SEDIMENT INVESTIGATION AND STABLE CHANNEL DESIGN FOR THE LOWER MUD RIVER

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Abstract: The U.S. Army Corps of Engineers, Huntington District (COE or the District) is currently developing a feasibility level investigation to determine viable alternatives for local flood protection for the town of Milton and vicinity in Cabell County, West Virginia. The principal flooding source is the Lower Mud River. In the current investigations several alternatives are being evaluated to determine the most cost effective project. These alternatives consist of a levee and channel widening/channel diversion as well as combinations of these options. A detailed stable channel and sediment transport analysis for the project was warranted to determine the aggradation/degradation tendencies that may have adverse impacts on the operational and functionality characteristics of the local protection project. The objectives of this study were to define both the long-term and short-term aggradation/degradation tendencies associated with the project. This information would be used as a basis for determining expected channel maintenance. Part of the scope for this study was to perform initial design of a new stable channel. Design of the new channel using an innovative combination of geomorphic and sediment transport analyses is described in the paper.

INTRODUCTION

The U.S. Army Corps of Engineers, Huntington District (COE or the District) is currently developing a feasibility level investigation to determine local flood protection alternatives for the town of Milton and vicinity in Cabell County, West Virginia. The principal flooding source is the Mud River.

Within the feasibility study conducted by the COE, several alternatives are being evaluated to determine the most cost effective project. These alternatives consist of a levee (with varying levels of protection) and channel widening/channel diversion as well as combinations of these options. One such alternative, consisting of a levee providing a 250-year level of protection and a new river channel, was identified as the preferred alternative (or PA) for purposes of this study.

A detailed sediment transport analysis for the project was warranted to determine the aggradation/degradation tendencies that may have adverse impacts on the operational and functionality characteristics of the local protection project. While hydrologic and hydraulic tasks were performed as part of the study, this paper will focus primarily on the efforts related to stable channel design with mention as well of the sedimentation analyses.

EXISTING DATA

The available data included an existing HEC-RAS hydraulic model of the study area, historic average daily flow data, historic flood hydrographs, current and historic stage-discharge rating curves, limited historic suspended sediment measurements, and historic aerial photography. WEST substantially modified portions of the hydraulic model in order to perform hydraulic and sediment transport simulations for this study. The corrected existing conditions model was used as a basis for the preferred alternative HEC-RAS model as well as both existing condition and preferred alternative HEC-6T sediment transport models (MBH, 2002). Historic stream discharge data were available from the U.S. Geological Survey (USGS) gage "Mud River near Milton, WV," active from 1938 until it was abandoned in 1980. No historical aerial photography was available from the COE. However, digital orthographic quadrangle maps (DOQs) for the project and model areas were available from the West Virginia Department of Environmental Protection. These photos were taken in April 1996 and April 1997. Additional aerial photography from March 1988 was available through the TerraServer website (MSN, no date). The COE provided copies of the Milton and Hurricane USGS 7.5 minute topographic maps. Raster graphic images of these same quadrangles were downloaded to use as background images within ArcView GIS. The GIS (geographic information system) was also used to overlay other information provided by the COE such as 1999 and 2001 elevation contours, levee alignment, and "new" cross section locations.

HYDROLOGY

Hydrologic data included both long-term flow data with a period of record equal to the 50-year design life of the project and a short-term flood hydrograph. The long-term data were used to develop an understanding of the long-term trends in aggradation/degradation that should be expected as a result of the project. The short-term data were used to understand the potential magnitude of aggradation/degradation associated with large flood events and what impact this may have on the hydraulics of the channel during a flood. Frequency flows were developed through statistical analysis, and used for fixed bed calibration of the HEC-6T models.

STABLE CHANNEL DESIGN

The preferred alternative developed by the COE consisted of a levee offering a 250-year level of protection and a reach of new river channel within the project limits. Part of the scope for this study was to perform initial design of the new channel such that, on average, it would neither aggrade nor degrade over time. The following sections describe the qualitative evaluation of the sediment transport characteristics of the river [in this discussion, the terms "bypassed channel" or "bypassed reach" refer to that portion of the river that will be cut off and no longer convey flow; the terms "new channel" or "new reach" refer to the newly constructed channel that will take the place of the bypassed channel].

The design of the new channel is presented here in four steps. The first step is the analysis of overall river stability. This evaluation is based on changes in the historic planform of the river, site observations, and a specific gage analysis for the Mud River gage. The second step is analysis of the current planform and hydraulic characteristics of the bypassed reach. The third step is the calculation of the theoretical planform for a "stable channel," using predictive techniques. The fourth step is the determination of a proposed alignment and cross sectional shape for the new channel.

RIVER STABILITY

Historic Planform: The available historic data showed little change in the planform of the Lower Mud River at Milton. Other than minor changes in vegetation, no significant channel movement or bar development was noted. Comparisons of March 1988 aerial photography with 1999/2001 mapping, and 1972 USGS topographic mapping with 1999/2001 mapping, revealed that the channel banks had no significant movement. Minor differences that were seen could be attributed to the water level at the time of each photograph or drawing. Comparison of this information with conditions observed in the field led to the conclusion that the planform of the Lower Mud River has remained relatively constant over the last 30 years.

Site Observations: An extensive field reconnaissance was performed to encompass both the project area and modeled reaches of the river. In general, the Lower Mud River appeared to be a stable river with little signs of active aggradation or degradation. Bank erosion was almost nonexistent.

Specific Gage Analysis: The USGS gage was located near a bridge about five miles upstream of Milton. The site of the gage was visited during the field reconnaissance and no hydraulic controls were noted at the site. Forty-four rating curves were obtained from the COE which cover the time period 1938 to 1978. Each rating curve was examined to determine the stage over time for a series of given flows from 50 to 10,000 cfs. The results of the specific gage analysis show that the Mud River near Milton was relatively stable between 1937 and 1980 (42 years).

EXISTING PLANFORM AND HYDRAULIC CHARACTERISTICS:

Channel Profile: The HEC-RAS model was used to determine the existing longitudinal slope in the bypassed reach. Water surface elevations at the ends of the bypassed reach were used to calculate a slope through the reach. Because determination of the in-bank channel slope was the objective, the highest discharge at which flow was in-bank at all cross sections was used. Based on model results, this discharge was found to be about 800 cfs. At 800 cfs, the average water surface slope is 0.00044.

Cross Section Geometry: The cross sections in the existing conditions HEC-RAS model from about river mile 20.144 to 19.413 were examined. This region includes nine cross sections. The approximate top of bank, toe of slope, and top of slope were estimated at each cross section. There was no discernable trend in the top width changes, except for a narrowing of the channel near the Fairgrounds Road Bridge. It was also found that the average bottom width is 49.4 feet, and the median bottom width is 41.5 feet. The average side slope for left and right banks is 2.2:1 (horizontal to vertical), and the median side slope is 2.28:1.

Amplitude, Meander Length and Radii of Curvature: These parameters were identified via use of a geographic information system for the existing reach to be bypassed.

STABLE CHANNEL PARAMETERS

The next step was to use predictive methods to calculate stable channel dimensions and expected planform parameters. A comparison of the predicted stable channel parameters with the actual parameters in the bypassed reach provides an indication of whether the bypassed reach is stable.

Predictions of stable dimensions for channel width, depth, and longitudinal slope were generated using two "regime" methods. The first method is Copeland's procedure (Copeland, 1994), which balances inflowing and outflowing sediment transport to predict stable channel dimensions. The second method is Julien and Wargadalam's method (Julien and Wargadalam, 1995), which uses a series of equations to predict channel dimensions. Predictions of meander length and amplitude were made using formulas developed by Leopold and Wolman, which predict these parameters based on channel width (Leopold and Wolman, 1960).

Channel Forming Discharge: The use of regime equations or SAM (USACE, 1998) to predict channel geometry requires specification of a channel-forming discharge. This discharge is a single representative flow that will theoretically produce (for stable rivers in regime) the same bankfull dimensions as the natural sequence of flow events.

Biedenharn et al., (2000) describe three ways to estimate channel-forming discharge. The first estimate is the bankfull discharge, the maximum discharge a stream can convey without overflowing its banks. Dunne recommends a 1.5-year recurrence interval as an estimate of bankfull discharge (Dunne et al., 1978). Another, less common, method to estimate bankfull discharge is through the use of the exceedance percentage. Nixon (1959) found that for rivers in England, the average bankfull discharge was equal to the 0.6% exceedance discharge of the mean daily discharges. When a hydraulic model is available, as is the case for this study, the bankfull discharge can also be estimated by modeling the highest discharge where the water stays in bank at each cross section.

The second estimate of channel-forming discharge described by Biedenharn et al., is based on flood recurrence intervals. The channel-forming discharge is generally between the 1-year and 5-year floods. Unfortunately, it is difficult to estimate precisely from this very wide range of discharges.

The third estimate of channel-forming discharge suggested by Biedenharn et al. is the effective discharge. Effective discharge is that discharge which transports the largest fraction of the average annual bed-material load.

The most practical approach, and the most common practice, is to use bankfull discharge to estimate the channel-forming discharge. That approach was adopted here. For this reach of the Lower Mud River, the 1.5-year flood (as suggested by Dunne to estimate bankfull) was not calculated, but the 1-year flood is 5,481 cfs and the 5-year flood is 11,259 cfs. It was found, however, than even the 1-year flood of 5,481 cfs caused most of the cross sections in the bypassed reach to overtop their banks in the HEC-RAS model. The hydraulic model results indicated that discharges somewhere between 3,500 to 4,500 cfs resulted in a bankfull condition at most cross sections in the bypassed reach. The exceedance percentage method of estimating bankfull showed that the 0.6% exceedance discharge equaled 4,080 cfs. Because both the HEC-RAS model and the exceedance discharge method indicate a lower channel-forming discharge than the 1.5-year flood method, these results were considered more reliable. The 4,080 cfs bankfull discharge obtained from the exceedance method was in the range of the bankfull discharges observed from the HEC-RAS model, and was adopted as the bankfull discharge and the channel-forming discharge for the purposes of this study.

Comparison of Predictions with Existing Conditions:

(1) Copeland Method. The agreement between cross section parameters in the bypassed channel reach and the SAM predictions is excellent. However, this is to be expected. The HEC-6T model was used to generate the sediment transport rates used as an input into SAM. It is therefore not surprising that SAM predicts stable channel parameters which are similar to the actual cross sections found in the bypassed reach. The SAM results are useful, however, for verifying the trapezoidal dimensions and slopes which provide similar sediment transport rates to the natural channel in the bypassed reach.

(2) Julien and Wargadalam's Method. Julien and Wargadalam's formulas provide a more independent prediction of stable channel parameters than the SAM method, since they do not depend on sediment transport rates calculated in HEC-6T. The predicted channel slope of 0.00045 is very close to the average slope of about 0.00044. The Julien and Wargadalam method, however, predicts a somewhat wider and shallower channel than found in the bypassed reach. The actual average depth is about 10 feet versus a predicted 7.2 feet. The actual widths range from below 100 feet to about 145 feet, versus a predicted 187 feet. One possible reason why the existing channel is narrower and deeper than the method's predictions may be the presence of cohesive bank material along much of the Lower Mud River (cohesive material is not considered in the method).

(3) Meander Wavelength and Amplitude. Empirical relations were used to predict the planform variables meander length and amplitude using channel width (w) as the input. The equations used were both from Leopold and Wolman (1960): $8=10.9w^{1.01}$ for meander length and $A=2.7w^{1.1}$ for meander amplitude.

The average existing meander length is 937 feet. The predicted stable meander length of 1234 feet is reasonably close to this value. The meanders in the bypassed reach have amplitudes of 77, 89, 154, 166, 194, and 298 feet. The average of these is 163 feet. The predicted meander amplitude of 466 feet is considerably higher than the existing values.

NEW REACH

Because the bypassed reach is apparently stable, the primary objective of the design of the new reach was to duplicate the parameters in the bypassed reach. For ease of design and analysis, a trapezoidal cross section was used for the new reach. Observation of cross sections from the HEC-RAS model shows that existing sections can also be reasonably approximated by a trapezoid.

The design of the new reach had the primary objectives of preserving the length and slope of the bypassed reach, and preserving the approximate channel geometry of sections in the bypassed reach. Secondary objectives were duplication of meander lengths and meander amplitudes of the existing

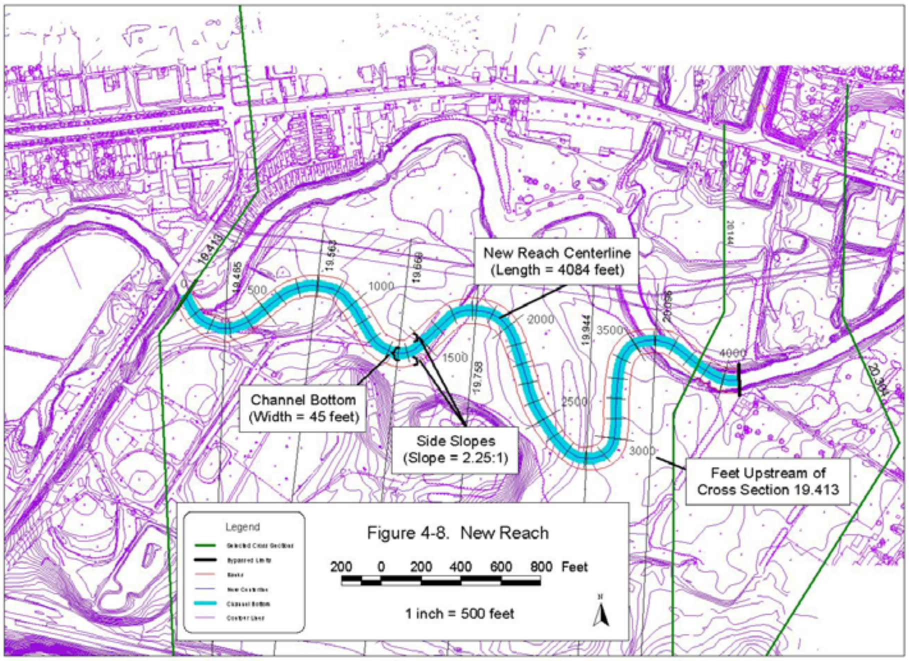

Figure 1 New reach layout.

channel. Another objective was that the meanders should maintain the left-right alternation found in natural channels. This last objective required, for example, that the first meander at the upstream limit of the new reach turn to the right since the previous upstream meander turned to the left.

Alignment: In addition to duplicating the previously measured parameters of the bypassed reach, the vertical position of the new channel needed to be established. The cross section of the existing (bypassed) channel near the upstream limit of the bypassed channel was used to establish the elevation of the new channel. For simplicity, the longitudinal slope of the new reach was set to a constant value, as was the bottom width and sides slopes. This was reasonable, as the existing slope of the bypassed reach is also more or less constant.

The proposed channel centerline is shown in Figure 1. This alignment is based on a 0.00045 longitudinal slope, 45 foot bottom width, 2.25:1 side slopes, and an elevation of 560.00 at cross section 20.144. The bank lines shown are based on where the side slopes would intersect the existing topography, assuming that the bypassed channel is filled in where it does not overlap the new channel. Meander amplitudes, lengths, and radii of curvature were kept within reasonable values as determined from analysis of the existing bypassed channel. Table 1 summarizes how the parameters of the new channel compare to those of the bypassed channel. Generally, the agreement between the new channel parameters and the bypassed channel parameters is good. The depth and width of the new channel depend on the existing terrain; because the terrain elevations were generally higher in the new reach than in the old, the channel tended to be slightly deeper and wider.

SEDIMENT TRANSPORT

Several HEC-6T models were developed to analyze sediment continuity in the Lower Mud River for two scenarios: future without project (FWOP) and future with the preferred alternative (PA).

Models were developed for long-term and single event hydrology, and for sensitivity analyses. Bed material samples were collected at various locations along the Lower Mud River and representative gradations were used in the modeling. From the field reconnaissance and plots of the cross sections, the lateral limits of erosion and deposition were determined at individual cross sections and input to the HEC-6T model. All cross sections were allowed to erode with the exception of those located at two weirs. For inflowing sediment load, the Colby method (Vanoni, 1975) was used to estimate total bed sediment discharge based on the stream measurements of suspended discharge. The values of mean channel velocity, stream width, and mean depth, necessary input for the method, were taken from the HEC-RAS results for the upstream most cross sections.

Table 1 Parameters of bypassed channel versus new channel.

Parameter	**Bypassed Channel ***	**New Channel ****
Channel Length	4,082 feet	4,084 feet
Longitudinal Slope	0.00042 – 0.00046 average across reach	0.00045 constant
Bottom Width	Average about 49 feet Median about 41 feet	45 feet
Side Slope	Average about 2.2:1 Median about 2.28: 1	2.25:1
Width From Top of Left Bank to Top of Right Bank	From about 71 to 141 feet. Average 108 feet	From 96 to 133 feet Average 117 feet
Vertical Position of Channel	Varies	Channel bottom at 560.00 feet at cross section 20.144
Average Depth	From about 6.1 to 11.8 feet Average 10.1 feet	From 4.6 to 12.9 feet*** Average 10.8 feet
Width to Average Depth Ratio at Bankfull	From 7.7:1 to 13.4:1 Average 10.8:1	From 10.2:1 to 15.2:1 Average 10.7:1
Meander Amplitude	From 77 to 298 feet Average 195 feet	From 162 to 356 feet Average 208 feet
Meander Length	From 643 to 1102 feet Average 890 feet	From 643 to 1102 feet Average 903 feet
Radius of Curvature of Bends	From 120 to 500 feet Average 228 feet	From 138 to 260 feet Average 194 feet

* Based on cross sections from the HEC-RAS model.
** Based on cross sections every 10 feet from 10 feet upstream of RM 19.413 to the upstream limit of the new reach.
*** A very low right bank for the first 100 feet upstream of RM 19.413, gives the lowest depths and top widths.

Hydraulic calibration of the sediment models was achieved by running the HEC-6T model in fixed bed mode and making adjustments to model parameters such as roughness values. The calibrated HEC-6T model was used for a long term simulation representative of "future without project" conditions.

Sediment and hydrologic data were added to the geometric data set to create the model. Time steps were adjusted as needed to allow the model to converge to reasonable solutions. The 100-year event hydrograph previously described was also simulated with the calibrated HEC-6T model to gage the response of the system to a single flood event. Because of the great deal of uncertainty in sediment transport modeling, a sensitivity analysis was performed to quantify the affects of several assumptions upon model results. Future without Project (FWOP) results were analyzed for variation of the following parameters: Manning's 'n', inflowing sediment load, and transport equation.

Preferred Alternative: The existing conditions models (HEC-RAS and HEC-6T) were adjusted to reflect the addition of the new channel and levee. Cross sections falling between RM 19.45 and RM 20.2 in the old model were replaced by new cross sections. Water surface and velocity profiles for the calibrated PA HEC-6T model also compared very well with the HEC-RAS model results, except again for expected discrepancies near the bridge locations. Manning's 'n' values were adjusted at these locations to help compensate for this known issue. Differences in channel velocities between the HEC-RAS and HEC-6T model were typically on the order of +/- 5% while water surface elevations were generally within +/- 2%. The 50-year synthetic hydrology record was used with the PA geometry to create the preferred alternative HEC-6T model. Similar to the FWOP results, a general lowering trend is predicted for the bed in the model reach and in the project reach. The PA model was also executed using the 100-year flood hydrograph described previously. The results closely resembled those found for the existing conditions single event model.

CONCLUSIONS

The study revealed that the Mud River in the vicinity of Milton, West Virginia appears to be a stream in quasi-equilibrium based upon historic data, a specific gage analysis and field observations. Numerical simulations of the stream behavior over the next 50 years (FWOP alternative) showed periods of erosion and deposition that varied over time. Overall, results showed that the stream will degrade slightly over much of its length during this period. However, given sensitivity analysis results and the above conclusion, the slight degradational trend may be caused by uncertainties in roughness values and inflowing sediment load and not necessarily by instability in the river system. Considering the long term numerical results in light of historic behavior suggests that the stream will be relatively stable over the next 50 years for the FWOP alternative.

The new channel was designed using geomorphic and engineering methods and is expected to be stable. Because the portion of the existing channel that will be bypassed appears to be stable, the new channel was also designed to match as closely as possible (within project constraints) the hydraulic and geomorphic parameters of the bypassed reach. Sedimentation modeling results support the assertion that the new channel will be stable in that the preferred alternative (PA) results show the same stable to slightly degradational trends as the FWOP results.

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