Case Study: Channel Stability of the Missouri River, Eastern Montana

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Abstract: The construction of Fort Peck Dam in the 1930s on the Missouri River, eastern Montana, initiated a series of changes in hydrologic conditions and channel morphology downstream from the dam that impacted channel stability. Impacts included streambed degradation of up to 3.6 m and substantially altered magnitude, frequency, and temporal distribution of flows. To investigate the effects of the altered flow regime and bed degradation on bank stability, two independent bank-stability analyses (one for planar failures, the other for rotational failures) were performed on 17 outside meanders. Both included the effects of matric suction and positive pore-water pressures, confining pressures, and layering. Instability occurred from the loss of matric suction and the generation of positive pore-water pressures. In this semiarid region, such hydrologic conditions are most likely to occur from the maintenance of moderate and high flows (greater than 425–566 m^3/s) for extended periods (5–10 days or more), thereby providing a mechanism for saturation of the streambank. For the postdam period, average annual frequencies of flows maintained above 566 m^3/s for 5- and 10-day durations are 149 and 257% greater, respectively. The analyses indicated that 30% of the sites were susceptible to planar failures while 53% of the sites were susceptible to rotational failures under sustained moderate- and high-flow conditions, while under a worst-case rapid drawdown scenario, 80% of the banks were susceptible to failure. Despite the negative effects of the altered flow regime, analysis of maps and aerial photographs shows that closure of Fort Peck Dam has resulted in a fourfold reduction of the average rate of long-term channel migration between the dam and the North Dakota border.

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Introduction

''The current ran at five miles per hour usually, but it sped up when it encountered encroaching bluffs, islands, sandbars, and narrow channels. The level was springtime high, almost flood stage. Incredible to behold were the obstacles—whole trees, huge trees, oaks and maples and cottonwoods, that had been uprooted when a bank caved in; hundreds of large and thousands of smaller branches; sawyers, trees whose roots were stuck in the bottom and whose limbs sawed back and forth in the current, often out of sight; great piles of driftwood clumped together, racing downriver...innumerable sandbars, always shifting; swirls and whirlpools beyond counting. This was worse than the Mississippi'' (Ambrose 1996, p. 140). These are the conditions along the Missouri River as experienced by Captain Meriwether Lewis and the Corps of Discovery in May 1804. Granted this passage refers to the Missouri River in the reaches just upstream of St. Louis, but it is interesting to note that almost 200 years ago river observers were referring to the effects of bank erosion.

Now, in the 21st century, stability of the Missouri River is still an important concern for local landowners and government agencies responsible for river management. A September 1995 field reconnaissance study of a 305 km reach of the Upper Missouri River between Fort Peck Dam, Mont. [river kilometer (RKM) 2,850] and the confluence of the Yellowstone River (RKM 2,545) indicated that approximately 50% of the riverbanks in this reach exhibit evidence of recent geotechnical failure and instability (Darby and Thorne 2000).

The closure of Fort Peck Dam in 1937 caused changes in the delivery of water and sediment to downstream reaches of the Missouri River. Previous studies have quantified channel changes, rates of bank erosion, and channel migration rates [U.S. Army Corps of Engineers (USACE) 1976; McCombs-Knutson Associates, Inc. 1984; Englebardt and Waren 1991; Wei 1997; Pokrefke et al. 1998] and identified bank-erosion mechanisms and the extent of bank instability (Simon and Darby 1996) for representative reaches between the dam and the North Dakota border (RKM 2,551). However, data describing the geotechnical characteristics of representative soil series and bank sections were not obtained and quantitative analyses of bank stability were not attempted. Accordingly, this paper presents analyses of the underlying causes of streambank and channel instability in the Fort Peck reach. A new planar-failure bank stability model incorporating detailed soil and pore-water pressure controls (Simon et al. 2000) is used in conjunction with a commercially available rotational failure model [(SLOPE/W 1998) (Note: Mention of trade names or commercial products in this article is solely for the purpose of pro-

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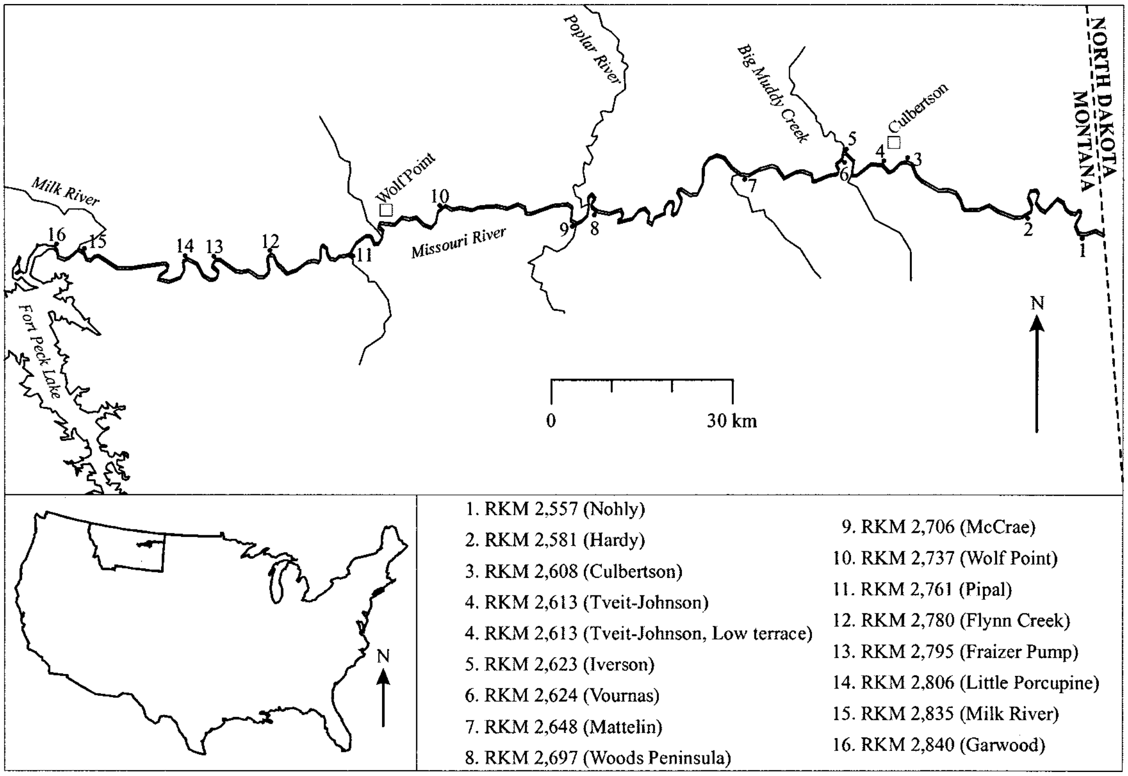

Fig. 1. Fort Peck Reach—location of study sites, tributaries, and main towns

viding specific information and does not imply recommendation or endorsement by the U.S. Department of Agriculture)] and more traditional analyses to examine the following issues:

1. Flow regimes before and after closure of Fort Peck Dam;
2. The impacts of changing flow regimes on bed- and bank-erosion processes;
3. The controlling factors in streambank stability in the reach; and
4. The relative stability of the channel banks.

Results indicate that dam closure has caused significant bed degradation in the study reach and, in particular, an increase in the occurrence of long-duration, medium- and high-stage flow events. Both have had deleterious effects on back stability. Analyses confirm that banks with low cohesion and erodible toes are particularly unstable and that those with high cohesion, few cracks, and unerodible toes are most stable. In addition, maintenance of high flows can cause bank saturation, eliminating matric suction and creating positive pore-water pressures significant enough to promote instability. Despite negative impacts on bank stability, postdam channel migration rates are almost four times less than predam rates.

General Characteristics of Study Area

The climate in northeastern Montana is semiarid, characterized by cold winters and hot summers, with an average annual precipitation of about 360 mm. The drainage area above the dam is about 149,000 km^2, while that contributing to the reach below the dam is about 93,300 km^2. About 66% of this is in the watershed of the Milk River, which empties into the Missouri River about 16 km downstream from the dam (USACE 1952; Wei 1997).

The study reach (Fig. 1), although generally meandering, contains several straight sections and numerous midchannel bars and islands (USACE 1952; Wei 1997) so that the overall channel pattern corresponds well to the "sinuous braided" river type (Brice 1984). The channel is between 244 and 366 m wide and occupies a floodplain that is between 1.6 and 6.5 km wide. Channel gradient is about 0.0002 and median bed-material size is about 0.25 mm, with occasional deposits of coarse gravel, cobbles, and dense clay.

Channel and Flow Response to Dam Construction

The major effect of Fort Peck Dam on channel geometry has been channel-bed degradation in the reach immediately downstream (Williams and Wolman 1984). Analysis of six cross sections located between the dam and 74.8 km downstream indicates that thalweg elevation decreased by as much as 4.6 m, with degradation being most rapid immediately after closure (Wei 1997). Channel widening at the six cross sections varied from 0% to 37% (Wei 1997).

Darby and Thorne (2000) estimated that 57% of the banks along the study reach were eroding by mass-wasting processes. Planar failures due to toe scour and oversteepening by fluvial undercutting were the most common mechanisms of failure. Comparison of 1975 and 1983 aerial photographs led to an annual-erosion estimate of 302 m^2/km (McCombs-Knutson Associates, Inc. 1984; Wei 1997). However, another study estimated that bank erosion in the reach led to an average annual loss of land of 1,182 m^2/km, almost four times greater than the first estimate (USACE 1995).

Changes in Bed Level

Dam closure typically initiates a characteristic sequence of changes in bed elevation in the downstream direction and, as a

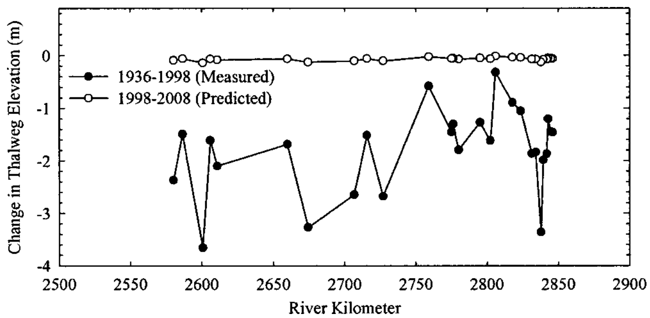

Fig. 2. Calculated changes in thalweg elevations between 1936 and 1998 with predicted erosion to 2008

result, has a significant impact on streambank stability by increasing bank heights. Bed-elevation data were obtained from the U.S. Army Corps of Engineers, Omaha District, and from Wei (1997) for the periods 1936, 1948, 1956, 1966, 1978, and 1994. Additional data originally derived from four gauging stations along the reach were also obtained from Williams and Wolman (1984).

As with other alterations to hydrologic regime or channel morphology, it is imperative to analyze bed-elevation data with respect to the predisturbed condition because response is most rapid immediately after dam closure (Williams and Wolman 1984). Data on predam conditions (1936 surveys) were obtained from the six measured cross sections (all within 74.8 km of the dam; RKM 2,776) and estimated for other sections in the reach downstream of RKM 2,776 by assuming vertically stable predam conditions and establishing linear regressions between RKM and elevation (see Simon et al. 1999).

Total bed-level changes were obtained by comparing 1936 bed elevations with those surveyed in 1994. Decreases in average bed elevations varied from 1.8 m near the dam to about 0.2 m near RKM 2,759, before increasing to 1.07 m toward RKM 2,639 (Simon et al. 1999). Decreases in thalweg elevations ranged from 0.2 m near RKM 2,806 to a maximum of 3.66 m near RKM 2,602 (Fig. 2). The changes represented by the thalweg data, particularly if the thalweg is adjacent to the bank, can be important in considering the most critical conditions regarding bank stability.

By extrapolating temporal changes in bed elevations utilizing a power function, future changes in both average and thalweg elevations are predicted to be less than 0.3 m over the period 1998 to 2008 (Fig. 2; Simon et al. 1999). Channel-bed degradation as a direct result of dam closure is, therefore, essentially complete. Results from four sites in this reach published by Williams and Wolman (1984) support these findings.

Changes in Flow Regime

Fort Peck Dam was closed in 1937, but it was not until 1942 that a minimum operation level was attained (Wei 1997). Releases from the dam were elevated somewhat between the late 1950s and the mid-1960s to fill downstream reservoirs during an extended drought (Shields et al. 2000). Hydropower units began operation in 1943 and 1961. Mean daily and peak flow conditions have undergone changes in magnitude and distribution since the closure of Fort Peck Dam in 1937. In general terms, the closure and operation of Fort Peck Dam resulted in a decrease in the magnitude of high flows, an increase in the magnitude of low flows, and a redistribution of high flows from the spring and early summer to the winter months.

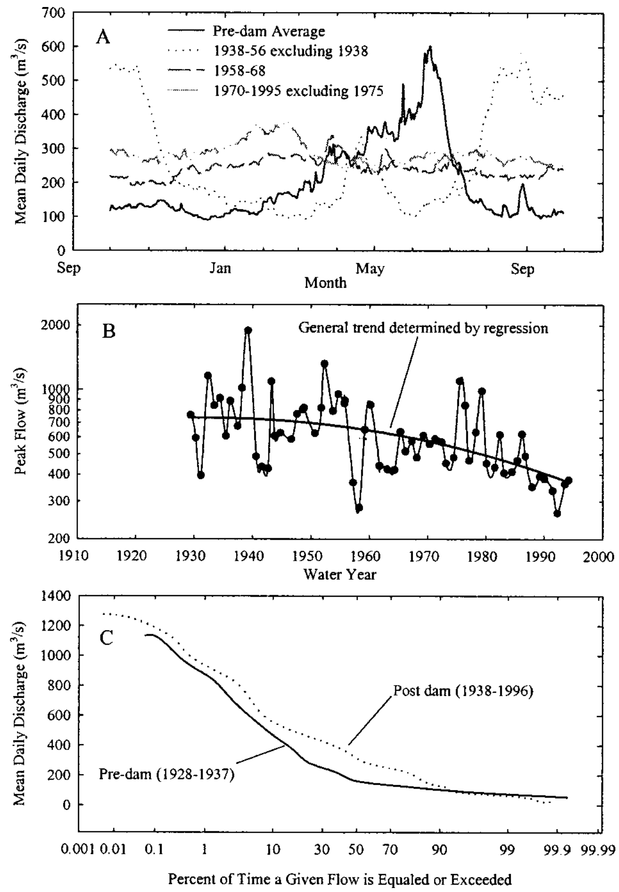

Fig. 3. Average of mean daily flows for four different periods at Missouri River at Wolf Point gauge showing (a) change in timing of high-flow season, (b) reduction of predam peaks, and (c) shift in flow-duration curve

To illustrate the changes in flow regimes, mean daily and peak discharge records of the U.S. Geological Survey (USGS) between 1928 and 1996 were analyzed for the gauge at Wolf Point, Mont., located at RKM 2,737, about 114 km downstream from the dam. Data were split into four periods: predam (1928–1937); postdam period I (1938–1956); postdam period II (1958–1967); postdam period III (1968–1995). Average daily flow values for each of the designated periods are shown in Fig. 3(a).

Predam annual hydrographs differ significantly in shape from the regulated flows of successive periods. In particular, it can be seen that high flows were shifted from June and July, reflecting snowmelt conditions, to between August and October (1938–1956). Subsequent to this, mean daily flows became more regular. More recently (1970–1995), peak mean daily flows have been shifted to the winter months (January and February) [Fig. 3(a)]. Through the four time periods, the average of the peak mean daily winter flows has steadily increased from about 142 to about 370 m^3/s through the 1990s. However, annual peak-flow data indicate a general decrease in the magnitude of peak flows following closure of the dam [Fig. 3(b)], signifying successful flood control.

An analysis of flow frequencies and durations for flows that now occur around 5% of the time (736 m^3/s), 60% of the time (283 m^3/s), and 90% of the time (142 m^3/s) was conducted (Table 1). Flow levels for these discharges were subsequently used in bank-stability analyses to represent a reasonable range of surface and groundwater conditions for the reach. The flow-duration series show higher discharges about 90% of the time, with mean

Table 1. Occurrence of Discharges Used in Bank-Stability Modeling and Average Number of Occurrences of 5- and 10-Day Duration Flows at Wolf Point Gauge, Showing Increased Occurrence of Long-Duration High Flows after Closure of Fort Peck Dam

Discharge	PREDAM 1928–1937				% total time	POSTDAM 1938–1996				% total time
Q	10-Day Duration		5-Day Duration		Q equaled	10-Day Duration	5-Day Duration			Q equaled
(m^3/s)	(% time)	(No./yr)	(% time)	(No./yr)	exceeded	(% time)	(No./yr)	(% time)	(No./yr)	exceeded
736	0.0	0.0	0.1	0.3	2.8	0.1	0.4	0.3	1.1	5.2
566	0.1	0.3	0.3	1.2	—	0.3	1.2	0.8	3.0	—
425	0.2	0.9	0.9	3.1	—	0.7	2.5	1.9	6.8	—
283	0.9	3.1	2.4	8.9	23.5	3.2	11.6	7.4	26.8	56.8
142	—	—	—	—	64.7	—	—	—	—	87.7

Note: Dash indicates data not available.

daily flows equal to or greater than 283 m^3/s now occurring on average 207 days/year, twice the note in the predam period [Fig. 3(c)]. This suggests the storage and redistribution of high spring runoff flows so that there are no extended periods of very low flows. Dam operation has also resulted in an increase in the frequency of moderate- and high-duration flows. As an example, for the postdam period average annual frequencies of flows maintained above 566 m^3/s for 5- and 10-day durations are 149 and 257% greater, respectively (Table 1).

Implications for Bank Stability

It is common along regulated rivers such as the Missouri for moderate and high stages to be maintained for a longer period of time than under predam conditions. If moderate to high stages are maintained for sufficient time, the near-bank region may become saturated by lateral infiltration at levels above low water. If stage is then decreased rapidly, a drawdown condition occurs, resulting in unfavorable pore-water pressure conditions. Additionally, the shifting of peak flows into the winter when parts of the river are frozen can have significant effects on channel morphology by generating positive pore-water pressures higher up the banks and by altering flow distributions, thereby providing a given discharge with a greater erosive power. However, if stage is reduced slowly, permitting positive pore-water pressures to dissipate, streambanks can sometimes maintain their strength and stability. The effects of pore-water pressures and confining pressures on bank stability have been explored by many other authors including Casagli et al. (1997), Simon and Curini (1998), Rinaldi and Casagli (1999), and Simon et al. (2000).

To illustrate the effects of high stage on soil moisture and pore-water pressures, it should be noted that clay banks have extremely low hydraulic conductivities. A value of 10^{-8} m/s is a reasonable representation for solid beds of clay without cracks or fissures (e.g., Rawls and Brakensiek 1985). This means that infiltration 0.9 m landward would occur in about three years, assuming zero head. However, since the banks along the study reach exhibit frequent cracks and macropores from desiccation, freeze-thaw, tension, rotted vegetation, etc., it is safe to assume that hydraulic conductivity is significantly greater. A value of 10^{-6} m/s is a reasonable estimate for silty materials and fissured clays (Rawls et al. 1992). Given this value and again assuming zero head, the zone of soil saturation can extend 0.9 m into the bank if high river stages are maintained for a 10-day period, and saturation of a 0.4 m deep zone can occur within 5 days. This scenario provides a mechanism for the maintenance of relatively high bank-saturation levels that can critically weaken streambanks in a semiarid region, since 0.4 and 0.9 m represent common failure block widths observed in the field.

Flow-duration information (Table 1) therefore indicates how often maintained surface-water levels could cause bank saturation at a given elevation to extend 0.4 and 0.9 m into the bank. Table 1 shows that predam, high-steady flows equaling or exceeding 566 m^3/s could not have caused 0.9 m of saturation into the bank, since flows of this magnitude were not maintained for long enough periods. During the postdam period, however, flows of 566 m^3/s or greater were sustained for 10 days or more 0.3% of the time, or on average just over once per year, allowing infiltration to help destabilize streambanks. It should be noted that the last time flow levels were maintained above 566 m^3/s for 5–10 days at the Wolf Point gauge was 1986. In the past 23 years this occurred in 1976, 1978, 1979, 1982, and 1986.

Five scenarios combining worst- and best-case combinations of groundwater and river stages were selected for modeling. The scenarios were as follows:

- Case 1. A river stage (RS) corresponding to the elevation of a flow of 736 m^3/s; groundwater elevation (GW) corresponding to the elevation of a flow of 736 m^3/s;
- Case 2. RS at 283 m^3/s; GW at 736 m^3/s;
- Case 3. RS at 283 m^3/s; GW at 283 m^3/s;
- Case 4. RS at 142 m^3/s; GW at 283 m^3/s; and
- Case 5. RS at 142 m^3/s; GW at 142 m^3/s.

Case 2, with high groundwater levels and relatively low flows in the channel, is considered to be a worst-case rapid-drawdown condition. Case 4 represents another drawdown case, but not as severe as Case 2. Conversely, Case 5, with low groundwater and surface-water levels, is considered the most conservative because of the important effects of negative pore-water pressures in enhancing bank strength. Elevations corresponding to the 283 and 736 m^3/s water surfaces were extracted for each site from measured water-surface profiles reported in Wei (1997). Elevations corresponding to the 142 m^3/s water surface were obtained by interpolating rating curves developed by Wei (1997) for seven gauges located between RKMs 2,608 and 2,843. For most of the banks, all groundwater levels above the elevation of the 283 m^3/s water surface would generate positive pore-water pressures on at least a portion of the assumed failure planes.

Site Selection

Data collection centered on characterizing the physical and geotechnical properties of a range of bank materials along particularly unstable channel sections. A primary consideration in site selection was that of soil series even though soil classification

Table 2. List of Study Sites, Representative Soil Series, and Date Visited

Site name	River kilometer	Soil series	Date tested
Nohly	2,557	Havrelon	09/09/97
Hardy	2,581	Lohler	09/10/97
Culbertson	2,608	Havrelon	09/08/97
Tveit-Johnson	2,613 (low terrace)	River Wash	08/19/96
Tveit-Johnson	2,613	Lohler	08/19/96
Iverson	2,623	Havrelon	08/20/96
Vournas	2,624	Lohler	08/20/96
Mattelin	2,648	Trembles	08/21/96
Woods Peninsula	2,697	Trembles	09/10/97
McCrae	2,706	Shambo	09/11/97
Wolf Point	2,737	Gerdrum	09/12/97
Pipal	2,761	Havre	08/21/96
Flynn Creek	2,780	Harlem	08/22/96
Fraizer Pump	2,795	Harlem	09/15/97
Little Porcupine	2,806	Harlem/Till	09/15/97
Milk River	2,835	Harlem	09/16/97
Garwood	2,840	Havre-Harlem	09/17/97

encompasses only the upper 1.52 m of the soil in what may be a 5 m high bank. Nevertheless, it was felt that by sorting the sites initially by soil series, information could be provided on the range of shear strengths of the upper part of the bank. During the field investigation, soils from eight soil series were evaluated at 17 sites on the outside of meander bends (Table 2; Fig. 1). Generally, these soils were fine grained and consisted of silty clay and clay loam, with some fine sand. Only the Riverwash and Trembles series consisted primarily of sand-sized materials.

Testing and Sampling of Bank Material

Direct measurements of the drained shear strength of the bank materials were made at each site with an Iowa borehole shear tester (BST). This instrument enables rapid in situ determination of soil shear strength (Lutenegger and Hallberg 1981). A series of tests was performed in each borehole at several depths as dictated by bank stratigraphy. Samples of streambank material were then removed from these boreholes to determine particle-size distribution, moisture content, and bulk unit weight. Finally, pore-water pressures were measured in situ at various depths with a miniature, digital tensiometer inserted into an ''undisturbed'' core. Results of the BST and other geotechnical tests are shown in Table 3.

Bank Stability Analyses

Two types of bank-stability analyses were conducted to compare the stability of the 17 study sites: a commercially available rotational failure analysis (SLOPE/W 1998) and the Agricultural Research Service (ARS) method for planar failures (Simon et al. 2000). To provide a more physically based analysis of bank-failure occurrence, conditions, and frequency, planar and rotational failures were modeled using geotechnical data specific to each bank layer (Table 3) and pore-water pressure distributions derived from assumed combinations of phreatic and water-surface (river stage) elevations. Suction values above the water table increased linearly upward away from the water table toward the floodplain surface. All elevations were referenced to the water-surface elevation of a given flow discharge.

Agricultural Research Service Method for Planar Failures

An analytical method that accounts for variations in bank material as well as the effects of pore-water and confining pressures was required to obtain a better understanding of the failure conditions in the study reach. A bank-stability algorithm for cohesive, layered banks has been developed by the ARS (Simon and Curini 1998; Simon et al. 2000), incorporating both the failure criterion of Mohr-Coulomb for the saturated part of the failure surface, and the failure criterion modified by Fredlund et al. (1978) for the unsaturated part of the failure surface.

The formulation for shear strength (S_r) utilizing the standard Mohr-Coulomb equation and incorporating the effects of matric suction produces (Fredlund et al. 1978)

$$S_r = c' + (\sigma - \mu_a)\tan\phi' + (\mu_a - \mu_w)\tan\phi^b \tag{1}$$

where c' = effective cohesion (kPa); σ = normal stress (kPa); μ_a = pore-airpressure (kPa); $(\sigma - \mu_a)$ = net normal stress on the failure plane (kPa); ϕ' = effective friction angle (deg); μ_w = pore-water pressure on the failure plane (kPa); $(\mu_a - \mu_w)$ = matric suction (ψ) (kPa); and ϕ^b = rate of increase in shear strength due to increasing matric suction (deg). The value of ϕ^b is generally between 10 and 20°, with a maximum value of ϕ' under saturated conditions (Fredlund and Rahardjo 1993; Simon et al. 2000).

The algorithm, developed by the ARS based on the limit equilibrium method, accounts explicitly for several additional forces acting on a planar failure surface that are not included in earlier bank-stability models such as that of Osman and Thorne (1988). These include the force produced by matric suction on the unsaturated part of the failure plane (S), the hydrostatic-uplift force due to positive pore-water pressures on the saturated part of the failure plane (U), and the hydrostatic-confining force provided by the water in the channel and acting on the bank surface (P) (Casagli et al. 1997; Simon and Curini 1998; Simon et al. 2000).

The hydrostatic-uplift (U) and confining (P) forces are calcu-

Table 3. Summary of Geotechnical Data Collected for Each Soil Unit for Bank-Stability Modeling at 17 Study Sites

River kilometer	Depth (m)	Failure surface depth[a] (m)	USCS	c_a (kPa)	c_u (kPa)	c' (kPa)	ϕ' (deg)	ϕ^b (deg)	ψ (kPa)	γ_{amb} (kN/m^3)	γ_{sat} (kN/m^3)
2,557	3.4	—	ML	17.8	—	13.2	30.1	17.0	15.0	14.2	21.4
2,557	5.1	4.7	CH-CL	9.1	—	7.3	29.1	17.0	5.8	17.9	22.2
2,581	2.7	—	CL	18.2	—	10.6	20.2	17.0	25.0	15.4	21.2
2,581	4.9	—	CH-CL	14.9	—	12.9	23.3	17.0	6.5	16.9	21.1
2,581	7.0	6.7	SM	1.9	—	1.7	32.9	17.0	0.5	16.3	20.9
2,608	2.4	—	CH-CL	17.3	—	8.7	29.5	17.0	28.0	16.6	21.3
2,608	3.2	5.2	CL	23.5	—	22.7	8.8	17.0	2.7	15.4	19.9
2,613 LT[b]	0.6	—	SM-ML	0.0	—	0.0	30.9	17.0	—	13.0	15.4
2,613 LT[b]	1.5	4.6	SP	0.0	—	0.0	26.4	17.0	—	16.9	21.1
2,613	4.1	—	CL	31.0	—	9.4	26.9	17.0	—	15.4	20.6
2,613	4.9	—	CL	32.3	—	31.5	5.5	17.0	—	16.9	20.8
2,613	5.5	5.9	SM	1.9	—	1.9	32.9	17.0	—	20.3	23.0
2,623	2.2	—	CL	16.2	—	12.5	28.3	17.0	—	15.8	20.7
2,623	3.7	—	SP	0.0	—	0.0	35.0	17.0	—	18.0	21.6
2,623	7.5	3.7	CL	—	77.8	—	0.0	17.0	—	21.0	21.6
2,624	2.1	—	ML-CL	16.0	—	8.5	20.9	17.0	—	16.6	21.3
2,624	2.4	4.6	SP	0.0	—	0.0	35.0	17.0	—	13.5	21.1
2,648	1.8	—	SM	0.0	—	0.0	38.1	17.0	—	13.5	20.0
2,648	2.1	—	ML	17.8	—	10.1	30.1	17.0	—	15.7	20.7
2,648	2.7	3.8	SP	4.6	—	0.0	35.4	17.0	—	16.5	21.3
2,697	3.4	—	SM	2.2		0.4	26.9	17.0	6.0	14.5	21.4
2,697	3.8	—	CL	—	77.8	—	0.0	17.0	6.0	21.0	21.6
2,697	4.0	5.9	SP	0.0	—	0.0	35.0	17.0	0.0	18.0	21.6
2,706	3.4	—	CL	9.2	—	0.0	33.6	17.0	30.0	14.1	20.9
2,706	4.1	—	SM	2.9	—	2.1	37.6	17.0	2.5	16.9	21.4
2,706	7.0	7.0	CL	33.0	—	32.2	14.0	17.0	2.5	17.1	21.4
2,737	2.7	—	CL	8.3	—	2.5	27.6	17.0	19.0	16.1	21.4
2,737	3.0	—	SC	0.2	—	0.0	35.0	17.0	6.0	15.4	21.1
2,737	6.7	6.1	CL	20.3	—	19.2	14.5	17.0	3.5	16.7	20.5
2,761	2.4	—	CL	0.0	—	0.0	37.7	17.0	—	14.8	21.0
2,761	3.8	—	CL-CH	25.1	—	22.3	13.4	17.0	—	17.3	21.4
2,761	5.5	5.5	SM	0.0	—	0.0	37.9	17.0	—	15.2	20.9
2,780	2.7	—	CL-CH	0.0	—	0.0	32.0	17.0	—	16.0	21.1
2,780	3.7	—	CH	21.0	—	8.5	26.1	17.0	—	16.9	21.0
2,780	5.5	5.5	CH	1.9	—	0.0	35.0	17.0	—	14.5	20.1
2,795	0.9	—	CL	9.2	—	8.1	28.8	17.0	—	15.4	20.4
2,795	1.2	—	CH	6.0	—	2.8	28.1	17.0	10.5	15.3	20.6
2,795	4.9	5.2	CL	9.2	—	8.1	28.8	17.0	3.5	15.4	20.4
2,806	1.8	—	CL-CH	10.2	—	5.0	33.2	17.0	17.0	15.2	21.1
2,806	6.1	6.4	CL	17.8	—	15.8	22.5	17.0	6.5	15.1	19.7
2,835	4.9	6.5	CH	29.7	—	27.7	9.9	17.0	6.5	16.3	20.2
2,840	2.8	—	CH	7.8	—	2.1	33.8	11.0	29.5	13.5	20.2
2,840	3.8	—	SM	1.9	—	1.9	32.9	11.0	—	14.5	20.1
2,840	4.3	4.3	CL	1.1	—	1.1	28.9	11.0	16.0	14.0	21.2
2,840	8.3	—	SM	1.9	—	1.9	32.9	11.0	—	14.5	20.1

Note: Dash indicates data not available.

[a]From bank top.

[b]Low terrace.

lated from the area of the pressure distribution of pore-water $(h_u \cdot \gamma_w)$ and confining $(h_{cp} \cdot \gamma_w)$ pressures (μ_w) by

$$U = \frac{\gamma_w h_u^2}{2} \quad (2)$$

$$P = \frac{\gamma_w h_{cp}^2}{2} \quad (3)$$

where $\gamma_w = 9.81\ \mathrm{kN/m^3}$; h_u = pore-water head (m); and h_{cp} = confining-water head (m). The loss of the hydrostatic-confining force (P) provided by the water in the channel is the primary reason bank failures often occur after the peak flow and on the recession of stormflow hydrographs.

Multiple layers are incorporated through summation of forces in a specific (ith) layer acting on the failure plane. The factor of safety (F_s) is (Simon et al. 2000)

Table 4. Minimum Factor of Safety for Planar Failures under Different Hydrologic Conditions

Site	Planar Failures				
	Minimum factor of safety for given hydrologic conditions				
	RS-736 GW-736	RS-283 GW-736	RS-283 GW-283	RS-142 GW-283	RS-142 GW-142
2,557	1.36	0.85	1.60	1.54	1.75
2,581	1.00	0.80	1.29	1.29	1.29
2,608	1.64	**1.41**	1.83	1.83	1.84
2,613 (low terrace)	0.52	—	0.81	—	0.91
2,613	1.23	0.83	1.58	1.50	1.67
2,623	1.49	0.61	1.92	1.92	1.92
2,624	0.77	0.11	1.12	1.09	1.20
2,648	1.05	0.80	1.43	1.43	1.43
2,697	1.20	0.95	1.56	1.56	1.56
2,706	1.52	**1.52**	1.52	1.52	1.52
2,737	1.23	0.99	1.44	1.42	1.52
2,761	0.94	0.57	1.27	1.26	1.36
2,780	0.81	0.60	1.16	1.16	1.16
2,795	1.63	0.97	1.88	1.82	2.01
2,806	1.92	**1.57**	2.15	2.11	2.28
2,835	2.85	**2.41**	2.90	2.89	2.91
2,840	0.99	0.64	1.30	1.30	1.35

Notes: RS=elevation of river at given discharge.
GW=elevation of groundwater equal to specified river discharge.

$$F_s=\frac{\Sigma\{c_i' L_i+(S_i \tan\phi_i^b)+[W_i\cos\beta-U_i+P_i\cos(\alpha-\beta)]\tan\phi_i'\}}{\Sigma[W_i\sin\beta-P_i\sin(\alpha-\beta)]} \tag{4}$$

where L_i=length of the failure plane incorporated within the ith layer (m); W_i=weight of the failure block (N); β=failure-plane angle (deg); and α=bank angle (deg). Eq. (4) represents the continued refinement of bank-failure analyses by incorporating additional forces and soil variability (Osman and Thorne 1988; Simon et al. 1991; Casagli et al. 1997; Simon and Curini 1998; Rinaldi and Casagli 1999; Simon et al. 2000).

Results of ARS Method for Planar Failures

Under the worst-case scenario (Case 2) all but four sites (RKMs 2,608, 2,706, 2,806, and 2,835) are unstable, with F_s less than 1.0 (Table 4). These four sites contain bank materials exhibiting some of the greatest cohesive strengths in the reach (Table 3), combined with relatively resistant bank-toe material. When both the river stage and groundwater levels are high (at the level corresponding to a 736 m^3/s flow), about 30% of the banks are still unstable. Results of all analyses are presented in Table 4, while Fig. 4 shows an idealized bank section displaying the modeled critical-failure surface.

Under the less critical drawdown scenario (Case 4) all of the modeled banks are stable, with the site at RKM 2,624 approaching a condition of instability. The shift from generally unstable to generally stable conditions is explained on the basis that saturation of the lower portion of the bank (as in Case 4) is generally below the base of the failure plane (Fig. 4). Because of this, the loss of suction at the base of the bank does not affect bank strength and F_s calculations along the failure plane.

Rotational Failures

Rotational failures were analyzed using *SLOPE/W*, a commercially available limit equilibrium method slope-stability software package (*SLOPE/W* 1998). Stability analyses of the Missouri River banks were performed using the Bishop method. This method allows evaluation of the stability of layered banks for a variety of slip-surface shapes and for a variety of pore-water pressure conditions and soil properties. In the Bishop method, the method of slices is employed to evaluate the forces between two adjacent slices (Bishop 1955). The F_s is found using Eq. (5)

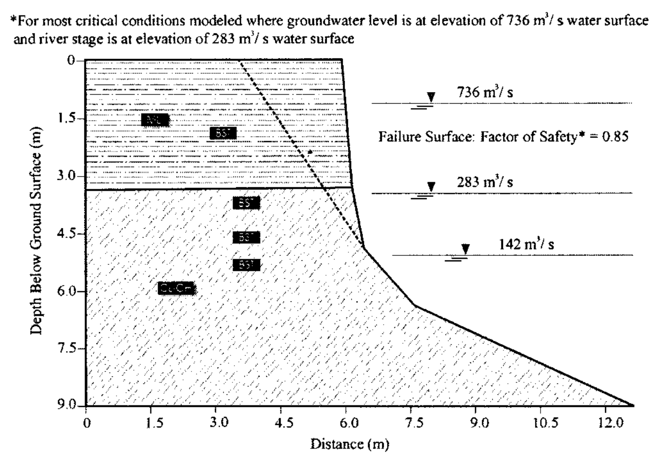

Fig. 4. Idealized bank section at river kilometer 2,557 (Nohly) showing modeled critical failure surface using Agricultural Research Service method for planar failures, factor of safety (F_s), river stage and groundwater levels, location of borehole shear tester tests, and differentiated soil units [ML=silt, CL—CH=clay (low to high plasticity)]

Table 5. Minimum factor of safety for rotational failures under different hydrologic conditions

Site	Rotational Failures: Minimum factor of safety for given hydrologic conditions				
	RS-736 GW-736	RS-283 GW-736	RS-283 GW-283	RS-142 GW-283	RS-142 GW-142
2,557	1.12	0.61	1.09	0.90	1.19
2,581	0.78	0.50	0.87	0.79	0.90
2,608	1.05	0.81	1.07	1.02	1.08
2,613 (low terrace)	0.52	0.10	0.56	0.52	0.68
2,613	0.96	0.79	1.27	0.95	1.07
2,623	1.29	0.34	1.52	1.50	1.67
2,624	0.83	0.34	0.87	0.78	0.89
2,648	0.76	0.75	0.80	0.75	0.96
2,697	0.91	0.88	1.52	1.53	1.63
2,706	1.64	**1.29**	1.57	1.65	1.59
2,737	0.97	0.97	1.60	1.59	1.69
2,761	1.06	0.83	1.17	1.17	1.25
2,780	0.75	0.63	1.06	1.06	1.15
2,795	1.32	0.95	1.40	1.32	1.48
2,806	1.53	**1.23**	1.59	1.54	1.68
2,835	2.45	**2.13**	2.31	2.25	2.29
2,840	0.72	0.51	0.94	0.94	0.97

Notes: RS=elevation of river at given discharge.
GW=elevation of groundwater equal to specified river discharge.

$$F_s=\frac{\Sigma(c'bR+[N-\mu_w b\tan\phi^b/\tan\phi'-\mu_a b(1-\tan\phi^b/\tan\phi')]R\tan\phi')}{\Sigma Wx} \tag{5}$$

where b=length of each slice at the base, in m; R=radius of the circular slip surface or the moment arm associated with the mobilized shear force for any shape of slip surface, in m; N =average normal stress at the base of each slice, in kN/m; W =total weight of a slice of width a and height h, in kN/m; and x=horizontal distance from the centerline of each slice to the center of rotation or to the center of moments, in m.

Results of Rotational Analysis

Under the worst-case scenario (Case 2) all but three of the modeled streambanks are found to be unstable. As with the planar-failure analyses, sites at RKMs 2,706, 2,806, and 2,835 appear stable (Table 5). All of these sites have cohesive materials at the bank toe, thereby slowing fluvial erosion and the steepening of the lower part of the bank. The sites that are unstable under all of the simulated conditions are located at RKMs 2,581, 2,613 low terrace, 2,624, 2,648, and 2,840. An example of the most critical failure surface, generated for the worst-case scenario, is shown in Fig. 5.

Channel Migration Rates

To provide a link between local bank stability and long-term lateral migration rates, an analysis of pre- and postdam channel migration was performed. Channel migration is defined here as the average rate of lateral migration along a river reach in dimensions of length per unit time. To calculate channel migration rates, available maps and photographs were organized to provide two predam coverages (1890 and 1913–1918) and two postdam coverages (1968–1971 and 1991). Because of limitations of available data, we determined channel migration by generating channel centerlines for each map or photo coverage, overlaying these traces, and calculating the areas enclosed by the intertwined digitized centerlines. Centerlines were used instead of banklines to reduce error introduced by different mapping conventions (Hooke 1997). Channel migration was then computed by dividing the sum of polygon areas for a given reach by the length of the earlier of the two centerlines and the time between the two coverages. Further details on the method of comparison and analysis can be found in Shields et al. (2000).

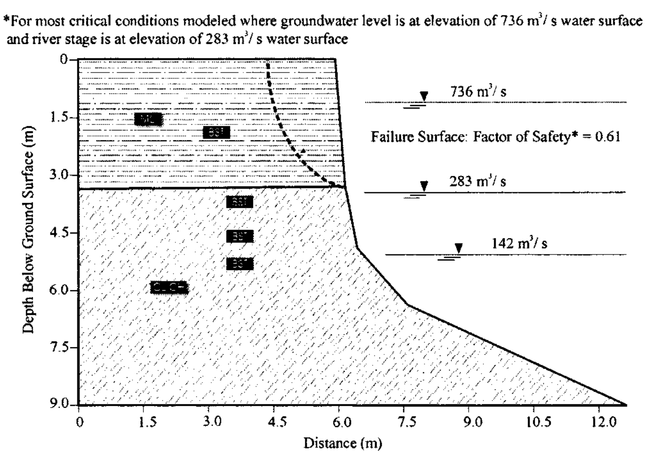

Fig. 5. Idealized bank section at river kilometer 2,557 (Nohly) showing modeled rotational critical failure surface, factor of safety (F_s), river stage and groundwater levels, location of borehole shear tester tests, and differentiated soil units [ML=silt, CL—CH=clay (low to high plasticity)]

Comparison of Pre- and Postdam Channel Migration

For the reach of the Missouri between Fort Peck Dam (RKM 2,850) and RKM 2,636, channel migration rates were considerably higher prior to dam closure than afterward. In this reach, about 3,079 ha of floodplain were enclosed by the two predam centerlines. This value includes areas affected by five channel avulsions that may not have been eroded by the river channel, but probably is a good estimate of the total area of floodplain reworked by the river. Only 722 ha were enclosed by centerlines derived from the two postdam coverages. Thus, the mean rate of predam channel migration was 6.6 m/year, while the mean postdam rate for the same reach between 1971 and 1991 was 1.8 m/year, representing a decrease of about 73% following dam closure.

Local "Activity" Rates

Local lateral bankline retreat rates at the 16 main study sites were calculated as the difference in area encompassed by digitized banklines bracketing time periods from 20 to 42.5 years. Coverages were selected from available maps dated from 1947 to 1971 and aerial photographs during the summer of 1991. In each case, the length of the subreach used for analysis was about 1.6 km. The net area encompassed by the banklines divided by the mean bank length provided an average distance that the bank had migrated over the period. Given the estimated error inherent in the analysis (0.2–0.9 m/year), average retreat distances ranged from about 0.0 m for two sites near the dam to close to 30 m at river kilometer 2,624. This error estimate does not include differences in bankline position created by differing definitions of bankline used by the makers of the 1947–1971 maps and our interpretation of 1991 aerial photographs. Dividing by the number of years between surveys normalizes the data and results in the rate of retreat in m/year (Fig. 6). For the ten sites upstream of RKM 2,636, the local channel migration rates calculated for the period prior to dam closure (via the method outlined above) have also been plotted in Fig. 6. Maximum bank retreat rates vary from 0.7 to 14.3 m/year, while average rates vary from 0.0 to 4.1 m/year. Fig. 6 clearly shows the considerable reduction in channel migration at these ten sites since closure, with an average reduction in retreat rate from a predam value of 9.4 m/year to a postdam value of 0.9 m/year.

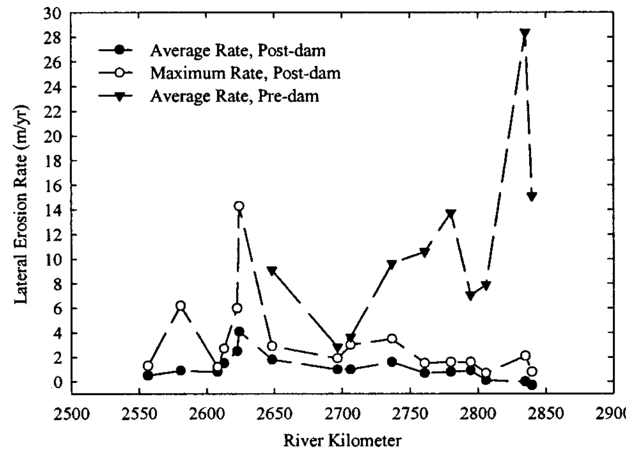

Fig. 6. Long-term lateral erosion rates for study sites

Summary of Results of Channel-Stability Analyses

Direct comparisons between absolute F_s values for the various hydrologic scenarios at a site (listed in Table 4) are difficult because *SLOPE/W* selects the most critical failure surface for the specified hydrologic conditions. Failure sizes and corresponding block widths are, therefore, of different sizes. Block widths for the simulated failures along the most critical failure surfaces were actually smaller than those for the modeled planar failures (Simon et al. 1999). This is contrary to what is predicted by theory (Carson and Kirkby 1972), but may be explained by the fact that the more cohesive clay materials are usually found low down the bank and, as rotational failure planes encompass a greater proportion of the lower units than do planar failure planes, the most critical rotational failure is often higher up the bank, resulting in smaller block widths.

Despite this, it is encouraging that totally different analytic techniques (the ARS method and *SLOPE/W*) show the same sites to be stable (or unstable) under similar conditions. When high groundwater levels are matched by high river levels, which occurs when flows are maintained long enough to allow saturation, sufficient positive pore-water pressures are developed to cause 30% of the sites to be susceptible to planar failures while 53% of the sites are susceptible to rotational failures. Results confirm previous findings that banks of low shear (and cohesive) strength as well as those with erodible bank-toe material (i.e., sand or silty sand) are generally unstable. Sites with low shear strengths are characterized by sandy or silty soils of the following soil series: Lohler, Havre-Harlem, Riverwash, and Trembles, while example locations of sites with sandy bank toes are RKMs 2,581, 2,613 low terrace, and 2,624. In contrast, banks with clay in the lowest portion of the bank and/or at the bank toe, and banks containing clays resistant to deep cracking, are the most stable. The site at RKM 2,835 is representative of the latter condition.

This research has highlighted additional hydrologic conditions that promote streambank instability. For the study reach

1. Banks are most stable during low stages and low groundwater heights corresponding to the elevation of the 142 m^3/s flow. Banks that are stable during low water often become unstable during drawdown conditions, such as a stage corresponding to 283 m^3/s and a groundwater height corresponding to 736 m^3/s, due to the increase of driving forces such as soil unit weight and positive pore-water pressure, and the reduction of resisting forces such as matric suction and confining pressure.
2. The destabilizing effect of high groundwater levels can be produced by maintaining flows greater than 425–566 m^3/s for five to ten days (or more). The frequency of flows of this magnitude and duration has increased since dam closure.

The hydrologic conditions that often result in bank instability are related to the frequency and duration of moderate and high flows. The elevation of flows greater than 283 m^3/s impinges on banks above the level of the base of the failure surface. The higher the flow level, the greater the proportion of the bank that is subject to saturation and positive pore-water pressures. Surface-water elevations associated with the 566 m^3/s and greater flows will have a significant negative impact on the shear strength of the bank material if the flow level is maintained for more than five days. Based on assumed values of hydraulic conductivity, saturation reaches 0.9 m into the bank mass after ten days. Rainfall in this region is not adequate to create these levels of saturation; thus maintenance of high flow levels creates the mechanism for failure. It is reasonable to assume that these effects would be greatly amplified during winter months when flow confinement by ice may generate pore-water pressures that are greater than those predicted hydrostatically.

Despite detrimental impacts on bank stability, closure of Fort Peck Dam has resulted in almost a fourfold reduction of the average rate of channel migration between Fort Peck Dam and RKM 2,636. Channel migration over the entire reach occurred at a rate 3.7 times greater prior to impoundment than presently, which is comparable to findings of others studying effects of similar flow regulation on other large, alluvial channels (Bradley and Smith 1984; Johnson 1992; Jiongxin 1997). This change is almost certainly due to the changes in peak-flow levels and frequency following dam closure, and may have negative ecological implications (Shields et al. 2000).

Conclusions

Analysis of bed-level trends in the Fort Peck Reach of the Missouri River has shown that bed degradation as a direct result of the 1937 closure of Fort Peck Dam has reduced thalweg elevations (and hence increased bank heights) by an average of 1.8 m. Future degradation as a result of dam closure is projected to be minimal. The application of two different streambank-stability models has shown that bank erosion by mass-wasting processes is an active process on the outside of meander bends. Instability occurs from the loss of matric suction and the generation of positive pore-water pressures during maintained high flows. In this semiarid region, flow modification ensures that bank saturation can occur several times during the year, but especially during the winter, when stages are maintained at a high level long enough to enable infiltration into the bank mass. Based on assumed rates of hydraulic conductivity, saturation 0.4 m into the bank can occur if flows are maintained for about 5 days or 0.9 m if maintained for 10 days. Once flows recede, a drawdown condition is created that triggers mass-wasting events. These types of drawdown conditions have increased in frequency since the closure of Fort Peck Dam in the late 1930s. A "worst-case" drawdown condition was represented by a phreatic surface elevation equal to the elevation produced by a 736 m^3/s discharge (presently equaled or exceeded about 5% of the time) and a water-surface elevation in the channel equal to the stage for a discharge of 283 m^3/s (presently equaled or exceeded about 60% of the time).

Despite negative impacts on bank stability, the closure of Fort Peck Dam has resulted in a fourfold reduction of the average rate of bank retreat. This reduction is probably linked to the reduction in overbank flows and a change to a more laterally stable channel pattern since channel incision. Avulsions such as neck cutoffs, which are included in the channel migration data, usually occur when overbank flows adopt advantageous routes (Brice 1973). The elimination or reduction of overbank flows in the postdam period implies that channel changes must now occur as a result of processes acting only on the banks and not on the banks, within the channel, or on floodplains as in the predam era. The scale of the reduction in bank retreat rates is comparable to those reported in other studies on the effects of flow regulation on large, alluvial channels, but direct quantitative comparison between bank-stability model results for the study reach and other large, regulated rivers is not appropriate. However, parallels can certainly be drawn from this study regarding the consequences of flow regulation on bank instabilities in other incised alluvial rivers.

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Notation

The following symbols are used in this paper:

b = length of each slice at base;
c_a = apparent or total cohesion;
c_u = undrained cohesion;
c' = effective cohesion;
h_{cp} = confining-water head;
h_u = pore-water head;
N = average normal stress at base of each slice;
R = radius of circular slip surface or moment arm associated with mobilized shear force for any shape of slip surface;
S_r = shear strength;
W = total weight;
x = horizontal distance from centerline of each slice to center of rotation or to center of moments;
α = bank angle;
β = failure-plane angle;
γ_{amb} = ambient (in situ) soil unit weight;
γ_{sat} = saturated soil unit weight;
γ_w = unit weight of water (9.81 kN/m);
μ_a = pore-air pressure;
μ_w = pore-water pressure;
σ = normal stress;
$(\sigma-\mu_a)$ = net normal stress;
ϕ^b = rate of increase in shear strength due to increasing matric suction;
ϕ' = effective angle of internal friction; and
ψ = matric suction $(\mu_a-\mu_w)$.

Subscripts

i = positive number index.

References

Ambrose, S. E. (1996). *Undaunted courage: Meriwether Lewis, Thomas Jefferson, and the opening of the American West*, Simon and Schuster, New York.

Bishop, A. W. (1955). "The use of the slip circle in the stability analysis of slopes." *Geotechnique*, 5, 7–17.

Bradley, C., and Smith, D. G. (1984). "Meandering channel response to altered flow regime: Milk River, Alberta and Montana." *Water Resour. Res.*, 20(12), 1913–1918.

Brice, J. C. (1973). "Meandering pattern of the White River in Indiana: An analysis." *Fluvial geomorphology*, M. Morisawa, ed., State Univ. of New York Press, Binghamton, N.Y., 178–200.

Brice, J. C. (1984). "River morphology and behavior: Problems of extrapolation." *River Meandering, Proc. of the Conf. Rivers '83*, ASCE, New York, 1–15.

Carson, M. A., and Kirkby, M. J. (1972). *Hillslope form and process*, Cambridge University Press, Cambridge, U.K.

Casagli, N., Curini, A., Gargini, A., Rinaldi, M., and Simon, A. (1997). "Effects of pore pressure on the stability of streambanks: Preliminary results from the Sieve River, Italy." *Management of landscapes disturbed by channel incision*, S. S. Y. Wang,

E. J. Langendoen, and F. D. Shields Jr., eds., Univ. of Mississippi Press, University, Miss., 243–248.

Darby, S. E., and Thorne, C. R. (2000). ''A river runs through it: Morphological and landowner sensitivities along the Upper Missouri River, Montana, USA.'' *Trans. Inst. British Geog. New Ser.,* 25(1), 91–107.

Englehardt, B., and Waren, K. (1991). ''Upper Missouri River bank erosion.'' *Unpublished Rep.*, North Dakota State Water Commission, Bismarck, N.D., and Montana Department of Natural Resources and Conservation, Helena, Mont.

Fredlund, D. G., Morgenstern, N. R., and Widger, R. A. (1978). ''The shear strength of unsaturated soils.'' *Can. Geotech. J.,* 15, 313–321.

Fredlund, D. G., and Rahardjo, H. (1993). *Soil mechanics of unsaturated soils*, Wiley, New York.

Hooke, J. M. (1997). ''Styles of channel change.'' *Fluvial geomorphology for river engineering and management,* C. R. Thorne, R. D. Hey, and M. D. Newson, eds., Wiley, Chichester, U.K., 237–268.

Jiongxin, X. (1997). ''Evolution of mid-channel bars in a braided river and complex response to reservoir construction: An example from the middle Hanjiang River, China.'' *Earth Surf. Processes Landforms,* 22(10), 953–65.

Johnson, W. C. (1992). ''Dams and riparian forests: Case study from the upper Missouri River.'' *River,* 3(4), 229–242.

Lutenegger, A. J., and Hallberg, B. R. (1981). ''Borehole shear test in geotechnical investigations.'' *ASTM Spec. Tech. Publ.,* 740, 566–578.

McCombs-Knutson Associates, Inc. (1984). ''Missouri River bank erosion study, Fort Peck Dam to the Yellowstone River for period July 1975 to August 1983.'' *Unpublished Rep. Submitted to the U.S. Army Corps of Engineers, Omaha, Neb.*, Plymouth, Minn.

Osman, A. M., and Thorne, C. R. (1988). ''Riverbank stability analysis I: Theory.'' *J. Hydraul. Eng.,* 114(2), 134–150.

Pokrefke, T. J., Abraham, D. A., Hoffman, P. H., Thomas, W. A., Darby, S. E., and Thorne, C. R. (1998). ''Cumulative erosion impacts analysis for the Missouri River master water control manual review and update study.'' *Tech. Rep. No. CHL-98-7*, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Rawls, W. J., Ahuja, L. R., Brakensiek, D. L., and Shirmohammadi, A. (1992). ''Infiltration and soil water movement.'' *Handbook of Hydrology*, D. R. Maidment, ed., McGraw Hill, New York, 5.1–5.51.

Rawls, W. J., and Brakensiek, D. L. (1985). ''Prediction of soil water properties for hydrologic modeling.'' *Watershed Management in the Eighties*, E. B. Jones and T. J. Ward, eds., ASCE, New York, 293–299.

Rinaldi, M., and Casagli, N. (1999). ''Stability of streambanks formed in partially saturated soils and effects of negative pore water pressures: The Sieve River (Italy).'' *Geomorphology,* 26(4), 253–277.

Shields, Jr., F. D., Simon, A., and Steffen, L. (2000). ''Reservoir effects on downstream river channel migration.'' *J. Environ. Conserv.,* 27(1), 54–66.

Simon, A., and Curini, A. (1998). ''Pore pressure and bank stability: The influence of matric suction.'' *Water Resources Engineering '98, Proc., Int. Water Resources Engineering Conf.*, ASCE, New York, 358–363.

Simon, A., Curini, A., Darby, S. E., and Langendoen, E. J. (2000). ''Bank and near-bank processes in an incised channel.'' *Geomorphology,* 35(3–4), 193–217.

Simon, A., and Darby, S. E. (1996). ''Preliminary analysis of bank stability problems on the upper Missouri River below Fort Peck Dam.'' *Unpublished Rep. submitted to Natural Resources Conservation Service*, USDA Agricultural Research Service, National Sedimentation Laboratory, Oxford, Miss.

Simon, A., Shields, F. D., Jr., Ettema, R., Alonso, C., Marshall-Garsjo, M., Curini, A., and Steffen, L. (1999). ''Channel erosion on the Missouri River, Montana between Fort Peck Dam and the North Dakota border.'' *Research Rep. No. 12*, USDA Agricultural Research Service, National Sedimentation Laboratory, Oxford, Miss.

Simon, A., Wolfe, W. J., and Molinas, A. (1991). ''Mass wasting algorithms in an alluvial channel model.'' *Proc., Fifth Federal Int. Sedimentation Conf.*, U.S. Government Printing Office, Washington D.C., 2, 8-22 to 8-29.

SLOPE/W users' guide version 4. (1998). GeoSlope International Ltd., Calgary, Alta.

U.S. Army Corps of Engineers (USACE). (1952). ''Report on degradation observations Missouri River downstream from Fort Peck Dam.'' *Unpublished Rep.*, Fort Peck District, Omaha, Neb.

U.S. Army Corps of Engineers (USACE). (1976). ''Fort Peck Dam to Yellowstone River bank erosion study, bank erosion rates for period 1938 to 1975.'' *Unpublished computation sheets (MRD form 68)*, Fort Peck District, Omaha, Neb.

U.S. Army Corps of Engineers (USACE). (1995). ''Missouri River main stem reservoirs annual operating plan and summary of actual 1994–1995 operations.'' *Rep*., Missouri River Division Reservoir Control Center, Omaha, Neb.

Wei, T. C. (1997). ''Downstream channel and sediment trends study.'' *Unpublished rep. prepared for U.S. Army Corps of Engineers Engineering Division, Omaha, Neb.*, Midwest International Inc.

Williams, G. P., and Wolman, M. G. (1984). ''Downstream effects of dams on alluvial rivers.'' *U.S. Geological Survey Professional Paper No. 1286*, U.S. Government Printing Office, Washington, D.C.