US Army Corps of Engineers
Kansas City District

Recommendations for a Plan to Regulate Commercial Dredging on the Kansas River

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Corps of Engineers
Kansas City, Missouri

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FINAL REPORT

RECOMMENDATIONS FOR A PLAN TO REGULATE COMMERCIAL DREDGING ON THE KANSAS RIVER

Prepared for

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EXECUTIVE SUMMARY

This report is a follow-up to the September 1984 report prepared by Simons, Li & Associates, Inc. (SLA) for the Corps of Engineers (COE), Kansas City District, entitled, "Analysis of Channel Degradation and Bank Erosion in the Lower Kansas River." That report provided a detailed study of the various factors which have been responsible for the severe bed degradation, channel widening, and bank erosion that has occurred in the lower Kansas River over the past several decades. Much of the data which is presented in this report was originally presented in the 1984 SLA report.

The objective of this study was to provide the COE, Kansas City District, with recommendations and pertinent data to be utilized in the District's development of a comprehensive management plan for the regulation of commercial sand and gravel dredging activities on the Kansas River and in the evaluation of individual permit applications for such dredging. The project area to be addressed under this scope of work includes two segments of the Kansas River. Segment 1 is the reach of river passing through Topeka and includes River Miles (RM) 80 to 90. Segment 2 extends from the confluence of the Kansas and Missouri Rivers to Bowersock Dam in Lawrence (RM 0 to 51.8).

Presented in this report are three main sets of recommendations for the COE to incorporate into its comprehensive management plan. The first set of recommendations is the concept of assigning a maximum amount of sand and gravel that can be dredged from a reach corresponding to a given level of impact (none, minor, moderate, or major). No judgment as to what level of impacts should be allowed is contained within this report. As part of the conditions for granting a dredging permit, it is recommended that the COE specify the maximum amount of sand and gravel that can be removed from the permit area.

Identification of the level of impact can be achieved by the use of a sediment continuity model that was developed for this study. The model provides the COE with the capability of assessing the amount of degradation induced by the removal of a given quantity of sand and gravel by a combination of dredges operating in a reach. The continuity model may be used for any future period of simulation. The volume of extraction in a reach can be varied from year to year. The continuity model was used to estimate the depths of aggradation/degradation for five different annual rates of dredging for the 30-year period, 1985 to 2015. The locations of dredging operations, and thus the volume of material removed from each reach were altered over time to simulate expected future trends. The modeling results were used to make a general estimate of how much material can be dredged under a given level of impact.

The second set of recommendations for the management plan is a monitoring scheme or data collection program. Recommendations are made for data collection of suspended sediment, bed-material samples, cross-sectional surveys, quantities of sand and gravel dredged, and the frequency with which aerial photos should be taken. Feedback from the monitoring program can be used to modify the criteria for permitting new dredging operations, for updating the continuity model, and for providing information on impacts induced by permitted dredges. All items discussed in the monitoring scheme have been prioritized so that items may be deleted if available funding is limited.

The third set of recommendations for inclusion into a management plan is a set of buffer distances in which no dredging should be allowed. The buffer distances relate to items such as bridge piers, pipeline crossings, dikes, islands, and the outside of sharp bends. A minimum buffer length between successive dredging operations is suggested to prevent the linking together of several dredge cuts.

I. INTRODUCTION

1.1 Study Objectives

This report is a follow-up to the September 1984 SLA report that provided a detailed study of the various factors which have been responsible for the severe bed degradation, channel widening, and bank erosion that has occurred in the lower Kansas River over the past several decades. The 1984 SLA report identified sand and gravel dredging as being one of the factors responsible for these changes. A general watershed map of the Kansas River basin is shown in Figure 1.1.

The Corps of Engineers (COE), Kansas City District, is the federal agency responsible for regulating dredging activities on the Kansas River. The objective of this study was to provide the COE with recommendations and pertinent data to be utilized in the District's development of a comprehensive management plan for the regulation of commercial sand and gravel dredging activities on the Kansas River and in the evaluation of individual permit applications for such dredging. The data presented will enable the District to assess the potential impacts of any probable number of dredges located in various reaches of the river relative to various quantities of material extracted.

1.2 Study Approach

The following scope of work was performed to meet the objectives of this study:

1. SLA staff visited the COE, Kansas City District office for meetings with COE staff, dredge operators, concerned landowners, and agencies. Following the meetings, SLA staff revisited dredge sites to become more familiar with dredging operations (dredge sites had been visited in preparation of SLA's 1984 report).

2. The mainstem of the Kansas River was subdivided into 22 reaches. The reach breakdown was based on consideration of tributary inflows, man-made and geologic controls, and the expected locations of future dredging operations.

3. Five different annual rates of sand and gravel extraction were developed for the Topeka area (RM 80 to 90) and for the reach below Bowersock Dam (RM 0 to 51.8). These five rates varied from no annual extraction to an initial rate of 2.70 million tons per year (2.30 million tons below Bowersock Dam and 0.40 million tons at Topeka) which was compounded by three percent annually for 30 years. Although not specifically required under the scope of work, annual extraction rates were also developed for the Wamego area (RM 126 to 127) and for the Manhattan area (RM 152 to

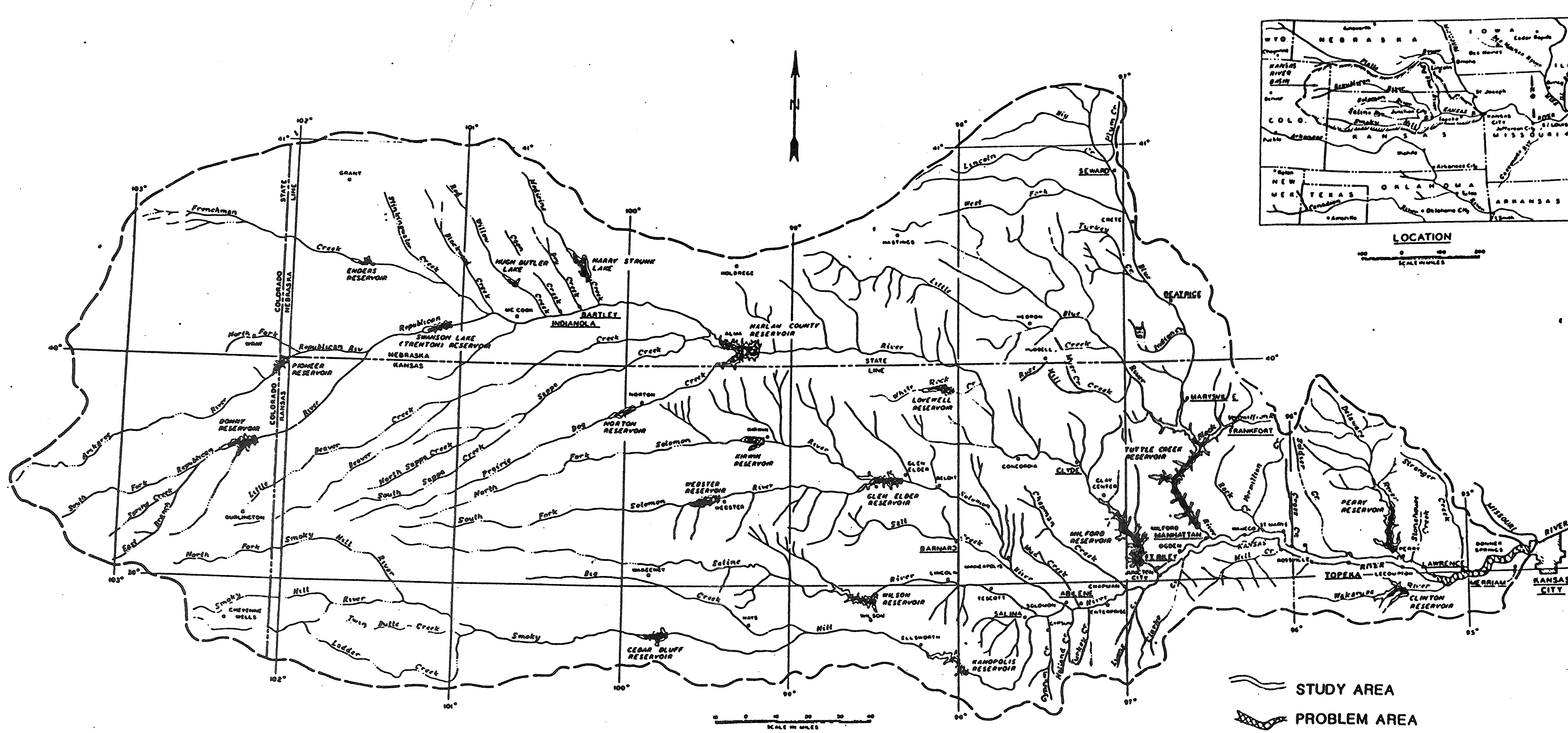

Figure 1.1. General watershed map for Kansas River (from Flood Plain Information Study, Kansas River, Kansas, U.S. Army Corps of Engineers, 1963).

153). The five rates were used in the analysis to determine the impacts associated with different quantities of material extraction.

4. The locations of dredging operations downstream of Bowersock Dam were predicted, by reach, for the 30-year period, 1985 to 2015. The dredge locations were selected by the COE after discussions with the dredging companies. The locations predicted are based on an expected trend to move dredging operations progressively upstream into undredged areas as downstream areas are depleted of suitable materials. Consideration was also given to the need to be close to present and future markets, and the location of existing and future haul routes.

5. The continuity model which was developed for the first report was modified to predict future changes in aggradation/degradation. The revised model, Program MINING, was run for the period 1985 to 2015 using the five rates of sand and gravel mining and the projected future locations of the dredge cuts. The results were tabulated on a reach by reach basis showing the depths of aggradation/degradation that the model estimates for the varying rates of extraction and locations of dredging operations. A classification system based on qualitative estimation of levels of impact (none, minor, moderate, and major) was developed and the continuity model results were retabulated based on this system. Definition of each level of impact was based on an interpretation of the potential consequences to the river system resulting from different depths of degradation. Estimates were made of how much sand and gravel can be dredged from each reach corresponding to a given level of impact. The model and documentation on its usage were supplied to the COE so that they may investigate any real or hypothetical future dredging rates and locations not specifically covered in this study.

6. The location of all bridges and pipeline crossings in the project area were plotted on a thalweg profile of the Kansas River. The stability of 13 bridges, Johnson County weir, and Bowersock Dam were discussed.

7. The computer program, HEC-2, was used to calculate backwater profiles through idealized dredge cuts. The depths of the dredge cuts were varied from 5 to 20 feet and the lengths were varied from 1,000 to 10,000 feet. The sediment trap efficiency was calculated for each case.

8. The existing COE regulations for minimum buffer distances (distance from structure or feature inside which no dredging is allowed) were evaluated. Recommendations were made to establish buffer distances from bridge piers, pipeline crossings, dikes, islands, and the outside of sharp bends, as well as a minimum buffer zone between successive dredging operations.

9. A data collection scheme or monitoring program was proposed. The components of the monitoring program were prioritized so that if funding is limited, the less-essential components may be deleted.

II. HYDRAULIC ANALYSIS OF DREDGE CUTS

The localized effects of dredge cuts include the acceleration of flow entering the upstream end of the dredge cuts and the slow-moving water flowing through the dredge cuts. These localized effects are different from the general aggradation/degradation of the channel bed that will be discussed in Chapter III. General aggradation/degradation of the stream bed occurs over larger reaches of river and longer periods of time. General aggradation/ degradation is the result of widespread and long-term imbalance between sediment supply and the river's capacity to transport the sediment. Excessive removal of sand and gravel throughout a large area can be one cause of general degradation. Localized effects do not extend a long distance from the point of disturbance and can occur over a relatively short period of time.

The COE computer model HEC-2 was used to perform a hydraulic analysis of a series of idealized dredge cuts ranging in length from 1,000 feet to 10,000 feet. Based on the results of the HEC-2 model and SLA experience with sand and gravel mining on other perennial rivers, dredge cuts in length of 1,000 feet have little potential for developing a headcut at their upstream face. (This would not be true for an ephemeral stream in which the cut or pit might not be filled with water continuously. In general, the dredge cut length required to produce headcutting would be shortened for steeper channels.) Longer pits of 10,000 feet have significant potential for headcutting. The potential for headcutting was evaluated based on the increase in velocity that occurs at the upstream end of the dredge cut. The larger the increase over existing conditions, the higher the potential for headcutting to develop. It is recommended that the COE establish guidelines for maximum permissible lengths of dredge cuts and minimum buffer distance between successive dredging operations in which no dredging is allowed. Recommendations for the lengths are presented in Section 5.3.

2.1 Existing Dredging Operations

Currently there exists 17 permitted dredging operations on the Kansas River. Table 2.1 lists the company names and the locations of these 17 permitted operations plus three other operations which have applied to the COE for permits. Figure 2.1 shows their locations. Cross-sectional data within a given dredging operation are scarce. In order to estimate a typical dredge cut length, the average lengths of the 17 permitted operations were calcu-

Table 2.1. Existing and Proposed Dredging Operations on the Kansas River.

Company	River Mile (RM)
Builders Sand	RM 6 - 7
Hub Material	RM 8 - 9.3
Hub Material	RM 12.3 - 12.9
Superior Sand and Gravel	RM 13.0 - 13.9
Builders Sand	RM 13.8 - 14.5
Holliday Sand and Gravel	RM 15.6 - 16.2
Holliday Sand and Gravel	RM 16.2 - 16.9
Builders Sand	RM 19.2 - 19.8
Holliday Sand and Gravel	RM 20.7 - 21.1
Holliday Sand and Gravel	RM 21.3 - 22.3
Kaw Sand (Proposed)	RM 26.3 - 27.9
Builders Sand (Proposed)	RM 31.2 - 32.2
Kaw Sand (Proposed)	RM 35.1 - 36.7
Penny's Ready Mixed Concrete	RM 50.1 - 51.3
Kansas Sand and Gravel	RM 83.0 - 85.2
Consumers Sand Co., Inc., River Sand Plant	RM 85.2 - 85.8
Victory Sand and Gravel	RM 86.3 - 86.6
Consumers Sand Co., Inc., River Sand Plant	RM 86.7 - 86.9
Wamego Sand	RM 126.5 - 126.9
Kershaw Ready Mix	RM 151.8 - 152.8

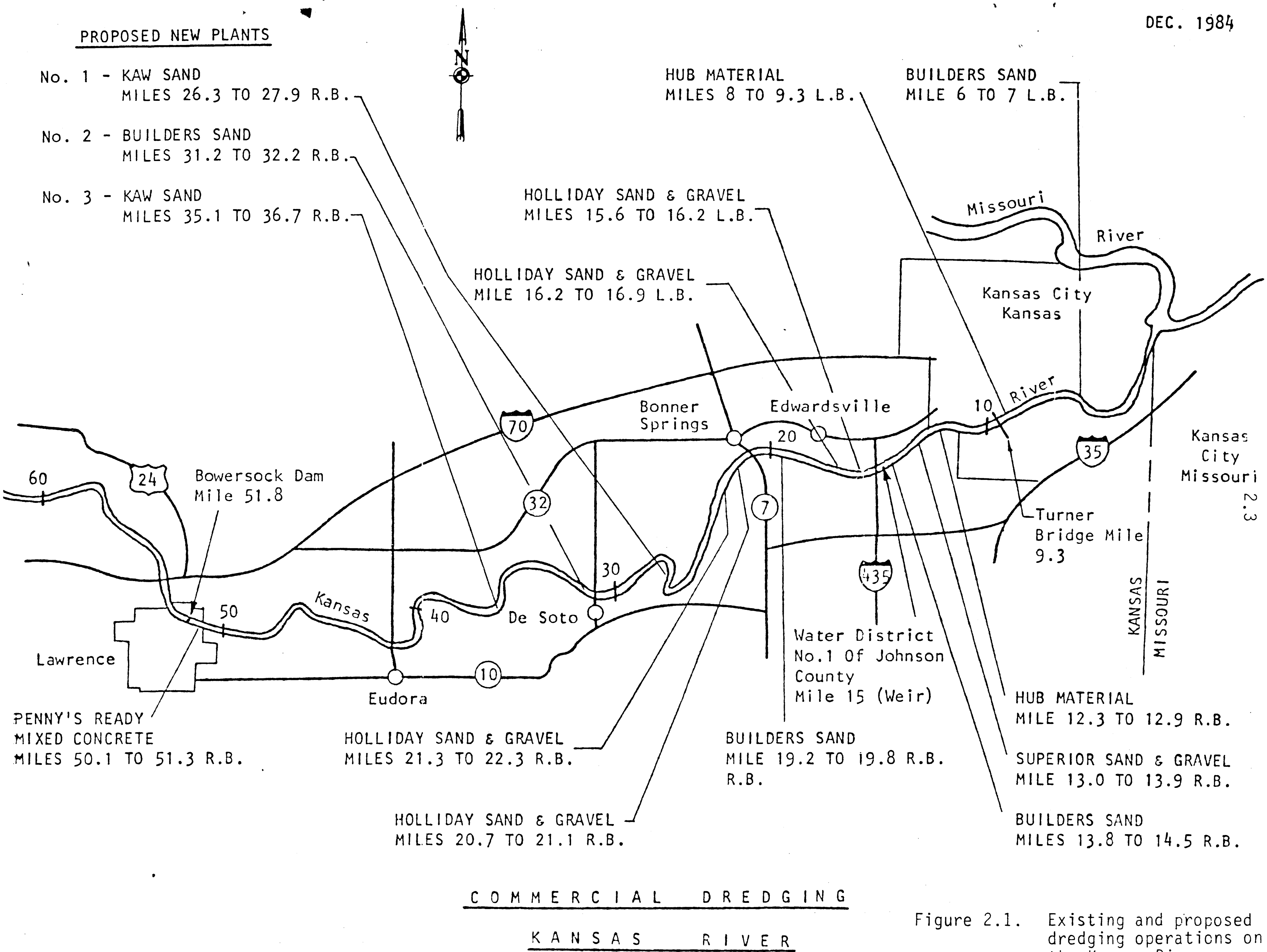

Figure 2.1. Existing and proposed dredging operations on the Kansas River.

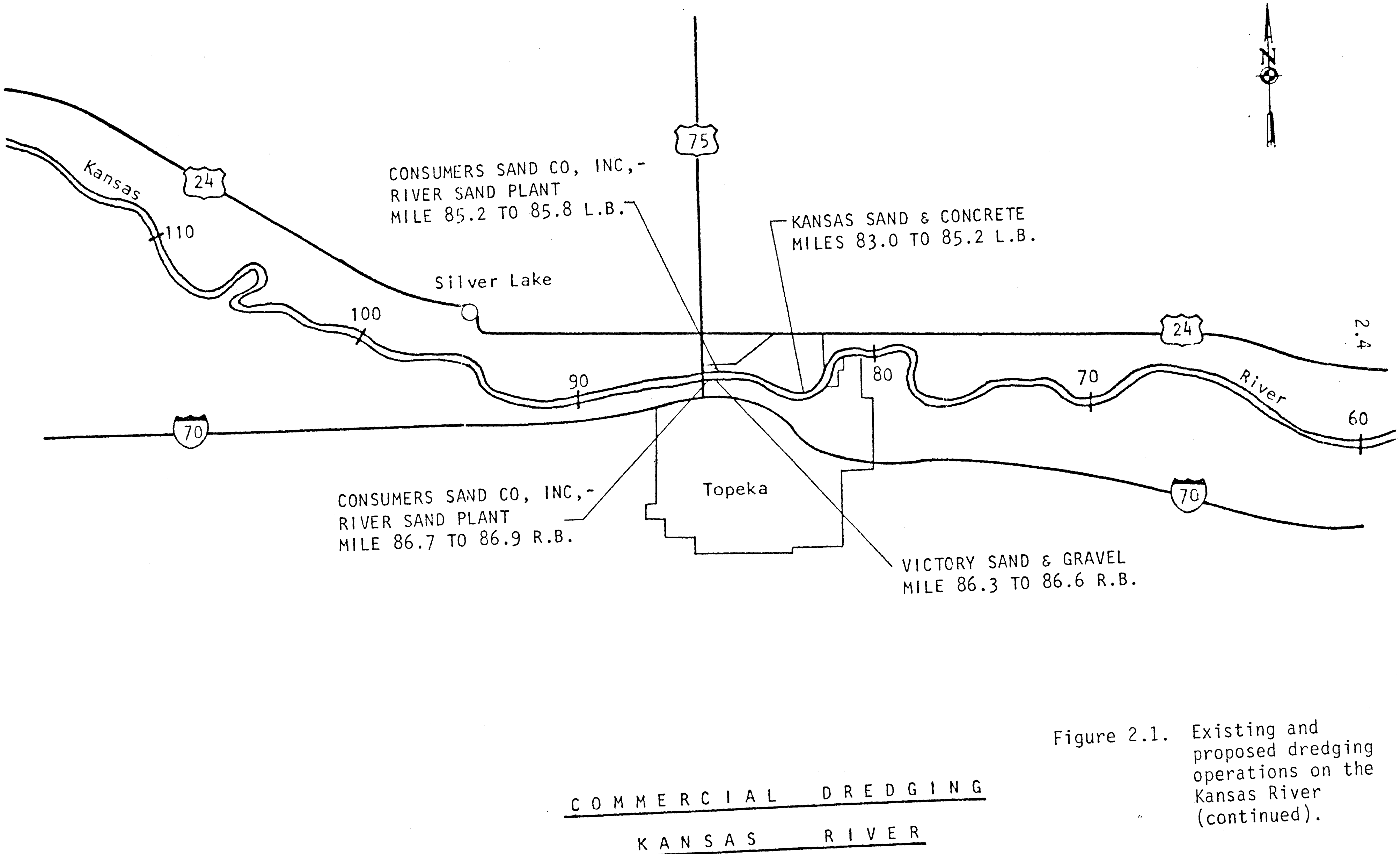

Figure 2.1. Existing and proposed dredging operations on the Kansas River (continued).

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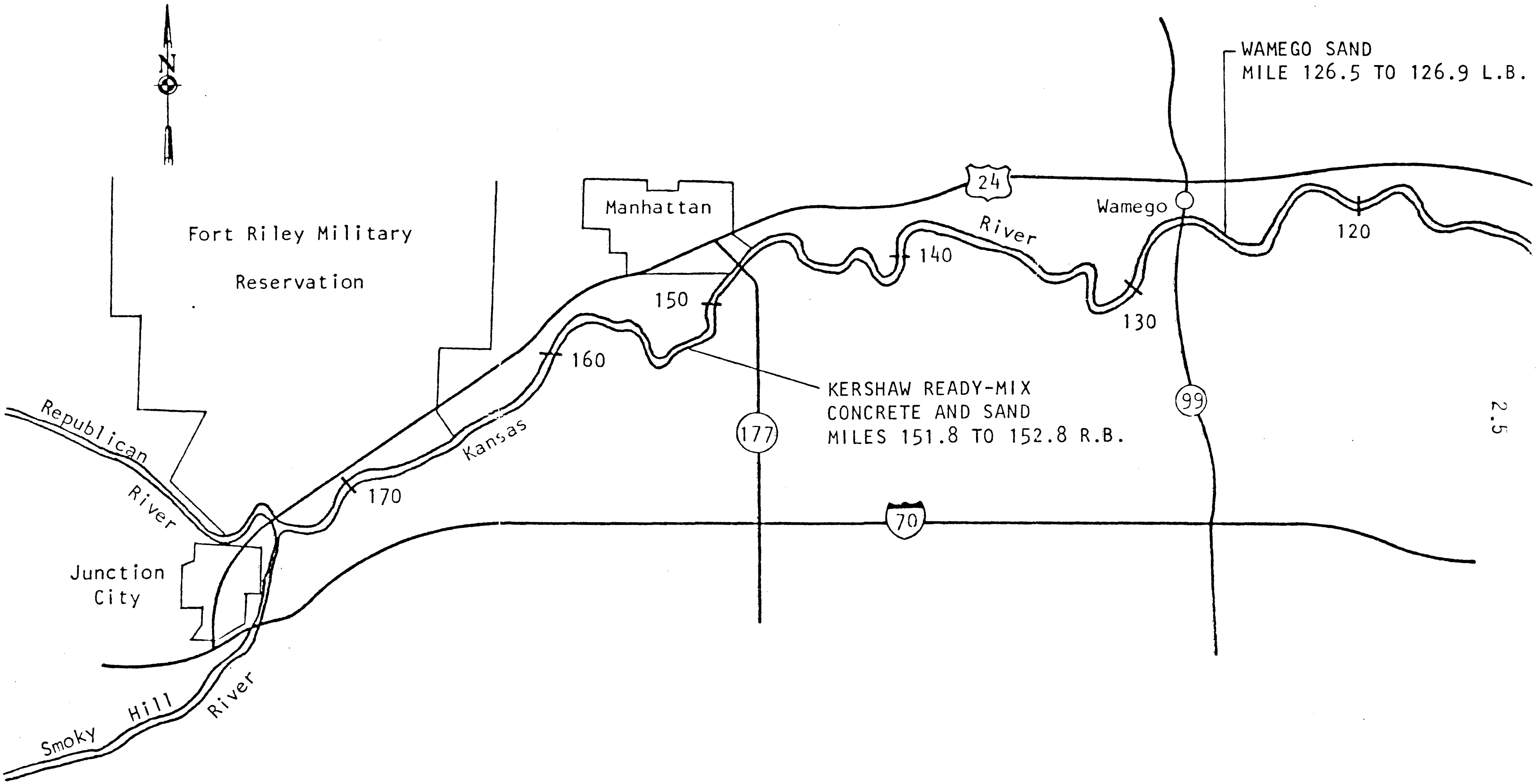

Figure 2.1. Existing and proposed dredging operations on the Kansas River (continued).

C O M M E R C I A L   D R E D G I N G

K A N S A S   R I V E R

lated and found to be approximately 4,900 feet. Hydraulic analysis was then performed on idealized dredge cuts 1,000, 5,000 and 10,000 feet in length. A cut of 1,000 feet is typical of a single-dredge operation. A 10,000-foot long cut represents the effects of several dredging operations linked together. A distinction must be made between the length of a dredge cut and the length of a permit granted to an individual dredging operation. Dredging companies move the locations of the dredges within their permit areas and it would be unusual for an operation to uniformly drop the river bed for the entire length of their permit. The hydraulic analysis thus represents the most severe lengths and depths of dredge cuts which could develop.

2.2 HEC-2 Analysis

The COE, HEC-2 computer model was used to perform the hydraulic analysis of the idealized dredge cuts. A trapezoidal approximation was made for the natural channel above and below the dredge cut. The natural channel trapezoidal approximation had an 800-foot wide base, 3:1 side slopes, a Manning's n value of 0.032, and a longitudinal slope of 0.00034. The dredge cut geometry was established for several depths of cuts by uniformly lowering the bed 5, 10, 15, and 20 feet. Twelve different combinations of dredge cut lengths (1,000, 5,000, and 10,000 feet) and depths (5, 10, 15, and 20 feet) were modeled. For each combination, six different discharges were investigated ranging from 1,000 to 100,000 cfs. The results of the HEC-2 analysis are presented in Table 2.2. Figures 2.2 to 2.4 are plots of the water-surface profiles for the three different lengths of cuts investigated. As mentioned in Section 2.1, the water-surface profile plots represent the most extreme hydraulic disturbances that could occur for permit lengths of 1,000, 5,000, and 10,000 feet.

Examination of Table 2.2 and the water-surface profiles yields the following qualitative conclusions:

1. The longer the dredge cut, the more pronounced the upstream water-surface drawdown, and thus the greater the potential for headcutting. A 1,000-foot long dredge cut had relatively minor effects on the hydraulics, while a 10,000-foot long cut had significant effects. Allowing different dredging operations to operate immediately one above the other could turn relatively small individual dredge cuts into one long continuous cut. A minimum buffer distance where no dredging is allowed should be maintained between successive dredging operations.

2. For a given sized pit, the hydraulic disturbance that the pit creates becomes drowned-out as the discharge increases.

Table 2.2. Analysis of Dredge Cuts With Respect to Velocities.

Discharge (cfs)	V Prior To Dredging (ft/sec)	1,000 ft Long Cut, 5 ft Deep: V through dredge cut (ft/sec)	1,000 ft Long Cut, 5 ft Deep: V at upper end of cut (ft/sec)	5,000 ft Long Cut, 5 ft Deep: V through dredge cut (ft/sec)	5,000 ft Long Cut, 5 ft Deep: V at upper end of cut (ft/sec)	10,000 ft Long Cut, 5 ft Deep: V through dredge cut (ft/sec)	10,000 ft Long Cut, 5 ft Deep: V at upper end of cut (ft/sec)
1,000	.99	.22 (22%)	1.58 (160%)	.25 (25%)	3.44 (347%)	.29 (29%)	3.43 (346%)
5,000	1.87	.80 (43%)	2.16 (116%)	.87 (47%)	3.98 (213%)	.98 (52%)	5.87 (314%)
10,000	2.47	1.31 (53%)	2.68 (109%)	1.40 (57%)	3.58 (145%)	1.52 (62%)	6.72 (272%)
20,000	3.24	2.06 (64%)	3.38 (104%)	2.15 (66%)	3.93 (121%)	2.28 (70%)	4.87 (150%)
50,000	4.61	3.47 (75%)	4.68 (102%)	3.57 (77%)	4.99 (108%)	3.67 (80%)	5.40 (117%)
100,000	5.92	4.92 (83%)	5.99 (101%)	5.00 (84%)	6.18 (104%)	5.08 (86%)	6.43 (109%)

Discharge (cfs)	V Prior To Dredging (ft/sec)	1,000 ft Long Cut, 20 ft Deep: V through dredge cut (ft/sec)	1,000 ft Long Cut, 20 ft Deep: V at upper end of cut (ft/sec)	5,000 ft Long Cut, 20 ft Deep: V through dredge cut (ft/sec)	5,000 ft Long Cut, 20 ft Deep: V at upper end of cut (ft/sec)	10,000 ft Long Cut, 20 ft Deep: V through dredge cut (ft/sec)	10,000 ft Long Cut, 20 ft Deep: V at upper end of pit (ft/sec)
1,000	.99	.06 (6%)	1.59 (161%)	.06 (6%)	3.44 (347%)	.06 (6%)	3.44 (347%)
5,000	1.87	.27 (14%)	2.18 (117%)	.28 (15%)	4.51 (241%)	.29 (16%)	5.85 (313%)
10,000	2.47	.50 (20%)	2.71 (110%)	.52 (21%)	3.90 (158%)	.54 (22%)	7.38 (299%)
20,000	3.24	.90 (28%)	3.42 (106%)	.93 (29%)	4.24 (131%)	.96 (30%)	6.37 (197%)
50,000	4.61	1.84 (40%)	4.72 (102%)	1.88 (41%)	5.28 (115%)	1.93 (42%)	6.25 (136%)
100,000	5.92	2.99 (51%)	6.01 (102%)	3.04 (51%)	6.44 (109%)	3.10 (52%)	7.09 (120%)

Note: The percentages in brackets represent the velocity in the dredge cut as a percent of the velocity prior to dredging.

V = Velocity

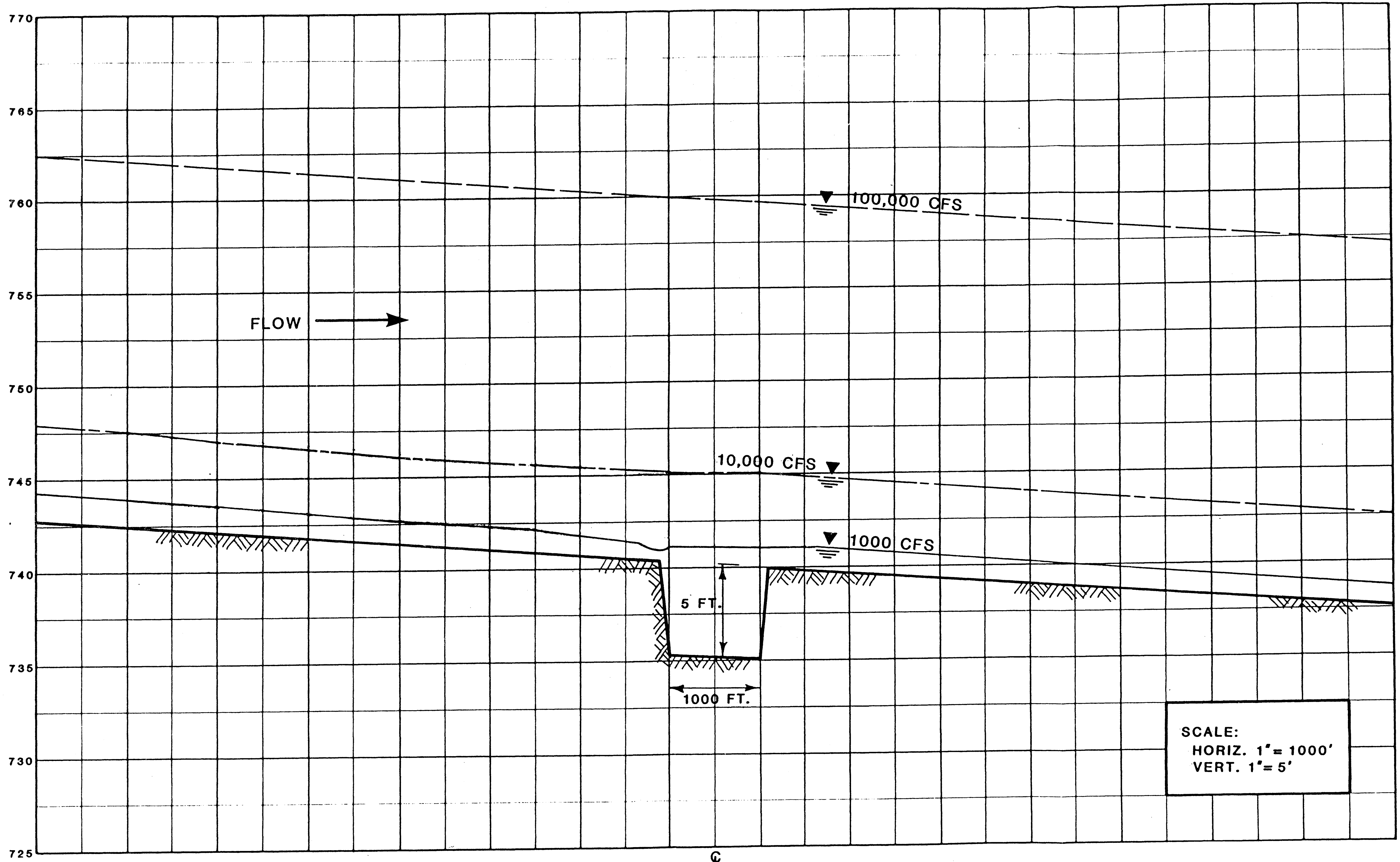

Figure 2.2. Water-surface profiles for a 1,000-ft long, 5-ft deep dredge cut.

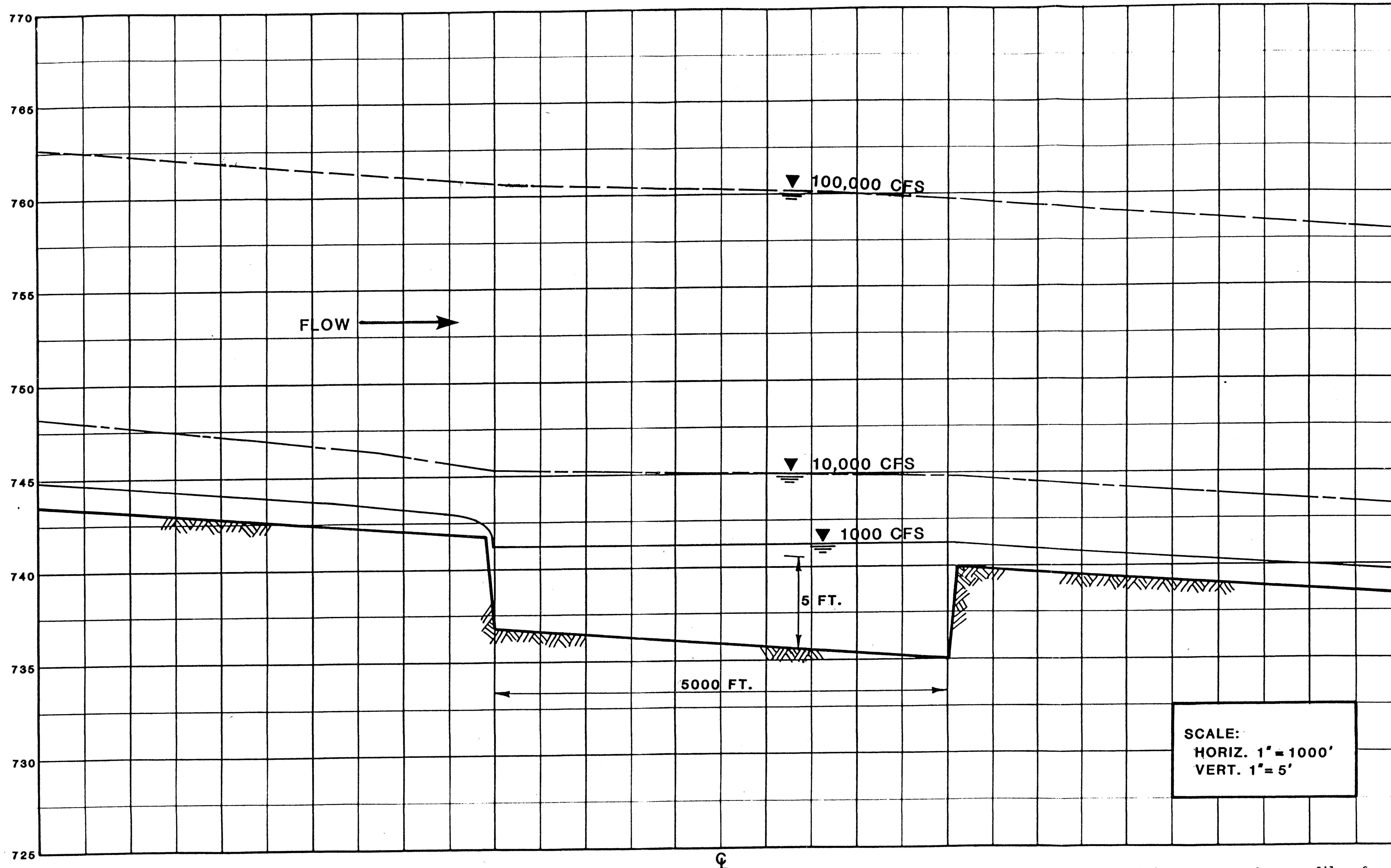

Figure 2.3. Water-surface profiles for a 5,000-ft long, 5-ft deep dredge cut.

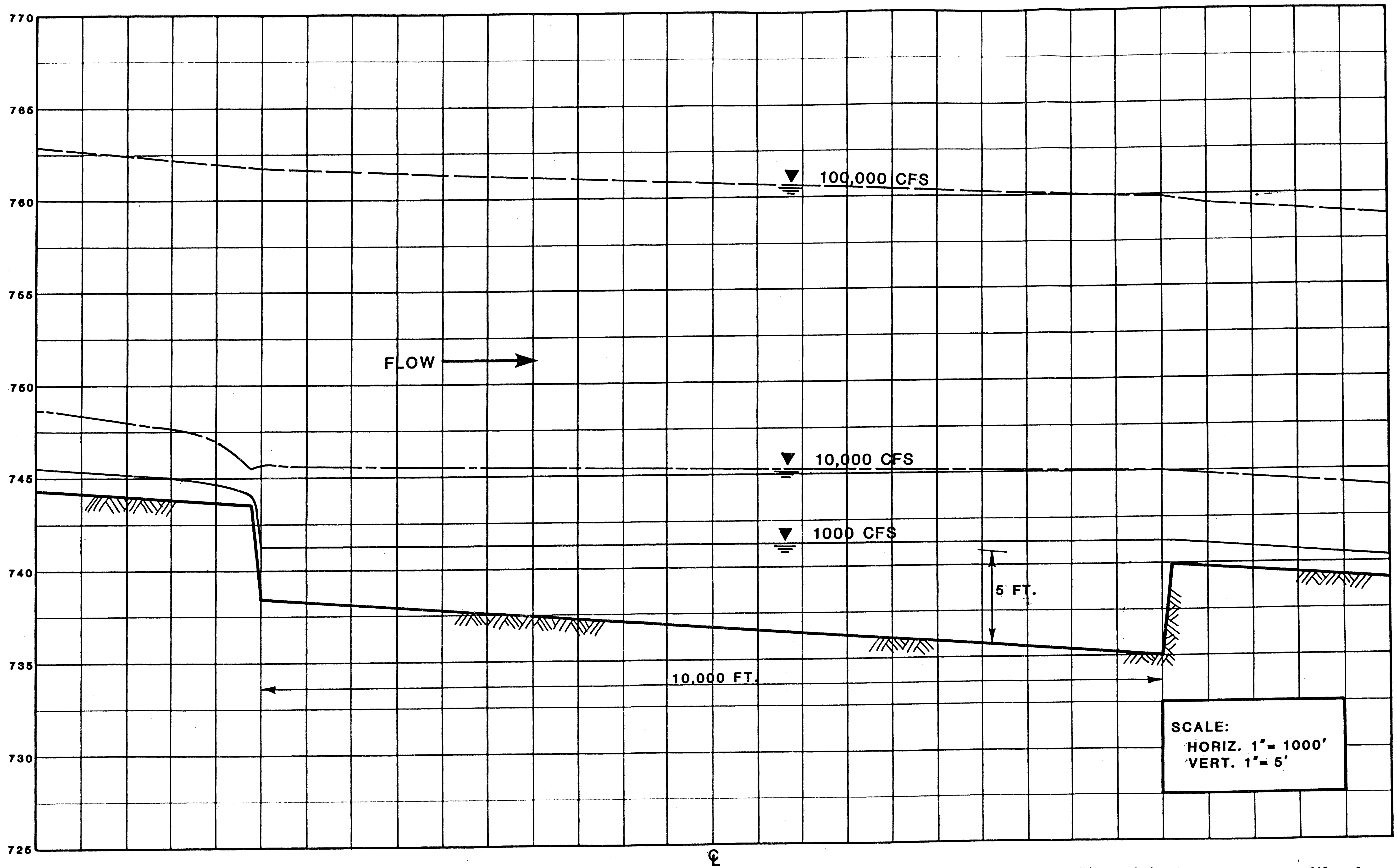

Figure 2.4. Water-surface profiles for a 10,000-ft long, 5-ft deep dredge cut.

3. As the depth of the pit is increased, the velocity through it decreases, which allows more time for the suspended sediments to settle out to the bottom of the dredge pit. This in turn increases the potential for downstream scour since relatively sediment-free water is flowing out of the dredge pit. Calculations of sediment trap efficiencies indicate that at a discharge of 20,000 cfs, and a 1,000-foot long, 5-foot deep pit, all the suspended sediment greater than 0.08 mm will be trapped in the pit. If the depth of the pit is increased to 20 feet, the velocity flowing through it will be further decreased and virtually all of the sand-sized sediments (> 0.062mm) will settle. The average trap efficiency of the entire width of the river is less than these values since the cut does not extend from bank to bank. Thus, the portion of the channel adjacent to the banks retains its original sediment-transort capacity. Figure 2.5 is a definition sketch illustrating this concept.

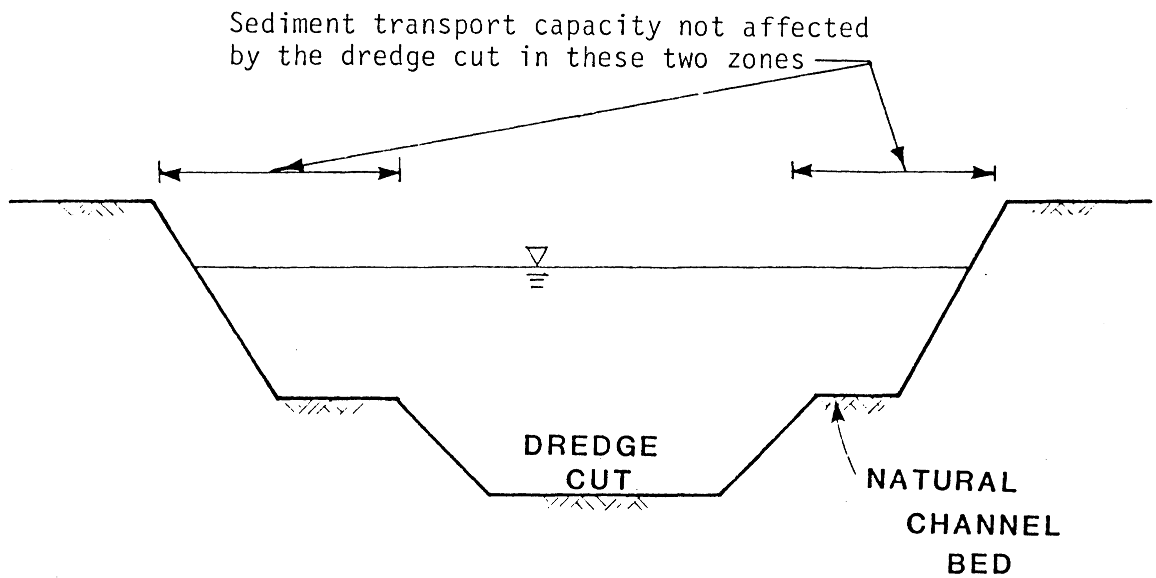

Figure 2.5. Definition sketch of a dredge cut.

III. CONTINUITY MODEL

A continuity-based water- and sediment-routing model was developed and applied to predict future changes in the base level of the Kansas River. The model was executed for the period 1985 to 2015 using five different annual rates of sand and gravel extraction. The location of the dredging operations were varied through the 30-year period to simulate their anticipated future locations.

3.1 Modifications Made to the Continuity Model

The continuity model was originally developed for the study associated with the 1984 SLA report. The original continuity model was used to simulate aggradation/degradation during the historical period from 1940 to 1973. Program MINING was developed to estimate the future response of the Kansas River channel for varying rates and locations of dredging from 1985 to 2015. To accomplish this, several modifications were made to the program for this study; the most signficant of which include:

1. The original continuity model utilized the USGS daily discharge records. Program MINING uses an average annual water yield which is constant from year to year. An average annual water yield is calculated at one reach based on a flow-duration curve. Average annual water yields are then calculated for the remaining reaches and tributaries using a hydrology ratio. The hydrology ratio is equal to the reach's (or tributary's) mean annual discharge divided by the mean annual discharges of the reach on which the flow duration is based. The flow-duration curve used in this study was based on 40 years (1935 to 1974) of synthesized data at Desoto for the with-reservoir, no-target low-flow conditions. The 1951 flood is included in the flow-duration curve. The flow-duration curve of Desoto was selected over the other three stations where flow-duration curves were available (Lecompton, Wamego, and Fort Riley) because of its proximity to the areas of intense dredging.

2. The sediment-transport relations developed in the first report were left unchanged. However, new sediment-transport relations were developed to reflect the reduced sediment-transport capabilities that are associated with the greater depths and slower velocities of dredge cuts. These new sediment-transport relationships are utilized when the depth of degradation reaches specified values.

The new sediment-transport equations are presented in Table 3.1. These equations are based on the hydraulic and sediment-transport considerations of idealized dredge cuts. Equations were developed for four different depths of channel degradation, 5, 10, 15, and 20 feet. A series of Q (water discharge) and Q_s (sand discharge) data points were generated for each depth of degra-

Table 3.1. Sediment Transport Equations for Different Levels of Degradation.

Fort Riley

Original equations for zero degradation (both Q and Q_s are in cfs):

$Q_s = 1.21 \times 10^{-5}\ Q^{1.28}$ for $Q \leq 20,000$ cfs

$Q = 4.58 \times 10^{-7}\ Q^{1.61}$ for $Q > 20,000$ cfs

New equations:

$Q_s = 1.39 \times 10^{-15}\ Q^{3.20}$ for 2.5 to 7.5 ft of degradation

$Q_s = 1.06 \times 10^{-18}\ Q^{3.70}$ for 7.5 to 12.5 ft of degradation

$Q_s = 1.77 \times 10^{-20}\ Q^{3.96}$ for 12.5 to 17.5 ft of degradation

$Q_s = 1.10 \times 10^{-21}\ Q^{4.12}$ for > 17.5 ft of degradation

Wamego

Original equations for zero degradation (both Q and Qs are in cfs):

$Q_s = 5.10 \times 10^{-8}\ Q^{1.75}$ for $Q \leq 22,000$ cfs

$Q_s = 1.26 \times 10^{-9}\ Q^{2.12}$ for $Q > 22,000$ cfs

New equations:

$Q_s = 2.25 \times 10^{-16}\ Q^{3.36}$ for 2.5 to 7.5 ft of degradation

$Q_s = 2.06 \times 10^{-19}\ Q^{3.86}$ for 7.5 to 12.5 ft of degradation

$Q_s = 3.24 \times 10^{-21}\ Q^{4.14}$ for 12.5 to 17.5 ft of degradation

$Q_s = 1.83 \times 10^{-22}\ Q^{4.31}$ for > 17.5 ft of degradation

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Table 3.1. Sediment Transport Equations for Different Levels of Degradation (continued).

Lecompton

Original equations for zero degradation (both Q and Q_s are in cfs):

$$Q_s = 3.86 \times 10^{-9} Q^{1.97} \text{ for } Q \leq 38{,}000 \text{ cfs}$$

$$Q_s = 3.24 \times 10^{-10} Q^{2.21} \text{ for } Q > 38{,}000 \text{ cfs}$$

New equations:

$$Q_s = 8.46 \times 10^{-17} Q^{3.42} \text{ for 2.5 to 7.5 ft of degradation}$$

$$Q_s = 6.64 \times 10^{-20} Q^{3.94} \text{ for 7.5 to 12.5 ft of degradation}$$

$$Q_s = 8.59 \times 10^{-22} Q^{4.24} \text{ for 12.5 to 17.5 ft of degradation}$$

$$Q_s = 4.09 \times 10^{-23} Q^{4.43} \text{ for > 17.5 ft of degradation}$$

Desoto

Original equations for zero degradation (both Q and Q_s are in cfs):

$$Q_s = 4.35 \times 10^{-8} Q^{1.74} \text{ for } Q \leq 28{,}000 \text{ cfs}$$

$$Q_s = 2.23 \times 10^{-9} Q^{2.03} \text{ for } Q > 28{,}000 \text{ cfs}$$

New equations:

$$Q_s = 3.09 \times 10^{-16} Q^{3.28} \text{ for 2.5 to 7.5 ft of degradation}$$

$$Q_s = 1.93 \times 10^{-19} Q^{3.82} \text{ for 7.5 to 12.5 ft of degradation}$$

$$Q_s = 2.19 \times 10^{-21} Q^{4.12} \text{ for 12.5 to 17.5 ft of degradation}$$

$$Q_s = 9.57 \times 10^{-23} Q^{4.32} \text{ for > 17.5 ft of degradation}$$

---

dation, and then a power curve was fitted to these data pairs. An example follows:

For the reach at Desoto, the HEC-2 results from the 1984 report give the following hydraulics:

Discharge $Q = 9{,}750$ cfs

Area $A = 3{,}693\ ft^2$

Velocity $V = 2.64$ ft/sec

Avg. Depth $Y = 4.7$ ft

The sediment-transport relation for the Desoto reach is given in the 1984 report as:

$$Q_s = 4.35 \times 10^{-8}\ Q^{1.74} \quad \text{for } Q \leq 28{,}000 \text{ cfs}$$

$$Q_s = 2.23 \times 10^{-9}\ Q^{2.03} \quad \text{for } Q > 28{,}000 \text{ cfs}$$

for $Q = 9{,}750$ cfs, $Q_s = 0.380$ cfs

If the depth is increased by 5 ft to 9.7 ft and the top width is held constant, the velocity drops to 1.28 ft/sec. A regression analysis for the Desoto reach has determined that the sediment transport is proportional to the velocity to approximately the sixth power. Therefore, the sediment-transport capacity will be lowered from 0.380 cfs to,

$$Q_s = 0.380 \left(\frac{1.28}{2.64}\right)^{6.0} = 0.00494 \text{ cfs}$$

This same procedure was applied for a variety of discharges to yield a series of Q and Q_s data pairs. Regression equations were fit to these data pairs to yield the equations presented in Table 3.1. These equations were read as input data into Program MINING. The program checks the cumulative depth of degradation from the previous time step before deciding which sediment-transport equation to use for the present time step. Adjusting the sediment-transport equations based on the depth of degradation gives more conservative results than the original continuity model presented in the 1984 SLA report. If the sediment-transport equations are held constant (as in the original continuity model) the predicted aggradation/degradation for each reach will be increased by as much as 50 percent. The results presented in this chapter are therefore on the low side (i.e., conservative).

The time step used in this study was one-twelfth of a year, or a monthly time step. At each time step, the program calculates the sediment-transport capacity by discretizing the flow-duration curve to obtain an average annual sediment-transport capacity (as in Section 4.5.1 of the 1984 SLA report) and then dividing this average annual sediment-transport capacity by the number of time steps per year. Since the sediment-transport equations can vary from one time step to the next, it is necessary to repeat these calculations each time step.

3.2 Reach Definition and Sand and Gravel Extraction Rates

The reach breakdown used for this study (Table 3.2) is similar to that used in the 1984 SLA report, but is more detailed. Fifteen reaches were modeled in the 1984 SLA report, while 22 reaches were modeled for this report. The extra reaches are mostly in the length of river around Topeka, and from Bowersock Dam to the Johnson County weir. The additional reaches were chosen to provide for a higher resolution of the locations of the dredging operations.

Five different rates of future sand and gravel extraction were modeled (Table 3.3). The extraction rate of 2.30 million tons a year below Bowersock Dam is representative of the average annual amount of material that has been removed from 1970 to 1981 between Bowersock Dam and the Missouri River confluence. Actual yearly rates fluctuate about the value depending on the demand. The extraction rate of 0.40 million tons a year represents the annual amount of sand and gravel that has been removed from 1978 to 1983 in the Topeka reach. Manhattan and Wamego's extraction rates of 0.12 and 0.04 million tons a year represent the average annual amount of material that has been removed from 1979 to 1983.

Rate B's annual extraction rates are one-half of the average annual extraction rates presented in Rate D. Rate C's annual extraction rate of 1.67 million tons a year corresponds to Desoto's average annual sand and gravel yield, based on an incremental analysis of the synthesized 1935 to 1974 flow-duration curve for modified conditions (page 4.23 of the 1984 SLA report). The average annual extraction rates presented in Rate D for Topeka, Wamego, and Manhattan were retained for Rate C, since the annual extraction rates at these locations do not exceed the annual sand yields.

Table 3.2. Reach Definitions.

Reach No.	River Mile	Sediment Transport Equation	Currently Dredged	Special Features
1	170.4 - 147.5	Fort Riley	Yes	
2	147.5 - 121.5	Wamego	Yes	
3	121.5 - 101.2	Wamego	No	
4	101.2 - 101.0	Wamego	No	Bedrock Outcrop
5	101.0 - 93.0	Lecompton	No	
6	93.0 - 88.0	Lecompton	Yes	
7	88.0 - 80.6	Lecompton	Yes	
8	80.6 - 64.5	Lecompton	No	
9	64.5 - 51.9	Lecompton	No	
10	51.9 - 51.7	Lecompton	No	Bowersock Dam
11	51.7 - 46.7	Desoto	Yes	
12	46.7 - 41.6	Desoto	No	
13	41.6 - 34.8	Desoto	Yes	
14	34.8 - 31.0	Desoto	Yes	
15	31.0 - 26.5	Desoto	Yes	
16	26.5 - 22.0	Desoto	Yes	
17	22.0 - 15.1	Lecompton	Yes	
18	15.1 - 14.9	Lecompton	No	Johnson County Weir
19	14.9 - 12.4	Lecompton	Yes	
20	12.4 - 12.2	Lecompton	No	Armor Bar
21	12.2 - 9.3	Lecompton	No	
22	9.3 - 0	Lecompton	No	

NOTE: "Sediment Transport" refers to the gaging station where the water and sediment data was collected for development of the sediment transport equation.

Table 3.3. Sand and Gravel Dredging Rates.

Rate	Description
A	No dredging
B	1985 to 2015 rates: 1.15 million tons a year below Bowersock Dam 0.20 million tons a year at Topeka 0.02 million tons a year at Wamego 0.06 million tons a year at Manhattan Total amount dredged from 1985 to 2015 is 43.20 million tons
C	1985 to 2015 rates: 1.67 million tons a year below Bowersock Dam 0.40 million tons a year at Topeka 0.04 million tons a year at Wamego 0.12 million tons a year at Manhattan Total amount dredged from 1985 to 2015 is 66.96 million tons
D	1985 to 2015 rates: 2.30 million tons a year below Bowersock Dam 0.40 million tons a year at Topeka 0.04 million tons a year at Wamego 0.12 million tons a year at Manhattan Total amount dredged from 1985 to 2015 is 85.80 million tons
E	1985 rates: 2.30 million tons a year below Bowersock Dam 0.40 million tons a year at Topeka 0.04 million tons a year at Wamego 0.12 million tons a year at Manhattan For Rate E only, the tons of material dredged from 1986 to 2015 have been compounded at an annual rate of 3 percent. Total amount dredged from 1985 to 2015 is 136.07 million tons

NOTE: These five rates were used with the MINING model to determine the potential for degradation of the riverbed associated with various rates of material extraction. Rate D is representative of the existing extraction rates. Rate B is 50 percent of Rate D's extraction. Rate C for the river below Bowersock Dam is representative of the average annual sand yield at Desoto as determined in Table 4.3 of SLA, 1984.

Mass curves of the five different rates of extraction are shown in Figure 3.1. The mass curves illustrate how significant a three percent annual growth becomes after 30 years. Rates D and E both start out removing the same annual amount of material, but after 30 years, Rate E is removing sand and gravel at an annual rate nearly two and one-half times as great as Rate D.

The locations of the dredges were varied throughout the 1985 to 2015 period of simulation. Table 3.4 presents how the locations of the dredges were varied with time. The general trend in Table 3.4 is that the dredging operations will move progressively upstream as the downstream areas are depleted of suitable materials. The rates and locations of dredging were based on all the information that was available at the time this report was prepared. They take into consideration the competitive need to be close to a market, the existing and probable future locations of roads required to access sand and gravel markets, the existing and probable future markets, and the depletion of suitable materials in various reaches of the river. The possibility of predicting a similar scenario for the Topeka area was examined and rejected. Such a prediction would be unreliable due to the smaller number of dredges operating in the Topeka area.

3.3 Results of Continuity Modeling

The results of the continuity modeling are presented in Tables 3.5 to 3.7. Table 3.5 presents the results for the 10-year period, 1985 to 1995; Table 3.6 is for the 20-year period, 1985 to 2005; and Table 3.7 gives the results for the 30-year period, 1985 to 2015. These results are long-term averages that were arrived at by using an average annual water yield which is based on the 1935 to 1974 flow-duration curve. A series of dry years will result in less aggradation/degradation than is shown in the results, while a series of wet years will result in increased aggradation/degradation. The aggradation/degradation shown is relative to the 1985 base level and does not include the actual aggradation or degradation that has occurred in the past.

Results are not presented for the lower 12 miles of the Kansas River (Reaches 21 and 22). As explained on page 5.1 of the 1984 SLA report, these reaches cannot be accurately modeled by Program MINING. The sediment-transport rates in these two reaches are dependent on both the hydraulics of the Kansas River and the stage of the Missouri River. Fortunately for the purposes of this study, the lower 12 miles is not the area of primary interest.

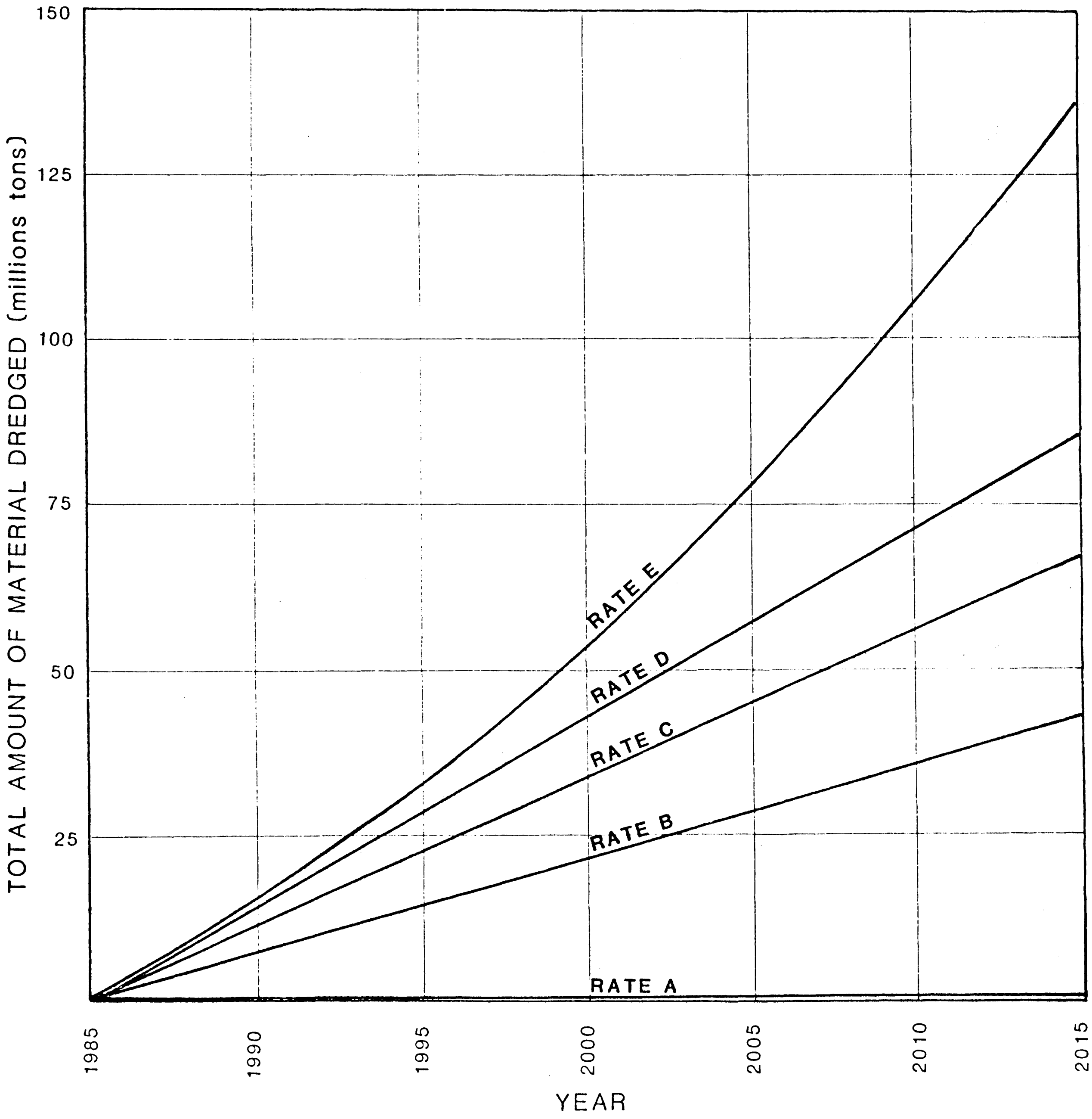


Figure 3.1. Mass curves for five rates of dredging.

Table 3.4. Projected Future Volume of Material Extracted From Each Reach Over Time.

Period From 1985 to 1995		
	Reach 11, RM 51.7-46.7	5%
	Reach 14, RM 34.8-31.0	10%
	Reach 15, RM 31.0-26.5	15%
	Reach 17, RM 22.0-15.1	45%
	Reach 19, RM 14.9-12.4	25%
		100%
Period From 1995 to 2005		
	Reach 11, RM 51.7-46.7	5%
	Reach 14, RM 34.8-31.0	20%
	Reach 15, RM 31.0-26.5	25%
	Reach 16, RM 26.5-22.0	20%
	Reach 17, RM 22.0-15.1	20%
	Reach 19, RM 14.9-12.4	10%
		100%
Period From 2005 to 2015		
	Reach 11, RM 51.7-46.7	5%
	Reach 13, RM 41.6-34.7	5%
	Reach 14, RM 34.8-31.0	20%
	Reach 15, RM 31.0-26.5	25%
	Reach 16, RM 26.5-22.0	25%
	Reach 17, RM 22.0-15.1	15%
	Reach 19, RM 14.9-12.4	5%
		100%

Note: 100% represents the total amount of sand and gravel dredged between Bowersock Dam and the confluence of the Kansas and Missouri River.

The rates of sand and gravel dredging listed in Table 3.3 were modeled in conjunction with the location of dredging operations shown above to yield the results of aggradation/degradation listed in Tables 3.5 to 3.7.

Table 3.5. Results of Continuity Model, 1985 to 1995.

Net aggradation (+) in feet for the 10-year period, 1985 to 1995
Net degradation (-) in feet for the 10-year period, 1985 to 1995

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	-1	-1	-1	-1	-1
2	147.5 - 121.5	+1	+1	0	0	0
3	121.5 - 101.2	0	0	0	0	0
4	101.2 - 101.0	0	0	0	0	0
5	101.0 - 93.0	+1	+1	+1	+1	+1
6	93.0 - 88.0	0	-1	-2	-2	-2
7	88.0 - 80.6	0	-1	-1	-1	-1
8	80.6 - 64.5	0	0	0	0	0
9	64.5 - 51.9	-1	-1	-1	-1	-1
10	51.9 - 51.7	0	0	0	0	0
11	51.7 - 46.7	-1	-1	-2	-2	-2
12	46.7 - 41.6	0	0	0	0	0
13	41.6 - 34.8	-1	-1	-1	-1	-1
14	34.8 - 31.0	-1	-2	-2	-2	-2
15	31.0 - 26.5	0	-2	-3	-3	-3
16	26.5 - 22.0	0	0	-1	-3	-2
17	22.0 - 15.1	0	-2	-2	-2	-3
18	15.1 - 14.9	0	0	0	0	0
19	14.9 - 12.4	0	-2	-3	-6	-11
20	12.4 - 12.2	0	0	0	0	0

Table 3.6. Results of Continuity Model, 1985 to 2005.

Net aggradation (+) in feet for the 20-year period, 1985 to 2005
Net degradation (-) in feet for the 20-year period, 1985 to 2005

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	-1	-2	-2	-2	-2
2	147.5 - 121.5	+1	+1	+1	+1	+1
3	121.5 - 101.2	-1	-1	-1	-1	-1
4	101.2 - 101.0	0	0	0	0	0
5	101.0 - 93.0	+2	+2	+2	+2	+2
6	93.0 - 88.0	0	-2	-2	-2	-2
7	88.0 - 80.6	0	-1	-3	-3	-3
8	80.6 - 64.5	0	0	-1	-1	-1
9	64.5 - 51.9	-2	-2	-2	-2	-2
10	51.9 - 51.7	0	0	0	0	0
11	51.7 - 46.7	-1	-2	-2	-2	-2
12	46.7 - 41.6	0	0	0	-1	-2
13	41.6 - 34.8	-1	-1	-1	-1	-1
14	34.8 - 31.0	-2	-2	-3	-2	-2
15	31.0 - 26.5	0	-2	-2	-3	-5
16	26.5 - 22.0	0	-3	-3	-4	-10
17	22.0 - 15.1	+1	-3	-3	-6	-8
18	15.1 - 14.9	0	0	0	0	0
19	14.9 - 12.4	0	-2	-7	-9	-16
20	12.4 - 12.2	0	0	0	0	0

Table 3.7. Results of Continuity Model, 1985 to 2015.

Net aggradation (+) in feet for the 30-year period, 1985 to 2015
Net degradation (-) in feet for the 30-year period, 1985 to 2015

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	-2	-2	-2	-2	-2
2	147.5 - 121.5	+2	+1	+1	+1	0
3	121.5 - 101.2	-1	-1	-1	-1	-1
4	101.2 - 101.0	0	0	0	0	0
5	101.0 - 93.0	+3	+3	+3	+3	+3
6	93.0 - 88.0	0	-2	-2	-2	-2
7	88.0 - 80.6	0	-2	-3	-3	-3
8	80.6 - 64.5	0	0	-2	-2	-2
9	64.5 - 51.9	-2	-3	-3	-3	-3
10	51.9 - 51.7	0	0	0	0	0
11	51.7 - 46.7	-2	-2	-2	-3	-4
12	46.7 - 41.6	-1	-2	-2	-2	-3
13	41.6 - 34.8	-2	-2	-3	-2	-3
14	34.8 - 31.0	-2	-2	-3	-3	-9
15	31.0 - 26.5	-1	-3	-3	-4	-15
16	26.5 - 22.0	0	-3	-3	-8	-21
17	22.0 - 15.1	+1	-3	-4	-8	-14
18	15.1 - 14.9	0	0	0	0	0
19	14.9 - 12.4	0	-3	-7	-10	-22
20	12.4 - 12.2	0	0	0	0	0

As explained in the 1984 SLA report, the model calculates the volume of aggradation/degradation within a reach from the equation:

$$\Delta S = \text{Supply} - \text{Capacity} - \text{Dredging}$$

where ΔS is the net sediment surplus or deficit (ft^3), Supply is the supply of sediment coming into the reach (ft^3), Capacity is the sediment-transport capacity of the reach (ft^3), and Dredging is volume of material dredged from the reach (ft^3). The depth of aggradation/ degradation ΔZ (ft) is then obtained from:

$$\Delta Z = (\Delta S * \text{Bulking Factor}) \div (\text{Length} * \text{Width})$$

where Length and Width are the reaches length and width in feet, and Bulking Factor is a factor used to convert from a volume of pure sediment (100 percent solids) to a volume of sediment that contains both solids and voids. The bulking factor equals $1 \div (1 - \text{porosity})$, or 1.67 for a porosity equal to 0.40.

The results in Tables 3.5 to 3.7 show the strong correlation between the amount of sand and gravel that is dredged and the net aggradation/degradation of the river. Within a given reach, degradation (a negative ΔZ) can result from a reduced supply of sediment coming into the reach, an increase in transport capacity as compared to the reach immediately upstream, or extensive dredging within the reach. Aggradation (a positive ΔZ) occurs when the supply of sediment coming into the reach is greater than the sediment-transport capacity of the reach. Thus, the depths of degradation shown in Tables 3.5 to 3.7 are a function of both the dredging within the reach itself, plus the reduced supply of sediment coming into the reach due to dredging upstream from the reach. For example, in Table 3.5 (1985 to 1995), Reach 11 experiences one foot of degradation for Rate A dredging and two feet of degradation for Rate E dredging. Reach 11 is dredged during this 10-year period, but the closest dredging upstream from Reach 11 does not occur until Wamego nearly 70 miles away. The extra foot of degradation at Reach 11 from Rate A to Rate E is thus attributable solely to the dredging within Reach 11. Conversely, for the same period, Reach 16 shows zero feet of degradation for Rate A dredging and two feet of degradation for Rate E dredging. Reach 16 is not dredged during this period but Reach 15 immediately upstream is. Reach 15 experiences three feet of degradation for Rate E dredging, which causes its sediment-transport capacity to drop due to the reduced velocities through the dredge cuts. This in turn results in a decreased supply of sediment coming out of Reach 15 and into Reach 16. Reach 16, therefore, degrades due to this decreased supply of incoming sediment.

These two examples illustrate how degradation can occur due to one of two distinct processes. For most of the reaches, aggradation/degradation occurs from the combination of dredging within the reach itself plus dredging upstream from the reach.

In any modeling effort of this nature, a certain amount of "noise" will be present in the results. The model results do not consider the change in bed elevation associated with the river's bed form. The bed form of the Kansas River is dunes (see Section 4.1.1 of 1984 SLA report). Dune heights are typically on the order of one-third to one-tenth of the flow depth so the bed level of the Kansas River can easily fluctuate from one-half to three feet depending on whether a dune crest or trough is present. Judgment says that predicting depths of aggradation/degradation 30 years into the future involves a level of uncertainty. Therefore, what is most important in Tables 3.5 to 3.7 is the trends predicted by the model rather than the precise value of the numbers themselves. The trend in these results is clearly that the more sand and gravel mined, the greater the drop will be in the Kansas River's bed level.

The values in Tables 3.5 to 3.7 have been rounded off to the nearest foot. Another way of looking at the same results is to classify them according to more qualitative levels (none, minor, moderate, and major impacts). This type of qualitative classification also serves to filter out some of the "noise" that is present in the results. Tables 3.8 to 3.10 categorize the results that are reported in Tables 3.5 to 3.7 according to the following four levels of impact:

Level 1: 0 to 2.5 ft of degradation. No impacts to structures or channel morphology.

Level 2: 2.5 to 5.0 ft of degradation. Minor impacts to structures and channel morphology. Minor bank erosion would be occurring at this level.

Level 3: 5.0 to 8.0 ft of degradation. Moderate impacts to structures and channel morphology. Bed degradation and bank erosion would be substantial and adjacent property owners would be threatened with substantial losses of land.

Table 3.8. Results of Continuity Model Classified By Level of Impact - 1985 to 1995.

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	1	1	1	1	1
2	147.5 - 121.5	1	1	1	1	1
3	121.5 - 101.2	1	1	1	1	1
4	101.2 - 101.0	1	1	1	1	1
5	101.0 - 93.0	1	1	1	1	1
6	93.0 - 88.0	1	1	1	1	1
7	88.0 - 80.6	1	1	1	1	1
8	80.6 - 64.5	1	1	1	1	1
9	64.5 - 51.9	1	1	1	1	1
10	51.9 - 51.7	1	1	1	1	1
11	51.7 - 46.7	1	1	1	1	1
12	46.7 - 41.6	1	1	1	1	1
13	41.6 - 34.8	1	1	1	1	1
14	34.8 - 31.0	1	1	1	1	1
15	31.0 - 26.5	1	1	2	2	2
16	26.5 - 22.0	1	1	1	2	1
17	22.0 - 15.1	1	1	1	1	2
18	15.1 - 14.9	1	1	1	1	1
19	14.9 - 12.4	1	1	2	3	4
20	12.4 - 12.2	1	1	1	1	1

NOTE: Data from Table 3.5 was evaluated to arrive at the above results.
Level 1 is 0 to 2.5 feet of degradation
Level 2 is 2.5 to 5.0 feet of degradation
Level 3 is 5.0 to 8.0 feet of degradation
Level 4 is > 8.0 feet of degradation

Table 3.9. Results of Continuity Model Classified By Level of Impact - 1985 to 2005.

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	1	1	1	1	1
2	147.5 - 121.5	1	1	1	1	1
3	121.5 - 101.2	1	1	1	1	1
4	101.2 - 101.0	1	1	1	1	1
5	101.0 - 93.0	1	1	1	1	1
6	93.0 - 88.0	1	1	1	1	1
7	88.0 - 80.6	1	1	2	2	2
8	80.6 - 64.5	1	1	1	1	1
9	64.5 - 51.9	1	1	1	1	1
10	51.9 - 51.7	1	1	1	1	1
11	51.7 - 46.7	1	1	1	1	1
12	46.7 - 41.6	1	1	1	1	1
13	41.6 - 34.8	1	1	1	1	1
14	34.8 - 31.0	1	1	2	1	1
15	31.0 - 26.5	1	1	1	2	3
16	26.5 - 22.0	1	2	2	2	4
17	22.0 - 15.1	1	2	2	3	4
18	15.1 - 14.9	1	1	1	1	1
19	14.9 - 12.4	1	1	3	4	4
20	12.4 - 12.2	1	1	1	1	1

NOTE: Data from Table 3.6 was evaluated to arrive at the above results.
Level 1 is 0 to 2.5 feet of degradation
Level 2 is 2.5 to 5.0 feet of degradation
Level 3 is 5.0 to 8.0 feet of degradation
Level 4 is > 8.0 feet of degradation

Table 3.10. Results of Continuity Model Classified By Level of Impact - 1985 to 2015.

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	1	1	1	1	1
2	147.5 - 121.5	1	1	1	1	1
3	121.5 - 101.2	1	1	1	1	1
4	101.2 - 101.0	1	1	1	1	1
5	101.0 - 93.0	1	1	1	1	1
6	93.0 - 88.0	1	1	1	1	1
7	88.0 - 80.6	1	1	2	2	2
8	80.6 - 64.5	1	1	1	1	1
9	64.5 - 51.9	1	2	2	2	2
10	51.9 - 51.7	1	1	1	1	1
11	51.7 - 46.7	1	1	1	2	2
12	46.7 - 41.6	1	1	1	1	2
13	41.6 - 34.8	1	1	2	1	2
14	34.8 - 31.0	1	1	2	2	4
15	31.0 - 26.5	1	2	2	2	4
16	26.5 - 22.0	1	2	2	3	4
17	22.0 - 15.1	1	2	2	4	4
18	15.1 - 14.9	1	1	1	1	1
19	14.9 - 12.4	1	2	3	4	4
20	12.4 - 12.2	1	1	1	1	1

NOTE: Data from Table 3.7 was evaluated to arrive at the above results.
Level I is 0 to 2.5 feet of degradation
Level II is 2.5 to 5.0 feet of degradation
Level III is 5.0 to 8.0 feet of degradation
Level IV is > 8.0 feet of degradation

Level 4: >8.0 ft of degradation. Major impacts to structures and channel morphology. Bed degradation and bank erosion would be severe and adjacent property owners would be experiencing a significant loss of land.

The depths of 2.5, 5.0, and 8.0 feet are based on engineering judgment and experience in river systems throughout the country. They are not absolute values, but general guidelines.

The results of Tables 3.8 to 3.10 indicate that for the case of no sand and gravel mining (Rate A), the river is essentially in an equilibrium condition with Level 1 impacts throughout. The introduction of dredging (Rate B) at a rate equal to one-half the present extraction levels increases the level of impact to Level 2. As the annual extraction rate is increased from Rate B to Rate E, the level of impact jumps from a minor to a major level.

When analyzing the level of impact, the depth of aggradation/degradation of the river bed is only one factor that needs to be considered. Another important consideration is the structures that will be affected by a change in the river's base level. For example, a reach of river that degrades down four feet will be classified as experiencing Level 2 minor impacts according to the above classification system. If there was a bridge located in that reach that has footings buried only four feet, then the bridge would suffer much more than just minor impacts. Chapter IV deals more with this subject.

IV. STRUCTURES IN THE KANSAS RIVER

A considerable number of man-made structures occur in the channel of the Kansas River. These structures include automotive and railroad bridges, weirs, pipelines, water supply intakes, river training works, and bank revetment. River training works and bank revetment are not considered in this chapter. Bridges, weirs, pipelines, and water supply intake locations are listed in Table 4.1 and plotted on a thalweg profile in Figure 4.1. Information is limited to two reaches of river (near Topeka and below Bowersock Dam) as required in the scope of work.

4.1 Bridges

Partial plans for 13 of the bridges on the Kansas River were made available to SLA by the COE, the Kansas Department of Transportation, the Atchison, Topeka, and Santa Fe Railway Company, and the SSW Railway Company. These bridges were analyzed to determine the damage each bridge would incur due to general channel degradation and local scour. Inspection of the available plans indicates that there are four general types of pier foundations in use on bridges in the Kansas River (Figure 4.2). In Type A pier foundations (Turner Bridge, I-435 Bridge, Desoto Bridge, Sardou Avenue Bridge, and Westgate Bridge), the piers are poured down to bedrock. With a Type B pier foundation (18th Street Bridge and the 3 railroad bridges), the piers are supported by piles that are driven to bedrock or as deep as the piles can penetrate. The bottom of the piers (or top of the piles) are situated in the alluvium about half way between the channel bottom and the bedrock. For Type C pier foundations (Eudora Bridge, Kansas Avenue Bridge and Topeka Avenue Bridge), the piers are supported on large (15- to 20-foot diameter) circular sheet piles that are driven to bedrock and filled with reinforced concrete. The bottom of the piers (or top of the piles) are situated at approximately the channel bottom. In Type D pier foundations (Highway 7 Bridge at Bonner Springs) the reinforced concrete bridge piers are poured down to bedrock and encased within corrugated metal pipes (CMP). The CMP's rest on rock sockets that penetrate into the bedrock and are filled with reinforced concrete.

Since the combined effects of general degradation and local scour at the piers cannot lower the channel bottom past the bedrock elevation, Pier Types A, C, and D will not be endangered by undermining. However, if the piers (Types A, C, and D) were designed such that the lateral resistance of the

Table 4.1. Structures in the Kansas River.

River Mile	Year Built	Description
Reach 22, RM 9.3-0		
0.2	-	RR Bridge
0.2	-	Lewis & Clark Viaduct Bridge
0.3	-	James St. Bridge
0.35	1965	36" Sewage Forcemain
0.7		RR Bridge
1.2		Central Ave. Bridge
1.5		RR Bridge
1.7		Stockyard Bridge
2.0		E. Kansas Ave. Bridge
2.5		RR Bridge
3.1	1975	30" Sewer Main
3.5		7th St. Bridge
4.4		12th St. Bridge
4.4	1940	24" Water Main
4.65	1966	30" Sewer Line
4.9		18 St. Bridge
5.3	1975	24" Sewer Line
5.8		Kansas Ave. Bridge
7.2		I-635 Bridge
9.0	1943	8" Pipeline
9.3		Turner Bridge
Reach 21, RM 12.2-9.3		
11.31	1963	10" Pipeline
11.6	1978	3-8", 2-12" Petroleum Pipelines
11.7	1950	12" Gasoline Pipeline
Reach 19, RM 14.9-12.4		
14.65	1954	2-20" Gas Lines
Reach 18, RM 15.1-14.9		
14.9		Johnson County Weir and Intake

Table 4.1. Structures in the Kansas River (continued).

River Mile	Year Built	Description
Reach 17, RM 22.0-15.1		
15.5	-	I-435 Bridge
16.0	1937	2-8" Pipelines
16.6	1980	12" Forcemain
19.4	1934	8" Gas line
20.3		Bonner Springs Hwy 7 Bridge
21.2		Atchison, Topeka, and Santa Fe RR Bridge
Reach 14, RM 34.8-31.0		
31.0		Desoto Bridge
31.2	1957	Telephone Cable
32.3	1966	16" Water Line
32.9	1944	Sunflower Plant Intake Structure
Reach 12, RM 46.7-41.6		
42.5		Eudora Bridge
Reach 11, RM 51.7-46.7		
49.6	1969	8" Fertilizer Line
49.75	1963	26" Gas Line
50.9	1976	18" Sewer Forcemain
50.9	1956	8" Sewer Forcemain
Reach 10, RM 51.9-51.7		
51.8		Bowersock Dam
51.8		Lawrence Bridge
Reach 7, RM 88.0-80.6		
82.8	1963	18" Sewage Forcemain
83.0		Sardou Ave. Bridge
83.7		Atchison, Topeka, and Santa Fe RR Bridge
84.2		Kansas Ave. Bridge
84.4		Topeka Avenue Bridge
84.5		SSW RR Bridge
87.7		Westgate Bridge

Table 4.1. Structures in the Kansas River (continued).

River Mile	Year Built	Description
<u>Reach 17, RM 22.0-15.1</u>		
15.5	-	I-435 Bridge
16.0	1937	2-8" Pipelines
16.6	1980	12" Forcemain
19.4	1934	8" Gas line
20.3		Bonner Springs Hwy 7 Bridge
21.2		Atchison, Topeka, and Santa Fe RR Bridge
<u>Reach 14, RM 34.8-31.0</u>		
31.0		Desoto Bridge
31.2	1957	Telephone Cable
32.3	1966	16" Water Line
32.9	1944	Sunflower Plant Intake Structure
<u>Reach 12, RM 46.7-41.6</u>		
42.5		Eudora Bridge
<u>Reach 11, RM 51.7-46.7</u>		
49.6	1969	8" Fertilizer Line
49.75	1963	26" Gas Line
50.9	1976	18" Sewer Forcemain
50.9	1956	8" Sewer Forcemain
<u>Reach 10, RM 51.9-51.7</u>		
51.8		Bowersock Dam
51.8		Lawrence Bridge
<u>Reach 7, RM 88.0-80.6</u>		
82.8	1963	18" Sewage Forcemain
83.0		Sardou Ave. Bridge
83.7		Atchison, Topeka, and Santa Fe RR Bridge
84.2		Kansas Ave. Bridge
84.4		Topeka Avenue Bridge
84.5		SSW ~~Southern~~ RR Bridge
87.7		Westgate Bridge

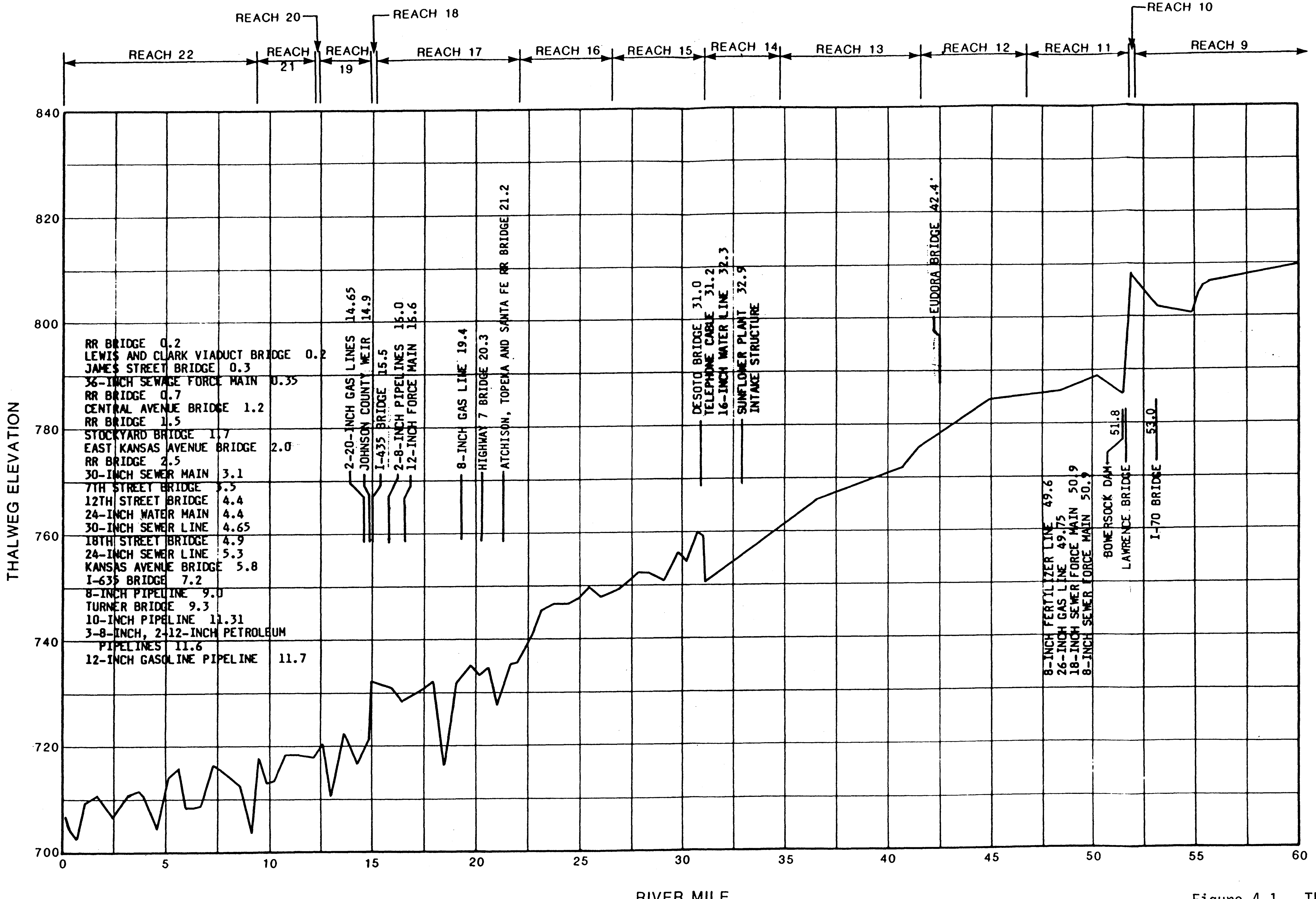


Figure 4.1. Thalweg profile of the Kansas River (continued).

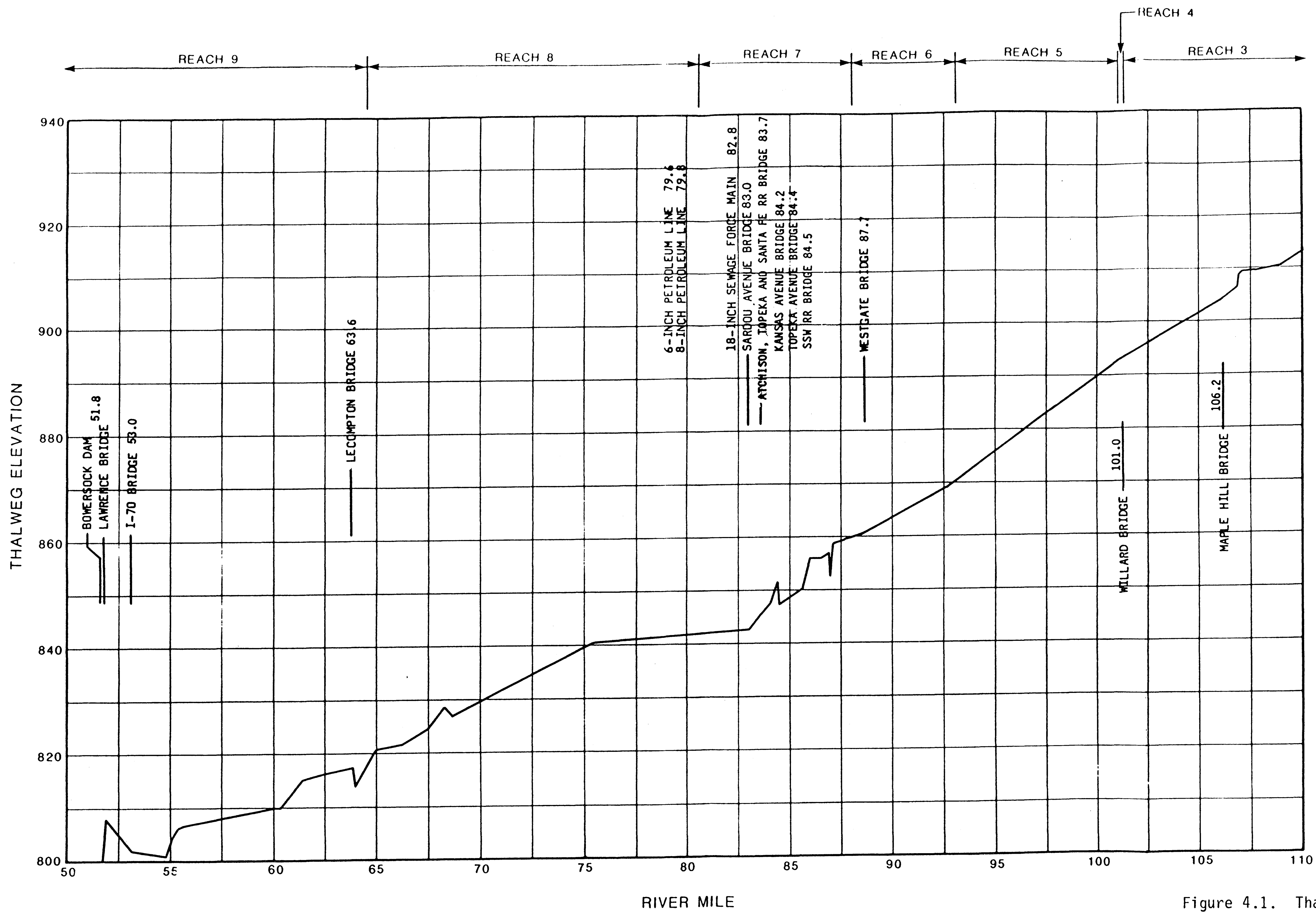


Figure 4.1. Thalweg profile of the Kansas River.

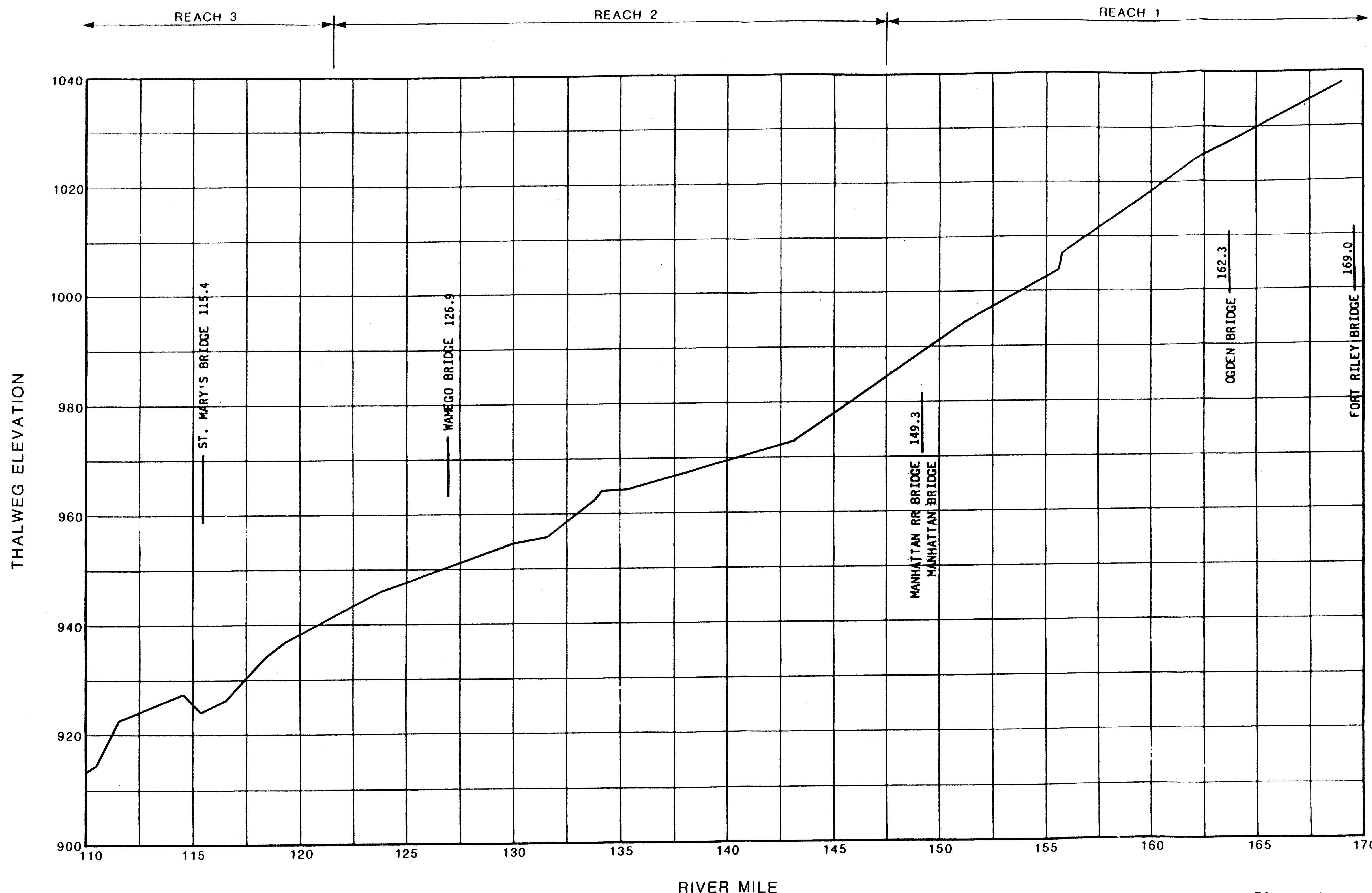
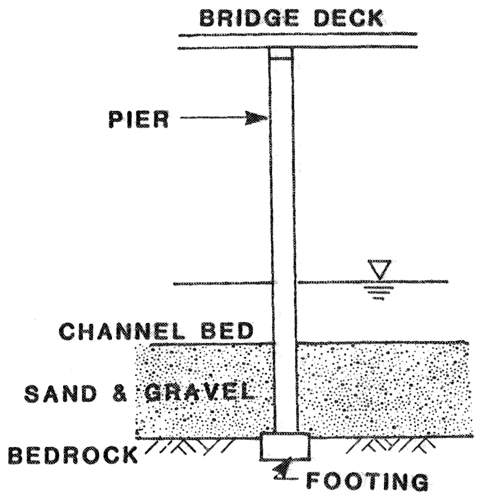
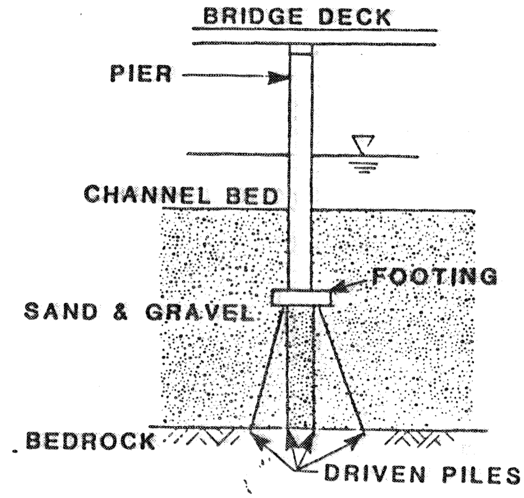


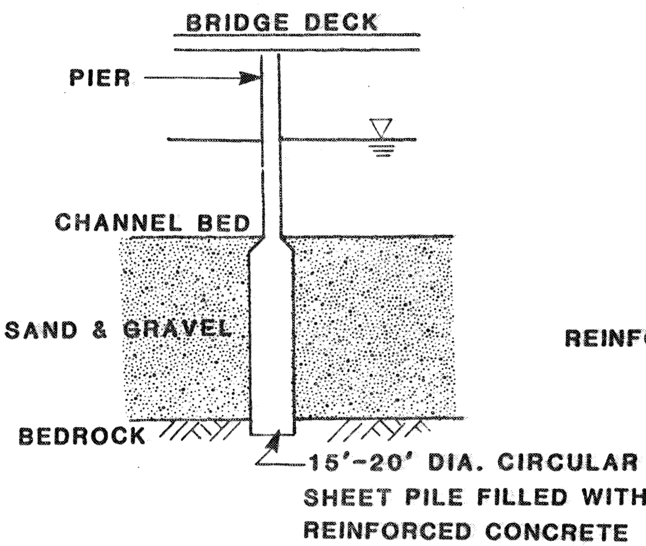
Figure 4.1. Thalweg profile of the Kansas River (continued).



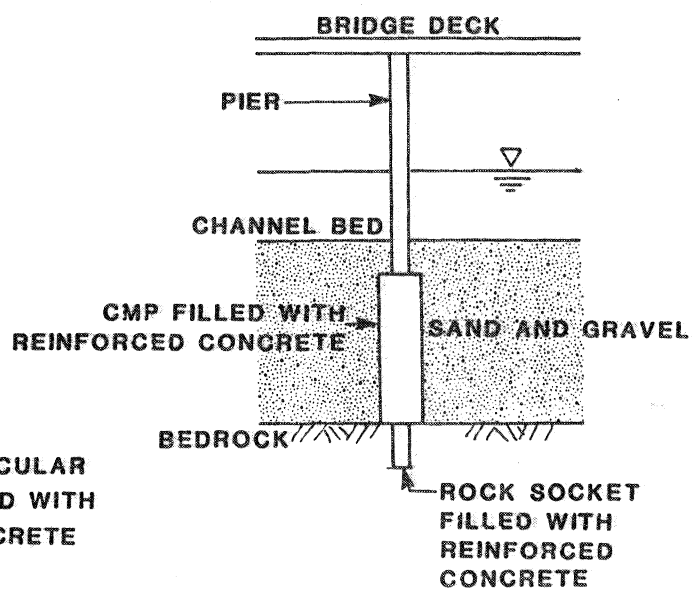
Type A - Spread footing on bedrock.



Type B - Spread footing supported by driven piles.



Type C - Circular sheet pile driven to bedrock and filled with reinforced concrete.



Type D - CMP driven to bedrock and filled with reinforced concrete, resting on a rock socket penetrating into bedrock and filled with reinforced concrete.

Figure 4.2. Commonly used types of bridge pier foundations on the Kansas River.

alluvial material they pass through is necessary for the structural stability of the pier, there will be a limiting depth of degradation and local scour beyond which the piers are structurally unsound. Evaluation of the structural stability of the piers is beyond the scope of this report and limited by the available data.

For piers of Type B, the combined general degradation and local scour should not exceed the top of the piles. If the driven piles become exposed, the pier may be in danger of failure. Depending on the design of the driven piles, some exposure during a flood may be allowable, but in general, this is considered an unsafe condition.

Local scour at bridge piers for a sand-bed channel with little armoring potential, such as the Kansas River, can be calculated from Neil's equation:

$$\frac{s}{d} = 2.0 \left(\frac{a}{d}\right)^{0.65} Fr^{0.43}$$

where s is the equlibrium local scour depth in feet, d is the depth of flow in feet, a is the pier width in feet, and Fr is the Froude number given by:

$$Fr = \frac{\upsilon}{\sqrt{gd}}$$

where υ is the velocity of flow in fps and g is the acceleration due to gravity.

Local scour and pertinent elevation data are given in Table 4.2. Further discussion of each bridge follows.

4.1.1 18th Street Bridge (RM 4.9)

The 18th Street Bridge is in the Missouri River backwater reach of the Kansas River. As shown in the results of the qualitative geomorphic analysis presented in the 1984 SLA report, this reach has experienced little net aggradation/degradation. Future problems with scour at bridge piers should be relatively minor for bridges in this reach.

This bridge has Type B pier foundations. Therefore, the combined local scour and general degradation should not exceed the elevation of the bottom of the pier footing. The highest pier footing bottom elevation at the 18th Street bridge is 695 feet msl, while the thalweg elevation is 705. Therefore, local scour and general degradation cannot exceed 10 feet at this location

Table 4.2. Bridge Data.

Bridge	River Mile	Date of Blueprint	Pier[1] Type	Elevation (ft. msl)[2] Bedrock	Thalweg	Bottom of Pier Footing[2]	Minimum Burial Depth of Pier Footing (ft)	Estimated Local Scour[3] (ft)
18th Street	4.9	1957	B	675	705	695	10	6
Turner	9.3	1955	A	681	715	680	35	16
I-435	15.5	1976	A	702	730	696	34	11
Hwy 7 at Bonner Springs	20.3	1982	D	709	734	708	26	16
Atchison, Topeka and Santa Fe RR at Bonner Springs	21.2	1938	B	711[5]	735	737	0	16
Desoto	31.0	1969	A	738	751	736	15	10
Eudora	42.4	1963	C	740[5]	779	738[5]	41	21
Sardou Ave.	83.0	*[6]	A	835	842	832	10	7[4]
Atchison, Topeka and Santa Fe RR at Topeka	83.7	1938	B	826[5]	846	856	0	12
Kansas Ave.	84.2	*[6]	C	820	849	819	30	22
Topeka Avenue	84.4	1937	C	795	851	794	57	22
SSW RR at Topeka	84.5	1939	B	795[5]	852	860	0	22
Westgate	87.7	1951	A	859	862	854	8	3[4]

[1]See Figure 4.2
[2]For highest pier in the main channel
[3]For 100-year flood (as controlled by reservoirs)
[4]Local scour to bedrock
[5]Approximate value
[6]Blueprints were available, but not dated

without possibly endangering the bridge piers. Using Neil's equation and hydraulic parameters from the 1984 SLA report, the maximum expected local scour at this location is approximately six feet for the 100-year flood. Therefore, general degradation must not exceed four feet to ensure the safety of the bridge piers.

4.1.2 Turner Bridge (RM 9.3)

This bridge has Type A pier foundations, which ensures that it cannot be undermined, due to local scour and/or general degradation. Since the local scour from the 100-year event (16 feet) is nearly 50 percent of the footing burial depth (34.9 feet), the scour could have an impact on the piers' lateral stability. Assessment of this type of stability should be performed by a structural engineer.

4.1.3 I-435 Bridge (RM 15.5)

Type A pier foundations support the I-435 Bridge. The bridge is a relatively new one with the available drawings dating to 1976. Local scour for this bridge is estimated at 11 feet for the 100-year flood event. Since the minimum depth of burial for the footings is 34 feet, the bridge has adequate protection against local scour at the present thalweg elevation. The continuity model results (Table 3.7) predict significant future degradation for this reach. The combination of degradation plus local scour could endanger the bridge's stability during the passage of future large flood events.

4.1.4 Highway 7 Bridge at Bonner Springs (RM 20.3)

The Highway 7 Bridge has Type D piers with 9-foot diameter rock sockets penetrating 10 feet into the bedrock. This bridge is new--construction was underway when the 1983 aerial photos were taken. Hence the foundation has not been affected by the 8.5-foot drop in stage elevation from 1950-1973 (see Section 3.3.5.1, SLA 1984). The foundation has adequate scour protection for the 100-year flood at present thalweg elevations. However, based on the future rates and locations of dredging operations presented in this report, this reach of river will continue to experience significant degradation. The combination of degradation plus local scour could endanger the bridge's stability during the passage of large flood events in the future.

4.1.5 Atchison, Topeka, and Santa Fe Railway Company Bridge at Bonner Springs (RM 21.2)

This railroad bridge has a Type B foundation with creosoted timber piles. Drawings dating back to 1938 show the bottom of the pier footings (top of pile caps) buried well below the channel bed at elevation 742. The degradation which this reach has experienced (See 4.1.4 above) has resulted in exposed timber piles at several piers. Repair work was performed on the bridge in 1975 to solidify these exposed piles. Sheet piling was driven around the perimeter of 3 piers, rock was added to fill the space between the sheet piling and the piers, and grout was pumped to fill in the voids. The work was performed at a cost of $73,000. The piles were made solid to a depth of 5 feet below the original pile caps or to elevation 737 (2 feet higher than the present thalweg). Information on the elevation to which the piles were originally driven is not available, but a company representative claimed the timber piles were 25 to 30 feet long. (This is consistent with the SSW Railroad Bridge in Topeka, which was built in the same period). This would put the bottoms of the piles at approximately elevation 718-713 or 17 to 22 feet below the present thalweg. Local scour for the 100-year flood is estimated to be 16 feet for this bridge, enough to endanger the structure at present thalweg levels. This bridge is in the same reach of river as the Highway 7 Bridge and will likely experience significant degradation in the future. The combination of degradation plus local scour during the passage of large flood events in the future would very likely cause the bridge to fail.

4.1.6 Desoto Bridge (RM 31.0)

The Desoto Bridge has Type A piers. Since the depth of bedrock from the channel bottom is less than 10 feet in places, it is reasonable to assume that the piers were designed to be structurally sound without the resistive forces of the alluvium. Therefore, this bridge should be unaffected by sand and gravel dredging even though the results of the continuity model (Table 3.7) predicts excessive degradation (15 feet) from 1985 to 2015 for Rate E dredging.

4.1.7 Eudora Bridge (RM 42.4)

The Eudora Bridge has Type C piers with the circular sheet pile cells 15 feet in diameter, driven to bedrock approximately 45 feet below the channel bed, and filled with reinforced concrete. Consequently, the piers should not fail due to undermining even under the most severe conditions. Since the piers are so wide, it appears that the slenderness ratio is low enough that lateral stability will not be a problem even with the estimated 21 feet of local scour. The continuity model predicts three feet of degradation for this reach of river from 1985 to 2015 with Rate E dredging.

4.1.8 Sardou Avenue Bridge (RM 83.0)

Available plans indicate that the Sardou Bridge has Type A piers, with footings one to two feet below the bedrock surface. Failure due to undermining should not occur. The lateral stability of the bridge should not be endangered by the three feet of general degradation associated with Rate E dredging for 30 years (Table 3.7).

4.1.9 Atchison, Topeka, and Santa Fe Railroad Company Bridge at Topeka (RM 83.7)

This railroad bridge has Type B foundations built on creosoted timber piles. The drawings for this bridge data to 1938 and show the bottom of the footings (and top of the piles) to be situated approximately flush with the channel bottom. Riprap was placed on all sides of the footings to reduce local scour. The thalweg is situated in the center of two piers and is at a lower elevation than the top of the wooden piles. The available drawings do not indicate the elevation of the bottom of the piles but a company representative stated that these piles were the same 25 to 30 foot-long type that were used on the railway bridge at Bonner Springs. Local scour is estimated at 12 feet for this bridge so it should be structurally safe during the passage of the 100-year flood (it survived the 1951 flood). Table 3.7 indicates degradation should not exceed three feet for this bridge, which would not endanger its stability.

4.1.10 Kansas Avenue Bridge (RM 84.2)

The Topeka Bridge has Type C piers and should not be susceptible to undermining. The present thalweg is at the same elevation as the top of the sheet pile cells or at approximate elevation 849 msl. Local scour is estimated to be 21 feet for the 100-year flood, but because the piers are so wide, the slenderness ratio is low, and the lateral stability should not be a problem. The continuity model results predicts three feet of general degradation in this reach for Rate E dredging.

4.1.11 Topeka Avenue Bridge (RM 84.4)

The Topeka Avenue Bridge has a Type C foundation with 20-foot diameter caissons situated on bedrock approximately 57 feet below the thalweg. The width of the piers results in a low slenderness ratio so the lateral stability of the piers should not be a problem. Local scour is estimated at 22 feet for this bridge while degradation is not expected to exceed 3 feet for this reach of river.

4.1.12 SSW Railroad Bridge at Topeka (formerly called the Chicago/Rock Island and Pacific Railroad)

The SSW Bridge has Type B pier foundations built on 25-foot long creosoted timber piles. The bridge was built around 1921 and drawings dating to 1939 show that sheet piling has been added around the perimeter of each pier. Concrete was placed inside the sheet piling at one of the piers.

As with the Atchison, Topeka, and Santa Fe Railroad Company Bridge at Topeka, the bottom of the footings (top of the piles) were originally located flush with the channel bottom. Riprap was placed on all sides of the footings to reduce local scour. The thalweg is located between 2 piers at a lower elevation than the top of the piles. Adding the sheet piling has increased the pier width resulting in an estimated 22 feet of local scour for the 100-year flood. Degradation is not expected to exceed 3 feet for this reach, but the combination of local scour plus degradation could endanger the bridge's stability during future large flood events.

4.1.13 Westgate Bridge (RM 87.7)

The Westgate Bridge has Type A piers, so undermining should not be a problem. Depth to bedrock is very shallow for this bridge, particularly along the southern bank (less than 10 feet). Therefore, lateral restraint from the alluvium is probably not necessary for structural stability. General degradation from dredging is not expected to exceed three feet for this reach and therefore should not affect the bridge's stability.

4.2 Weirs and Dams

4.2.1 Johnson County Weir (RM 14.9)

The Johnson County Weir is a dumped-stone structure built in response to a rapidly degrading channel bed. Its purpose is to raise the water-surface elevation upstream of the weir enough to ensure adequate operation of the

Johnson County municipal water intake structure located adjacent to and upstream of the weir. Because of the approximate 8- to 10-foot drop across the structure, critical velocity (i.e., free overfall) occurs for all but extreme flood events. Assuming critical velocity occurs for the 100-year event, velocities of 18 fps may occur. Using Shield's relation, it can be shown that this velocity will move particles less than or equal to approximately one foot in diameter. Field inspection indicates that the d_{50} of stone used to construct the weir is one to two feet in diameter. Therefore, many of the smaller-sized stones in the structure will be dislodged during the 100-year flood. The results of the continuity model (Table 3.7) predict substantial degradation for Reach 19 immediately downstream of the weir for most of the extraction rates examined. The substantial degradation below the weir, combined with the local scour and movement of smaller-sized stones during large flood events, could seriously endanger the stability of the structure.

4.2.2 Bowersock Dam at Lawrence (RM 51.8)

A stability analysis of the Bowersock Dam has been performed by the COE. It is believed that the structure rests on a timber crib foundation and the analysis indicated that the factor of safety for sliding failure was marginal. Additionally, the structure has been steadily eroding and has required considerable repairs in recent years. Therefore, dredging activities below the structure have been carefully monitored and limited. It is recommended that dredging activities in the reach immediately below the dam continue to be very limited in extent due to the probable instability of this structure.

4.3 Pipeline Crossings

Numerous gas, oil, sewer, and water lines cross the Kansas River. The locations of these pipelines are given on Table 4.1 and Figure 4.1. Pipeline owners were contacted and pipeline crossing elevations obtained. When these elevations were checked against thalweg elevations, approximately half the pipeline crossings were found to be 5 to 15 feet above the present channel thalweg. In several cases, the pipeline crossing elevations were 5 to 10 feet above the historical thalweg elevation at the time the pipelines were built. For this reason, the accuracy of the obtained pipeline crossing elevations is suspect, and the adequacy of burial of the lines impossible to determine. The discrepancy of pipeline and channel thalweg elevations is most likely due to lack of a common datum between the pipeline construction survey and the river surveys.

Since data on specific pipelines are suspect, the depth of burial of pipelines in general will be addressed. Experience gained by SLA in evaluating the design of the 36-inch force main sewer crossing at Topeka shows that, for the conditions at Topeka, pipelines buried 10 feet or lower below the thalweg will be adequately protected against general scour during the 100-year flood. However, for a given pipeline, local conditions must be evaluated and considered before the adequacy of burial can be determined. Specifically, conditions which must be considered include the presence of bridges and dredge cuts, which may induce local scour or headcutting that could expose the pipeline during a flood.

The combined effect of contraction scour due to contraction of the flow through a bridge opening and resulting increase in velocity and local scour at the piers may lower the bed sufficiently to uncover any pipelines located directly above or below bridges. Evidence of headcutting due to dredge cuts on the Kansas River was presented in the 1984 SLA report and discussed in Chapter II of this study. Headcutting at the upstream end of dredge cuts is a natural and expected effect due to the local lowering of the channel bed at the dredging site.

A discussion of an adequate buffer zone for dredging near pipeline crossings is given in Section 5.3 of this report. Determination of adequate burial depth for pipelines near bridges or other structures is site specific since the contraction scour and local scour at piers are functions of the bridge design.

V. RECOMMENDATIONS FOR INCLUSION IN THE MANAGEMENT PLAN

Included in this chapter are three main sets of recommendations to be considered for inclusion into the COE's management plan. The recommendations deal with allowable dredging rates based on given levels of impact, a monitoring program to evaluate the impacts of dredging activities on the morphology of the river and structures in the river, and minimum buffer distances from critical structures and landforms. The allowable dredging rates are concerned with limiting the general degradation to a given level of impact, while the minimum buffer distances deal with local scour associated with critical structures and landforms.

5.1 Allowable Dredging Rates

As discussed in Chapter III, the continuity model MINING was used to calculate the depths of aggradation/degradation for the period 1985 to 2015 using five different rates of sand and gravel mining. For each rate of dredging, the levels of impact after 10, 20, and 30 years were determined and are shown in Tables 3.8 to 3.10. These three tables show the level of impact corresponding to a given rate of dredging. However, for the management of sand and gravel dredging operations, the information needed is the rate of extraction that will produce a given level of impact. The approach taken to produce this information is outlined below.

Table 5.1 lists the total amount of sand and gravel that was dredged on a reach by reach basis for the five different extraction rates presented in Chapter III. These values are difficult to compare because all the reaches are of different lengths. The values from Table 5.1 were divided by their reach lengths and then by the 30 years of simulation to arrive at average annual amounts of material dredged on a per mile basis. These values are presented in Table 5.2. When these new average annual values are plotted against the depths of aggradation/degradtion, the trend becomes apparent. Figure 5.1 shows a plot of the annual extraction rate versus the depth of degradation for Reach 17. Once these plots have been established, it is possible to make an estimate of how much material can be dredged for any given depth of degradation. Estimates of the average annual amount of sand and gravel that can be removed over a 30-year period to achieve a specified level of impact are presented in Table 5.3.

The rates in Table 5.3 are based on the results from Reaches 16, 17, and 19, reaches that all have dredging upstream from them, (note that Reach 18 is

Table 5.1. Total Amount of Sand and Gravel Dredged (million tons) from 1985 to 2015.

Reach Number	River Mile	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	0	1.80	3.60	3.60	5.71
2	147.5 - 121.5	0	0.60	1.20	1.20	1.90
3	121.5 - 101.2	0	0	0	0	0
4	101.2 - 101.0	0	0	0	0	0
5	101.0 - 93.0	0	0	0	0	0
6	93.0 - 88.0	0	3.00	6.00	6.00	9.52
7	88.0 - 80.6	0	3.00	6.00	6.00	9.52
8	80.6 - 64.5	0	0	0	0	0
9	64.5 - 51.9	0	0	0	0	0
10	51.9 - 51.7	0	0	0	0	0
11	51.7 - 46.7	0	1.80	2.52	3.45	5.47
12	46.7 - 41.6	0	0	0	0	0
13	41.6 - 34.8	0	0.60	0.84	1.15	2.38
14	34.8 - 31.0	0	5.80	8.35	11.50	19.25
15	31.0 - 26.5	0	7.50	10.87	14.95	24.72
16	26.5 - 22.0	0	5.20	7.52	10.35	18.99
17	22.0 - 15.1	0	9.20	13.37	18.40	26.09
18	15.1 - 14.9	0	0	0	0	0
19	14.9 - 12.4	0	4.70	6.69	9.20	12.52
20	12.4 - 12.2	0	0	0	0	0
TOTAL		0	43.20	66.96	85.80	136.07

Table 5.2. Total Amount of Material Dredged on a Unit Basis (million tons per year per mile).

Reach Number	River Mile	Length Miles	Rate A Mining	Rate B Mining	Rate C Mining	Rate D Mining	Rate E Mining
1	170.4 - 147.5	22.9	0	0.003	0.005	0.005	0.008
2	147.5 - 121.5	26.0	0	0.001	0.002	0.002	0.002
3	121.5 - 101.2	20.3	0	0	0	0	0
4	101.2 - 101.0	0.2	0	0	0	0	0
5	101.0 - 93.0	8.0	0	0	0	0	0
6	93.0 - 88.0	5.0	0	0.020	0.040	0.040	0.063
7	88.0 - 80.6	7.4	0	0.014	0.027	0.027	0.043
8	80.6 - 64.5	16.1	0	0	0	0	0
9	64.5 - 51.9	12.6	0	0	0	0	0
10	51.9 - 51.7	0.2	0	0	0	0	0
11	51.7 - 46.7	5.0	0	0.012	0.017	0.023	0.036
12	46.7 - 41.6	5.1	0	0	0	0	0
13	41.6 - 34.8	6.8	0	0.003	0.004	0.006	0.012
14	34.8 - 31.0	3.8	0	0.051	0.073	0.101	0.169
15	31.0 - 26.5	4.5	0	0.056	0.081	0.111	0.183
16	26.5 - 22.0	4.5	0	0.039	0.056	0.077	0.141
17	22.0 - 15.1	6.9	0	0.044	0.065	0.089	0.126
18	15.1 - 14.9	0.2	0	0	0	0	0
19	14.9 - 12.4	2.5	0	0.063	0.089	0.123	0.167
20	12.4 - 12.2	0.2	0	0	0	0	0

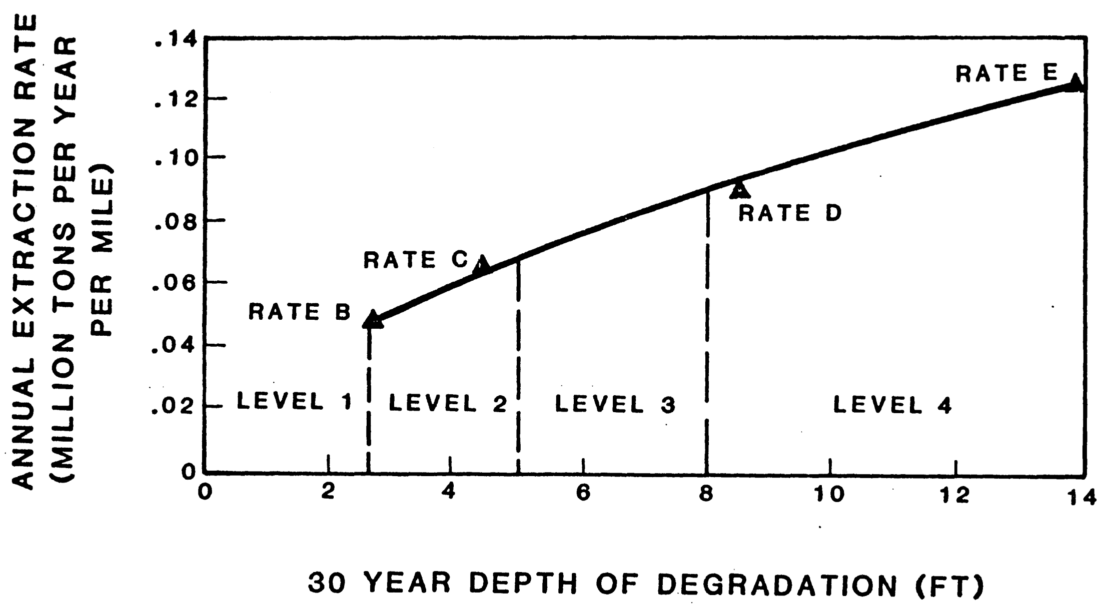


Figure 5.1. Degradation of Reach 17 for different rates of dredging.

Table 5.3. Annual Dredging Rates (30-Year Period) Associated With a Given Level of Impact For Reaches Where Dredging Occurs Immediately Upstream.

Level of Impact	Depth of Degradation (ft)	Annual Dredging Rate Over a 30-Year Period (million tons per year per mile)
1	0 to 2.5	0 to 0.05
2	2.5 to 5.0	0.05 to 0.07
3	5.0 to 8.0	0.07 to 0.09
4	> 8.0	> 0.09

Table 5.4. Annual Dredging Rates (30-Year Period) Associated With a Given Level of Impact For Reaches Where No Dredging Occurs Immediately Upstream.

Level of Impact	Depth of Degradation (ft)	Annual Dredging Rate Over a 30-Year Period (million tons per year per mile)
1	0 to 2.5	0 to 0.08
2	2.5 to 5.0	0.08 to 0.12
3	5.0 to 8.0	0.12 to 0.15
4	> 8.0	> 0.15

a control reach - Johnson County Weir). As discussed in Section 3.3, the cause of degradation in these reaches is due to both the dredging in the reaches themselves, and the reduced supply of sediment coming into the reaches due to upstream dredging. More material can be removed from Reaches 14 and 15 because these reaches are farther upstream and have a greater supply of inflowing sediment. The estimated average annual amount of sand and gravel that can be removed from Reaches 14 and 15 are presented in Table 5.4.

In the five different extraction rates considered in Chapter III, Reaches 11, 12, and 13 were not heavily dredged. If the level of dredging were increased in these reaches, then the supply of sediment flowing into Reaches 14 and 15 would be reduced. Under such conditions, the allowable dredging rates for Reaches 14 and 15 should be lowered to those values presented in Table 5.3. The values in Table 5.4 would then be applicable to Reaches 11, 12, and 13.

The two reaches at Topeka (Reaches 6 and 7) were also not heavily impacted by the five different dredging rates. Dredging at Topeka could be allowed to increase by three percent a year for 30 years without exceeding Level 2 impacts.

The average annual dredge rates in Tables 5.3 and 5.4 should only be used as a guide for estimating how much material can be removed from each reach. These rates are average annual rates that were developed by dividing the total amount of material dredged by the 30 years of simulation. These rates can be increased for reaches where dredging will occur for less than the next 30 years. If dredging will occur for only 10 or 20 years, then the rates in Tables 5.3 and 5.4 could be multiplied for factors of 3.0 and 1.5, respectively. Once the decision is made as to what level of impact is acceptable, then these average annual rates can be used to develop a first estimate of how much dredging can be allowed in each reach, and the continuity model MINING can then be used to "fine tune" the scenario. As part of the conditions for granting a dredging permit, it is recommended that the COE specify the maximum amount of sand and gravel that can be removed from the permit area.

Because the depth of degradation within a reach depends on both the rate of dredging in the reach itself plus the upstream dredging, there are many combinations of dredging rates and locations that will result in the same level of impact within a given reach. Fortunately, by using the continuity model, it is relatively simple to investigate all probable rates and locations of dredging operations and determine what the corresponding levels of impacts will be throughout the river.

5.2 Monitoring Program

A considerable amount of data already exists for analyzing the hydrology, hydraulics, and erosion and sedimentation of the Kansas River. However, some revisions to the present data collection system would make the data more compatible with its end use. A monitoring program is suggested which would serve the above-mentioned purpose. The program consists of three main components. The following outline documents the essential level of effort that should be directed towards data collection. These data will provide basic information that is required to assess the impacts of dredging operations, data for detailed mathematical models of water and sediment routing, and data to better understand the response of the physical system. The system-related data are essential to answer more complex questions about the river, including the effects of the federal reservoirs on tributary sediment loadings as well as the impact of gravel mining.

5.2.1 Data Required to Monitor the Local Impacts of Dredging Operations

The first component of the monitoring program identifies the data required to establish and monitor the local impacts of the dredging operations. Five different data collection items are included in this first component.

1. Keep a continuous record of dredge locations, quantity of material extracted on an annual basis, depth of mining, and the aerial extent and location of dredge cuts.

2. Establish cross sections throughout the permit area to monitor for possible headcuts. Cross sections should be located at the extreme upper and lower ends of the permit area as well as at least every 2,000 feet within the permit area. These cross sections should be resurveyed annually.

3. Table 4.1 of the 1984 SLA report lists 135 stations at which cross-sectional information is available. Approximately 50 of these 135 stations are ranges with permanent monuments on either side of the river bank. Most of the ranges are located between Desoto and the mouth of the Kansas River and immediately below the junction of the Delaware and Kansas Rivers. The COE can no longer gain access to many of the ranges because of farming operations. All ranges below Desoto should be resurveyed annually. Other ranges should be resurveyed every five years and

after major flow events. A minimum of three additional ranges should be established between Desoto and Eudora.

4. Samples of bed material should be collected in each range at the time of the resurvey to identify if any changes are occurring in the size of the bed material. At least 10 samples should be collected at each range. The samples may be analyzed for size distribution individually or they may be composited and analyzed as a single sample. Bank-material samples vary greatly and are not considered as important to the analysis.

5. Based upon data gathered from the monitored ranges and the cross sections within the permit areas, establish the profile of the river bed along the thalweg. Plot this profile and compare it with the thalweg profiles of previous years. Show the locations of bridge foundations, pipeline crossings, geologic controls, and man-related controls. Identify significant changes, such as headcuts or endangered structures.

5.2.2 Data Required to Upgrade the Water and Sediment Continuity Model

The following data collection items are required for upgrading and fine tuning the continuity model

1. The USGS presently collects suspended sediment data at Wamego, Lecompton, and Desoto. The COE presently has observers collecting suspended sediment samples at Wamego, Lecompton, Desoto, and Turner Bridge. In previous years, the COE also had its observers collecting sediment samples at Fort Riley, Lawrence, and Eudora. The COE sediment sampling should be continued, but the station at Turner Bridge could be abandoned because this data is so strongly influenced by the stage of the Missouri River. The observer station at Desoto is of primary importance due to the anticipated dredging activity in this reach of river.

2. Under the present COE observer sediment sampling program, samples are collected at a single vertical in a cross section. An alternative program, practiced by the USGS involves taking a composite sample from several verticals in the cross section. While it may not be feasible for the COE observers to collect composite samples, the COE should consider having COE staff or the USGS collect composite samples several times yearly from each station.

5.2.3 Data Required to Better Understand the Long-Term Response of the Entire River System to the Cumulative Impacts of Dredging

The following data collection items are recommended to better understand the long-term response of the entire river system:

1. Establish and maintain a ground-control system for aerial photography. Obtain aerial photos of the river on a five-year basis and after major floods. The photos may be black and white or color. The scale chosen should be the same for every series of photos (approximately 1 inch = 400 feet). The photos can be overlaid to evaluate lateral migration of the river.

2. Continue plotting up the changes in the USGS rating curves as shown in Figures 3.24 to 3.26 of the 1984 SLA report. Establish similar curves for Desoto and Eudora. As a minimum, the validity of these rating curves should be checked annually and after major floods.

3. Resurvey the sections shown in Table 4.1 that are cross sections (as opposed to permanent ranges) every 10 years or after every major flood.

4. Establish new ranges upstream from Bowersock Dam. At a minimum, ranges should be located every 10 miles.

5. On the major tributaries that are affected by large federal reservoirs (the Republican, Smoky Hill, Big Blue, and Delaware River), collect data similar to that collected on the Kansas River (cross section, ranges, bed-material samples, and aerial photos). Use this data to estimate changes in profile, degradation downstream of the dams, bank stability, and supply of sediment to the mainstem.

5.3 Minimum Buffer Distances

In the dredging permit, the COE currently prohibits dredging within certain distances of structures. The following are typical special conditions of a dredging permit, "the permittee agrees not to dredge within 500 feet of any levee centerline, bridge pier or abutment, or water-supply intake; nor within 200 feet of any dike, revetment, or other structure built by or authorized by the U.S. Government; nor within 100 feet of any normal bank, island, or tributary mouth without special authorization."

The above buffer distances were originally developed for dredging operations on the Missouri River and they have proven to be satisfactory on that river. It is recommended that these buffer distances be added into the management plan, with one modification and several additions. Prohibiting dredging within 500 feet of a levee centerline would seem to be overly conservative, since the Kansas River is considerably narrower than the Missouri River. This buffer could be reduced to 350 feet for the Kansas River.

The 1983 aerial photos were examined to determine the locations of islands to which the above 100-foot buffer distances should apply. The following islands were identified:

a. RM 5.3
b. RM 15.1
c. RM 37.3
d. RM 46.3
e. RM 80.8
f. RM 90.5

It is recommended that the following additions be added to the special conditions of a dredging permit.

1. No dredging shall be allowed within 500 feet of any pipeline that is buried less than 10 feet deep without written authorization from the pipeline owner. The buffer could be decreased to 200 feet for pipelines that have more than 10 feet of burial depth. Unlike bank revetment, and levees that are situated along the river banks, bridge piers and pipeline crossings are located in the middle of the river at right angles to the flow. Dredge cuts lengthen more at their upstream and downstream ends rather than at their sides, so buffer distances for structures such as bridge piers and pipelines need to be larger than buffers for structures located along the river banks. The buffer length for pipelines can be smaller than the buffer for bridge piers since there is not the additional component of degradation due to local scour at a pipeline as there is at a bridge.

2. No dredging should be allowed within 200 feet of the bank on the outside of sharp bends in the river. The typical flow pattern around a bend is for a point bar to form on the inside of the bend, while the main current is directed at the banks on the outside of the bend. This results in the common occurrence of erosion of the banks around the outside of bends. Allowing dredging to occur on the point bars can actually alleviate some of the erosional potential on the opposite banks. Referring to Figure 5.2 (from Appendix B of 1984 SLA report), it is recommended that this increased buffer distance be applied to the following bends:

   a. RM 26 - 29
   b. RM 34 - 35.5
   c. RM 35.5 - 37
   d. RM 38.5 - 40
   e. RM 40.5 - 42
   f. RM 43 - 44
   g. RM 46.5 - 48

   These bends all have a radius of curvature of 4,000 feet or less.

3. Dredging should only be allowed on the point bars on the insides of bends in reaches of the river that have experienced significant lateral erosion in recent history. Two such areas which were identified on the channel migration maps (Figure 5.2) are Eudora bend (RM 40.5 - 42) and a bend immediately upstream from the Mud Creek junction (RM 47.5 - 48.0).

4. Based on the results of Chapter II, and the average length of existing dredging operations, it is recommended that the maximum length of any dredging operation be limited to one mile. To prevent different dredging operations from possibly linking together and forming one long continuous dredge cut, it is recommended that a minimum buffer zone of 2,000 feet be maintained between successive permits. If a situation arises where the 2,000-foot buffer is thought to be excessive, then a hydraulic analysis similar to that performed in Chapter II should be performed.

The selection of adequate buffer distances is a qualitative procedure based on engineering judgement, experience with the response of numerous other

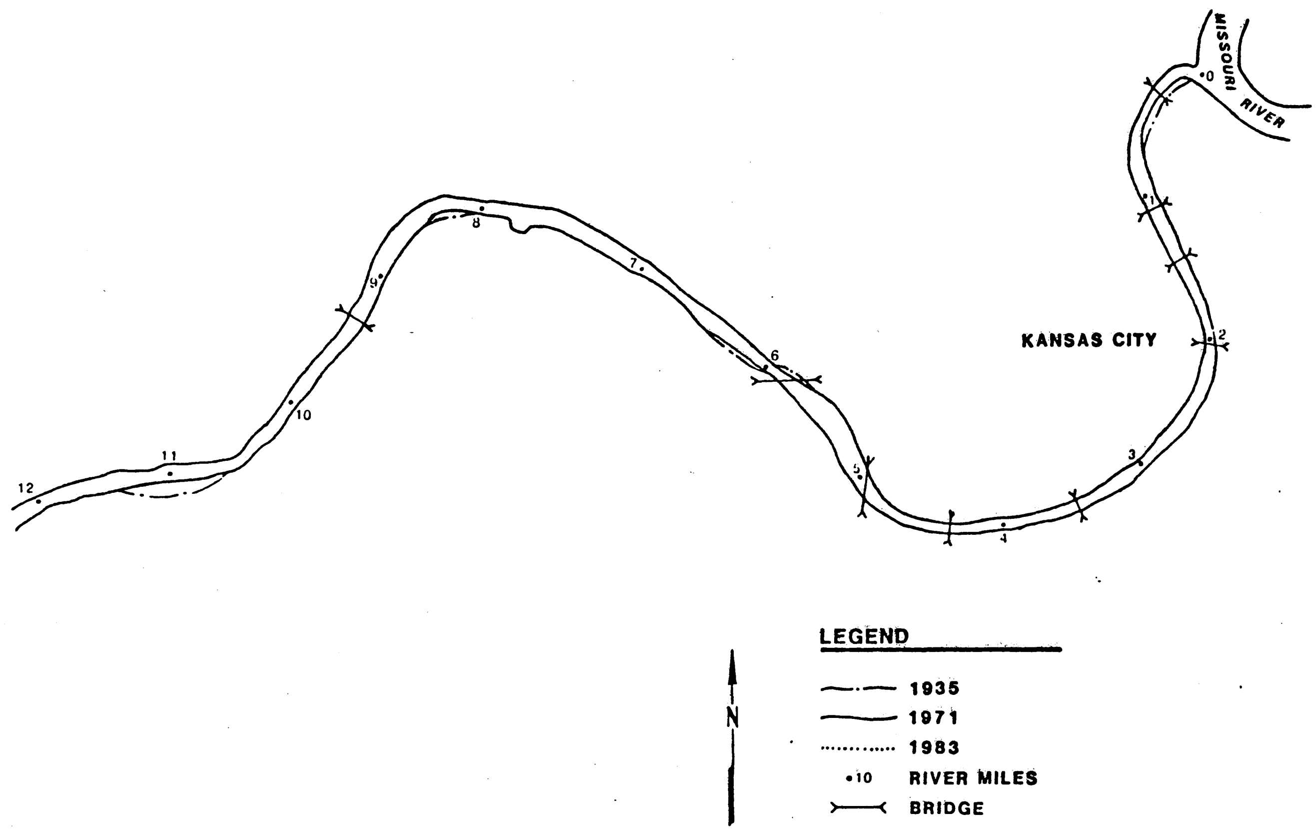


Figure 5.2. Historic channel migration maps.

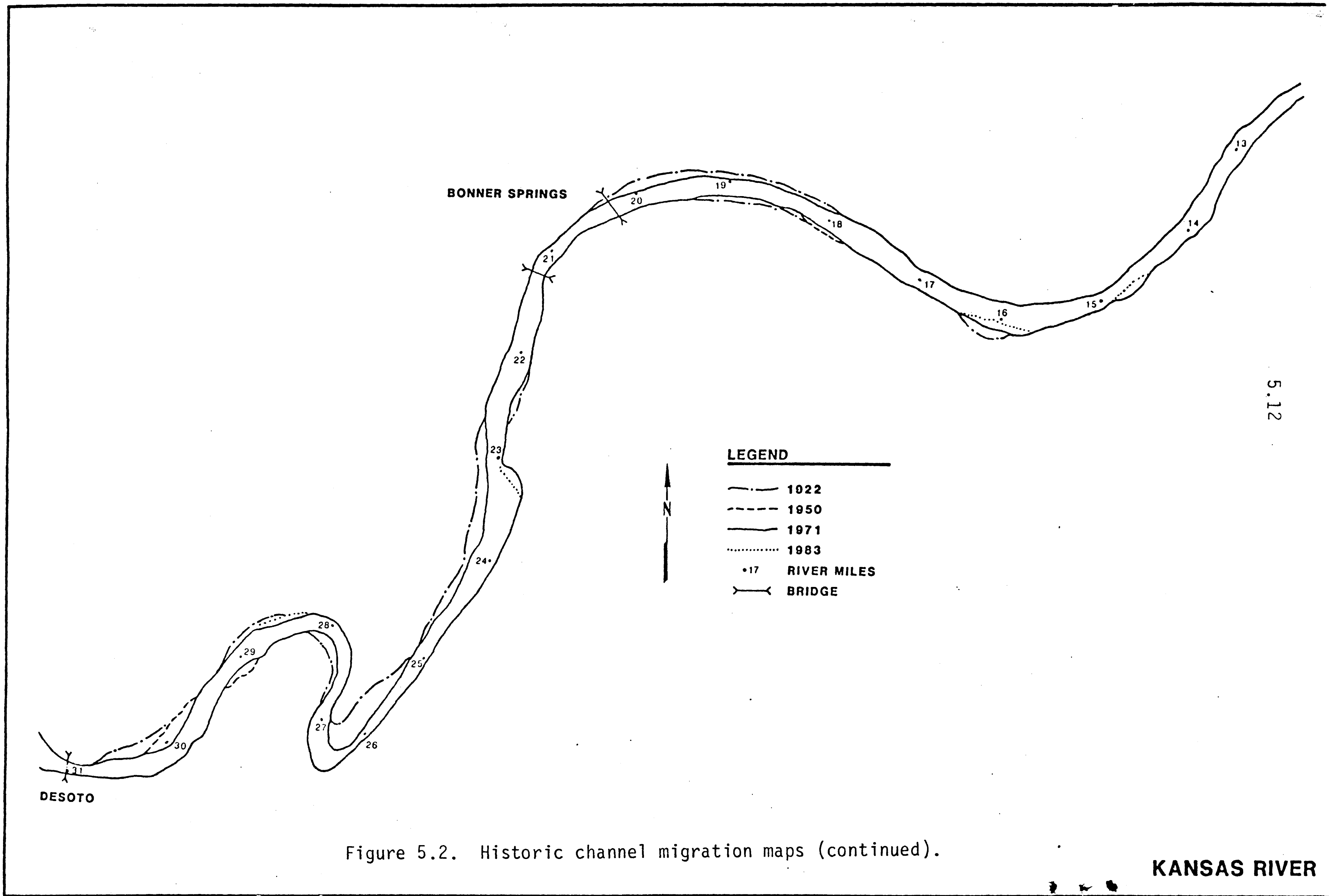


Figure 5.2. Historic channel migration maps (continued).

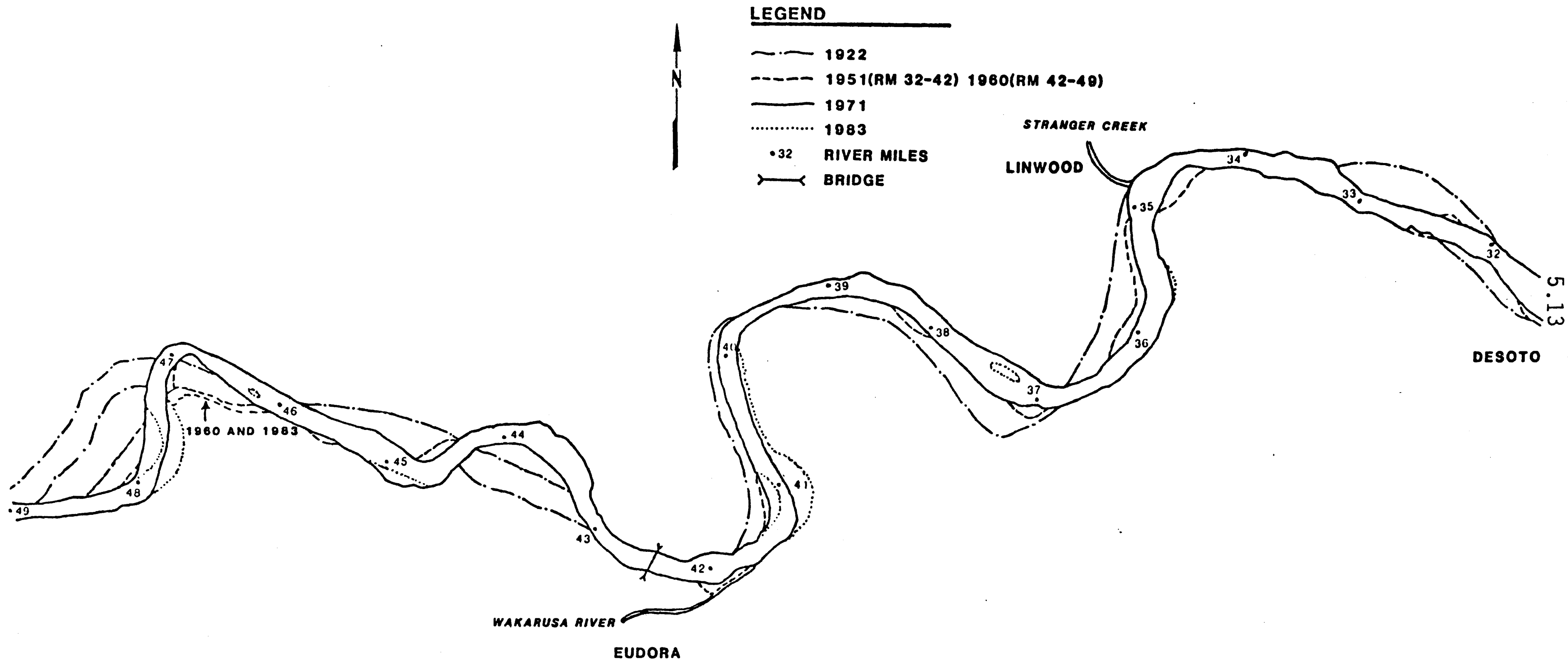


Figure 5.2. Historic channel migration maps (continued).

KANSAS RIVE

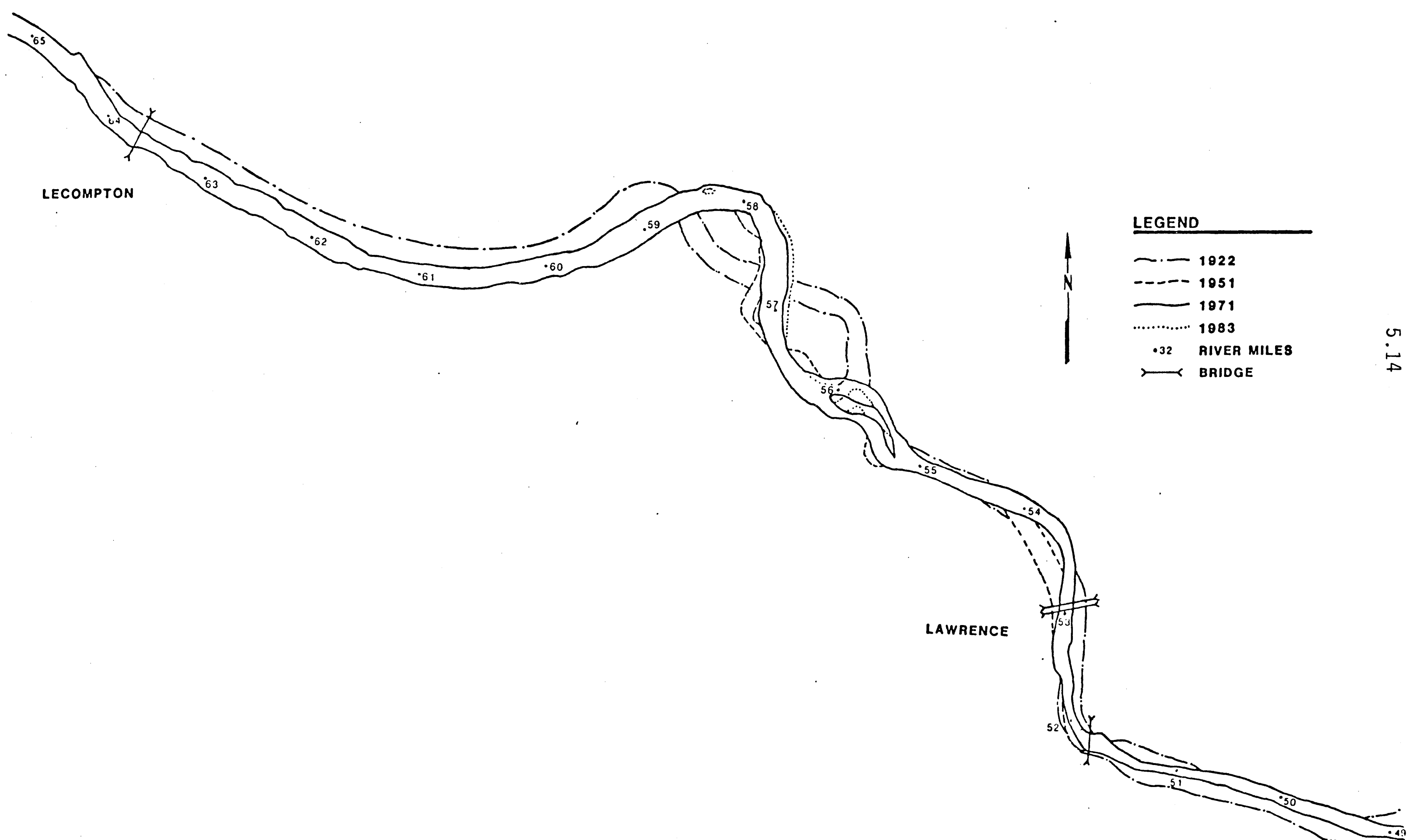


Figure 5.2. Historic channel migration maps (continued).

KANSAS RIVER

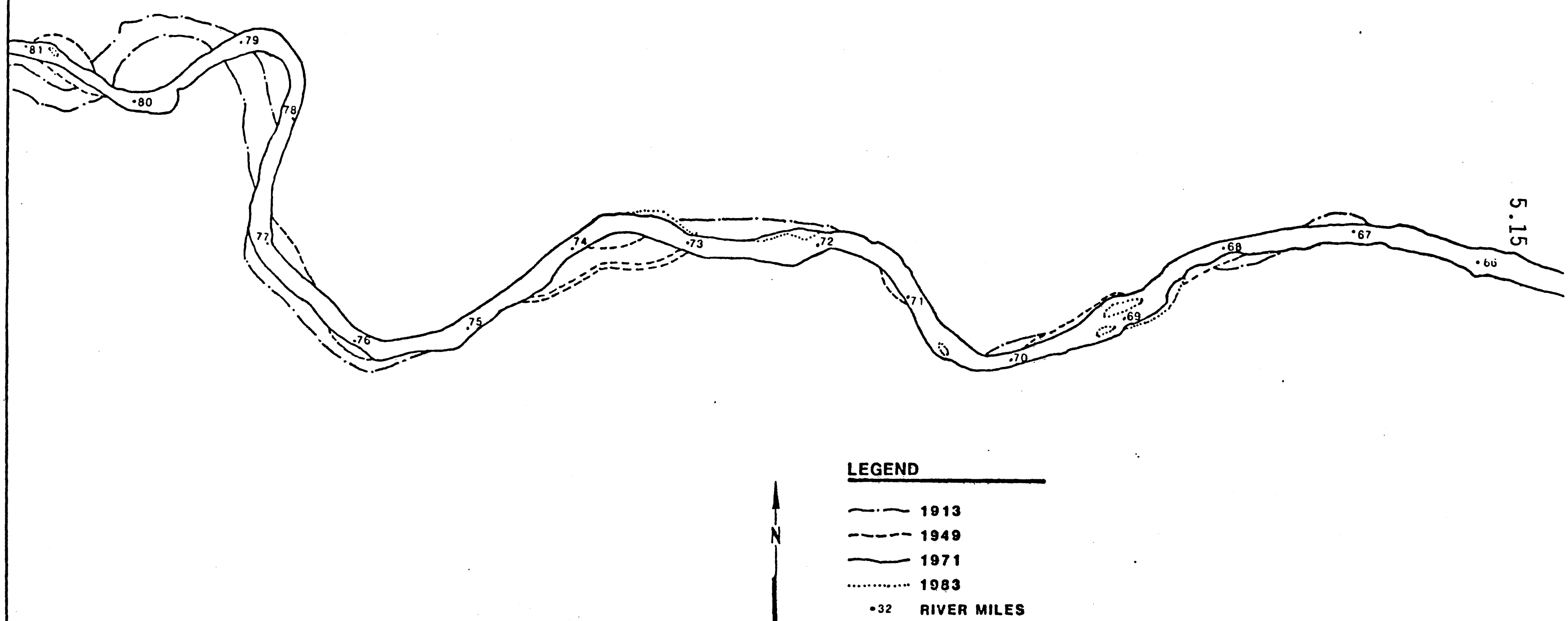


Figure 5.2. Historic channel migration maps (continued).

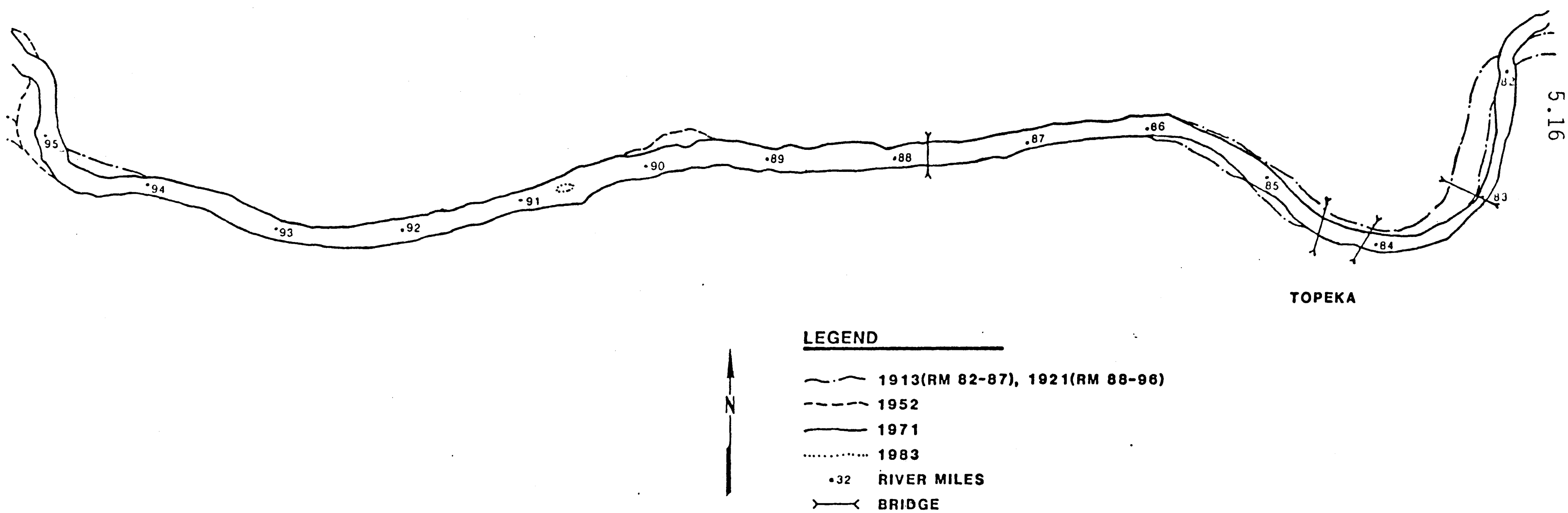


Figure 5.2. Historic channel migration maps (continued).

KANSAS RIVER

river systems, and interpretation of data and observations on the Kansas River. It is recommended that the COE incorporate the above-mentioned buffer distances into its management plan and then use the field data gathered from the monitoring program to evaluate their adequacy.

5.4 Interaction Between the Recommendations

Three main sets of recommendations are presented in this chapter for inclusion into a comprehensive management plan. These recommendations should not be viewed as independent of each other. Rather, the COE will need to coordinate the information obtained from each set of recommendations for the management plan to be most successful. For example, the buffer distances presented in Section 5.3 were necessarily based on engineering judgment. Once field data collected from the monitoring program becomes available, it should be used to verify these buffer distances or modify them as required. Similarly, certain features of the continuity model could be updated as more field data is collected from the monitoring program. The present flow-duration curve is based on 40 years of synthesized data. It should be revised in the future to reflect the hydrology that has actually occurred since closure of the federal reservoirs on the Kansas River tributaries. Included in the monitoring program is the continuation of the suspended sediment sampling program. After a few more years of suspended sediment sampling, the transport equations should be re-checked and the coefficients adjusted as necessary to match the observed changes in the base level of the river.

VI. SUMMARY AND CONCLUSIONS

The purpose of this report was to provide recommendations and pertinent data to the Kansas City District, COE that can be used as a basis for both regulating sand and gravel dredging activities on the Kansas River and for evaluating individual dredging applications. Presented in the study are three sets of recommendations for the COE to incorporate into the management plan.

The first set of recommendations is the concept of regulating the total amount of sand and gravel that can be dredged from each reach according to a given level of impact (none, minor, moderate, or major impact). A continuity-based water-and sediment-routing model was developed and used to estimate what the level of impact will be for five different rates and locations of dredging for the period, 1985 to 2015. From this information, extraction rates were provided for regulating the volume of dredging to achieve the level of impact selected by the COE. The model and documentation on its usage have been supplied to the COE so that they may investigate any future scenarios not specifically covered in this study.

The second set of recommendations is a monitoring program for data collection. Twelve data collection items were discussed in terms of their importance, how their results should be interpreted and the frequency with which the data should be collected. In the event that funding for data collection is limited, each item was prioritized according to the following three categories:

1. Data required to monitor the river system's local response to dredge cuts.
2. Data required for updating and "fine tuning" the continuity model.
3. Data required to understand the overall long-term response of the river system to the cumulative impacts of dredging.

The final set of recommendations identifies a series of buffer distances in which no dredging should be allowed. The current buffer distances, developed for dredging operations on the Missouri River, were evaluated. One modification and three additions were suggested.

These recommendations are not independent of each other. As discussed in Section 5.4, the COE will need to coordinate the information obtained from each set of recommendations for the management plan to be most successful.

Feedback from the monitoring program should be used for verifying and modifying both the continuity model and the minimum buffer distances.

In conclusion, we believe that incorporating these three sets of recommendations into the management plan will provide the basis for a plan that considers the interests of all parties involved, including the commerical dredging industry, public interests, non-dredging private interests, and concerned government agencies.