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SEPTEMBER 1980

Final Report on Phase I

ALTERNATE METHODS OF STABILIZING DEGRADING STREAM CHANNELS IN WESTERN IOWA

Submitted to
the Highway Division of
the Iowa Department of Transportation
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ISU-ERI-Ames-81047
Project 1421

Iowa DOT HR-208

ENGINEERING RESEARCH INSTITUTE
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AMES, IOWA 50010 USA

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1. INTRODUCTION

1.1. Problem Statement

Since the beginning of channel straightening at the turn of the century, the streams of western Iowa have degraded 1.5 to 5 times their original depth. This vertical degradation (Plate 1) is often accompanied by increases in channel widths of 2 to 4 times the original widths. The deepening and widening of these streams has jeopardized the structural safety of many bridges by undercutting footings or pile caps (Plate 2), exposing considerable length of piling (Plate 3), and removing soil beneath and adjacent to abutments (Plate 4).

Various types of flume and drop structures have been introduced in an effort to partially or totally stabilize these channels, protecting or replacing bridge structures. Although there has always been a need for economical grade stabilization structures to stop stream channel degradation and protect highway bridges and culverts, the problem is especially critical at the present time due to rapidly increasing construction costs and decreasing revenues. Benefits derived from stabilization extend beyond the transportation sector to the agricultural sector, and increased public interest and attention is needed.

1.2. Objectives

The long range objective of this research is to develop effective and economical methods of stabilizing the degrading stream channels of western Iowa. The study is divided into two phases. The results of

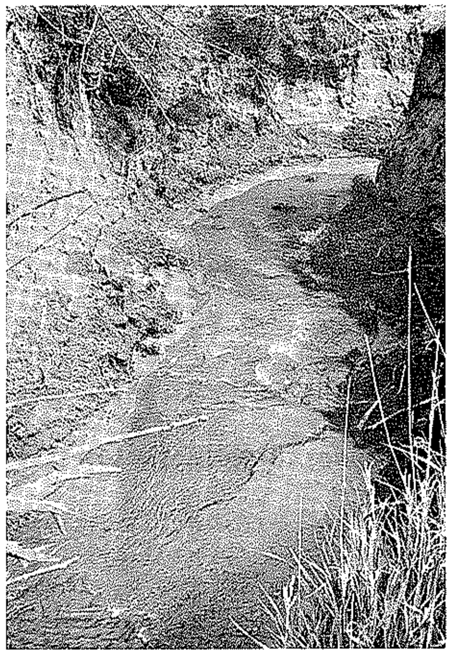


Plate 1. An example of a degrading stream in Monona County. Note the blocks of soil sliding down the channel sides.

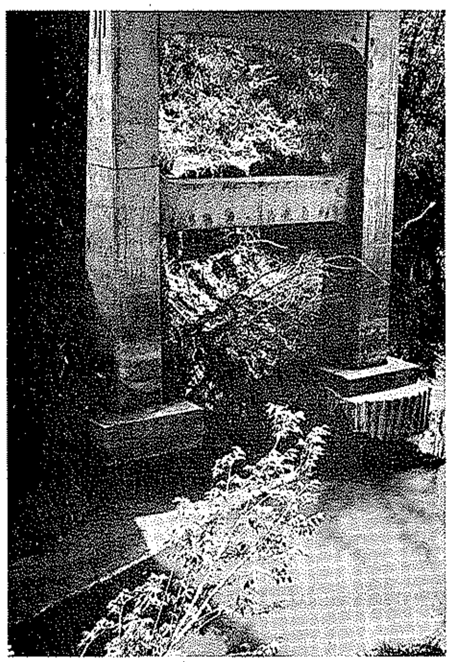


Plate 2. Degradation exposed pile cap of pier at left. Pier at right has had remedial work.

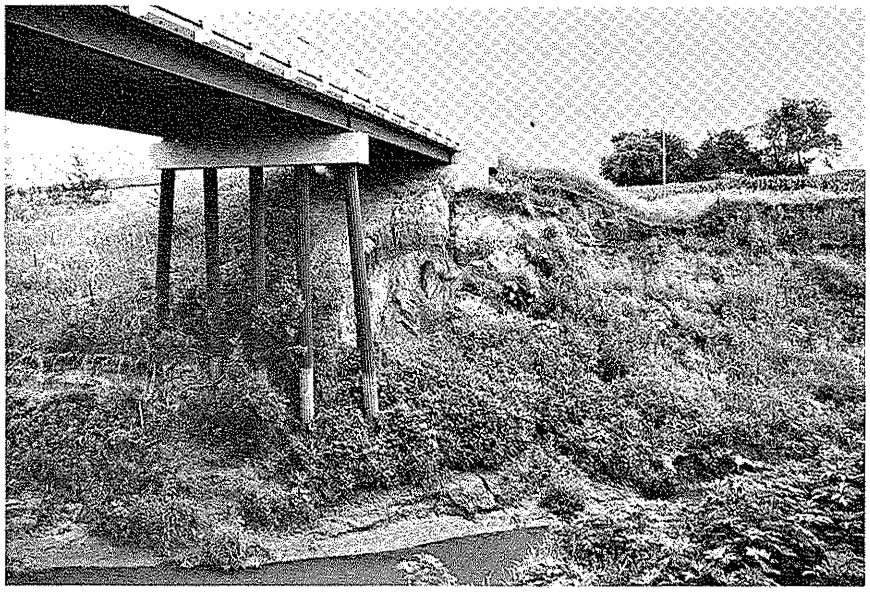


Plate 3. Channel degradation has exposed portions of these piles. Former ground level is indicated by the top of the light-colored portion at bottom of piles. Note also the soil pulling away from the abutment.

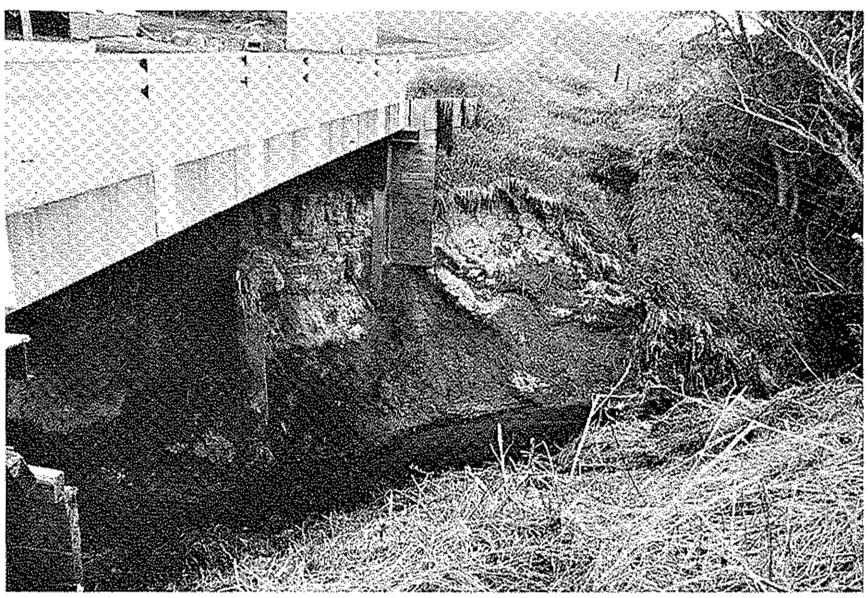


Plate 4. Degradation and the associated landslide has removed soil adjacent to the abutment on this Shelby County bridge.

the first phase are the subject of this report. The specific objectives of Phase I are to inventory existing grade stabilization structures and to evaluate their performance relative to the geologic and hydrologic environment and to their structural design. These analyses should point to possible design weaknesses and potential remedial actions that may be taken on existing structures. Also, the analyses should provide insights that will lead to innovative designs. The grade stabilization structure which is analyzed in some detail is the flume bridge, sometimes called the Greenwood Flume. One innovation given a preliminary evaluation in Phase I is the substitution of soil-cement for portland cement concrete elements in grade stabilization structures. Finally, some tentative recommendations will be made to provide a method for predicting the rate and maximum depth of degradation at a given position on a stream. Phase II will involve the design, construction and evaluation of demonstration grade stabilization structures.

1.3. Location of the Study Area

This research inventoried and evaluated structures within a 13-county area along the western Iowa border. The Missouri River provides the base level from which all streams in the area are graded. The counties included in this study are: Sioux, Plymouth, Woodbury, Ida, Crawford, Monona, Harrison, Shelby, Pottawattamie, Mills, Montgomery, Fremont, and Page. Figure 1 shows the study area. Selected tributaries to the Missouri River are identified in Fig. 2.

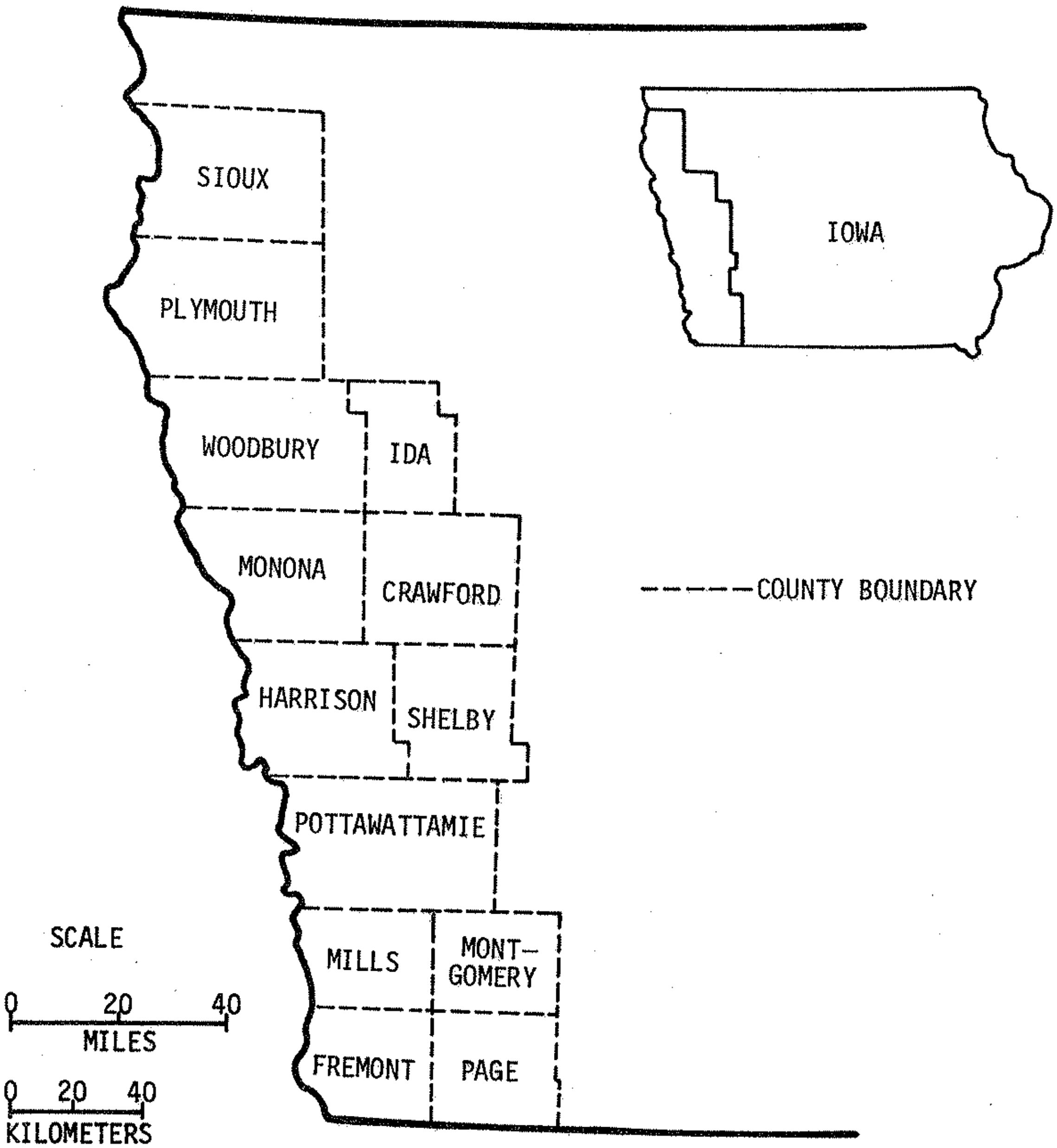


Fig. 1. Location of study area.

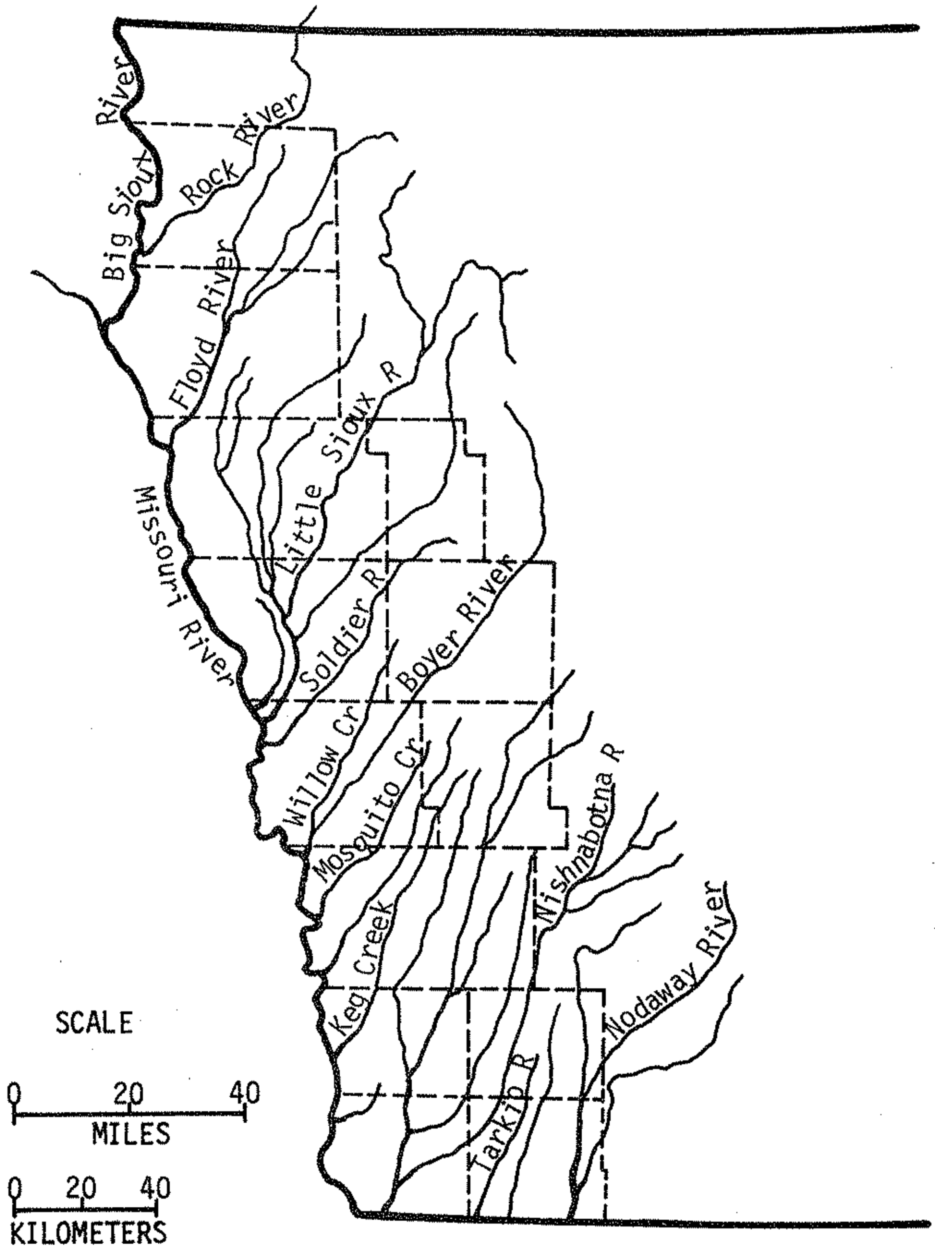


Fig. 2. Selected streams in study area.

2. GEOLOGIC AND HYDROLOGIC SETTING

2.1. Surficial Geology and Geotechnical Properties of Soils

The surficial geology of the uplands and the geotechnical properties of the soils of this 13-county area are well known from numerous studies. The alluvium of the Nishnabotna and Little Sioux Rivers has also been the subject of geologic and geotechnical research (Knochenmus, 1963; Pedersen, 1962).

Figure 3 shows the surficial geology of the uplands as reported in Dahl et al. (1958). The two northernmost counties of the study area are covered by loess deposits from 5 to 20 ft thick over glacial till. Loess is wind-blown silt from the Missouri River floodplain deposited in this area more than 14,000 years ago (Ruhe, 1969). The loess is thickest at the western edge of the counties and thins eastward. The southern counties have much thicker loess deposits ranging from over 100 ft in the bluffs adjacent to the Missouri River to 15 ft on the eastern sides of Page and Montgomery Counties.

Associated with the eastward thinning of the loess deposits are systematic trends in the properties of the upland loess. Figure 4 from Dahl et al. (1958) shows that clay content increases with increasing distance from the Missouri River. Davidson and Handy (1952) show that in-place dry density varies linearly from 69 pcf at the bluff line to about 84 pcf at a distance 78 miles from the bluff, whereas the plasticity index increases and the shrinkage limit decreases with increasing distance from the river. Both median particle diameter and percent

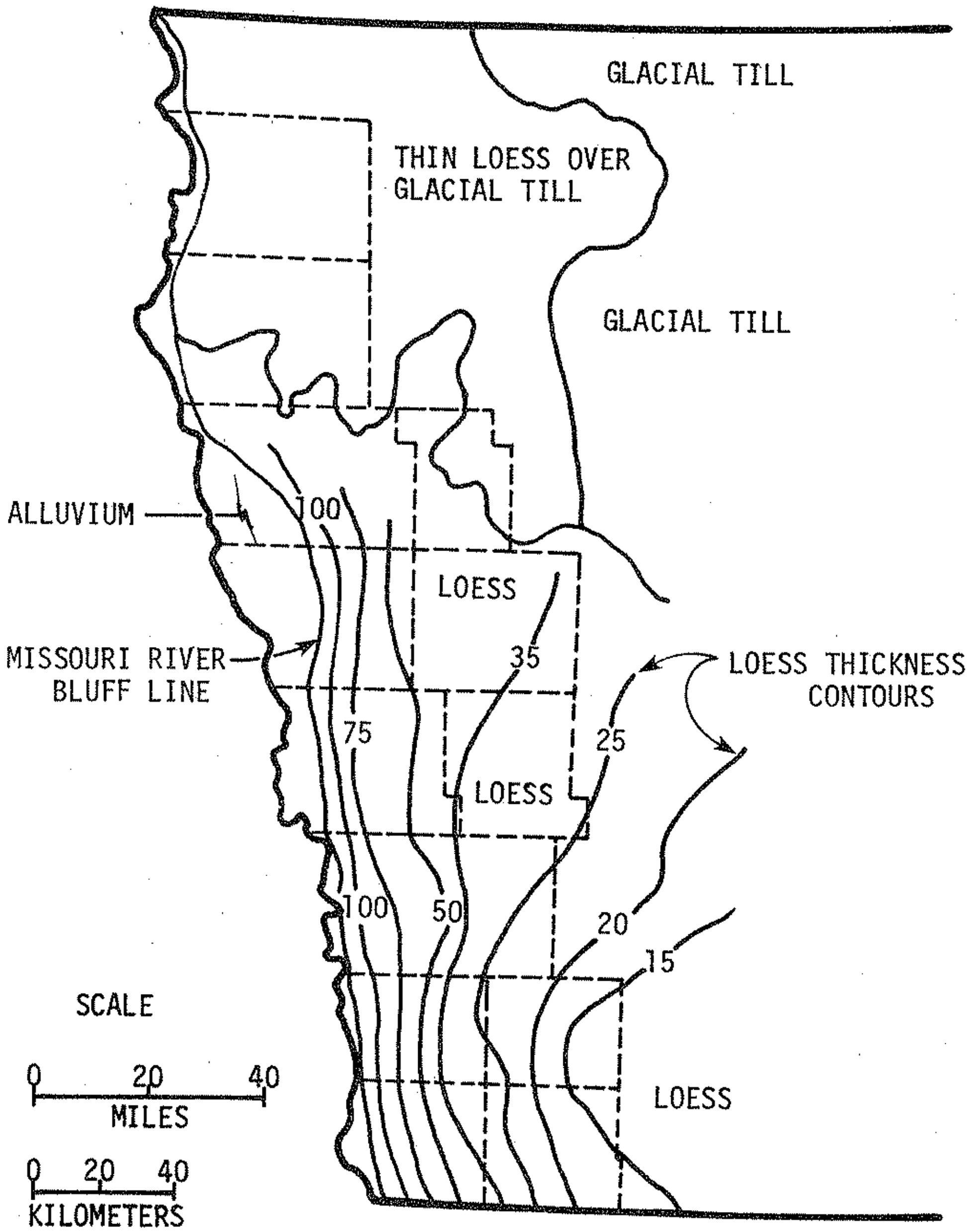


Fig. 3. Surficial geology of western Iowa showing thickness of loess. After Dahl et al. (1958).

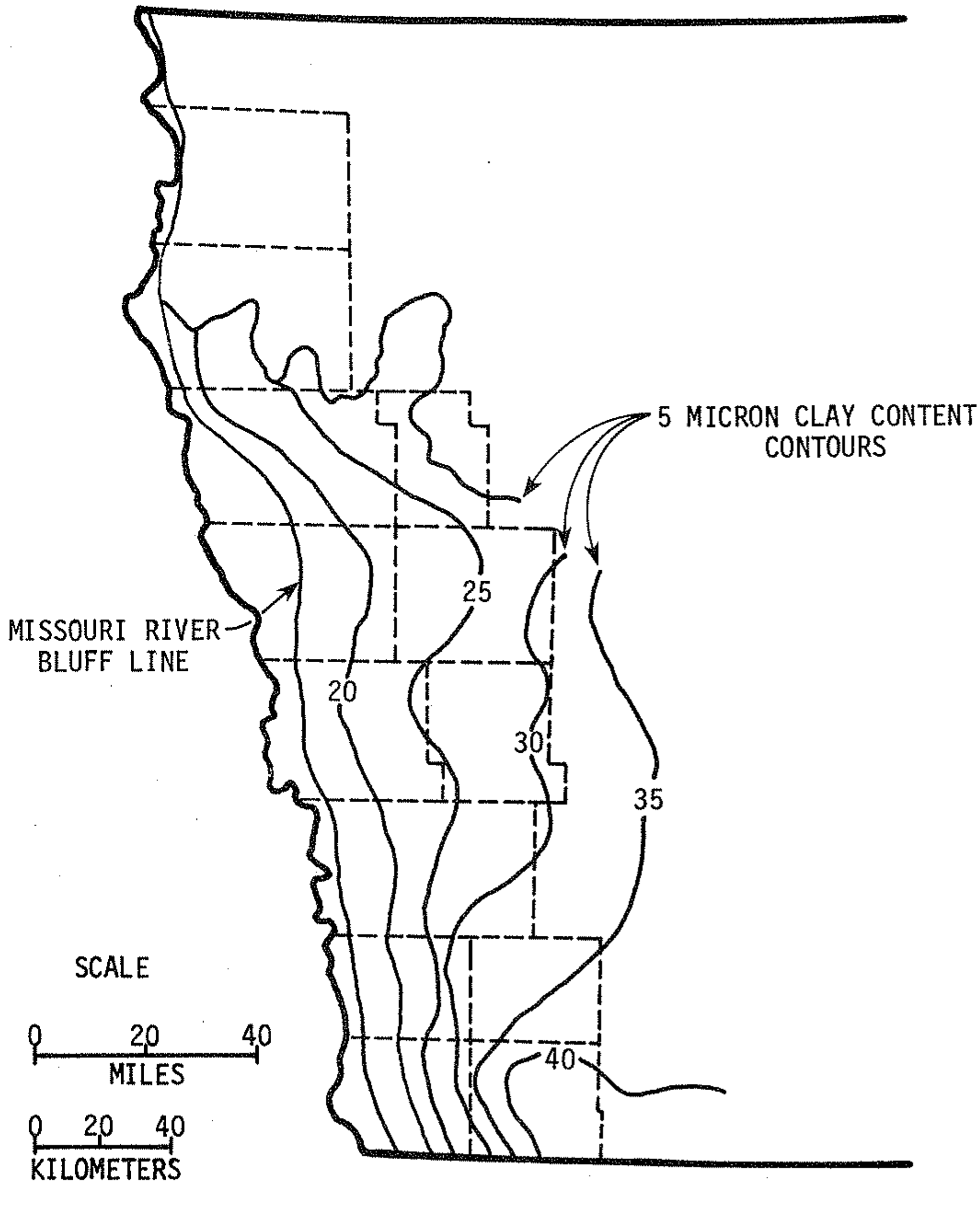


Fig. 4. Geographic distribution of 5 micron clay (in percent) contours in loess of western Iowa. After Dahl et al. (1958).

silt content decrease with increasing distance from the bluff line (Davidson and Handy, 1952). The regional variation in engineering classification of the western Iowa loess is shown in Fig. 5 with the A-4 soils closest to the valley wall, A-6 soils at intermediate distances and A-7 soils at the southeastern extreme of the study area (Hansen et al., 1959).

Although the upland soils have been described extensively, only two studies have characterized the regional variations of the alluvium or secondary loess in the valleys. Engineering data for the pedologically classified soils in western Iowa provide a basis for comparing and contrasting the alluvium with the upland soils. The principal pedological soil associations of western Iowa are shown in Fig. 6 (from Simonson et al., 1952). In the southern, deep loess portion of the study area the Monona-Ida-Hamburg (MIH) soil association occupies the western portion, with the Marshall association eastward. The alluvial soils of the MIH association are the McPaul and Napier, whereas the upland soils of the MIH are the Monona, Ida, and Hamburg. The Hamburg soils occur on the steeply sloping hills of the extreme western bluffs. In the Marshall association the alluvial soils are the Wabash-Judson complex with the Marshall as the principal upland soil. Some of the engineering data for these soil series is reported by Miller et al. (1978). These data indicate that, although the alluvium tends to be somewhat higher in clay content and more plastic than the upland soils, the engineering classification of these two groups of soils is similar with both alluvium and upland soils classifying as A-4 in the extreme west, A-6 in the mid region, and A-7 in the eastern portion of the study area.

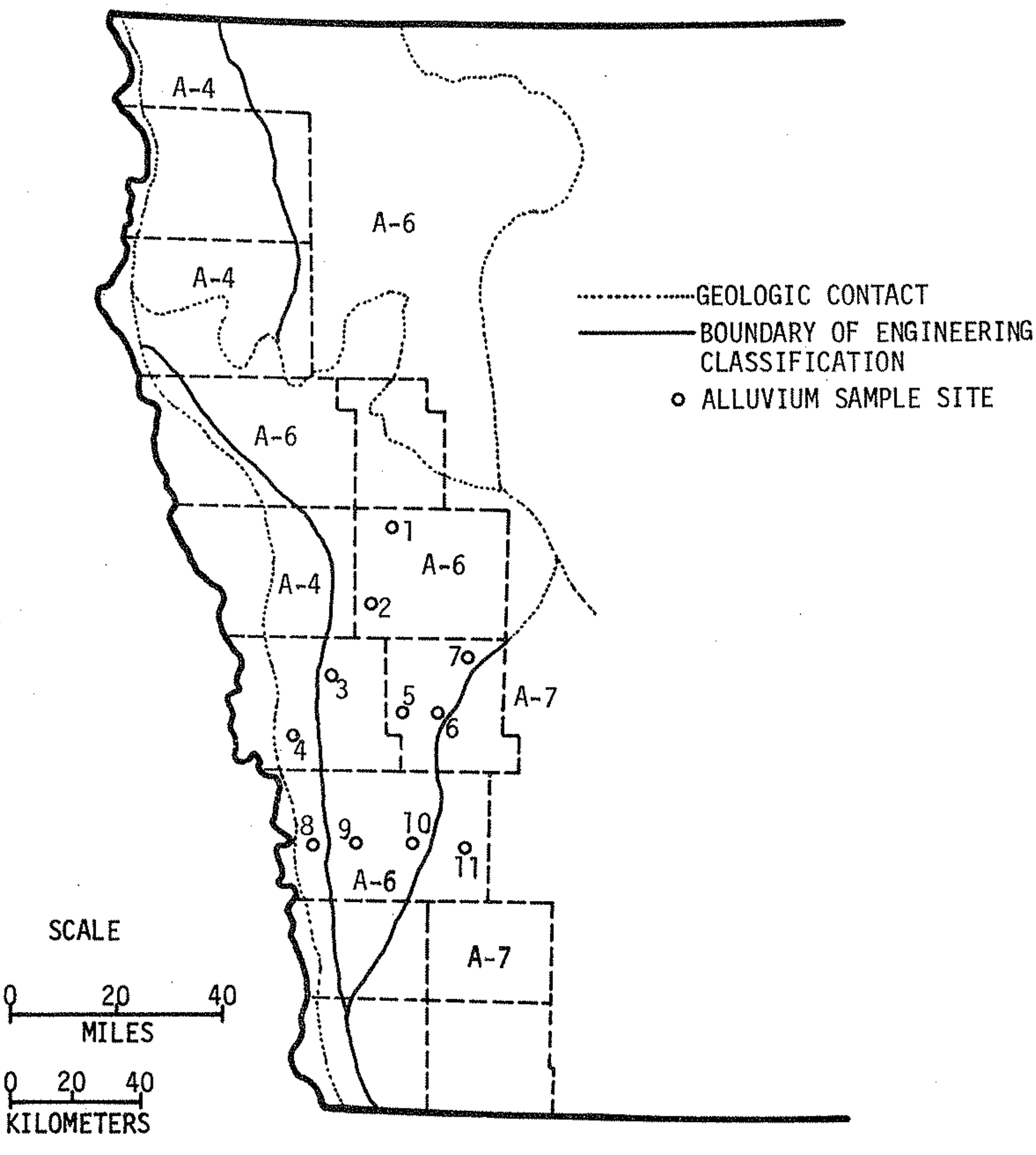


Fig. 5. Engineering classification of loess in western Iowa. After Hansen et al. (1959) and location of alluvium sample sites.

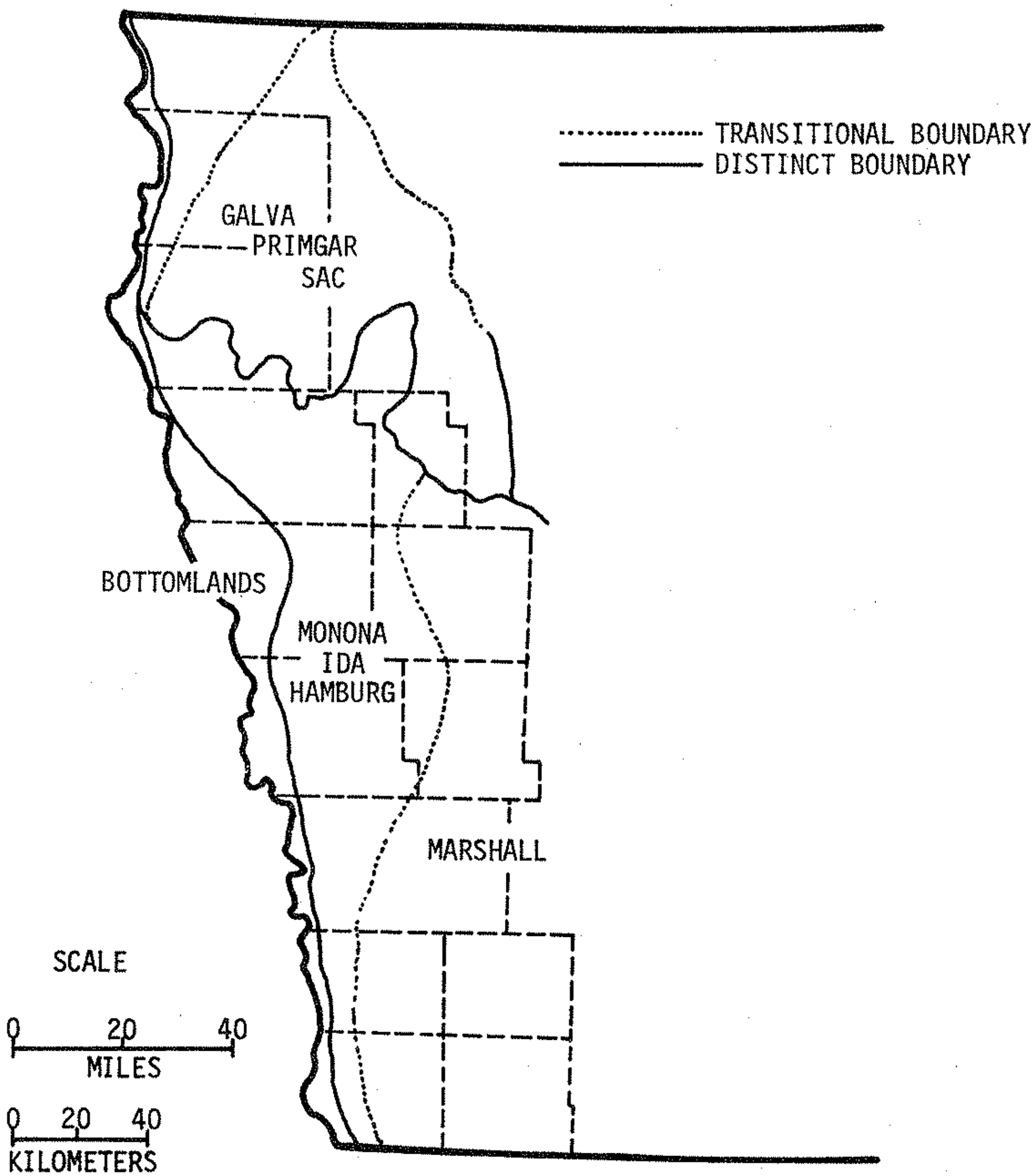


Fig. 6. Principal soil associations in western Iowa. After Simonson et al. (1952).

This generalization is not supported by data collected as part of this study. Figure 5 shows the location of sites where alluvium samples were collected. Table 1 shows the results of the laboratory geotechnical tests. These data suggest that the alluvium is less plastic and higher in sand content than the adjacent upland soils. More geotechnical studies of loess-derived alluvium are needed; however, it can be seen that most alluvium is a clayey silt similar to the upland loess.

Loess is well known for its high degree of erodibility; thus, it is desirable to know the thickness of the loess-derived alluvium beneath the active channel of the streams. Once the streams degrade to the more resistant material beneath the alluvium, it is possible the more resistant material will form a base level and inhibit the erosion process. The alluvial thickness is defined as the thickness of alluvium beneath the active channel. The map shown in Fig. 7 was compiled by taking data from the boring logs file at the Iowa Department of Transportation. This is a modification of the map found in Antosch and Joens (1979). The Antosch and Joens report shows that, except for the extreme southern portion of the study area, the alluvium is underlain primarily by glacial till. In the south, the base material is mainly Pennsylvanian shale. It would be expected that the alluvium thickness would decrease in the upstream direction as it does in the Little Sioux and Floyd River valleys. However, most of the southern streams show an irregular pattern of thickening and thinning alluvium from their mouths to their headwaters. This irregular pattern may be due to buried valleys (Antosch

Table 1. Geotechnical Properties of Loess-Derived Alluvium.

	Particle Size Distribution							
Sample Number*	Percent Sand	Percent Silt	Percent Clay	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity	AASHTO Class
1	3	69	29	38	23	15	--	A-6(16)
2	1	65	34	53	31	22	--	A-7-6(27)
3	27	61	12	28	24	4	--	A-4(2)
4	20	65	15	31	21	10	--	A-4(7)
5	1	73	26	41	31	10	2.67	A-5(13)
6	13	68	19	39	28	11	2.69	A-6(11)
7	35	45	20	28	21	7	2.70	A-4(3)
8	18	65	17	31	19	12	--	A-6(9)
9	29	50	21	31	23	8	2.70	A-4(5)
10	17	65	18	24	23	6	2.70	A-4(4)
11	23	63	14	35	23	11	--	A-6(8)

*See Fig. 5 for location.

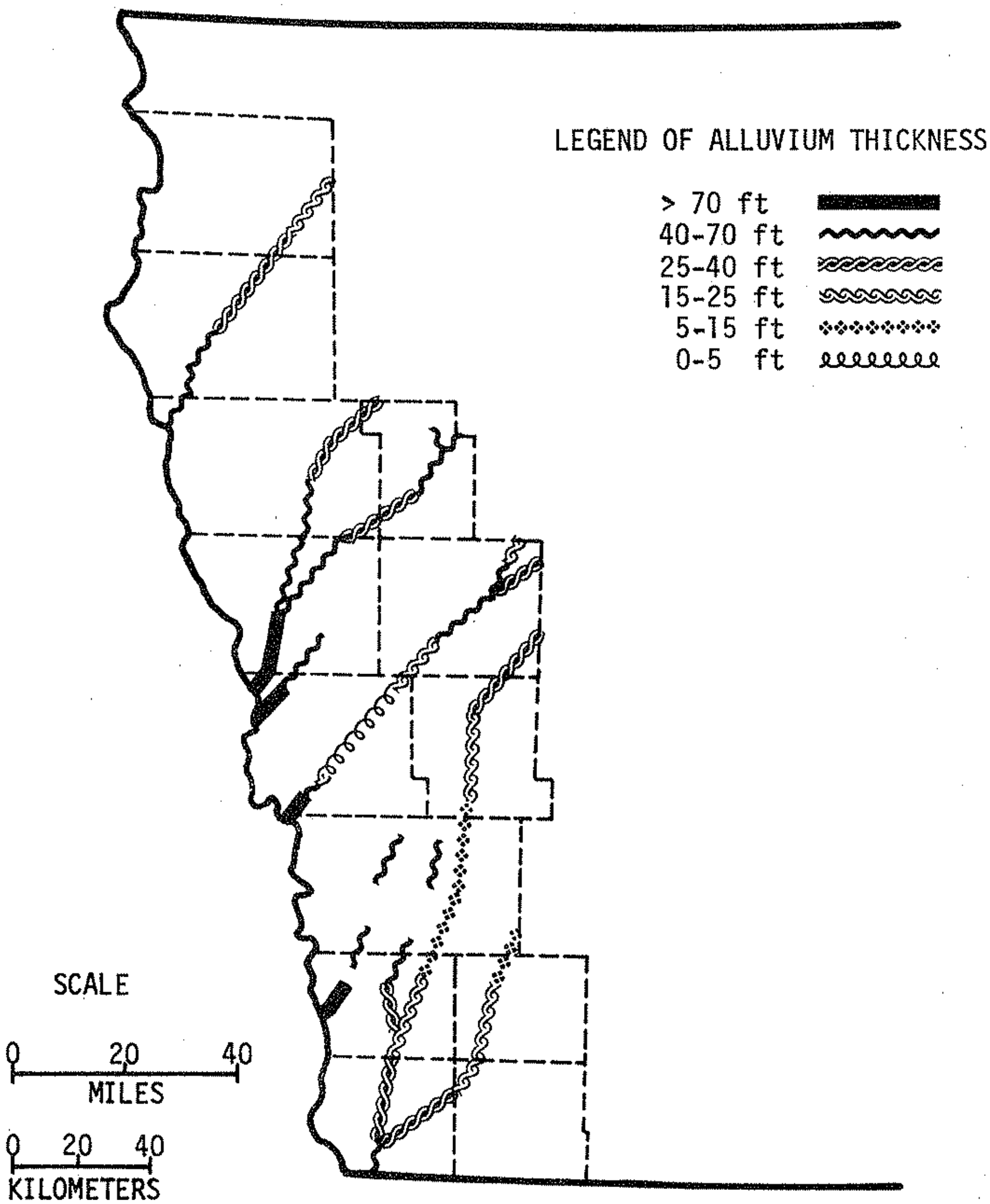


Fig. 7. Thickness of alluvium beneath channel bottom. Modified from Antosch and Joens (1979).

and Joens, 1979) or due to degradation. In any event it can be seen that the alluvium is greater than 15 ft, thus in most cases, the chance of the streams degrading to a more resistant base level is not likely to occur for many years.

2.2 Topography and Regional Geomorphology

The tributaries to the Missouri in western Iowa are elongated basins which trend to the south and the southwest. Morphometric parameters measured on 1:250,000 topographic maps for some basins are shown in Table 2.

The drainage divide between the Missouri and Mississippi tributaries is approximately 400 to 500 ft above the Missouri River floodplain. Lara (1973) has computed the average slopes of the major streams and has shown that the Little Sioux has the most gentle profile, with the average channel slope upstream of the mouth at 2 feet per mile (ft/mi) and increasing to between 5 and 6 ft/mi for the channel reaches near the headwaters. The Floyd, Boyer, West and East Nishnabotna Rivers are similar with slopes between 3 and 5 ft/mi near their mouths and increasing to 6 to 8 ft/mi near their heads. The Maple has the steepest longitudinal profile with an average slope of about 4.5 ft/mi near the mouth increasing in upstream reaches to more than 11 ft/mi.

Lohnes and Joshi (1967) studied the regional variations of topography in the loess area and found that valley side slopes are steepest nearest the Missouri and decrease with increasing distance from the bluffs according to a power function. Similarly, the absolute drainage

Table 2. Quantitative Geomorphology of Selected Western Iowa Streams.

Stream	Basin Order	Area (mi^2)	Drainage Density (mi^{-1})	Length of Main Stream (mi)	Circularity Ratio	Maximum Relief (ft)
Boyer	4	1,188	0.31	100	0.268	400
Keg	2	190†	0.32	65	---	525
Floyd	5	921	0.50	80	0.420	-
Little Sioux	6	4,507	0.23	130	0.664	430*
Soldier	3	445	0.33	65	0.266	575
Nishnabotna	5	2,819†	0.36	120*	---	650
Mosquito	2	267	0.41	60	0.277	450
Tarkio	3*	540	0.88	70	---	380
Willow	2	146	--	45	---	475

*Parameters for portions of streams within Iowa only.

†Measured 4.6 miles above mouth of stream.

density (total length of streams within a circular area of one square mile) decreases with increasing distance. Maximum relief (the elevation difference between the valley bottom and drainage divide) ranged from 220 ft near the Missouri River bluffs to 90 ft about 16 miles from the river, but no systematic relationship between maximum relief and distance from the river was observed. Ruhe (1969), however, observed a linear decrease in local relief with distance from the bluffs as well as a linear increase in summit widths with increasing distance. Recently, Hallberg (1979) studied the alignment of stream systems developed in loess. That research showed that first-order streams in northwest and west central Iowa have a preferred alignment North 40-50° West, but no alignment is apparent in southwest Iowa. The alignment of the higher order streams somewhat follows the joint patterns of the underlying glacial till. Hallberg concludes that the lower order streams are controlled by wind alignment, whereas the higher order streams are controlled by the till landscape beneath the loess.

An interesting feature of the streams which flow through the thick loess area is the elongated shape of the drainage basins. Table 2 lists the circularity ratio of several of the tributaries to the Missouri River. Circularity ratio is the area of the drainage basin divided by the area of a circle having the same perimeter as the basin. Thus, a circular basin would have a circularity ratio of one. The more elongated the basin, the lower the ratio. Both the Floyd and the Little Sioux Rivers, with large portions of their drainage areas in glacial till as well as loess, have circularity ratios of about 0.4 and 0.6, respectively. The remaining basins which are predominantly in loess have circularity ratios less than 0.3.

Because the drainage density (total stream length divided by basin area) in the loess area is relatively high, the stream discharge will respond more quickly to precipitation because the drainage network is more efficient in conducting the water off the landscape. The low circularity ratios of the basins suggest an attenuation characteristic for flood discharges.

2.3. Climate and Hydrology

In general, the western part of Iowa is the portion of the state with the lowest amounts of precipitation and runoff. Figure 8 from the U.S. Geological Survey (1978) shows the normal amounts of annual precipitation within the study area. The lowest amount of precipitation is 25 in. in the extreme northwest corner of the state. The amount of annual precipitation increases to the south-southeast with the southeast corner of Page County receiving as much as 35 in. per year. A similar geographic trend is in the variation of average annual runoff as shown in Fig. 9 (U.S. Geological Survey, 1978). The northwest corner of the state has runoff of about 2 in., whereas the southeast portion of the study area has runoff between 5 and 6 in. per year. Table 3 shows mean annual discharge for the major stream basins in western Iowa where gaging data are available, and estimated discharge for other selected

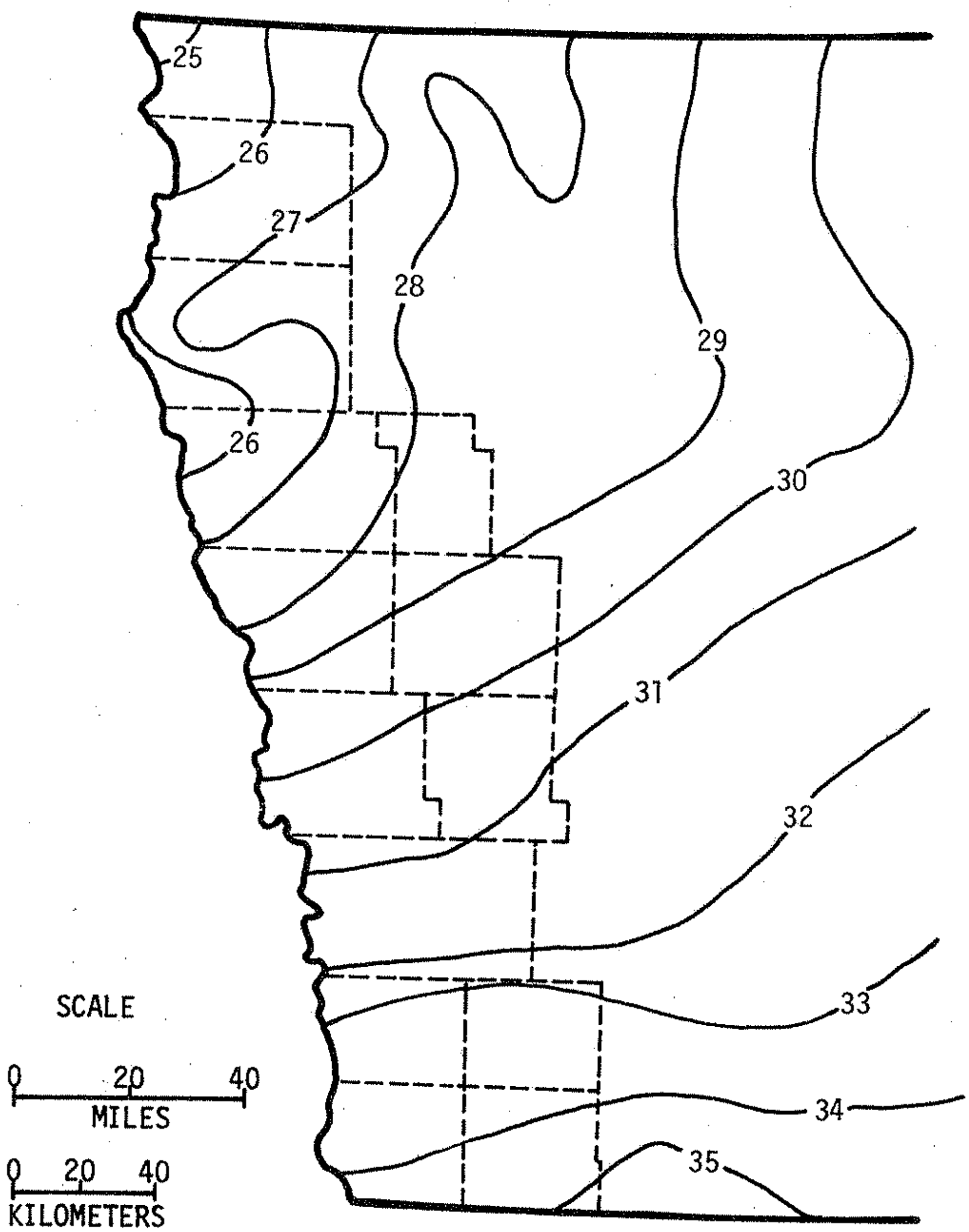


Fig. 8. Normal amounts of annual precipitation in inches. From U.S. Geological Survey (1978).

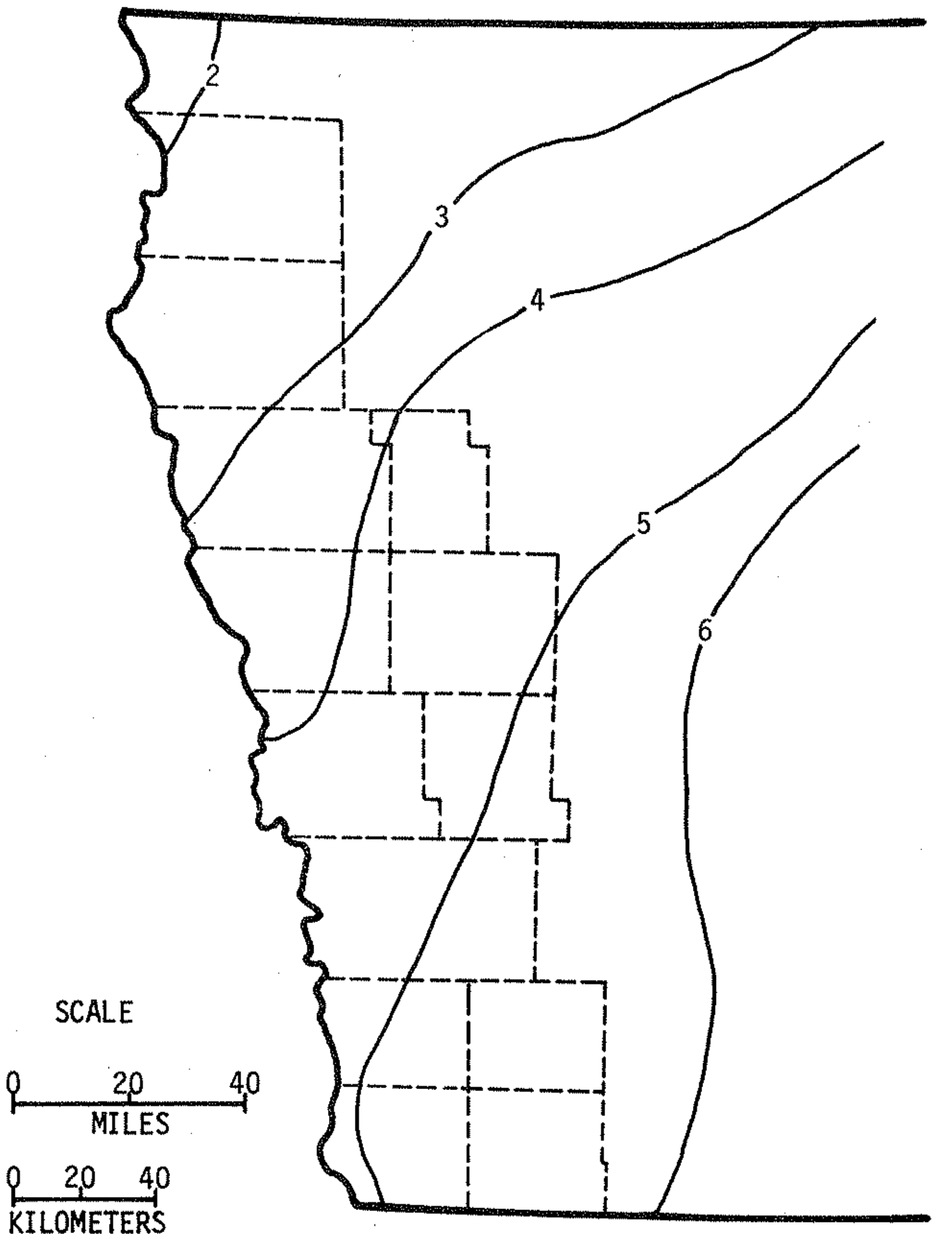


Fig. 9. Average annual runoff in inches. From U.S. Geological Survey (1978).

Table 3. Average Annual Discharge for Selected Western Iowa Streams.

Stream	Gaged Discharge* (cfs)	Drainage Area at Gage* (mi^2)	Estimated Discharge† (cfs)	Drainage Basin Area (mi^2)
Boyer	326	810	278	1,188
Keg	-	-	85	190
Floyd	207	918	236	921
Little Sioux	310	4,460	1,269	4,507
Soldier	141	417	98	445
Nishnabotna	933	2,800	1,373	2,819≠
Mosquito	-	-	64	267
Tarkio	43	200	-	-
Willow	-	-	34	146

*Gages not at mouth.

†From equations of Lara (1979).

≠Within Iowa only.

streams where the discharge is estimated from the equations based on the regional analysis of Lara (1979). Estimated average annual discharges range from 33 to over 1200 cfs.

The magnitude and frequency of flood discharges in Iowa have been evaluated by Lara (1973). The two primary physiographic variables are drainage area and average stream slope. Stream slope becomes increasingly important in the smaller drainage basins less than 1000 mi^2. Peak discharges for the streams gaged by the U.S. Geological Survey in the study area vary from 29 cfs/mi^2 (871 mi^2, Boyer River at Logan) to 1,028 cfs/mi^2 (4.7 mi^2, East Tarkio River near Stanton).

3. CHANNEL DEGRADATION

3.1. History of Channel Degradation

Daniels (1960) described the entrenchment of Willow River between 1919 and 1958 in his benchmark paper. Subsequently, two other papers (Ruhe and Daniels, 1965; Daniels and Jordan, 1966) provided expanded data and analyses of the historical erosion in the Willow. Beer (1962) provided some data on the degradation of Steer Creek. More recently, the historical deepening of the Tarkio River has been documented (Piest et al., 1976; Piest et al., 1977). This study updates the history of the Willow by compiling longitudinal profiles of the stream in 1966 and 1980 and provides new information on the degradational history of Keg Creek. The new data are synthesized with the published data to show the magnitude of the degradation problem and to provide a tentative approach to estimating the amount of degradation in the streams. Table 4 summarizes the results of this and previous studies.

For all of these historical studies of channel degradation, data were obtained from old land surveys, drainage district records and plans, and bridge plans and inspection reports. Notes from the original land surveys indicate that the streams of western Iowa had swamps and marshes associated with their floodplains and that the streams were subject to frequent flooding, making the bottomland unfit for cultivation (Daniels, 1958; Piest et al., 1976). Around 1920, programs of stream straightening began and continued for about 40 years in order to achieve better drainage and to open bottomlands to farming.

Table 4. Degradation and Aggradation in Western Iowa Streams.

Stream	Distance from Mouth (mi)	Date	Channel Depth (ft)	Rate of Erosion (ft/yr)
Willow	10.7	1919	15	
		1931	21	0.5
		1936	25	0.8
		1958	29	0.2
		1966	33	0.5
		1980	32	aggrading
Willow	16.0	1919	16	
		1931	22	0.5
		1958	33	0.4
		1966	22	aggrading
		1980	27	0.4
Willow	19.1	1920	12	
		1929	19	0.8
		1942	30	0.9
		1958	36	0.4
		1966	33	aggrading
		1980	32	aggrading
Tarkio	12	1846	6	
		1932	16	0.1
		1975	26	0.2
Tarkio	27	1921	7	
		1939	20	0.7
		1963	30	0.4
Steer	--	1932	3	
		1942	17	1.4
		1961	30	0.7
Steer	--	1932	3	
		1942	21	1.8
		1961	28	0.4
Keg	23.0	1954	28	
		1973	32	0.2
		1980	30	aggrading
Keg	25.1	1927	21	
		1952	30	0.4
		1976	32	0.1

Table 4. (continued).

Stream	Distance from Mouth (mi)	Date	Channel Depth (ft)	Rate of Erosion (ft/yr)
Keg (continued)		1978	32	stable
		1980	32	stable
Keg	28.0	1954	17	
		1973	24	0.4
		1980	25	0.1
Keg	28.5	1958	14	
		1973	24	0.7
		1980	27	0.4

One example of stream degradation is on the Tarkio River, which has its headwaters in southwest Iowa and empties into the Missouri River in northwest Missouri. One indication of the magnitude of the shortening of the Tarkio is that the present confluence of the Missouri and the Tarkio River is now 16 miles upstream from its former, prestraightened location. Data reported by Piest and his associates are shown in Table 4 where a fourfold increase in channel depth has occurred. Piest et al. (1976) used data dating back to 1846 from 20 sections in Atchinson County, Missouri, to calculate degradation rates of 0.1 ft/yr during the first 86 years of settlement and 0.2 ft/yr between 1932 and 1975. On the West Tarkio Creek near the Iowa-Missouri border they estimate that the annual channel erosion rate within a 15 mile length has been 131,000 ton/yr since dredging began in 1920.

Perhaps the best documented stream degradation history is that of Willow Creek which had its channel shortened in Harrison County from 26.3 to 20.2 miles between 1916 and 1919. Daniels (1960) reports that degradation began almost immediately and that at one point (between 1919 and 1924) degradation proceeded at a rate of 1.2 ft/yr. Subsequent analyses of downcutting rates indicate that the degradation rate decreases with time. Table 4 shows data on the amount of degradation at three sections on the Willow as reported by Ruhe and Daniels (1965) between 1919 and 1958, as well as data from this study for 1966 and 1980. The channel in this reach was uncontrolled until the early 1970s when two flume bridges were constructed. Longitudinal profiles reconstructed from Daniels' (1960) data for the period 1919 to 1958 are shown in Fig. 10. Longitudinal profiles from 1966 to 1980 as well as Daniels 1958 profile are shown in Fig. 11. The data between 1919 and 1958 clearly indicate that the channel was degrading at rates varying from 0.8 ft/yr to 0.18 ft/yr with the maximum amount of degradation at 24 ft and a maximum depth of channel below the floodplain of nearly 40 ft!

Ruhe and Daniels (1965) used these data to calculate an erosion volume of 1.9 yd^3/yr per foot of channel length. If this erosion rate is calculated for a 15-mile reach of stream, and a sediment unit weight of 70 pcf is assumed, the erosion rate of about 145,000 tons per year for the Willow is comparable with the erosion rate for the Tarkio as reported by Piest and his associates.

A comparison of the 1958 and 1966 profiles raises some questions. At 11 sections there is evidence of degradation with a maximum depth of 3.2 ft, whereas at 7 sections the measurements indicate aggradation with accumulations of up to 3.6 ft. Thus, about half of the straightened portion of Willow Creek appears to have been aggrading while the other half appears to be degrading with a sequence of aggradation, degradation, aggradation, degradation, progressing upstream. These data suggest that the channel bottom has been essentially stable between 1958 and 1966 and confirms the Daniels and Jordan (1966) speculation that this reach of the channel was stable by 1958. In the early 1970s two flume bridges were constructed as grade stabilization structures. A comparison of the 1966 and 1980 profiles shows the effectiveness of the flume bridges in sedimenting the degraded channels. The flume bridge with a 17-ft drop has backed up sediment for approximately 2.5 miles

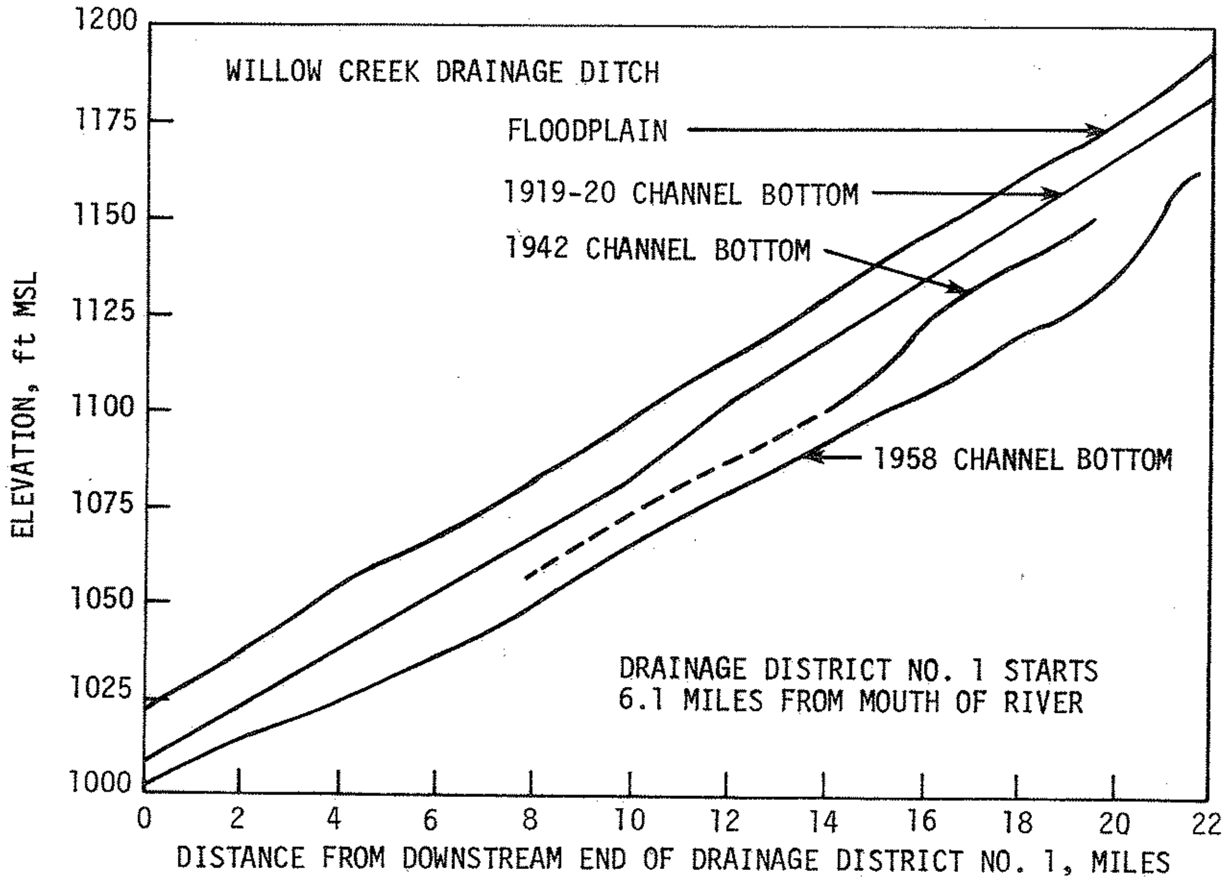


Fig. 10. Longitudinal profile of Willow drainage ditch in Drainage Districts 1 and 2. After Daniels (1960).

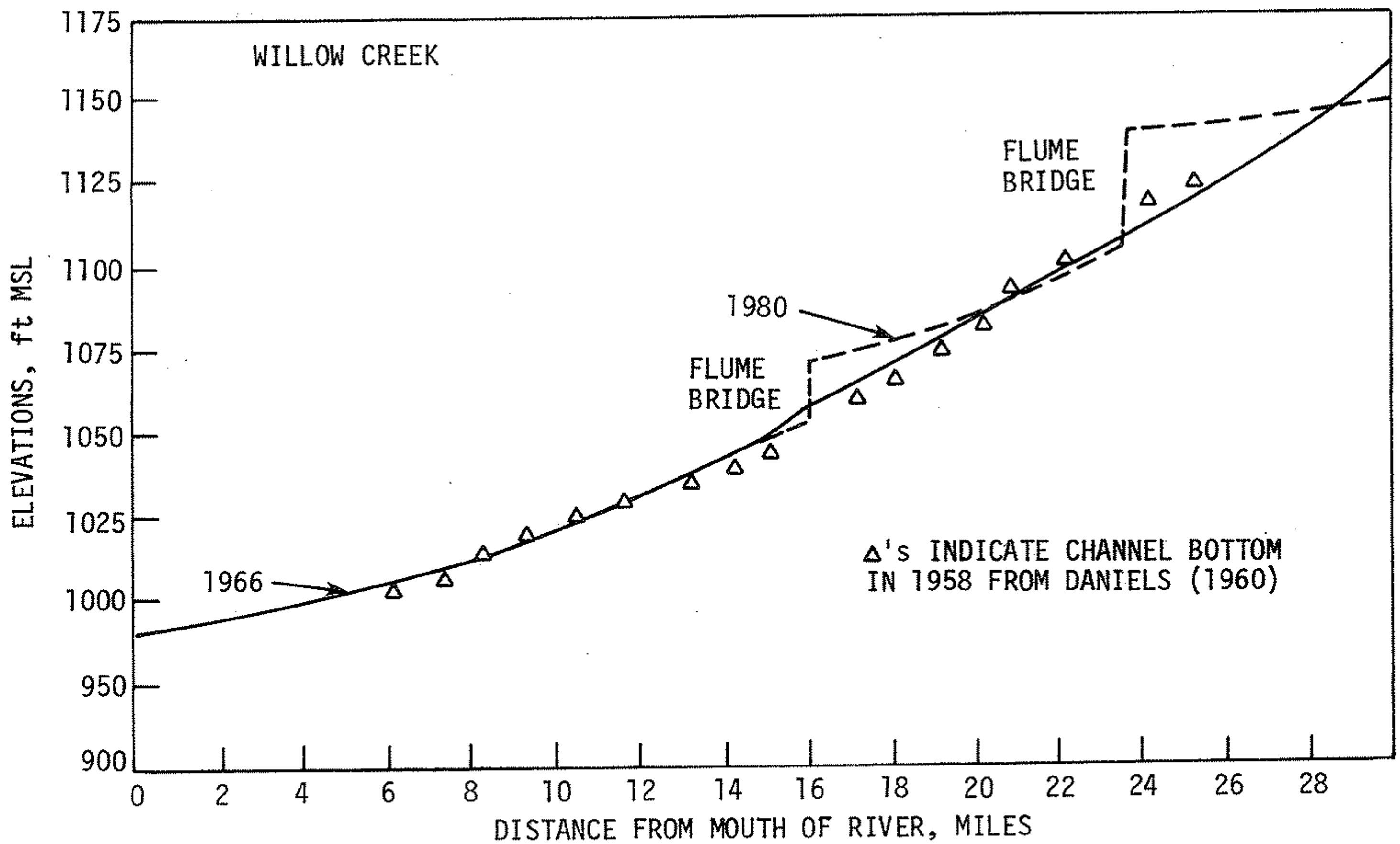


Fig. 11. Longitudinal profiles of Willow Creek 1958, 1966, and 1980.

upstream, whereas the flume with a 34-ft drop has caused sedimentation over 6 miles upstream. It appears that the aggradation is effective only to the elevation of the crest of the structure in the case of the lower structure, but aggradation has extended further upstream in the other flume. Although there are no data to indicate the dates at which the sedimentation was complete, the fact that both structures are full 8 or 9 years after construction indicates that the sedimentation rate is rapid.

The degradation in Steer Creek between 1932 and 1942 at two stations is described by Beer (1962). The maximum depth increase is from 2 ft to over 30 ft. There was a four to fivefold increase in depth in the first 10 years with less than a doubling of depth in the latter 19 years. This is further evidence of a decreasing degradation rate with time.

Keg Creek provides additional documentation of stream degradation in western Iowa; however, the data are more recent than those of the streams previously discussed. Keg Creek was straightened throughout half its length, but the original maps or plans are not available. The only source of data has been bridge plans and subsequent inspection reports. Figure 12 shows the 1980 longitudinal profile of Keg Creek with some data going back to 1954. The 1972 profile of the stream is 2 to 3 ft above the 1980 profile to mile 14.5. Upstream from mile 14.5 to mile 28.5 the 1973-74 profile is 1 to 3 ft below the 1980 profile. Comparing the 1954 profile with the two more recent profiles between

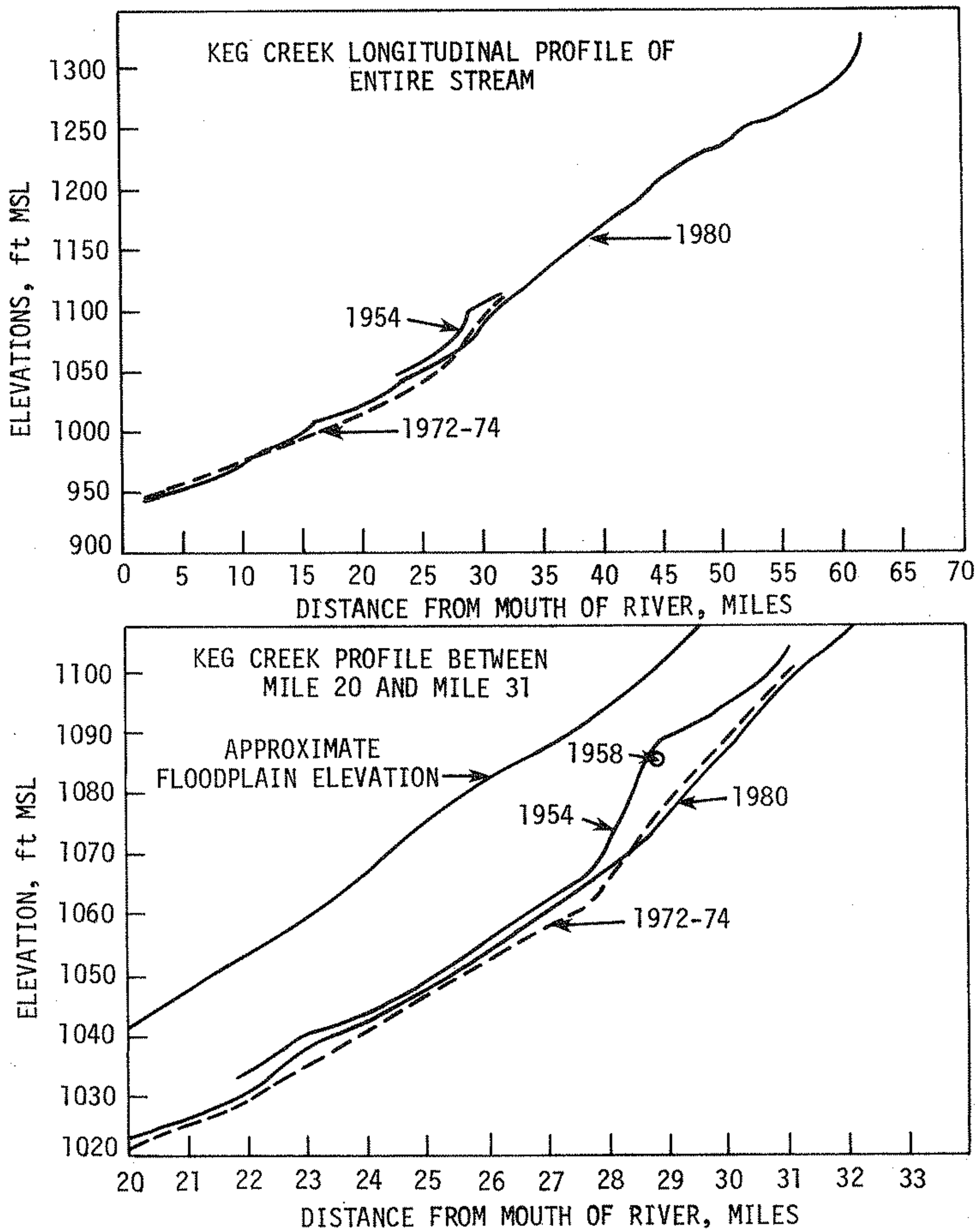


Fig. 12. Longitudinal profile of Keg Creek.

mile 22 and mile 27.8, the 1954 profile is 1 to 2.5 ft above the 1980 profile. The interpretation is that this lower reach of the stream has been relatively stable with regard to degradation in the last 25 years. However, between mile 28.5 and mile 31 there is evidence of degradation of as much as 10 ft within this same time span. This suggests that a knickpoint (Plate 5) has moved through this reach of river. According to reports, a knickpoint about 5 ft high was downstream of mile 28.5 in the early 1970s. The knickpoint which existed in 1979 was observed by the authors in the reach approximately at mile 29 and consisted of 3 riffles 1 to 3 ft in height. During the summer of 1980, four riffles from 1 to 1.5 ft high were observed between mile 29.5 and 31.5. It is concluded that this reach of the stream is actively degrading. Little difference exists between the profiles of 1976 and 1980 in the upper reaches where the 1976 channel bottom is only 2.5 ft above the 1980 channel bottom. Thus, there is little evidence for degradation in the upper reaches of Keg Creek. This is verified by the county engineer who stated that this reach of the stream is not creating any bridge problems at present.

The tentative conclusion of this historical study of degradation in western Iowa is that, although degradation may precede the passage of a knickpoint, the channel seems to be vertically stable after it passes. Further, the effect of grade stabilization structures is to cause aggradation upstream to the elevation of the crest of the structure. Obviously, the higher the structure and the less the channel gradient, the more miles upstream the channel will be stabilized. It is apparent

Plate 5. An example of a knickpoint eroding headward in a small tributary stream.

from field inspection that additional degradation has occurred immediately downstream as a result of some of these grade stabilization structures. This is believed to be due in part to the supercritical flow at the toe of the flumes and resultant energy dissipation back to subcritical flow further downstream. The overall long-term effect of the two previously described structures in stopping degradation is questionable because the stream appeared to be reaching stability in that reach. The structures, however, do assist in restoring a higher bed level and permit using a much shorter bridge for transportation purposes.

3.2. Causes and Mechanism of Degradation

Three causes of stream degradation in western Iowa have been suggested: stream shortening, increased runoff, and degradation of the Missouri River. The problem of interpretation is complicated by the fact that these processes have gone on periodically at different times in the area. The earliest reports are that the floodplains of these streams were poorly drained and the streams sluggish, thereby implying that the streams were at grade or equilibrium. However, by 1900 Udden reported that in Pottawattamie County, "many small creeks which now have well established furrows twenty feet deep, requiring good bridges for wagon roads, could be crossed by teams and heavy vehicles almost anywhere in the early days before the country was settled" (Udden, 1900). Although the bulk of the data on degradation is after stream straightening began (about 1919), the observation on the Tarkio indicates that degradation had occurred between 1846 and 1932. Piest and his associates argue that degradation in these streams resulted from increased runoff as the land was cultivated and the native prairie plowed. Normal runoff due to intensive cultivation has increased 2 to 3 times and peak flow rates more than 10 to 50 times the presettlement amounts (Piest et al., 1976; Piest et al., 1977).

Using geological evidence, Daniels and Jordan (1966) conclude that there were two cycles of degradation in Harrison County. One started about 1880 as a result of increased cultivation and a second after the construction of the Willow drainage ditch. In the case of the Willow ditch, the gradient of the channel was increased by straightening from 5.18 to 7.66 ft/mi in the lower reaches and from 7.50 to 8.48 ft/mi in the upper reaches. This increase in slope would initially increase the velocity of the water in the channel, thereby increasing the erosive potential of the stream.

There is evidence for two cycles of degradation in the Missouri River along the western border of Iowa. Lohnes et al. (1977) used geomorphic and historical evidence to conclude that vertical degradation of 10 to 12 ft occurred on the Missouri in the reach midway between Sioux City and Omaha sometime between 1804 and 1879. These same data indicate that the vertical position of the channel was stable between 1879 and 1952. This could have been the response of increased discharge resulting from the transition from prairie to farmland, or from large-scale, regional geologic dynamics. Sayre and Kennedy (1978) used stage-discharge data to document degradation on the Missouri between 1952 and 1978.

The data indicate that although there is degradation at Sioux City of about 8 ft, there is no apparent degradation at Omaha; and at Kansas City the river is aggrading. The combined roles of channelization and clear water release from the upstream dams in causing this modern degradation cannot easily be differentiated. In any event, most of the western Iowa streams that degrade flow into the Missouri near Omaha. Thus, modern entrenchment of the Missouri is not a cause for these degrading tributaries. However, streams with confluences at and upstream of the Little Sioux River, including the Soldier River, could be influenced by the Missouri River degradation.

The mechanism of channel degradation in the loess hills of western Iowa was interpreted by Daniels, based on his observations. This will be modified, based on the expanded set of observations. Daniels' ideas were influenced by his observation of the headward migration of a knickpoint 4,730 ft between November, 1956 and August, 1958. His observations are shown graphically in Fig. 13 where it can be seen that the headward migration goes on at a sporadic rate varying from 0 to 150 ft/day with an average rate of 7.3 ft/day. Daniels (1960) concludes that degradation or entrenchment is a process of upstream migration of knickpoints. He states that upstream of a knickpoint there is little degradation but that entrenchment continues downstream after passage of the knickpoint. This is true when the process is viewed from the short-term data, i.e., less than two years, and over a length of stream less than a mile. However, Daniels (1960) indicates that the reach of stream where he observed the knickpoint migration also experienced over

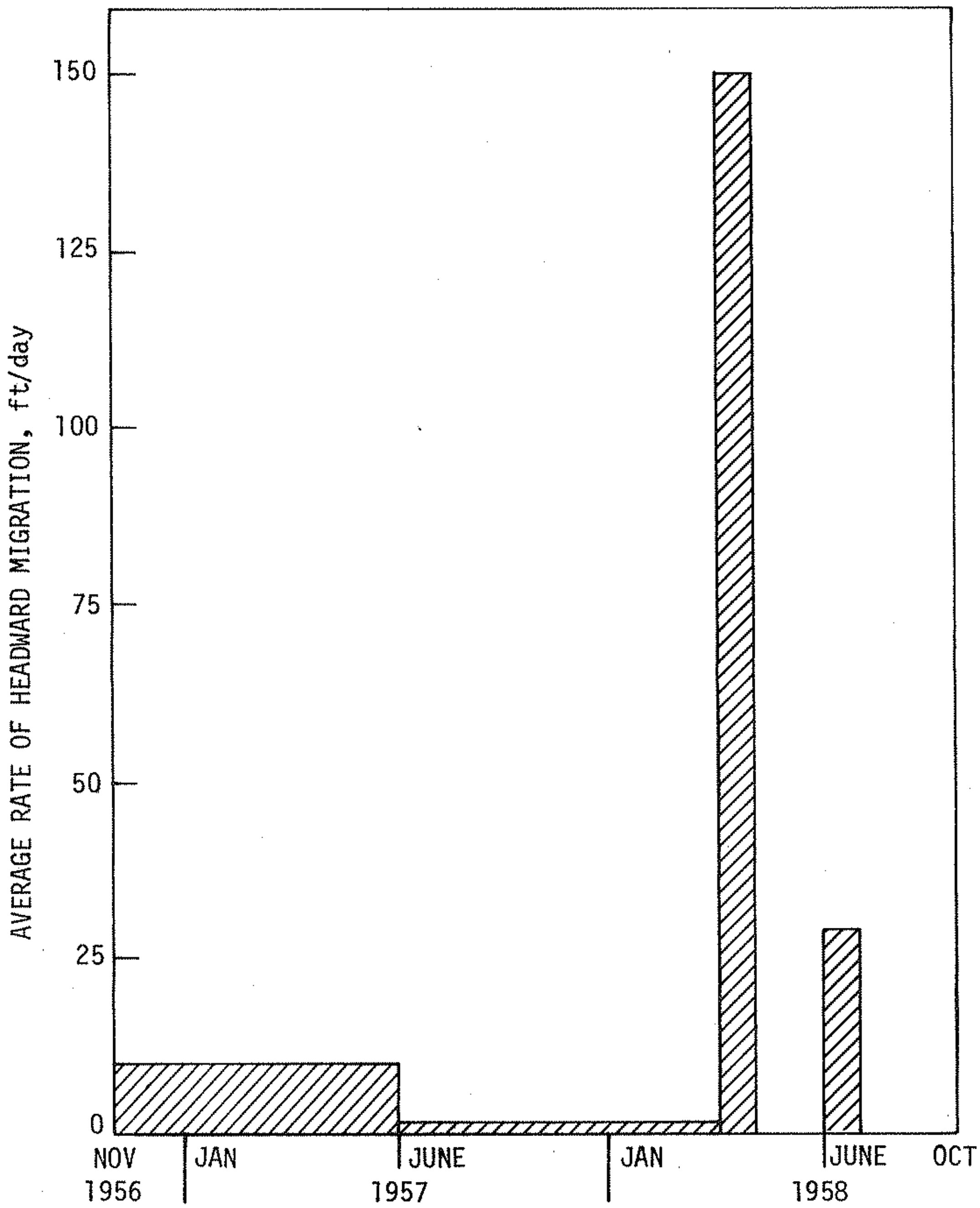


Fig. 13. Average Rate of knickpoint migration on Willow Creek. Plotted from data reported by Daniels (1960).

20 ft of degradation prior to the passage of the knickpoint. Also, in a later paper (Daniels and Jordan, 1966) it was concluded that in 1958 the portion of the channel downstream from the knickpoint was a stable channel. This later interpretation is supported by the 1966 measurements on Willow Creek and the data on Keg Creek. Thus it is concluded that degradation begins when the equilibrium of the stream is disturbed, and that significant amounts of downcutting may occur prior to the passage of knickpoints through a particular reach of the stream. Flume studies (Holland and Pickup, 1976) have demonstrated that some degradation may proceed upstream from a knickpoint. When a knickpoint does pass through a reach there will be a dramatic increase in channel depth and downcutting may continue for a short distance downstream from the knickpoint. However, once the knickpoint has passed, the streams appear stable with regard to vertical degradation.

Daniels and Jordan (1966) used the empirical equation of Hack (1957) to predict the amount of downcutting upstream from the 1958 knickpoint. Hack (1957) demonstrated that the longitudinal profile of a stream in equilibrium, and flowing over uniform materials throughout its length, is approximated by the equation

$$B = C - k \ln(L)$$

where B is the altitude, L is the distance along the stream measured from the head of the stream, k is the slope of the line on a semilog plot, and C is a constant. Note that all previous data on stream length in this report are plotted as distance from the mouth. Since the total length of Willow Creek is 45 miles, all distances in previous plots have to be subtracted from 45 to get L. Daniels and Jordan (1966) predicted an additional degradation of 35 ft at mile 29 and and additional 37 ft at mile 39 by extrapolating the value of k from that reach of the stream which they presumed to be in equilibrium. The analysis of the data from mile 30 between 1947 and 1968 indicates 20 ft of degradation. At mile 30 Daniels and Jordan predicted a maximum of 20 ft after 1958. The observed amount of degradation is less than predicted but lends credibility to the method of prediction.

Associated with the progressive deepening of the streams is an increase in channel width. Ruhe and Daniels (1965) used the systematic trends in increasing width and depth of the Willow to illustrate that the stream was adjusting its hydraulic geometry including width, depth and slope to achieve new equilibrium in response to the man-made changes in discharge and/or channel slope. Also, mass movement of the soil along the channel sides will occur as the channel deepens and the shearing forces resulting from soil weight and seepage exceed the shearing resistance of the soil. Field shear strength measurements and the Culmann analysis have been used on friable loess in western Iowa to demonstrate that, on the average, a vertical cut can exist to a depth of 15.5 ft. Once this depth is exceeded a landslide will occur, producing an average slope angle of 77°. Assuming the debris at the toe of the slope is removed, the resulting slope will be stable until the downcutting exceeds a depth of 22.7 ft, whereupon a second episode of slope failure will occur (Lohnes and Handy, 1968). The foregoing analysis ignored the effect of seepage forces which would decrease the maximum stable channel side slope height. It has been observed that in the lower reaches of the Willow below mile 20, the channel side slopes are vertical; but upstream from mile 20 there is considerable evidence of slumping (Daniels and Jordan, 1966). The maximum depth of channel also occurs at about mile 20 where the distance from the channel bottom to the floodplain is about 40 ft. This implies that the shear strength of the alluvium is greater than that of the friable loess in the uplands.

4. BRIDGE PROBLEMS ASSOCIATED WITH DEGRADATION

4.1. Perceptions of the County Engineers

The purpose of stream straightening was to decrease flooding and improve drainage, an objective which has been met. For example, there has been no overbank flooding along straightened channels in Monona County since 1942; even under the most extreme flooding conditions, the channels in the upper reaches of the Willow were flowing only half full. In the lower reaches of the streams, flooding occurs only occasionally (Daniels, 1960; Daniels and Jordan, 1966). However, the degradation of the streams, the associated entrenchment of their tributaries, and the formation of new gullies in the upland has taken its toll on the agricultural land. It was estimated in 1962 that damage associated with gullying averaged $\$719/mi^2$ in 8 watersheds (Beer, 1962).

No estimates on the cost of damage to highway bridges in this region were found, but engineers in western Iowa are concerned about the effects of degradation. Vertical degradation undercuts footings and exposes piles. The associated mass movement and widening of channels has removed soil adjacent to and beneath abutments and in many cases made it necessary to add approach spans to bridges built before degradation occurred.

In an attempt to define the scope of the problem, county engineers in 12 of the 13 counties were interviewed to obtain their perceptions of the bridge problems associated with degradation. Figure 14 is a map showing the location of structures that are of most concern to

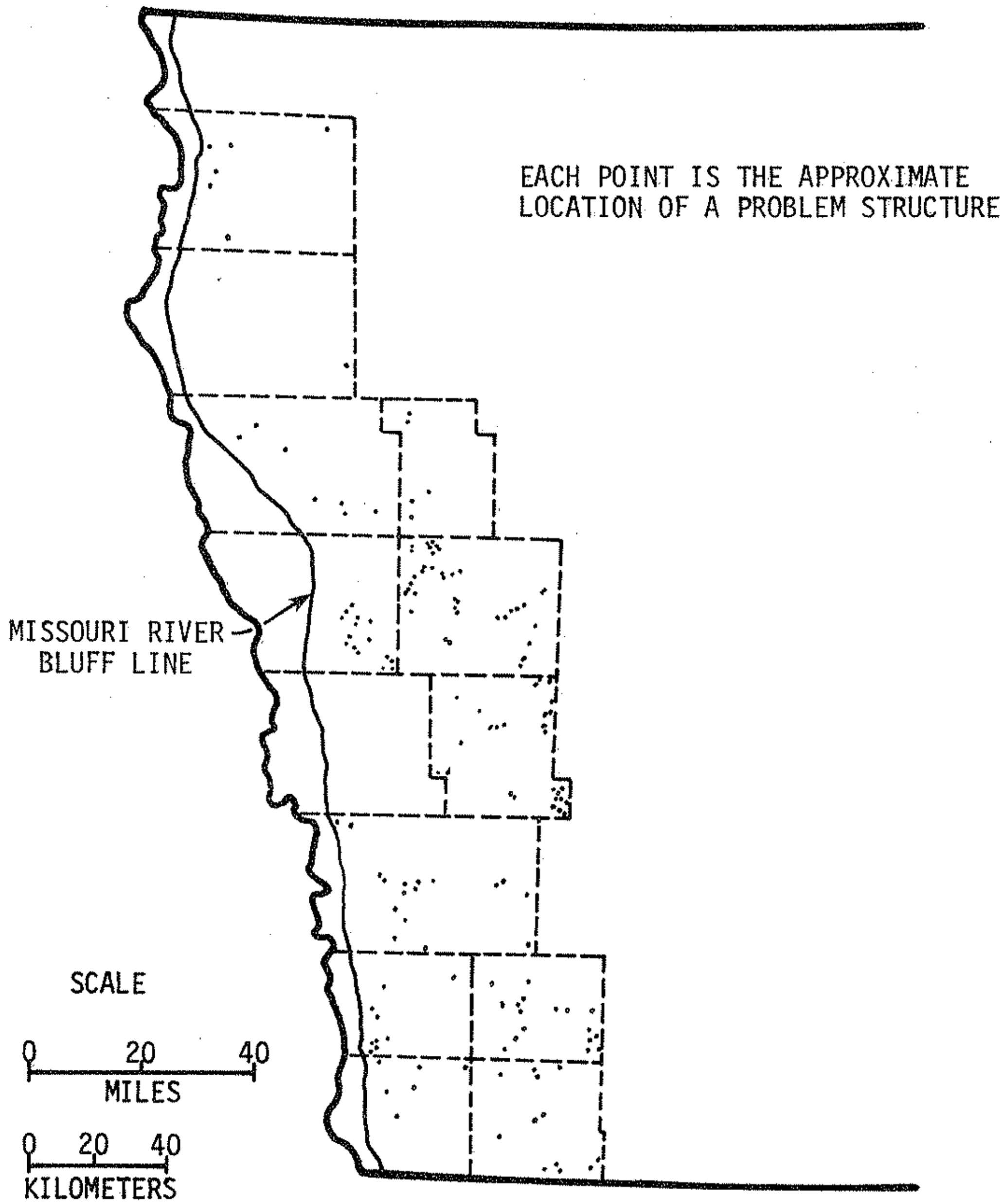


Fig. 14. Geographic distribution of structures endangered by degradation as perceived by county engineers.

these engineers. It is clear that in the engineers' view the area of thick loess deposition is the most problematic. Also, the engineers' concerns seem to be associated with structures in the upstream reaches of the drainage system. This observation reinforces the earlier conjecture that downstream portions of the western Iowa streams may be at equilibrium and no longer downcutting.

4.2. Bridge Inventory Data

It is recognized that this type of problem assessment is subjective and so an attempt was made to be more objective and quantitative by using data from the bridge inventory of the state highway department. The Facility Record Section of the Highway Division, Iowa Department of Transportation, currently maintains base record files on primary, secondary and municipal structures in Iowa. These records are complete except for some counties where data are still being collected. In addition to basic information, these files included structure inventory and appraisal information. After reviewing the various categories of appraisal information available and discussing it with maintenance personnel of the Iowa DOT, the channel condition rating and waterway adequacy were selected as being indicative of structures in difficulty due to channel problems. Channel ratings vary from 9 (new condition) to 0 (bridge conditions beyond repair--danger of immediate collapse). A rating of 6 (major items in need of repair) or less was selected as indicative of a problem structure. Channel rating includes such items as stream stability, conditions of riprap, condition of spurs, etc. Waterway adequacy also varies from 9 to 0 with 9 indicating conditions superior to present desirable criteria and 0 indicating immediate replacement necessary to put back in service. An adequacy code of 4 (condition meeting minimum tolerable limits to be left in place as is) or less was construed as indicating a structure with a problem or a potential problem. The waterway adequacy includes such items as slope protection, scour erosion, condition of slope protection, etc.

Data obtained from a computer review of the base record files are given in Table 5. Except for the total number of bridges in each county, the data obtained for five of the counties are incomplete; these counties are indicated in the table with an asterisk. These data exclude structures on the Missouri River floodplain. Table 5 presents percentage of bridges in each county with a channel rating of 6 or less, a waterway adequacy of 4 or less, or a combination of the two. Bridges having both a channel rating and a water adequacy less than the previously described limits will be referred to as problem structures. The geographical distribution of problem structures is shown in Figs. 15 and 16.

In interpreting these data, it should be remembered that in some counties the data are incomplete and in some instances the channel rating and waterway adequacy problems may be caused by lateral channel migration as well as by degradation. However, based on field observations, it is felt that the vast majority of problems are the result of degradation. Figure 15 shows both the percentage and the number of problem structures. In the 13 counties, 750 bridges

Table 5. Results of Bridge Inventory Rating of Counties in Western Iowa.

County	Number of Bridges	Percentage of Bridges with Channel Rating of 6 or Less	Percentage of Bridges with Waterway Adequacy of 4 or Less	Percentage of Bridges with Channel ≤ 6 and Waterway ≤ 4
Crawford	449	53.9	51.9	43.0
Fremont	238	13.4	23.9	8.8
Harrison*	302	4.0	2.0	1.0
Ida*	222	3.2	1.4	0.9
Mills	283	35.0	35.0	26.2
Monona	220	27.3	23.2	16.8
Montgomery	288	31.9	31.9	22.6
Page	349	10.6	9.2	3.4
Plymouth*	600	2.2	0.5	0.2
Pottawattamie	877	42.8	38.3	31.2
Shelby	378	30.4	31.7	26.5
Sioux*	425	5.2	1.2	0.2
Woodbury*	598	5.5	1.8	1.3

Table 5. (continued).

County	Number of Bridges	Percentage of Bridges with Channel Rating of 6 or Less	Percentage of Bridges with Waterway Adequacy of 4 or Less	Percentage of Bridges with Channel ≤ 6 and Waterway ≤ 4
Total (All 13 Counties)	5229	21.8	20.0	15.1
Total (Only 8 Counties with Complete Records)	3082	34.1	33.1	25.2

*Counties with incomplete data.

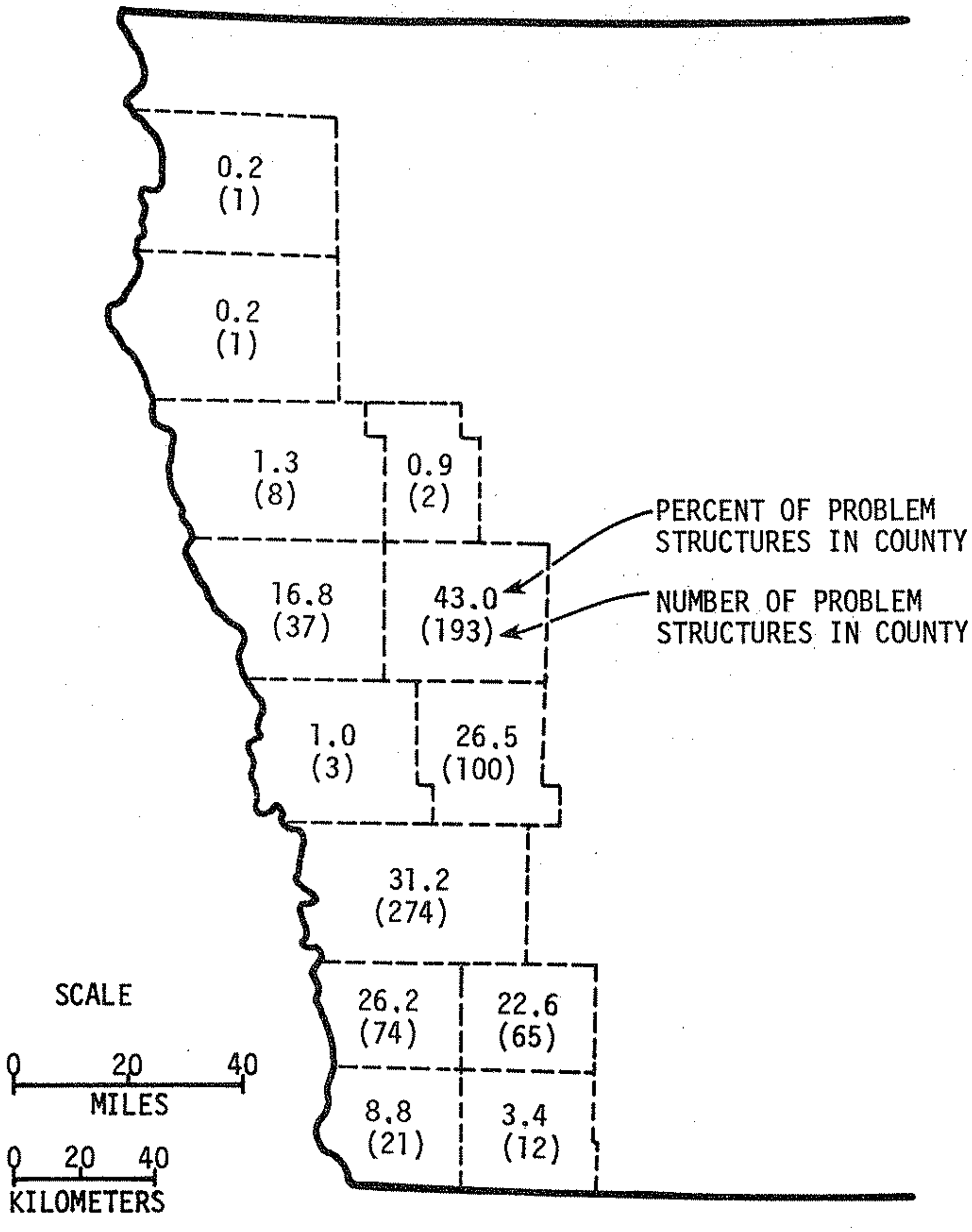


Fig. 15. Percentage and number of problem structures in western Iowa.

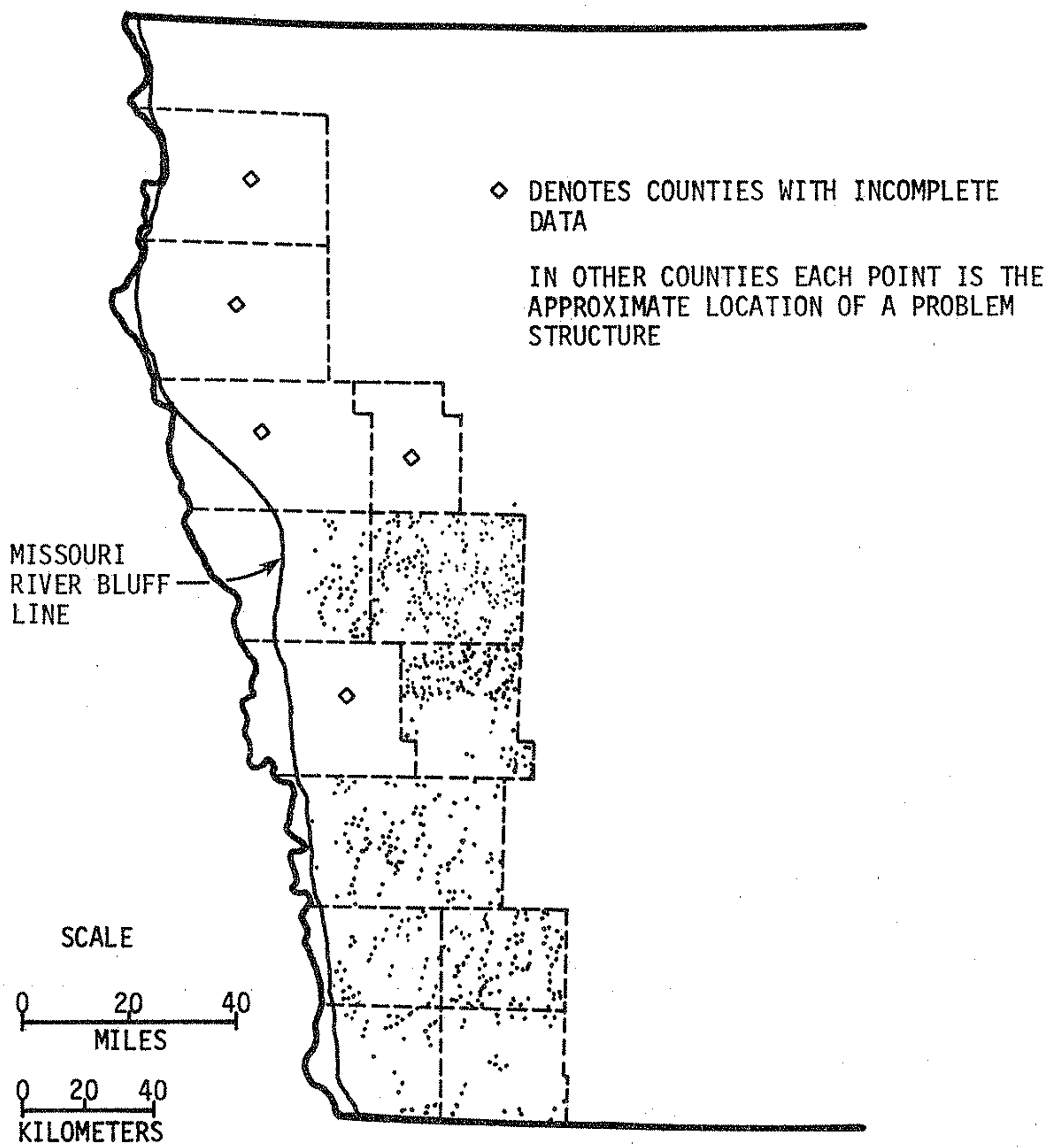


Fig. 16. Location of problem structures as determined by data from Iowa DOT bridge inventory.

are problem structures. This represents over 25% of the existing bridges. It also appears that the majority of the problems are associated with the deep loess deposits in the southern portion of the study area and that the upstream reaches of the rivers present most of the problems. In general, the computer survey from the bridge inventory showed trends similar to the county engineers' perceptions.

As pointed out earlier, channel degradation leads to landslides which cause an increase in channel width. An attempt was made to quantitatively define the problem by determining the number of bridges in the area with approach spans. In western Iowa, presence of approach spans usually indicates that spans have been added since construction (Plate 6). The Iowa DOT bridge inventory data on the number of bridges with approach spans are summarized in Table 6. These data indicate that problem structures are more abundant in counties with deep loess deposits. The bar graph in Fig. 17 shows (for the entire 13-county area) the distribution of the number of approach spans among the bridges having such spans. Although the majority of the bridges have fewer than 3 approach spans, there are 14 bridges with 8 or more approach spans. Of the 5,223 bridges in the 13-county area, nearly 18% have one or more approach spans.

Plate 6. Stream degradation has exposed the bottom of the piles in this Crawford County bridge and caused a landslide which pulled soil from behind the abutment. Note that an approach span has been added to accommodate the increased valley width.

Table 6. Data on Approach Spans.

County	Number of Bridges	Percentage of Bridges with Approach Spans	Maximum Number of Approach Spans on a Bridge
Crawford	449	25.4	8
Fremont	238	9.7	4
Harrison	302	19.5	8
Ida	222	5.0	4
Mills	283	17.0	10
Monona	220	15.9	9
Montgomery	288	23.3	8
Page	349	18.9	7
Plymouth	600	7.7	6
Pottawattamie	874	12.8	7
Shelby	378	28.3	6
Sioux	423	12.7	10
Woodbury	597	31.9	8

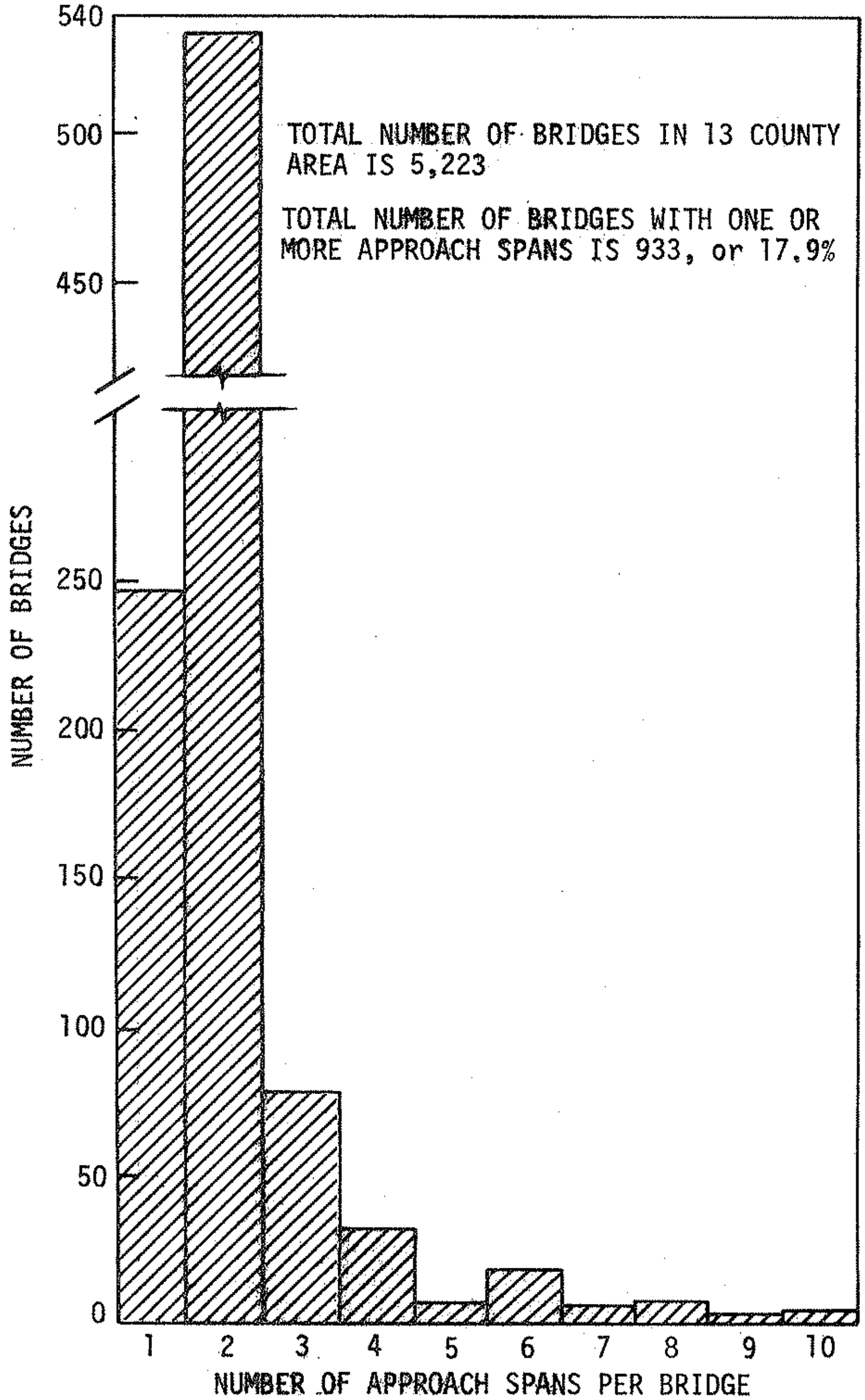


Fig. 17. Number of bridges having various numbers of approach spans.

5. PREDICTION OF DEGRADATION

5.1. Estimating Depth of Entrenchment

In the case where degradation is occurring and endangering a bridge, it would be useful to have a method for estimating how deep the channel will degrade, the rate of downcutting, and the width of the channel once the stream has ceased its entrenchment. An analysis of the history of the degradation of Willow and Keg Creeks suggests an approach. The methods outlined here are based on limited data and therefore are tentative.

The assumptions underlying the prediction method are that the most recent period of downcutting in western Iowa is the result of channel straightening and that the average discharge has remained essentially constant since the turn of the century. Also, it is presumed that the streams were near equilibrium prior to straightening. After straightening, the streams have approached a new equilibrium by adjusting their channel geometry. The variables that would adjust to the increased velocity resulting from the higher gradient of the straightened channel are width, depth, and slope. It is also assumed that magnitude of the average discharge over the years has been most important in shaping channel geometry.

The earlier sections of this report have offered historical data on the longitudinal profiles as evidence that the lower portions of the stream achieved equilibrium within 40 years after straightening. An additional argument can be made, using principles of hydraulics, to show how the channels have adjusted width, depth, and slope. The equations used in this analysis are the equation for the channel discharge and Manning's equation. The discharge flowing through a channel will be

$$Q = vA$$

where Q is the discharge, v is the velocity, and A is the cross sectional area of the channel. Manning's equation relates the velocity of flow to channel geometry according to

$$v = 1.49 \frac{R^{2/3} s^{1/2}}{n}$$

where R is the wetted perimeter or A/(2d + w) with d being depth of flow and w being the width of channel, s is the slope, and n is the roughness coefficient. In this analysis n is assumed to be 0.025 and the area is approximated by depth times width. Thus, the preceding equation becomes

$$v = 59.6 \left[\frac{dw}{2d + w}\right]^{2/3} s^{1/2}$$

This gives two equations which can be used to show how velocity at a given section in the channel has changed with time. From the published data, channel width and slope are known, but depth of flow and velocity are unknown. The two equations with the two unknowns allow the calculation of velocity for a given discharge. Table 7 shows the discharge, estimated from Lara's equation, and data on the geometry of the channel at various times since

Table 7. Channel Velocity Variations with Time for Sections of Willow Creek.

Station	Distance From Mouth	Discharge*	Date	Slope (%)	Channel Depth (ft)	Width† (ft)	Depth of Flow (ft)	Velocity (ft/sec)
			1919	0.145	15	22	0.725	1.76
			1935	0.161	23	45	0.44	1.39
1	11.4 miles	28 cfs	1936	0.166	27	49	0.42	1.34
			1958	0.127	32	66	0.38	1.1
			1966	0.127	39	68	0.375	1.09
			1919	0.145	15	22	0.68	1.68
2	15.4 miles	25 cfs	1931	0.161≠	24	40	0.44	1.4
			1958	0.127	33	66	0.36	1.06
			1966	0.127	35	69	0.35	1.04
			1920	0.160	11	20	0.53	1.52
			1929	0.161	20	18	0.57	1.56
3	19.7 miles	16 cfs	1942	0.165	32	33	0.37	1.31
			1958	0.127	37.5	53	0.31	0.95
			1966	0.127	47	60	0.29	0.92

*Estimated from $Q = 0.17A^{1.06}$ (Lara, 1979).

†As measured 5 ft above channel bottom.

≠Estimated, no data.

straightening. Figure 18 shows a plot of velocity versus time for the three sections supposedly in equilibrium. These graphs seem to support the conclusion that the velocity has tended to approach a limiting value within the last 25 years, and suggests that the channels have now adjusted their geometries such that the velocity is stable with respect to time.

If a portion of the channel is in equilibrium, the key to predicting the amount of downcutting in a nonequilibrium or degrading portion of the channel would be the longitudinal profile of the channel bottom or the flowline. The longitudinal profile of a stream in equilibrium and flowing over uniform material is concave upward. Hack has related the slope of the channel to the median particle diameter of the bed material such that the greater the decrease in particle size with increasing distance from the headwaters, the more concave the longitudinal profile. Leopold et al. (1964) showed that for a stream with bed material of constant diameter downstream, it is still possible for the stream to have a profile concave to the sky. Hack (1957) has shown that the longitudinal profile of a stream can be described by the equation

$$B = C - k \ln(L)$$

as introduced earlier in this report. This relationship plots as a straight line on semilog paper. Daniels plotted the 1958 profile of the Willow and noted that in the lower reaches, one slope was apparent whereas in the upper reaches a different slope value of k existed. He then assumed that the lower reaches of the stream were in equilibrium

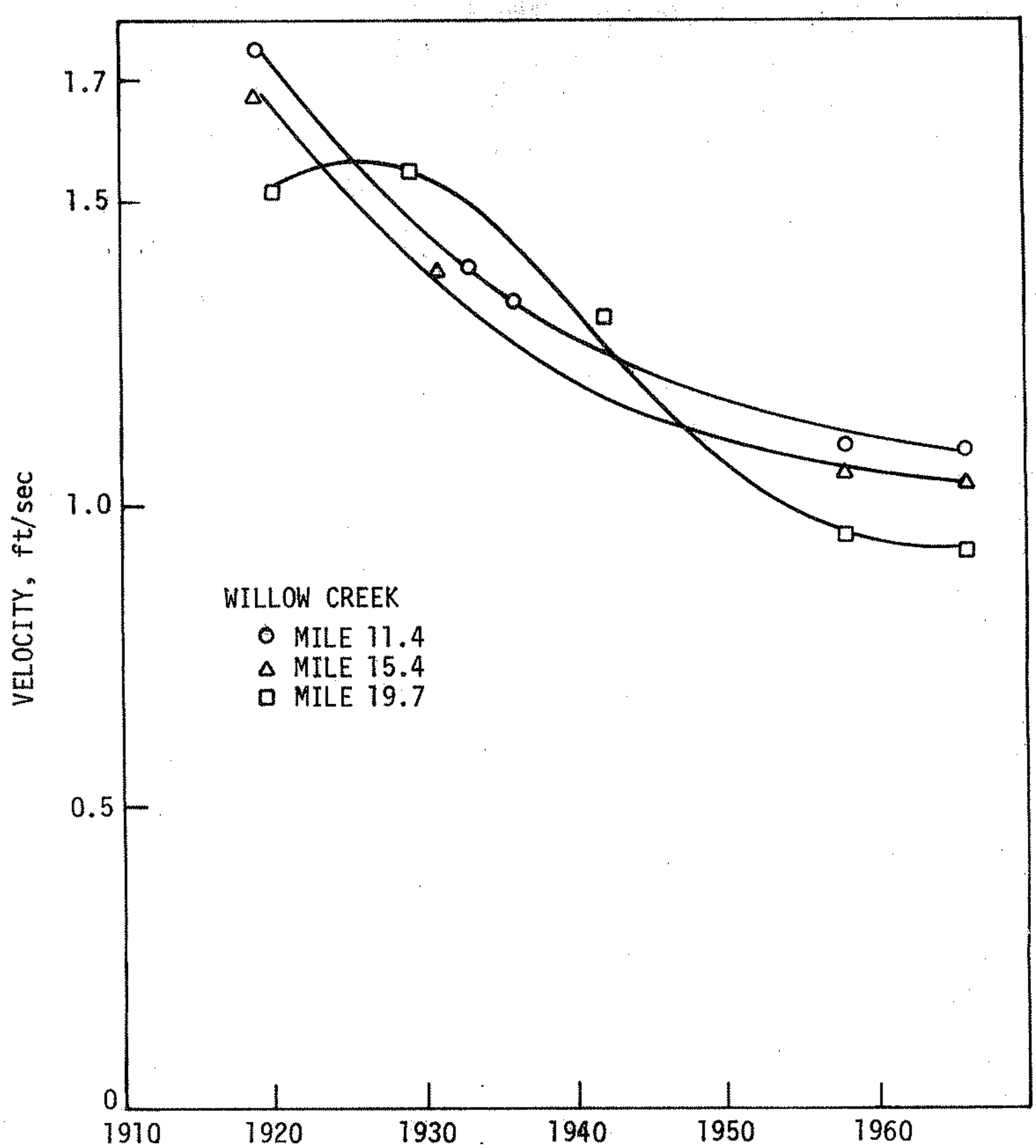


Fig. 18. Velocity variation with time for 3 sections of the Willow Creek.

and that the difference in the slopes of the two lines on the semilog plot indicated the amount of entrenchment the upper reaches of the stream would undergo before reaching equilibrium. By projecting the line of the downstream reach upstream on the graph and comparing the present elevation at that point with the elevation on the linear extrapolation, he predicted the amount of downcutting that would occur before the stream reached equilibrium. Figure 19 shows the semilog plot of elevation versus distance from the head of the stream for the 1958 and 1966 data. Note that the 1966 and 1958 profiles coincide below mile 19.5, but diverge upstream. The data points at mile 17.4 and 18.4 fit nicely on the same line established by the lower reaches of the stream. It can also be seen that the reaches of a stream below Willow Drainage District No. 1 (miles 39 to 45) also fit the semilog relationship very well. Figure 19 is simply a graphical presentation of the conclusion stated in a previous section of this report, i.e., Hack's equation for the longitudinal profile of a stream can be used to predict the amount of degradation by extrapolating the straight line on a semilog plot upstream from the equilibrium reaches to those reaches actively downcutting and noting the elevation difference between the degrading channel and the extrapolated line. The difference is the amount of entrenchment that will occur before equilibrium is reached. Having estimated the amount of downcutting, the next problem is to determine a method for estimating the rate of degradation.

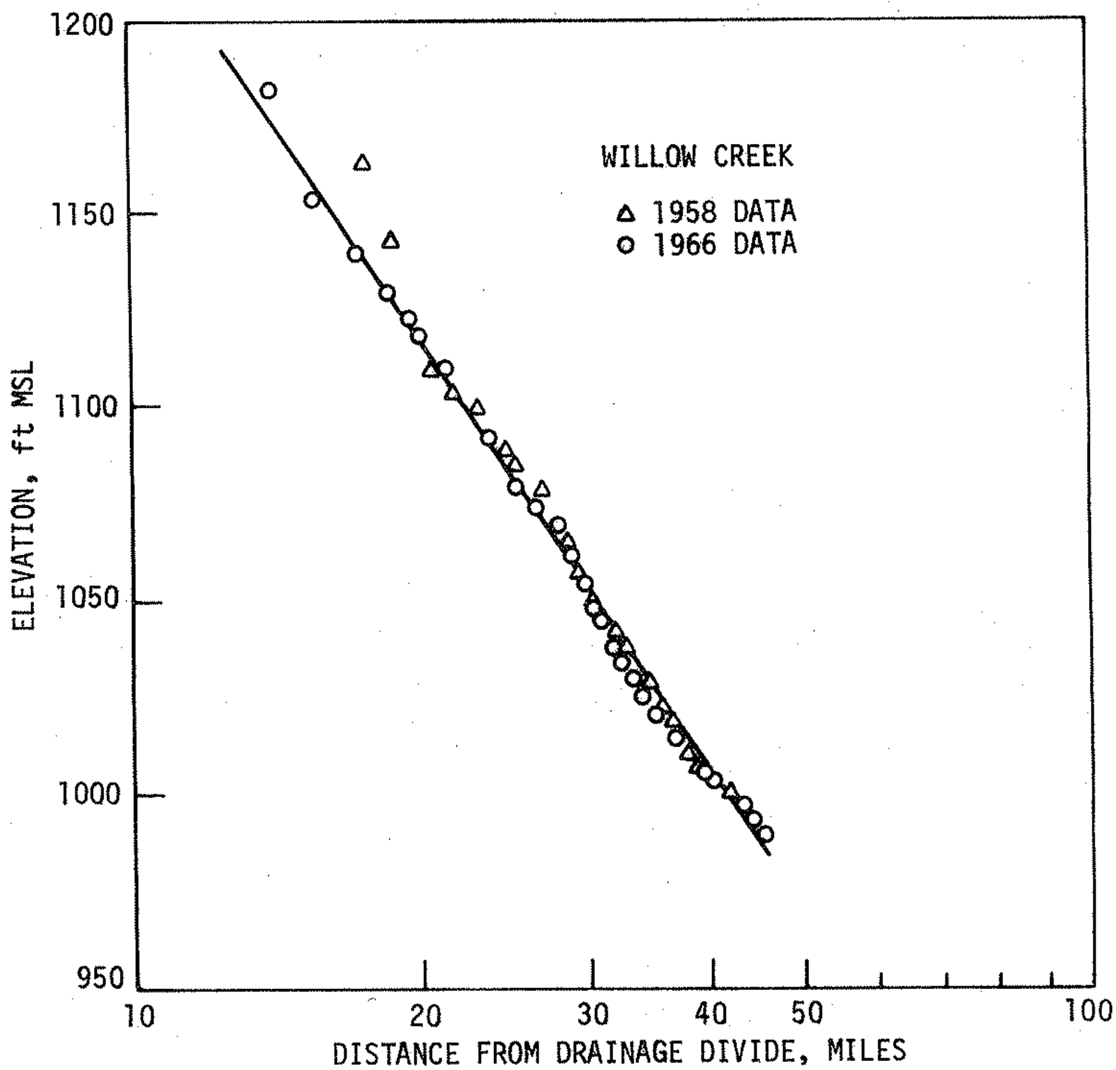


Fig. 19. Longitudinal profile of Willow Creek on semilog plot.

5.2. Rate of Degradation

A discussion of the rate of degradation becomes complex because of the role of knickpoint migration. It has been observed that the "movement of knickpoints is erratic" and that "there is no statistical correlation between movement of knickpoints, size of watershed, and proportion of watershed under cultivation" (Daniels and Jordan, 1966). The graphical display of Daniels' (1960) data in Fig. 13 amplifies that observation. However, 90% of the knickpoint growth occurs between November and April (Daniels and Jordan, 1966) suggesting that freeze-thaw action combined with high runoff tends to make the knickpoints migrate faster. As stated earlier in this report, knickpoint migration is one of the more dramatic aspects of channel degradation and one of the most observable over the short term, but there is laboratory and field evidence to indicate that degradation goes on prior to and following the passage of a knickpoint.

The rate of vertical channel degradation decreases with increasing time. This is supported by the observations of Daniels (1960, pp. 167-168) and by Daniels and Jordan (1966, p. 65) although neither makes that statement explicitly. Ruhe and Daniels (1965) empirically derived an equation describing the rate of vertical degradation decreasing with time. If depth of channel is D and time is T then

$$D = 1.8 + 20.9 \log T$$

No explanation or theory is given for this relationship.

In this report a rational theory of the rate of vertical degradation is proposed and evidence from previous work and this study will be offered in support of the theory.

It must be recognized that on the basis of short-term observations, the rate of entrenchment will appear to be sporadic; however, when averaged out over the long term there is a systematic decrease in the rate of downcutting with increasing time.

The theory presented here is that the rate of downcutting at a given reach of river is proportional to the elevation of that reach above base level of the stream. This can be expressed as

$$dh/dt = -k'h$$

where dh/dt is the rate of vertical degradation, h is the elevation of that reach of stream above base level, and k' is a constant describing the rate of degradation. The rate constant, k', should be a function of the discharge through that section. This conjecture is supported qualitatively by Daniels and Jordan (1966) who state that "the rate of deepening in Thompson Creek has been greatest in the lower reaches...." This concept assumes that discharge remains constant with time at a given reach of the stream, but increases downstream as the stream accumulates more drainage area. By separating variables and putting on the boundary conditions that h_o exists at $t = 0$ and h_1 occurs at t_1, the differential equation can be solved and takes the form

$$\ln(h/h_o) = -k'(t_1 - t_o)$$

or, if time is measured from the date of channel straightening

$$\ln(h_1/h_o) = -k' t$$

where t is the time since channel straightening, h_o is the original elevation above base level, and h is the elevation at some time after straightening. This equation is significantly different from the Ruhe and Daniels equation and has a rational basis. It is intuitively pleasing that if a stream at equilibrium is perturbed, the stream will adjust to a new equilibrium with the rate of adjustment decreasing as the new equilibrium is approached. This logic is identical to the logic used in explaining the rate of stream sinuosity increase after cutoff (Handy, 1972; Lohnes et al., 1979). The second equation indicates that a plot of ln(h) versus t should be linear. Also the slope of the line, -k, should be proportional to discharge at that reach. Figure 20 shows these plots for the reaches of the Willow using Ruhe and Daniels' (1965) data and for the Tarkio using data from Piest et al. (1977). It can be seen that there is a good linear relationship. Further, the rate constant, -k, for each reach is proportional to discharge at that reach. These observations tend to support the theoretical equation, but it must be recognized that this relationship is based on limited data. However, if this relationship is verified by further observations, it should be a valuable tool for predicting the rate of entrenchment of streams degrading as a response to straightening.

An obvious limitation of the rate equation presented here is that theoretically the channel would never reach equilibrium but

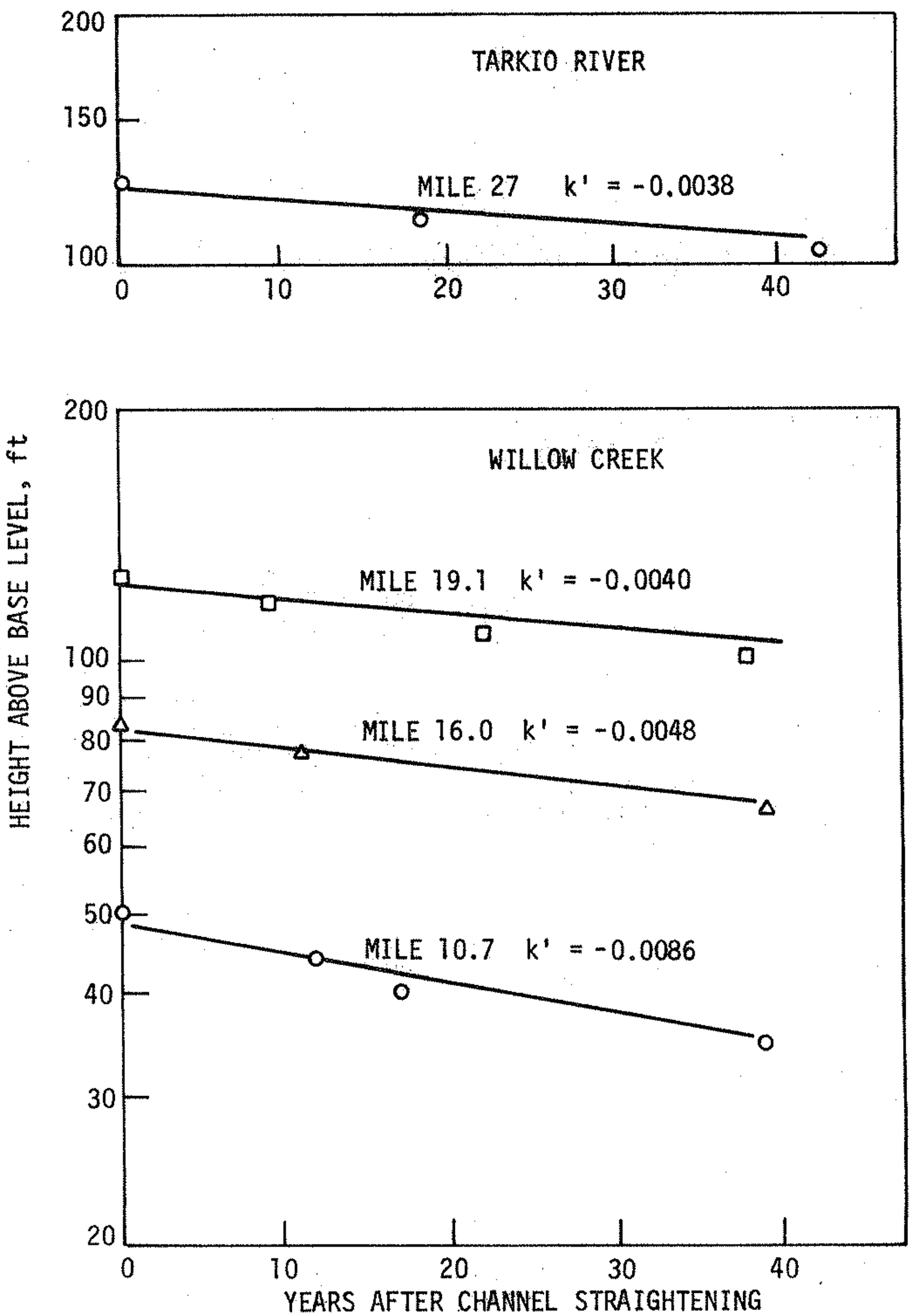


Fig. 20. Time versus log height above base level for reaches of Willow and Tarkio Rivers.

would approach the equilibrium depth at an ever decreasing rate. The analysis of the velocity data in Fig. 18 also shows the velocity approaching a limiting value asymptotically. From a practical standpoint, the very slow rate of degradation in the latter stage of the downcutting event indicates that entrenchment has essentially ceased until the equilibrium is once again perturbed.

5.3. Estimating Channel Width

The width of the channel at equilibrium can be estimated by applying the statistical observation (Ruhe and Daniels, 1965) that the width-to-depth ratio has remained nearly constant through time. Thus, if the maximum depth of the channel has been estimated, the equilibrium width can be calculated by multiplying the depth by the empirically determined width-depth ratio. It is suggested that this method can be refined by applying principles of soil mechanics and slope stability as suggested previously in this report. Unfortunately there was insufficient time to gather specific strength data as part of the present study. It is hoped that future studies will explore this area.

Once again, the methods suggested here are tentative because of limited data. These ideas will be explored as part of the thesis research of Dirks (in preparation) and Massoudi (in preparation) and the interested reader is referred to their work for verification or modification of these concepts.

6. GRADE STABILIZATION STRUCTURES

The preceding sections of this report are devoted to developing a better understanding of the degradation process in western Iowa and to developing methods of estimating the amount and rate of degradation at a given location to help in the planning of grade stabilization structures. The remainder of this report will discuss factors to be considered in the design of grade stabilization structures.

It is recognized that grade stabilization structures are needed in situations where streams are actively degrading or where streams, although in equilibrium, have weakened the foundations of bridges from previous degradation.

The full flow structures currently used in western Iowa include flume bridges (Plate 7), flexible drop structures of riprap or derrick stone with a sheetpile control crest (Plate 8), a double row of sheetpile as a control crest using less riprap (Plate 9), earth embankments with drop inlets or broken back culverts, reinforced concrete vertical drop structures of Soil Conservation Service (SCS) design (Plate 10), or reinforced concrete chute spillway structures with a Saint Anthony Falls (SAF) stilling basin at the end of the chute. In most cases all of these structures have performed well. In cases where grade stabilization structures have not performed well, it appears that the seepage through the earth embankment portions of the structure has not been controlled, either in design or construction. In other situations erosion at the downstream

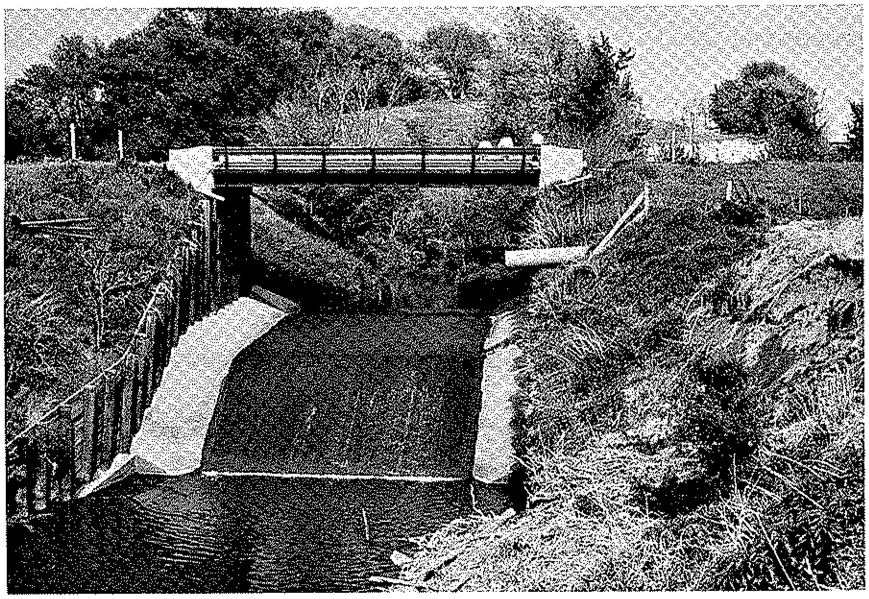


Plate 7. Flume bridge in Fremont County. Note wood piles inside the flume.

Plate 8. Grade stabilization structure consisting of a single row of sheetpile reinforced with derrick stone.

Plate 9. Grade stabilization structure consisting of double row of steel sheetpile with a concrete core and cap. Note large crack in right side resulting from adjacent landslide caused by downstream scour in plunge pool.

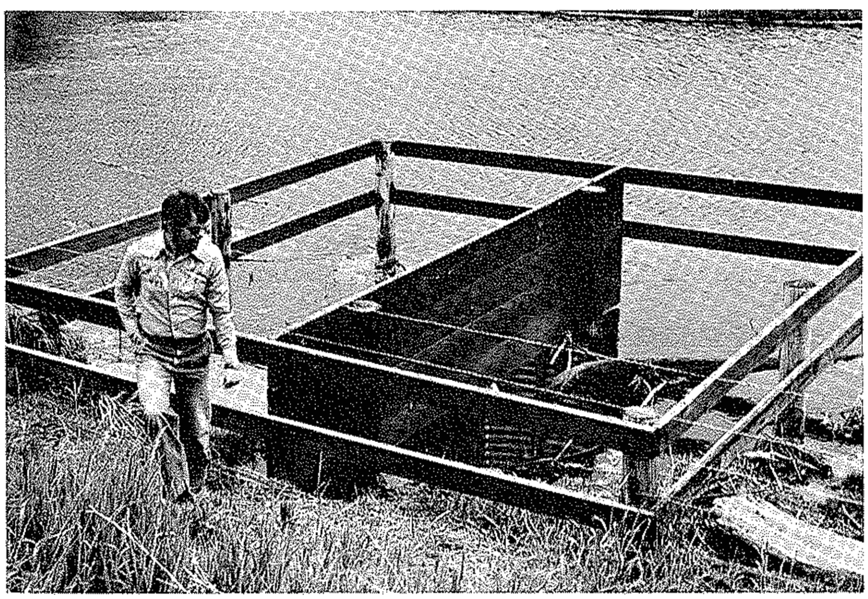


Plate 10. An SCS-type drop inlet for a grade stabilization structure. Note baffle plate to disrupt vortices and trash rack.

side of the structure has caused problems. For these reasons a subsequent portion of this report considers energy dissipation downstream of the drop, and seepage through the embankments.

A recent nationwide study (Brice et al., 1978) sponsored by the Federal Highway Administration (FHWA) consisted of case studies of countermeasures for hydraulic problems at bridges and, in some cases, evaluation of their performance. Of the 283 case studies cited, 61 concerned degradation problems; 42 of these were evaluated. This report synthesizes their observations with regard to degradation problems and countermeasures and assigns a numerical rating to the performance evaluation.

The countermeasure performance rating included a threefold rating: of function, of damage to countermeasure, and, in some cases, of unwanted countermeasures effects. If the structure prevented or controlled the hydraulic problem, it was given a function rating of 4. If the countermeasure partially controlled the problem, it received 2 points. And if it had no effect, it received 0 points for the function rating. For the damage to countermeasure criterion, a grade of 4 was assigned if the structure sustained no damage and a 0 if it sustained damage. Only three countermeasures produced unwanted effects and these were scour downstream from the check dams.

Table 8 summarizes the performance ratings of the countermeasures studied. Main channel check dams consisted of riprap, gabions, concrete, rock-and-wire mattress, sheetpile, and concrete fabric mat. Check dams have the highest combined rating except for the grout curtain around the

Table 8. Performance Rating of Countermeasures for Degradation Problems at Bridges. Data from Brice et al. (1978).

Countermeasure	Performance		Number of Samples Evaluated	Number of Total Used	Percent of Total Used
	Function	Damage to Counter-Measure			
Main Channel Check Dams	5.9	3.4	7	11	18
Rigid Revetment and Bed Armor	3.0	2.8	12	16	25
Flexible Revetment and Bed Armor	1.6	1.8	18	21	33
Car Body	2.0	0	1	1	2
Underpinning or Jacketing Pier or Abutment	2.0	2.0	2	4	6
Addition of Bridge Spans	0	0	1	3	5
Driving Piles Deeper	---	---	---	2	3
Sheetpile Around Pier	---	---	---	1	2
Grout Curtain Around Foundation	4	4	1	1	2
Concrete Curtain Wall Between Steelpile	---	---	---	1	2

foundation (which was used in only one instance). The check dams as a group are the third most popular countermeasure. Of the 11 check dams documented, 3 were sheetpile, 3 gabions, and 2 riprap. None of the types of check dams appeared substantially more effective or ineffective than the others.

Flexible revetment and bed armor consist of riprap and rock-and-wire mattresses. Although this countermeasure is the most popular, it has the lowest performance rating. A comparison of the two subclasses indicates that dumped riprap contributed most to the low rating, whereas the rock-and-wire mattress seemed fairly effective. Rigid revetments included concrete pavement, sacked concrete, and concrete grouted riprap. Rigid revetments rank second in use and have a high performance rating. Little can be said regarding the other countermeasures because of the limited number of each type.

It appears from the FHWA study that grade stabilization structures (or check dams) and rigid revetments have been the most effective countermeasures for controlling channel degradation.

7. SOIL-CEMENT POTENTIAL FOR GRADE STABILIZATION STRUCTURES

Both the FHWA study and the experience of engineers in Iowa indicate that full flow grade stabilization structures and rigid revetment can be effective in controlling degradation around bridge piers. Both of these solutions employ concrete elements in the structure or use of portland cement grout; therefore the remedy is expensive. In an attempt to find a low-cost chemical stabilizing agent for western Iowa loess, a literature search was conducted regarding chemical stabilization of soils for erosion resistance (for example, Holtz and Hansen, 1976; Wilder, 1977; Morrison and Simmons, 1977). In addition, the authors met with William Morrison of the Bureau of Reclamation and with Kenneth Hansen of Portland Cement Association (PCA).

As a result of these meetings it was concluded that none of the chemicals studied by Morrison and his associates would be adequate for long-term stabilization of western Iowa loess; however recommendations were made regarding some chemicals that should work for short-term stabilization against erosion. The names of these chemicals, their manufacturer and the Bureau of Reclamations laboratory sample number are shown in Table 9.

Thus, of the stabilizing agents considered, portland cement remained as a long-term possibility. Although PCA does not recommend that portland cement be used in water resource applications with soils containing less than 55% passing the No. 4 sieve and more than 35% passing the No. 200 sieve (PCA, 1975), previous research at Iowa State University has shown that portland cement can be used effectively in highway applications.

Table 9. Chemicals with Potential for Short-Term Stabilization of Iowa Loess Against Erosion (Source: personal communication W. R. Morrison, Bureau of Reclamation, Engineering and Research Center, Denver, Colorado).

Manufacturer's Identification	Manufacturer	Bureau of Reclamation Lab Sample Number
Soil Seal	Soil Seal Corp.	B-5778
Aerospray 70 Binder	American Cyanamid Corp.	B-6471
Curasol AK	American Hoecht Corp.	B-6513
Crust 500	Firewater Co.	B-6654
Terra Krete No. 2	Terra Krete Co.	B-6738

Also, virtually no studies have been done on portland cement stabilization of loess for water resources applications.* The design of cement stabilized soils to resist erosion is based on durability and not on strength; it was the feeling of the authors that the conventional wire brush testing for durability (PCA, 1959) might be too severe for applications in low head grade stabilization structures. As a result, a preliminary testing program was established to evaluate the feasibility of using soil-cement in grade stabilization structures. This section will summarize the findings of the preliminary investigation of soil-cement in resisting erosion. For more complete information, the thesis of Travis (in preparation) and the report of Yang (in preparation) should be consulted.

7.1. Test Procedures

In the evaluation of cement-stabilized loess for erosion resistance many variables could influence the results of erosion and freeze-thaw tests, including: density, moisture content at compaction, percent cement, soil type, and size and shape of specimens. In order to limit the variables in this initial phase of the project, sample density and moisture content at compaction were determined in accordance with American Society for Testing and Materials (ASTM) 558. This resulted in specimens compacted to approximately Standard Proctor density at optimum moisture content. An additional variable was eliminated by using one

*Personal communication, K. D. Hansen, PCA, Denver, Colorado.

type of loess from a site at Turin in Monona County. This site has produced relatively consistent samples and a soil reasonably representative of loess-derived alluvium. The American Association of State Highway and Transportation Officials (AASHTO) classification for this soil is A-4(8). A list of chemical and geotechnical properties of the loess (known as a friable loess with highly dispersive properties) used in this part of the research is given in Table 10.

Previous tests (Handy et al., 1954) indicate that a soil-cement which would pass the freeze-thaw test (ASTM D560) requires inordinate amounts of cement. This is due in part to the fact that the loess particles are very fine and relatively uniform in size, resulting in a high void ratio and a large surface area of particles. However, it was also suggested that the freeze-thaw durability of cement-stabilized loess could be improved by the addition of sand (Handy and Davidson, 1957). Therefore, two series of test specimens were molded, one consisting of loess and cement and another with loess, cement, and coarse sand. The coarse sand constituted 20% by weight of the soil portion of the specimens. Cement contents of 3, 5, 7, 9, 11, and 13% were tested. The optimum moisture and density for the test specimens were 19% at 104.5 pcf with no sand, and 16% at 110.2 pcf with 20% coarse sand.

The size and shape of the specimens were designed to provide a large surface area for erosion tests. The dimensions were 12 in. × 12 in. × 2 in. thick, with a specimen weight of approximately 21 pounds. The specimens were pressed to the desired dimensions and density in a large steel mold (Plate 11) which can be disassembled for sample removal.

Table 10. Properties of Loess Used in Soil-Cement Tests.

Textural Class	Friable Loess Silty Clay Loam
Percent Sand	1
Percent Silt	82
Percent Clay	17
Liquid Limit, Percent	33
Plastic Limit, Percent	21
Plasticity Index	12
AASHTO Class	A-4(8)
CEC, me/100 gm	13.4
pH	7.8
Principal Clay Minerals	montmorillonite and illite
Probable Predominant Exchangeable Cation	calcium
Organic Matter, Percent	0.2
Carbonate Content, Percent	10.2

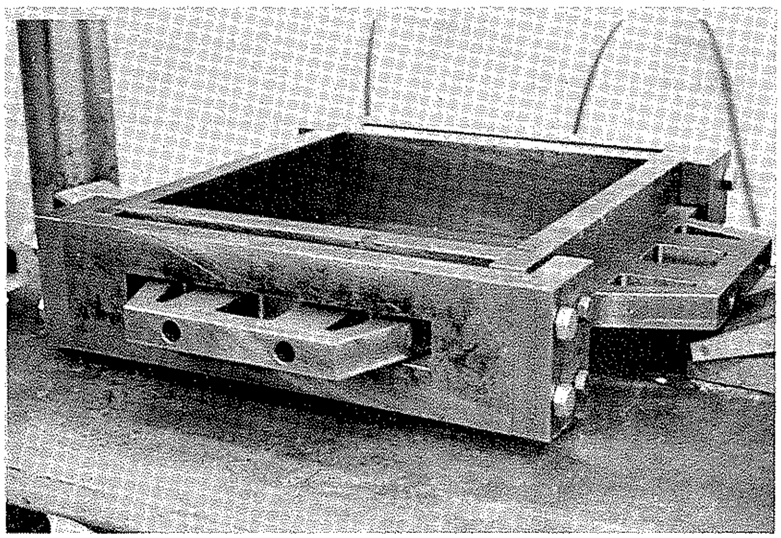


Plate 11. Mold used for preparing soil-cement specimens for erosion tests.

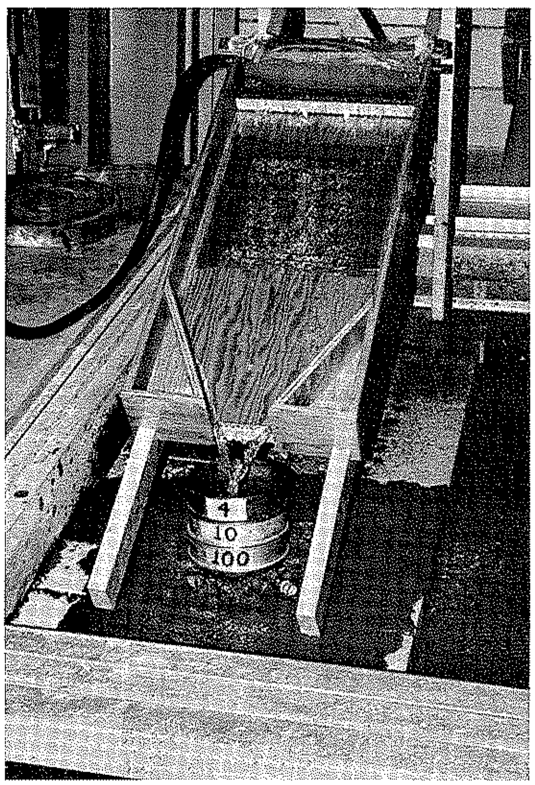


Plate 12. Erosion flume used in cement stabilization study.

The necessary force for achieving compaction is provided by a universal testing machine; fixed internal dimensions and rigidity of the mold assured consistent dimensions for the samples.

With the moisture content, density and sample size held constant, it is possible to determine the influence of cement content of loess and the loess-sand mixtures on erosion resistance after 10 cycles of freeze-thaw.

All test specimens were subjected to repeated freeze-thaw cycles followed by an erosion test after the 10th cycle. A full cycle of freeze-thaw consisted of 24 hours in the freezer at -20 ^{o}C (-4 ^{o}F) followed by 24 hours on saturated muslin cloth in a humidity room at temperatures ranging from 79 ^{o}F to 99 ^{o}F. The wide range in temperatures in the humidity room resulted from difficulties with the air-conditioner during the early part of the project. The samples were not brushed with a wire brush as prescribed in ASTM D560. Instead, any softened material was allowed to accumulate and washed away in the subsequent erosion test. As the mold and sample weigh about 280 pounds, sample preparation and handling is an arduous task. It was impossible to move the entire assembly into the humidity room for curing; therefore, it was necessary to disassemble the mold and remove the sample before the cement hydrated while the sample was still relatively fragile. This resulted in some cracked samples and some samples with edge damage prior to curing; this does not appear to have affected any test results.

Erosion testing of the test specimens was conducted to determine the relative erodibility of the various specimens at different cement contents. After initial curing for seven days and subsequent exposure to freeze-thaw, each specimen was "imbedded" into the channel of an 18-in. wide plywood flume (Plate 12). Water was supplied to the top of the flume via a pipe with 19 equally spaced orifices. The final configuration of the flume has 1/8-in. orifices with a weir downslope to even the flow. For comparison testing, the flow was set at 10 gpm at a two-to-one slope (horizontal to vertical). Each sample was weighed, eroded for a period of one hour, then weighed again to determine the amount of material eroded. It was not practical to trap and weigh the detritus because of the large volume of water involved and the very fine grained nature of the material. A nest of sieves was used on some tests, however, to determine the gradation of the aggregates formed by the soil-cement as it deteriorates during freeze-thaw.

Flow rates were stabilized with a pressure regulator, and were monitored with a rotometer. Cumulative flows were measured with a trident flow meter.

The tractive force on the surface of the specimen may be determined by

$$\tau_0 = \gamma ds$$

where τ_0 is tractive force, γ is the unit weight of water, d is the depth of flow, and s is the slope. Depth of flow varies with the roughness of the bed which, in turn, varies as the specimen erodes. However, with a relatively smooth specimen the depth of flow is on the order of 0.011 ft as measured with a micrometer point gage. With a slope of two-to-one, this yields a tractive force of 0.34 lb/ft^2. This is roughly equivalent to water flowing at a depth of nearly 5 ft down a loess channel with a slope of 0.0011.

7.2. Soil-Cement Test Results

Prior to erosion testing, it was possible to visually judge the reaction to freeze-thaw in a qualitative sense. Compacted samples with low cement content tend to exhibit surface scaling while those with higher cement content were not visibly affected. Also, those samples with 20% coarse sand displayed much less effect of freeze-thaw than those composed of loess only. Specimens that deteriorated visibly under freeze-thaw conditions also eroded easily after freeze-thaw action was completed.

As a basis of comparison, a sample of pure loess compacted to maximum density at optimum moisture content was subjected to the freeze-thaw and erosion tests. It was observed that this same loess disintegrated to loose silt when submerged and thus would erode completely if submerged prior to testing. In the test which involved simply eroding the zero cement content loess, 35,693 gm/m^2 of soil was lost in erosion. The initial soil weight in each test was about 21 lb or 9.55 kg. Recognizing that the area of the eroded specimen is 0.1 m^2, that means over one-third of the pure loess was eroded. The erosion quantities are calculated as weight per unit area in order to express the results in a manner somewhat consistent with traditional means of expressing soil loss by erosion. In this test and subsequent tests it was observed that essentially all of the soil loss took place within the first hour of erosion as indicated by several long duration tests. Once a stable configuration is achieved, further erosion is negligible until the specimen is subjected to additional freeze-thaw action. This proved to true even when flow was increased from 10 gpm to 200 gpm on the test flume.

Table 11 shows the results of the erosion tests on the loess and loess-sand mixture at cement content between 0 and 9%. An addition of 3% cement to loess reduces the soil loss to less than 50% and 7% cement reduces erosion to about 4% of that experienced by pure loess. The effect of adding sand is even more dramatic. At 3% cement content with 20% sand, the loss is slightly more than 1% of that lost by the pure loess. At 7% cement in the loess-sand mixture there is no measurable soil loss. Cement increases the erosion resistance of loess and a loess-sand-cement mixture is even more durable.

Because of the practical implications of adding sand to the loess, a simple economic analysis compares the cost of loess with 7% cement with the loess-sand mixture at 3% cement content. Cement cost was assumed to be $65/ton and sand $10/ton*. The loess-sand mixture cost is $\$18.19/yd^3$, whereas the stabilized loess is $\$18.50/yd^3$. Recognizing that the 3% cement content loess-sand mixture performed better than the 7% cement content loess by a factor of nearly 3, it is concluded that the addition of 20% sand to the loess shows promise of providing an effective material for grade stabilization structures. The calculations for the economic analysis are given in Table 12.

7.3. Special Tests Using Plastic Soil-Cement

As mentioned previously, problems with grade stabilization structures often are the result of poor seepage control. Recognizing that cutoff walls of plastic soil-cement might be economical and easily constructed

*Personal communication, Brian Hunter, Pottawattamie County Engineer's Office, Council Bluffs, Iowa.

Table 11. Results of Erosion Tests on Cement-Stabilized Loess after 10 Freeze-Thaw Cycles.

Compacted Specimens

Percent Cement	Erosion Loss (gm/m^2) Loess-Sand	Erosion Loss (gm/m^2) Loess
0	---	35,693
3	388	16,006
5	183	4,241
7	0	1,281
9	0	--

Plastic Mix Specimens

Percent Cement	Erosion Loss (gm/m^2)	Moisture Content	Remarks
5	27,437	42.9%	Tested at 4 F-T Cycles
5	14,843	32.9%	---
5	11,571	28.9%	20% Sand

Table 12. Sample Cost Analysis--Per Cubic Yard of Soil-Cement.

Cement $\cong$ \$65/ton

Gravelly Sand $\cong$ \$10/ton

Specimen with 3% Cement and 20% Sand

<u>With Sand</u>: Assume 110 pcf compacted-in-place dry density or:

2970 lb/yd^3	processing cost* $\cong$		\$12.50/yd^3
At 3% cement	86.5 lb	\$2.81	
At 20% sand	577 lb	\$2.88	
	Materials	\$5.69	
	Total		\$18.19/yd^3

<u>With No Sand</u>: Assume 104.5 pcf dry density or:

2821.5 lb/yd^3	processing cost* $\cong$		\$12.50/yd^3
At 7% cement	184.6 lb	\$6.00	
	Materials	\$6.00	
	Total		\$18.50/yd^3

*Processing cost estimated from 1979 data provided by The Portland Cement Association for projects less than 5,000 yd^3.

seepage barriers, an additional set of tests was performed. Several samples were molded with sufficient excess water to create a plastic mix. This mix was then troweled into wood forms to form specimens the same size as the compacted specimens. These specimens have a very high moisture content and low density, so it was expected they would be susceptible to freeze-thaw action. Thus, it is desirable to use the least possible amount of water while maintaining workability. A minimum of 30% to 33% moisture content produced a moderately stiff mix.

Plastic soil-cement specimens did not perform well, as shown in Table 11. The visible effects of freeze-thaw action are much more noticeable in the plastic soil-cement specimens than in those compacted at optimum moisture and density. The plastic specimens expand as much as 10% and generally display signs of distress, such as surface softening and internal cracking or splitting. Although this result was anticipated, the disparity between platic and compacted soil-cement was greater than expected. This indicates that plastic soil-cement should only be used where freeze-thaw action is minimal.

The ease of mixing and placing the plastic soil-cement, however, makes it useful for placing in cutoff trenches below frost zone or for temporary structures at construction sites. It could possibly be placed in the interior of erosion control structures with compacted soil-cement exterior. Plastic soil-cement also might be used for remedial action where piping or erosion has occurred leaving voids beneath or behind structures such as wingwalls where compaction of backfill is difficult or impossible. The upper or outer protions of the voids produced by piping should be backfilled with soil-cement containing sand and compacted at optimum moisture content to provide frost protection to the plastic soil-cement.

Finally, in order to evaluate the effectiveness of plastic soil-cement as cutoff walls, permeability tests were conducted. Two specimens of plastic soil-cement were formed in 3in. Shelby tubes and subjected to falling head permeability tests after curing seven days. The specimens were formed of excess material from erosion test specimens and designated 29T and 30T. Data for these specimens are shown in Table 13 where the permeability values are about 10^7 cm/sec. These values compare with 1×10^5 for loess dynamically compacted to similar densities without additives (Badger, 1972). Although this appears to indicate a substantial reduction in permeability, Badger (1972) managed to vary the permeability from 4.2×10^5 to 7.8×10^7 by varying the molding moisture content of samples with a nominal dry unit weight of 85 pcf. Thus, the advantages of plastic soil-cement over compacted loess in cutoff trenches are ease of placement and resistance to piping, but not necessarily reduction in permeability.

The permeability test specimens were apparently completely saturated during the tests. The initial saturation of the specimen with no sand was 94% with an increase to 99.9%. The initial saturation of the sample with 20% sand was 95.3% with a calculated increase to 101.6%. These figures consider the change in volume of the cement gel upon

Table 13. Permeability and Related Properties of Plastic Soil-Cement.

Specimen Number	29T	30T
Dimensions	6.7 in. × 2.85 in.	8.25 in. × 2.85 in.
Dry Unit Weight	86.1 pcf	92.7 pcf
Percent Sand	None	20%
Initial Moisture Content	32.9%	28.9%
Permeability, cm/sec	3.8×10^{-7}	4.7×10^{-7}
Moisture Content after Testing	33.2%	29.2%

hydration. Neither sample appeared to be saturated upon extrusion and crushing, however, and it should be noted that neither sample shrank during testing, as both were very difficult to extrude from the Shelby Tubes. Permeability tests have not been run on samples following freeze-thaw action.

8. FLUME BRIDGE: A GRADE STABILIZATION STRUCTURE

8.1. Background and History

As pointed out earlier in this report, one type of grade stabilization used in western Iowa is the flume bridge. It has been used to span deep and wide gullies with the added advantage of causing aggradation upstream of the structure. An example of a flume bridge is shown in Plates 7 and 14. The flume bridge consists of an earthen embankment including the approach grades, thus spanning the valley width, with a relatively short bridge span over a hydraulic concrete flume designed for full flood flow. Although the floors of the modern flumes are concrete, the side walls are timber planking supported by piles. About 250 flume bridges are found in five counties throughout western Iowa. They are used most often in Monona and Fremont Counties. Fremont County alone has about 150 flume bridges (small and large).

The design of flume bridges in western Iowa seems to have envolved as a local concept, slightly modified through time with changes of need, construction methods, and materials. No written history of this structure was available to the authors; the brief historical data are those related to Robert Goehring, an Iowa State University graduate student, by Carl Coffman, a retired Fremont County bridge foreman. The first flume bridge was built in 1935 or 1936 based on a design by a young engineer from Iowa State College. The plans were obtained by Bill McCosh, the county engineer, to replace a 160-ft long three span bridge over an unnamed stream flowing into the Missouri River floodplain. The structure consists of a twin 8 × 10 ft cast-in-place concrete box placed beneath the roadway and a flume, a rectangular box with wood flooring and sidewalls (Plate 13). The floor is double planked with no overlapping joints and rests on either 4 in. × 6 in. or 6 in. × 6 in. posts on grade. The sidewalls are double tongue-in-groove planking with steel straps holding the planking to a frame of 4 in. × 6 in. posts on the exterior of the flume box. Although the top of the box is open, a few posts span the top for lateral wall support. At the bottom of the flume is a cast-in-place concrete stilling basin with an energy dissipator. Originally, the dissipator consisted of three 9-in. high walls on each side of the basin forming a wedge pattern pointing downstream. A slot extended down the center to accommodate low flows. This dissipator apparently functioned very well during the first flood until a log came down the flume and broke the tops off two of the walls. As a result of that incident, energy dissipators were eliminated from later flume bridges. To take advantage of low flow conditions, construction took place during late fall and early winter; so the hand mixed concrete and forms were heated with steam pipes and the green concrete insulated with straw after it was in place. The flume, box, and stilling basin all appeared to be in good condition in 1979 except for the damage to the energy dissipators. Four additional wooden box flumes of this original type were built in the next few years.

A short time later, Ralph Greenwood, a strong advocate of this type of grade stabilization structure, became county engineer. Because of his enthusiasm, these structures in western Iowa are still often referred to as "Greenwood Flumes." Greenwood and Coffman continued building these structures modifying their design for economy. A major change in

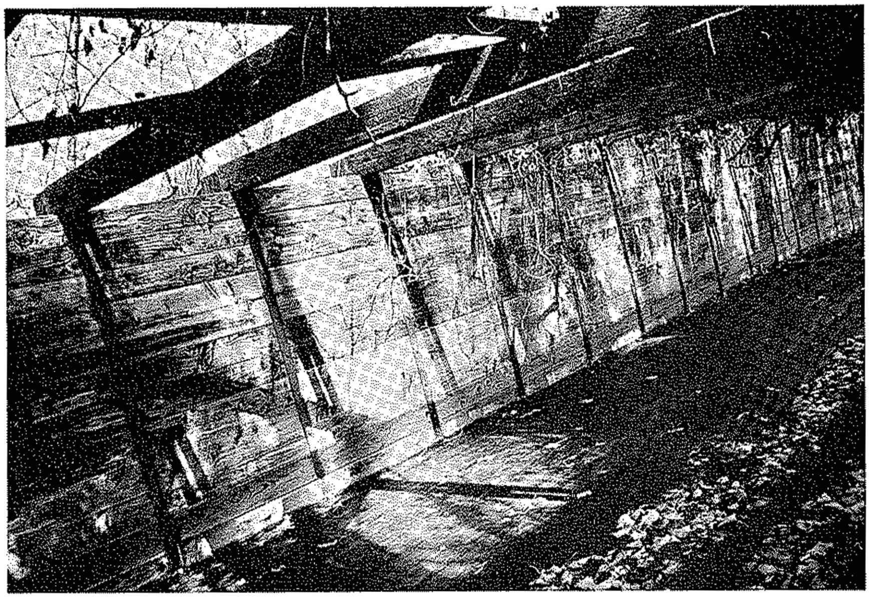


Plate 13. Flume of the original flume bridge in Fremont County. Note wood sidewalls and floor, cross braces on top, and absence of piles in the interior of the flume.

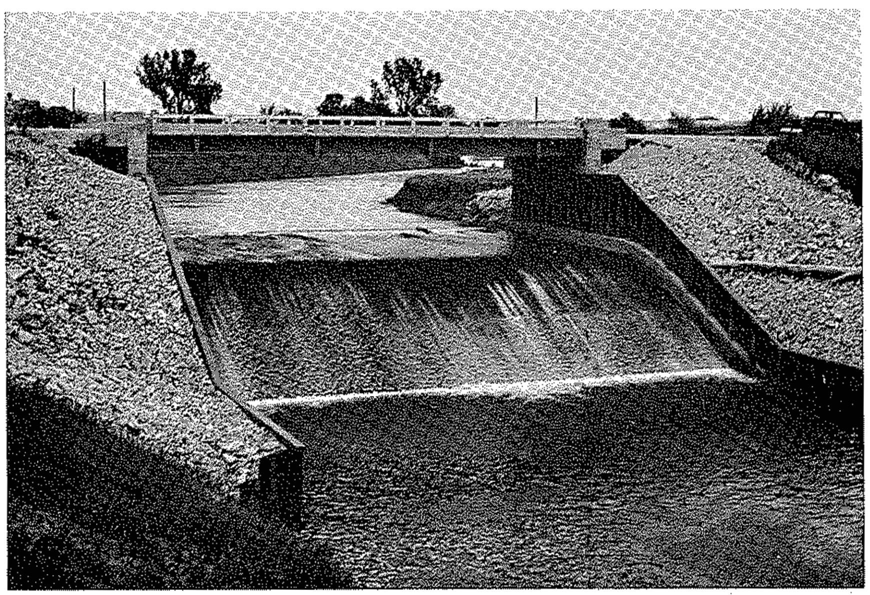


Plate 14. Recently built flume bridge in Harrison County. Note the use of steel piles in the sidewalls and bridge alignment relative to stream channel.

the construction of the flume box came about by driving piles on the inside of the box and placing the planking on the outside. The planking was nailed to the piles and held by soil pressure. The wood planking of the floor was replaced by concrete. This design was cheaper and easier to build. According to Coffman, this new design could be constructed in eleven days at a cost of about $3500.

Goehring found the original five wooden box structures and reported that three are in good condition; two have rotted floors and sagging walls. There is no record of any provision for seepage control in these early structures.

Most structures built at the present time have sand drains and cutoff walls for seepage control. The only major recent modification in the structure itself is the replacement of wood piles by steel H-piles to support the side walls (Plate 14).

8.2. Influence of Flume Bridges in Causing Aggradation Upstream

One of the benefits of a flume bridge as perceived by the engineers who advocate it is that, in addition to stopping degradation in the immediate vicinity of the stream where the structure is located, it will induce alluviation upstream (Plate 15). The effect of a main channel check dam on the alluviation upstream is somewhat unclear from published reports. One view is that there may be aggradation continuing upstream above the crest elevation for several miles depending upon stream gradient and the gradation of the sediment (Gottschalk, 1964). According to

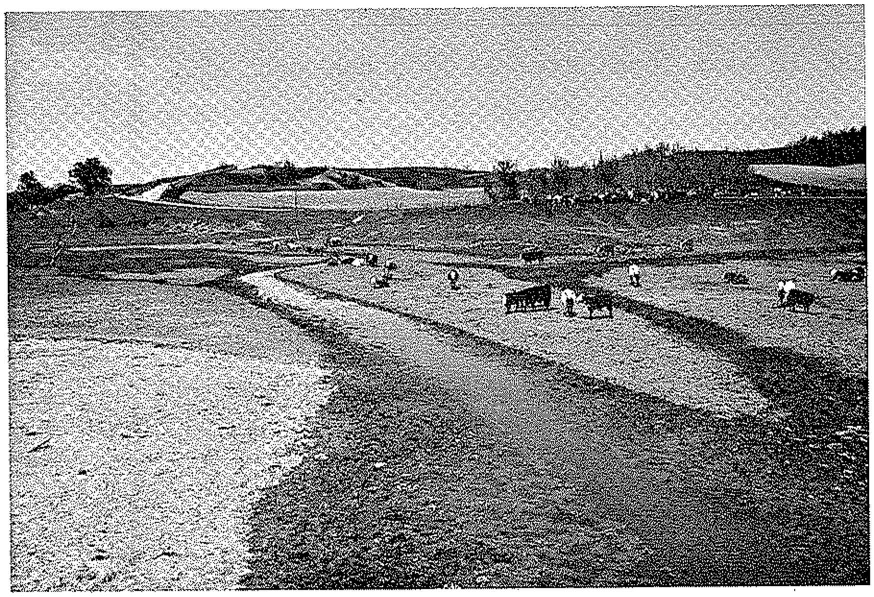


Plate 15a. The aggraded prism of sediment deposited upstream of a flume bridge in Monona County. Compare this photo with the downstream view in 15b.

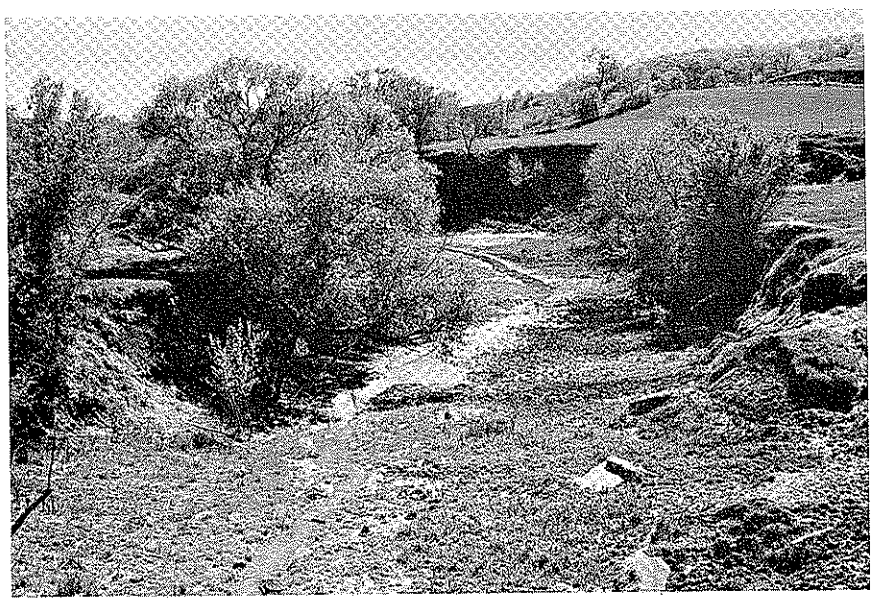


Plate 15b. A degraded channel downstream from a flume bridge. Compare this photo with the upstream view in 15a.

this view, low-gradient streams transporting heavy sand loads may cause aggradation many miles upstream of the reservoir. The opposing view is that the rise in base level created by a dam will affect only limited reaches upstream and the gradient of the deposited sediment wedge is appreciably less than that of the original channel (Leopold et al., 1964). These same authors recognize that in cases where the evidence points to this conclusion there may have been insufficient time to observe the "ultimate" aggradation. It is their opinion, however, that flattening of slope is compensated for by other changes in channel geometry which maintain the continuity of sediment and water transport. Perhaps the most honest appraisal of the problem has been given by Vanoni (1975, p. 18):

> "Despite the substantial volume of study devoted to sedimentation mechanics, it is still not possible to predict the sediment discharge of an alluvial stream with a degree of certainty for most engineering purposes."

Vanoni continues by stating that amounts of aggradation and degradation associated with dam construction are not predictable. Thus it can be seen that a state-of-the-art review cannot give a good evaluation of the effectiveness of flume bridges in creating aggradation and upstream stabilization of degrading channels. In a previous section of this report, reference is made to the influence of flume bridges in causing upstream aggradation and those data are expanded and discussed in more detail here.

Figure 21 shows a larger scale drawing of the effect of alluviation of the 34-ft high flume bridge at about mile 23.5 on Willow Creek. For reference, the profile of the floodplain and the 1916 channel is shown.

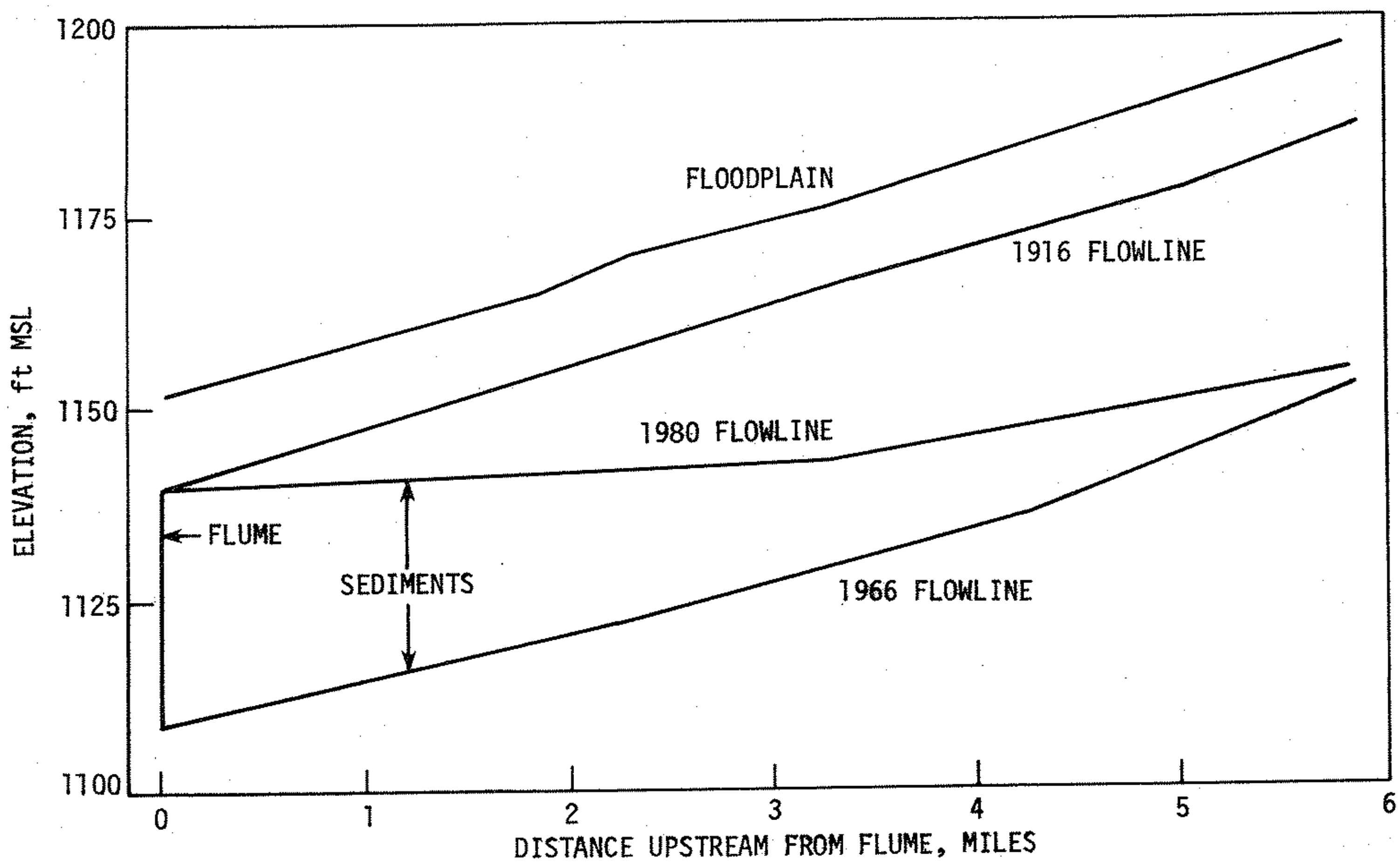


Fig. 21. Flume bridge at mile 23.5 on Willow Creek showing sediment deposition.

The area between the 1966 flowline and the 1980 flowline represents the prism of deposited sediment created by the flume bridge. By planimetering this area and multiplying by the average channel cross section, it is possible to calculate the volume of sediment entrapped by the flume. This structure has collected 850 acre-ft (1,371,333 yd^3) of sediment since construction in 1970. It is not known when this level of sediment was reached but the average rate of sediment accumulation is 85 acre-ft/year. The 1980 slope ranges from 0.00017 near the crest of the structure to 0.00087 near the upstream end of the sediment prism. The slopes of the 1966 channel ranged from 0.00112 to 0.00203. Thus the channel has been flattened considerably by the aggradation. The measurable sediment prism extends over five miles upstream and attains a maximum elevation over 10 ft above the flume crest elevation.

A second flume bridge at mile 16 was subjected to a similar analysis. This flume is 17 ft high and the sediment was deposited to an elevation about 2.5 ft above its crest and extended about 2.5 miles upstream from the dam. The total volume of sediment trapped is 183 acre-ft. The flume was constructed in 1972; thus the minimum rate of sedimentation is about 23 acre-ft/year. In comparing the two flumes it can be seen that the upstream dam has entrapped 27 acre-ft of sediment per foot of height, whereas the downstream structure has about 11 acre-ft of sediment per foot. The upstream channel distance subjected to aggradation is 0.2 miles per foot of dam height for the structure at mile 23.5 and 0.15 miles per foot of dam height for the flume at mile 16. The 1966 channel gradient at the location of the downstream flume was 0.0016, which is essentially the same as the original gradient at the upstream flume. So the lower efficiency of the downstream flume in causing aggradation upstream cannot be attributed to slope differences. Perhaps the clear water discharge of the upstream flume had some effect on this. This preliminary study suggests that a more thorough study of the history and extent of sedimentation behind many grade stabilization structures may lead to recommendations regarding optimum spacing and maximum heights of check dams to achieve maximum aggradation in these streams.

The hydraulic design of the upstream flume was based upon the 25-year flood with a discharge of 5800 cfs. In order to determine if the 50-year flood with a discharge of 7000 cfs would cause overbank flooding, the HEC-2 water surface program was utilized. It was calculated that the 50-year flood would be 5 ft below floodplain elevation at the crest of the dam, therefore the presence of the flume does not remove the flood transporting capacity of the straightened stream.

Although the flume bridges in most instances have been successful as grade stabilization structures, they are no panacea. Even some of the most modern structures have failed through piping of the soil in the embankment adjacent to the structure or through excessive uplift pressures beneath the concrete floor of the flume (Plate 16). Another common problem involves excessive erosion of the channel downstream from the stilling basin (Plate 17). Although these problems have been recognized, there has been no previous attempt to quantitatively analyze the structures and make recommendations regarding possible design

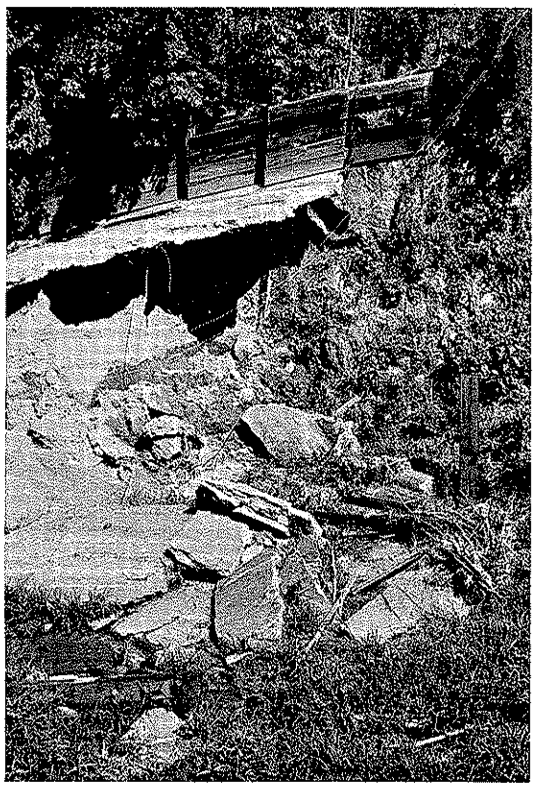


Plate 16. The remainder of the concrete floor and timber sidewalls of a flume which has failed, probably through piping.

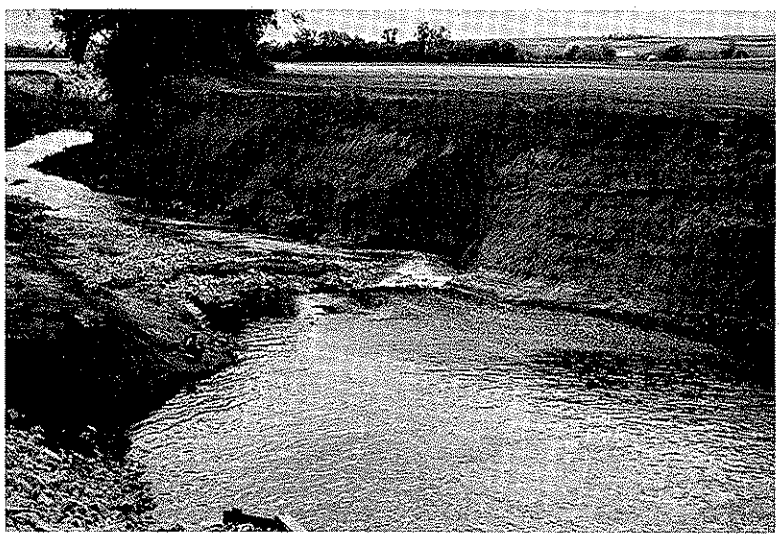


Plate 17. Scour hole at stilling basin downstream from flume bridge. Note how deflection of channel is causing lateral erosion.

improvements. Therefore, the following sections of this report will analyze both seepage and downstream energy dissipation associated with flume bridges. These sections of the report contain an overview of this aspect of the study, but more complete information can be found in the thesis by Goehring (in preparation).

8.3. Hydraulic Analysis of Stilling Basins

Field observations of flume bridges and other types of grade stabilization structures in western Iowa indicate that erosion at the tailwater or stilling basin area of the structures in a common occurrence (Plate 18). Therefore it seems appropriate to analyze the energy dissipation at the tailwater of some representative drop structures. Two flumes were analyzed: one 18.9 ft high, 77.8 ft wide with a stilling basin 21.3 ft long and a design discharge of 8100 cfs; and a second 18.2 ft high, 18.5 ft wide with a stilling basin 24 ft long and unknown design discharge. Flume No. 2 has no baffles, whereas Flume No. 1 does have them.

Three methods of analyzing the energy dissipation were employed: 1) Soil Conservation Service (SCS) or Saint Anthony Falls (SAF) stilling basin design, 2) Bureau of Reclamation method, and 3) a theoretical approach.

The SCS method was developed at the SAF Hydraulic Laboratories (Blaisdell, 1948) in the early 1940s for universal application to small drainage basins and is based on relatively small and infrequent flows. The design requires use of baffle blocks, chute blocks, and end sills

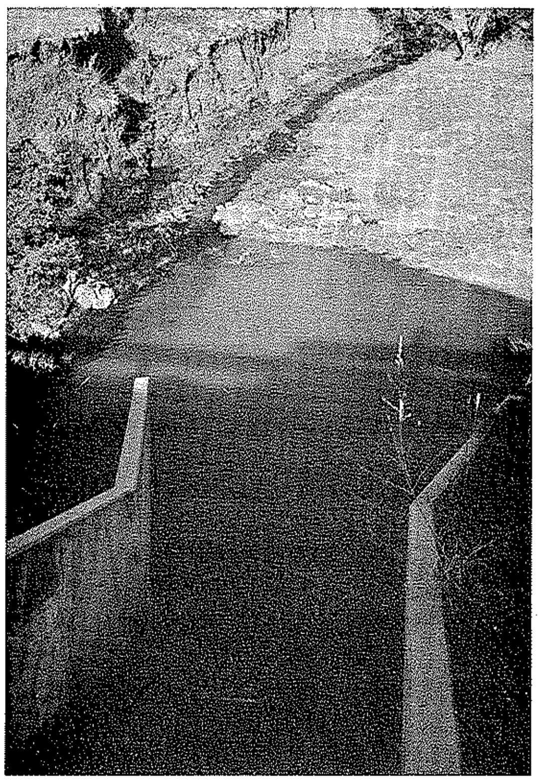


Plate 18. Undissipated energy on the downstream side of this box culvert having an IDOT flume outlet resulted in erosion of a scour hole and bank instability.

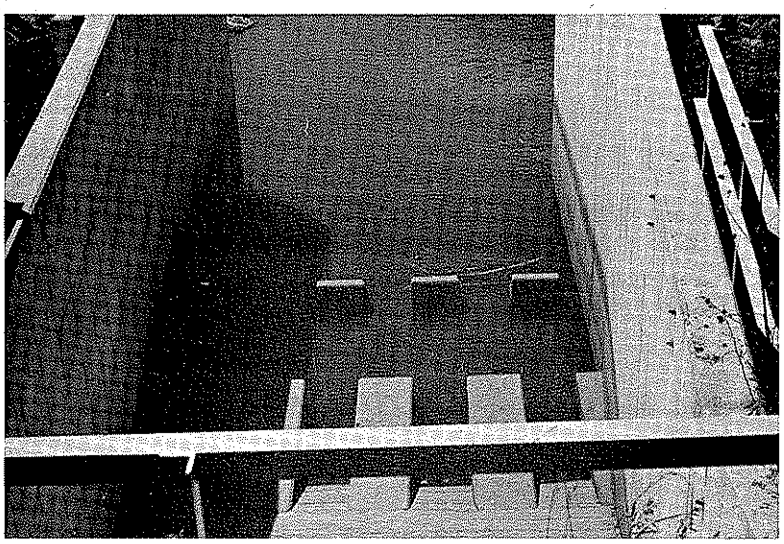


Plate 19. Chutes and baffle blocks for energy dissipation on downstream side of box-culvert type of grade stabilization structure.

(Plate 19). The method is easily programmed on a hand calculator. Often, the basin length computed by this method is increased by 50% to insure complete energy dissipation.

The design philosophy of the Bureau of Reclamation (BR) method (Peterka, 1978) is different from the SAF because the BR method anticipates larger and more constant flows. The BR method makes extensive use of design charts and, like the SAF method, is easy to apply.

The theoretical method employed is described in any standard hydraulics text; an especially good description is found in Eleavorski (1959). The length of stilling basin calculated using the theoretical approach is greater than the length calculated by either the BR or SAF methods. This is because the latter two methods both assume that riprap will be required downstream of the concrete basin whereas the theoretical method assumes the entire stilling basin will be paved. The details of each of these methods can be found in the references cited previously or in the thesis by Goehring (in preparation).

Table 14 shows the results of the length of stilling basin calculations for the two flumes using all three methods for various discharges. In the case of the smaller flume, No. 2, discharges were assumed with a maximum slightly above the inflow capacity of the structure. These calculations show that the average basin with baffles is of the order of 20 to 35 ft for moderate to high discharges; however, without baffles the stilling basins are of the order of 30 to 70 ft. Flume No. 1 requires a stilling basin about 29 ft long for 8100 cfs; however, the length was 21 ft or 72% of that required. The basin is safe in Flume

Table 14. Stilling Basin Lengths Calculated for Two Flume Bridges in Western Iowa.

		Stilling Basin Length			
Flume Number	Discharge (cfs)	SAF (ft)	BR (ft)	Theory (ft)	Actual Basin Length (ft)
	8100	35.0	28.7	47.8	
1	4500	21.4	23.6	39.3	21.3 (with baffles)
	3500	17.3	21.5	35.9	
	2000	36.5	28.4	72.5	
2	1250	24.6	24.5	62.6	24 (no baffles)
	200	5.2	11.7	29.9	

Note: Both SAF and BR methods assume baffles are present. SAF method includes additional 50% increase.

No. 1 for about 4500 cfs, which is 55% of the design flow. Flume No. 2 requires about a 72 ft long stilling basin for a flow of 2000 cfs. It has 32% of the length required by this analysis. The basin is safe for 10% of the design discharge it will be required to accommodate.

If the flumes analyzed here are representative of most flume bridges, it can be seen that the stilling basins are marginal at best with the stilling basins capable of handling only a fraction of the design flow. In future designs more attention to stilling basin length should result in much less erosion on the downstream side of grade stabilization structures. Also, the effectiveness of baffles in dissipating energy suggests that existing stilling basins which are too short without baffles and have severe erosion around the tailwater can be improved by the addition of baffles.

8.4. Seepage Analysis

Field observation of embankment instability induced by seepage through the embankment (Plate 20) suggests that some flume bridge embankments have been designed as highway embankments when in fact they should have been considered earth dams. It is also the opinion of many of the engineers in the area that failures in some cases have been caused by piping through the embankments or in other situations by uplift pressures and/or piping beneath the concrete. It is recognized that piping problems may result from poor construction rather than from poor design. The aforementioned application of plastic soil-cement may provide remedial measures when piping is observed but before a major

Plate 20. A foreshortened flume outlet; the irregular topography in the soil to the left of the flume is the result of landslides caused by seepage forces in the embankment.

failure takes place. Also, it can be speculated that the success of some of the very early wood floored flumes may be the result of release of porewater pressure beneath the floor through cracks.

The recognition of seepage problems in grade stabilization structures provides the incentive to quantitatively analyze the seepage in typical flume bridges. The approach was to draw flow nets beneath flume bridges for various hypothetical cutoff structures to evaluate their effect on quantity of seepage, piping potential, and uplift pressures. Methods of flow net analysis can be found in Cedergren (1977) and the specifics of the analysis of the western Iowa structures can be found in Goehring (in preparation).

Two flume bridges were selected for analysis of seepage. One of these, Flume No. 2, was used in the stilling basin analysis. A third flume is used here to compare and contrast the effect of an impermeable zone at different depths beneath the structures. Flume No. 3, designed for a 5527-cfs discharge is 30.9 ft high, 67.5 ft wide with a stilling basin 24 ft long. Boring logs from Flume No. 3 indicate glacial till 15 ft below the alluvium, whereas beneath Flume No. 2 the glacial till is estimated from the alluvial thickness map to be about 40 ft. The flow nets for Flume No. 2 and Flume No. 3 are shown in Figs. 22 and 23, respectively. These nets do not show the structures as built, but represent cases without seepage control. It can be seen that the exit gradients are higher beneath Flume No. 3 than beneath Flume No. 2. There are cutoff walls on the upstream side of both structures. Although

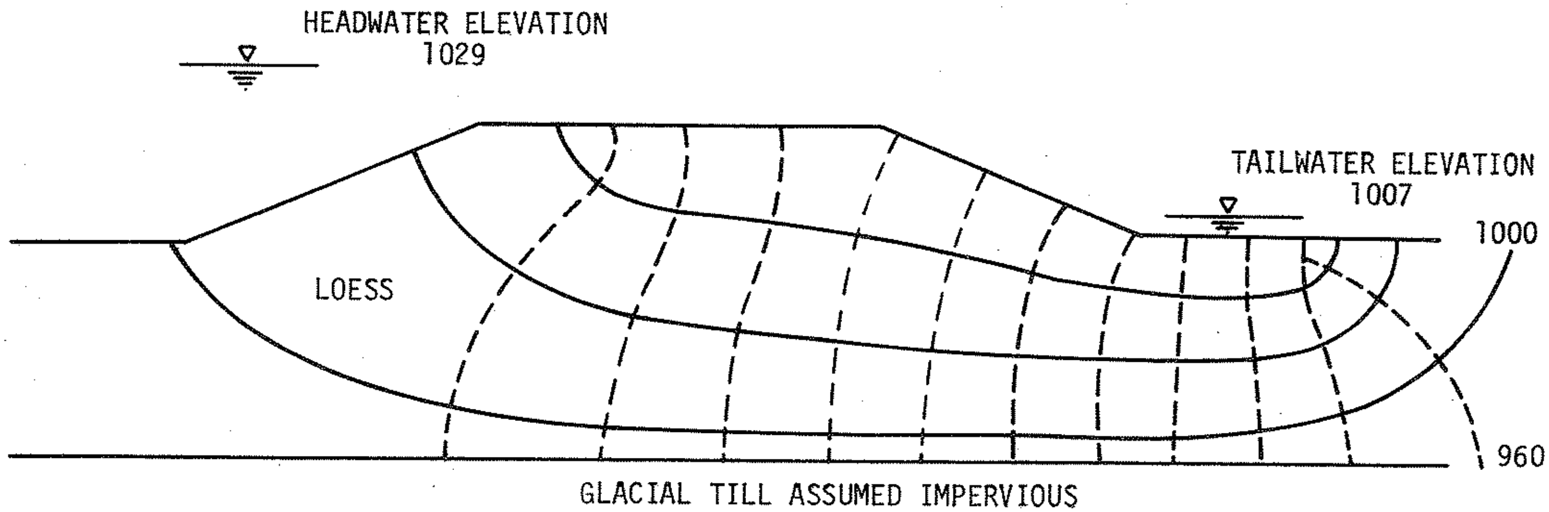


Fig. 22. Flow net beneath structure for Flume No. 2.

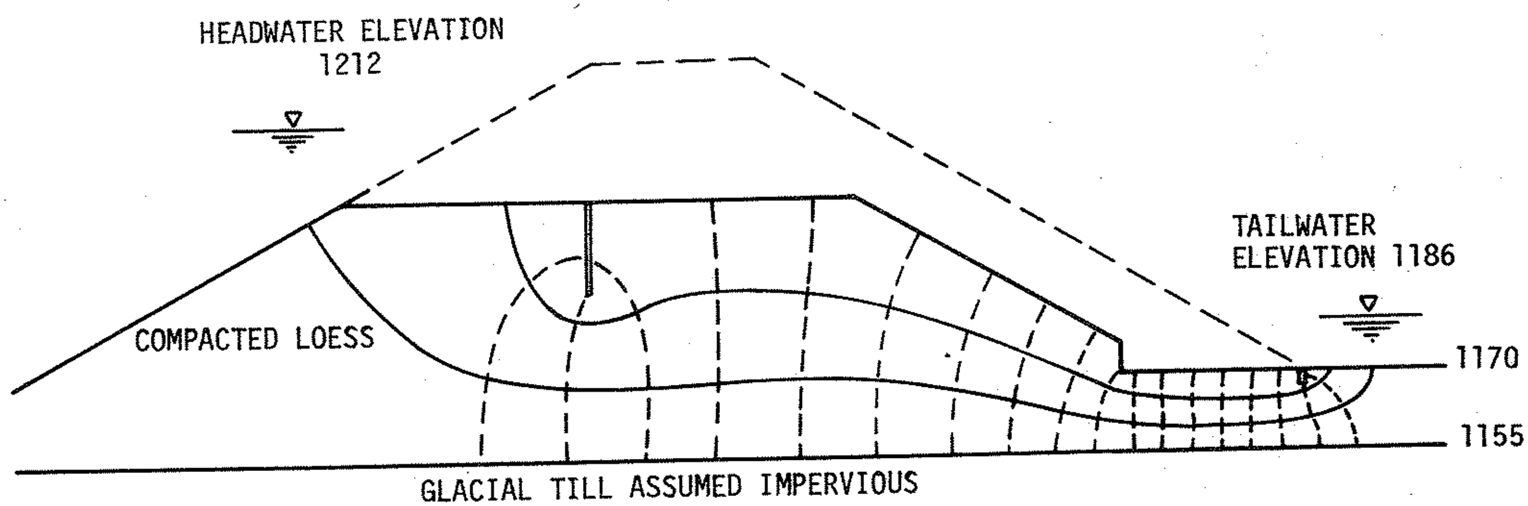


Fig. 23. Flow net beneath structure for Flume No. 3.

the 18-ft deep cutoff wall in Flume No. 3 is effective in increasing the length of flow lines, the 6-ft cutoff wall has no effect on decreasing the flow beneath the structure.

Figure 24 shows the flow net through the embankment adjacent to the flume for Flume No. 2. The occurrence of the phreatic surface high on the embankment with no cutoff or drains to control seepage indicates the possibility of slope instability and the need for seepage control within the embankment.

A series of flow nets for sections beneath the flume simulated various lengths of upstream cutoff walls and allowed an analysis of the influence of seepage control on uplift pressures and piping potential. Uplift pressures in Flume No. 3 show no significant reduction by increasing cutoff depth. A small reduction of uplift pressure occurs as the cutoff is increased beneath Flume No. 2. The uplift pressure analysis indicates that, without drains beneath the spillway of Flume No. 3, the situation would be critical; however, uplift pressures are tolerable beneath Flume No. 2. The implications of these uplift pressures are discussed in the section on the structural analysis of the slab.

The piping potential of the seepage beneath the flume and the influence of the cutoffs can be evaluated by calculating the exit gradients from the various flow nets and comparing these exit gradients with the critical gradient of loess. Using the in-place void ratio of 0.92 for undisturbed loess and a specific gravity of 2.7 (Badger, 1972), a critical gradient of 0.88 is obtained from the equation

$$i_c = \frac{G - 1}{1 + e}$$

where G is the specific gravity of the solids and e is the void ratio. Table 15 shows the values for exit gradients for the two flumes with various cutoff percentages. Flume No. 3 has the greatest danger of piping with no cutoff. It can be seen that in both structures the factor of safety is increased by an increase in cutoff length. In the case of Flume No. 2 the factor of safety approaches 1.5 with a cutoff of about 10%, whereas a cutoff depth of nearly 70% is needed to achieve a factor of safety of 1.5 in Flume No. 3. These analyses suggest that flow nets are necessary to understand seepage associated with these grade stabilization structures and to provide an efficient method for their design.

The above analysis was made on the basis of no seepage control under the flume slab. In some of the newer structures, drainage blankets and cutoff walls have been used to relieve the uplift pressure and to further control seepage.

The design of flume bridges in the future should include these provisions since the flume actually is a chute spillway. Criteria for these are available from several engineering sources in the literature (Bureau of Reclamation, 1970).

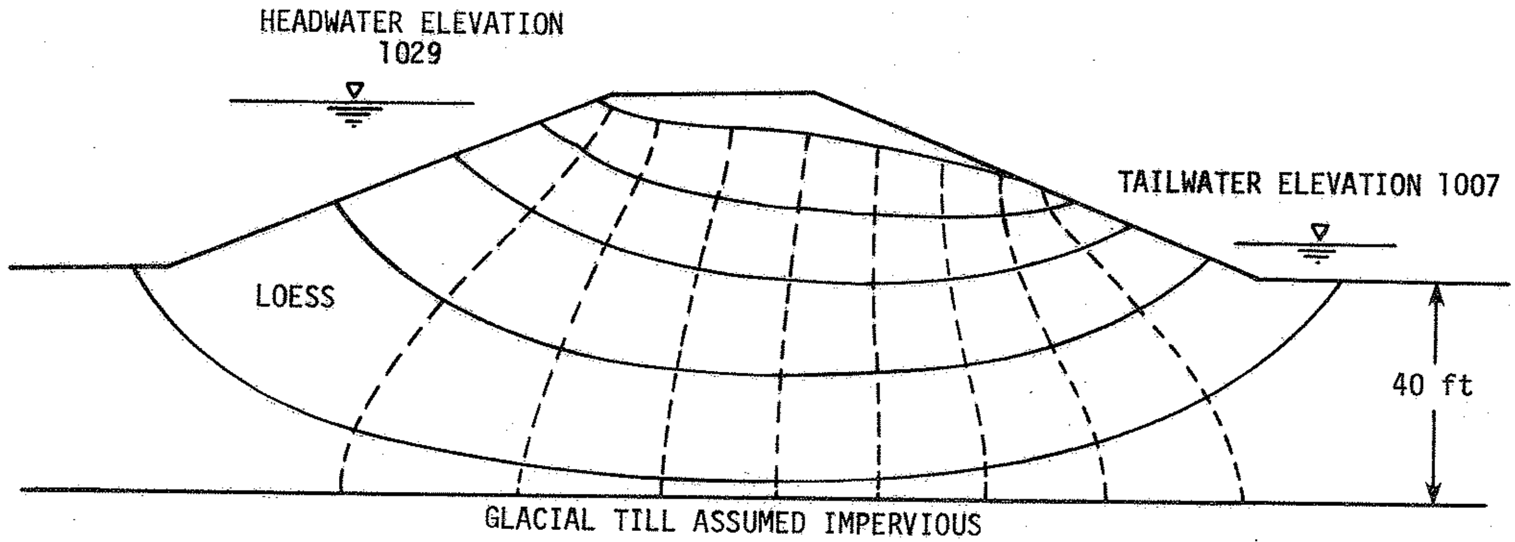


Fig. 24. Flow net through embankment adjacent to structure for Flume No. 2.

Table 15. Exit Gradients and Factors of Safety Against Piping for Various Lengths of Cutoff in Two Flume Bridges.

Flume Number	Percent Cutoff	Exit Gradient	Factor of Safety
	0	0.67	1.31
2	10	0.61	1.44
	39	0.49	1.80
	70	0.40	2.20
	0	0.77	1.14
3	36	0.68	1.39
	70	0.58	1.51
	90	0.46	1.91

8.5. Structural Analysis of Stilling Basin

Because the stilling basin in a flume-type structure is subjected to the greatest forces, it is one of the more critical elements in a structural sense. Thus, as an example of what can be done, a stilling basin was idealized, analyzed and designed for Flume No. 2. The basin was modeled as a 9-in. thick rectangular plate, 21 ft wide and 24 ft long.

After examining several techniques for analyzing the plate, it was decided the most expedient method would be to use an existing finite element computer program. The plate was subjected to five different loading cases simulating various combinations of hydraulic jump and uplift forces; the cases considered, as well as the resulting magnitude of force and distribution, are presented in Fig. 25. For cases IV and V to exist, there must be adequate drainage and pressure relief. The loads were determined using the flow net of Fig. 22 and an assumed flow of 2000 cfs. Figure 26 illustrates plan views of the stilling basin with the two edge conditions assumed. The only difference in the two conditions is that in Condition 1, the edges parallel to the direction of flow are assumed pinned (free to rotate but not to deflect) while in Condition 2, they are assumed fixed (restrained against both deflection and rotation). Actual field conditions (effects of the wood piling in restraining the slab edges) probably fall between the two. Condition 1 produces the largest negative moment (tension in the top) in the slab.

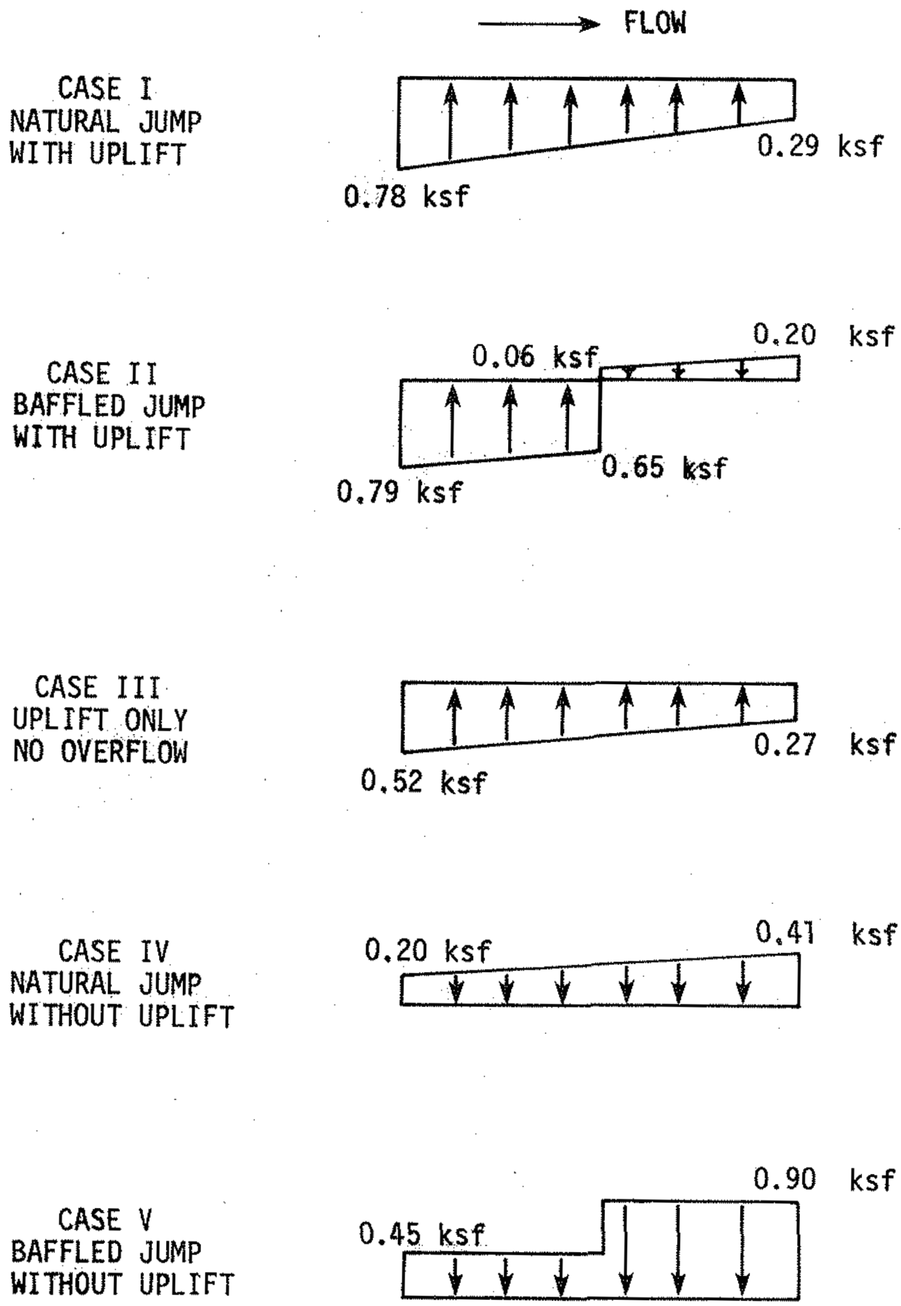


Fig. 25. Loading cases considered--Flume No. 2.

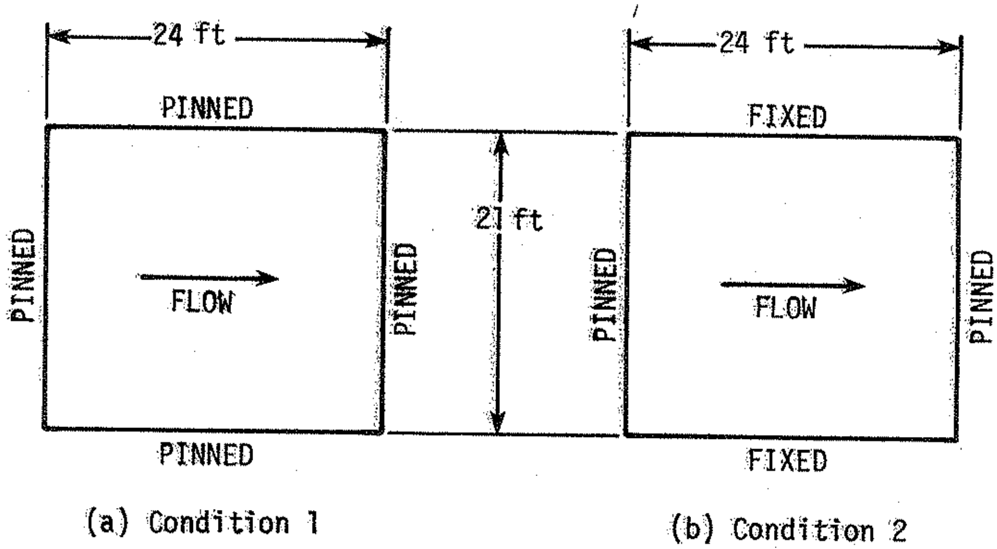


Fig. 26. Plan view of stilling basin with assumed edge conditions -- Flume No. 2.

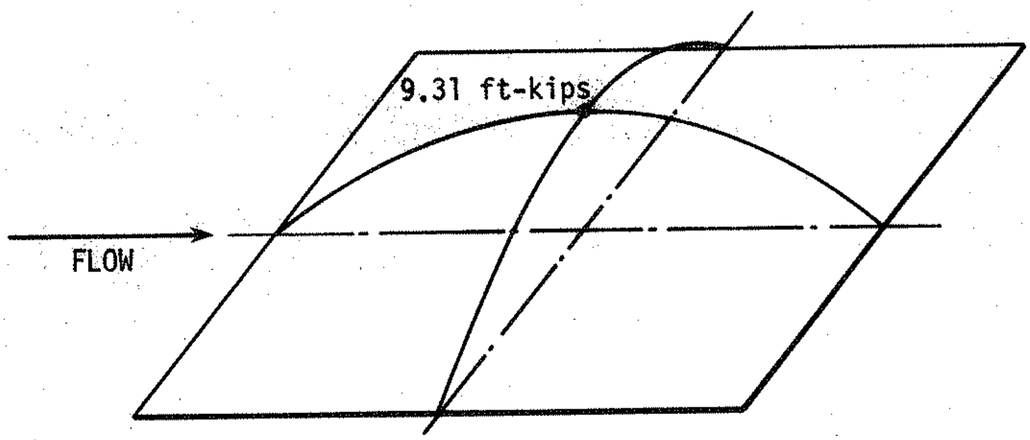
However, Condition 2 produces the largest positive moments in the slab (tension in the bottom). Thus both support conditions should be considered.

Table 16 presents the maximum centerline moments determined for each of the two edge conditions considered and for the first three loading conditions previously discussed. Loading cases IV and V, as may be seen in Fig. 25, subject the slab to downward loading, which is less than the bearing capacity of the loess. Thus the only reinforcement required for Cases IV and V would be temperature and shrinkage steel. Moments are given as M_L (moment about longitudinal axis of basin--axis parallel to direction of flow) and as M_T (moment about transverse axis of basin--axis normal to direction of flow). Also given in Table 16 are the amounts of reinforcement required to withstand the various moments. The area of steel calculations were based on the following assumptions: f'_c = 4000 psi, f_y = 60,000 psi, slab thickness = 9 in., top cover = 2 in., and bottom cover = 3 in. Although not apparent, as only maximum centerline moments are given in Table 16, some of the loadings and support conditions require the steel to be placed in the top of the slab while others require the steel to be placed in the bottom.

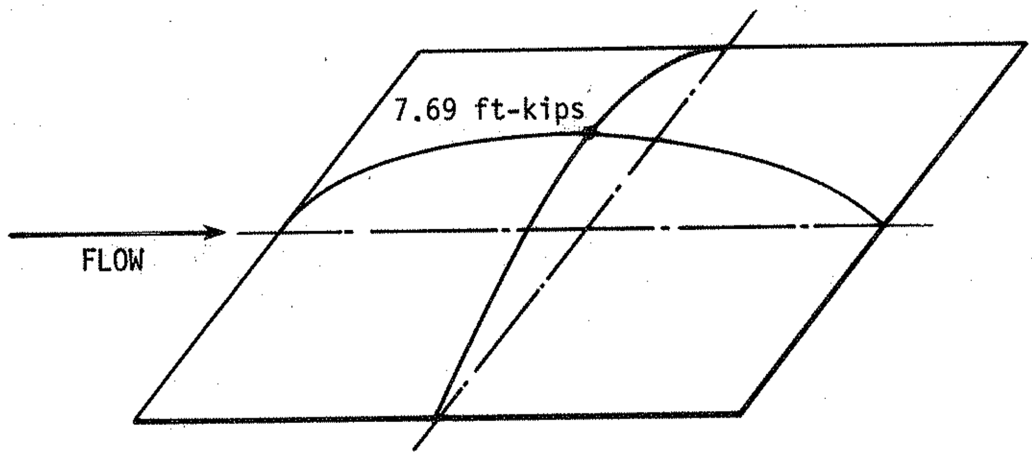
As previously noted, Table 16 presents the maximum moments in the slab resulting from the various loading and support conditions. Obviously one does not have to reinforce the slabs for these moments at all locations. As an example, variations in the moments along the basin centerlines resulting from Case III, support condition 1, are shown in Fig. 27. As may be seen, the moments decrease from the maximum value

Table 16. Maximum Centerline Moments and Corresponding Amount of Reinforcement Required.

	Condition 1				Condition 2			
Case	M_L (ft-kip/ft)	As_L (in.^{2}/ft)	M_T (ft-kip/ft)	As_T (in.^{2}/ft)	M_L (ft-kip/ft)	As_L (in.^{2}/ft)	M_T (ft-kip/ft)	As_T (in.^{2}/ft)
I Natural Jump with Uplift	12.67	0.67	10.69	0.56	8.24	0.42	4.59	0.23
II Baffled Jump with Uplift	7.95	0.42	10.19	0.55	6.00	0.30	6.37	0.33
III Uplift Only	9.31	0.45	7.69	0.37	6.03	0.29	3.73	0.19



(a) Longitudinal Moments , M_L



(b) Transverse Moments , M_T

Fig. 27. Variation along centerlines of M_L and M_T--Case III loading, condition 1 support.

given in Table 16 to zero at the edges; therefore the amount of steel required varies from that given in Table 16 to the minimum required for temperature and shrinkage steel.

In summary, the importance of determining the various loading conditions to which elements of a flume-type structure may be subjected during its life cannot be overemphasized. Proper modeling of the structure is essential in obtaining moments, forces, etc. for use in design. Uplift forces, estimated from flow net analysis, must be included not only to determine the proper amount of reinforcement required but also to determine proper positioning of it in the slab.

9. CONCLUSIONS

Since the latter part of the last century, the streams of western Iowa have degraded 1.5 to 5 times their original depth. This vertical degradation has been accompanied often by increases in channel widths, often from 2 to 4 times the original widths. The causes of the degradation are: 1) increased runoff resulting from the transition from native prairie to row crops in the mid to late 1800s, and 2) higher stream velocities due to channel straightening projects in the 1920s to 1950s. The degradation of the Missouri River to which these streams are graded does not appear to be a cause of their entrenchment, particularly downstream of the Little Sioux River.

Historical studies of Willow and Keg Creeks suggest that about 40 years after straightening, degradation in the lower reaches of the streams has stopped and the streams have achieved a stable, graded profile. The rate of downcutting decreases with time. Daniels has suggested that the equation

$$B = C - k \ln(L)$$

can be used to estimate the ultimate amount of entrenchment that will occur in a degrading reach of a stream, provided the longitudinal profile in a downstream stable reach is known. This conjecture is somewhat verified by the fact that the amount of downcutting he predicted after 1958 is greater than the actual degradation observed between 1947 and 1966.

The historical data suggest that grade stabilization structures on the lower Willow Creek may have questionable effectiveness in stopping future vertical degradation because the stream in that reach had been stable for 10 years prior to construction of the flume bridges. However, the flumes were effective in rapidly trapping sediment and did cause aggradation upstream to the elevation of the flume crest in one case and 10 ft above the crest in another case.

Because aggradation may be effective for short distances upstream, it may be more economical to build a series of very low channel check dams to protect individual bridge structures, and not attempt to stabilize many miles of channel with the construction of one high structure. The rate of vertical downcutting has decreased with time as the streams degrade to their equilibrium height above base level. Application of fluid mechanics shows that channel width and slope adjust to produce slower velocities as the streams entrench. This decreasing velocity also is approaching a limiting value asymtotically. A rational equation supported by experimental evidence describes the rate of downcutting

$$\ln(h/h_o) = -k't$$

This equation can be used to predict the time to reach equlibrium.

It is also observed that at a given section the stream channel has maintained a constant width through time. Thus, once the channel depth at equilibrium is predicted, the final width can be estimated by multiplying by width-to-depth ratio.

The majority of bridge problems caused by channel degradation in western Iowa are associated with the areas of thick loess accumulation. The alluvium in the valleys in this region is deep and relatively erodible so there are no resistant geological strata to retard downcutting. The problem structures seem to be associated more with the upstream reaches of the stream. As many as 25%, or a total of 750 bridges in the 13-county area, may be considered problem structures.

A previously published national survey indicates that main channel check dams (i.e., grade stabilization structures) and rigid bed armor are two of the most effective measures to control degradation. Most of these methods are expensive, so preliminary tests on cement-stabilized loess were conducted to evaluate the possibility of using loess soil-cement in grade stabilization structures. Although loess with about 7% cement can withstand erosion after 10 freeze-thaw cycles, a mixture of 20% sand and loess with 3% cement performed better. A cost analysis showed that the sand-loess-cement mixture was slightly more economical than the loess with the higher cement content. Plastic soil-cement (high water content soil-cement) using loess was also evaluated. These tests indicate that the plastic loess soil-cement is not durable to freeze-thaw action and thus easily eroded after freeze-thaw. However, the plastic soil-cement has permeabilities of 10^{-7} cm/sec, which is equivalent to compacted loess. The material shows promise of providing subsurface seepage protection when used as a slurry trench below frost line or as a remedial material injected into earthen embankments that are beginning to pipe.

A hydraulic analysis of stilling basins associated with flume bridges suggests that often the stilling basins are too short for effective energy dissipation. If this situation has caused erosion problems downstream of the stilling basin, the basin's energy dissipation effectiveness can be improved by the addition of baffle blocks.

Flow net analysis of seepage through earth embankments which are a part of flume bridges produces factors of safety with respect to piping as low as 1.2 in some cases; but the addition of cutoff walls and drains can improve the safety. It is recommended that seepage analyses become a routine part of the design of the soil embankments which are a part of grade stabilization structures.

Determination of the various loading conditions which the components of a flume-type structure may be subjected to during its life cannot be overemphasized. Uplift forces, estimated from flow net analysis, must be included not only to determine the proper amount of reinforcement required but also to determine the proper positioning in the slab.

Finally, the scope of this project has been broad and the time for study has been short. This report has suggested some very specific techniques for predicting channel degradation and some potential remedies. The authors are aware of the statement made by Mark Twain: ". . . something fascinating about science. One gets such wholesale returns of conjecture out of a trifling investment of fact." The reader must also be aware that the ideas suggested here need verification by expanded study.

The several theses being prepared will contain recommendations appropriate with the research findings to date. Detailed recommendations for the Phase II study will be included in the Phase II proposal. These will include:

1. Proposed preparation of guidelines for check structures, including necessary alternative construction features.
2. Further study of the hydraulics of knickpoints and control structures, consisting of a multiple-drop concept (several drops of smaller height), using the new hydraulic flume now available in the Water Resources Laboratory of the Civil Engineering Department.
3. Stability and structural study of two or three types of innovative channel stabilization structures.
4. With assistance of participating counties, prepare design plans for the above-mentioned structures which will be studied in a demonstration project.

10. ACKNOWLEDGMENTS

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The opinions, findings, and conclusions of this report are those of the authors and not necessarily those of the Highway Division of the Iowa Department of Transportation.

11. LITERATURE CITED

Antosch, L. and J. Joens, 1979, Iowa's lowhead dams: their past, present and future roles, Iowa State Water Resources Research Institute, No. 96, Special Report.

Badger, W. W., 1972, Structure of friable Iowa loess, Unpublished Ph.D. Dissertation, Iowa State University, Ames, Iowa.

Beer, C. E., 1962, Relationship of factors contributing to gulley development in loess soils of western Iowa, Unpublished Ph.D. Dissertation, Iowa State University, Ames, Iowa.

Blaisdell, F. W., 1948, Development and hydraulic design, Saint Anthony Falls stilling basin, Trans. Am. Soc. Civ. Eng. 113.

Brice, J. C., et.al., 1978, Countermeasures for hydraulic problems at bridges, Federal Highway Administration Report No. FHWA-RD-78-163, Vols. 1 and 2, National Technical Information Service, Springfield, Virginia.

Bureau of Reclamation, 1970, Design of Small Dams, U.S. Government Printing Office, Washington, D.C.

Cedergren, H. R., 1977, Seepage, Drainage, and Flow Nets, 2nd Ed., Wiley, New York.

Dahl, A. R., R. L. Handy and D. T. Davidson, 1958, Variation of loess thickness and clay content in southern Iowa, Iowa Academy of Science Proceedings 64:393-399.

Daniels, R. B., 1960, Entrenchment of the Willow drainage ditch, Harrison County, Iowa, Am. J. Sci. 258:161-176.

Daniels, R. B. and R. H. Jordan, 1966, Physiographic history and the soils, entrenched stream systems and gullies, Harrison County, Iowa, U. S. Department of Agriculture Technical Bulletin 1348.

Davidson, D. T. and R. L. Handy, 1952, Property variations in the Peorian (Wisconsin) loess of southwestern Iowa, Iowa Academy of Science Proceedings 59:248-265.

Dirks, W. K., in preparation, Geomorphic approach to predicting degradation of streams in western Iowa, Unpublished M.S. Thesis, Iowa State University, Ames, Iowa.

Eleavorski, E. A., 1959, Hydraulic Energy Dissipators, McGraw Hill, New York.

Goehring, R., in preparation, Preliminary structural and hydraulic analysis of flume bridges, in western Iowa, Unpublished M.S. Thesis, Iowa State University, Ames, Iowa.

Gottschalk, L. C., 1964, Reservoir Sedimentation in Handbook of Applied Hydrology, N. T. Chow (Ed.), McGraw Hill, New York, pp. 17-1 to 17-34.

Hack, J. T. 1957, Studies of longitudinal stream profiles in Virginia and Maryland, U. S. Geological Survey Professional Paper 294-B: 45-97.

Hallberg, G. R., 1979, Wind-aligned drainage in loess in Iowa, Iowa Academy of Science Proceedings 86:4-9.

Handy, R. L., D. T. Davidson and T. Y. Chu, 1954, Effect of petrographic variations of southwestern Iowa loess on stabilization with portland cement, Highway Reserve Board Bulletin 98:1-20.

Handy, R. L. and D. T. Davidson, 1957, Portland cement contents required to stabilize eastern and western Iowa loess, Iowa Academy of Science Proceedings 64:276-313.

Handy, R. L., 1972, Alluvial cutoff dating from subsequent growth of a meander, Geol. Soc. Am. Bull. 83:475-480.

Hansen, J. A., A. R. Dahl and D. T. Davidson, 1959, Further studies of loess in Iowa: thickness, clay content, and engineering classification, Iowa Academy of Science Proceedings 65:317-322.

Holland, W. N. and G. Pickup, 1976, Flume study of knickpoint development in stratified sediment, Geol. Soc. Am. Bull. 87:76-82.

Holtz, W. G. and K. D. Hansen, 1976, The use of compacted soil cement in water control structures, 12th Congress on Large Dams Q44 R13: 251-278.

Iowa Geological Survey, 1956, Surface water resources of Iowa, Oct. 1, 1950 to Sept. 30, 1955, Water-supply Bulletin No. 6.

Knochenmus, D., 1963, Alluvial morphology of the Nishnabotna River, Unpublished M.S. Thesis, Iowa State University, Ames, Iowa.

Lara, O. G., 1973, Floods in Iowa: technical manual for estimating their magnitude and frequency, Iowa Natural Resources Council Bulletin No. 11.

Lara, O. G., 1979, Annual and seasonal low-flow characteristics of Iowa streams, U. S. Geological Survey Open-File Report 79-555, Iowa City, Iowa.

Leopold, L. B., M. G. Wolman and J. P. Miller, 1964, Fluvial Processes in Geomorphology, W. H. Freeman and Co., San Francisco.

Lohnes, R. A., M. D. Dougal, M. Johnson and R. Bachmann, 1977, Water management, water quality and alluvial morphology of oxbow lakes, Final Report, Phase I, Engineering Research Institute Project 1279, Iowa State University, Ames, Iowa.

Lohnes, R. A. and R. C. Joshi, 1967, Quantitative variations in loess topography, Iowa Academy of Science Proceedings 74:160-167.

Lohnes, R. A. and R. L. Handy, 1968, Slope angles in friable loess, Geol. 76:247-258.

Lohnes, R. A., D. E. Wolff and M. D. Dougal, 1979, Cutoff dates of oxbow lakes from first order rate equation, Geol. Soc. Am. Prog. Abst. 11:468.

Massoudi, H., in preparation, A hydraulic model for predicting stream degradation as a result of straightening, Unpublished Ph.D. Dissertation, Iowa State University, Ames, Iowa.

Miller, G. A., J. D. Highland and G. R. Hallberg, Highway soil engineering data for major soils in Iowa, Technical Information Series No. 7, Iowa Geological Survey, Iowa City, Iowa.

Morrison, W. R. and L. R. Simmons, 1977, Chemical and vegetative stabilization of soils, Report No. REC-ERC-76-13, Engineering and Research Center, Bureau of Reclamation, Denver, Colorado.

Pedersen, D., 1962, Alluvial morphology of the Little Sioux River, Unpublished M.S. Thesis, Iowa State University, Ames, Iowa.

Peterka, A. J., 1978, Hydraulic design of stilling basins and energy dissipators, Bureau of Reclamation Engineering Monograph No. 25.

Piest, R. F., C. E. Beer and R. G. Spomer, 1976, Entrenchment of drainage systems in western Iowa and northwestern Missouri, Proceedings of 3rd Federal Inter-agency Sedimentation Conference 5-48 to 5-60.

Piest, R. F., L. S. Elliott and R. G. Spomer, 1977, Erosion of the Tarkio drainage system, 1845-1976, Trans. Am. Soc. Ag. Eng. 20:485-488.

Portland Cement Association, 1959, Soil-Cement Laboratory Handbook, Portland Cement Association, Chicago, Illinois.

Portland Cement Association, 1975, Soil-Cement Slope Protection for Embankments: Planning and Design, Portland Cement Association Chicago, Illinois.

Ruhe, R. V., 1969, Quaternary Landscapes in Iowa, Iowa State Univesity Press, Ames, Iowa.

Ruhe, R. V. and R. B. Daniels, 1965, Landscape erosion-geologic and historic, J. Soil Water Conserv. 20:52-57.

Simonson, R. W., F. F. Riecken and G. D. Smith, 1952, Understanding Iowa Soils, Wm. C. Brown Company, Dubuque, Iowa.

Sayre, W. W. and J. E. Kennedy (Eds.), Degradation and aggradation of the Missouri River, Proceedings of a Workshop in Omaha, Nebraska January 23-25, 1978, Iowa Institute of Hydraulic Research Report No. 215, University of Iowa, Iowa City, Iowa.

Travis, C. B., in preparation, Preliminary evaluation of cement for stabilizing loess in water resources applications, Unpublished M.S. Thesis, Iowa State University, Ames, Iowa.

Udden, J. A., 1901, Geology of Pottawattamie County, Iowa Geological Survey Report 11:199-279.

U. S. Geological Survey, 1978, Water resources investigations in Iowa, 1978, U. S. Geological Survey, Iowa City, Iowa.

Vanoni, V. A. (Ed.), 1975, Sedimentation Engineering ASCE Manual on Engineering Practice No. 54, Am. Soc. Civ. Eng., New York.

Wilder, C. R., 1977, Soil-cement for water resource structures, Trans. Am. Soc. Ag. Eng. 20:109-112.

Yang, Pe-shen, in preparation, Strength of loess stabilized with Portland Cement for possible water resources applications, CE-690 Report, Civil Engineering Department, Iowa State University, Ames, Iowa.