H-1

MISCELLANEOUS

VARIOUS MODEL STUDIES

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Design and Development of Bendway Weirs for the Dogtooth Bend Reach, Mississippi River

Hydraulic Model Investigation

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Final report

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4 Discussion of Results and Conclusions

Interpretation of Model Results

In any analysis and evaluation of the results of this study, the limitations of the model should be considered based on the model verification, the base test, hydrographs used, and the condition of the model bed at the time that a plan or modification to a plan was installed. Comparison of the final model verification run (verification run 5, Plate 4) with the prototype survey of April 1983 (Plate 3) indicated that the model had a greater tendency to scour at dike 35.0 (R). The navigation channel was wider and shallower from miles 37.5 to 36.3 and miles 23.0 to 22.5, and wider and deeper from miles 32.1 to 31.4. The model exhibited greater deposition than the prototype between miles 31.0 to 30.6, miles 28.8 to 27.5, and miles 24.5 to 23.7, narrowing the navigation channel in these areas.

These tendencies should be considered in the evaluation of the model results. Tests of improvement plans (Plans A through R) should be based only on those changes caused by the plans compared to results reproduced in the model during the base test (Base Test rerun run 5). It should also be considered that the model does not reproduce the movement of material in suspension, and that the bank lines were fixed, with no attempt made to reproduce the degree of erodibility of the banks and sandbars. All dikes and weirs were also fixed with no attempt made to model any structure deterioration or failure. Also to be considered are the average annual and 1983 flood hydrographs used for the testing of plans, which could be considerably different from what actually occurs in the river in the future, and the fact that all model surveys were taken after a low-water period.

All model tests involving weirs were performed in a bendway with the outer bank line of the bend *revetted*. No assumptions or extrapolations based on the results of this model study should be made regarding performance of bendway weirs in an *unrevetted* bend. Information on the design and test results of a very limited series of tests of bendway weirs in an unrevetted bendway can be found in Pokrefke.[1] Also, model tests have not been performed with bendway weirs in a *pool* section of the river; therefore, weir performance under those conditions is yet to be determined.

Summary of Results and Conclusions

The following definitions, results, and general indications were developed from the model study:

a. Analysis of the results of Plans A through F-1 indicated that while many types of river training structures were tested, none were successful in significantly reducing the prototype problems (outlined in the section "Descriptions of Prototype Problems") encountered in this reach of river.

b. Visual observations aided by floating confetti indicated that dikes angled downstream (Plans A through E-4) redirected the surface water currents toward the outer bank of the bend. This would adversely affect navigation, and the increased hydraulic forces on the revetment could threaten the integrity and stability of the bank and would likely result in increased maintenance costs. This increased pressure on the outer bank is of particular concern at Dogtooth Bend where there is already a high natural concentration of currents. These concentrated high-velocity currents were a major factor in the bank failure at Dry Bayou during a high-water event in 1983.

c. Compared to all other plans tested in the model, Plan G-2 was the best at solving the complex multitude of problems associated with this study reach. Results after six runs demonstrated that the bendway weirs of Plan G-2 were the most effective in improving the alignment and widening the navigation channel through the bends and downstream crossings; constructively redistributing flow patterns; depositing significant amounts of sediment on the toe of the outside bank revetment; redistributing velocities in a more uniform manner; and improving the navigation channel in the crossing downstream of the bend.

d. A bendway weir is defined as a rock structure located in the navigation channel of a bend, ideally angled 30 deg upstream of a line drawn perpendicular to the bank line at the bank end of the weir. In cases where the outer bank of the bend does not have a constant radius, one should be calculated and employed when laying out weir position (this allows the bendway weirs within a field to act as a coherent unit). The bendway weir is level-crested at an elevation low enough to allow

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[1] Thomas J. Pokrefke. (1993). "Demonstration Erosion Control Project Monitoring Program, Fiscal Year 1992 Report; Volume VII: Appendix F, Model study of bendway weirs as bank protection," Technical Report HL-93-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

normal river traffic to pass over the weir unimpeded. The bendway weir must be of adequate height and length to intercept a large enough percentage of flow at the river cross section where the weir is located to produce the following six hydraulic improvements: a wider navigation channel through the bend, deposition at the toe of the revetment on the outside of the bend, more uniform flow velocities at any bend cross section, surface water currents that do not concentrate on the outside bank of the bend, an improved navigation channel in the crossing downstream of the bend, and an improved alignment of the navigation channel throughout the bend and downstream crossing.

e. In addition to the hydraulic improvements outlined in the preceding paragraph, the following environmental benefits of properly placed and angled bendway weirs could be realized: water quality should not be adversely affected as the weirs are deeply submerged 100 percent of the time; the stone bendway weir itself creates diversity of habitat within the channel, provides cover and protection for small fishes, and creates a firm, stable substrate to which benthic invertebrates can attach themselves and flourish; and the cross-sectional shape of the bendway channel is changed from a deep triangle to a wider, shallower trapezoid, thus improving and increasing the usable aquatic habitat area (studies have shown that few fish live in the deepest sections of the Mississippi River). The weirs should have a minimal impact on the least tern, a federally protected endangered species of sea bird that inhabits the bendway point bars. Since the weirs will be submerged in the deepest section of the river at all times, the major negative environmental consequence associated with emergent dikes (the gradual conversion of aquatic habitat areas to terrestrial habitat due to deposition within the dike field) will be avoided. Also, the improvements in the navigation channel through the bends and downstream crossings should result in less maintenance dredging, which would result in a decrease in dredged material. With stricter environmental regulations being enacted this is very important now, and will become even more so in the future. In conclusion, since the weirs have less impact on the environment than traditional river training structures, the future could see more widespread use.

f. Model tests analyzing weir angle (Plan G-2, weirs angled 30 deg upstream; Plan N, 45 deg upstream; Plan O, 35 deg upstream; and Plan R, 25 deg upstream) indicated that bendway weirs angled 30 deg upstream were the most effective in solving the navigation problems in this model reach (a detailed comparison of the results of these tests is presented in the section "Conclusions from Tests of Different Bendway Weir Angles"). Results of the 25- and 35-deg angled plans demonstrated that weir angle is an extremely important parameter. With only a 5-deg difference in angle, the results from these tests showed the weirs to be less effective than the 30-deg angled weirs of Plan G-2.

g. While an entire series of tests involving weir heights and lengths was not performed, the poor results of Plan J demonstrated that the length of the weirs, the angle of flow entering the weir field, and the width of water the weir influences relative to the overall width of the river at the location of the weir (percentage of flow captured) are of critical importance. These parameters have not been thoroughly investigated; therefore, the reader is hereby cautioned to determine their effect and importance on any proposed bendway weir design.

h. Various tests were performed with weirs constructed at el -18 and -15. Since during this study weir length was always a function of weir height, a direct comparison involving two identical plans with weirs at different heights was not conducted. However, comparing the best test results employing weirs at el -15 (Plan G-2, run 6) with the best test results involving weirs at el -18 (Plan K, run 1) showed the weirs at el -15 to be much more effective (a detailed comparison is contained in the section "Plan L").

i. Using the basic design parameters of Plan G-2, bendway weirs have been constructed in the prototype (the river) at Dogtooth Bend (1989 and 1990), and Price's Landing Bend (1991). Detailed weir theory, design, construction methods, and prototype performance at the Dogtooth Bend location are contained in the publication "Bendway Weir Design Manual."[1] Furthermore, as stated in the first paragraph of this report, results of this model were analyzed and used as a guide to design bendway weirs for two troublesome reaches of river not included in this study. On the middle Mississippi River in 1992, eight weirs were installed at Cape Rock Bend (mile 54), and nine weirs were installed at Red Rock (mile 94).

j. Response from the towing industry regarding the improvements in navigation due to prototype installation of bendway weirs at these four sites on the middle Mississippi River has been enthusiastic. "This is the best thing to happen on the river in a hundred years," said Andy Cannava of the American Commercial Barge Line, a regular user of the study reach of the river.

[1] Robert D. Davinroy. n.d. "Bendway weir design manual," U.S. Army Engineer District, St. Louis, St. Louis, MO.

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Redeye Crossing Reach, Lower Mississippi River

Report 1
Sediment Investigation

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Report 1 of a series

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4 Analysis of Results and Conclusions

Redeye Crossing

Analysis of Study Results

The sedimentation study plan included use of both numerical and physical models. The numerical models were to be used during the period required for physical model construction and verification to determine an approximate contraction width and provide preliminary guidance on dike spacing and location. This screening process was adopted to minimize the testing program required in the physical movable-bed model. Although adjusted somewhat by changed study requirements in this case (Chapter 1), this basic approach is still considered valid. After initial calculations using the 1-D numerical sedimentation model (TABS-1) to estimate the required contraction and provide boundary conditions, the 2-D numerical modeling system (TABS-2) was used to test a number of alternative dike plans at Redeye Crossing. Eleven different dike plans were tested with the 40-ft navigation channel to provide insight concerning the number of dikes and their crest elevation, location, and length. Four additional dike plans were tested to provide similar information for a 45-ft navigation channel. The optimum plans based on numerical model results, Plan 5AO for the 40-ft channel (Figure 14 and Chapter 2, paragraph entitled "Dike crest elevations and lengths") and Plan 8AO for the 45-ft channel (Figure 17 and Chapter 2, paragraph entitled "Dike crest elevations and lengths"), differed in dike crest elevation with the dikes for the 45-ft channel being 7 ft higher than those for the 40-ft channel and in the length of the three downstream-most left-bank dikes being extended by 200 ft for the 45-ft channel. In the interest of time, all physical movable-bed model tests were conducted with dike Plan 8AO whether the navigation channel was dredged to el -40 or -45 ft.

Although test schedule requirements resulted in physical model tests which were not directly comparable to previous numerical model tests, there are sufficient similarities between 40-ft channel tests in the two models to make some qualitative comparisons. That is not true for the 45-ft channel tests since no base test without dikes was conducted in the physical model. However, for the 40-ft channel, most general tendencies from the two modeling approaches were consistent. Both models indicated that dikes would reduce shoaling in the crossing channel but that dike elevations would have to be substantially higher than those originally proposed by LMN and based on the conveyance procedure developed from prototype experience with dike fields and naturally maintaining crossings in shallow-draft channels. However, numerical model estimates of dike effectiveness were much higher than physical model estimates (discussed below). The numerical model results indicated that dikes proposed on the right descending bank were generally ineffective and, although no specific tests were conducted in the physical model with dikes on the right bank, physical model observations supported the indication that they would be ineffective. Results from both models indicated that the volume of channel shoaling with the dike plans is highly dependent on the flow hydrograph, with the effectiveness decreasing as the magnitude of the flow hydrograph increases. Two significant differences also were evident from analysis of numerical and physical model results: (a) a tendency for the crossing channel to shift toward the dike field in the numerical model results that was not exhibited in physical model results and (b) numerical model estimates of shoaling reduction were substantially higher than estimates based on physical model results. The two situations are discussed in the following paragraphs.

Tendency of channel to rotate toward dike field

The numerical sediment transport computations with the TABS-2 modeling system indicated a tendency for the downstream end of the crossing channel to migrate toward the dike field leading to a rotation of the navigation channel alignment through the crossing. Later tests of dike plans in the physical movable-bed model did not demonstrate that tendency. This leads to the conclusion that the numerical model tendency for the channel to rotate toward the dike field is probably an artifact of its inability to capture the three-dimensional (3-D) character (rotational flow) of flow in the bendway just downstream of Redeye Crossing. The depth-averaged numerical model has a tendency to move the strength of flow to the inside of the bend resulting in poor lateral distribution of flow in the bendway. This known weakness of the depth-averaged model also prevents the model from properly building the point bar which also may contribute to the computed channel migration tendency. These factors lead to the conclusion that installation of the tested dike field should not significantly impact present navigation channel alignment at Redeye Crossing.

Effectiveness of dike plans in reducing channel shoaling

As noted earlier, while the general tendencies of the numerical and physical model results for the 40-ft channel tests were consistent, there were substantial differences in the estimates of shoaling reductions achieved by the tested dike plans. The physical model test results indicated less benefit from dike field installation than the numerical model had indicated. The discussion of these differences should begin with the acknowledgment that both approaches are approximations of a very complex physical process, and their validity is impacted by a variety of factors. The following paragraphs discussing each approach are an attempt to put some of those factors into perspective.

Numerical model (TABS-2). The basic limitation of the 2-D model in not computing the rotational flow in the bendway is equally relevant to the computation of shoaling estimates. The approximation of flow conditions through the upstream portion of the crossing should be realistic with some loss of validity as the bendway is approached, and the inability of the model to properly reproduce the point bar development becomes more important.

Another factor influencing accuracy of predictions is model adjustment and associated data limitations. The Redeye Crossing Reach has a very flat energy slope, and limited data were available for use in hydrodynamic model adjustment. Stage data from the nearest upstream and downstream gauges were used to adjust water-surface profiles in the 1-D model and then the 2-D model was adjusted to stage-discharge relationships developed from the 1-D calculations. Very limited prototype velocity data were available, primarily data collected to help establish the proper flow distribution across Manchac Point during high flows. Overall, hydrodynamic model adjustment was accomplished with much less field data than is typically available for 2-D model verification. Nevertheless, in general, computed stage and velocity information appeared consistent and reasonable.

As is typical of many sediment transport studies, available data for verification were limited. Sediment inflow concentrations for the 2-D study were taken from the 1-D model which was adjusted using measured concentrations at selected Mississippi River gauge locations. Bed sediment data were available from the dredged crossing channel and, from these data, a medium sand (0.25 mm) was selected as the single grain size for transport. Other input data and coefficients were given in Chapter 2 of this report. The model was adjusted for the 43-year-average-annual hydrograph to yield a channel shoaling volume similar to the average maintenance dredging reported in years when peak daily discharge did not exceed 1,000,000 cfs. Time constraints did not permit significant sensitivity tests with higher energy hydrographs during the adjustment process. One problem encountered during the adjustment process was the tendency to erode the point bar downstream of Redeye Crossing. Since the bar was downstream of the study area and time was critical, the bar was hardened (i.e., no erosion allowed). The question remains as to whether this trend was an artifact of the depth-averaged model's inability to correctly compute the lateral flow distribution in the bendway or some other factor.

While it is not possible to quantify the accuracy of the numerical model results, the cumulative effect of these factors is probably optimistic estimates of shoaling reduction from dike plans. As noted earlier, use of the depth-averaged numerical model was proposed as a screening tool to limit the test program in the physical model. Circumstances which developed resulted in its use for developing design refinements in spite of the inherent limitations of the 2-D code in modeling the important 3-D aspects of the Redeye Crossing Reach. Some additional insight concerning dike field effectiveness could probably have been developed through additional sensitivity testing, etc., but a fully 3-D flow/sediment transport code is needed for substantial improvement in numerical model shoaling predictions at Redeye Crossing.

Physical movable-bed model. The ability of a physical movable-bed model to reproduce conditions similar to those that can be expected in the prototype is highly dependent upon the success of model verification efforts. Although rooted in science, physical movable-bed modeling is largely an art due to the inability to scale all pertinent phenomena. It is an art which has been highly developed and successfully applied for study of shallow-draft problem areas over the past 40 to 50 years. However, the Redeye Crossing study is the first WES application of this movable-bed model technology to a deep-draft, open river problem area. Consequently, the verification process was complicated by the necessity to extrapolate techniques and procedures which worked in shallow-draft situations to similar techniques and procedures which would work in the deep-draft environment. (The positive side of that situation is that the flatter portion of that learning curve is now history and future deep draft studies will benefit.)

The Redeye Crossing Reach also presented unique problems. The reach includes a relatively short radius, large curvature bend just downstream of the crossing and available field data indicated the elevation of the point bar did not follow the "normal" tendency of alternating point bars to be very dependent on the flow hydrograph. These factors coupled with the necessity of finding the right distortion for discharge and bed slope to encourage appropriate movement of the model bed material made the verification process difficult and time consuming.

As noted earlier, the urgency in the spring of 1991 for guidance on dike design to help finalize a plan for construction in 1991 led to termination of verification refinements and an abbreviated physical model testing program. As with the numerical model results, it is not possible to quantify the accuracy of the movable-bed physical model results. However, the judgment is that physical model results are qualitative and that the estimates of shoaling reduction from dike plans are conservative. Had circumstances on the physical movable-bed model permitted additional refinement during the verification, then the test results could be extrapolated to a more precise prediction of the plan results on the channel development. In other words, the more refined the verification, the more comfortable one would feel that any particular model plan result would be predicting the effect of that plan in the prototype.

Conclusions

The Redeye Crossing Reach is a very complex physical system with the crossing followed immediately by a relatively short radius, large curvature bend. Field data indicated the added complexity that the elevation of the point bar in this bend does not follow the normal tendency of alternating point bars to be very dependent on the flow hydrograph.

The study plan concept of using both numerical models and a physical movable-bed model was a useful approach. The numerical model was used to screen several dike plans to select the most promising plans for more extensive testing with the numerical model and in the physical model.

For the 40-ft channel tests where test conditions were similar enough for qualitative comparison, most general tendencies from the two modeling approaches were consistent.

Both numerical and physical models documented that the effectiveness of dike plans in reducing channel shoaling is hydrograph dependent with the effectiveness decreasing as the magnitude of the flow hydrograph increases.

Both numerical and physical models indicated that, for the 40-ft project, dikes would reduce shoaling in the crossing channel. The models also indicated that dike crest elevations would have to be substantially higher than those originally proposed based on the conveyance procedure developed from prototype experience with dike fields and naturally maintaining crossings in shallow-draft channels. However, numerical model estimates of dike effectiveness were much higher than physical model estimates.

40-ft channel. Relative to model results of tests for the 40-ft channel, the following conclusions are presented:

a. Numerical model test results indicated Dike Plan 5AO (6 dikes on left-descending bank, crest elevations -5, -5, 0, 0, 0 upstream (U/S) to downstream (D/S)) would reduce channel shoaling by about 90 percent for the 43-yr-average-annual hydrograph and 50 to 60 percent for the 1990 hydrograph.

b. Physical movable-bed model tests results indicated Dike Plan 8AO (6 dikes on left descending bank, crest elevations 2, 2, 7, 7, 7, 7 U/S to D/S) would reduce channel shoaling by about 60 percent for the 43-yr-average-annual hydrograph and about 27 percent for the Sep 82-Aug 83 hydrograph.

c. Both projections are estimates with current judgment that numerical model estimates are optimistic and physical model estimates are conservative.

45-ft channel. Relative to model results of tests for the 45-ft channel, the following conclusions are presented:

a. Numerical model test results indicated Dike Plan 8AO (described above) would reduce channel shoaling by about 90 percent for the 43-year-average-annual hydrograph. No test was run with a higher flow hydrograph.

b. Physical movable-bed model tests were conducted with Dike Plan 8AO but, since no base test was run, a percent reduction in channel shoaling can not be presented. The volume of channel shoaling with Plan 8AO and the Sep 82-Aug 83 hydrograph was much higher than the similar test with the 40-ft channel.

c. Judgment is that numerical model estimates are optimistic and physical model results are conservative.

Installation of the tested dike field should not significantly impact present navigation channel alignment at Redeye Crossing. The numerical model tendency for the downstream end of the crossing to rotate toward the dike field is probably an artifact of its inability to capture the 3-D character (rotational flow) of flow in the bendway just downstream of Redeye Crossing. Physical model test results did not indicate a tendency for the alignment to rotate.

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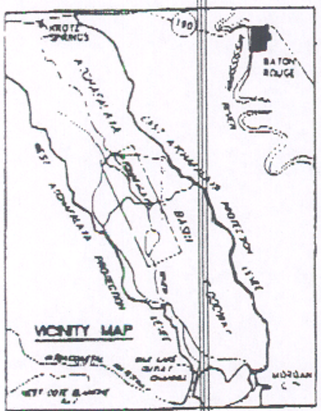

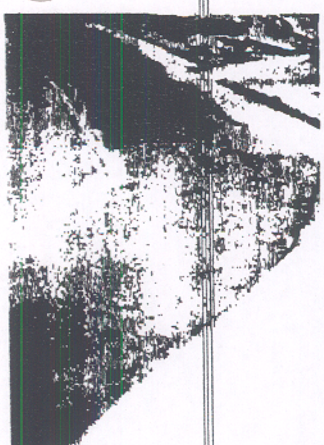

HYDRAULICS LABORATORY

TECHNICAL REPORT HL-89-2

WEST ACCESS CHANNEL REALIGNMENT ATCHAFALAYA RIVER

Hydraulic Model Investigation

by

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West Access

PART IV: DISCUSSION OF RESULTS AND CONCLUSIONS

Interpretation of Model Results

15. The limitations of the model in reproducing all of the factors affecting developments in the reach and the differences between the model and prototype indicated by the results of verification tests must be considered in the evaluation of the model results. Results of tests of improvement plans should be based on only those changes caused by these plans compared with the results reproduced in the model during the verification. It should also be considered that the model does not reproduce the movement of material in suspension, and that the bank lines were fixed, with no attempt made to reproduce the degree of erodibility of the banks and sandbars. Also to be considered is the 1976 hydrograph used for testing of plans, which could be considerably different from what actually occurs in the river in the future, and the fact that the model surveys were always made during low-water periods.

Summary of Results and Conclusions

16. The indications and conclusions developed from the results of model tests are summarized as follows:

- a. Realigning of the entrance of the West Access Channel in Plan A reduced the bed-load sediments entering to less than 1 percent of existing conditions.
- b. The realignment in Plan A produced some shoaling of the deeper portions of the Atchafalaya River, below el -40, but it is believed that this change was not enough to significantly affect stages on the river below the realigned channel.
- c. Alignment of the new entrance of the West Access Channel in Plan A produced unsatisfactory current patterns that could cause severe attack on the downstream side of the entrance.
- d. Realignment of the entrance of the new channel in Plan A-Modified produced satisfactory current patterns entering the entrance of the channel.
- e. Plan A-Modified produced the same diverted bed-load sediments and effects on the Atchafalaya River Channel as obtained with Plan A.

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Chipola Cutoff Reach, Apalachicola River Movable-Bed Model Study

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Chipola Cutoff

4 Conclusions

The limitations of the model in reproducing all of the factors affecting developments in the reach and the differences between the model and prototype indicated by the results of the verification tests must be considered in the evaluation of model results. The model was not able to reproduce all of the overbank and the flow that passes over the banks during flood stages. The model also tended to scour deeper in the bendways and shoal higher on the point bars than the prototype surveys indicated. In spite of these limitations, adjustment and verification of the model were considered sufficient to indicate trends that can be expected under the conditions imposed for each plan or modification tested and the relative effectiveness of each plan.

Conclusions reached from the study are summarized as follows:

a. Thalweg disposal of dredged material will have little effect on channel dredging requirements.

b. Disposal material placed in the thalweg of the lower bend (navigation mile 40.20 to 39.85) tends to remain in place. Some increase of material passing out of the bend was noted.

c. The capacity of the bend for storage of dredged material decreases with each hydrograph.

d. Within-bank disposal of dredged material will have little effect on channel dredging required to maintain the navigation channel.

e. Within-bank disposal improves the channel from navigation mile 41.5 to 40.8 but deteriorates the channel in the lower bend (navigation mile 40.20 to 39.70) during average water years.

f. The channel will deteriorate following high water years with within-bank disposal.

g. Once filled, within-bank disposal will provide little or no additional storage capacity for dredged material due to the lack of erosion at these sites.

h. The system of conventional spur, L-head, and vane dikes and submerged sills used in Plan A-32 will not provide an adequate navigation channel.

i. The system of spur, L-head, and submerged vane dikes (Plan A-42) will develop and maintain an adequate navigation channel throughout the model reach except for some possible shoaling of material at navigation mile 41.1 and 39.9 following a high water year.

TECHNICAL REPORT H-73-1

CHANNEL CONDITIONS, DEVIL'S ISLAND REACH, MISSISSIPPI RIVER, MISSOURI AND ILLINOIS

Hydraulic Model Investigation

by

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Hydraulics Laboratory
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PART IV: EVALUATION OF RESULTS AND CONCLUSIONS

Limitation of Model Results

Devils Island

33. In evaluating the results of channel regulating works, it should be considered that the conventional process of verifying a movable-bed model was precluded in this case by the lack of necessary recorded data for the reach of the Mississippi River reproduced. The scale ratios established, therefore, can be considered to approximate only in a general way the relations between the model and the various reaches of the prototype. The time scale, in particular, was by necessity established arbitrarily; and, although it was used as a basis for reproducing the stage hydrograph (plate 2), it should not be considered an accurate indication of the time required in the prototype for model-predicted developments to occur under similar conditions.

34. Developments in the model occurred mostly as a result of bed movement. Although some of the material forming the bed of the model channel was thrown into suspension during the higher stages, the model did not even roughly simulate the effects of suspended sediment in a river of this type. Shoaling in the backwater areas resulting from deposition of sediment in suspension, therefore, could not be reproduced in this model.

35. The results of the model tests have to be based on the results of the model verification or on the accuracy with which the model was adjusted to reproduce prototype conditions. It should be considered that the model was operated by reproducing a flow hydrograph considered to be reasonably typical of flow conditions in that reach (plate 2). Flow conditions that vary appreciably from this hydrograph could affect developments within the channel. Consideration must also be given to the fact that tests were continued only until the general effects of the plan being tested were indicated and not until ultimate development was reached.

36. It is believed that, in spite of the above limitations, the model adjustment resulted in the development of operating techniques and

only bed character.

scale ratios sufficiently accurate to permit satisfactory reproduction of the bed movement characteristics of the river and thus should indicate the relative effectiveness of the various plans and modifications.

Summary of Results and Conclusions

37. The results and indications developed during the model study were as follows:

a. The original plan as developed and submitted for testing (plan A) would provide adequate depths through the reach, except at two of the six crossings. Some of the dikes proposed would be subjected to considerable attack by currents.

b. In general, the results indicate that contracting the channel to a control width of 1500 ft would provide satisfactory channel depths and alignment. Special treatment would be required in some of the crossings, depending on the alignment of the channel and the degree of sinuosity available.

c. Where special treatment is required in the crossings, the control channel width could be reduced with dikes of lower elevation than normally used for the 1500-ft control width. With this system, the amount of contraction would not be any greater than with a 1500-ft control width, except during low flows.

d. The most economical plan developed that would provide adequate channel dimensions was plan C. However, in this plan, some of the dikes were subjected to strong current attack.

e. The cost of construction could be reduced by using, where feasible, the stepped-down principle in the design of dike systems. With this system, the alignment and depth of the channel in crossings can be better controlled without overcontraction of the higher flows.

f. Plan D using the stepped-down principle produced the best overall results and would involve less construction than with plan A, which provided for a channel with a 1500-ft control width. With this plan, the average elevation of the new dikes was reduced by about 5 ft, with some reduction in the total length of dikes required.

g. Placing gaps in the dikes crossing the back chutes or channels, as tested in plan E, would have little effect on channel development when one end of the channel is closed. Properly designed L heads on the ends of spur dikes can be used to reduce deposition landward of the ends of the dikes and maintain slack-water areas behind and downstream of the dikes.

38. The effectiveness of L sections in maintaining slack-water areas behind the spur dike would depend on the relative elevation of the L section being lower than the spur dike, so that surface flow could move over the top of the L section and prevent bottom currents from moving around the end of the dike. Also, flow over the dike would tend to cause scouring along the landward side of the dike. Shoaling would occur near the end of the L section during low water, when the section is not overtopped. The L section would tend to reduce but not eliminate deposition some distance landward of the section from settlement of fine sediment that might be in suspension.