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Operation Procedures of Mid-Barataria Sediment Diversion Model to Reproduce Hydraulics and Sediment Transport

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OPERATION PROCEDURES OF MID-BARATARIA SEDIMENT DIVERSION MODEL TO REPRODUCE HYDRAULICS AND SEDIMENT TRANSPORT

A Thesis

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College
in partial fulfillment of the requirements for the degree of Master of Science
in
The Department of Civil and Environmental Engineering

by
Chase Alexander Holston
B.S., Louisiana State University, 2017
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Abstract

The Lower Mississippi River Physical Model (LMRPM) is a distorted, movable bed model that simulates the hydraulics and sediment transport in the lower 195 miles of the Mississippi River. Carved into the model are both existing hydraulic structures, such as the Bonnet Carré Spillway, and proposed structures, such as the Mid-Barataria Sediment Diversion (MBSD). The MBSD will be the first sediment diversion built as part of Louisiana’s Coastal Master Plan, so incorporating the diversion in the LMRPM will be a useful tool to understanding how the diversion can impact the river hydraulics and bedload transport. The objective of this thesis was to perform preliminary tests on the model MBSD, which is incorporated as a sluice gate and conveyance channel, to understand the relationships controlling diverted discharge at different river discharges and future sea level conditions. The results of these preliminary tests were used to create operating procedures to model the expected performance of the prototype diversion. These operating procedures were tested using historical hydrographs. The results of these tests showed that the operating procedures were able to achieve the target diverted discharges well, though improvements can be made to reduce the amount of error. The diverted discharge sediment fraction ($\delta$) had values greater than or equal to 1, as well as greater variance in values, for higher river discharges. Detailed bed level measurements at the diversion did not show the expected short-term impact of erosion upstream and deposition downstream. Finally, dye studies showed that the presence of the operating diversion appears to cause significant increases in turbulence in the model river, especially for higher discharges.
Chapter 1. Introduction

Coastal Louisiana is home to estuaries that provide important resources to the state (CPRA “Louisiana’s Comprehensive Master Plan”, 2017). Land loss in coastal Louisiana is a well-documented issue that has been investigated for decades (Morgan & Larimore, 1957; Britsch & Dunbar, 1993; Couvillion et al., 2017). This land loss is due to several factors, including natural subsidence of degrading deltas and eustatic sea level rise ("relative sea level rise" when considered together), marsh degradation due to factors like saltwater intrusion and poor water quality, and reduction in the sediment supply from the Mississippi River for building and maintaining land (CPRA “Louisiana’s Comprehensive Master Plan”, 2017). The presence of dredged canals in wetlands has also been linked to land loss (Bass & Turner, 1997). Today, there is a large effort to address land loss and other coastal issues in Louisiana using many different project types; this effort is described in detail in the state’s Coastal Master Plan (CPRA “Louisiana’s Comprehensive Master Plan”, 2017). One of the most sustainable solutions in this plan is river sediment diversions.

1.1. The Lower Mississippi River and Coastal Louisiana

The Mississippi River is an influential part of Louisiana. The water that flows through the Bird’s Foot Delta in the Gulf of Mexico is sourced from a watershed that covers 41% of the lower 48 states of the U.S. Coastal Louisiana was built by several historical delta complexes that would deliver sediment and build land across the state (Roberts, 1997). Today, the river is unable to provide sediment to coastal wetlands to the same degree that it did historically. The first reason is that the river has been contained by levees to help control navigation conditions and to provide flood protection to local communities and industries. The second is that, due to engineering projects upstream (e.g., dams), the river does not carry as much sediment as it once did. The estimated total suspended load of the Mississippi and Atchafalaya rivers together is 205 MT per year, which is about half of the amount that was available before these projects were implemented (Blum & Roberts, 2009).

Since losing the Mississippi River as a sediment source, large areas of coastal wetlands have been unable to maintain or increase elevation quickly enough to keep up with subsidence and eustatic sea level rise. Since 1932, it is estimated that Louisiana has lost about 1,900 mi² of coastal land (Couvillion et al., 2017). These wetlands are important to coastal Louisiana. Large areas of wetlands have the potential to reduce storm surge (Wamsley et al., 2009; Barbier et al., 2013). Having a healthy coastal ecosystem is vital for Louisiana for what it provides for the fishing and seafood industry, the oil and gas industry, and navigation and transportation of goods up and down the river (CPRA “Louisiana’s Comprehensive Master Plan”, 2017). As subsidence, eustatic sea level rise and saltwater intrusion continue to threaten large areas of wetlands, it will be important to combat the loss of these areas with projects such as sediment diversions.

1.2. Sediment Diversions and the Lower Mississippi River Physical Model

There has been great interest in studying sediment diversions in the Lower Mississippi River. For proposed structures, like the Mid-Barataria Sediment Diversion (MBSD), where no real-world data is available, it is important to have many modeling tools to understand the impact and behavior of the structure. Although the study of real structures such as the Bonnet Carré...
Spillway and Old River Control Structure can influence decisions when designing sediment diversions (Brown et al., 2013), in some respects these structures are quite different from sediment diversions. Computer models have been an important part of understanding and designing sediment diversions (Meselhe et al. 2012; Viparelli et al., 2015; Gaweesh & Meselhe, 2016). It was modeling results that helped inform the Louisiana Coastal Protection and Restoration Authority (CPRA)’s decision to send the MBSD and Mid-Breton sediment diversion projects to the engineering and design phase (CPRA “Louisiana’s Comprehensive Master Plan” 2017).

Physical models are another tool for studying diversions. Physical and numerical model have different strengths and weaknesses, so a deeper understanding of a system can be achieved by combining them in what is called composite modeling (Sutherland & Barfuss, 2011). In 2018, the Lower Mississippi River Physical Model (LMRPM) became another modeling tool to study sediment transport and hydraulics. This model has several sediment diversions, including the MBSD, routed into the model. The LMRPM can model many different scenarios by adjusting the river flows, sea level conditions, the number of operating diversions, and the capacity of each diversion. There are also other physical models of the MBSD, such as those at Alden Labs, which use different scalings and domains and are focused on answering different questions about the diversion.

1.3. Objective

The overall objective of this thesis research was to create operating procedures for the LMRPM’s MBSD so that the model diversion captured the desired diverted discharges over a range of river and receiving basin conditions. In order to accomplish this objective, a significant amount of data was collected that was useful for not just developing the procedures but for improved understanding of river and diversion hydraulics (stage and discharge) and bedload (sand) transport with and without diversion operation.

Preliminary tests were performed to determine the relationships that govern the diversion (sluice) gate opening. These preliminary tests included: flume test to determine a way to use surface particle tracking to estimate the cross-sectional average velocity; sensor location test to find the optimal location for measuring the conveyance channel water levels; gate test to relate the gate opening height to diverted discharge as a function of river stage; backwater effects test to examine what, if any, impact higher water levels in the diversion receiving area would have on diverted discharges; and sediment concentration test to measure the amounts of diverted sediment. The results of the preliminary tests informed development of diversion gate operating procedures. The operating procedures were tested using historical hydrographs, and an initial analysis of short-term impacts of operating the diversion was done.
Chapter 2. Sediment Diversions

The purpose of sediment diversions in the Coastal Master Plan is to capture sediment and freshwater from the Mississippi River and deliver it to coastal wetlands and basins (CPRA “Louisiana’s Comprehensive Master Plan”, 2017). The sediment will help wetlands build and maintain land. The location of the diversions, the designs of the structures, and the characteristics of the diversion receiving area are taken into consideration to maximize the benefits to the target areas. The 2017 Coastal Master Plan has ten sediment diversion projects planned for the first implementation period (CPRA “Appendix A”, 2017).

The MBSD will be the first sediment diversion built as part of the state’s Coastal Master Plan. The MBSD will be located on the west bank of the Mississippi River at approximately river mile 60.7 (Figure 2.1.). The diversion will capture sediment from the river and transport it over 2 miles to be deposited in wetlands in the Mid-Barataria Basin, north of Barataria Bay. The current plan is to divert a range of discharges, with a maximum of 75,000 cfs. While the goal of the structure is to deliver sediment, there are concerns that there may be negative effects that arise from operating it as well, such as changing river hydraulics impacting navigation; shoaling downstream of the diversion; increased flooding in communities near the diversion; and reduced water quality and wetland health (USACE “Mid-Barataria Sediment Diversion Project”, 2021). The impact of the diversion on the receiving areas will be investigated and addressed through the Environmental Impact Statement (USACE “Environmental Impact Statement”, 2021).

![Project Location](image)

**Figure 2.1.** Project Location for the MBSD (CPRA “Mississippi River”, 2021).

2.1. Overview of Sediment Diversions

While the prototype MBSD is currently designed to control discharge with a set of 4 radial gates (CPRA “Mid-Barataria Sediment Diversion Program”, 2019), the model diversion controls diverted discharge with a single submerged sluice gate, which has a width of 16 mm
model and a maximum opening height of 22.5 mm model (Figure 2.2.). The diversion gate allows for control of the diverted discharge over the entire range of river discharges and stages.

![Figure 2.2. LMRPM MBSD Gate Dimensions.](image)

Figure 2.2. LMRPM MBSD Gate Dimensions. The gate is 16 mm model wide, with a maximum gate height of 22.5 mm model.

A semi-empirical equation for discharge through a submerged sluice gate is given by Lozano et al. (2009):

\[
q = C_c C_vf C_{va} d_{gate} \sqrt{2 g \Delta h} = C_d d_{gate} \sqrt{2 g \Delta h}
\]

(2.1.)

where
\( q \) = discharge per unit gate width,
\( C_c \) = contraction coefficient,
\( C_vf \) = dimensionless velocity distribution and friction losses coefficient,
\( C_{va} \) = dimensionless approach velocity head coefficient,
\( d_{gate} \) = height of diversion gate,
\( g \) = gravitational acceleration,
\( \Delta h \) = head differential between water levels upstream and downstream of the gate,
and \( C_d \) = effective discharge coefficient.

For head differentials smaller than 6 mm, \( C_c \) may approach unity (USBR, 1997), so the equation may not be valid for the LMRPM.
The amount of water that flows through a diversion can be considered as a fraction of the river discharge (Letter et al., 2008):

\[ Q_{div} = \beta Q_{river} \]

(2.2.)

where

- \( Q_{div} \) = diverted discharge,
- \( \beta \) = diverted discharge fraction,
- and \( Q_{river} \) = river discharge upstream of the diversion.

Per guidance from the CPRA, the target operation for the model MBSD (Figure 2.3.) is \( Q_{div} = 35,000 \) cfs at \( Q_{river} = 450,000 \) cfs, increasing to a maximum \( Q_{div} = 75,000 \) at \( Q_{river} = 1,250,000 \) cfs, so the \( \beta \) for the MBSD will range between 0.060 – 0.078.

Figure 2.3. Target Operation of MBSD. Target refers to the desired diverted discharge for the given river discharge.

The transport of sediment in the river increases as stream power increases. Stream power (Bagnold, 1966) can be represented as:

\[ \Omega = \gamma Q_{river} S \]

(2.3.)

where

- \( \Omega \) = cross-sectional total stream power,
- \( \gamma \) = unit weight of the water,
- and \( S \) = energy slope.
The sediment transport and distribution in the river will determine how much sediment is available to be diverted.

Sediment transport in rivers is driven by water discharges (Letter et al., 2008):

\[ Q_{se} = Q_{river} C_e \text{ and } C_e \approx A Q_{river}^{\alpha} \text{ then } Q_{se} \approx A Q_{river}^{(1+\alpha)} \]  

(2.4.)

where

\( Q_{se} \) = equilibrium sediment flux (total sediment load),
\( C_e \) = equilibrium mean sediment concentration upstream of the diversion,
A = empirical water discharge coefficient,
and \( \alpha \) = empirical coefficient on the concentration dependence.

The amount of mobile sediment will determine how much sediment the diversion can capture. The diverted sediment concentration, \( C_{div} \), can be represented as (Letter et al., 2008):

\[ C_{div} = \delta C_e \]  

(2.5.)

where

\( \delta \) = diverted discharge sediment fraction
and \( C_e \) = equilibrium mean sediment concentration upstream of the diversion.

The conversion of sediment load to sediment concentration can be done using the first part of Equation 2.4.

Because the goal of sediment diversions is to capture sediment from the river, it is desirable to have \( \delta > 1 \). Modeling efforts and field measurements have suggested that \( \delta \) for the MBSD can be increased by: building it in the presence of a lateral bar; increasing the depth of water captured; having a large and straight diversion channel; and increasing \( \beta \) up to 0.1 – 0.2 (Meselhe et al., 2012; Gaweesh & Meselhe, 2016).

A river’s equilibrium conditions for surface elevation slope, bed slope, and sediment concentration will be changed when a sediment diversion begins to operate. The expected short-term impact to the bed during diversion operation is erosion upstream and deposition downstream of the diversion (Figure 2.4.).

![Figure 2.4. Short Term Effects on River Morphology Due to the Presence of Sediment Diversion (Brown et al., 2013). Upstream of the diversion there is increased slope of the water and erosion of the bed, and downstream there is deposition.](image)
Chapter 3. Physical Modeling and the LMRPM

The LMRPM is located at the LSU Center for River Studies under a collaborative agreement with the CPRA. The model covers 14,000 square miles of southeastern Louisiana, including the 195 miles of the Mississippi River from Donaldsonville to the Bird’s Foot Delta and the nearby Gulf of Mexico. The model is used to study the hydraulics and sediment transport of historical hydrographs of the river, as well as future scenarios involving changes such as sea level rise and the introduction of sediment diversions.

The LMRPM is a mobile bed model, which is preferred over rigid bed when modeling sediment transport conditions such as those found in the Lower Mississippi River. This model combines the rational and empirical methodologies for modeling river hydraulics. The LMRPM meets the rational method’s criteria for similitude for the Froude number, critical Reynolds particle number, and critical shields parameter; and it meets the empirical method’s criteria replicating sediment transport while relaxing other similitude criteria (Warnock, 1950).

The LMRPM is run using the system design platform LabVIEW. Acoustic sensors record water levels in the river and the Gulf of Mexico. Other measurements, like recording bed level, are taken manually between tests. Return pumps control the sea level on the model based on water level readings of sensors in the Gulf of Mexico.

3.1. Geometric Scaling

The geometric scaling of the LMRPM was chosen in order to cover a large domain and to have flow conditions that model sediment transport in the river. The horizontal scale ($x_r$ and $y_r$, the ratios of prototype to model values for the downstream and lateral dimensions, respectively) is 6,000 and the vertical scale $z_r = 400$. This geometric scaling results in a distortion $D = \frac{y_r}{z_r}$ of 15, which is higher than the suggested range of 5 - 10 for the rational methodology for modeling river hydraulics (Julien, 2002; Chanson, 2004; Shen, 2012). A vertical distortion is common for river models, allowing for larger domains to be modeled in reasonably sized facilities (Hughes & Pizzo, 2003). However, distortion can cause scale effects that impact the flow characteristics.

3.2. Dynamic Scaling

Dynamic scaling refers to the ratio of mass-related parameters such as density and specific weight (Ettema et al., 2000). The Froude and Reynolds number criterion are important for determining the dynamic scaling of a model, and can be respectively presented as:

$$Fr = \frac{U}{\sqrt{gL}}$$

(3.1.)

and

$$Re = \frac{UL}{v}$$

(3.2.)
where
- \( U = \) cross-sectional average velocity,
- \( L = \) a reference length (i.e. depth, \( d \)),
- \( \nu = \) kinematic viscosity.

The LMRPM was designed to achieve Froude similitude, while relaxing the Reynolds number similitude (Green, 2014).

### 3.3. Kinematic Scaling

Kinematic scaling refers to ratios of vectorial motions such as velocity and acceleration. From the geometric and Froude scaling, the velocity ratio and water discharge ratio for the model are given respectively by:

\[
V_r = \sqrt{z_r}
\]  
(3.3.)

and

\[
Q_r = z_r^{3/2} x_r
\]  
(3.4.)

The LMRPM has a velocity ratio \( V_r = 20 \) and a discharge ratio \( Q_r = 48,000,000. \) While the average cross-sectional velocity is properly scaled at 20, there will be differences in the detailed horizontal and vertical velocities and accelerations (Hughes & Pizzo, 2003).

### 3.4. Time Scaling

While the hydraulic time scale can be calculated from Froude similitude, the LMRPM was designed to be operated at the sediment time scale. The hydraulic time scale ratio is given by:

\[
T_r = \frac{x_r}{\sqrt{z_r}}
\]  
(3.5.)

and is equal to 300. The sediment time scale ratio, \( T_{st} \), was determined through empirical calibrations (Hooper, 2019) to be 6600.

### 3.5. Model Sediment Scaling

The model sediment is ground unexpanded polystyrene that has a specific gravity of 1.05 g/cm\(^3\). The LMRPM model sediment was designed based on similitude of the critical particle Reynolds number and critical Shields parameter. The sediment size ratio, \( d_r \), is given by (BCG Engineering & Consulting, Inc., 2015):
\[ d_r = (S - 1)^{1/3} = 3.2 \] (3.6.)

The LMRPM sediment size distribution is a scaled representation of the prototype river size distribution for sand sized particles (Table 3.1.).

Table 3.1. Design Characteristic Sediment Sizes for Both Prototype and Model (BCG Engineering & Consulting, Inc., 2015).

<table>
<thead>
<tr>
<th>Type</th>
<th>(D_{10}) (mm)</th>
<th>(D_{50}) (mm)</th>
<th>(D_{90}) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prototype</td>
<td>0.08</td>
<td>0.12 – 0.14</td>
<td>0.25</td>
</tr>
<tr>
<td>Model</td>
<td>0.25</td>
<td>0.40 – 0.45</td>
<td>0.8</td>
</tr>
</tbody>
</table>

3.6. Model Sediment Discharge

The sediment discharge scaling for the LMRPM is represented by:

\[
(q_{bv})_r = \frac{x_r z_r}{T_{sr}(C_{bv})_r}
\] (3.7.)

and

\[
(Q_{bv})_r = (q_{bv})_r x_r
\] (3.8.)

where

- \((q_{bv})_r\) = volumetric transport rate per unit width scale,
- \((C_{bv})_r\) = volume concentration of sediment in the bed scale,
- \((Q_{bv})_r\) = total volumetric transport rate scale.

For the LMRPM, \((C_{bv})_r\), \((q_{bv})_r\), and \((Q_{bv})_r\) were calculated to be 1.33, 273, and 1,640,000, respectively.

The sediment discharge for the LMRPM was based on the analysis by Thomas (2014) of a HEC-6T sediment transport model. The HEC-6T model covered the river from Tarbert Landing to the mouth of Southwest Pass (18 river miles below Head of Passes). The model results for sediment concentration of very fine, fine, medium, and coarse sand at river mile 76 (Belle Chasse) were verified by comparing them to USGS suspended sediment concentrations at Belle Chasse. Polynomial regression equations for suspended sediment concentration based on river discharge were created for each of the grain sizes. The LMRPM sediment input was determined by summing all the regression equations and scaling the results using Equations 3.7 and 3.8.

It was determined the saturated model sediment volume of 1 mL has an approximate mass of 0.982 g. The equations for converting saturated sediment mass to dry prototype sediment mass are:
\[ m_{dry} = 0.534m_{sat} \]  
(3.9.)

and

\[ (m_{dry})_r = x_r y_r z_r \rho_r \]  
(3.10.)

where
\[ m_{dry} = \text{dry sediment mass}, \]
\[ m_{sat} = \text{saturated sediment mass}, \]
\[ (m_{dry})_r = \text{dry sediment mass scale} \]
\[ \text{and } \rho_r = \text{sediment density scale}. \]

For the LMRPM, \( \rho_r \) and \( (m_{dry})_r \) are 2.53 and 3.63E+10, respectively.

3.7. Limitations of the LMRPM

The LMRPM was designed to model the movement of water and sediment in the river. The model was not designed to recreate the hydraulics of wetlands and shallow bays, so the influence of friction is exaggerated for these areas on the model. This means that the transport and deposition of diverted sediment in the MBSD receiving area will not be representative of the prototype behavior. Other models with different scalings will be able to investigate movement of sediment once it leaves the diversion channel.

It should also be noted that the LMRPM does not model the effect of tides, waves, or storm events. During operation of the prototype diversions, conditions in receiving area are a large concern for both the safety of local communities and for the successful building of new land. It will be important to consider how various factors such as tides, wave action, changing vegetation, and weather events such as hurricanes and cold fronts influence the operation of prototype sediment diversions and the distribution of the diverted sediment in the receiving areas (Bevington et al., 2017; Hiatt et al., 2019).

Also, there are different subsidence rates in coastal Louisiana: at the Bird’s Foot, the subsidence rate is 22.3 mm/yr; for the section of the Mississippi River around MBSD, it is 10.7 mm/yr (Demarco et al., 2012). Based on the findings of Olivier (2016), sea level rise on the LMRPM is set to the relative sea level rise at the Bird’s Foot Delta. So, the impact of sea level rise on the model MBSD will be exaggerated because the relative sea level was higher at the diversion than will be expected in prototype.
Chapter 4. Preliminary Testing

The overall purpose of this thesis research was to determine operating procedures for the LMRPM’s MBSD (Figure 4.1) to divert the correct discharges and sediment concentrations that CPRA has chosen for the prototype structure. More specifically, to determine the relationships that represent the function for diverted discharge:

\[ Q_{div} = f(g, h_{river}, d_{gate}, h_{rb}) \]  

(4.1.)

where

- \( h_{river} \) = river stage,
- \( d_{gate} \) = diversion gate height opening,
- \( h_{rb} \) = water level in the receiving basin.

![Figure 4.1. LMRPM MBSD, Downstream of the Mid-Breton Sediment Diversion. The Mississippi River is flowing from the top left corner to the middle right edge. Other diversions were closed off during the testing for this thesis.](image)

The goal was to define the relationship between \( h_{river}, d_{gate}, h_{rb}, \) and \( Q_{div}. \) Diverted discharge can be calculated using:

\[ Q_{div} = V_{avg}A_{cs} \]  

(4.2.)

where

- \( V_{avg} \) = average velocity in diversion channel,
- \( A_{cs} \) = cross-section area.

The river stage, \( h_{river} \), used in this relationship is from the sensor near Alliance, upstream of the diversion at river mile 62.5 on the model. The rating curve at Alliance shows the
relationship between river stage and river discharge. The rating curve was an important relationship for these experiments because the target diverted discharge is based on river discharge, but the elevation difference between the river stage and diversion channel stage will determine the required gate height.

This relationship to determine the required gate opening needed to be understood for model river discharges between 4.2 gpm (450,000 cfs prototype) and 11.7 gpm (1,250,000 cfs prototype). CPRA’s plan for operating the structure will divert 35,000 cfs at 450,000 cfs in the river, and diverted flow will increase with river flow until it diverts a maximum flow of 75,000 cfs at 1,250,000 cfs in the river. It was also important to know the potential impact of backwater effects due to 2 factors: extended periods of operation (if there was an insufficient “draining rate” of the diversion receiving area) and relative sea level rise.

4.1 Methods

In order to determine the relationship in Equation 4.1., 5 preliminary experiments were performed: flume test, sensor location test, gate test, backwater effects test, and sediment concentration test. 5-Year Hydrograph Tests of this thesis covers the sixth experiment, which was running 5 years of historical river hydrographs to determine whether the operating procedures were able to produce the desired diverted discharges. The flume test, sensor location test, backwater effects test, and sediment concentration test were run only once, but with appropriate replicates to determine the statistics. The gate test and 5 years of hydrographs test were run for 3 sea levels: the current sea level of 1.3 ft (SL1), 2.3 ft (SL2), and 3.3 ft (SL3) (all in prototype NAVD88). These sea levels were chosen because they cover the range of sea levels used for the LMRPM 50-year projection experiments.

4.1.1 Flume Test

The diverted discharge can be calculated using the cross-sectional average velocity and area. The small size of the diversion channel limits the ability to directly measure the cross-sectional average velocity. However, the surface velocity can be measured using neutrally buoyant particles (Scott, 2019). Therefore, a series of flume tests were run to determine the relationship:

\[ V_{avg} = f(V_{surf}, h_{div}) \]  

(4.3.)

where

- \( V_{avg} \) = average velocity in the diversion channel,
- \( V_{surf} \) = surface velocity,
- \( h_{div} \) = diversion flow depth.

An elongated channel (i.e., the “flume”) was constructed, composed of the same dimensions (V-shaped channel with 1:1 side slopes) and materials of the LMRPM MBSD diversion channel and placed just downstream of the LMRPM head box (Figure 4.2.). The head box gate was open about 10% and LabVIEW was used to maintain the appropriate diverted flows. To find the relationship between the total discharge before the head box gate and flume
channel discharge, a container and a stopwatch were used to record flume channel discharge values over time.

The experiment covered 5 model diversion discharges between approximately 0.28 gpm model (30,000 cfs prototype) and 0.79 gpm model (85,000 cfs prototype) and 3 tailwater conditions representing different sea levels. This experiment required particle velocity tests. The particles used were 5/32 in. diameter, neutrally buoyant (0.95 g/cm3 density), HDPE plastic balls coated with phosphorescent green paint; these particles were used for quantitative flow visualization tests by Scott (2019). The particles were placed in center of the approximately 4 cm wide channel and the time it took each particle to travel a measured distance was recorded. The time duration and distance traveled was used to calculate the average surface velocity in the center of the channel. Measured water depths were used to calculate the cross-sectional area.

Equation 4.2 was then solved for average velocity, \( V_{avg} = \frac{Q}{A} \). At each combination of discharge and tailwater conditions, average results from particle tests were used to calculate the relationship between surface velocity and cross-sectional averaged velocity.

### 4.1.2. Sensor Location Test

The intake of the model diversion channel has a complex geometry and a gate structure that constricts the flow of water. Therefore, the placement of the water level sensor, required to calculate the diverted discharge cross-sectional area, needs to be in a location that gives representative water elevations that will not be impacted by unusual hydraulic characteristics like hydraulic jumps that may result from the water flowing through the intake geometry and gate structure.
The sensor location test was used to determine the best location along the 18 inches of model diversion channel to place the water level sensor (Parallax PING Ultrasonic Distance Sensor) used to calculate the water depth (and subsequently the flow cross-sectional area). Sensors were placed at 5 in, 9 in, and 13 in model from the diversion gate structure (Figure 4.3.). Hydrographs were run using model discharges of 4.2 gpm, 6.1 gpm, 7.9 gpm, 9.8 gpm, and 11.7 gpm (prototype discharges of 450,000 cfs, 650,000 cfs, 850,000 cfs, 1,050,000 cfs, and 1,250,000 cfs, respectively). The diversion gate was open 75%, which is 1.7 cm model height. The 3 sensors recorded water elevations for each model river discharge. The test was repeated 3 times, and averaged results were used to select the best location for recording representative elevations that allow for the accurate calculation of cross-sectional flow area.

![Sensor Location Test Setup Over the Diversion Channel](image)

Figure 4.3. Sensor Location Test Setup Over the Diversion Channel. Flow through the diversion in this image is from right to left. The sensors are located at 5 in, 9 in, and 13 in from the diversion gate.

### 4.1.3. Gate Test

The gate test was done to determine the diversion gate height, \(d_{\text{gate}}\), required to achieve target diverted discharge for the range of river discharges. This test was necessary to determine the relationship described by (4.1.) A hydrograph with the same 5 discharges used in the sensor location test was used. Gate heights of 22.5 mm, 19 mm, 15.5 mm, and 12 mm model were used and surface particle velocities, \(V_{\text{surf}}\), were measured for each of the 5 discharges. \(V_{\text{avg}}\) was then calculated using the relationship from the flume test. The area, \(A_{cs}\), was calculated from the \(h_{\text{dev}}\), based on the known geometry of the diversion channel lining. \(V_{\text{avg}}\) and \(A_{cs}\) were used to calculate the diverted discharge. The gate test was performed at 3 tailwater conditions: \(SL_1\), \(SL_2\), and \(SL_3\). It was important to perform the test at all 3 sea levels to determine how changing tailwater conditions (i.e., higher receiving basin water surface elevations) impact the diverted discharge.
4.1.4. Backwater Effects Test

The objective of this experiment was to determine the impacts of backwater on the diverted discharges. As discussed earlier, the water levels in the prototype receiving area can be influenced by many factors that are not modeled by the LMRPM. This test was to check whether there were backwater effects from operating the diversion over time due to the “drainage rate” of the receiving area being smaller than the diverted discharge.

This test used the 5 model river discharges from the previous 2 tests. For each of the 5 river discharge conditions, the diverted discharge was maintained for a total 11 minutes, with the diversion gate open to the appropriate amount. 1 minute was allowed for the flow conditions to stabilize. Particle velocity tests were performed at 1 minute, 6 minutes, and 11 minutes, and the diverted discharge was calculated using the previously described methods. The results were used to show whether, for a given river discharge, there was a difference in diversion channel water depth and diverted discharge over time due to backwater effects. If there were backwater effects, the results would be used to determine drainage rates of the model receiving area and adjust operation procedures of the diversion gate.

This test was performed only at the first sea level. If there were backwater effects due to operating the diversion over time, they would initially appear for the first sea level, where the
lower water level would mean friction would have a greater impact on flowing water and reduce the “drainage rate”.

4.1.5. Sediment Concentration Test

The objective of this experiment was to determine diverted sediment concentrations for a range of river discharges and sediment concentrations. Because the purpose of sediment diversions is to capture sand from river flows and deliver it to build land in nearby wetlands, it was important to see how well the model diversion captured sediment. This test used the 5 model river discharges from the previous tests. Flow was maintained at each of the discharges for a total 11 minutes, with the diversion gate open to the appropriate amount to achieve the target \( Q_{\text{div}} \). For each discharge, the average diverted discharge sediment concentration \( C_{\text{sed div avg}} \) was determined by:

\[
C_{\text{sed div avg}} = \frac{\sum V_{\text{div sed}}}{\sum V_{\text{div water}}}
\]

where

\[ V_{\text{div sed}} = \text{volume of diverted sediment.} \]

\[ V_{\text{div water}} = \text{volume of diverted water.} \]

This test was performed at SL_1.

4.2. Results and Discussion

4.2.1. Flume Test Results

The flume test results are presented in Figure 4.5. Each data point shows the measured \( V_{\text{surf}} \), determined from the particle tests, and the corresponding \( V_{\text{avg}} \), determined from the discharge and cross-sectional area. The error bars show the standard deviation from the particle velocity tests. This figure shows that there was a strong linear relationship between the measured \( V_{\text{surf}} \) and the calculated \( V_{\text{avg}} \). Adjusting the tailwater conditions during the testing did not have a noticeable impact on the relationship between \( V_{\text{surf}} \) and \( V_{\text{avg}} \). A linear equation was fit to the datapoints to determine the relationship described in Equation 4.3. and found to be:

\[
V_{\text{surf}} \text{ (ft/s prototype)} = 1.163V_{\text{avg}} + 0.743; \quad R^2 = 0.968
\]

The theoretical linear trendline that goes through the origin is shown in Figure 4.5. as well, although it is unclear whether the relationship between \( V_{\text{surf}} \) and \( V_{\text{avg}} \) would be linear for ranges outside of what was measured. For the purposes of this study, Equation 4.5. was used to calculate average velocity from surface velocity measurements.

Most likely, the surface velocities were larger than the cross-sectional average velocities because the size of the channel and flow depths were very small (about 18 – 21 mm model), so there was greater impact of wall friction losses on the flow velocities. Some previous testing was done with different channel dimensions for the LMRPM MBSD channel lining, and the ratio
$V_{surf}/V_{avg}$ was much greater than it is for the V-shaped channel. The small flow velocities in these previous channel geometries allowed too much sediment to deposit in the diversion channel, which was why the V-shaped channel was chosen.

![Graph of $V_{surf}$ vs. $V_{avg}$](image)

**Figure 4.5.** Flume Test, $V_{surf}$ vs. $V_{avg}$. The 1:1 relationship between $V_{surf}$ and $V_{avg}$, the theoretical linear trendline of the measurements going through the origin, and the linear trendline of the measurements used are graphed.

### 4.2.2. Sensor Location Test Results

The sensor location test results are presented in Figures Figure 4.6. - Figure 4.10. Each data point shows the average $h_{div}$ for the given sensor location and $Q_{river}$, with error bars showing the standard deviation from the elevation measurements. Test A, Test B, and Test C were the 3 replicates of the test. The data from the 3 replicate tests on the plots are slightly offset at each sensor location for presentation purposes: the locations were 5 in, 9 in, and 13 in model from the diversion gate. These results show that the diversion channel flow did not have any unusual hydraulic characteristics. Tests A and C show decreasing $h_{div}$ as distance from the diversion gate increases, especially for higher $Q_{river}$ and between the sensors at 5 in and 9 in. Test B shows flat water levels, or perhaps a small dip at the center location. The differences are very small compared to the magnitude of $h_{div}$ (the range of the vertical axis for Figures 4.6. – 4.10. was selected to show the small differences). Such a small difference in elevation has minimal impact on the cross-sectional area. For example, a model diversion channel water depth of 19 mm ± 0.25 mm converted to prototype values is 24.9 ft ± 4 in. Based on these results, the middle location was chosen as the point to place the sensor.
Figure 4.6. Sensor Location Test, 450,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Data are offset for display purposes; the sensor locations are 5 in, 9 in, and 13 in.

Figure 4.7. Sensor Location Test, 650,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Data are offset for display purposes; the sensor locations are 5 in, 9 in, and 13 in.
Figure 4.8. Sensor Location Test, 850,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Data are offset for display purposes; the sensor locations are 5 in, 9 in, and 13 in.

Figure 4.9. Sensor Location Test, 1,050,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Data are offset for display purposes; the sensor locations are 5 in, 9 in, and 13 in.
Figure 4.10. Sensor Location Test, 1,250,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Data are offset for display purposes; the sensor locations are 5 in, 9 in, and 13 in.

4.2.3. Gate Test Results

The results of the gate tests for the 3 different sea level scenarios and different gate openings are shown in Figures 4.11 to 4.13. Each data point shows an average $Q_{\text{div}}$ calculated from particle velocity measurements that were related to cross-sectional average velocities, and the cross-sectional area, calculated from the water depth and cross-sectional profile. Error bars show the standard deviation of the particle velocity tests. The $h_{\text{river}}$ values were obtained from the LMRPM water level sensor, located at Alliance. Each gate opening scenario shows a strong linear relationship between $Q_{\text{div}}$ and $h_{\text{river}}$. At $Q_{\text{river}} = 450,000$ cfs prototype for SL1 and SL2, the stage in the river was too low to have submerged gate conditions, so those lowest points were not included in calculating the linear equations. For these conditions, the diversion was unable to divert the target $Q_{\text{div}}$ of 35,000 cfs prototype. The results for the gate tests were necessary for determining how to operate the model diversion, but they do not represent expected prototype relationship between gate heights and diverted discharges.

The discharge coefficient $C_d$ (2.1.) was calculated for the results of the gate tests. The values for SL1, SL2, and SL3 were 0.657 ± 0.0476, 0.690 ± 0.0506, and 0.632 ± 0.0365, respectively. These calculated values for $C_d$ are lower than expected values in similar prototype structures. The USACE Hydraulic Design of Reservoir Outlet Works (1980) shows $C_d$ ranging between approximately 0.72 – 0.82. The $C_d$ calculated for this thesis did not have strong correlations with $d_{\text{gate}}$ or $Q_{\text{div}}$, which may have been due to the scaling of the model and the unreliability of this equation for small head differentials (USBR, 1997).
Figure 4.11. SL₁ Gate Test. The results of the gate test for $d_{\text{gate}}$ of 22.5 mm, 19 mm, 15.5 mm, and 12 mm model.

Figure 4.12. SL₂ Gate Test. The results of the gate test for $d_{\text{gate}}$ of 22.5 mm, 19 mm, 15.5 mm, and 12 mm model.
Figure 4.13. SL3 Gate Test. The results of the gate test for d\textsubscript{gate} of 22.5 mm, 19 mm, 15.5 mm, and 12 mm model.

The results for SL\textsubscript{1} and SL\textsubscript{2} are similar. For SL\textsubscript{3}, the results show a shift in the relationships – for a given target diverted discharge, the gate height required for a given river stage condition will be greater at SL\textsubscript{3} that at SL\textsubscript{1} or SL\textsubscript{2}. This is likely due to the higher sea level conditions increasing the flow depth in the diversion channel. This means that the head difference between the river and the diversion is reduced, so a larger gate opening is required to achieve the same target diverted discharge. However, as sea level increased, so did the model stages in the rating curve at Alliance. Karadogan et al. (2009) modeled the impact of sea level rise on stages in the last 100 miles of the Mississippi River. A comparison (Figure 4.14.) of stages from those numerical modeling results and this report’s tests on the LMRPM show that sea level rise correlated with greater stage increases for the LMRPM than for the numerical models. Stages for similar flow and sea level were also higher on the LMRPM than for the numerical model.
The numerical model (Karadogan et al., 2009) looked at sea level conditions that were similar to SL1 and SL2. Stages for the 5-Yr Hydrograph Tests are shown for SL1, SL2, and SL3.

4.2.4. Backwater Effects Test Results

The results of the backwater effects test are shown in Figure 4.15. Each data point shows an average $Q_{\text{div}}$ calculated from a particle velocity test, with error bars showing the standard deviation of the particle velocity tests. The plotted data have slightly offset times for presentation purposes, the measurement times were 1 min, 6 min, and 11 min. The target $Q_{\text{div}}$ is shown by the dotted line. As seen in the gate test, the diverted discharge was less than the target for the lowest $Q_{\text{river}}$ condition of 450,000 cfs prototype (Figure 4.15.).

For the $Q_{\text{river}} = 450,000$ cfs condition (Figure 4.15.), there was a decrease in $Q_{\text{div}}$ between times 1 min and 6 min (between 2,424 – 7,599 cfs prototype), then a much smaller increase in $Q_{\text{div}}$ between times 6 min and 11 min (between 481 – 2,250 cfs prototype). This decrease in $Q_{\text{div}}$ could be due to the small decrease in stage at Alliance of about 0.2 feet prototype for Test A and B, although Test C had a small increase in stage of about 0.2 feet. If this decrease in $Q_{\text{div}}$ for the $Q_{\text{river}} = 450,000$ cfs was due to backwater effects because the drainage rate of the receiving area was less than the $Q_{\text{div}}$, then the tests with higher $Q_{\text{div}}$ should have backwater effects as well. However, these other tests did not show decreasing $Q_{\text{div}}$ over time, so it was determined that there were not backwater effects from operating the diversion.

The backwater effects test was necessary for understanding the model MBSD. The results of this test should not be considered as representing accurate conditions in the prototype MBSD receiving area.
Figure 4.15. Backwater Effects Test, 450,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Target refers to the desired diverted discharge for the given river discharge. Data are offset for display purposes; the times are 1 min, 6 min, and 11 min.

Figure 4.16. Backwater Effects Test, 650,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Target refers to the desired diverted discharge for the given river discharge. Data are offset for display purposes; the times are 1 min, 6 min, and 11 min.
Figure 4.17. Backwater Effects Test, 850,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Target refers to the desired diverted discharge for the given river discharge. Data are offset for display purposes; the times are 1 min, 6 min, and 11 min.

Figure 4.18. Backwater Effects Test, 1,050,000 cfs Prototype. Test A, B, and C are the 3 repetitions of the test. Target refers to the desired diverted discharge for the given river discharge. Data are offset for display purposes; the times are 1 min, 6 min, and 11 min.
4.2.5. Sediment Concentration Test Results

The results of the sediment concentration test are shown in Figure 4.20. Each data point shows the average of the calculated $C_{\text{sed div avg}}$ from the 3 repetitions of the test, with error bars showing the standard deviation from the average sediment concentrations. The dotted line shows the relationship between sediment concentration in the river, $C_{\text{sed HEC-6T}}$, and $Q_{\text{river}}$, which was calculated from the HEC-6T sediment transport model described in Thomas (2014).

As expected, $\delta$ increased as $Q_{\text{river}}$ increased. At $Q_{\text{river}} = 450,000$ cfs prototype, $\delta = 0.11$. When $Q_{\text{river}} \approx 1,050,000$ cfs prototype, $\delta = 1$ and average diverted sediment concentration is 150 mg/L prototype. At $Q_{\text{river}} = 1,250,000$ cfs prototype, $\delta = 1.52$. As $Q_{\text{river}}$ and the corresponding river stages increase, the gate height required to achieve the target $Q_{\text{div}}$ decreases. So the flow approach velocity as well as the velocity through the diversion gate entrance increases as (1) $Q_{\text{div}}$ increases and (2) gate opening area decreases. This increasing approach velocity into the diversion intake may increase erosion of sediment on the bed and in the river in front of the diversion, leading to higher $C_{\text{sed div avg}}$ and $\delta$. Meselhe et al. (2012) also modeled low $\delta$ for low $Q_{\text{river}}$, although the maximum $\delta$ only reached about 1.15.
Figure 4.20. Sediment Concentration Test, $C_{sed \; div \; avg}$ and Target $C_{sed \; HEC-6T}$ from HEC-6T Study (Thomas, 2014).

One equation for $C_{sed \; div \; avg}$ as a function of $Q_{river}$ could not be determined. Therefore, the relationship was broken up into 3 intervals.

If $Q_{river} < 450,000$ cfs prototype, then

$$C_{sed \; div \; avg} \; (mg/L \; prototype) = 0$$

(4.6.)

If $450,000 \leq Q_{river} < 592,000$ cfs prototype, then

$$C_{sed \; div \; avg} \; (mg/L \; prototype) = 26.6 \ln(Q_{river}) - 342.01; \; R^2 = 1$$

(4.7.)

If $Q_{river} \geq 592,000$ cfs prototype, then

$$C_{sed \; div \; avg} \; (mg/L \; prototype) = 5.53^{-28}Q_{river}^{4.89}; \; R^2 = 0.994$$

(4.8.)

During the experiments, it was noticed that as dunes moved past the diversion intake during high discharge conditions, the dune will climb up the side of the model river wall and “feed” the diversion with sediment (Figure 4.21.). While most of the diverted sediment was suspended load, this “feeding” effect likely contributed to variability in $C_{sed \; div \; avg}$ and a $\delta > 1$. 

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Figure 4.21. Dune Feeding the Diversion with Sediment. The river is flowing from left to right and the diversion intake branching down from the river.
Chapter 5. 5-Year Hydrograph Tests

5.1. Experimental Setup

The results of the previous tests were used to create comprehensive operating procedures for SL1, SL2, and SL3 (Figure 5.1.). The operating procedures were created by using the gate test results to calculate the required gate height to achieve the target diverted discharge for a given river discharge and stage at Alliance. These final operating procedures were tested by measuring diverted discharges and average sediment concentrations using 5 real years of river discharge and sediment hydrographs, 1995 – 1999. The 5-year hydrograph test was performed at SL1, SL2, and SL3.

This range of years was chosen because they contain the full range of $Q_{\text{river}}$. Particle velocity tests were done to determine how well the operating procedures were able to reproduce the target diverted discharges during a natural hydrograph. In each year, 4 discharges were selected to measure the diverted discharge using particle tests. The 20 total particle tests were selected to cover a full range of $Q_{\text{div}}$, as well as rising and falling limbs of the river hydrograph.

Scott (2019) used dye as a qualitative tool for visualizing flow conditions and hydrodynamics of the LMRPM. In this work, dye was used in the 5-year hydrograph test to qualitatively show how the diversion captures water from the river and how operating the diversion impacts flow characteristics in the river. 1 discharge for each year was selected to perform dye injection tests in the river at the diversion intake. The 5 total dye tests cover the full range of $Q_{\text{river}}$. After the 5-year hydrograph tests, a separate run was done to record dye tests of the river with the diversion gate closed. The dye tests in this separate run were performed at the 5 river discharges that were used in the preliminary tests.
For each year from 1995 – 1999, expected total yearly diverted sediment volumes (based on the results of the sediment concentration test) were calculated using Equations 4.6. – 4.8. After each year of the experiments, the volume of diverted sediment was recorded and compared to the expected diverted sediment volume for that year.

Sieve analyses of the diverted sediment were done to determine size distribution. This was done to determine what size particles the diversion is capturing and to compare that to the target sediment that is injected into the model. For each year, the diverted sediment that was collected were sieved using No. 20, 30, 45, 50, 80, and 100 sieves. The D_{10}, D_{50}, and D_{90} for was calculated from the sieved sediment. For each year, sieve analysis was done on up to 3 samples, depending on the amount of sediment diverted during that year.

In order to determine the short-term impacts of operating the model MBSD, detailed bed measurements of the mobile riverbed were recorded between river miles 63 and 58 at half-mile increments (Figure 5.2.) following the LMRPM standard operating procedures. At each river mile, a caliper was used to measure the bed elevation of the model sediment at 1 cm (model) increments. The measurements at each river mile were compared to determine bed level changes over the course of the 5-year hydrograph tests.

Figure 5.2. Locations of Detailed Bed Measurements at the MBSD. The river is flowing from left to right and the diversion branching down from the river.

5.2. Results

5.1.1. 5-Year Hydrograph Diverted Discharge Results

The results of the diverted discharge tests are shown in Figure 5.3. - Figure 5.5. Each data point shows a calculated average Q_{div}, with error bars showing the standard deviation from the particle velocity tests. The target diverted discharge is shown by the dotted line. The average absolute differences between the measured Q_{div} and the target Q_{div} were 3,429 ± 2,885, 4,688 ±
2,678, and $5,387 \pm 4,273$ cfs prototype for SL$_1$, SL$_2$, and SL$_3$, respectively. Note that 1 of the planned particle velocity measurements was not done at the peak of SL$_3$ 1997.

Figure 5.3. Diverted Discharge Results for 5-Year Hydrograph Test, SL$_1$. Target refers to the desired diverted discharge for the given river discharge.

Figure 5.4. Diverted Discharge Results for 5-Year Hydrograph Test, SL$_2$. Target refers to the desired diverted discharge for the given river discharge.
The largest difference in $Q_{\text{div}}$ was 17,649 cfs prototype more than the target of 48,343 cfs prototype. For the data points that have large differences between measured and target $Q_{\text{div}}$, there was a correlation with differences in the measured stage at the Alliance gage. Figure 5.6. shows the absolute value of the differences in measured and target $Q_{\text{div}}$ and stage for particle tests where the percent error was greater than ±10%. There can be variance in river stages due to the headbox discharge being higher or lower than the target. Hysteresis can also produce differences in stages at the same flowrate, depending on whether it is the rising or falling limb of a hydrograph. Overall, the operating procedures produced acceptable errors in diverted discharge, but the performance would be improved by accounting for the variance in river stage.
5.2.3. Diverted Volumes and Sieve Analysis Results

Figure 5.7. and Figure 5.8. show photographs of the diverted sediment distribution using SL1 years 1995 and 1997, respectively. The results for yearly diverted sediment volumes are shown in Figure 5.9. 5-Year Hydrograph Tests Diverted Sediment Volumes. Each bar shows the total volume of sediment diverted in a year. The solid bar shows the expected annual sediment volume calculated using Equations 4.6. – 4.8. Most of the volumes are close to the expected volume, but for some measurements there were large deviations from the expected volume. The SL2 and SL3 volumes for 1997 were lower than the expected volume, with percentage errors of 26.5% and 38.2%, respectively. The SL1 volumes for 1998 and 1999 were higher than the expected volumes, with percentage errors of 46.7% and 67.4%, respectively. The SL2 volume for 1998 was higher than the expected volume, with a percentage error of 50.0%. The differences in diverted sediment volumes were likely due to variations in the location of dunes and the upstream availability of sediment to be diverted, especially during times with high $Q_{\text{river}}$. 
Figure 5.7. Diverted Sediment After SL\textsubscript{1} 1995.

Figure 5.8. Diverted Sediment After SL\textsubscript{1} 1997.
Figure 5.9. 5-Year Hydrograph Tests Diverted Sediment Volumes. The expected volume was calculated using Equations 4.6. – 4.8. (derived from the sediment concentration test performed at SL1).

Table 5.1. Diverted Sediment $D_{10}$, $D_{50}$, and $D_{90}$ (in mm) for SL1, SL2, and SL3. Average diameter and standard deviation are shown.

<table>
<thead>
<tr>
<th>Test Year</th>
<th>$D_{10}$ (mm)</th>
<th>Stdev</th>
<th>$D_{50}$ (mm)</th>
<th>Stdev</th>
<th>$D_{90}$ (mm)</th>
<th>Stdev</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL1 1995</td>
<td>0.185</td>
<td>0.003</td>
<td>0.365</td>
<td>0.021</td>
<td>0.764</td>
<td>0.011</td>
</tr>
<tr>
<td>SL1 1996</td>
<td>0.183</td>
<td>-</td>
<td>0.344</td>
<td>-</td>
<td>0.669</td>
<td>-</td>
</tr>
<tr>
<td>SL1 1997</td>
<td>0.192</td>
<td>0.004</td>
<td>0.390</td>
<td>0.006</td>
<td>0.753</td>
<td>0.011</td>
</tr>
<tr>
<td>SL1 1998</td>
<td>0.194</td>
<td>0.005</td>
<td>0.416</td>
<td>0.014</td>
<td>0.786</td>
<td>0.012</td>
</tr>
<tr>
<td>SL1 1999</td>
<td>0.212</td>
<td>0.012</td>
<td>0.449</td>
<td>0.029</td>
<td>0.828</td>
<td>0.026</td>
</tr>
<tr>
<td>SL2 1995</td>
<td>0.187</td>
<td>0.002</td>
<td>0.343</td>
<td>0.003</td>
<td>0.661</td>
<td>0.015</td>
</tr>
<tr>
<td>SL2 1996</td>
<td>0.181</td>
<td>-</td>
<td>0.329</td>
<td>-</td>
<td>0.579</td>
<td>-</td>
</tr>
<tr>
<td>SL2 1997</td>
<td>0.186</td>
<td>0.008</td>
<td>0.367</td>
<td>0.021</td>
<td>0.713</td>
<td>0.032</td>
</tr>
<tr>
<td>SL2 1998</td>
<td>0.186</td>
<td>0.008</td>
<td>0.384</td>
<td>0.022</td>
<td>0.741</td>
<td>0.023</td>
</tr>
<tr>
<td>SL2 1999</td>
<td>0.186</td>
<td>0.004</td>
<td>0.366</td>
<td>0.027</td>
<td>0.721</td>
<td>0.027</td>
</tr>
<tr>
<td>SL3 1995</td>
<td>0.176</td>
<td>0.009</td>
<td>0.339</td>
<td>0.011</td>
<td>0.706</td>
<td>0.023</td>
</tr>
<tr>
<td>SL3 1996</td>
<td>0.191</td>
<td>0.005</td>
<td>0.379</td>
<td>0.016</td>
<td>0.756</td>
<td>0.025</td>
</tr>
<tr>
<td>SL3 1997</td>
<td>0.189</td>
<td>0.002</td>
<td>0.350</td>
<td>0.011</td>
<td>0.731</td>
<td>0.016</td>
</tr>
<tr>
<td>SL3 1998</td>
<td>0.178</td>
<td>0.005</td>
<td>0.358</td>
<td>0.007</td>
<td>0.713</td>
<td>0.017</td>
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<tr>
<td>SL3 1999</td>
<td>0.180</td>
<td>0.003</td>
<td>0.348</td>
<td>0.009</td>
<td>0.745</td>
<td>0.012</td>
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</table>

Sieve analyses of the diverted sediment samples shows that the $D_{10}$, $D_{50}$, and $D_{90}$ increased as the SL1 experiment progressed, but the increases were less than 0.1 mm (Table 5.1.). The sieve analyses for SL2 and SL3 did not show increasing $D_{10}$, $D_{50}$, and $D_{90}$ as seen in SL1.
Overall, the size distributions were similar for the 3 sea level conditions. The $D_{10}$, $D_{50}$, and $D_{90}$ of what is injected (0.25 mm, 0.40 - 0.45 mm, and 0.80 mm, respectively) were larger than what was diverted. The downstream fining of sediment is commonly found in real rivers (Morris & Williams, 1999).

5.2.4. Dye Test Results

For the dye tests with the diversion closed off, there was more turbulence for the high discharge scenario (Figure 5.14.) than the low discharge scenario (Figure 5.10. Dye Tests without Diversion, Low River Discharge (450,000 cfs Prototype).). At this location in the river, the flow features that came off the leeside of the dunes tended to produce more turbulence on the diversion side of the river. Having the diversion open caused more turbulence in river flow near the diversion. For the low discharge scenario with the diversion open (Figure 5.11.), approximately $\frac{1}{5} - \frac{1}{4}$ of the river channel just upstream of the diversion and approximately $\frac{1}{5} - \frac{1}{2}$ of the channel downstream was impacted by increased turbulence. For the medium (Figure 5.13.) and high (Figure 5.15.) discharge scenarios with the diversion open, approximately $\frac{1}{4} - \frac{1}{5}$ of the channel upstream of the diversion and approximately $\frac{1}{2} - \frac{2}{3}$ of the channel downstream was impacted by increased turbulence. The high discharge scenario with the diversion open had the most turbulent river flows. The turbulence was likely due to the diversion intake disrupting the flow in the river. It should be noted that the results of the dye tests showed a 2-dimensional representation of flows near the top of the river, but do not offer a 3-dimensional description of flow characteristics.
Figure 5.10. Dye Tests without Diversion, Low River Discharge (450,000 cfs Prototype).

Figure 5.11. Dye Test with Diversion, Low River Discharge (450,000 cfs Prototype).
Figure 5.12. Dye Test without Diversion, Medium River Discharge (850,000 cfs Prototype).

Figure 5.13. Dye Test with Diversion, Medium River Discharge (850,000 cfs Prototype).
5.2.5. Detailed Bed Measurement Results

The detailed bed measurements showed that there was some erosion upstream of the diversion for the SL₁ (e.g., Figure 5.16.) and SL₂ (e.g., Figure 5.17.) tests, but there was no deposition downstream. There was not a clear pattern of erosion upstream or deposition downstream for the SL₃ tests (e.g., Figure 5.18.). Overall, the detailed bed measurements did not show the expected short-term impact of erosion upstream and deposition downstream of the diversion. Bed level changes near the diversion may have been small because the amount of
water and sediment diverted was a small fraction of the total discharge of water and sediment in the river.

Figure 5.16. SL1 Detailed Bed Measurements at River Mile 62. The solid black line represents the model, and the dotted line represents the target sediment bed level.

Figure 5.17. SL2 Detailed Bed Measurements at River Mile 62. The solid black line represents the model, and the dotted line represents the target sediment bed level.
Figure 5.18. SL3 Detailed Bed Measurements at River Mile 62. The solid black line represents the model, and the dotted line represents the target sediment bed level.

5.3. Comparison

The results of the 5-year hydrograph tests were compared to 2 tests that included runs at similar sea levels, but without the diversion operating. These 2 tests were repetitions of a 50-year future-without-action test (50-yr FWA I and II), which model future flows with the moderate projection for sea level rise. SL1, SL2, and SL3 were compared to 6-year periods in the 50-year FWA tests that had approximately the same sea level conditions. The comparison between the 5-year hydrograph tests and the relevant years from the 50-year FWA tests looked at river stages, bed level measurements, and dredged material. The river stages and bed level measurements were from 6 locations: Reserve, Carrollton, Alliance, Empire, Venice, and Southwest Pass (Figure 5.19).
5.3.1. Stages Comparison

The 5-year hydrograph tests had stages at Reserve that were 0.5 – 2 prototype ft higher than the 50-yr FWA I and II Reserve stages (Figure 5.20.). At Carrollton, the stages for the 5-year hydrograph tests at SL1 and SL3 were 1 – 2 ft higher than the stages for 50-yr FWA I and II during high river discharges, but they were similar for medium to low river discharges. The 5-year hydrograph test at SL2 had similar stages to 50-yr FWA I and II. At Alliance (Figure 5.21.), Empire, Venice (Figure 5.22.), and SWP, there were similar stages for the 5-year hydrograph tests and 50-yr FWA I and II. The most important stage location for this study was Alliance because water elevations at that point in the river determine the required height of the diversion gate. The similarity of stages at locations at the end of the river (Venice and SWP) show that the tailwater conditions were consistent for the three tests that were compared.
Figure 5.20. Reserve Stage Comparison at SL₁ Conditions.

Figure 5.21. Alliance Stage Comparison at SL₂ Conditions.
5.3.2. Regular Bed Measurements Comparison

The bed level conditions for the 5-year hydrograph tests and the 50-yr FWA I and II tests were similar. At a few locations there were differences, but most of the regular bed measurements taken at Reserve, Carrollton, Alliance, Empire, Venice, and SWP were at approximately the same elevations (Figures Figure 5.23 & Figure 5.24.). The LMRPM shows a large variance for some of the bed level measurement locations; the LMRPM range of elevations can exceed 30 ft prototype. This variance is likely due to the movement of dunes from year to year. In the prototype Lower Mississippi River, Nittouer et al. (2008) measured dunes with heights of up to 33 ft during high discharge conditions; medium discharge conditions had dune heights of up to 10 ft.
Figure 5.23. Regular Bed Measurement Comparison, Alliance (River Mile 64) at SL1. The solid black line represents the model, and the dotted line represents the target sediment bed level.

Figure 5.24. Regular Bed Measurement Comparison, Venice (River Mile 7.5) at SL3. The solid black line represents the model, and the dotted line represents the target sediment bed level.
Chapter 6. Conclusions

The results of the preliminary tests showed the relationship between river stage at the diversion, height of the diversion gate opening, water level in the receiving area, and diverted discharge. It was determined that there were no backwater effects from an insufficient “drainage rate” in the LMRPM receiving area. The relationship between river discharge and diverted discharge sediment concentration was measured. The results of the preliminary tests were used to create equations that were incorporated into the diversion operating procedures.

The operating procedures were effective at producing the target diverted discharges. When the measured diverted discharge deviated from the target, it correlated with higher river stages than the target stage based on the rating curves that were used. There was variability in the yearly diverted sediment volume, especially for the high discharge year, 1997. This was likely due to the location of dunes at or just upstream of the diversion, something that will most likely occur in the prototype river. The dye tests showed that having the diversion open and operating created much more turbulence in the river flow than when the diversion was closed off. The detailed bed measurements near the diversion did not show the expected short-term impact from operating the diversion.

The conditions of the 5-year hydrograph tests were similar the 50-yr FWA I and II. The stages upstream of the diversion were higher for the 5-year hydrograph tests, but at and downstream of the diversion the stages were similar. Similar bed levels were found for the 5-year hydrograph tests and the 50-yr FWA I and II.

6.1. Limitations

The limitations of the LMRPM and the findings of this thesis report should be noted once more. The LMRPM was not designed to model wetland hydraulics, so the findings of this report should not be the basis for predicting actual diverted sediment concentrations or diversion receiving area conditions such as water levels or sediment distribution patterns. As mentioned in earlier chapters, water levels and sediment transport in wetlands are complex phenomena influenced by a large number of factors which are not represented well by the design of the LMRPM. Other models, both physical and numerical, will be better equipped to model the receiving area. The measurements taken in the diversion channel (flow depths, velocities, etc.) should not be considered as representative of the expected prototype diversion flow features.

6.2. Recommendations

The results of this thesis report can be used as the basis for determining the operating procedures for other diversions on the LMRPM. Once this is done, the impact of operating more than one diversion can be studied. It may be useful to see how changing the operating procedures for the diversion impact the river. If future tests have a sea level greater than SL3, it will be necessary to do additional gate tests to determine backwater effects from sea levels that were not tested in this work. Determining how to predict hysteresis on the LMRPM will help reduce the error in diverted discharges. For future experiments like the 50-year tests that cover long periods of time, the long-term (decadal) impacts of operating the diversion can be studied. It could be useful to measure suspended sediment concentrations in the river at the diversion, which would allow for calculating diversion efficiency. Placing model structures in the river may be used to
increase the diverted discharge sediment fraction. A quicker method of measuring surface velocity in the diversion channel would be useful, though it seems like most velocimeters are too large to fit in the model diversion channel, or they are very expensive.
References


Vita

Chase “Alex” Holston is from Crowley, Louisiana. He attended Louisiana State University and received his Bachelor’s degree in Environmental Engineering in May 2017. He entered graduate school at LSU and was awarded a graduate assistantship with the LSU Center for River Studies to research hydraulics and sediment transport in the Lower Mississippi River Physical Model under the guidance of Dr. Clinton Willson in August 2018. He anticipates receiving his Master of Science degree in May 2021.