., 1997) and the James River (Shen and Kuo, 1999). The proof of a well developed hydrodynamic model warrants the establishment of a comprehensive sediment transport module that can meet or exceed EPA’s specifications for sediment transport studies in the Chesapeake Bay and its tributaries (1999 Chesapeake Bay...
The York River System (Fig. 1-1) was selected to implement a complete sediment transport model because of the availability of a computation bathymetry grid, a well documented characteristics of hydrodynamics and salinity intrusion, a basic understanding of the suspended sediment distribution, and *in-situ* measurements of erosion rates (Maa and Kim, 2003). Likewise, studies on sediment accumulation rates, sediment composition, and bed shear stress measurements have been also conducted (Kim et al., 2000). Indeed, only marginal field work is required to complete the data sets, greatly facilitating the development of a complete 3-D sediment transport model. Building upon investments in HEM-3D will therefore produce more fruitful results at greater cost effectiveness than starting a new.

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**Fig. 1-1.** A General Map of the York River System, a Subestuary of the Chesapeake Bay.
The available CAST data (Conductivity and Temperature profiles with Total Suspended Solid (TSS) data at three elevations, from June 1996 to June 1997) spanned an entire year with a sampling interval of roughly one month. For comparing the model results with these field data, it would require a model to run for about 13 to 14 months in order to have a fair comparison. This is not an unreasonable requirement if a model is already developed and requires validation for long-term simulation. From a model developer’s point of view, however, a short simulation period, e.g., one or two months, with more data in the simulation period (e.g., four sets of data in one month) would be more reasonable because the dynamics of the York River can be closely monitored and simulated.

For the above stated reason, two one-month periods of model simulation were planned to mimic the possible dry (October) and wet (March) seasons. In order to have enough data for calibrating the model, four slack water surveys were planned during each period to measure Conductivity, Temperature and TSS profiles. With an Optical Backscatter Sensor (OBS, Downing, 1983) mounted with the CTD profiler, we can obtain TSS profiles with a much better vertical resolution. Details of the surveys and survey results are given in Chapter 2. All of the in-situ calibration data for OBS and evidences of sediment sources are given in Appendix I. In order to show the complexity of OBS measurement results (i.e., the possible TSS profiles), the raw OBS profiles from the first slack water survey are given in Appendix II.

To carry out a good numerical simulation of sediment transport, one basic requirement is to have enough resolution to address the geometry, e.g., channels and flats. Originally we plan to use the old bathymetric grid generated by a previous study as mentioned in our proposal. When further examining the grid, we found that the resolution is not good enough. The old channel at Clay Bank and the navigation channel near West Point are not shown in the old bathymetry. We foresee that this inaccurate bathymetric grid neither will have good results on the simulation of salinity intrusion, nor good results on sediment transport. For this reason, we have to produce a new bathymetric grid that meet our objectives. Details of the reasons to
develop a new grid and details of the grid generation are given in Chapter 3.

With this new grid, we have to go through the calibration on tidal wave propagation, as well as the salinity intrusion. Although these two are not the ultimate objectives in our study, they have to be right in order to justify a correct simulation of sediment transport. Details of the calibration on tidal wave propagation and salinity intrusion are given in Chapter 4. Both the upstream and downstream boundary conditions for salinity and TSS for the simulation period are also given in this Chapter.

One of the objectives of this study is to simulate cohesive sediment transport in the York River system. The four major processes that describe the dynamics of cohesive sediment transport are (1) erosion, (2) deposition, (3) settling, and (4) consolidation. In our proposal, we suggested using the Owen tube method to measure the settling velocity for cohesive sediments. Results of this effort are given in Chapter 5. Since there are wind measurements at VIMS campus site, this supplemental information for the simulation period is also included in this chapter. Wave erosion on shoreline has been claimed as an important potential source for sediment influx from lateral boundary. For this reason, we proposed to monitor the wave and TSS information at a shallow water site at VIMS’ Ferry Pier. Details of this effort was also given in this chapter.

To integrate the erosion and deposition processes together is another proposed study objective. This is a great simplification of modeling cohesive sediment transport because we are not attempt to study estuary morphology, and thus, consolidation process can be ignored because it is not playing an important role in our study. Since the original HEM-3D already has a module (however, not verified yet) for cohesive sediment transport, we replaced the old codes and implemented the above mentioned integration of erosion and deposition processes for cohesive sediment. Details on the implementation are given in Chapter 6. During this studying period, we found that there is no any data to address the sediment flocculation process. We also found that the cohesive sediment transport module works reasonable well, at least for the
two simulated period in the York River. Thus, there is no need to add another size class for cohesive sediment.

We added another module in the HEM-3D to simulate suspended non-cohesive sediment transport. The effort and results of adding this sub-module are given in chapter 7. The calculated vertical profiles of non-cohesive sediment concentration are reasonable, but there is no data to calibrate the results. Inasmuch of the small area for non-cohesive sediment bed in the York River, the contribution of calculated non-cohesive sediment has little contribution to the measured TTS profiles.

Discussion on several interest phenomena (e.g., residual current under the action of M\textsubscript{2} tide; methods to compare measurements and model simulated results) and problems (e.g., numerical stability when involving sediment transport; sediment availability; interactions between suspended cohesive sediments) are given in Chapter 8 and followed by the conclusions.

All of the input datafile names are listed in Appendix III with a brief description of the purpose of each file, but the contents are not listed to save space. These input datafiles are given in the attached CD under a subdirectory INPUT. The source codes of the two sediment transport modules are also given in Appendix III with sufficient comments to describe the content.
CHAPTER 2. FIELD SURVEYS

We proposed to collect two one-month comprehensive data sets on the salinity, temperature, and total suspended solid (TSS) profiles in the York River system, including tidal fresh waters. The objective of acquiring these data sets is to verify the HEM-3D on both hydrodynamic and the proposed modification on sediment transport modules.

SAMPLING STATIONS

Twenty-five stations along the main channel of the York River and the Pamunkey River

Fig. 2-1. Water Sampling Stations for Slack Water Surveys along the York River System.
were selected (Fig. 2-1). Because of the limited resources and the relatively small dynamic range of freshwater discharge in the Mattaponi River (Fig. 2-2), the slack water stations on the upstream side were selected along the Pamunkey River. Notice that the discharge information was obtained from two USGS stations: one is near Hanover on the Pamunkey River (about 170 km from the York River mouth) and the other is near Beulahville (about 135 km from the York River mouth). Although our original objective was to have one survey period for a dry season and the other for a wet season (Fig. 2-2), the extremely dry year following July 2001 caused the two data set to be very similar. This is unexpected and beyond our control. The first survey period was extended a little because of the bad weather and the school closed at the year end.

Fig. 2-2. Historiography of Freshwater Discharge and Durations for the Slack Water Surveys in the York River System.
The coordinates of the 25 stations are given in Table 2-1. Notice that the distance between each station was short (between 4 to 5 km) because the objective was to obtain a better axial resolution of salinity and TSS gradient at the place where the TSS gradient was large. It is obvious that not all stations were needed if the axial gradient is small. For this reason, some upstream stations (i.e., YR17 and YR24) moved a little in each survey to find the maximum TSS profiles. Also because of this reason, not all of the surveys had measurements at all of the stations listed in Table 2-1. The negative longitude given in Table 2-1 stands for the West hemisphere. All stations were located in the main channel in order to get the maximum salinity and TSS information. Also notice that these surveys were carried out at local slack tide, either after a flood or after an ebb.

INSTRUMENTS

The Conductivity, Temperature and Depth (CTD) profiles were measured using an Apply Micro CTD profiler, model 663. This device was checked and calibrated before each survey. A Seapoint Optical Backscatter Strength (OBS) sensor and a water pump were mounted with the CTD profiler to get continuous readings on the TSS vertical profiles. The pump inlet was aligned to have the same elevation as the OBS for taking water samples whenever the OBS reading showed a significant change. Later, water samples were taken at almost all of the surveyed stations to establish an in-situ calibration equation for converting the OBS readings to TSS readings.

There were two sets of OBS sensors with different settings: one with a 12 bits resolution but saved in 16 bits data formats, and the other with a 12 bits resolution and also saved in 12 bits
data format. For this reason, readings from the 16 bits data format were 16 times larger than those from the 12 bits data. Nevertheless, through the in-situ calibration process, they all can be converted to TSS readings.

Table 2-1. Water Sampling Stations Coordinates for the Slack Water Surveys.

<table>
<thead>
<tr>
<th>Station</th>
<th>Long. (deg)</th>
<th>Lat. (deg)</th>
<th>Dist.(km)</th>
<th>Depth(m)</th>
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SALINITY AND OBS PROFILES

All the CTD profiles were reasonably smooth and can be used directly to construct the “snap” shots of salinity distribution. The OBS readings, however, required further processing because of the reasons given next.

Raw OBS readings showed a large fluctuation in almost all of the profiles because of the possibility that a fish, a sea glass, or any solid subject moved around the OBS sensor and caused a spike or abnormal readings. For this reason, it was necessary to remove these spikes and construct smoothed OBS profiles (Fig. 2-3).

![Diagram of OBS readings](image)

Fig. 2-3. Examples of Measured Vertical Profiles of OBS Readings. “+” are raw data with spikes removed, “o” are averaged.
Fig. 2-3. (continued).
Notice that not all the OBS profiles showed a high gradient near bottom. For these downstream stations (i.e., YR01 and YR02 in general, or YR06 in Fig. 2-3) and at the upstream end when the water depth was small (i.e., YR24 in Fig. 2-3), the OBS profiles were nearly uniform. Only when local convergence was possible or at the primary turbidity maximum, did the OBS profiles have a significant gradient for the lower part of the water column.

An OBS sensor was very sensitive to particle size and the reflection index of suspended particles (Maa et al., 1992). Thus, it was necessary to conduct in-situ calibration during the surveys. Also for this reason, an OBS sensor might respond differently when it was in the top of
water column or in the bottom of water column. This is because the size of suspended solid may be different in the water column. We did find that the OBS calibration curves are slightly different because of the sensor location: at the top or at the bottom of the water column, at the upstream or downstream side of the York River (Fig. 2-4). Fortunately, the difference was not significant, and when considering the data scatter, it was not worth to use different calibration equations for each section. As a matter of fact, using all the in-situ calibration data points to construct an OBS calibration equation was reasonably good (Fig. 2-5). Thus, eight calibration equations were developed to convert the smoothed OBS vertical profiles to TSS profiles (see Appendix I and II).

Fig. 2-4. OBS Calibration Data for Slack Water Survey on 04/11/2002. Different equations were used for the downstream section, the middle section near the water surface or the near bed area for all locations.
SURVEY RESULTS

Survey results were very similar because of the extremely dry year. Nevertheless, the two data sets provided an extreme case for checking the performance of HEM-3D on salinity intrusion, and for validation of TSS distribution on the module developed.

Salinity Distributions

Figures 2-6 to 2-13 are the eight salinity distributions along the York-Pamunkey Rivers. In general, the salinity at the York River mouth was around 24 - 25 ppt. The salinity was still
Fig. 2-6. Salinity Distribution for the 11/29/2001 Slack Water Survey.

Fig. 2-7. Salinity Distribution for the 12/05/2001 Slack Water Survey.
Fig. 2-8. Salinity Distribution for the 12/10/2001 Slack Water Survey.

Fig. 2-9. Salinity Distribution for the 1/18/2002 Slack Water Survey.
Fig. 2-10. Salinity Distribution for Slack Water Survey on 3/19/2002.

Fig. 2-11. Salinity Distribution for Slack Water Survey on 3/25/2002.
Fig. 2-12. Salinity Distribution for Slack Water Survey on 4/02/2002.

Fig. 2-13. Salinity Distribution for Slack Water Survey on 4/11/2002.
high (i.e., around 15 ppt) at West Point which is about 50 km upstream from the York River mouth. For a normal year, the salinity was about 5 ppt at West Point, and thus, the salinity distributions demonstrated severe dry conditions. The maximum salinity intrusion distance was about 90 km from the York River mouth.

The stratification caused by salinity distribution was not strong; most of the time, it was uniform in the vertical direction.

**TSS Distributions**

At the downstream side of the York River, the TSS profiles clearly indicated a slow increase with water depth. Even at depths that were close to the bottom, the TSS values were still small, and only increased about 10 to 20 mg/L (Fig. 2-3). At stations near the upstream turbidity maximum, the TSS profiles increased quickly and had a significant gradient at the middle water depth (Fig. 2-3). For stations at the middle section of the York River, all kinds of vertical profiles were found (see Appendix II). Nevertheless, all available vertical profiles for one survey were smoothed and used to construct a “snap” shot of the TSS distribution for that particular survey.

The survey results are given in Figs 2-14 to 2-21. In general, the TSS concentrations were low and about the same near the York River mouth. The existence of the primary turbidity maximum was obvious and located in the area where the salinity varies from 10 to 4 ppt. Among these survey results, Fig. 2-17 and Fig. 2-21 did not show a clear turbidity maximum.
Fig. 2-14. TSS Concentration Distribution for the 11/29/2001 Slack Water Survey.

Fig. 2-15. TSS Concentration Distribution for the 12/05/2001 Slack Water Survey.
Fig. 2-16. TSS Concentration Distribution for Slack Water Survey on 12/10/2001.

Fig. 2-17. TSS Concentration Distribution for Slack Water Survey on 1/18/2002.
Fig. 2-18. TSS Concentration Distribution for the 3/19/2002 Slack Water Survey.

Fig. 2-19. TSS Concentration Distribution for the 3/25/2002 Slack Water Survey.
Fig. 2-20. TSS Concentration Distribution for the 4/02/2002 Slack Water Survey.

Fig. 2-21. TSS Concentration Distribution for the 4/11/2002 Slack Water Survey.
Downstream Boundary Conditions

There are two possible approaches for obtaining the required downstream side boundary conditions for modeling salinity and TSS distributions in the York River. It was originally proposed to deploy a S4 with an OBS sensor at a location near Station YR01. This approach would provide continuous records of salinity and TSS at a middle elevation of the mouth. However, this approach could not obtain the important near-bottom TSS and Salinity information at the deep channel because a S4 could not be placed in the channel. Considering that the near-bottom TSS and salinity information in the deep channel are more critical for a better simulation of salinity and TSS distributions and a better understanding of the source for TSS, we selected to extend the monitoring at Station YR01 to get the required downstream side boundary conditions. Details are described next.

Every two or three days, salinity and TSS profiles at Station YR01 were measured at a slack tide during the two one-month survey periods. Thus, the TSS and salinity information was available from the channel bottom to the water surface at Station YR01. For other places within this cross section, salinity and TSS profiles were assumed to be the same as those measured at the same elevation in the channel. The survey results were summarized in Figs 2-22 and 2-23. When using this approach for modeling the TSS and the salinity distributions, we would have to assume an amplitude for salinity and TSS variation. Fortunately, these two amplitudes were small, and thus, even assumed zero amplitudes, we were still able to obtain reasonably good results.

In general, the York River mouth is not a source of TSS because of the small TSS values that were measured during the two observation periods. The change of salinity and TSS
boundary conditions were also limited. For example, the maximum change of salinity was only 4 ppt (from 26 to 22 ppt). Time series of the TSS profiles also indicated that TSS was low (around 10 to 20 mg/L) and the change of TSS was small, especially for the planned dry season (10-16 mg/L). For the planned wet season, the change of TSS was also small, from 10 to 30 mg/L.

Fig. 2-22. Measured Downstream Side Boundary for the Planned Dry Season. (a) Salinity; (b) TSS Profiles.
Fig. 2-23. Measured Downstream Side Boundary Conditions for the Planned Wet Season. (a) Salinity; (b) TSS Profiles.
Upstream Boundary Conditions.

Cohn et al. (1992) developed a seven-parameter equation (Eq. 2-1) to estimate the TSS concentration based on daily mean discharge and the number of Julian day in a year for tributaries of the Chesapeake Bay.

\[
\ln(C) = \beta_0 + \beta_1 \ln(Q/Q) + \beta_2 \ln(\ln(Q/Q)^2 + \beta_3 (T-T) + \beta_4 (T-T)^2 + \\
\beta_5 \sin(2\pi T) + \beta_6 \cos(2\pi T) + \epsilon
\]  

(2-1)

where \(\ln\) is the natural logarithm function, \(Q\) is the daily mean fresh water discharge (in \(\text{ft}^3/\text{s}\)), \(T\) is the Julian day measured in a year, \(\beta_0\) to \(\beta_6\) are constant coefficients, \(\hat{Q}\) and \(\hat{T}\) are defined in Eqs. 2-2 and 2-3, \(\epsilon\) is the error assumed to be independent and normally distributed with zero mean and variance \(\sigma^2\).

\[
\hat{T} = \bar{T} + \frac{\sum_{i=1}^{N} [T_i - \bar{T}]^3}{2 \sum_{i=1}^{N} [T_i - \bar{T}]^2}
\]  

(2-2)

\[
\hat{Q} = \bar{Q} + \frac{\sum_{i=1}^{N} [Q_i - \bar{Q}]^3}{2 \sum_{i=1}^{N} [Q_i - \bar{Q}]^2}
\]  

(2-3)

where \(\bar{T}\) and \(\bar{Q}\) are the average time and average discharge over one year.

In a previous study, Lin (2001) used 15 years (1979-1994) data of the suspended sediment concentration influx and the fresh water discharge at the two USGS gauging stations.
(i.e., at Hanover for the Pamunkey River and Belahville for the Mattaponi River) to work out the best fitted seven coefficients $\beta_0$ to $\beta_6$ (table 2-1).

Based on the above formulation, the TSS influx condition at the upstream sides of the York River can be calculated for the two periods of slack water surveys and the results are given in Figs. 2-24 and 2-25.

Table 2-1. Values for the seven parameters used in the USGS sediment influx model

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Fig. 2-24. Measured Freshwater Dischages and Calculated TSS Concentrations as Input at the Upperstream Boundary during the Planned Dry Season.
Fig. 2-25. Measured Freshwater Discharges and Calculated TSS Concentrations as Input at the Uperstream Boundary during the Planned Wet Season.
CHAPTER 3, NEW BATHYMETRICAL GRID

It is our original intension to use the bathymetric grid generated from a previous study and skip the checking of bathymetry and hydrodynamics. It was found later during the slack water surveys that the available bathymetric grid does not have the resolution we needed to properly address the true bathymetry. For this reason, the salinity intrusion cannot be accurately described using the HEM-3D model, and more important, it would be impossible to reveal the true reason(s) of having the observed Total Suspended Solid (TSS) distribution in the York River System. For this reason, the deficiency of the old grid was demonstrated first and followed by the details on the construction of a new grid.

DEFICIENCY OF THE OLD GRID

It is important to have a correct representation of channels in any estuary modeling effort because a channel is the main route for salt water intrusion which is the reason of producing a turbidity maximum near the end of salinity intrusion. While examining the available bathymetric grid, it was noticed that the old rectangular grid does not have enough resolution, and thus, it changed an original continuous channel to be discontinued (Fig. 3-1). Also because of the rather large cell size, and lack of sufficient number of raw data, some of the channel disappeared in the numerical grid. For example, the old channel at Clay Bank (Fig. 3-1) and the channel that approaches West Point were all missing (Fig. 3-2).

The above two important deficiencies will definitely affect salinity intrusion, especially for the extreme dry year we have encountered. Salinity at West Point was about 15-19 ppt
during our survey periods (see Figs. 2-6 to 2-13). Because of a relatively large fresh water discharge for the early study period, the salinity is low at West Point (around 1-3 ppt). Therefore, the old grid, although not good to represent the channel, may not have too much influence on the early study. Nevertheless, an accurate bathymetry is definitely important for this study, and thus, we have to re-generate a better grid. Details are given next.

Fig. 3-1. The Old Rectangular Grid with a Relatively Large Grid Size Changes an Original Continuous Channel to a Discontinued Channel near Clay bank. The old channel at Clay Bank is also missing.
GRID GENERATION

There is a FORTRAN software named GEFDC.FOR in the HAM-3D package for generating curve-linear orthogonal grid. Actually a Cartesian rectangular grid is a special case of curve-linear orthogonal grids. For the reason to better fit curve channels in estuaries, it is decided to use the general curve-linear orthogonal grid.

A digital bathymetric data set is available for the Chesapeake Bay and its tributaries from NOAA. The quality of this data set is excellent. Because of the high density of raw data (Fig. 3-
3), it is possible to reduce the grid size to 100 m x 200 m in transverse and axial directions, respectively.

Fig. 3-3. The Density of Raw Bathymetric Data at the Downstream Section. Blue points are the currently selected and crosses were used in the previous modeling.
Fig. 3-3. (Continued).
In order to use GEFDC to generate a good curve-linear orthogonal grid, high resolution seeding pairs along the axial direction and the transverse directions should be given first. It is best to use an example to demonstrate the above statement. Figure 3-4 shows an ideal curve channel with outer radius of 75 m and inner radius of 50 m. A relatively low resolution seeding pairs (21 pairs in the $z$ direction with $\hat{E}_2 = 9^\circ$) on the outer and inner perimeters failed to generate a good enough curve-linear orthogonal grid (Fig. 3-4a). When improving the resolution to $\hat{E}_2 = 1.8^\circ$ (i.e., 101 pairs in the $z$ direction), the generated curve linear grid is excellent and almost identical to that from an analytical solution (Fig. 3-4b). It is worth to point out that in this particular example, the curve-linear orthogonal grid is only required for the $z$ direction (i.e., axial direction). Therefore the resolution on the transverse direction (4 pairs with $\hat{E}_r = 5$ m) has no effect at all. In reality, however, a curve-linear orthogonal grid may be needed in both axial and transverse directions. Thus, the selection of high resolution seeding pairs on both directions are necessary in order to produce a good grid.

Notice in Fig. 3-4, the perimeter of the channel is exactly where the seeding pairs shall be located for generating interior grid points. In reality, an estuary usually has an irregular width, and thus, an artificial perimeter should be drawn first along the main channel of the estuary. Then the seeding pairs will be selected on the artificial perimeter. For this reason, some of the interior cells may actually represent land and should be excluded in the computation of hydrodynamics and sediment transport when using HEM-3D.
Fig. 3-4. Effect of Seeds Resolution on the Generation of Curve-linear Orthogonal Grid. 
(a) Using a large seeding space ($\Delta \theta = 9^\circ$ and $\Delta r = 5$ m) along the perimeter; 
(b) Using a small seeding space ($\Delta \theta = 1.8^\circ$ and $\Delta r = 5$ m). Circles are the ideal grid points, and crosses are the GEFDC results.
It required a quite extensive seeding pairs for generating a curve-linear orthogonal grid for the York River. Figure 3-5 shows a small portion of the seeding pairs near Gloucester Point. All the preparation of seeding pairs was done using ARC-INFO software package. After running the GEFDC, a curve-linear orthogonal grid was generated. The next step was to remove all those cells that are on land and a final grid was then established.

Fig. 3-5. Part of the Seeding Pairs (Blue and Green dots) Along the Axial Direction near Gloucester Point. They are roughly parallel to the channel.
NEW CURVE-LINEAR ORTHOGONAL GRID

Figure 3-6 is a partial plot of the new curve-linear orthogonal grid. The associated water depth are also plotted using different colors. The important issue is that the channel approaches Clay Bank are clearly represented by two or three cells in transverse direction. Thus it will not be an obstacle for prevent salinity intrusion. Another important transition was necessary to gradually change the horizontal Two-Dimensional (2-D) grid system into an One-Dimensional (1-D) grid (Fig. 3-7). At the upstream side, the Pamunkey and Mattaponi rivers are still represented by using an 1-D system because of the narrow channel.

Fig. 3-6. The New Curve-linear Grid and Bathymetry at the Downstream Section. Color shows the water depth. Notice the great water depth at Gloucester Point caused by the narrow channel geometry.
Even plot the York River in a relatively large scale now (Fig. 3-8), the main channel and the old channel can be clearly seen at Clay Bank. The channel that goes to a paper mill at the West Point also can be clearly seen. We can get this good results because of the following two factors: (1) A much better bathymetrical raw data set, and (2) a fine curve-linear orthogonal grid.

We further checked the bathymetric cross sections at three selected locations indicated in Fig. 3-9 with a relatively rough field survey by running a small boat across the channel. The
echo sounding results indicate that the modeled bathymetrical grid is sufficiently good to represent the reality (Fig. 3-9 - 3-11).

Fig. 3-8. The New Bathymetry for the York River Clearly Shows the Main Channel that Goes to West Point and the Old Channel at Clay Bank.
Fig. 3-9. Comparison of the Model Used Depth and Surveyed Depth at York River Mouth.

Fig. 3-10. Comparison of the Model Used Depth and Surveyed Depth at Clay Bank.
Fig. 3-11. Comparison of the Model Used Depth and Surveyed Depth at West Point.
CHAPTER 4. VERIFICATION OF HYDRODYNAMICS

Since the bathymetry grid was changed to a high resolution curve-linear orthogonal grid (see Chapter 3), we have no choice but to re-check the performance of the HEM-3D on tidal hydrodynamic and salinity intrusion. Details are given next.

TIDES

Because the performance of HEM-3D has been proved before for the York River, there is no need to re-check every details. For example, if the behavior of $M_2$ tide is correct, then there is no reason to think that the behavior of $S_2$ tide will be incorrect. For this reason, we only checked the performance of $M_2$ tide. Also because the energy of $M_2$ tide alone is about 89% of all the major seven constitutive tides ($M_2$, $S_2$, $N_2$, $K_1$, $M_4$, $O_1$, and $M_6$, Sisson et al., 1997), to check $M_2$ alone is sufficient for our purposes.

Along the York River System, there is one NOAA-VIMS cooperative tide station at the Research Pier of VIMS at Gloucester Point with more than 20 years data. There are other 14 short term tide stations established by VIMS during or before the previous study (Sisson et al., 1997, also see Fig. 4-1). All the $M_2$ amplitudes at these 14 short term stations were adjusted to become the long term $M_2$ amplitudes according to the same period of short term tidal records obtained from the NOAA-VIMS station. Details were given in Sisson et al. (1997), and thus, not repeated here. Notice that the majority of tide stations were located on the north side of the York River. For a channel width on the order of 5 km for the York River, the possible effect of tidal range amplification caused by the possible Kelvin waves would be minimum because of the small channel width.
The amplitude of $M_2$ tide at Goodwin Islands (0.34 m) was used as the downstream side boundary condition at the York River mouth. At the upstream side, the recorded fresh water discharges were used with the radiation boundary condition to allow tidal waves to travel further upstream if needed.

For the bottom friction, the HEM-3D only allowed using one single bottom roughness, $r$, to represent the entire estuary, and a typical value of $r = 0.001$ was used.

Because of using the cold start, the HEM-3D was run for 5 cycles of $M_2$ tide to approach the equilibrium state, and the tidal range along the cells that coincide with the channel were “measured” by checking the time series of water surface elevations for lows and highs. We can

Fig.4-1. Tide Stations along the York River System
do this because we are using a curve linear orthogonal grid, and thus, an along-channel profile of tidal range was constructed (the lines in Fig. 4-2). The comparison of calculated and measured tidal ranges (crosses in Fig. 4-2) indicates a satisfactory agreement.

![Diagram showing mean tidal ranges across different rivers](image)

**Fig. 4-2.** Comparison of Calculated and Observed Mean M$_2$ Tidal Range using the New Curve-linear Grid.

**SALINITY**

For checking salinity distribution, we used the measured salinity distributions given in Chapter 2. Because the downstream side salinity boundary condition and upstream freshwater discharges were also given in Chapter 2, they were not repeated here.
The bottom roughness height, \( r = 0.001 \) m, was also applied in this case. The anti-numerical diffusion mechanism was on, and the diffusion coefficient was set to zero for this study.

The HEM-3D started with a “cold start” using the tidal records obtained from the NOAA-VIMS tidal station. Because of the amplification effect shown in Fig. 4-2, the tidal elevations were reduced by 10% and then used as the boundary condition at the York River mouth. Because of using the cold start, the HEM-3D was run for seven days without activated the calculation of salinity. This seven days can be considered as sufficient for the HEM-3D to achieve an equilibrium state in tidal simulation.

The salinity transport was then activated at the end of this seven-day period and the HEM-3D was continued to run for another 3 days without change the salinity boundary condition (Fig. 4-3). At the onset of simulating salinity transport, the initial salinity for all water cells were selected according to the measured salinity distribution obtained from the first slack water survey. Linear interpretation along the channel direction was used to obtain salinity information for all the along-channel cells which are the deepest cell in the corresponding cross sections. The initial salinity for all other cells in a cross section were then estimated based on the following two assumptions: (1) The surface salinity of all the side cells are the same as the surface salinity at the deepest channel, and (2) the vertical profiles of salinity distribution are the same as that in the channel, but stopped at a shallow water depth.

At the downstream boundary site, the actual salinity boundary condition may change with time. During a flood tide, higher salinity from further downstream, ocean side may come to this boundary. Similarly, lower salinity from upstream side may come to this location during an
ebb tide. Fortunately, the change of salinity was not significant, *e.g.*, 3 - 4 ppt, and thus, the error caused by this inaccuracy in boundary condition was limited.

For the first set of slack water survey, the fourth survey was about one month apart from the other three surveys because of the school close at year end, and thus, we don’t have enough boundary conditions between Dec. 11, 2001 and January 7, 2002. For this reason, the

![Salinity transport was activated](image)

**Fig. 4-3. Salinity Boundary Conditions Specified at the York River Mouth for the First Set of Slack Water Survey.**

comparison of calculated and measured salinity distribution was only made for the first three surveys (Figs. 4-4 to 4-6). The date of these three surveys were also marked in Fig. 4-3. Notice the same salinity at bottom and surface at the York River mouth between December 3 and 5 was probably caused by a two-day long North wind (see later in Chapter 6), which may produce a strong lateral circulation at the mouth.
Fig. 4-4. Comparison of Measured and Calculated Salinity Distribution for the 11/29/2001 Slack Water Survey.
Fig. 4-5. Comparison of Measured and Calculated Salinity Distribution for the 12/05/2001 Slack Water Survey.
Fig. 4-6. Comparison of Measured and Calculated Salinity Distribution for Slack Water Survey on Dec. 10, 2001.
For the second set of slack water survey, a similar “cold start” with the boundary condition given in Chapter 2 and a similar process of initial condition were used. During this period, the salinity difference between bottom and surface of the water column at the downstream boundary was also small, e.g., around 2-3 ppt (Fig. 4-7). The comparison with the four slack water survey results are given in Figs. 4-8 to 4-12.

Fig. 4-7. Salinity Boundary Conditions Specified at the York River Mouth for the Second Set of Slack Water Survey.
Fig. 4-8. Comparison of Measured and Calculated Salinity Distribution for Slack Water Survey on 03/19/2002.
Fig. 4-9. Comparison of Measured and Calculated Salinity Distribution for Slack Water Survey on 03/25/2002.
Fig. 4-10. Comparison of Measured and Calculated Salinity Distribution for Slack Water Survey on 4/02/2002.
Fig. 4-11. Comparison of Measured and Calculated Salinity Distribution for Slack Water Survey on 4/11/2002.
CONCLUSIONS

In general, the simulated results indicate that salty water can intrude into the two upstream branches (i.e., Pamunkey and Mattaponi Rivers) with a slightly low salinity in the two upstream rivers. The maximum salinity intrusion distances for all the 7 cases simulated are roughly matched with the measurements. The most significant differences occurred in Figs. 4-4, 4-11, and 4-13 when the measured gradients of salinity were large near West Point (approximately between 50 and 60 km from the York River mouth). Near West Point, the navigation channel is narrow, and most important, we don’t have a high resolution/accurate bathymetry grid for the two upstream rivers. We improved the resolution and accuracy of bathymetry for the York River, but only did a small improvement for the Pamunkey and Mattaponi Rivers which are upstream from West Point. This may cause the high gradient of salinity along that section.

If not use the anti-numerical diffusion mechanism and increases slightly the diffusion coefficient, we shall have more salty water intrusion. Nevertheless, the above simulation results warrant the simulation of suspended sediment transport.
CHAPTER 5. SETTLING VELOCITY, WIND SPEED, AND WAVE RESUSPENSION

The major differences between cohesive and non-cohesive sediment transport are settling velocity, erosion rate, critical bed shear stress for erosion, and consolidation rate. This chapter shows the most up-to-date formulation of settling velocity for non-cohesive sediment, the parameters that affect the settling velocity for cohesive sediment, a realistic approach for measuring settling velocity for cohesive sediment in the York River, and preliminary results of our measurements. Wind records measured at the VIMS meteorological station during the two periods of slack water surveys are also displayed. The possible relationship between wave resuspension at shallow water using wave and OBS data measured from the VIMS Ferry Pier site is also presented.

SETTLING VELOCITY FOR NON-COHESIVE SEDIMENT

For non-cohesive sediment, there is little sediment-sediment interaction except when the suspended sediment concentration reaches a level of 10 g/L or more. For this reason, each sediment grain can be treated as if it is the only sediment grain in the water.

The terminal settling velocity, $w_s$, of a granular sediment particle is a function of grain size, $D$, kinematic viscosity of water, $\nu$, and the relative grain density, $\rho' = (\rho_s - \rho \)/\rho_s$, where $\rho_s$ and $\rho$ are solid mass density and water mass density, respectively. Hallermeier (1981) presented three universal equations to determine $w_s$ based on the Archimedes buoyancy index, $A$, which is defined as $A = \rho' D^3 / \nu$ where $g$ is the gravitational acceleration. Later Ahrens (2000) merged the three equations into one for easy to use, and later Chang and Liou (2001) further improved
the formulation for a better use of the one equation for \( w_s \) (Eq. 5 - 1).

\[
\frac{w_s}{D} = \frac{\nu}{18} \left( \frac{aD^n}{1 + aA^{n-1}} \right)
\]

where \( a \) and \( n \) are two constants and they suggested \( a = 30.22 \) and \( n = 0.463 \) for general use.

Equation 5-1 will be reduced to the Stokes’ (1851) falling velocity \( (i.e., w_s = \frac{\nu gD^2}{18}) \) for a small sphere with the Reynolds number \( R = \frac{w_sD}{\nu} \) less than one.

For a small granular sediment particle \( (i.e., D < 0.2 \text{ mm}) \), it only takes a few millimeters to approach the terminal settling velocity. Considering that the minimum water depth in any numerical modeling for estuarial flows is usually on the order of 0.5 m which is much larger than that required to approach the terminal settling velocity. Thus, only the terminal settling velocity should be considered in the modeling of sediment transport.

SETTLING VELOCITY FOR COHESIVE SEDIMENT

For cohesive sediments, the settling velocity is a complicated process for modeling. For primary particles, the Stokes formula can be used to estimate the settling velocity. Unfortunately, suspended cohesive sediments rarely exist in primary particle forms. Most likely, they exist as sediment flocs.

Floc Properties

Because of the crystal structure, a primary particle of cohesive sediments has a large ratio
of surface area to grain volume. This means the shape of a primary particle of cohesive sediment looks like a piece of paper, or a book. The alignment of crystal structure also bring negative charges to the large flat surfaces and leave positive charges on the edges (Fig. 5-1). Thus, these primary particles will attract water to form moving clusters, or attract other primary particles to form flocs. It is also possible for primary particles to attract organic matters (if available) and form hybrid components.
Before the fresh sediment-laden water meets seawater (i.e., no salt), the repulsive electric force on a primary particle’s surface is much strong than the attractive force (mainly the Van der Waal’s force). For this reason, cohesive sediments are most likely to form face-to-edge flocs (also called non-salt flocculation, see Fig. 5-2a). Thus the void ratio (the ratio of empty space to solid space within a floc) is large. After the fresh sediment-laden water meets seawater (i.e., with salt), sodium ions in seawater will replace the attached water layer and depress the repulsive force. Thus, cohesive sediments are much easy to form face-to-face flocs (Fig. 5-2b, also called salt-type flocculation) when there are sodium ions around. For this reason, the void ratio of sediment flocs in salt water is relatively small and the floc density is relatively large when compared with those of sediment flocs in fresh water.

Turbulence and Total Suspended Solid (TSS) concentration are the other two factors that affect the formation of flocs. Turbulence can speed up the formation of flocs (if the turbulence is weak) or break flocs (if the turbulence is strong). TSS concentration indicates the abundance or availability of sediment to form flocs. In other words, sediment in water is a necessary

![Diagram of floc types]

**(a)** Edge-to-face flocculated and aggregated   
**(b)** Face-to-face flocculated and aggregated

**Fig. 5-2. Two Major Types of Clay Flocculation. (a) Non-salt Flocculation; (b) Salt flocculation.**
condition to form flocs. In summary, the amount of cohesive sediment, the ambient turbulence, and the existence of salt will determine the floc size and density, and thus, the settling velocity.

The above statements suggest that the best approach for obtaining the true settling velocity would be to carry out in-situ measurements with the above three parameters not affected. In reality, however, a perfect approach for measuring the settling velocity for cohesive sediments does not exist yet. Even the most popular Owen tube method is not perfect because it blocks out the turbulence. Nevertheless, the Owen tube method is still the most popular method and was used in this study.

**Owen Tube Method**

Owen tube is not a commercially available product. However, some descriptions of the Owen tube and details on the method of data analysis are available (Owen, 1976). For this reason, the details of data analysis has been omitted and only the details of the Owen tube manufactured for this project are given here.

The Owen tube consisted of two 1.2 m long plexiglass tubes with inside diameters of 5.4 cm and 10 cm, respectively. These two plexiglass tubes were placed together to have the same center, and the space between the outer and the inner tube was filled with ambient water while being lowered into the river to form a thermal isolation layer (Fig. 5-3a). The Owen tube was set at a horizontal position and opened at the two ends when lowered into the water (Fig. 5-3b). It was left at the sampling location for a few minutes to let tidal current flushed through the inner tube and filled up the space between the inner and outer tubes. After closed the two ends by using an trigger device, the tube was lifted into a boat and turned to a vertical position (Fig. 5-
3a) in order to start the test of measuring settling velocity. Our first design of the tube (one on the right hand side in Figure 5-3a) is a failure because of the incorrect end closing mechanism. We used almost all of our boat time to learned the reason, and continued to modify the closing mechanism in laboratory to make it works. Because of the lack of boat time, we continued our experiment in laboratory using tap water. For this reason, the settling velocity presented in Fig. 5-4 is for fresh water. There, the settling velocity presented in Fig. 5-4 is on the low end. The settling velocity in salty water should be higher. Currently, there is no settling velocity data for salty water environments, but the formula presented in Fig. 5-4 is the basis for tuning in the numerical modeling.

Fig. 5-3. Measurement of Settling Velocity using the Owen Tube Method.
**Measurement Results**

Owen’s (1976) procedures were followed to calculate the settling velocity for surficial sediment collected from the York River at the Clay Bank site (Fig. 5-4). Because the maximum TSS concentration measured in the York River was about 200 mg/L, there is no need to find a regression equation for the settling velocity of high TSS concentration, at least not for the current modeling effort. For TSS concentration less than 200 mg/L, the measured settling velocity can be estimated using the formula given next

\[ w_s = aC^{0.375} \]  \hspace{1cm} (5-2)

where C is the local TSS concentration in mg/L, and a is a constant equal to 4.64x10^{-6} m/s for fresh water. With salinity and turbulence, the value of a should have been higher because of the face-to-face flocculation and better chance to form aggregation. However, the proper value for the constant a will be determined at the time of modeling to better fit the slack water survey results.

**WIND RECORDS**

The Virginia Institute of Marine Science maintains a weather station on the roof of Byrd Hall. The instruments are 19.5 m above the ground. Wind velocity, rainfall, and solar radiation were observed continuously 24 hours a day for 365 days a year. The observations were averaged every 6 minutes and then kept as one record. Thus, there are 10 raw data in each hour and 240 raw data in each day. The maximum and mean values within one hour, as well as within one
day, were further archived for use. These data can be accessed by going to the world wide web
http://www.vims.edu/resources/databases.html#pier/ and then selecting a time period for data
retrieval. During the two slack water surveys, the rainfall records were all zero, and thus, were
not displayed. The wind velocities during the time of two slack water surveys are given in Figs.
5-5 to 5-7. During the first slack water survey (started Nov. 29, 2001), the wind was relatively
weak with an average wind velocity of 3.75 m/s and did not exceed 10 m/s for the entire 40 day
periods. During the second slack water survey (started March 19, 2002), the wind was relatively
strong with an average speed of 5.5 m/s and a maximum speed of 15 m/s. Fig. 5-6 is an enlarged
plot of wind speed and direction for the period from Nov. 29, 2001 to Dec. 5, 2002. It shows the
northern wind, which is perpendicular to the lower York River from Gloucester Point to the

![Graph of Measured Settling Velocity for York River Sediment](image)

Fig. 5-4. Measured Settling Velocity for York River Sediment.
York River mouth, with a nearly constant speed of 4 m/s for two days might induce enough across-channel circulation to fully mix the salinity at the York River mouth. This is one of the possible reasons for having a uniform salinity distribution during this period of time (see Fig. 4-3).

Fig. 5-5. **Hourly Maximum (6 minutes averaged) Wind Speed and Direction** Measured at the VIMS Campus from Nov. 1 to Dec. 10, 2001.

Fig. 5-6. **Detailed Hourly Maximum Wind Speed and Average Direction** Measured at the VIMS Campus from Nov. 29 to Dec. 5, 2001.
SIGNIFICANCE OF WAVE RESUSPENSION IN SHALLOW WATER

Water waves are known to be an effective erosion force because of the small wave boundary layer thickness, especially in shallow waters. This is a reliable statement for open waters that wave force is much stronger compared with tidal current. In estuaries with a limited wind fetch length, the possible wave heights and wave periods are usually limited. For this reason, the significance of wave erosion become unclear. It was suggested to establish a pilot observation station to monitor the possible relationship between wave height and TSS concentration. It is understood that data from one observation station may not represent the entire York River, but the results may give some suggestions as what shall be done next.

We used a pressure gage (Druck, model PDCR130/W), an OBS (Seapoint Technology), an Onset Model 4A data logger, and a 12 inches long, 5 inches PVC tubes to build a underwater
unit for sensing the pressure, temperature, power voltage, and OBS reading. This unit was mounted on a 2 inches rod which was tied to the VIMS’ Ferry Pier (Fig. 5-8). The mounting position was selected such that after deployment, the OBS sensor would be about 60 cm from the sea floor. A cable connected this underwater unit to a PC located in a laboratory on the Ferry Pier. The PC controled the underwater unit, collected the data measured, and stored the data in compact flash memory. Data stored in memory were downloaded every week or so, depending on the pace of marine fouling. When the water temperature exceeded 25°C, we have to clean the OBS every two or three days.

We first deployed the observation station in June 2002. It worked well until mid-July when the system was destroyed by thunder strikes that cause a total loss of about $5000. We replaced everything necessary and re-started the monitoring in late September, 2002, but the system did not work well until November, 2002. In mid-December, the system failed again due to a leakage problem, and the data logger was lost. Because wave activity and TSS concentration were low in winter, we delayed the re-installation until the following spring. This gave us the time needed to work out the leakage problem and to order new parts. On April 3, 2003, we returned to the Ferry Pier and the observations continued until June 3, 2003, at which time, the pressure gage failed. Considered the observations covered a sufficient long period of time, the observation was stopped, and the collected data were analyzed as given next (Figs 5-9 to 5-14).
Fig. 5-8. Location of Wave Height and TSS Observation Station at the VIMS’ Ferry Pier Site. The coordinates are given in the diagram.

Fig. 5-9. Observed Wave, Tide, and TSS Concentration at the VIMS’ Ferry Pier Site in June 2002.
Fig. 5-10. Observed Wave, Tide, and TSS Concentration at the VIMS' Ferry Pier Site in July 2002.

Fig. 5-11. Observed Wave, Tide, and TSS Concentration at the VIMS' Ferry Pier Site in November 2002.
Fig. 5-12. Observed Wave, Tide, and TSS Concentration at the VIMS’ Ferry Pier Site in December 2002.

Fig. 5-13. Observed Wave, Tide, and TSS Concentration at the VIMS’ Ferry Pier Site in April 2003.
Fig. 5-14. Observed Wave, Tide, and TSS Concentration at VIMS’ Ferry Pier Site in May 2003.

Fig. 5-15. Hourly Maximum (6 minutes averaged) Wind Speed and Direction Measured at VIMS Campus from April 9 to April 19, 2003.
The above figures indicate that TSS concentration did not correlate well with wave height. For example, Fig. 5-10 shows a plume was detected at the Ferry Pier site from July 3 to 5, but the waves were small during the same period. Another example can be found in April 2003 (Fig. 5-13). Waves were practically the same, and the tide seemed affected by strong northern winds from April 8 to 12 and NE winds on April 17 (see Fig. 5-15). From April 15 to April 23, the TSS concentration gradually increased to a relatively high value and then decreased back to its normal background. In this particular case, wind-induced secondary circulation might be responsible for a slowly transport of a high TSS plume to the observation site. The role of wave erosion and bank erosion caused by breaking waves does not show any significance at this site.
CHAPTER 6. COHESIVE SEDIMENT TRANSPORT

The current sediment transport module in HEM-3D available at VIMS only support a single class of sediment sizes, and the codes has not been checked for its performance yet. This project used the existed frame but modified the formulation for erosion and deposition to build a fully verified module for simulating cohesive sediment transport. Despite the dynamic behaviors are different between cohesive and non-cohesive sediment, the basic transport equations are the same. The major differences are in settling velocity, erosion rate, critical bed shear stress for erosion, and consolidation rate. In Chapter 5, a formulation for the minimum settling velocity for cohesive sediment in the York River was suggested. In this chapter, an innovative approach for addressing the erosion and deposition processes together for simulating cohesive sediment transport is presented.

GOVERNING EQUATIONS

The following generalized equation for sediment mass conservation will be used,

\[
\frac{\partial C}{\partial t} + u \frac{\partial C}{\partial x} + v \frac{\partial C}{\partial y} + (w - W_s) \frac{\partial C}{\partial z} = \frac{\partial}{\partial x} \left( -u'C' \right) + \frac{\partial}{\partial y} \left( -v'C' \right) + \frac{\partial}{\partial z} \left( -w'C' \right) \quad (6-1)
\]

where \( C \) is the sediment concentrations for suspended sediment, \( u, v, \) and \( w \) are the three flow velocity components, \( W_s \) is the settling velocities for the suspended sediment, \( t \) is time, \( <> \) represent time-averaged values, and \( x, y, \) and \( z \) represent the three coordinates. Equation 5-1 is difficult to be solved accurately even with the best numerical model currently available. This is because of the changing of settling velocity with time and ambient environments, the exchange
of sediment particle/floc by flocculation and break-up, the uncertainty in turbulence, and the uncertainty of boundary conditions at bottom, sides, upstream, and downstream borders.

Regarding to turbulence mixing, the last three terms (i.e., \( M^{-u'C}/M \), \( M^{-v'C}/M \), and \( M^{-w'C}/M \)) in Eq. 6-1 can be replaced by \( M(D_x(MuC/M_x)) / M_x \), \( M(D_y(MvC/M_y)) / M_y \), and \( M(D_z(MwC/M_z)) / M_z \), where \( D_x \), \( D_y \), and \( D_z \) are the three components of turbulent diffusion coefficient in \( x \), \( y \), and \( z \) directions, respectively. For simulating cohesive sediment transport, we may assume that the turbulent diffusion coefficient equals the turbulent eddy viscosity.

It is understood that the turbulent dispersion coefficient should not be used in this study. This is because the turbulent dispersion coefficient was caused by averaging the observed data across a cross-section to obtain an averaged TTS or salinity reading for that cross-section. Thus, the turbulent dispersion coefficient was set to zero because we do not have this kind of data. All of our measurements were obtained at one location in the channel. However, the HEM-3D allows a user to modify the turbulent diffusion coefficient.

**BOUNDARY CONDITIONS**

In order to solve Eq. 6-1, the following boundary conditions must be specified. It was assumed that the amount of water, salt, and sediment that comes from precipitation over the York River water surface is negligible, and thus, excluded in the modeling. This is a very good assumption because the precipitation records at VIMS showed that there is no rain fall during the two periods of slack water surveys. The wind shear stresses that acted on the water surface is also negligible to affect the circulation pattern. This is because wind records at VIMS station between Nov. 2001 and April 2002 indicate that the averaged wind speed were not strong, on the
order of 3.75 m/s and 5.5 m/s with all possible wind directions, see Chapter 5. We also assumed that sediment input from side boundary is relatively small, and thus, negligible. Our limited observations based on a monitoring station established at VIMS also support this approach (see Chapter 5). The remaining terms are discussed below:

**Upstream/Downstream Boundary Conditions**

In Chapter 2, we have discussed the approach for acquiring vertical profiles of salinity and TSS at the York River mouth. Those data will be used directly as the downstream boundary conditions.

We also presented the results for upstream side boundary conditions in Chapter 2. Although no vertical distribution information is available, it can be safely assuming a uniform vertical distribution because of the shallow water depth, low freshwater discharge, low salinity (< 1 ppt), and low TSS concentration (< 60 mg/L). Notice that this assumption may be changed if the above three conditions change.

Because the freshwater discharge and TSS readings were all small for our two study periods, it would be logical to assume that majority of the TSS are fine clay particles. For our study period, we may assume that there was no sandy sediment influx from the upstreams.

**Bottom Boundary Condition: A Novel Approach**

For any traditional sediment transport models, the processes that need to be addressed at the bottom boundary include: erosion, deposition, and consolidation. Maa and Kim (2002) used the VIMS Sea Carousel to measure erosion rates at the Clay Bank site in the middle section of
the York River for four seasons. Their objective was to check the seasonal variability of erosion. That is probably the only attempt to directly measure the erosion rate in this river. With no data for other place of this river which spans more than 100 km long, it is difficult, if not impossible, to accurately simulate the bottom boundary condition for the entire York River. For this reason, we would like to simplify this bottom boundary condition by combining the erosion and deposition processes into one process which can be described as “erosion occurs only when tidal flows are accelerating and a constant erosion rate may be applied.” This simplification also implies that the change of estuarial morphology is not included in this study. As a matter of fact, this modeling effort concentrates on the transport of sediment, and the formation of turbidity maximum.

The simplified bottom boundary condition is also justified from the four in-situ erosion experiments (using the VIMS Sea Carousel) carried out at the Clay Bank site in the York River (Maa and Kim, 2002). Because this is an important simplification of the sediment transport modeling, further discussion is given as follows.

Using the VIMS Sea Carousel for erosion tests (Maa, 1993; Maa et al., 1993; 1998), the observed erosion behavior is always the “Type 1 behavior,” which means that for a given excess bed shear stress, the eroded sediment mass decreases with time because of the increase of critical bed shear stress \( \tau_c(z) \) with depth. Type 1 erosion behavior can be expressed as follows

\[
\varepsilon(t) = \varepsilon_o \, e^{-\lambda t}
\]

(6–2)

where \( \varepsilon_o \) is the erosion rate at \( t = 0 \) for a given excess bed shear stress, \( \varepsilon \) (t) is the erosion rate at
Given a elapsed time, t, and \( \lambda \) is the rate constant.

In general, the rate constant, \( \lambda \), 0.005 s\(^{-1}\), appears to be an universal constant if the content of sediment is more than 30% of clay. For example, in the clay-rich Baltimore Harbor and San Diego Bay, \( \lambda \) are also around 0.005 s\(^{-1}\) (Maa et al., 1998). Figure 6-1 further shows the results of \( \lambda \) for the Clay Bank site in the York River. The physical meaning of \( \lambda \), 0.005 s\(^{-1}\) is that \( \lambda (t) \) of 0 in 900 seconds (15 minutes). This means that after 15 minutes, the resuspension process is almost completely ceased. That is a condition when the bed shear stress, \( \lambda_b \), equals the critical bed shear stress, \( \lambda_{cr}(z) \). Because tidal flows (i.e., tidal induced \( \lambda_b \)'s) do not change significantly in 20 minutes, tidal erosion is always nearly in equilibrium, and the excess bed

![Diagram](image)

*Fig. 6-1. VIMS Sea Carousel Measured Erosion Rate Constant: (a) \( \lambda \) and (b) \( \varepsilon_o \).*
shear stress is always small. Thus, we may use a constant erosion rate for tidal flows.

The traditional formulation for erosion of cohesive sediments (Eq. 6-3) also suggests a constant erosion rate if the change of $J_b$ is small. In Eq. 6-3, M and n are two constants. Based on an analytical study, Parchure and Mehta (1985) found that $n$ should be 1/2. For practical applications, however, $n = 1$ is often used for its simplicity. Even with $n = 1$ (or 1/2), Eq. 6-3 remains impractical because there is no way to accurately know $J_{cr}(z)$, especially in the top few centimeters of sediment beds. Therefore, an assumption of $J_{cr}(z)$ must be made for modeling purposes. This leads to the unavoidable tuning of $M$ and $J_{cr}(z)$ in any modeling study of cohesive sediment transport.

$$
\varepsilon = M \left( \frac{\tau_b}{\tau_{cr}(z)} - 1 \right)^n
$$

(6-3)

Furthermore, since erosion and deposition may occur alternatively and frequently, this implies that the profile of $J_{cr}(z)$ must be frequently adjusted. This further complicates the simulation and increases the difficulty of using Eq. 6-3 directly.

VIMS Sea Carousel studies suggest that the variation of $[\tau_b/\tau_{cr}(z) - 1]$ remains a reasonably small positive constant during the entire tidal acceleration phase. During the tidal deceleration phase, however, $[\tau_b/\tau_{cr}(z) - 1]$ is a negative number because $\tau_b < \tau_{cr}$. Therefore, it is reasonable to assume that Eq. 6-3 can be reduced to

$$
\varepsilon = \text{constant} \quad \text{for tidal acceleration phases}
$$

$$
\varepsilon = 0 \quad \text{for other phases}
$$
As the tide changes from neap to spring, however, the constant \( k \) may also change. Fortunately, the range of \( k \) is clearly specified in this chapter. As indicated in Fig. 6-1b, the history of deposition and consolidation can affect the erosion rate significantly, and a proper selection of the constant erosion rate depends on the bulk density of surficial sediment.

Finally, a constant erosion rate is further supported by tripod observations obtained at the Clay Bank site (Fig. 6-2). Notice that SSC (Fig. 6-2b) always increases during tidal acceleration phases. The decrease in SSC during the deceleration phases indicates that upward diffusion gives way to downward flux. The net downward flux will increase the sediment concentration at a near-bed elevation. When \( J_b \) is sufficiently small, suspended sediment will deposit and start consolidation.

![Graph](image)

*Fig. 6-2. Tripod Measured (a) Tidal Current and (b) Total suspended matter in the York River.*
If straight line (i.e., results from a constant erosion rate) segments are used to link the peaks and minimums of SSC (see the dashed lines in Fig. 6 - 2b) at tidal acceleration phases, one can see that a constant erosion rate can be applied to find the maximum SSC’s at 10 cm above the bed for all the erosion cycles shown in Fig. 6-2.

IMPLEMENTATION OF THE CONSTANT EROSION RATE

According to the location, the maximum bed shear stress induced by tidal current may vary significantly in an estuary. For this reason, it is not logical to use the same constant erosion rate for the entire estuary. Since the information of maximum bed shear stress, $J_{b_{\text{max}}(i,j)}$, at each horizontal water cell (here $i$ and $j$ are cell identification number) can be saved after checking the tidal hydrodynamics and salinity intrusion, it is not difficult to categorize the cell according to the maximum bed shear stress acted on that cell. The first step is to find the absolutely maximum bed shear stress for the entire water cells, $J_{b_{\text{max}}}$. The next step is to categorize each cell according to its own $J_{b_{\text{max}}(i,j)}$. Assuming N categories can be established with an interval of $J_{b_{\text{max}}}/N$, the constant erosion rate for each cell can be prorated according to its category. It has been found that N can be selected more than 8 for having enough resolution. Figure 6-3 shows the maximum bed shear stress distribution with eight categories.
Notice that the area with large maximum bed shear stress matches with the deep channel at Gloucester Point, between Gloucester point and West Point, and in the Pamunkey river. On the other hand, the maximum bed shear stress is small at shallow areas and downstream from Gloucester Point. The erosion experiments carried out during 1995 deployed the VIMS Sea Carousel in the secondary channel near Clay Bank (see the mark in Fig. 6-3). This site had a

**Fig. 6-3. Distribution of HEM-3D Calculated Maximum Bed Shear Stresses.**
water depth of about 5 m and it is on the south side of the main channel. The maximum bed shear stress at this site is about 0.8 Pa. Since the duration of a tidal acceleration phase is about 4 hours (i.e., 240 minutes, see Fig. 6-2) and the tidal force only changes slightly within 20 minutes, the 0.8 Pa bed shear stress must be increased across this period of time. If assuming it increases linearly, the excess bed shear stress can be estimated as 0.8 Pa/ (240 min / 20 min) = 0.0667 Pa.

Using this information, the erosion rate can be found varies between 0.02 - 0.1 g/m²/s (Fig. 6-1). Of course, this is just one estimation of the possible erosion rate, and the actually erosion rate may change with season significantly. Nevertheless, this procedure provided a base of selecting the “constant erosion rate.” The simulation results given next was based on a constant erosion rate of 0.04 g/m²/s.

It is understood that at those places that the bed shear stress is larger than 0.8 Pa, the erosion rate should be larger than 0.04 g/m²/s, and similarly, the less the maximum bed shear stress, the less the erosion rate. For this reason, we established a table with eight different constant erosion rates and each erosion rate was assigned to a cell according to their category in the maximum bed shear stress table. Notice that the vertical axis of Fig. 6-1b is in log scale. This means the difference in the above mentioned table for eight constant erosion rates will not be uniform. At this time, we selected a ratio that is given in Fig. 6-3 with the number 1.0 stands for 0.04 g/m²/s.
AREAS OF COHESIVE SEDIMENT

Although in the York River, most of the river bed material can be classified as cohesive sediment with a clay content more than %30, however, a small portion of the York River bed can be classified as sandy bed. Nichols et al. (1991) surveyed the York River and provided a map, which was adopted in this report (Fig. 6-4), and the above mentioned processes was applied over the area that the bed material is cohesive sediment.

Fig. 6-4. A Map to Show Areas with Different Sediment Beds. Red areas are for non-cohesive sediment, the rest is for cohesive sediment bed.
Other parameters used for simulating cohesive sediment transport include: (1) In Input Card # 6, the switch for anti-numerical diffusion was turned off; (2) In Input Card # 12, a dimensionless horizontal momentum diffusivity of 0.003 was used to slightly increase the horizontal transport; (3) The settling velocity was selected as \( w_s = 3.5 \times 10^{-5} \cdot C^{0.375} \); (4) the time increment for the HEM-3D model, \( \Delta t \), decreases from 30 s (for simulating tidal and salinity) to 5 s (including the simulation of sediment transport) for maintaining numerically stability.

RESULTS AND DISCUSSION

The above conditions were used to simulate the distribution of TSS concentration for the two periods of slack water surveys. Only one class of cohesive sediment was used. The simulation results were compared with the measurements and presented in Figs. 6-5 to 6-11.

In general, the model simulated TSS concentration is slightly higher than those measured and the location of turbidity maximum is also off a little, on the order of 5 to 10 km. Using the selected approach, erosion at Clay bank area is always existed, and that may contribute to the existing of secondary turbidity maximum.

Because of the decrease of time interval caused by numerical instability (i.e., \( \Delta t = 5 \) s), the computing time for an one-month simulation using a 2.8 Ghz Pentium 4 PC with the fastest RDRAM (1066 Mhz) and the fastest hard disk (SCSI 160 at 10000 rpm) still requires about 35 hours to complete. For this reason, we only tried four times to adjust the settling velocity and erosion rate constants. This means further improvements on model performance is possible by further turning these two parameters. Nevertheless, the results presented here indicate that the proposed processes are good enough to reproduce the turbidity maximum.
Fig. 6-5. Comparison of (a) Measured and (b) Modeled TSS Distribution for the Nov. 29, 2001 Survey.
Fig. 6-6. Comparison of (a) Measured and (b) Modeled TSS Distribution for the Dec. 5, 2001 Survey.
Fig. 6-7. Comparison of (a) Measured and (b) Modeled TSS Distribution for the Dec. 10, 2001 Survey.
Fig. 6-8. Comparison of (a) Measured and (b) Modeled TSS Distribution for the March 19, 2002 Survey.
Fig. 6-9. Comparison of (a) Measured and (b) Modeled TSS Distribution for the March 25, 2002 Survey.
Fig. 6-10. Comparison of (a) Measured and (b) Modeled TSS Distribution for the April 2, 2002 Survey.
Fig. 6-11. Comparison of (a) Measured and (b) Modeled TSS Distribution for the April 11, 2002 Survey.
CHAPTER 7. NON-COHESIVE SEDIMENT TRANSPORT

It is understood that the transport equations are the same for both non-cohesive and cohesive sediments, and the differences between these two sediments are the formulation of erosion, settling velocity, consolidation, bed roughness, and the existence of bed load transport. For cohesive sediments, there is no bed load because of the small particle size. Any eroded material is suspended immediately. For non-cohesive, granular sediments, bed load is another important transport mechanism. Because of the following reasons, consolidation and bed load transport were also excluded in this study. These reasons are: (1) The consolidation pace is fast for granular sediments, it almost completed once deposit; (2) This study is concentrated on the simulation of suspended transport process; (3) Bed load of granular sediment has limited effect on suspended load. Since the settling velocities for both kinds of sediments were described in Chapter 5, only the erosion process for non-cohesive sediments and the effect of bed roughness are presented in this chapter.

TRADITION APPROACH

It would be better to explain the erosion process starting from a simple case on the simulation of non-cohesive sediment transport. This is because an analytical solutions of Eq. 6-1 is available for this simple case: no horizontal gradients, the suspended sediment is a uniform sandy sediment, the suspended concentration is low (i.e., less than 10 g/L, and thus, \( w_s \) is a constant because of no particle-to-particle interactions), and the flow is steady or quasi-steady without salinity vertical gradient (i.e., a parabolic distribution of \( D_z \)). The following Rouse equation for the vertical profile of suspended sediment concentration, \( C(z) \), can be obtained
(HENDERSON, 1966; van Rijn, 1984).

\[
\frac{C(z)}{C_a} = \left[ \frac{z_a (h - z)}{z (h - z_a)} \right]^{u_s/u_s}
\]  

(7-1)

where \( \theta \) is the Von Karman constant, \( h \) is the water depth, \( z \) is the vertical coordinate with \( z = h \) at water surface, \( u_s \) is the bed shear velocity, \( w_s \) is the settling velocity, and \( C_a \) is the reference concentration at a near-bed elevation, \( z_a \). Notice that Eq. 7-1 is useless if \( C_a \) is not given, and the settling velocity, \( w_s \), only varies with grain size and water viscosity, \( i.e. \) water temperature (see Chapter 5 for the formulation).

To simulate the above given TSS vertical profile, one only needs to use a parabolic distribution of diffusion coefficient, \( D_z \), in the vertical direction. For estuary environments, the \( D_z \) profile is much complicated because of the vertical gradient on salinity and the influence caused by tidal acceleration. Horizontal diffusion might also has contribution to the local vertical TSS profile. Fortunately, the simulation of \( D_x \), \( D_y \), and \( D_z \) have been established in the HEM-3D and they include the effect of salinity and suspended sediment concentration. Thus, the only process left is erosion at the water-sediment interface.

As mentioned before, any numerical solution for the TSS concentration distribution would not be a complete solution if \( C_a \) is not given. Thus, the key issue on erosion of non-cohesive sediment is to address the \( C_a \). Once \( C_a \) is known, the sediment amount available for transport can be calculated. In other words, the erodable sediment amount is available. From here on, the turbulence and settling taking over to control the transport.
Resuspension Coefficient

SMITH (1977), building on the pioneering works of LANE and KALINSKE (1941), EINSTEIN (1950), and YALIN (1977), proposed the following formulation:

\[
C_a = C_b \gamma_o S / (1 + \gamma_o S) \tag{7-2}
\]

where \( C_b \) is the bed sediment concentration of the surficial sediment, and \( S \) is a non-dimensional excess bed shear stress defined as \( S = (J_b - J_c)/J_c \), where \( J_b \) is the bed shear stress that can be provided by the numerical model, HEM-3D, and \( J_c \) is the critical bed shear stress for incipient motion. Shields (1936) provided the results of his study, and we will be back on this term later.

The most critical parameter in Eq. 7-2 is probably \( \gamma_o \). SMITH and MCLEAN (1977), from their measurements in the Columbia River, found that \( \gamma_o = 0.0024 \) for coarse to medium sand (mean diameter = 0.27 mm). GLENN (1983) found a similar value of \( \gamma_o = 0.003 \) for pure sand with a grain size ranging from 0.15 to 2.82 mm. For silt, however, GLENN (1983) estimated that \( \gamma_o \) reduced to \( 5 \times 10^{-4} \). WIBERG and SMITH (1983) found that for silt with \( D_{50} = 70 \mu \text{m} \) at Norton Sound, Alaska, \( \gamma_o = 1 \times 10^{-5} \). HILL et al. (1988) evaluated \( \gamma_o \) for medium to fine sand (\( D_{50} = 0.17 \text{ mm} \)) in a laboratory flume and concluded that \( \gamma_o = 1.3 \times 10^{-4} \). DRAKE and CACCHIONE (1989) used the Geoprobe at two sites in the "mid-shelf silt belt" on the California Shelf and found that \( \gamma_o \) may vary from \( 1.5 \times 10^{-5} \) to \( 3 \times 10^{-4} \) with an average of \( \gamma_o = 1.6 \times 10^{-4} \). The dispersed mean particle sizes are 16 and 24 \( \mu \text{m} \) for the two sites, respectively. MADSEN et al. (1993) found a remarkable consistency among their estimations of \( \gamma_o \) for Duck sediments (\( D_{50} = 0.1 \text{ mm} \)) under severe sea conditions. The average value is \( 4 \times 10^{-4} \) (with \( z_a = 7D_{50} \)).

Other investigators found that \( \gamma_o \) may change with the bed shear stress. STERNBERG et
al. (1986) found that \( \Theta \) may be linearly proportional to the nondimensional excess bed shear stress, S. Using Optical Backscatter Sensors (OBS, DOWNING 1983) and velocity measurements for a well-sorted to moderately-sorted fine sand at the Duck experimental site, KIM (1990) found that \( \Theta \) decreases with S. For example, in fair weather, \( \Theta \) was 0.002 but for severe seas, \( \Theta \) was reduced to 0.0004.

Figure 7-1 summarizes the information about \( \Theta \) mentioned in the previous paragraphs and indicates a possible influential parameter, sediment grain size. Because each investigator had their own selection of \( C_a \) and \( z_w \), a direct comparison of these results might not be meaningful. On the other hand, if most of the study results indicate a trend, it may be a signal
for a missing parameter, in this case, the sediment grain size. The line in this figure is added to show the possible influence of sediment grain size on \( \phi \).

The above data indicate that although \( \phi \) was originally defined as a constant, it was treated as a tunable coefficient to fit the measured suspended sediment concentration profiles, and Fig. 7-1 indicates the range of possible selection.

**Critical Bed Shear Stress**

In the formulation of excess bed shear stress, \( S \), the determination of the critical bed shear stress, \( J_c \), is another important issue. Back to 1936, Shields published his results (i.e., the Shields Diagram in Fig. 7-2) on how to find the parameter. In Fig. 7-2, the range of grain Reynolds number, \( R_g \), varies from 1 to 1000. This range includes all the possible flow conditions, from laminar flow to turbulent flow. For practical application, however, \( R_g \) is always in the range of fully turbulent flow, i.e., the grain Reynolds number is larger. For this flow condition, the critical bed shear stress can be estimated as

\[
\tau_c = 0.06 \ g \ (\rho_s - \rho) \ d
\]

(7-3)

where \( \rho_s \) is the density of sediment, usually varies between 2.5 to 2.6 g/cm\(^3\) and \( \rho \) is the density of water, usually varies from 1.0 to 1.03 g/cm\(^3\) for estuarial environments. Thus, the critical bed shear stress for a bed with uniform sediment grain of 0.1 mm is about 0.8 dyne/cm\(^2\).

**Armoring Effects and Effective Erosion Rate**

If a nature sediment bed have a uniform grain size distribution, then all the erosion process have been discussed. However, most natural sediment bed do not have a uniform grain
size. Thus, after the available small grains were all suspended from the active layer, i.e., a layer with thickness about the ripple height, erosion ceases even the excess bed shear stress remain the same. This is because the large particles that remained on bed surface formed an armor layer which prevent further erosion.

Mathematically, the armoring effect can be described by using the following function

\[ ARM(t) = e^{-pt} \]  \hspace{1cm} (7-4)

where \( t \) is the elapsed time (in second) start counting when the bed shear stress is larger than the critical bed shear stress, \( p \) is a undetermined constant which may depends on the grain size distribution as well as the excess bed shear stress. A commonly agreeable value is not available.

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**Fig. 7-2. Critical Bed Shear Stress for Non-cohesive Sediment (after Shields, 1936)**

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yet, and further study is needed on this formulation. Nevertheless, the product of $C_a$ and $\text{ARM}(t)$ will produce an effective sediment concentration, $C_e$, at the reference elevation $z_a$.

NEW APPROACH

After solved the $C_e$, the traditional next step would be to properly select a turbulent diffusion coefficient to bring sediment up into water column. Here in this study, $C_e$ was further transfer to “erosion rate” and with other modifications (given next), the computer codes for cohesive sediment transport were used to simply the coding.

Erosion Rate

The option to transfer $C_e$ to erosion rate involves an assumption. This is because $C_e$ is in unit of g/m$^3$, but erosion rate is in g/m$^2$/s. In the formulation of $C_e$, time is never mentioned. Therefore it implies that $C_e$ is for a unit time. Assuming this concentration is good for a layer with thickness of 3 times grain size instead of a point, the erosion rate can be calculated as $3dC_e$. This rate will be used to calculate the transport of non-cohesive sediments. The selection of $3d$ for the layer thickness is arbitrary and subject to change. In practical application, however, it may not make too much difference and can be combined with $\ell_o$ to become a new tunable constant coefficient.

Diffusion Coefficient for Ripple Beds

Although the upward diffusion coefficient, $D_z$, was calculated in the HEM-3D. There is a need to modify it because of the different bed roughness between cohesive sediment and non-
cohesive sediment. Since there is no bed form, \textit{i.e.,} ripples or dunes, at the water-sediment interface for a cohesive sediment bed, and the only roughness is biological roughness. Most of the times, the biological roughness was ignored because of the uncertainty of existence and the relatively insignificance. On the other hand, ripples and dunes for a non-cohesive sediment bed increase the near-bed vertical diffusion coefficient significantly. Nielson (1992) reported that the pick-up capability of vortices generated by wave flows over ripples can trap sediment particles and transport them upward to a distance much higher than the ripple height. For this reason, the near-bed upward transport capability is much stronger for sandy beds at where ripples and dunes are always existed.

Notice that ripples are formed under actions of waves, and dunes are formed under the actions of tidal currents. To accurate estimate the combined effect of waves and currents, one needs to know wave conditions to estimate ripple geometry, and then, the combined nonlinear wave and current effects. Since there is no wave-information for used in the HEM-3D, there is no way to calculate the wave-current boundary layer thickness. In reality, however, it is well known that waves are important at shallow water because even small waves can contribute to the development of ripples, and thus, a strong up-lifting capability.

To address the problem mentioned above without acquiring wave information (because it is another difficult task even with sufficient resources) nor to follow the rigorous approach for calculating wave-current boundary layer thickness and the associated diffusion coefficient modified by the wave-current boundary layer, it is suggested to use the following simple assumption: “The diffusion coefficient is a constant at the bottom 50 cm, and this constant equals the HEM-3D calculated value at the elevation that is around 50 cm from bottom.” By doing so,
there will be enough upward diffusion force to bring non-cohesive sediment (which has a much large settling velocity) into water column. The selection of 50 cm was arbitrary and may subject to change. Unfortunately there was no data to fine tune this selection. Nevertheless, this selection provided reasonable profiles of suspended granular sediment concentration (see Fig. 7-5 given later).

RESULTS

The above formulation was applied to the bed areas that was classified as a sandy bed (Fig. 7-3). The time series of bed shear stress at the shoal cell is given in Fig. 7-4, and details of the vertical suspended sediment concentration profiles at two selected times (as marked in Fig. 7-4) are given in Fig. 7-5. All the parameters used in this simulation are also given in the figures.

Since the non-cohesive sediment has a much large settling velocity, e.g., for the selected 0.1 mm very fine sand, $w_s = 0.0075 \text{ m/s}$. They cannot transport far away from their source. The low concentration of sandy sediment at upper water column from the erosion site also indicates this phenomenon (Fig.7-5). Thus, the concentration of suspended sandy sediment is close to zero at the channel even during the maximum flood (Figs. 7-6 and 7-7). For this reason, there is no contribution from non-cohesive sediment to the TSS measurements obtained from the slack water surveys.
Fig. 7-3. Map to Show the Locations of Suspended Non-cohesive Sediment Concentration Profile.

Fig. 7-4. Record of Bed Shear Stress at the Shoal Cell Marked in Fig. 7-3.
Fig. 7-5. Vertical Profile of Suspended Non-cohesive Sediment. (a) At the maximum Bed shear stress marked in Fig. 7-4; (b) at another marked time is Fig. 7-4.
Fig. 7-6. Calculated Suspended Non-cohesive Sediment Concentration when Maximum Flood Occurred at Gloucester Point during Dry Season. (a) at middle level; (b) at bottom level.
In conclusions, the selected approach works fine and the only issue remained is a field verification which includes a fine turning of the resuspension coefficient, $\omega$, and a possible choice of other bottom boundary layer thickness for the rough ripple or dune bed. Because of the lack of data in this study, these two tasks were left for future studies.

*Fig. 7-7. Time Series of Suspended Non-cohesive Sediment Concentration at Bottom Layers of Marked Cell in Fig. 7-3. (a) channel; (b) Shoal.*
CHAPTER 8. DISCUSSION AND CONCLUSIONS

Several important issues (numerical stability, residual current, sediment availability, etc.) that observed during this exercise are discussed as follows.

NUMERICAL STABILITY

The inclusion of sediment transport simulation degraded the numerical stability significantly. For simulating tidal hydraulic and salinity intrusion with the current fine grid size, a time interval of 30 seconds (\( t = 30s \)) was used and it required about 6 hours to simulate a period of one-month using our high-end personal computer. In the HEM-3D, the codes for simulating salinity and cohesive sediments were very similar: The advective transport of salt and suspended sediment were all handled by the same subroutine CALCONC.F. This subroutine called CALTRAN.F to calculate vertical distribution of suspended sediment or salinity. For calculating salinity distribution, that was the end. When calculating suspended sediment, CALCONC.F also called CALSED.F to estimate the loss of suspended sediment due to settling and the gain of suspended sediment due to erosion/suspension. When including the simulation of suspended sediment, we have had a small time interval, \( t = 5 \) s, in order to have a stable run. This is an indication that suspended sediment stratification caused the instability problem. It is important to know how the numerical stability degraded and how to improve it. Significant computing time can be saved (change from 35 hours back to about 6 hours for an one-month simulation) if there is a practical solution, and that is another topic to work on in the near future.
RESIDUAL CURRENT

When examining the calculated TSS distribution with the selected settling velocity, erosion rates, and diffusion coefficient, it seems that there are always two areas with high TSS concentration for the currently low freshwater discharge condition. One of these two areas was always at the downstream side of West Point. The other was at the upstream side of West Point. To further study what is the possible reason(s), we checked the residual current caused by M₂ tide. Since M₂ tide has 89% of the total tidal energy for the York River system, this residual current is sufficient to reveal the possible reason(s) on what was observed from the model results.

What we did was to simulate M₂ tidal circulation with salinity for a period of 60 tidal cycles. During the entire period of simulation, the upstream side freshwater discharge and the downstream side salinity vertical profile and tidal range were all maintained the same. We then averaged the calculated tidal current in the last tidal cycle to obtain the residual currents.

At Gloucester Point the relatively fresher water (in red) flowed toward downstream, and the more salty water flowed toward upstream direction at the bottom (Fig 8-1). Notice that the flow pattern was not symmetric because of the bending at Gloucester Point. Thus, the eccentrical force must played a role at this location. This phenomenon may worth for further study in the future.

Similarly, the intrusion of salinity at Clay Bank, where there are two channels, was also clearly observed (Fig. 8-2). Residual current in the deep channel had a stronger velocity that moving toward upstream direction. In the shallow channel, the upstream-going residual current was weak. At surface the residual current was mainly going toward the downstream direction.
Fig. 8-1. Residual Current Caused by M₂ Tide at Gloucester Point (11.98 km from mouth). Green is flowing upstream and red is downstream.

Fig. 8-2. Residual Current Caused by M₂ Tide at Clay Bank Site (26.61 km from Mouth). Green is flowing upstream and red is downstream.
The residual current for the bottom cells near West Point (Figs. 8-3) indicates that there are cross sections (Fig. 8-4 and 8_5) that could block the upstream-moving residual current (caused by $M_2$ tide) for this low freshwater discharge conditions. This information may explain why the downstream high turbidity plume was hard to move further upstream.

Fig. 8-3. Residual Current at bottom cells near West Point.
Fig. 8-4. Residual Current at Cross-section (a) 143; (b) 146.

Fig. 8-5. Residual Current at Cross-section 161.
SEDIMENT AVAILABILITY

In our approach for erosion and deposition formulation, we assumed that downward settling occurs all the time, but erosion occurs only when the tidal flow is in an acceleration stages, i.e., the bed shear stress at next time step is larger than the bed shear stress at the current stage. This assumption seems ignoring the possible problem of sediment availability, but actually it is not. This is because between two tidal acceleration phases, there is a sufficient time (2 to 3 hours) for suspended sediment to deposit on the sea floor, see an example of the time interval between to two red-dashed lines in Fig. 6-2.

When considering the residual current for a fixed upstream and downstream boundary conditions and simulated for a long enough time, the available sediment may be depleted. But this condition is not a realistic one because in reality it is impossible to have a constant boundary conditions.

METHOD TO COMPARE RESULTS

One slack water survey took about one day to finish and we usually started at the York River mouth when it was a slack tide there, either before flood or before ebb. After getting the survey data, we moved toward a upstream station. The pace of our movement usually matched with the tidal propagation, so our measurements were done always near slack tide at all the survey stations. Sometimes it was impossible to catch with the tide, and some degrees of mismatch is possible. Nevertheless, the survey results are not “snap shots” of estuarial processes. We may construct contours of salinity, TSS, (e.g., all contour plots in Chapter 2) but they are not exactly “snap shots.”
The model calculated results (*i.e.*, water level, current velocity, salinity, TSS, etc.), however, are saved instantaneously for the entire York River system. They are “snap shots.” For this reason, model output has to go through a post process to find the calculated results with their times match with the times of survey at each station. The results of this post process were used to construct contour plots, and then, compared with the survey results. All of the contour plots generated from model results were produced by this manner.

This approach is more accurate when compared with other approach that average the results over one tidal cycle. The second approach actually smooths the output and sometime is hard to compare with the measurements because they represent two different conditions. The development of this post processing process is already a step toward better understanding a model’s capability, and thus, worth to mark it in the report.

**INTERACTION BETWEEN COHESIVE SEDIMENTS**

So far it is still not realistic to include an aggregation/flocculation sub-module in the cohesive sediment transport module. There are studies on this kind of modeling (*e.g.*, Chisholm, 1999; McAnally, 1999). However, there is no any data to prove any of the assumption or results. For this reason, it is not recommended to include an aggregation or flocculation module in the codes.

Current available field instruments for measuring sediment floc size distribution are not reliable or sufficient yet. For example, using the principal of laser light diffraction, an instruments called LISST-100 (developed by Sequoia Scientific, Inc.) has been available on the market for about 10 years. This instrument claimed that it can identify grain size from 1.25-250
microns (Type-B) or 2.5-500 microns (Type-C). An early version on this instrument, however, failed a rigorous test conducted by USGS for cohesive sediments. It was told that a significant improvement has been made after that, but there is no second, independent test to confirm that statement yet.

Using an underwater film camera with two strobes for light, Knowles and Wells (1998) have developed a system to take pictures for identifying floc size distribution. The minimum size of floc or particle that can be identified is about 20 microns, which is marginal for study cohesive sediment. An ideal minimum size should be around 2 microns. Our preliminary study indicates that it would not be difficult to extend the capability of their device to identify floc with a size of about 5 microns.

Combine these two instruments (LISST-100 and underwater photo system) together for field measurements on floc size distributions would be the ideal approach. It is highly recommended that a study on this subject should be supported in the near future.

CONCLUSIONS

The followings are summarized of what have been done and what are the major findings during this study:

1. Totally eight field surveys on the salinity and Total Suspended Sediment (TSS) profiles along the York River were successfully conducted during two one-month periods (Nov. - Dec. 2001 and March - April, 2002). The survey results indicate an extremely dry season during that periods. For this reason, the survey results are very close.
2. The survey results show that the estuarial turbidity maximum (ETM) was abnormally located
about 30 km upstream from West Point. For a normal hydrological year, the ETM was usually located on the downstream side of West Point. A Secondary Turbidity Maximum (STM) was also identified in the York River with a much weak signal.

3. Because of the abnormal location of ETM, the bathymetric grid we originally proposed for use was not good because of the low resolution. Several important channels that may affect salinity intrusion as well as TSS distribution were missing. For this reason, we need a new bathymetric grid to improve the accuracy of describing channels.

4. A new curve-linear orthogonal bathymetric grid for the York River system was developed. This new grid clearly shows all the navigation channels. Because of this new grid, we have to recheck the hydrodynamic and salinity intrusion. As expected, this new grid provides a much better performance in term of hydrodynamic and salinity intrusion.

5. One Owen tube was successfully manufactured after a series of field tests and modifications. The final series of experiments on the settling velocity for the York River sediment indicates that the settling velocity does affected by the concentration of TSS.

6. Numerical experiments using HEM-3D with the freedom to change settling velocity confirmed that turbulence and salinity also have a significant contribution on the ultimate settling velocity.

7. The observed wave heights and TSS data at a shallow water site (i.e., VIMS' Ferry Pier) do not provide evidence for a strong correlation between these two parameters. However, it is not sufficient to prove that there is no correlation neither because the data only come from one observation station. Much observation stations are needed.

8. Using a much simple model for erosion and deposition, i.e., erosion occurs only when the tidal
flow is in acceleration phases, and during this period of time, a constant erosion rate
which was selected based on the local maximum bed shear stress, were successfully used
to simulate the turbidity maximums in the York River.

9. The HEM-3D required a significant computing resources, our high-end personal computer
needed about 6 hour to run for an one-month simulation of tidal hydrodynamics and
salinity intrusion. But it required about 35 hours to simulate cohesive sediment transport
for the same time span because of a numerical stability problem. Further code
parallelization is the only possible solution for addressing this time consuming problem.

10. Although flocculation process can be modeled, there is no data to assess the performance of
any available model yet. In other words, the advance of flocculation modeling is still far
behind when compared with other modeling. Collection of field data is the most
important step to improve the flocculation modeling.

11. Although the numerical simulation results are not perfect yet, it is reasonable to use only one
size class for the entire cohesive sediment. There is neither no data, nor no significant
benefit to justify the use of multiple size classes on cohesive sediment.

12. A rather simple approach to simulate non-cohesive sediment transport was selected. The
commonly used approach by estimating $C_a$ (i.e., $(a)$ was not changed. The difference are
(a) converting $C_a$ information to an effective erosion rate, and (b) modifying the vertical
diffusion coefficient for the near bed zone (i.e., 50 cm above bed) to include the effect of
ripples and dunes. The rest computer codes are the same as those for simulating cohesive
sediment transport.

13. The contribution from sandy sediment to the measured TSS along the channel is negligible.
This is because (a) the area of sandy sediment bed is small and (2) the large settling velocity of sandy sediment practically limited the transport of granular sediment.

14. The study on residual current caused by M_2 tide reveals that there is an area near West Point that is difficult for the near-bed sediment-laden water produced at the middle section of York River to move further upstream. For further study this phenomenon, a bathymetrical grid with high resolution is need.

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Appendix I. Calibration Data from the Eight Slack-water Surveys.

Because of the different data storage methods (i.e., stored in 12 bits or 16 bits), Figs. I-6 and I-8 have a much large raw count for OBS, see Chapter II for details.

The survey results indicate that at the downstream end (i.e., the York River Mouth), the Total Suspended Sediment (TSS) values near the water surface were close to that at the bottom for most of the time (Figs. I-9 and I-10). Only occasionally, the TSS values had a small difference between the surface and bottom at the downstream end (Fig. I-11). This implies that the sediment source was not coming from the Chesapeake Bay.

Fig. I-9 shows a clearly secondary turbidity maximum at 30 km from the York River mouth. But even at this condition, the surface TSS readings were still low, less than 40 mg/L.
Fig. I-1. OBS Calibration for the 11/29/2001 Slack Water Survey. Number represents the Sample station, B and S stand for a near bottom or a near surface sample.

Fig. I-2. OBS Calibration for the 12/05/2001 Slack Water Survey. Number represents the Sample station, B and S stand for a near bottom or a near surface sample, 19B and 20B are saturated, and thus, excluded for the calibration purpose.
Fig. I-3. OBS Calibration Data for the Slack Water Survey on Dec. 10, 2001.

Fig. I-4. OBS Calibration Data for the Slack Water Survey on Jan. 18, 2002.
Fig. I-5. OBS Calibration Data for the Slack Water Survey on March 19, 2002

Fig. I-6. OBS Calibration Data for the Slack Water Survey on March 25, 2002.
Fig. I-7. OBS Calibration Data for the Slack Water Survey on April 02, 2002.

Fig. I-8. OBS Calibration Data for the Slack Water Survey on April 11, 2002.
Fig. I-9. Comparison of Bottom and Surface TSS Concentration for the 12/05/2001 Survey.

Fig. I-10. Comparison of Bottom and Surface TSS Concentration for the 12/10/2001 Survey.
Fig. I-11. Comparison of Bottom and Surface TSS Concentration for the 4/11/2002 Survey.
Appendix II. OBS Profiles for the Nov. 29, 2001 Slack Water Survey

The spikes-removed OBS measurements and the smoothed OBS profiles for the slack water survey carried out on Nov. 29, 2001 are displayed to show the pattern of possible OBS and converted Total Suspended Solid (TSS) profiles. The objective is to indicate the difficulty to obtain an excellent simulation on what was measured. This is because, sometimes, the measurements are just hard to explain with a limited input from upstream and downstream conditions.

Fig. II-1. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR01; (b) YR02; and (c) YR03.
Fig. II-2. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR04; and (b) YR05.

Fig. II-3. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR06; (b) YR07; and (c) YR08.
Fig. II-4. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR09; (b) YR10; and (c) YR11.

Fig. II-5. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR12; (b) YR13; and (c) YR14.
Fig. II-6. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR15; (b) YR16; and (c) YR17.

Fig. II-7. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR18; (b) YR19; and (c) YR20.
Fig. II-8. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR21; (b) YR22.

Fig. II-9. Smoothed OBS Raw count for the 11/29/01 Slack Water Survey. (a) YR23; (b) YR24.
Appendix III. Required Input Files for HEM-3D and Source Codes for Sediment Transport

A: Input Files

This appendix contains all the input files required for executing the HEM-3D for this project. Here in this appendix only the purposes of each input files are explained. The content of each file are not displayed as text in this report, but included in the attached CD. Some of the files are not needed by HEM-3D for calibration or verification purposes. If there is no specific notification about an input file, then it is always required for HEM-3D simulation.

In the attached CD, there are one directory named EPA and two subdirectories: (1) HEM3d_codes and (2) Input_files. Under “Input_files”, there are three lower level subdirectories: (a) Calibration, (b) Dry, and (3) Wet. In each of these lower level directory, there are input files for the cases we simulated.

Please notice that the HEM-3D codes will not work for the input files provided in the Calibration case. This is because the later revised master input file EFDC.INP needs to read two card 29 (29a and 29b) for sediment information. But the early EFDC.INP, which is used for the Calibration cases, there is only one card 29 (because the early work do not have sediment involved). Uses need to add another card 29b to it in order to run the newly improved HEM-3D.

1. Aser.inp - time series of atmospheric input, especially wind speed and direction. In this project wind effect is ignored, and therefore, wind speeds were all set = 0.
2. Cell.inp - specifies the horizontal grid of cells which is defined by a cell type array.
3. Celllt.inp - is used for saving mean mass transport and is identical to the file cell.inp in this project.
4. Dxdy.inp - specifies horizontal grid spacing, depth, bottom elevation, bottom roughness and vegetation classes for either Cartesian or curvilinear-orthogonal grid.
5. *Efdc.par* - contains a parameter statement specifying the dimensions of arrayed variables.

6. *Efdc.com* - contains common block declarations for the hundreds of arrays required by HEM-3D.

7. *Efdc.inp* - The main input file for HEM-3D, *efdc.inp*, provides control parameters, output control, and physical information describing the model domain and external forcing functions.

8. *Gwater.inp* - specifies the characteristic of a simple soil moisture model. Not activated in this project.

9. *Lxly.inp* - specifies both the horizontal cell center coordinates and the cell orientations (Cartesian or curvilinear).

10. *Mask.inp* - is used to insert thin barriers, which block flow across specified cell faces, but there is no barrier in current model domain.

11. *Modchan.inp* - is used to activate and specify data for a subgrid scale channel model, but there is no subgrid in current model domain.

12. *Moddxdy.inp* - allows for quick modification of cell sizes, specified as dx and dy in the *dxdy.inp* file. It is needed for a quick adjustment of subgrid channel sections lengths and widths. This input file contains no data because there is no subgrid in current model domain.

13. *Pser.inp* - is used to specify surface elevation time series at open boundaries. It is used for salinity verification and sediment transport simulations.

14. *Qser.inp* - set time series of fresh water discharge data. It is also used for salinity verification and sediment transport simulations.

15. *Restart.inp* - is used to specify initial conditions for running the HEM-3D model in the restart mode. This file is obtained by renaming the *restart.out* file. This input file could be used in many simulations to save the warm-up time.
16. *Salt.inp* - is used to initialize the model domain with a pre-determined salinity field and it is required for salinity verification and sediment transport simulations.

17. *Sdser.inp* - specifies time series of cohesive sediment concentrations at the boundary cells. It is required only for simulate cohesive sediment transport.

18. *Snser.inp* - specifies time series of non-cohesive sediment concentrations at the boundary cells. It is required only for simulate non-cohesive sediment transport.

19. *Sser.inp* - specifies time series of salinity at the open boundary cells. It is used for simulating salinity and sediment transport.

20. *Taub_coef.inp* - specifies coefficients for resuspension rate for cohesive sediment transport and defines cohesive or non-cohesive cells. This one is required only for Simulating sediment transport.
SUBROUTINE CALSED_COHE(ISTL,CORDT)
C
C ** LAST MODIFIED BY JOHN HAMRICK ON 8 SEPTEMBER 1995
C ** MODIFIED BY J. KWON JUNE 2003
C**********************************************************************C
C
C ** SUBROUTINE CALSED_COHE CALCULATES BOTTOM AND INTERNAL
C ** COHESIVE SEDIMENT SOURCES AND SINKS (ORIGINAL FORMUALITON)
C**********************************************************************C
C
INCLUDE 'efdc.par'
INCLUDE 'efdc.com'
DIMENSION CTMPDRY(LCM)
DOUBLEPRECISION TAUB_COEF2
C
DELT=DT2
S3TL=1.0
S2TL=0.0
ISUD=1
IF (ISTL.NE.3) THEN
DELT=DT
S3TL=0.0
S2TL=1.0
ISUD=0
END IF
C
C**********************************************************************C
C
C INITIALIZE ARRAYS
C
DO L=2,LA
SEDF(L,KC)=0.
CTMPDRY(L)=1.
IF(ISCDRY(L).NE.0) CTMPDRY(L)=0.
END DO
C
C**********************************************************************C
C
C CALCULATE DOWN FLUXES
C
IF(SEXP.EQ.0.) THEN
IF (KC.GT.1) THEN
DO K=0,KS
DO L=2,LA
SEDF(L,K)=-WSEDO*SED(L,K+1)
END DO
END IF
ELSE
    DO L=2,LA
        SEDF(L,0)=-WSEDO*SED(L,1)
    END DO
END IF
C ELSE
C IF (KC.GT.1) THEN
    DO K=0,KS
        DO L=2,LA
            C WSEDO AND SEXP ARE INPUT PARAMETERS IN EFDC.INP ON CARD 29A.
            C SETTLING VELOCITY, Ws = 3.5*10^-5*C^0.375: BASED ON LAB. EXPERIMENT
            C USING OWEN TUBE WITH THE SEDIMENT AT CLAY BANK IN THE YORK
            C RIVER, VA.
            C
            WS_TMP(L,K+1) = WSEDO*(SED(L,K+1)**SEXP)
            SEDF(L,K)     = -WS_TMP(L,K+1)*SED(L,K+1)
        END DO
    END DO
END IF
C END IF
C**********************************************************************C
C**********************************************************************C
C CALCULATE RESUSPENSION
C
C NEW EROSION/DEPOSITION SCHEME IS APPLIED BASED ON MAA AND KIM(2002).
C EROSION OCCURS ONLY WHEN TIDE IS ACCELERATING, AND EROSION IS
C ALWAYS CLOSE TO THE EQUILIBRIUM.
C THUS, A CONSTANT EROSION RATE IS USED.
C FOR YORK RIVER, EROSION RATE RANGES FROM 0.006 ~ 2.4 g cm^{-2} s^{-1}
C EROSION RATE (WRSPO) IS INPUT PARAMETER IN EFDC.INP ON CARD 29A.
C
    DO L = 2, LA
        TAUB_OLD    = TAUB_NEW(L)
        TAUB_NEW(L) = QQ(L,0)/CTURB2
        D_TAUB  = TAUB_NEW(L) - TAUB_OLD
        TAUB_COEF2 = TAUB_COEF(L)*TAUB_COEF2
    END DO
C WRSPO(EROSION RATE) MODIFIED BY COHC_COEF(LCM)
C: RESUSPENSION OCCURS ONLY AT COHESIVE CELLS
C COHE_COEF(L) HAS 1 OR 0: 1 AT COHESIVE CELLS
     : 0 AT NON-COHESIVE CELLS
C
WRSP_TMP(L) = WRSP*COHE_COEF(L)
C
IF (D_TAUB.GT.0) THEN ! WHEN TIDE IS IN ACCELERATE PHASE
C MODIFY EROSION RATE(WRSP_TMP) WITH TAUB_COEF2.
C THERE ARE 8 REGIONS FROM THE BOTTOM SHEAR DISTRIBUTION AND EACH
C CELL HAS CORRESPONDING COEFFICIENT (1/8, 2/8,....,1) THAT NAMED
C TAUB_COEF(L) AND HERE SQUARE OF TAUB_COEF, TAUB_COEF2(L) IS APPLIED.
C
    WRSP = WRSP_TMP(L)*TAUB_COEF2
    WRSP_TMP(L) = WRSP
ELSE
    WRSP = 0. ! NO EROSION UNLESS TIDE IS IN ACCELERATE PHASE
    WRSP_TMP(L) = 0.
ENDIF
C
NOW ADDS UP RESUSPENSION INTO FLUX TERM AT THE BOTTOM CELLS
C
SEDF(L,0)=SEDF(L,0)+ WRSP
C
END DO
C**********************************************************************C
C SEDIMENT AND BED ELEVATION INTERCHANGE
C
IF (ISTL.EQ.3.AND.SDBLV.NE.0.) THEN
    DO L=2,LA
        BELV(L)=BELV(L)-CTMPDRY(L)*CORDT*DT*SDBLV*SEDF(L,0)
    END DO
END IF
C
NOW UPDATE COHESIVE SEDIMENT CONCENTRATIONS
C
IF(KC.GT.1) THEN
    DO K=1,KC
        DO L=2,LA
            SED(L,K)=SED(L,K)
            + CTMPDRY(L)*CORDT*DELT*(SEDF(L,K-1)-SEDF(L,K))/(HP(L)*DZC(K))
        END DO
    END DO
ELSE
    DO L=2,LA
        SED(L,1)=SED(L,1)+CTMPDRY(L)*CORDT*DELT*SEDF(L,0)/HP(L)
    END DO
END IF
C
RETURN
END
C. Sediment Transport Modules for Non-cohesive Sediment Transport (CALSED_NONC.FOR)

SUBROUTINE CALSED_NONC(ISTL,CORDT)
C
C ** LAST MODIFIED BY JOHN HAMRICK ON 8 SEPTEMBER 1995
C ** MODIFIED BY J. KWON JUNE 2003
C**********************************************************************C
C
C ** SUBROUTINE CALSED_NONC CALCULATES BOTTOM AND INTERNAL
C ** NON-COHESIVE SEDIMENT SOURCES AND SINKS (ORIGINAL
C ** FORMUALITON)
C
C**********************************************************************C
C
INCLUDE 'efdc.par'
INCLUDE 'efdc.com'
DIMENSION CTMPDRY(LCM)
C
RHO = 1025. ! WATER DENSITY IN KG/M3
DELT=DT2
S3TL=1.0
S2TL=0.0
ISUD=1
IF (ISTL.NE.3) THEN
DELT=DT
S3TL=0.0
S2TL=1.0
ISUD=0
END IF
C
C INITIALIZE ARRAYS
C
DO L=2,LA
SEDF(L,KC)=0.
CTMPDRY(L)=1.
IF(ISCDRY(L).NE.0) CTMPDRY(L)=0.
END DO
C
C**********************************************************************C
C CALCULATE DOWN FLUXES
C
C
C A CONSTANT SETTLING VELOCITY(WSEDO,INPUT PARAMETER IN EFDC.INP ON
C CARD 29B) WAS CALCULATED USING EQ. FROM CHANG AND LIOU(2001)
C
IF (KC.GT.1) THEN
DO K=0,KS
DO L=2,LA
SEDF(L,K) = -WSEDO*SND(L,K+1)
END DO
END IF
DNF_NONC(L,K) = SEDF(L,K)
END DO
END DO
ELSE
DO L=2,LA
SEDF(L,0)= -WSEDO*SND(L,1)
DNF_NONC(L,K) = SEDF(L,K)
END DO
END IF
C**********************************************************************
C**********************************************************************
C CALCULATE RESUSPENSION
C CALCULATE RESUSPENSION AT THE BOTTOM CELLS AND ADDS IT INTO FLUX
C TERM AT THE BOTTOM CELLS
C : TRADITIONAL EROSION SCHEME IS USED -> EROSION OCCURES ONLY
C WHEN BOTTOM SHEAR STRESS IS LARGER THAN CRITICAL SHEAR STRESS FOR
C EROSION
C
DO L=2,LA
TAUB = QQ(L,0)/CTURB2         ! SHEAR VELOCITY SQUIRE (M2/S2)
TAUB = TAUB*RHO                  ! CONVERT UNIT TO SHEAR STRESS (N/M2)
TAUB_NEW(L) = TAUB

TAUE=(TAUB-TAUR)/TAUN    ! EXCESS SHEAR STRESS (NON-DIMENSION)
TMP_TAUE(L) = TAUE
TAUE=MAX(TAUE,0.)               ! EROSION OCCURES ONLY
       ! WHEN TAUB > CRITICAL TAUB FOR EROSION
C
SEXP(GAMMA0) MODIFIED BY COEF_NONC(LCM) :
C SEXP IS 0.002 IN USED HERE.
C COEF_NONC(L) HAS 1 OR 0 : 1 AT NON-COHESIVE CELLS
C : 0 AT COHESIVE CELLS
C THEREFORE EROSION OCCURES ONLY AT NON-COHESIVE CELLS
C
SEXP_TMP(L) = SEXP*COEF_NONC(L)

C CALCULATE Ca(REFERENCE CONC. AT Za) AND CONVERT A RESUSPENSION
C RATE(WRSP) : IT IS ASSUMED THAT Za IS 3 TIMES OF GRAIN SIZE
C SEDN : Cb, SEDIMENT CONC. OF THE SURFICIAL SEDIMENT (IN HERE 1.7E+6
mg/L)
C Ca IS ACTUALLY INSTANEOUS CONC. THEREFORE IT IMPLIES SEDIMENT CONC.
C PER UNIT TIME --> ITS UNIT IS mg/L/S
C
Ca          = SEXP_TMP(L)*SEDN*TAUE/(1.+SEXP_TMP(L)*TAUE)          ! Ca (mg/L/S)
TMP_CREF(L) = Ca
GRAIN_SIZE  = 0.1E-3                                                                  ! GRAIN SIZE IS 0.1 mm
Za          = 3 * GRAIN_SIZE

WRSP        = Ca*Za                                ! Convert  Ca(mg/L/S) to erosion rate
WRSP_TMP(L) = WRSP
C NOW ADDS UP RESUSPENSION INTO FLUX TERM AT THE BOTTOM CELLS
C
  SEDF(L,0) = SEDF(L,0) + WRSP
ENDDO

C**********************************************************************C
C
C SEDIMENT AND BED ELEVATION INTERCHANGE
C
IF (ISTL.EQ.3.AND.SDBLV.NE.0.) THEN
  DO L=2,LA
    BELV(L)=BELV(L)-CTMPDRY(L)*CORDT*DT*SDBLV*SEDF(L,0)
  END DO
END IF

C

C UPDATES NON-COHESIVE SEDIMENT CONC.
C
IF(KC.GT.1) THEN
  DO K=1,KC
    DO L=2,LA
      SND(L,K)=SND(L,K)
      + CTMPDRY(L)*CORDT*DELT*(SEDF(L,K-1)-SEDF(L,K))/(HP(L)*DZC(K))
    END DO
  END DO
ELSE
  DO L=2,LA
    SND(L,1)=SND(L,1)+CTMPDRY(L)*CORDT*DELT*SEDF(L,0)/HP(L)
  END DO
ENDIF

C**********************************************************************C
C
RETURN
END