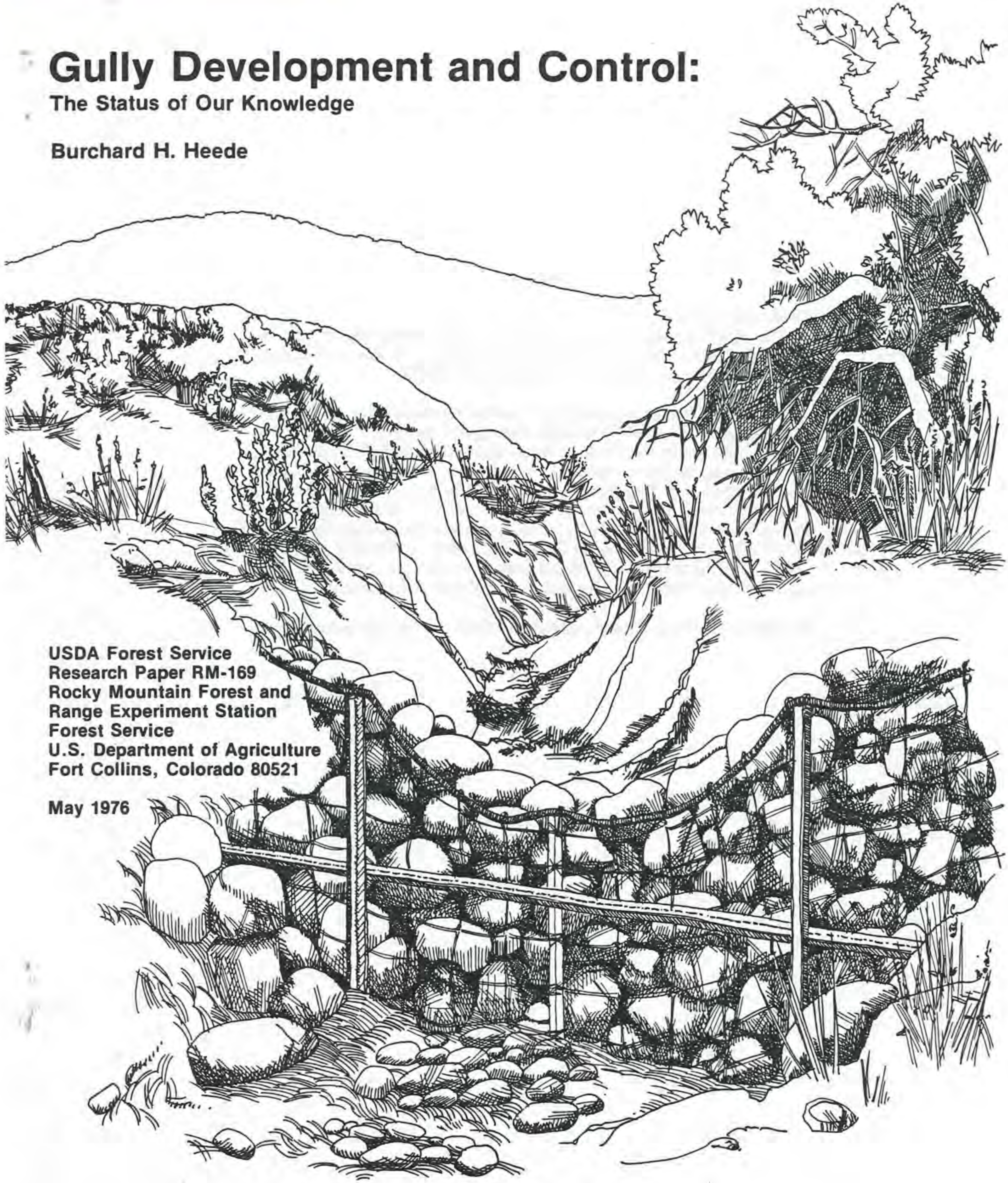
Gully Development and Control:

The Status of Our Knowledge

Burchard H. Heede



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Abstract

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Gully formation is discussed in terms of mechanics, processes, morphology, and growth models. Design of gully controls should draw on our understanding of these aspects. Establishment of an effective vegetation cover is the long-term objective. Structures are often required. The least expensive, simply built structures are loose-rock check dams, usually constructed with single- or double-wire fences. Prefabricated concrete dams are also effective. Functional relationships between dams, sediment catch, and costs, as well as a critical review of construction procedures, should aid the land manager in design and installation of gully treatments.

Keywords: Gullies, erosion, geomorphology, erosion control, dams.

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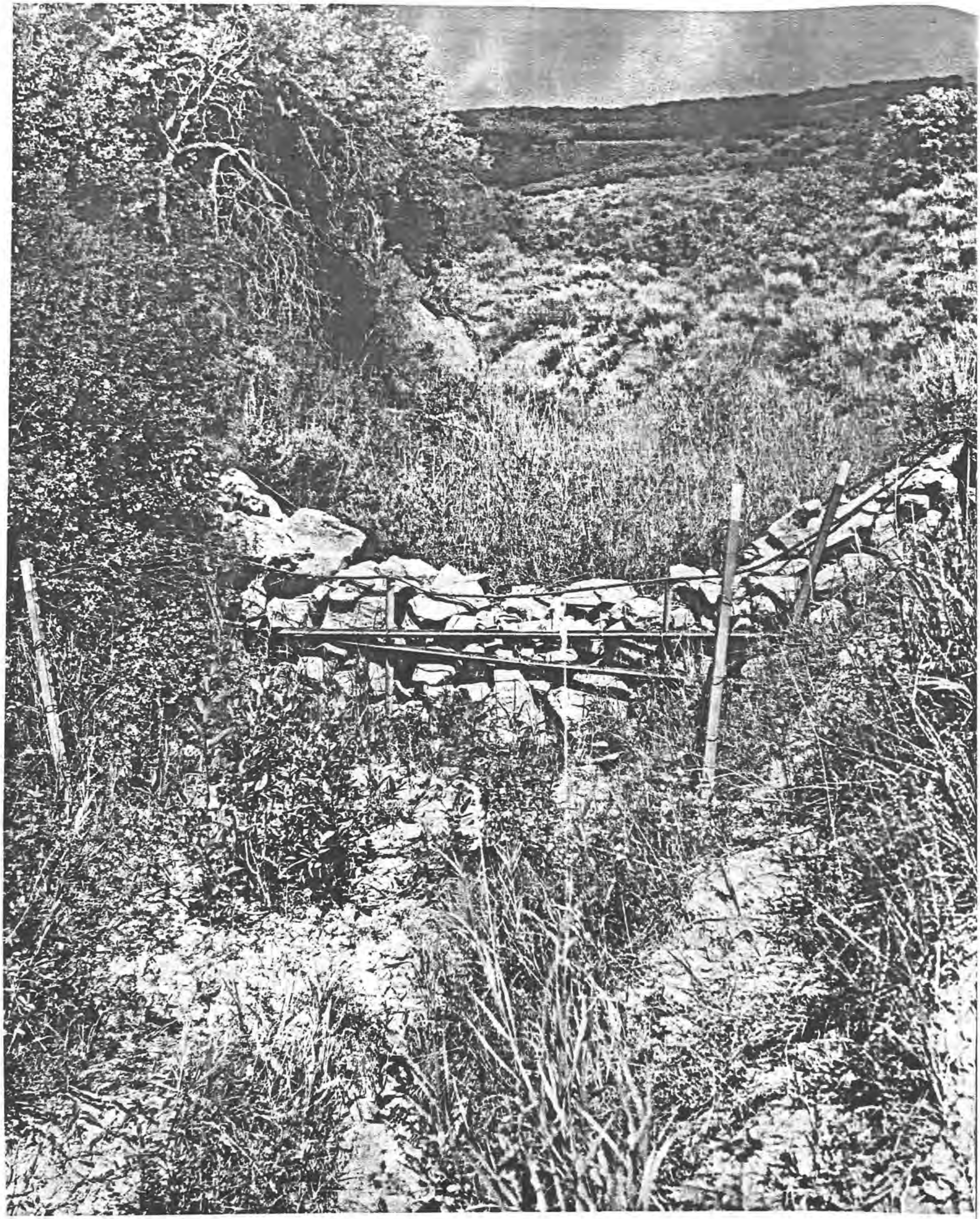

GULLY DEVELOPMENT AND CONTROL:
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HISTORICAL BACKGROUND

Early man was less mobile and more dependent on the surrounding land than his modern descendant. In many desert and semidesert regions, he not only learned to live with gullies, but utilized them for the collection of water and the production of food. Such desert agriculture was practiced in North Africa, Syria, Transjordan, southern Arabia, and North and South America. Thus, many areas in the world once supported more people than today. The Sierra Madre Occidental, Mexico, had a much higher population density 1,000 years ago than at present (Dennis and Griffin 1971), as did the Negev Desert in Israel (Evenari 1974).

Gully control on these ancient farms was not an end in itself, but a means for food production. Evenari et al. (1961) found well-defined "runoff farms" in the Negev Desert of Israel dating back to the Iron Age, 3,000 years ago. The climate, undoubtedly not different today, is characterized by an average yearly precipitation of 95 mm (3.7 inches), most of which falls in relatively small showers. Precipitation exceeds 10 mm (0.39 inch) on an average of only 2 days per year. Still, runoff farms, using check dams and water spreaders in wadis, gullies, and on hillsides, were able to support dense populations until the Negev was occupied by nomadic Bedouins after the Arab conquest in the 7th century A.D.

At least 900 years ago, the aborigines of the northern Sierra Madre Mountains of Chihuahua and Sonora, Mexico, developed an intensive field system by altering the natural environment with the help of trincheras (Herold 1965). Trincheras of the Sierran type—check dams built from loose rock—created field and garden plots within gullies and valleys by sediment accumulation, increased water storage within the deposits, and spread the flows on the deposits during storms. Similar but less developed systems were built sporadically in Arizona, New Mexico, and southwestern Colorado. About 1450 A.D. this flourishing agriculture disappeared.

With the age of industrialization, man lost his close dependence on the land. Population densities increased, land was fenced, and roads and communication systems mushroomed. This rapid change caused a different philosophy in the approach to gullies. Gullies were visualized as destroyers of lives and property, and as barriers to speedy communication. It is not surprising, therefore, that the first textbook on gullies or torrents, published in the 1860's in France, dealt with control only. Others followed quickly in Austria, Italy, Germany, and later in Japan.

It is not surprising that our knowledge on the mechanics of gullying is meager if we consider that, during the last 100 years, torrent and gully control were emphasized. Gully control research focused on engineering aspects—structural dimensions, types of structures, and adaptation of advances in civil engineering elsewhere. When in the middle 1950's interest was awakened in gully processes, efforts concentrated on mathematical and statistical, rather than physical, relationships.

The time has come to concentrate our efforts on understanding gully mechanics, and to reassess our philosophy on gully control. The objectives must be broadened beyond those of defense, and incorporate those of agricultural production, water yield, and environmental values. This task will not be easy, and in many cases tradeoffs will be required.

In areas of food shortage, the most pressing objective in gully control may be agricultural production. Food-short areas are often arid or semiarid, where gullies are the only streambeds supporting flow at times, and gully bottoms are closest to the low-lying water table. Gully flows as well as moisture storage both were utilized for plant growth by ancient man. Modern man may have to relearn the forgotten art of gully management in desert farming. This possibility is better for many developing countries; in highly industrialized countries, the present cost-price structure will seldom permit successful gully management for food production in deserts and mountain lands.

In the United States, however, gully management has been successfully practiced on agricultural lowlands at least since the 1930's, when conservation farming was introduced on a large scale. Farmers converted gullies into grassed waterways to serve the dual purpose of safe conveyance of surplus irrigation water and forage production. Often the Federal Government subsidized this work by extending technical and monetary help.

In contrast to agricultural lowlands, we know very little about gully management on mountain lands, where we have been mainly concerned with control. In the United States, a first approach to gully management on mountain slopes was Heede's (1968a) installation of vegetation-lined waterways in the Colorado Rocky Mountains. Converted areas lost 91 percent less soil than untreated gullies, and the unpalatable plant cover, consisting mainly of sagebrush, was changed to a palatable one, adding to the grazing resource.

In Italy, intensive hand labor, plowing, and manmade torrent streams reshaped gullied mountain slopes of the Apennines into gentle hillsides that could support pastures, vineyards, and orchards. The reshaping, called hydraulic reclamation (Heede 1965a), was justified by efforts to place Italian agriculture on a competitive basis when it would join the Common Market Community in the late 1960's.[2]

Modern check dam systems can also benefit water yield. Brown (1963) reported on the conversion of ephemeral flows to perennial streams below check dams. Heede[3] obtained perennial flow 7 years after installation of a check dam system where only ephemeral flow had occurred during the previous 50 years. It is postulated that this change is due to water storage in the sediment accumulations above the dams. Considerable vegetation develops within the gullies as well as on the watershed. Although this additional vegetation undoubtedly uses water, the evapotranspiration loss is more than offset by increased soil infiltration rates, resulting from vegetation cover improvement, which benefit soil water storage at times of high flows. The duration of significant flows increased, but total water yield did not.

[2] *Heede, Burchard H., 1962. A report on a visit of research stations, torrent control, and land reclamation projects in France, Italy and Austria. 73 p. (On file, Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)*

[3] *Heede, Burchard H. Evaluation of an early soil and water rehabilitation project—Alkali Creek watershed, Colorado. (Research Paper in preparation at Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)*

The environmental value of gullies is assessed differently by different people. To some, gullies may represent a typical landform of the Old West, a dear sentiment, adding to environmental quality. To them, gully control should be attempted only if needed to meet pressing land management objectives. To others, gullies may offer only an unsightly scene, and the conversion of raw gully walls into green stable slopes is a desirable goal. Our approaches to gully management must therefore remain flexible.

It is the objective of this paper to show progress and limits in our knowledge of gullies and their control, and thus to help the land manager achieve his goals.

SCOPE

This paper attempts to summarize the available body of knowledge and hypotheses on gully formation and control. As illustrated by the historical development of gully management, gully control currently comprises the larger body of knowledge. Of necessity, the discussion of gully control will be based mainly on works in the Colorado Rocky Mountains, where considerable effort has been invested since the work of the Civilian Conservation Corps in the 1930's.

Gully formation will be divided into three aspects: mechanics, processes and resulting morphology, and growth models. The individual aspects of gully formation must be considered not only by the control engineer, but also by the land manager who may decide not to interfere. If noninterference is the decision—and it will be in most cases—the consequences should be considered in the management plan to avoid future "surprises." Should gully management be planned for food or forage production, however, knowledge of these aspects of gullying will improve the design. Thus this report should be a helpful tool, whatever the land management decision may be.

GULLY FORMATION

Gullies develop in different vegetation types. In the West, gullies often develop in open ponderosa pine forests (fig. 1) or grasslands (fig. 2), the latter often heavily mixed with sagebrush (Heede 1970).

Gully development and processes have been studied by many investigators. A basic question raised was, why did gully cutting accelerate in the 1880's in the West, as documented for many locations? Schumm and Hadley (1957) argued that the sudden rapid development of gullies followed the sharp increase in cattle grazing around 1870. Leopold (1951) cited an additional influential factor—exceptionally frequent high-intensity storms at this time. Thus overgrazing may only have been the trigger. Yet Peterson (1950) stated that gully formation started in some locations before they were overgrazed, while other areas never experienced gully erosion after grazing. Other investigators stressed climatic change as the chief cause (Gregory 1917, Bryan 1925, Richardson 1945).

Neither the short-time climatological records, nor other approaches such as tree ring studies and pollen analysis, permit us to realistically assess the possible relationship between climatic change and gully cutting. I agree with Hastings (1959) that, recognizing the fragile condition of much western plant cover, any trigger effect could damage the cover to an extent where bare soil and runoff could increase drastically. Overgrazing and other land abuses such as poor road construction and location certainly were triggers. Once gully scarps formed, the development of gully networks was inevitable, because during the last half of the 19th century, the agricultural industry of the West was one of exploitation, not conservation.

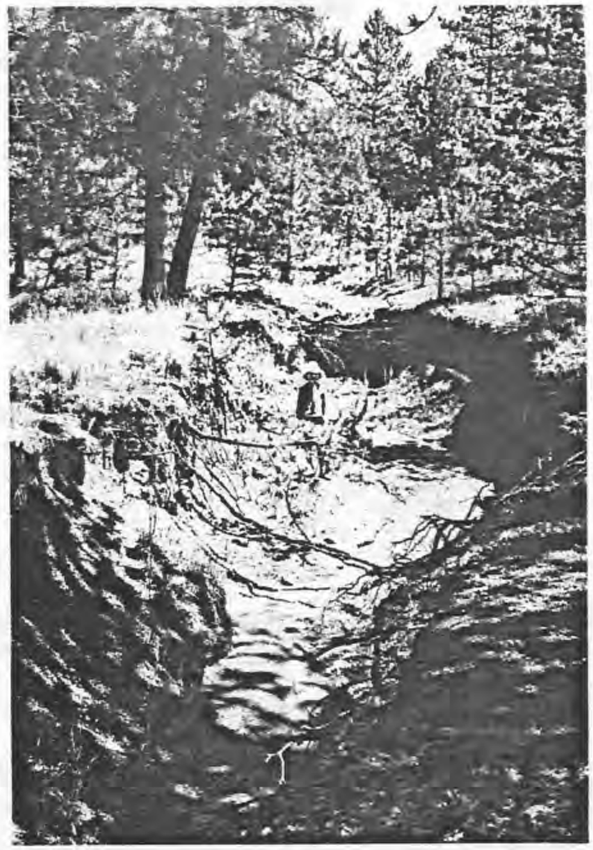


Figure 1.—This discontinuous gully advances through a ponderosa pine forest with an understory of grasses and other herbaceous vegetation. Location is the Manitou Experimental Forest on the eastern slope of the Rocky Mountains in the Colorado Front Range.

Figure 2.—This gully developed on a valley bottom covered by a fine stand of bunchgrasses on the Manitou Experimental Forest, Colorado Front Range. The view is across the reach close to the gully mouth.

Mechanics

Piest et al. (1973, 1975) deserve the credit for beginning gully mechanics investigations on agricultural croplands. Their studies showed that tractive force and stream power of the flow were not sufficient for a significant detachment of erodible loess soil overlying glacial till in the rolling countryside of western Iowa. Tractive force (τ) was defined as:

$$\tau = \gamma R_1 S_1 \qquad (1)$$

where γ is the specific weight of the fluid, R_1 is the hydraulic radius, and S_1 represents the slope of the energy gradient. The investigators determined the stream power per unit length of gully (ω) by

$$\omega = \tau P V \qquad (2)$$

where P is the wetted perimeter and V is the mean stream velocity. Since flow width (w) and wetted perimeter were approximately equal, w, the factor usually included in the equation, was substituted with P.

Calculations of unit stream power gave estimated values only, since the roughness coefficient (n) had to be estimated in the Manning's equation. Stage-discharge records as well as current meter measurements were used as checks, however. These calculations explain much of the "abnormal" behavior of flow and sediment relations observed by Heede (1964,[4] 1975a) and Piest et al. (1973, 1975): flow and sediment concentration in gullies are not necessarily related.

Concentration is related to the time since beginning of the particular flow event, however (table 1). During early flow, sediment concentrations and loads are high and then decrease with time until the easily available sediment derived from mass wasting processes within the gully has been removed. The last recession flows may be nearly clear water. This time-dependent characteristic of sediment concentration makes it possible that a high stream discharge may carry a much smaller load than a small one if the former occurs at a later date. Thus, if concentration is plotted over discharge, a hysteresis effect becomes visible.

Piest et al. (1975) stress that a sediment concentration parameter is usually a better erosion indicator than sediment discharge for testing erosion-causing variables. Two main reasons were given: (1) Sediment discharge is the product of flow and sediment concentration, which introduces a statistical bias into any relationship that may be runoff correlated; (2) runoff is not a basic variable, and would mask other, more basic variables since it usually is well correlated with the erosion condition of the watershed.

In the Iowa study, mass wasting of gully banks and headcuts were the prime erosion processes, not tractive force or stream power. Piest et al. (1973) found that height of water table, soil cohesive strength, and rate of water infiltration were the main factors controlling stability of gully banks. At Alkali Creek in western Colorado, where soils have up to 60 percent clay, mass wasting of gully banks takes place mainly during rainfalls that are sufficient to wet and thus change the cohesiveness of the banks, but insufficient to cause gully flows (fig. 3).

Processes and Morphology

Discontinuous Gullies

Leopold and Miller (1956) classified gullies as discontinuous or continuous. Discontinuous gullies may be found at any location on a hillslope. Their start is signified by an abrupt headcut. Normally, gully depth decreases rapidly downstream. A fan forms where the gully intersects the valley. Discontinuous gullies may occur singly or in a system of chains (Heede 1967) in which one gully follows the next downslope. These gullies may be incorporated into a continuous system either by fusion with a tributary, or may become a tributary to the continuous stream net themselves by a process similar to stream "capture." In the latter case, shifts on the alluvial fan cause the flow from a discontinuous gully to be diverted into a gully, falling over the gully bank. At this point, a headcut will develop that proceeds upstream into the discontinuous channel where it will form a nickpoint. Headward advance of the nickpoint will lead to gully deepening.

A chain of discontinuous gullies can be expected to fuse into a single continuous channel. Heede (1967) described the case history of such a fusion. Within three storm events of less than exceptional magnitude, the headcut of the downhill gully advanced 13 m to the next uphill channel, removing 70 m^3 of soil and forming one gully.

Vegetation types on the eastern and western flanks of the Colorado Rocky Mountains have not controlled the advance of headcuts of discon-

[4]Heede, Burchard H., 1964. A study to investigate gully-control measures on the Alkali Creek watershed, White River National Forest. Progress Report No. 3. 29 p. (On file at Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)

Table 1.--Suspended sediment samples from gully flows on Alkali Creek watershed, Colorado Rocky Mountains, 1964[1] and 1975[2]

Sampling station	Watershed area	Date	Flow		Sediment				
			Average velocity	Discharge	Concentration	Discharge	Sand	Silt	Clay
	km^2		*m/s*	*m^3/s*	*p.p.m.*	*kg/s*	*Percent*		
		1964							
Gully 3	0.5	April 16	0.7	0.20	35,706	7.17	12.9	53.4	33.7
		April 29	.5	.02	12,402	.23	7.5	54.6	37.9
Main Gully A	2.8	April 14	.3	.23	20,766	4.81	1.5	57.9	40.6
		April 16	.4	.55	13,432	7.39	2.5	62.3	35.2
		April 28	.5	.19	4,499	.86	5.9	63.5	30.6
		May 26	.3	.02	19	.0003	--	--	--
Main Gully B	26.9	April 15	1.5	2.25	63,855	143.52	34.1	45.1	20.8
		April 16	2.0	3.04	35,134	106.59	51.4	24.9	23.7
		April 29	1.0	.99	4,628	4.58	17.3	73.8	8.9
		May 26	.4	.03	12	.0004	--	--	--
		1975							
Gully 3	0.5	April 25	.4	.06	2,775	.18	--	--	--
		April 26	.6	.04	1,255	.05	--	--	--
Main Gully A	2.8	April 24	.7	.19	2,377	.44	--	--	--
		April 26	.7	.25	932	.23	--	--	--
		April 28	.6	.04	178	.01	--	--	--

[1]The flow of 1964, caused by snowmelt, was preceded by a dry channel period of 1 year.
[2]Since 1971, the flows are perennial but decrease to magnitudes of less than 0.028 m^3/s by midsummer, except after intense rains. The 1975 flow was mainly snowmelt runoff.

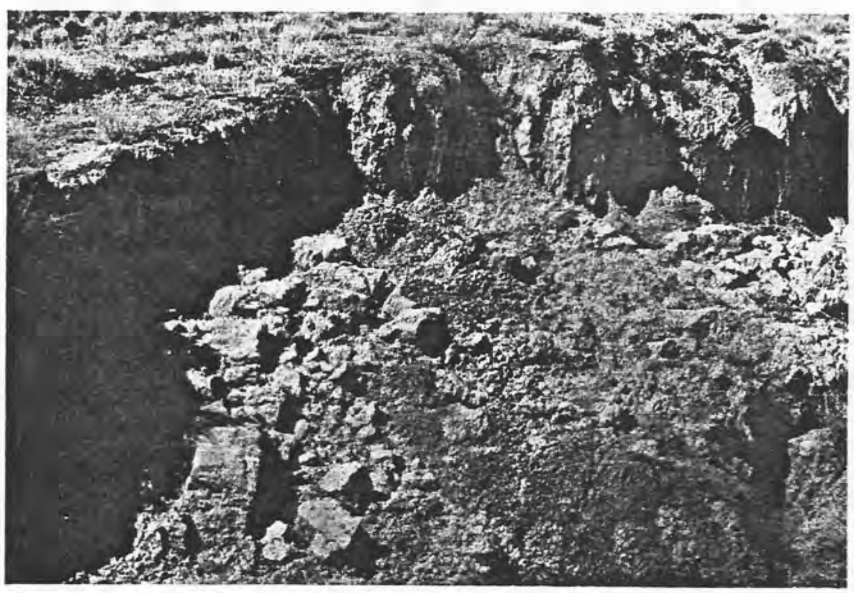


Figure 3.—Bank sloughing in gully on Alkali Creek watershed, western Colorado, during a period with no channel flow.

tinuous gullies (fig. 4). Ponderosa pine and Douglas-fir types, both with understory of grasses and other herbaceous vegetation, grew on the eastern flank; grass and sagebrush dominated the western flank. Since dense root mats of all these species occur at a depth below ground surface of only 0.3 to 0.6 m, undercutting by the waterfall over the headcut lip renders the mats ineffective.

Investigation of valley fill profiles and discontinuous gullies in Wyoming and New Mexico showed that discontinuous gullies formed on reaches of steeper gradient within a valley (Schumm and Hadley 1957). The authors postulated that overly steep gradients within alluviated valleys could be explained by deficiency of water in relation to sediment. In arid and semiarid areas, water losses along stream courses are well known (Murphey et al. 1972). The maintenance of stable alluvial streambeds is related to the quantity of water and the quantity and type of sediment moving through the system (Schumm 1969).

On the Alkali Creek watershed, evidence suggests that discontinuous gullies began to form at locations on the mountain slopes that were characterized by a break in slope gradient. This observation coincides with Schumm and Hadley's (1957) survey on valley floor and discontinuous gully profiles in Wyoming and New Mexico, and with Patton and Schumm's (1975) investigations of gullies in the oil-shale mountains of western Colorado. There, the breaks in valley gradients constituted a critical oversteepening of the valley slope. The oversteepening was the product of tributary streams that deposited large alluvial fans on the valley floor. Since flow data were not available, Patton and Schumm related the valley slope to drainage area. Discriminant-function analysis showed that, for areas larger than 10 km^2, a highly significant relationship existed between slope gradient, drainage area, and gullying. Discontinuous gullies occurred only above a critical slope value for a given area. The authors suggested that the results may be applicable only for the study region, since climate, vegetation and geology were considered constants. Yet for this particular region, the land manager obtained a valuable tool that tells him where discontinuous gullies may form.

The initiation of a discontinuous gully may also be explained by piping collapse (Hamilton 1970). Leopold et al. (1964) reported soil pipes to be an important element in the headward extension of this gully type. Since soil piping may be related to soil sodium, soil chemistry must also be regarded as a factor in gully formation, as demonstrated on the Alkali Creek watershed (Heede 1971). Piping soils (fig. 5), which caused gully widening and the formation of tributary gullies (fig. 6), had a significantly higher exchangeable sodium percentage (ESP) than nonpiping soils. The sodium decreased the layer permeability of the soils by 88 to 98 percent. Other prerequisites for the occurrence of pipes were low gypsum content, fine-textured soils with montmorillonite clay, and hydraulic head.

Older soil piping areas showed that extensive presence of pipes leads to a karstlike topography (fig. 7). The mechanical breakdown of the soils under such conditions facilitates leaching of the sodium from the soils, which in turn benefits plants. The new topography, characterized by more gentle gully side slopes compared with the former vertical walls of sodium soils, permits increased water infiltration, and natural rehabilitation of the gully by vegetation.

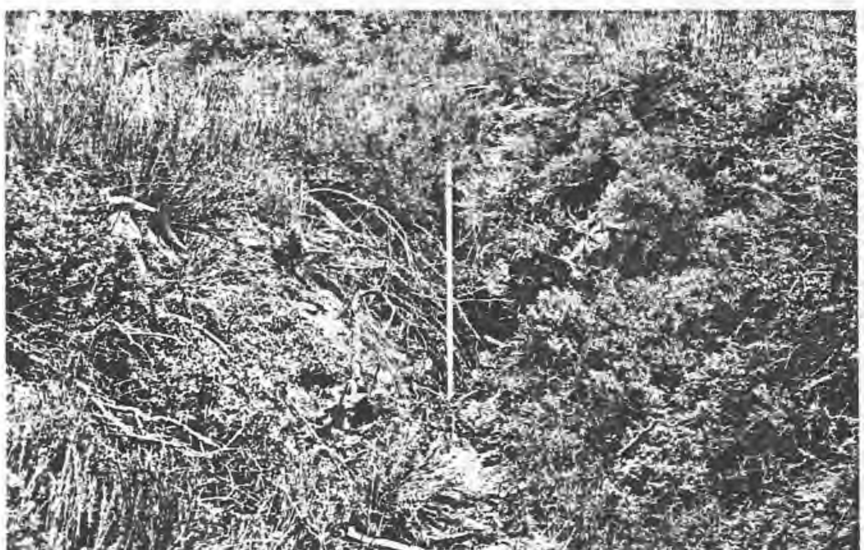
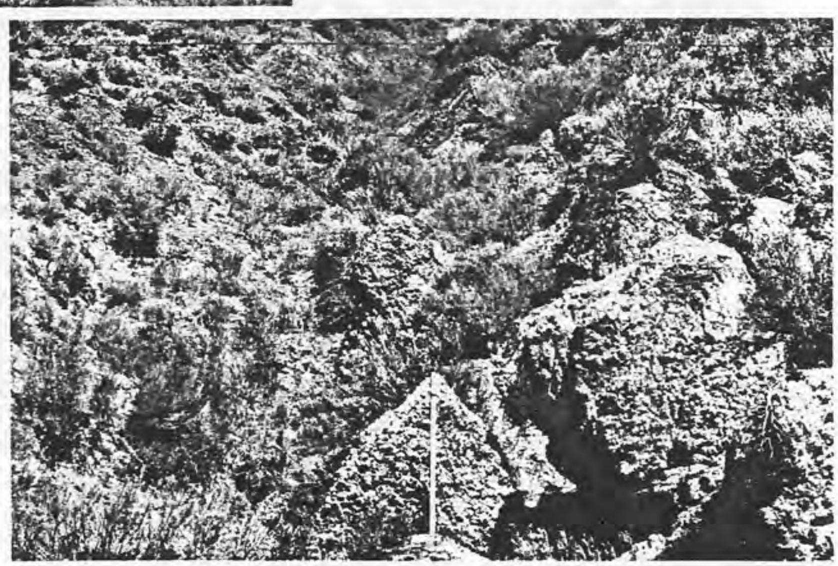


Figure 4.—Upstream view of headcut in gully 4, Alkali Creek watershed, before treatment. Length of rod is 1.7 m.

Figure 5.— Soil pipes on the Alkali Creek watershed drain runoff into the gully.

Figure 6.—This tributary to the main gully of Alkali Creek developed after the roof of the soil pipe collapsed.

Figure 7.— After the collapse of the soil pipes, lining several reaches of gullies on the Alkali Creek watershed, the vertical gully walls have begun to break down. The resultant topography begins to resemble that of a karst surface of mature to old-age stage.



Continuous Gullies

The continuous gully begins with many fingerlike extensions into the headwater area. It gains depth rapidly in the downstream direction, and maintains approximately this depth to the gully mouth. Continuous gullies nearly always form systems (stream nets). They are found in different vegetation types, but are prominent in the semiarid and arid regions. It appears that localized or regional depletion of any vegetation cover can lead to gully formation and gully stream nets, if other factors such as topography and soils are conducive to gully initiation. Several studies have demonstrated, however, that vegetation and soil type predominantly influence the morphology of gullies.

Schumm (1960) found that, in western channels, the type of material in banks and bottoms controls the cross-sectional channel shape. When the mechanical analysis of the soils was related to the width-depth ratio (upper width versus mean depth), linear regression indicated that increases in the ratio conformed with the increases of the average percent sand in the measured load. This relationship was also established by the Soil Conservation Service at Chickasha, Oklahoma (unpublished report). On Alkali Creek, where extensive sampling showed no significant differences in the texture of the soils, meaningful correlations between the width-depth ratio and thalweg length could not be established (Heede 1970).

Tuan (1966) reported, in a critical review of literature on gullies in New Mexico, that channels developed in a semiarid upland environment were of moderate depth, and cut into sandy alluvium. Deep trenches were rare. The influence of sand in gully bank material on sediment production was shown when upland gullies were studied in the loess hills of Mississippi (Miller et al. 1962). They found that the annual volume of sediment produced ranged from 0.091 to 0.425 m^3 per hectare of exposed gully surface. The lower rate was associated with an average 6-m vertical gully wall having a low percentage of uncemented sand, while the higher rate was found in gullies with 12 m vertical walls and a high percentage of uncemented sand. As illustrated by gully erosion on the Alkali Creek watershed, sediment production may also be related to the chemical composition of the soils (Heede 1971).

That vegetation surrounding a gully may exert stronger influences on the channel morphology than the soils was shown for small streams in northern Vermont (Zimmerman et al. 1967), and also for a large gully in California (Orme and Bailey 1971). The California gully occupied a 354-ha watershed in the San Gabriel Mountains. In an experiment to increase water yield, the riparian woodland was removed and replaced by grasses. Two years later, a wildfire destroyed all vegetation on the watershed, and 57 ha of side slopes (16 percent of total area) were seeded to grass. The vegetative conversion was maintained by aerial sprays with selective herbicides. When high-intensity storms hit the watershed 6 and 9 years after conversion, stream discharge rates and sediment loads increased to previously unknown magnitudes. Changes in longitudinal profile and channel cross sections were spectacular. The gully "survived" as a relict feature partially clogged with storm debris, but had not regained its hydraulic efficiency by mid-1971.

In the Vermont streams, encroachment and disturbance by vegetation eliminated the geomorphic effect of channel width increase in the downstream direction, a normal stream behavior. In contrast, on the Alkali Creek watershed where gullies did not experience severe encroachment or disturbance by the sagebrush-grass cover, this geomorphic effect was eliminated by local rock outcrops and soils with low permeability due to high sodium (Heede 1970).

To establish gully morphology and possible stages of gully development, Heede (1974) analyzed the hydraulic geometry of 17 Alkali Creek gullies. Stream order analysis showed that 67 percent of the area of a fourth-order basin was drained by first-order streams. This is contrasted to 1 percent, the average for similar river basins in the United States (Leopold et al. 1964). Since the Alkali Creek gully system is still in the process of enlargement toward headwaters, the drainage area of the first-order streams will decrease with time. The longitudinal profiles of the gullies exhibited weak concavities, and it was argued that concavity would increase with future gully development.

The shape factor of the gullies, relating maximum to mean depth and expressing channel shape, had relatively high values (average 2.0). These values represent cross sections with large wetted perimeters that in turn indicate hydraulic inefficiency of the gullies.

The tested hydraulic parameters—drainage net, profile, and shape factor—were interpreted as indicating juvenile stages of gully development (termed youthful and early mature). Thus it can be argued that gully development should be recognized in terms of landform evolution, proceeding from young to old age stages. If stages of development could be expressed in terms of erosion rates and sediment yields, a useful tool would be provided for the watershed manager.

When the hydraulic geometry of the gullies was compared with that of rivers, it was suggested that the mature gully stage should be characterized by dynamic equilibrium. The condition of dynamic equilibrium does not represent a true balance between the opposing forces, but includes the capability to adjust to changes in short timespans, and thus regain equilibirum (Heede 1975b). Although some gullies of the Alkali Creek watershed approached this condition, it must be realized that in ephemeral gullies, a mature stage may not be defined by stream equilibrium alone, but may include other aspects of stability such as channel vegetation. Invasion of vegetation into the gully is stimulated during dry channel periods.

During the youthful stage, gully processes proceed toward the attainment of dynamic equilibrium, while in the old age stage, a gully loses the characteristics for which it is named, and resembles a river or "normal" stream. Gully development may not end with old age, however. Environmental changes such as induced by new land use (Nir and Klein 1974) and climatic fluctuation or uplift, may lead to rejuvenation, throwing the gully back into the youthful stage.

The condition of steady state, representing true equilibrium, is a theoretical one and can hardly be conceived to apply to gully systems, with the possible exception of very short timespans. Schumm and Lichty (1965) expressed a similar view when they stated that only certain components of a drainage basin may be in steady state.

We must also recognize that gully development is not necessarily an "orderly" process, proceeding from one condition to the next "advanced" one. Erosion processes accelerate at certain times, and at others apparently stand still. For example, Harris (1959) established four epicycles of erosion during the last 8,000 years for Boxelder Creek in northern Colorado. During the interims, the stream was in dynamic equilibrium most of the time. In a case study on ephemeral gullies, it was demonstrated that flows alter the channel, at times leaving a more stable, at others a very unstable, condition (Heede 1967). The latter internal condition leads to the well-known explosive behavior of geomorphic systems (Thornes 1974). External events, however, such as flooding in natural streams, may also lead to rapid, drastic changes (Schumm and Lichty 1963).

Growth Models

At present, no physical formula or model is available that describes the advancement of gullies, although several statistical models have been devised. In the badlands of southern Israel, which are severely dissected by gullies, field data were statistically analyzed and a simple model for gully advance established (Seginer 1966). Seginer tested three geometric parameters of the watershed that can easily be measured: watershed area, length of watershed along the main depression, and maximum elevation difference in the watershed. Of course, these parameters are interrelated. Regression analyses for several combinations indicated that watershed area was the most important single factor explaining the deviations about the mean; additional factors did not supply more information.

The prediction equation derived was as follows:

$$E = C_1 A^{0.50} \tag{3}$$

where E is the advancement rate of the gully headcut, A is the watershed area draining into the headcut, and C_1 is a constant that varies from watershed to watershed.

It is obvious that a simplified approach to the quantification of gully processes, such as described above, at best presents empirical relationships valid for a given watershed at a given point in time. Assumptions of uniform distribution of rainfall (expressed by watershed area), uniform geology, soils, and vegetation, unchanged land uses, to name just a few, do not permit formulation of meaningful predictions.

The limitation of prediction equations based on statistical relations of a few selected parameters and factors was also illustrated by other studies. Thompson (1964) investigated the quantitative effect of independent watershed variables on rate of gully-head advancement. Variables were: drainage area above the gully head, slope of approach channel above the gully head, summation of rainfall from 24-hour rains equal to or greater than 13 mm, and a soil factor—the approximate clay content (0.005 mm or smaller) of the soil profile through which the head cut is advancing. Regression analysis showed that 77 percent of the variance was explained by the four variables. The t-test indicated that only drainage area, precipitation, and soils were highly significant in the regression equation at the 5 percent level to express the rate of headcut advancement. An R^2 value of 0.77 appears to signify an efficient relationship, yet about one-fourth of the variance is due to other, not measured variables. This unexplained fourth will prohibit the use of the prediction equation for most projects.

While Thompson (1964) chose the linear advancement of gully headcuts, Beer and Johnson (1963) selected the changes in gully surface area as the dependent variable. In addition to the independent variables used in the 1964 study, Beer and Johnson included an estimate of an index of surface runoff. The results showed that the gullying process was best represented by a logarithmic model, as contrasted with Thompson's linear model. All variables were evaluated from the past growth of the gullies. No controlled studies of the individual components responsible for the gullying process have been made.

The above-mentioned statistical investigations threw light on the important variables in gully growth, and thus added to our understanding of gullying. But quantification and prediction of growth still lack precision because past rates of gullying do not necessarily indicate future rates. Stages can be recognized in the development of gullies, and erosion and sediment production change between the stages (Heede 1974). Gully growth predictions without recognition of stage development may not be meaningful.

A deterministic growth model for gullies was proposed based on investigations in the badlands of S.E. Alberta, Canada, where climate, lithology, and total available relief are uniform (Faulkner 1974). Vegetation is practically absent. The constraints on the model are quite drastic in view of the variability of environments supporting gully systems. The model is an extension of Woldenberg's (1966) gradient derived from the allometric growth law (Huxley 1954), defined as

$$x = c_1 y^{d^1} \tag{4}$$

where x is the size of an organ, y represents the size of the organism to which the organ belongs, and c_1 and d^1 are constants.

Usually, nonuniformity of environmental factors such as soils and vegetation is the rule. The intermittent flow of ephemeral gullies adds another formidable task in making the present law sufficiently flexible to take care of the numerous field combinations. For most situations, the present model will therefore not yield results useful to the land manager.

The above compendium illustrates that our knowledge on gully mechanics and processes is limited. As we will recognize in the following chapters, art and judgment are still required in many phases of gully control.

OBJECTIVES IN GULLY CONTROL

Main Processes of Gully Erosion As Related To Control

The mechanics of gully erosion can be reduced to two main processes: downcutting and headcutting. Downcutting of the gully bottom leads to gully deepening and widening. Headcutting extends the channel into ungullied headwater areas, and increases the stream net and its density by developing tributaries. Thus, effective gully control must stabilize both the channel gradient and channel headcuts.

Long-Term Objective of Controls—Vegetation

In gully control, it is of benefit to recognize long- and short-term objectives because often it is very difficult or impossible to reach the long-term goal—vegetation—directly; gully conditions must be altered first. Required alterations are the immediate objectives.

Where an effective vegetation cover will grow, gradients may be controlled by the establishment of plants without supplemental mechanical measures. Only rarely can vegetation alone stabilize headcuts, however, because of the concentrated forces of flow at these locations. The most effective cover in gullies is characterized by great plant density, deep and dense root systems, and low plant height. Long, flexible plants, on the other hand, such as certain tall grasses, lie down on the gully bottom under impact of flow. They provide a smooth interface between flow and original bed, and may substantially increase flow velocities. These higher velocities may endanger meandering gully banks and, in spite of bottom protection, widen the gully. Trees, especially if grown beyond sapling stage, may restrict the flow and cause diversion against the bank. Where such restrictions are concentrated, the flows may leave the gully. This is very undesirable because, in many cases, new gullies develop and new headcuts form where the flow reenters the original channel.

Engineers' Measures—An Aid to Vegetation Recovery

If growing conditions do not permit the direct establishment of vegetation (due to climatic or site restrictions, or to severity of gully erosion) engineering measures will be required. These measures are nearly always required at the critical locations where channel changes invariably take place. Examples are nickpoints on the gully bed, headcuts, and gully reaches close to the gully mouth where deepening, widening, and deposition alternate frequently with different flows (see fig. 2). Nickpoints signify longitudinal gradient changes; a gentler gradient is being extended toward headwaters by headcutting on the bed (fig. 8). Normally, critical locations are easily definable since the active stage of erosion at these sites leaves bed and banks in a raw, disturbed condition.

The designer must keep in mind that well-established vegetation perpetuates itself and thus represents a permanent type of control. In contrast, engineering measures always require some degree of maintenance. Because maintenance costs time and money, projects should be planned so that maintenance is not required indefinitely.

An effective engineering design must help establish and rehabilitate vegetation. Revegetation of a site can be aided in different ways. If the gully gradient is stabilized, vegetation can become established on the bed. Stabilized gully bottoms will make possible the stabilization of banks, since the toe of the gully side slopes is at rest (fig. 9). This process can be speeded up mechanically by sloughing gully banks where steep banks would prevent vegetation establishment. Banks should be sloughed only after the bottom is stable, however.

Vegetation rehabilitation is also speeded if large and deep deposits of sediment accumulate in the gully above engineering works. Such alluvial deposits make excellent aquifers, increase channel storage capacity, decrease channel gradients, and thus, decrease peak flows. Channel deposits may also raise the water table on the land outside the gully. They may reactivate dried-up springs, or may convert ephemeral

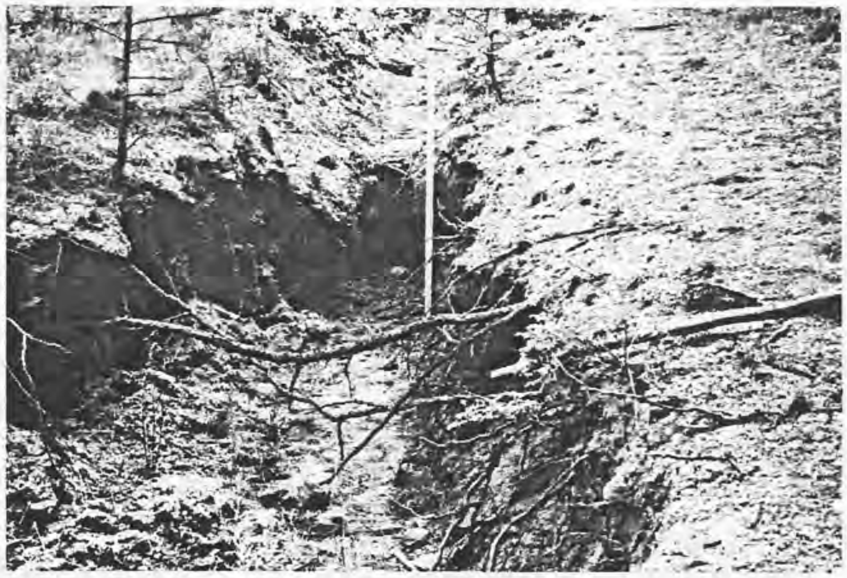


Figure 8.—The nickpoint, located on the gully bottom and indicated by a survey rod, has a depth of about 0.5 m. Although this gully appears to be stabilized by the invasion of vegetation, rejuvenation must be expected by the upstream advance of the nickpoint. The root systems will be undercut and gully depth and width will increase. Length of the rod is 1.7 m.

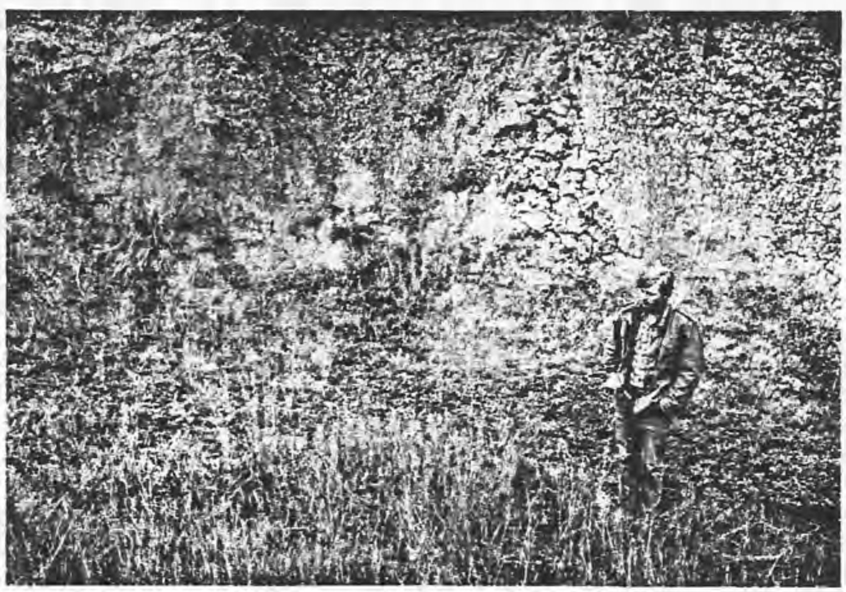


Figure 9.—The bank of Main gully, Alkali Creek watershed, 12 years after installation of check dams. Stabilization of the gully bottom made possible the invasion of dense vegetation that now is creeping up the bank. The man stands at the toe of the bank.

springs to perennial flow. All these results create conditions much more favorable to plant growth than those existing before control.

Watershed Restoration Aids Gully Control Measures

Measures taken outside the channel can also aid revegetation processes in the gully. Improvements on the watershed that (1) increase infiltration and decrease overland flow, and (2) spread instead of concentrate this flow, will benefit gully healing processes. A study on sediment control measures showed that sediment yields were reduced 25 to 60 percent by land treatment and land use adjustments, as surveyed at 15- to 20-year-old flood-water-retarding structures in the southern Great Plains (Renfro 1972). But when combined land treatment and structural measures were applied, sediment yields were reduced 60 to 75 percent.

Normally, however, gully improvements can be attained quicker within the gully than outside, because of concentration of treatment and availability of higher soil moisture in the defined channel.

Many types of watershed restoration measures have been devised, and the literature on the subject is abundant: Poncet (1965) described an integrated approach to erosion control on the watershed and in gullies; Copeland (1960) presented a photo-record of watershed slope stabilization in the Wasatch Mountains of Utah; and Bailey and Copeland (1961) analyzed the behavior of slope stabilization structures.

Since watershed restoration measures are only supplemental to gully control, some examples will suffice here: seeding and planting with and without land preparation and fertilization; vegetation cover conversions; and engineering works such as reservoirs, water diversions, benches, terraces, trenches, and furrows.

Immediate Objectives of Control

Different types of measures benefit plants in different ways. It is therefore important to clarify the type of help vegetation establishment requires most. Questions should be answered such as: Is the present moisture regime of the gully bottom sufficient to support plants, or should the bottom be raised to increase moisture availability? One must recognize that a continuous, even raising of the bottom is not possible. Due to the processes of sedimentation above check dams, deposits have a wedge-shaped cross section if plotted along the thalweg.

The immediate objectives of a gully treatment must consider other aspects in addition to plant cover. Usually, these considerations involve hydraulics, sedimentation, soils, and sometimes the logistics required for the management of the watershed. For instance, management may call for deposits of maximum possible depth at strategic locations to provide shallow gully crossings. Thus, if sediment catch is a desirable objective, large dams should be built. But if esthetic considerations make check dams undesirable (and watershed logistics and revegetation offer no problems), the gully bottom may be stabilized with dams submerged into the bed, and thus invisible to the casual observer.

These examples illustrate how important it is to clarify the immediate and overall objectives of a planned treatment before deciding on approaches and measures. The objectives determine the measures; the measures, the type of result.

GULLY CONTROL STRUCTURES AND SYSTEMS

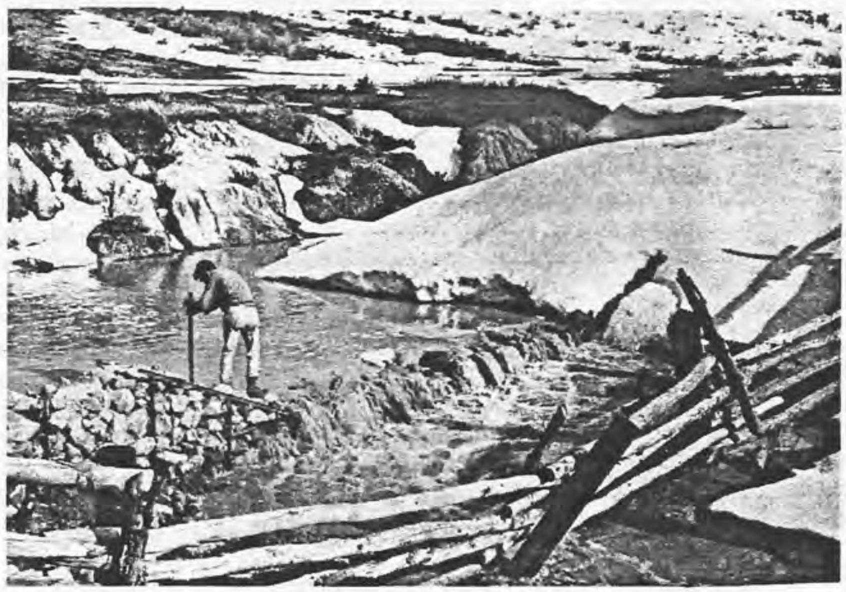
Types of Porous Check Dams

The most commonly applied engineering measure is the check dam. Forces acting on a check dam depend on design and type of construction material. Nonporous dams with no weep holes, such as those built from concrete (Poncet 1963, Heede 1965b, Kronfellner-Kraus 1971), sheet steel, wet masonry, and fiberglass, receive a strong impact from the dynamic and hydrostatic forces of the flow (fig. 10). These forces require strong anchoring of the dam into the gully banks, to which most of the pressure is transmitted. In contrast, porous dams release part of the flow through the structure, and thereby decrease the head of flow over the spillway and the dynamic and hydrostatic forces against the dam (fig. 11). Much less pressure is received at the banks than with nonporous dams. Since gullies generally are eroded from relatively soft soils, it is easier to design effective porous check dams than nonporous ones. Once the catch basin of either porous or nonporous dams is filled by sediment deposits, however, structural stability is less critical because the dam crest has become a new level of the upstream gully floor.

Loose rock can be used in different types of check dams. Dams may be built of loose rock only, or the rock may be reinforced by wire mesh, steel posts, or other materials. The reinforce-

Figure 10.—This prefabricated, prestressed concrete check dam accumulated sediment readily because the structure is not porous. At the same time, dynamic and hydrostatic forces of the flow on the dam are much stronger than those at a porous rock check dam. The discharge over the spillway of this structure, installed on the Alkali Creek watershed, is about 0.4 m^3/s.

Figure 11.—As contrasted to impervious dams, rock check dams such as this double-fence structure release much of the flow, and hence hydrostatic pressure, through the structure.



ments may influence rock size requirements. If wire mesh with small openings is used, rocks may be smaller than otherwise required by the design flow.

Some different types of check dams will be described, but the field of check dam design is wide open. Many variations are possible. The torrent-control engineers of Europe have been especially successful with filter or open dams. Most of their designs are for large torrents where stresses on the structures are much greater than those in gullies, generally. Clauzel and Poncet (1963) developed a concrete dam whose spillway is a concrete chute with a steel grid as the chute bottom. This grid acts as a filter for the bedload. Periodic cleaning of the dam is required, however.

Other types of filter dams have vertical grids, or grids installed at an angle to the vertical. Such dams are described by Puglisi (1967), Kronfellner-Kraus (1970), and Fattorelli (1971).

All the torrent control dams are quite sophisticated, and thus costly. Such high costs are often justified in Europe, however, since population densities require the most effective and lasting control measures. These qualities are especially important if the basic geologic instability of the alpine torrents is considered. In contrast, most gullies in the western United States are caused by soil failure, and life and high-cost property are not usually endangered. Simpler, low-cost structures will therefore be preferable. Some of the most effective and inexpensive dams are buil mainly from loose rock. They will, therefore, be emphasized in the descriptions that follow.

Loose Rock

The basic design of a loose-rock check dam is illustrated in figure 12. If facilities are not available to use the computer program developed by Heede and Mufich (1974), volumes of excavation and of rocks required in the construction can be calculated from the drawings. Rock volumes can also be obtained from an equation discussed in the section on Equations for Volume Calculations. In a Colorado project, the drawings also served well in the field as construction plans (Heede 1966).

Since loose-rock dams are not reinforced, the angle of rest of the rock should determine the slopes of the dam sides. This angle depends on the type of rock, the weight, size, and shape of the individual rocks, and their size distribution. If the dam sides are constructed at an angle steeper than that of rest, the structure will be unstable and may lose its shape during the first heavy runoff. For the design of check dams, the following rule of thumb can be used: the angle of rest for angular rock corresponds to a slope ratio of 1.25 to 1.00; for round rock, 1.50 to 1.00. Figure 13 illustrates a dam built from angular loose rock.

Wire-Bound Loose Rock

A wire-bound check dam is identical in shape to that of a loose-rock dam, but the loose rock is enclosed in wire mesh to reinforce the structure. The flexibility within the wire mesh is sufficient to permit adjustments in the structural shape, if the dam sides are not initially sloped to the angle of rest. Therefore, the same rock design criteria are required for a wire-bound dam as for a loose-rock structure.

The wire mesh should: (1) be resistant to corrosion, (2) be of sufficient strength to withstand the pressure exerted by flow and rocks, and (3) have openings not larger than the average rock size in the dam. Wire mesh may not be effective in boulder-strewn gullies supporting flows with heavy, coarse loads.

Single Fence

Single-fence rock check dams (figs. 14, 15) differ greatly in shape and requirements of construction materials from the loose-rock and wire-bound dams. These structures consist of (1) a wire-mesh fence, fastened to steel fenceposts and strung at right angles across the gully, and (2) a loose-rock fill, piled from upstream against the fence. The rock fill can be constructed at an angle steeper than that of rest for two reasons:

1. The impact of flows will tend to push the individual rock into the fill and against the dam.
2. Sediment deposits will add stability to the fill and will eventually cover it.

The design of this type of check dam should emphasize specifications for the wire mesh, and the setting, spacing, and securing of the steel fenceposts. The wire mesh specifications will be the same as those for the wire-bound dams.

The steel fenceposts should be sufficiently strong to resist the pressure of the rock fill and the flows, and must be driven into the gully bot-

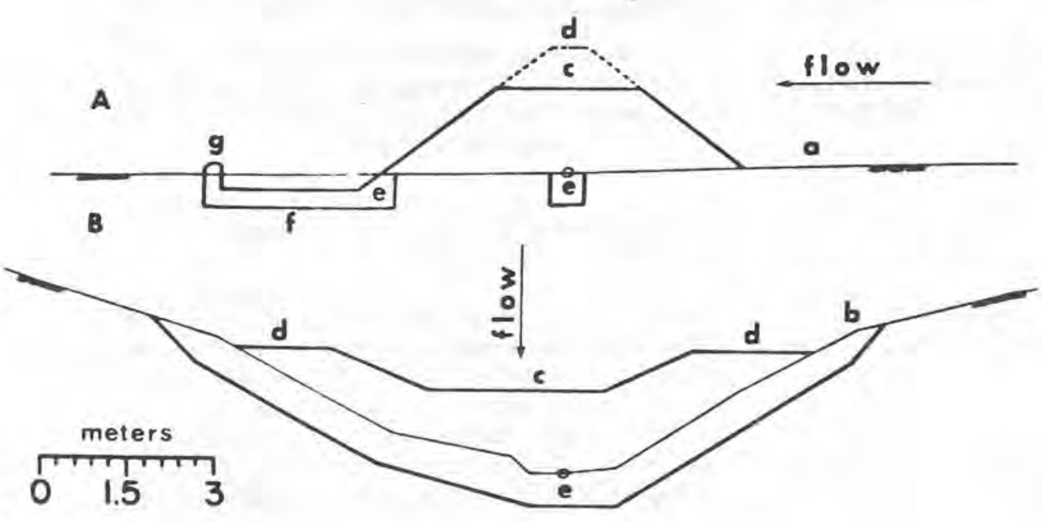


Figure 12.—Construction plans for a loose-rock check dam.
A, Section of the dam parallel to the centerline of the gully.
B, Section of the dam at the cross section of the gully. a = original gully bottom; b = original gully cross section; c = spillway; d = crest of freeboard; e = excavation for apron; g = end sill.

Figure 13.—Upstream view of a loose-rock check dam. The catchment basin filled with sediment during the first spring runoff after construction. Rod is 1.7 m high.

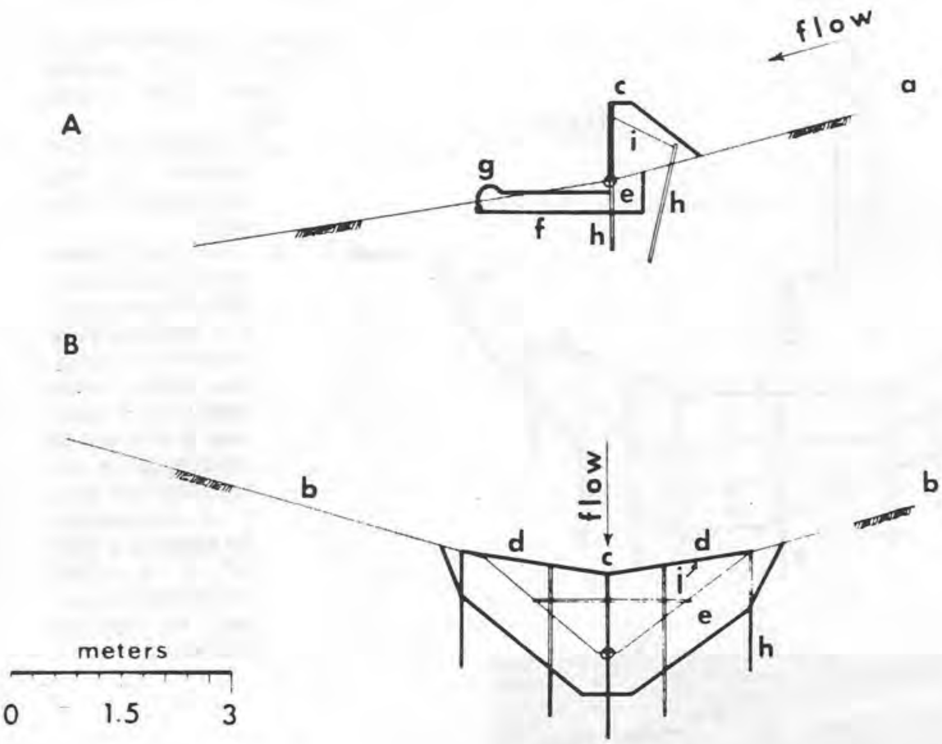
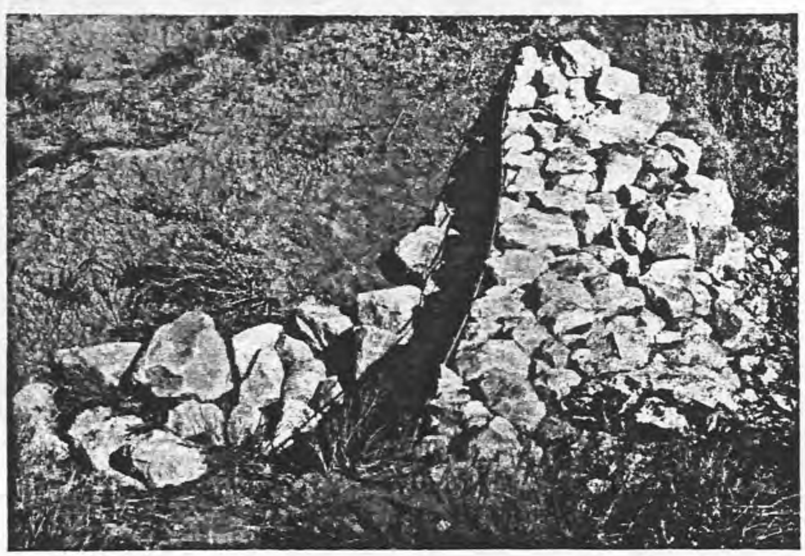


Figure 14.—Construction plans for a single-fence rock check dam.
A, Section of the dam parallel to the centerline of the gully.
B, Section of the dam at the cross section of the gully. a = original gully bottom; b = original gully cross section; c = spillway; d = crest of freeboard; e = excavation for key; f = excavation for apron; g = end sill; h = steel fencepost; k = guys; j = rebar, 13 mm in diameter.

Figure 15.—View across a single-fence dam. Apron and gully bank protection are to the left of the dam crest.



tom and side slopes to a depth that insures their stability in saturated soil. If it is impractical to drive posts to sufficient depths, the stability of the posts should be enhanced by guys. These guys should be anchored to other posts that will be covered and thus held in place by the rock fill.

In general, spacing between the fenceposts should not be more than 1.2 m to prevent excessive pouching (stretching) of the wire mesh. Where conditions do not allow this spacing, a maximum of 1.5 m can be used but the fence must be reinforced by steel posts fastened horizontally between the vertical posts. Excessive pouching of the wire mesh reduces the structural height and impairs the stability of the dam.

Double Fence

The double-fence rock check dam has two wire mesh fences, strung at a distance from each other across the channel (fig. 16). In this type of dam, a well-graded supply of rocks is essential, otherwise the relative thinness of the structure would permit rapid throughflow, resulting in water jets. Double-fence dams should only be built if an effective rock gradation can be obtained.

In Colorado, parallel fences were spaced 0.6 m (Heede 1966). Peak flows did not exceed 0.7 m^3/s, and loads consisted mainly of finer material. Dams were no taller than 1.8 m (fig. 17). At many dam sites, maintenance and repairs

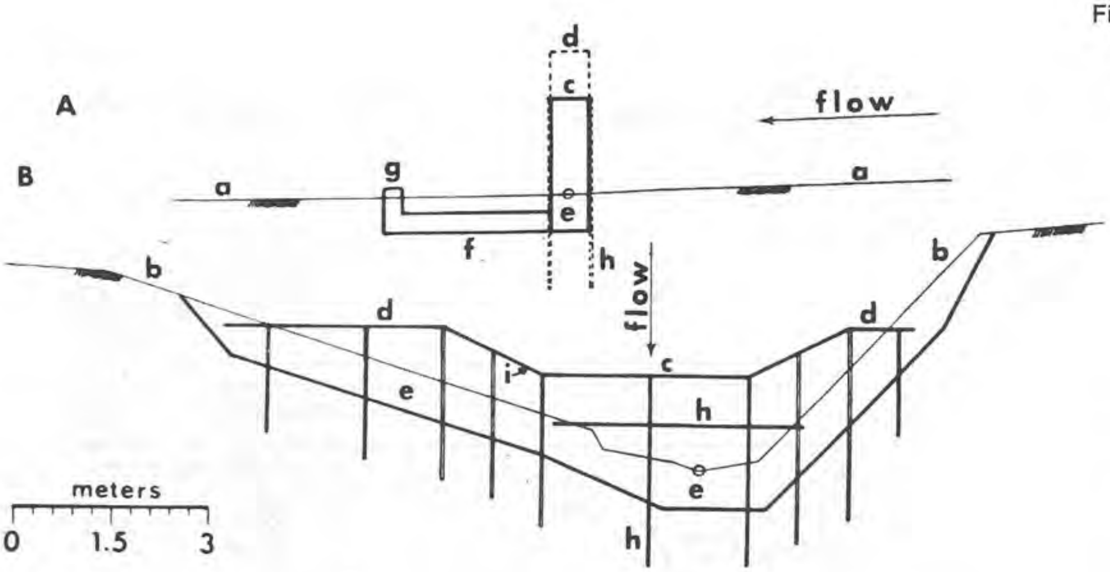


Figure 16.—Construction plans for a double-fence rock check dam.
A, Section of the dam parallel to the centerline of the gully.
B, Section of the dam at the cross section of the gully. a = original gully bottom; b = original gully cross section; c = spillway; d = crest of freeboard; e = excavation for key; f = excavation for apron; g = end sill; h = steel fencepost; i = rebar, 13 mm in diameter.

Figure 17.—Upstream view of a double-fence dam. Note the bank protection work. The apron is covered by water. Length of rod is 1.7 m.

were required because excessive water jetting through the structures caused bank damage. The percentage of small rock sizes was too low.

When flows of large magnitude, say 2 m^3/s, or gullies on steep hillsides are encountered, the base of the double-fence dam should be wider than the crest. This will add structural stability and increase the length of the flow through the lower part of the dam.

Gabion

A gabion check dam consists of prefabricated wire cages that are filled with loose rock. Individual cages are placed beside and onto each other to obtain the dam shape. Normally, this dam is more esthetically pleasing, but it is more costly than loose-rock or wire-bound rock check dams.

Headcut Control

Headcuts can be stabilized by different types of structures, but all have two important requirements: (1) porosity in order to avoid excessive pressures and thus eliminate the need for large, heavy structural foundations; and (2) some type of inverted filter that leads the seepage gradually from smaller to the larger openings in the structure. Otherwise, the soils will be carried through the control, resulting in erosion. An inverted filter can be obtained if the headcut wall is sloughed to such an angle that material can be placed in layers of increasing particle size, from fine to coarse sand and on to fine and coarse gravel. Good results may also be obtained by use of erosion cloth, a plastic sheet available in two degrees of porosity.

If rock walls reinforced by wire mesh and steel posts are used, site preparation can be minimized. Loose rock can be an effective headcut control (Heede 1966) if the flow through the structure is controlled also. As in loose-rock check dams, the size, shape, and size distribution of the rock are of special importance to the success of the structure. The wall of the headcut must be sloped back so the rock can be placed against it.

If the toe of the rock fill should be eroded away, the fill would be lost. Therefore, stabilization of this toe must be emphasized in the design. A loose-rock dam can be designed to dissipate energy from the chuting flows, and to catch sediment (fig. 18). Sediment depositions will further stabilize the toe of the rock fill by encouraging vegetation during periods with no or low channel flow.

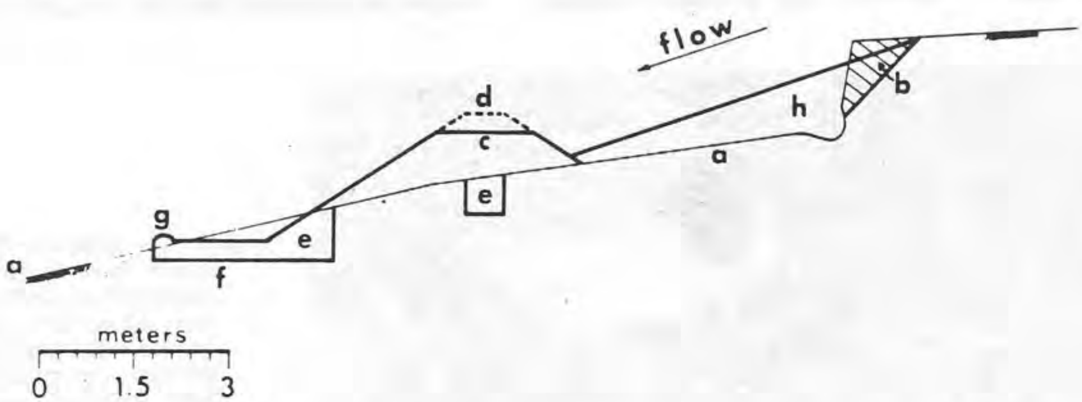


Figure 18.—Construction plan for a gully headcut control with a loose-rock check dam. The section of the structure is parallel to the centerline of the gully. a = original gully bottom; b = excavated area of headcut wall; c = spillway; d = crest of freeboard; e = excavation for key; f = excavation for apron; g = end sill; h = rock fill.

General Design Criteria

Loose Rock

Loose rock has proved to be a very suitable construction material if used correctly. Often it is found on the land and thus eliminates expenditures for long hauls. Machine and/or hand labor may be used. The quality, shape, size, and size distribution of the rock used in construction of a check dam affect the success and lifespan of the structure.

Obviously, rock that disintegrates rapidly when exposed to water and atmosphere will have a short structural life. Further, if only small rocks are used in a dam, they may be moved by the impact of the first large water flow, and the dam quickly destroyed. In contrast, a check dam constructed of only large rocks that leave large voids in the structure will offer resistance to the flow, but may create water jets through the voids (fig. 19). These jets can be highly destructive if directed toward openings in the bank protection work or other unprotected parts of the channel. Large voids in check dams also prevent the accumulation of sediment above the structures. In general, this accumulation is desirable because it increases the stability of structures and enhances stabilization of the gully.

Large voids will be avoided if the rock is well graded. Well-graded rock will permit some flow through the structure. The majority of the rock should be large enough to resist the flow.

Since required size and gradation of rock depend on size of dam and magnitude of flow, strict rules for effective rock gradation cannot be given. The recommendations given below are empirical values derived from gully treatments in the Colorado Rocky Mountains, and should be evaluated accordingly. The designer should use these values only as a guide for his decision.

As a general rule, rock diameters should not be less than 10 cm, and 25 percent of all rocks should fall into the 10- to 14-cm size class. The upper size limit will be determined by the size of the dam; large dams can include larger rock than small ones. Flat and round rock, such as river material, should be avoided. Both types slip out of a structure more easily than broken rocks, which anchor well with each other.

In general, large design peak flows will require larger rock sizes than small flows. As an example, assume that the designed total dam height ranges between 1 and 2 m, where total height is measured from the bottom of the dam to the crest of the freeboard. Type of dam is loose rock without reinforcement. Design peak flow is estimated not to exceed 1 m^3/s. An effective rock gradation would call for a distribution of size classes as follows:

Size	Percent
10–14 cm	25
15–19 cm	20
20–30 cm	25
31–45 cm	30

If, on the other hand, dam height would be increased to 3 m, rock up to 1 m diameter, constituting 15 percent of the volume, could be placed into the base of the dam and the second size class decreased by this portion. If peak flow

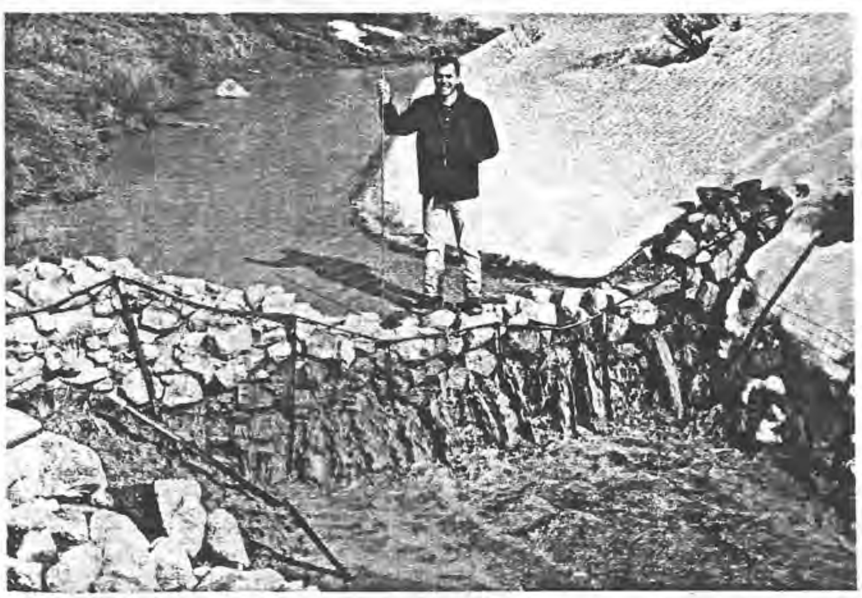


Figure 19.—Because this double-fence rock check dam was built with an insufficient portion of small rocks, many large voids allow water jets through the structure. Note that water is not running over the spillway. The jets endanger the stability of the structural keys and bank protection work.

was estimated not to exceed 0.75 m^3/s, the 31- to 45-cm size class could be eliminated and 55 percent of the volume could be in the 20- to 30-cm class.

In ephemeral gullies, only in exceptional cases will meaningful flow information be available that permits a realistic estimate of average velocities at the dam sites. If flow information is available, an equation developed by Isbach and quoted by Leliavsky (1957) may be used to check the suitability of the larger sizes. The equation relates the weight of rock to the mean velocity of the flow as follows:

$$W = 2.44(10^{-5})V^6 \qquad (5)$$

where W is the weight of rock related to D_{65} of the rocks, and V is stream velocity. D_{65} is the sieve size that allows 65 percent of the material to pass through. This equation states that 65 percent of the rocks can be smaller and 35 percent larger than the calculated weight. As stated above, the smallest size should have a diameter of 10 cm.

Spacing

The location of a check dam will be determined primarily by the required spacing of the structures. Requirements for spacing depend on the gradients of the sediment deposits expected to accumulate above the dams, the effective heights of the dams, the available funds, and the objective of the gully treatment. If, for instance, the objective is to achieve the greatest possible deposition of sediment, high, widely spaced dams would be constructed. On the other hand, if the objective is mainly to stabilize the gully gradient, the spacing would be relatively close and the dams low.

In general, the most efficient and most economical spacing is obtained if a check dam is placed at the upstream toe of the final sediment deposits of the next dam downstream. This ideal spacing can only be estimated, of course, to obtain guidelines for construction plans.

Normally, objectives of gully control require spacings of check dams great enough to allow the full utilization of the sediment-holding capacity of the structures. Determination of this spacing requires definite knowledge of the relationship between the original gradient of the gully channel and that of sediment deposits above check dams placed in the gully. This relationship has been hypothesized by several authors.

Kaetz and Rich[5] were the first known investigators to propose a relationship between the slope of sediment deposits above structures and that of the original thalweg. They concluded that the ratio varied between 0.3 and 0.6. The steeper deposition slopes were found in channels carrying coarse gravel, in contrast to the flatter slopes associated with fine loads. When some of the same structures were resurveyed 22 years later (Myrick Survey, as quoted by Leopold et al. 1964), the sediment wedge had lengthened only slightly since the time of the first survey. The increase in length was accompanied by a slight steepening of the deposition slope.

The Los Angeles County Flood Control District, engaged in gully control since the 1930's, used an empirically established ratio of 0.7 between deposition and original bed slope (Ferrell 1959, Ferrell and Barr 1963). In a sediment trend study, conducted 9 years after installation of a check dam treatment, the validity of this ratio could not be confirmed (Ruby 1973). It appears that a 9-year period is not sufficiently long to prove or disprove the rule of thumb.

Deposition of sediment above dams is a dynamic process dependent on regimen and magnitudes of flows during the treatment period. In a laboratory study on low-drop structures for alluvial flood channels, it was demonstrated that the regimen of flow exerts an overriding influence on channel grade (Vanoni and Pollak 1959). Also, Ruby (1973) stated that the system is constantly changing. But it is important to note that in the Los Angeles treatment, all sediment deposits have consistently aggraded, and not one has yet degraded. This suggests that sediment is still accumulating above the check dams.

Heede (1960) evaluated 20- to 26-year-old check dams in the Colorado Front Range (eastern flank) of the Rocky Mountains, and found the ratio of deposition to original bed slope fluctuating between 0.5 and 0.65. The soils had a large amount of coarse particles, and clay content was low. A check of 15-year-old earth check dams and stock pond structures on the western flank of the Colorado Rocky Mountains showed an average ratio of 0.7 (Heede 1966). This ratio was applied to an extensive watershed restoration

[5]*Kaetz, A. G., and L. R. Rich, 1939. Report of survey made to determine grade of deposition above silt and gravel barriers. (Unpublished memo, dated Dec. 5, 1939, on file, U.S. Soil Conserv. Serv. library, Albuquerque, N.M.)*

project on the western slope of the Rocky Mountains in 1963. That project is now being evaluated.[6]

Channel structures were investigated in Arizona washes by Hadley (1963). He concluded that a rise in base level, as represented by a dam, reduced the channel slope and caused aggradation upstream to a higher elevation than that of the channel control (dam). From the field observations, he inferred that the extent of deposition is determined by valley width, channel slope, particle size of the material, and vegetation. A ratio was not established.

The deposition slopes behind the impermeable structures of the Arizona washes were compared with those of permeable structures in the upper Rio Puerco Basin of New Mexico (Lusby and Hadley 1967). The latter developed steeper slopes than the impermeable dams. Impermeable structures, placed on gentle hill slopes, consisted of wooden fenceposts and woven-wire fencing material, and were set into the ground so that 0.3 m was above the original land surface.

A general flattening of the deposition slope, as compared with the original thalweg, was also found in field investigations on 25-year-old gully control structures in Wisconsin (Woolhiser and Miller 1963). The ratio ranged between 0.29 and 1.22. Interestingly, the authors recognized the classic aggradation-degradation pattern between structures; it showed degradation and the associated flattening of the channel slope caused by a reduction in the sediment load.

Woolhiser and Lenz (1965) also demonstrated that not only the original channel gradient influences the deposition slope, but also the width of the channel at the structure, and the crest height of the spillway above the original channel bottom. These authors found an average slope ratio of 0.52. Where original slopes were less than 14 percent, the average ratio was raised to 0.66; the ratios tended to be smaller as the original slope increased.

As the above discussion demonstrated, relationships developed so far have been entirely empirical, and further research is necessary to establish the theoretical basis.

In Colorado, earth dams were examined for guidance in determining the spacing of dams (Heede 1966). Data indicated that, in gullies of less than 20 percent gradient, the dams would not interfere with sediment catch if their spacing was based on the expected slope of the deposits being 0.7 of the original gully gradient. For gully gradients exceeding 20 percent, expected sediment deposits would have a gradient of 0.5 that of the gully. Heede and Mufich (1973) developed an equation to simplify the calculation of spacing as follows:

$$S = \frac{H_E}{K\ G \cos \alpha} \tag{6}$$

where S is the spacing, H_E is effective dam height as measured from gully bottom to spillway crest, G represents the gully gradient as a ratio, α is the angle corresponding to the gully gradient ($G = \tan \alpha$), and K is a constant. The equation is based on the assumption that the gradient of the sediment deposits is $(1-K)G$. In the Colorado example, values for K were:

$$K = 0.3 \text{ for } G \leq 0.20 \tag{7}$$
$$K = 0.5 \text{ for } G > 0.20 \tag{8}$$

The generalized equation (6) can be used by the designer, after the applicable K value has been determined for the treatment area. Works older than 10 years should be inspected for this determination. Figure 20 illustrates the relationship between dam spacing, height, and gully gradient. For a given gully, the required number of dams decreases with increasing spacing or increasing effective dam height, and increases with increasing gully gradient. An example for a 600 m gully segment is given in figure 21.

Keys

Keying a check dam into the side slopes and bottom of the gully greatly enhances the stability of the structure. Such keying is important in gullies where expected peak flow is large, and where soils are highly erosive (such as soils with high sand content). Loose-rock check dams without keys were successfully installed in soils derived from Pikes Peak granite, but estimated peak flows did not exceed 0.2 m^3/s (Heede 1960).

The objective of extending the key into the gully side slopes is to prevent destructive flows of water around the dam and consequent scouring of the banks. Scouring could lead to gaps between dam and bank that would render the structure ineffective. The keys minimize the danger of scouring and tunneling around check dams because the route of seepage is considerably lengthened. As voids in the keys become plugged, the length of the seepage route increases. This in-

[6] *Heede, Burchard H. Evaluation of an early soil and water rehabilitation project—Alkali Creek watershed, Colorado. (Research Paper in preparation at Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)*

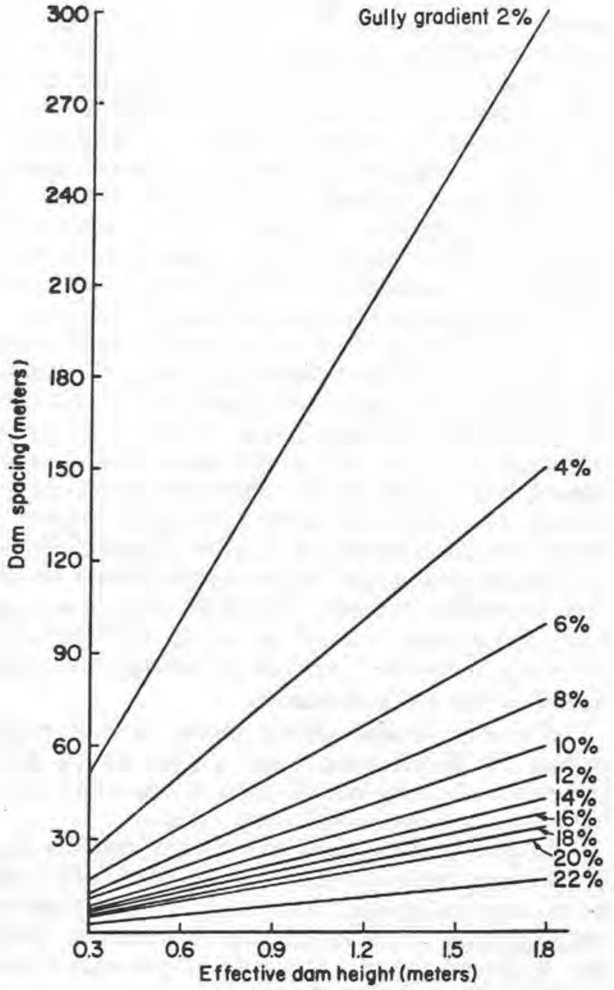


Figure 20.—Spacing of check dams, installed in gullies with different gradients, as a function of effective dam height.

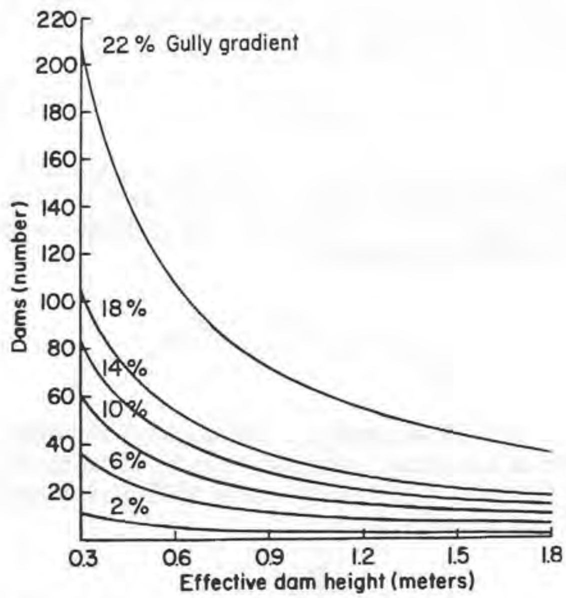


Figure 21.—Number of dams required in gullies, 600 m long and with different gradients, as a function of effective dam height.

crease causes a decrease in the flow velocity of the seepage water and, in turn, a decrease of the erosion energy.

The part of the key placed into the gully bottom is designed to safeguard the check dam against undercutting at the downstream side. Therefore, the base of the key, which constitutes the footing of the dam, must be designed to be below the surface of the apron. This is of particular importance for fence-type and impervious structures because of the greater danger of scouring at the foot of these dams. The water flowing over the spillway forms a chute that creates a main critical area of impact where the hydraulic jump strikes the gully bottom. This location is away from the structure. The sides of loose-rock and wire-bound check dams slope onto the apron, on the other hand, and no freefall of water occurs.

The design of the keys calls for a trench, usually 0.6 m deep and wide, dug across the channel. Where excessive instability is demonstrated by large amounts of loose materials on the lower part of the channel side slopes or by large cracks and fissures in the bank walls, the depth of the trench should be increased to 1.2 or 1.8 m.

Dam construction starts with the filling of the key with loose rock. Then the dam is erected on the rock fill. Rock size distribution in the key should be watched carefully. If voids in the key are large, velocities of flow within the key may lead to washouts of the bank materials. Since the rock of the keys is embedded in the trench and therefore cannot be easily moved, it is advantageous to use smaller materials, such as a mixture with 80 percent smaller than 14 cm.

Height

The effective height of a check dam (H_E) is the elevation of the crest of the spillway above the original gully bottom. The height not only influences structural spacing but also volume of sediment deposits.

Heede and Mufich (1973) developed an equation that relates the volume of sediment deposits to spacing and effective height of dam:

$$V_S = \tfrac{1}{2} H_E S \cos \alpha L_{HE} \tag{9}$$

where V_S is the sediment volume, S represents the spacing, and L_{HE} is the average length of dam, considered for effective dam height and calculated by the equation:

$$L_{HE} = L_B + \frac{L_U - L_B}{2D} H_E \tag{10}$$

where L_B is the bottom width and L_U the bank width of the gully, measured from brink to brink, and D is the depth of the gully. If S in equation (9) is substituted, then

$$V_S = \frac{H_E^2}{2KG} L_{HE} \tag{11}$$

where the constant K has the values found to be applicable to the treatment area. Equation (11) indicates that sediment deposits increase as the square of effective dam height (fig. 22).

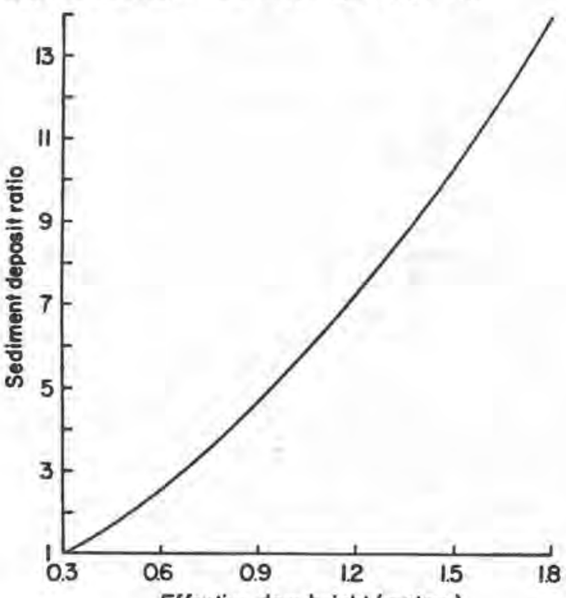


Figure 22.—Expected sediment deposits retained by check dam treatment as a function of effective dam height. The sediment deposit ratio relates the volume of sediment deposits to the volume of sediment deposits at effective dam height of 0.3 m. Thus, deposits in a treatment with 1.2 m dams are more than seven times larger than those caught by 0.3 m dams.

For practical purposes, based on the sediment deposit model, the sediment curve in figure 22 is valid for treatments in gullies with identical cross sections and gradients ranging from 1 to 30 percent. At this range, the difference is 4.5 percent with smaller deposits on the steeper gradients, a negligible fraction in such estimates. The volume of deposits, compared with that on a 1 percent gradient, decreases by 10 percent on a gradient of 45 percent, if the cross sections are constant. Magnitudes of cross sections, of course, exert strong influences on sediment deposition.

In most cases, dam height will be restricted by one or all of the following criteria: (1) costs, (2) stability, and (3) channel geometry in relation to spillway requirements. Cost relations between different types of rock check dams will be discussed later. Stability of impervious check dams should be calculated where life and/or property would be endangered by failure. Heede (1965b) presented an example for these calculations which can be easily followed. Pervious dams such as rock check dams cannot be easily analyzed for stability, however, because of unknowns such as the porosity of a structure.

Severely tested check dams in Colorado (Heede 1966) had maximum heights as follows: loose-rock and wire-bound dams, 2.2 m; and fence-type dams (thickness of 0.6 m), 1.8 m.

In gullies with small widths and depths but large magnitudes of flow, the effective height of dams may be greatly restricted by the spillway requirements. This restriction may result from the spillway depth necessary to accommodate expected debris-laden flows.

Spillway

Since spillways of rock check dams may be considered broad-crested weirs (fig. 23), the discharge equation for that type of weir is applicable:

$$Q = CLH^{3/2} \tag{12}$$

where Q = discharge in m^3/s, C = coefficient of the weir, L = effective length of the weir in m, and H = head of flow above the weir crest in m.

The value of C varies. The exact value depends on the roughness as well as the breadth and shape of the weir and the depth of flow. Since in rock check dams, breadth of weir changes within a structure from one spillway side to the other, and shape and roughness of the rocks lining the spillway also change, C would have to be determined experimentally for each dam. This, of course, is not practical and it is recommended, therefore, to

Figure 23.—Upstream view of a loose-rock check dam supporting a discharge of about 0.3 m³/s. Effective dam height is 1.7 m.

use a mean value of 1.65. This value appears reasonable in the light of other inaccuracies that are introduced in calculating the design storm and its expected peak flow. For this reason also, the discharge calculations would not be significantly improved if they were corrected for the velocity of approach above a dam. Such a correction would amount to an increase of 5 percent of the calculated discharge at a head of flow of 0.6 m over a dam 0.75 m high, or 8 percent if the flow had a 0.9 m head.

Most gullies have either trapezoidal, rectangular, or V-shaped cross sections. Heede and Mufich (1973) developed equations for the calculation of spillway dimensions for check dams placed in these gully shapes. In trapezoidal gullies, the equation for length of spillway can be adjusted to prevent the water overfall from hitting the gully side slopes, thus eliminating the need for extensive bank protection. In V-shaped gullies this is not possible, generally. In rectangular ones, adjustment of the equation is not required because the freeboard requirement prevents the water from falling directly on the banks. One equation was established, therefore, for V-shaped and rectangular gullies as follows:

$$H_{SV} = \left(\frac{Q}{CL_{AS}} \right)^{2/3} \qquad (13)$$

where H_{SV} is spillway depth, the constant C is taken as 1.65, and L_{AS}, the effective length of spillway, was derived from the equation

$$L_{AS} = \frac{L_U}{D} H_E - f \qquad (14)$$

in which f is a constant, referring to the length of the freeboard. In gullies with a depth of 1.5 m or less, the f value should not be less than 0.15 m; in gullies deeper than 1.5 m, the minimum value should be at least 0.3 m.

For structural gully control, design storms should be of 25 years magnitude, and, as a minimum, spillways should accommodate the expected peak flow from such a storm. In mountainous watersheds, however, where forests and brushlands often contribute large amounts of debris to the flow, the size and the shape of spillways should be determined by this expected organic material. As a result, required spillway sizes will be much larger than if the flow could be considered alone. Spillways designed with great lengths relative to their depths are very important here. Yet, spillway length can be extended only within limits because a sufficient contraction of the flow over the spillway is needed to form larger depths of flows to float larger loads over the crest. The obstruction of a spillway by debris is undesirable since it may cause the flow to overtop the freeboard of the check dam and lead to its destruction.

The characteristics of the sides of a spillway are also important for the release of debris over the structure. Spillways with perpendicular sides will retain debris much easier than those with sloping sides; in other words, trapezoidal cross sections are preferable to rectangular ones. A trapezoidal shape introduces another benefit by increasing the effective length of the spillway with increasing magnitudes of flow.

The length of the spillway relative to the width of the gully bottom is important for the protection of the channel and the structure. Normally, it is desirable to design spillways with a length not greater than the available gully bottom width so that the waterfall from the dam will strike the gully bottom. There, due to the stilling-basin effects of the dam apron, the turbulence of the flow is better controlled than if the water first strikes against the banks. Splashing of water against the channel side slopes should be kept at a minimum to prevent new erosion. Generally, spillway length will exceed gully bottom width in gullies with V-shaped cross sections, or where large flows of water and debris are expected relative to the available bottom width. In such cases, intensive protection of the gully side slopes below the structures is required.

Equation (13) includes a safety margin, because the effective length of the spillway is calculated with reference to the width of the gully at the elevation of the spillway bottom, instead of that at half the depth of the spillway. At spillway bottom elevation, gullies are generally narrower than at the location of the effective spillway length. This results in somewhat smaller spillway lengths, which will benefit the fit of the spillway into the dam and the gully.

If the spillway sides are sloped 1:1, it follows that in V-shaped and rectangular gullies, the bottom length of the spillway (L_{BSV}) is derived from the equation

$$L_{BSV} = L_{AS} - H_{SV} \tag{15}$$

and the length between the brinks of the spillway (L_{USV}) is given by the equation

$$L_{USV} = L_{AS} + H_{SV} \tag{16}$$

In trapezoidal gullies, the effective length of the spillway equals the bottom width of the gully. From the discharge equation for broad-crested weirs, it follows that the depth of spillway (H_S) in these gullies is given by the equation

$$H_S = \left(\frac{Q}{CL_B}\right)^{2/3} \tag{17}$$

in which the coefficient of the weir (C) is taken as 1.65.

Lengths at the bottom (L_{BS}) and between the brinks of the spillway (L_{US}) are calculated by the equations

$$L_{BS} = L_B - H_S \tag{18}$$

and

$$L_{US} = L_B + H_S \tag{19}$$

respectively.

Rock-fill dams were also designed with builtin spillways (Parkin 1963). At minimum depth of flow, the flow passes on a plane through the crest of the spillway and inclines at 45° to the vertical at the downstream side. The design does not appear to be suitable for most gully control situations due to the high sediment loads, which rapidly clog the structural voids. The detailed discussions and design equations could be helpful, however, in testing rock stability as related to specific gravity and diameter of rock as well as in estimating the void ratio. The interested reader should, however, be aware that the design criteria are for large dams, supporting discharges between 28 and 85 m^3/s, and that flow information must be available. In most situations, conservation programs have to be started to meet public demand even though adequate hydrologic data are not available. Not much has changed since this observation was made by Peterson and Hadley (1960).

Apron

Aprons must be installed on the gully bottom and protective works on the gully side slopes below the check dams, otherwise flows may easily undercut the structures from downstream and destroy them.

Apron length below a loose-rock check dam cannot be calculated without field and laboratory investigations on prototypes. Different structures may have different roughness coefficients of the dam side slope that forms a chute to the flow if tailwater depth is low. Differences in rock gradation may be mainly responsible for the different roughness values.

The design procedures for the loose-rock aprons were therefore simplified and a rule of thumb adopted: the length of the apron was taken as 1.5 times the height of the structure in channels where the gradient did not exceed 15 percent, and 1.75 times where the gradient was steeper than 15 percent. The resulting apron lengths included a sufficient margin of safety to prevent the waterfall from hitting the unprotected gully bottom. The design provided for embedding the apron into the channel floor so that its surface would be roughly level and about 0.3 m below the original bottom elevation.

In contrast, for straight-drop structures such as dams built from steel sheets or fence-type dams, apron length can be calculated if gully flows are known. In such a case, the trajectory of the nappe can be computed as follows:

$$V_O = x \sqrt{\frac{g}{2(-z)}} \tag{20}$$

in which x and z are the horizontal and vertical coordinates of a point on the trajectory referred to the midpoint of the spillway as the origin, and g is the acceleration due to gravity, taken as 9.81 m/s^2 (Howe 1950). Thus, if V_O is substituted for V_C, the critical velocity at dam crest, and z is the effective dam height, x will yield the distance from the structure at which the waterfall will hit the apron. Depending on magnitude of flow, one or several meters should be added to this distance.

The procedure for calculating critical depth and critical velocity over a check dam is as follows: The critical depth equation is

$$\frac{V_C^2}{2g} = \frac{Y_C}{2} \tag{21}$$

where V_C is the critical velocity, and Y_C is the critical depth. The continuity equation for open channel flow is

$$q = AV \tag{22}$$

where q is the flow rate of unit width of flow, A is the cross section of flow, and V represents the average velocity in the cross section. q is derived from the estimated rate of flow Q by dividing Q by L_{AS}, the effective length of spillway. Since q refers to unit width of flow, A can be replaced by Y_C and equation (22) becomes

$$V_C = \frac{q}{Y_C} \tag{23}$$

If V_C in equation (21) is replaced by (23),

$$\frac{q^2}{g} = Y_C^3 \tag{24}$$

By placing the value of Y_C, the depth of flow over the spillway, into equation (23), the critical velocity (V_C) can be obtained.

At the downstream end of the apron, a loose-rock sill should be built 0.15 m high, measured from channel bottom elevation to the crest of the sill. This end still creates a pool in which the water will cushion the impact of the waterfall.

The installation of an end sill provides another benefit for the structure. Generally, aprons are endangered by the so-called ground roller that develops where the hydraulic jump of the water hits the gully bottom. These vertical ground rollers of the flow rotate upstream, and where they strike the gully floor, scouring takes place. Thus, if the hydraulic jump is close to the apron, the ground roller may undermine the apron and destroy it (Vanoni and Pollak 1959). The end sill will shift the hydraulic jump farther downstream, and with it the dangerous ground roller. The higher the end sill, the farther downstream the jump will occur. Since data on sediment and flow are not usually available, a uniform height of sill may be used for all structures.

Ephemeral gullies carry frequent flows of small magnitudes. Therefore, it is advisable not to raise the crest of the end sills more than 0.15 to 0.25 m above the gully bottom. End sills, if not submerged by the water, are dams and create waterfalls that may scour the ground below the sill. At higher flows, some tailwater usually exists below a sill and cushions to some extent the impact from the waterfall over the sill.

Where the downstream nature of the gully is such that appreciable depth of tailwater is expected, the installation of end sills is not critically important. The hydraulic jump will strike the water surface and ground rollers will be weak.

Bank Protection

Investigations have shown (Heede 1960) that check dams may be destroyed if flows scour the gully side slopes below the structures and produce a gap between the dam and the bank. Since water below a check dam is turbulent, eddies develop that flow upstream along each gully side slope. These eddies are the cutting forces.

Several types of material are suitable for bank protection. Loose rock is effective, but should be reinforced with wire-mesh fence, secured to steel posts, on all slopes steeper than 1.25 to 1.00 (see fig. 17). The design should provide for excavation of the side slopes to a depth of about 0.3 m so that the rock can be placed flush with the surrounding side slope surface to increase stability of the protection. Excavation of surface materials also assures that the rock would not be set on vegetation. Banks should be protected for the full length of the apron.

The height of the bank protection depends on the characteristics of channel, flow, and structure. Where gullies have wide bottoms and spillways are designed to shed the water only on the channel floor, the height should equal total dam height at the structure, but can rapidly decrease with distance from the structure. In contrast, where the waterfall from a check dam will strike against the gully banks, the height of the bank protection should not decrease with distance from dam to prevent the water from splashing against unprotected banks.

In gullies with V-shaped cross sections, the height of the bank protection should be equal to the elevation of the upper edges of the freeboards of the dam. In general, the height of the bank protection can decrease with increasing distance from the dam.

Equations for Volume Calculations

After the dam locations have been determined in the field, based on spacing requirements and suitability of the site for a dam, gully cross sections at these locations should be surveyed and plotted. If possible, use the computer program developed by Heede and Mufich (1974) to design the dams. Otherwise the dams must be designed from the plotted gully cross sections. Structural and gully dimensions can be used in equations developed by the above authors.

Loose-Rock and Wire-Bound Dams

The volume equation for the dam proper of loose-rock and wire-bound dams considers either angular or round rock, because the angle of repose varies with rock shape and influences the side slopes of the dam. The generalized equation is

$$V_{LR} = \left(\frac{H_D^2}{\tan A_R} + 0.6\, H_D \right) L_A - V_{SP} \qquad (25)$$

where V_{LR} is the volume of the dam proper, H_D represents dam height, 0.6 is a constant that refers to the breadth of dam, L_A is the average length of the dam, $\tan A_R$ is the tangent of the angle of repose of the rock type, and V_{SP} is the volume of the spillway. It is assumed that the angle of repose for angular rock is represented by a slope of 1.25:1.00, corresponding to a tangent of 0.8002; for round rock, the slope is 1.50:1.00 with a tangent of 0.6590. L_A is given by the equation

$$L_A = L_B + \frac{L_U - L_B}{2D} H_D \qquad (26)$$

where L_B is the length of dam at the bottom, L_U represents the length of dam measured at freeboard elevation, and D is the depth of the gully. V_{SP} is calculated by the equation

$$V_{SP} = H_S L_{AS} B_A \qquad (27)$$

where H_S is the depth and L_{AS} is the effective length of the spillway; B_A is the breadth of the dam, measured at half the depth of the spillway and derived from the equation

$$B_A = \frac{H_S}{\tan A_R} + 0.6 \qquad (28)$$

Angular rock is preferable to round rock because less is required, and it enhances dam stability.

The equation for volumes of loose-rock and wire-bound loose-rock dams (eq. 25) was simplified by assuming a zero gully gradient. This assumption results in an underestimate of volumes in gullies with steep gradients. To offset this underestimate on gradients larger than 15 percent, 10 percent should be added to the calculated volume.

If the design peak flow is larger than 0.3 m^3/s, all types of check dams must be keyed into the gully banks and bottom. Under varied conditions in Colorado, it was found that a bottom key of 0.6 m depth and width was sufficient for check dams up to 2 m high. A width of 0.6 m was also adequate for the bank keys. The depth of the keys, however, must be adjusted according to characteristics of the soils. Thus, the equation for the volume of the key is generalized as follows:

$$V_K = (L_A + 2R)(0.6H_D + 0.36) - 0.6H_D L_A \qquad (29)$$

where R represents the depth of key and 0.6 m and 0.36 m^2 are constants, referring to depth and width of bottom key and width of bank key, respectively.

In the construction plan, the volume V_K should be kept separate from that of the dam proper because, generally, a finer rock gradation is required for the keys.

Apron and bank protection below the structure are always required at check dams. The equation developed for the volume calculations is:

$$V_A = \left(cH_D L_B + dH_D^2 \right) 0.3 \qquad (30)$$

in which V_A is the rock volume of the apron and bank protection, and c and d are constants whose values depend on gully gradient. For gradients $\leq$ 15 percent, c = 1.5 and d = 3.0; for gradients > 15 percent, c = 1.75 and d = 3.5.

The total rock volume required for a loose-rock dam with keys is the sum of equations (25), (29), and (30).

Besides rock, wire mesh and steel fenceposts are used in most of the dams. If dam height is equal to or larger than 1.2 m, reinforcement of the bank protection work by wire mesh and fenceposts will generally be required. The equation for amount of wire mesh and number of posts includes a margin of safety to offset unforeseen additional needs. To assist in construction, dimensions of the mesh are given in length and width. The length measured along the thalweg is

$$M_{LB} = 3.50\, H_D \qquad (31)$$

where M_{LB} is the length of the wire mesh for the bank protection, and 3.50 is a constant. The width of the wire mesh, measured from the apron to the top of the bank protection at the dam, equals the total dam height.

The number of fenceposts is calculated by the equation

$$N_B = 2\left[\left(\frac{1.75 H_D}{1.2}\right) + 1\right] \qquad (32)$$

where N_B is the number of fenceposts for the bank protection, rounded up to a whole number divisible by 4, and 2 is for the two banks, 1.75 H_D is the length of bank protection for gradients >15 percent, 1.2 is for the 1.2 m spacing of the posts, and 1 is for an end post. Of the total number of posts, half should be 0.75 m taller than the dam; the other half are of dam height.

For wire-bound dams, the length of the wire mesh is taken as that of the dam crest, which includes a safety margin and is calculated by the equation

$$M_L = L_B + \frac{L_U - L_B}{D} H_D \qquad (33)$$

where M_L is the length of the wire mesh. The width of the mesh, measured parallel to the thalweg, depends not only on dam height but also on rock shape. The equation for the width of the wire mesh is

$$M_w = 2\left(\frac{H_D}{\tan A_R} + \frac{H_D}{\sin A_R} + 0.6\right) + 1.6 \qquad (34)$$

where M_w is the width and A_R the angle of repose of the rock. For angular rock, this angle is assumed to be 38° 40′, corresponding to a dam side slope of 1.25:1.00, and for round rocks 33° 25′, representing a slope of 1.50:1.00. The terms 0.6, 1.8 and 2 are constants. Equation (34) provides for an overlapping of the mesh by 1.8 m.

Single-Fence Dams

A zero gully gradient was assumed for calculating rock volume for the dam proper of single-fence dams. This results in overestimates that compensate for simplification of the equation for volume calculation. If the construction plan calls for a dam with a 0.6 m breadth, for ease of calculation, the cross section of the dam parallel to the thalweg is taken as a right triangle with a dam side slope of 1.25:1.00 in the equation:

$$V_{SF} = \frac{H_D^2}{2(0.80020)} L_A - V_{SSF} \qquad (35)$$

where V_{SF} is the rock volume of the dam proper, 2 is constant, and 0.80020 represents the tangent of a slope of 1.25:1.00. V_{SSF} is the volume of the spillway, calculated by the equation

$$V_{SSF} = H_S L_{AS} B_{SF} \qquad (36)$$

where B_{SF} is the breadth of the dam, measured at half the depth of the spillway and given by the equation

$$B_{SF} = \frac{H_S}{2(0.80020)} \qquad (37)$$

The length of wire mesh for a single-fence dam is given by equation (33), while the width equals dam height. The number of fenceposts is calculated by the equation

$$N_{SF} = \frac{L_B}{1.2} + \frac{L_U - L_B}{1.2D} H_D + 1 \qquad (38)$$

where N_{SF} is the number of posts of the dam proper of a single-fence dam, rounded up to a whole number; 1.2 signifies a distance of 1.2 m between the posts; and 1 is a constant. Of the total number of posts, half are 0.75 m taller than the dam; the other half are dam height.

Double-Fence Dams

The equation for rock volume of a double-fence dam with vertical fences, 0.6 m apart, is:

$$V_{DF} = 0.6H_DL_A - V_{SDF} \qquad (39)$$

where V_{DF} is the volume, 0.6 is a constant, and V_{SDF} is the volume of the spillway, computed by the equation

$$V_{SDF} = 0.6H_SL_{AS} \qquad (40)$$

where 0.6 represents the standard breadth of the dam, in meters.

The length of wire mesh is given by

$$M_{LD} = 2L_B + \frac{L_U - L_B}{D}2H_D \qquad (41)$$

where M_{LD} is the length of the mesh. The width of the wire mesh equals dam height. The number of fenceposts is computed by the equation

$$N_{DF}=2\left(\frac{1}{1.2}\left[L_B+\frac{L_U-L_B}{D}H_D\right]+1\right) \qquad (42)$$

where N_{DF} is the number of posts of the dam proper of a double-fence dam, rounded up to a whole even number, and 1, 1.2 and 2 are constants. The equation is based on a post spacing of 1.2 m. Half of the posts are dam height, while the other half are 0.75 m taller than the dam.

Headcut Control

The volume requirements for a headcut control structure are given by the equation

$$V_{HC}=\left(\frac{3D^2}{2}\right)\left(\frac{L_U+3L_B}{4}\right) \qquad (43)$$

where V_{HC} is the rock volume, D is the depth of the gully at the headcut, and 3 is a coefficient that refers to a structure with a slope gradient of 3:1. If a slope gradient different from 3:1 is selected, the value of the coefficient in the equation should be changed to correspond to that gradient.

Rock Volume Relations Among Dam Types

In the Colorado project (Heede and Mufich 1973), rock volumes required for the various types of check dams were expressed graphically (fig. 24). If this graph is used for decisionmaking, it must be recognized that double-fence dams had parallel faces 0.6 m apart. Where double-fence structures with bases wider than the breadth of dam are required, rock volume requirements will be larger. The graph shows that a loose-rock or wire-bound dam with effective height of 1.8 m requires about 3 times more rock than a double-fence dam.

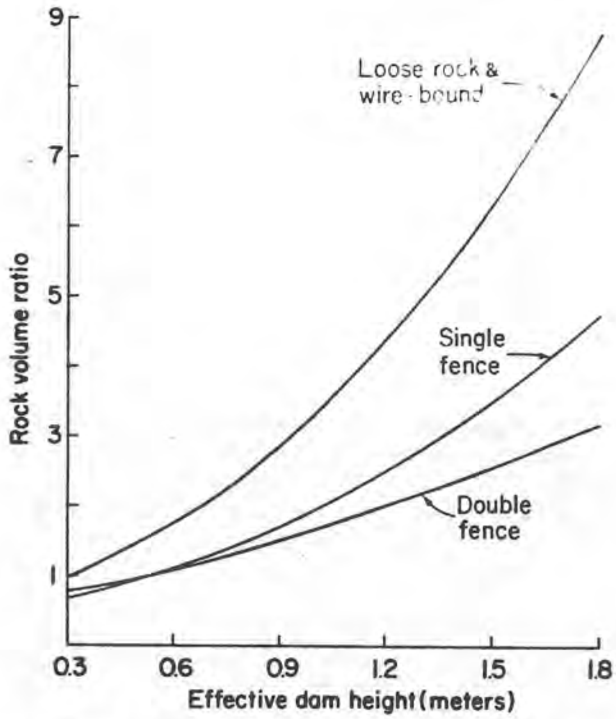


Figure _4.—Required volumes of angular rock for four different dam types as a function of effective dam height. The rock volume ratio relates the rock volume to that of a loose-rock dam 0.3 m high.

Construction Procedures

Before construction starts, the following design features should be staked and flagged conspicuously:

1. Mark the centerline of the dam and the key trenches, respectively, on each bank. Set the stakes away from the gully edge to protect them during construction.

2. Designate the crest of the spillway by a temporary bench-mark in the gully side slope sufficiently close to be of value for the installation of the dam.

3. Mark the downstream end of the apron.

4. For loose-rock and wire-bound dams, flag the upstream and downstream toes of the dam proper.

Caution is required during excavation to avoid destroying the stakes before the main work of installation begins.

The construction of all dams should start with the excavation for the structural key (fig. 25), the apron, and the bank protection. This very important work can be performed by a backhoe or hand labor. Vegetation and loose material should be cleaned from the site at the same time.

The trenches for the structural keys will usually have a width of 0.6 m, therefore a 0.5-m-wide bucket can be used on the backhoe. If the construction plan calls for motorized equipment, two types of backhoes can be used. One, mounted on a rubber-wheeled vehicle and operating from a turntable, permits the backhoe to rotate 360°. This machine travels rapidly between locations where the ground surfaces are not rough, and works very efficiently in gullies whose side slopes and bottoms can be excavated from one or both channel banks. The other type can be attached to a crawler tractor. This type proves to be advantageous at gullies that are difficult to reach, and with widths and depths so large that the backhoe has to descend into the channel to excavate. In deep gullies with V-shaped cross sections, temporary benches on the side slopes may be necessary. Often, the bench can be constructed by a tractor with blade before the backhoe arrives.

The excavated material should be placed upstream from the dam site in the gully. The excavated trench and apron should then be filled with rock. Since a special graded rock is required for the keys, rock piles for keys must be separate from those used in the apron and dam proper. Excavations can be filled by dumping from a dump truck or by hand labor. During dumping operations, the fill must be checked for voids, which should be eliminated.

If dump trucks are loaded by a bucket loader, some soil may be scooped up along with the rock. Soil is undesirable in a rock structure because of the danger of washouts. To avoid soil additions, use a bucket with a grilled bottom that can be shaken before the truck is loaded. Other devices such as a grilled loading chute would also be appropriate.

Dumping rock into the dam proper has two advantages: The structure will attain greater density, and rocks will be closer to their angle of repose than if placed by hand. Hand labor can never be completely avoided, however, since plugging larger voids and the final dam shape require hand placement. Where gullies are deep and dumping is impractical, rock chutes may be used.

Often, gully control projects are planned to provide employment for numbers of people. This objective can easily be accomplished if sufficient supervision is available for the individual steps in the construction. Special attention is needed at the spillway and freeboard. In loose-rock and wire-bound structures, where the shape of the dam is not outlined by a fence as in the other types, experience shows there is a tendency to construct the spillways smaller than designed.

Figure 25.—The key for a rock check dam is efficiently excavated with a backhoe.

In wire-bound dams, a commercial, galvanized stock fence, usually about 1.2 m wide, can be used. The stay and line wires should not be less than 12½-gage low-carbon steel, the top and bottom wires 10-gage low-carbon steel, and the openings in the mesh 0.15 m. To connect ends of the fence or to attach the fence to steel posts, a galvanized 12½-gage coil wire is sufficiently strong.

The wire mesh of required length and width should be placed over the gully bottom and side slopes after the trench and apron have been filled with rock (fig. 26). Generally, several widths of mesh will be needed to cover the surface from bank to bank. If several widths are required, they should be wired together with coil wire where they will be covered with rocks. The parts not to be covered should be left unattached to facilitate the fence-stringing operations around the structure.

Before the rock is placed on the wire mesh for the installation of the dam proper, the mesh should be temporarily attached to the gully banks. Otherwise, the wire mesh lying on the gully side slopes will be pushed into the gully bottom by the falling rock and buried. Usually, stakes are used to hold the wire mesh on the banks.

After the dam proper is placed and shaped, the fence can be bound around the structure. Fence stretchers should be applied to pull the upstream ends of the fence material down tightly over the downstream ends, where they will be fastened together with coil wire. Then the bank protection below the dam should be installed.

The installation of single- and double-fence dams begins with the construction of the fences after excavation is completed (fig. 27). Construc-

Figure 27.—Parallel fences for double-fence dam (see fig. 11) are being installed. Note the excavations for key, apron, and bank protection (the latter two to right of the structure).

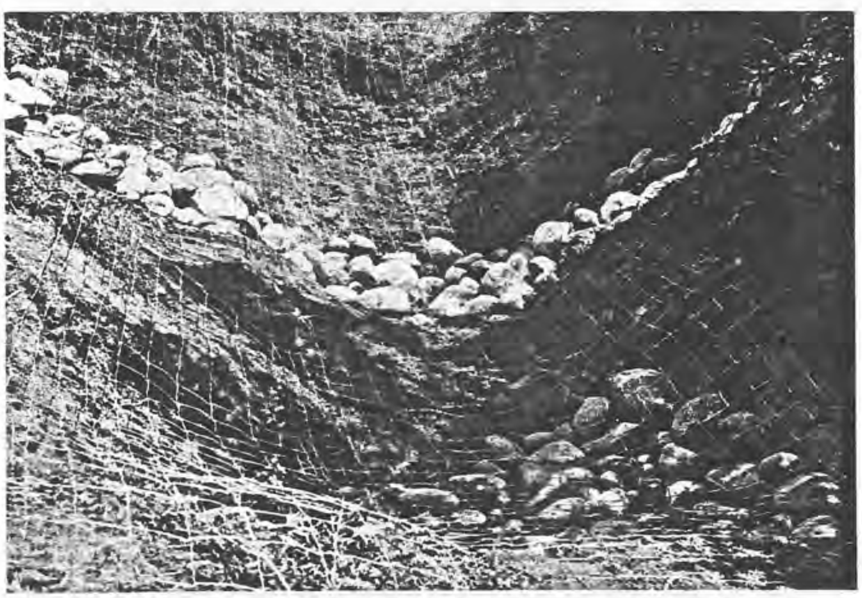


Figure 26.—Upstream view of a construction site for a wire-bound dam. Note that key and apron excavations were filled with rock before wire mesh was placed on the bed banks. The site is prepared for the construction of the dam proper.

tion drawings should be followed closely here, because the final shape of the dams will be determined by the fences. Conventional steel fence-posts can be used. In some locations, the great height of posts may offer difficulties for the operator of the driving equipment, and scaffolds should be improvised. A pneumatically driven pavement breaker with an attachment designed by Heede (1964) can be used to ease the job of driving. Since relatively great lengths of hose may be attached, this tool may be used in deep gullies and on sites with difficult access.

At single-fence dams, dumping of rock is practical if the gully is not excessively deep or wide. At double-fence structures, hand labor, or a backhoe or clamshell (fig. 28) will be required. The rock should be placed in layers and each layer inspected for large voids, which should be closed manually by rearranging rocks.

Much time and effort can be saved during construction if a realistic equipment plan is established beforehand. Such a plan requires an intimate knowledge of the cross-sectional dimensions of the gullies and their accessibility to motorized equipment. Pioneer roads that might be needed because of lack of access are not only important for equipment considerations, but will also enter into the cost of the construction.

If equipment is to be used, as a general rule, it appears to be advantageous to use heavier and larger machines if their mobility is adequate. Although hourly costs for heavier machines are usually greater, the total cost for a job is reduced.

With few exceptions, conventional construction equipment is not sufficiently mobile to operate in rough topography without pioneer roads. In watershed rehabilitation projects such as gully control, road construction is undesirable because it disturbs the ground surface and may lead to new erosion. It is therefore desirable to consider crawler-type equipment only.

Cost Relations

Relationships between the installation costs of the four different types of rock check dams described here are based on research in Colorado (Heede 1966). The relationships are expressed by ratios (fig. 29) to avoid specific dollar comparisons. When considering the cost ratio, one must keep in mind that differential inflation may have offset some finer differences in cost. It is advisable, therefore, to test the cost of individual structures by using material and volume requirements as given by the equations. The cost ratios in figure 29 can then be adjusted, if necessary.

In a given gully, for example, a double-fence dam with an effective height of 1.8 m costs only about four times as much as a 0.3 m loose-rock dam, while a wire-bound dam 1.8 m high costs 8.5 times as much. Costs will change with different sizes and gradients of gullies, but the general relationships will not change.

It is obvious that the cost of installing a complete gully treatment increases with gully gradient because the required number of dams increases. Figure 30 indicates there is one effective dam height at which the cost is lowest. In the sample gully, this optimum height for loose-rock dams is about 0.6 m, for single-fence dams 0.7 m, and for double-fence dams 1.1 m. A con-

Figure 28.—Using clamshell to place rock into a double-fence dam. The man steadies the clamshell with a long rope.

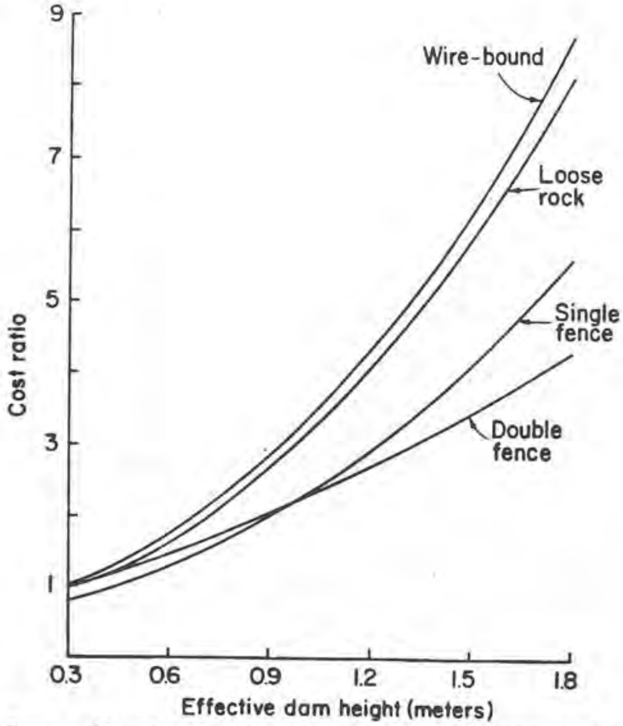


Figure 29.—Installation cost of four different types of check dams as a function of effective dam height. The cost ratio is the cost of a dam related to the cost of a loose-rock dam, 0.3-m-high, built with angular rock.

stant gully cross section was assumed. In reality, of course, gully cross sections usually change between dam sites. The optimum height for lowest treatment costs is not a constant, but changes between gullies, depending on shape and magnitude of the gully cross sections at the dam sites.

Since the cost of the dam is directly proportional to the rock volume, figure 30 also expresses the relationship between rock requirement and effective dam height. This means that, in a given gully, there is one dam height at which rock requirements for a treatment are smallest.

A treatment cannot be evaluated on the basis of cost of installation alone, because recognition of benefits is part of the decisionmaking process. Sediment deposits retained by check dams can be incorporated into a cost ratio that brings one tangible benefit into perspective. Sediment has been cited as the nation's most serious pollutant (Allen and Welch 1971). The sediment-cost ratio increases (treatment is increasingly beneficial) with dam height and decreases with increasing gradient (fig. 31). The example in figure 31 shows that a treatment consisting of loose-rock dams on a 2 percent gradient has a sediment-cost ratio larger than 1.0 for effective dam heights of 0.75 m and above.

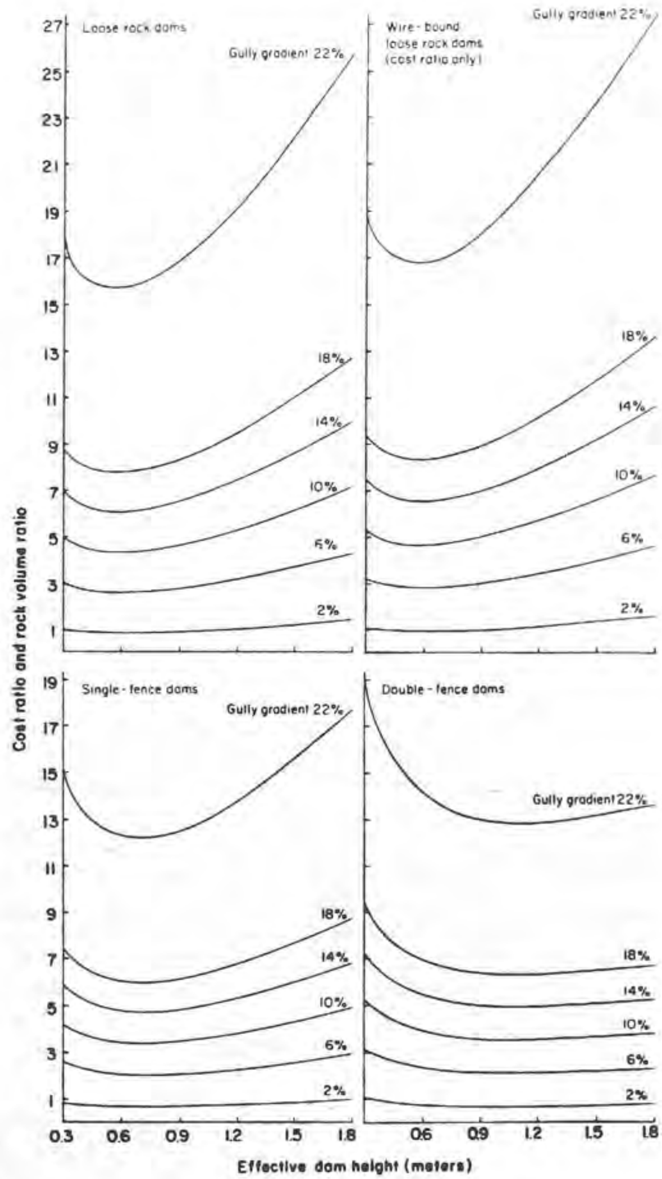
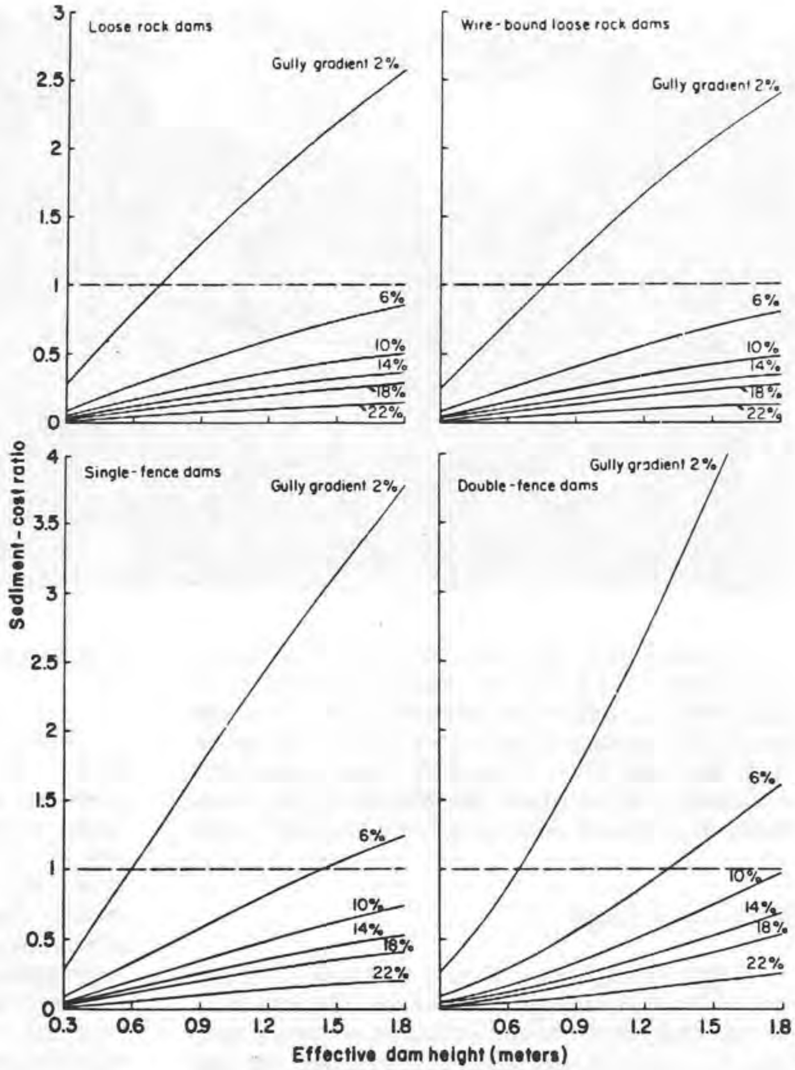


Figure 30.—Relative cost of installation of check-dam treatments and relative angular rock volume requirements in gullies with different gradients as a function of effective dam height. The cost and rock volume ratios relate the cost and rock volume of a treatment to those of a treatment with loose-rock dams 0.3 m high installed on a 2 percent gradient.

The large ratio is explained by the fact that a gully with a 2 percent gradient requires only a small number of dams (see fig. 21), while volumes of sediment deposits do not decrease significantly with number of dams or with gradient.

Since single-fence and double-fence dams cost less than loose-rock and wire-bound loose-rock dams for an effective height greater than 0.3 m, the sediment-cost ratio is more favorable for the

Figure 31.—The sediment-cost ratio relates the value of the expected sediment deposits to the cost of treatment. The graphs show this ratio as a function of effective dam height on gully gradients ranging from 2 to 22 percent. The base cost was taken as \$20/m³ of angular rock dam; the value of 1 m³ of sediment deposits was assumed to be one-tenth of that cost.



fence-type structures. The ratios remain smaller than 1.0 on all gradients larger than 5 percent for treatments with loose-rock and wire-bound loose-rock dams, and on gradients larger than 7 and 9 percent for treatments with single-fence and double-fence dams, respectively.

The importance of sediment-cost ratios in relation to gully gradient and effective dam height becomes apparent in situations where not all gullies of a watershed can be treated. Gullies with the smallest gradient and largest depth, and highest possible fence-type dams should be chosen if other aspects such as access or esthetic value are not dominant.

Other Gully Control Structures and Systems

Nonporous Check Dams

Rock can be used for the construction of wet masonry dams. Limitations in available masonry skills, however, may not permit this approach. A prefabricated concrete dam was designed (Heede 1965b) and a prototype installed in Colorado (see fig. 10). It required very little time and no special skills for installation (fig. 32). The capital investment for this dam is larger than for a rock structure, however. A prestressed concrete manufacturer must be available reasonably close to the project area, and the construction sites must be accessible to motorized equipment. Where esthetic considerations and land values are high—recreational sites and parks, for example —a prestressed, prefabricated concrete check dam may be the answer.

Many different designs of concrete dams for torrent control were published in recent years. Some references are: Fattorelli (1970, 1971), Puglisi (1970), Benini et al. (1972), IUFRO-Working Group on Torrents, Snow and Avalanches (1973). Nearly all torrent dams would be overdesigned if installed in western gullies, however.

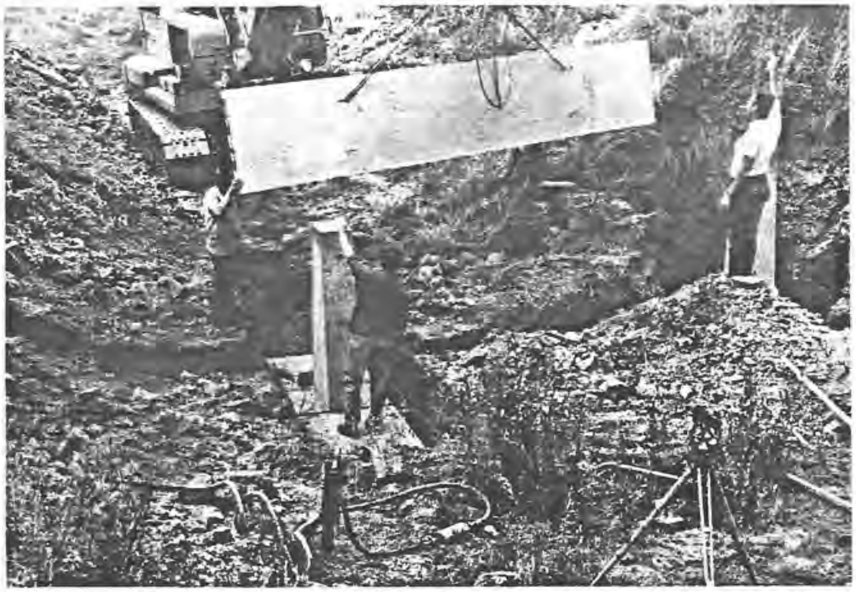


Figure 32.—Placement of a prestressed concrete slab against the buttresses of a prefabricated dam on Alkali Creek watershed. Backhoe proved to be sufficient for excavation of key and for structural installation. View is downstream.

Check dams may also be built from corrugated sheet steel. For successful application, a pile driver is required to assure proper fit of the sheets. Excavating trenches for the sheets jeopardizes dam stability if the refill is not compacted sufficiently. Quite often, insufficient depth of soil above the bedrock does not permit this dam type.

Earth Check Dams

Earth check dams should be used for gully control only in exceptional cases. Basically, it was the failure of the construction material, soil, that—in combination with concentrated surface runoff—caused the gully. Gullies with very little flow may be an exception if the emergency spillway safely releases the flow onto the land outside the gully. The released flow should not concentrate, but should spread out on an area stabilized by an effective vegetation cover or by some other type of protection such as a gravel field. Most gullied watersheds do not support areas for safe water discharge.

Standpipes or culverts in earth check dams generally create problems, because of the danger of clogging the pipe or culvert inlet, and the difficulty in estimating peak flows. Therefore, additional spillways are required.

If soil is the only dam material available, additional watershed restoration measures (such as vegetation cover improvement work and contour trenches) should be installed to improve soil infiltration rates, to enhance water retention and storage, and thus decrease magnitude and peak of gully flows.

Vegetation-Lined Waterways

With the exception of earth check dams, gully control measures described previously treat the flow where it is—in the gully. In contrast, treatments by waterways take the water out of the gully by changing the topography (figs. 33, 34). Check dams and waterways both modify the regimen of the flow by decreasing the erosive forces of the flow to a level that permits vegetation to grow. In waterways, however, flow is modified compared with the original gully, in two ways (Heede 1968a): (1) Lengthening the watercourse results in a gentler bed gradient; and (2) widening the cross section of flow provides very gentle channel side slopes. This latter measure leads to shallow flows with a large wetted perimeter (increase in roughness parameter). Both measures substantially decrease flow velocities, which in turn decrease the erosive forces.

Contrasted with check dam control, waterway projects strive to establish a vegetation cover when land reshaping is finished. Indeed, a quick establishment of an effective vegetation lining is the key to successful waterways. It follows that the prime requisites for a successful application are precipitation, temperature, and fertility of soils, all favorable to plant growth. Other requisites are:

1. Size of gully should not be larger than the available fill volumes;
2. Width of valley bottom must be sufficient for the placement of a waterway with greater length than that of the gully;

Figure 33.—Looking upstream on gully No. 6 of Alkali Creek watershed before treatment, November 14, 1961. Mean gully depth was 0.9 m and mean width from bank to bank 4.0 m.

Figure 34.—Repeat photograph of figure 33 taken on September 2, 1964, three growing seasons after conversion of gully to vegetation-lined waterway. The annual pioneer cover, consisting mainly of ryegrass (*Lolium* sp., annual variety) has been replaced by perennial herbaceous plants — smooth brome (*Bromus inermis*) and intermediate wheatgrass (*Agropyron intermedium*) are the main species.

3. Depth of soil mantle adequate to permit shaping of the topography; and
4. Depth of topsoil sufficient to permit later spreading on all disturbed areas (fig. 35).

Design criteria or prerequisites in terms of hydraulic geometry are not yet available, but the literature discussed below is relevant.

Few studies are available on flow in vegetation-lined channels or waterways. The investigation by Ree and Palmer (1949) may be a classic. They planted grasses that are widespread in the southeastern and southcentral States. Outdoor test channels and flumes were located in the Piedmont plateau, South Carolina. Permissible velocities (threshold values before beginning of erosion) were established. The study gained valuable insight into the change of the roughness parameter (n) with the growth of the grasses. The species they used do not normally grow in the West, however.

Parsons (1963), basing his work on that of Ree and Palmer (1949), established equivalent stone sizes for Bermudagrass streambank linings by relating the allowable shear stress on the grass lining to equivalent stone diameter. Useful guiding principles for successful application of vegetation for stream bank erosion control were given.

Kouwen et al. (1969) avoided the Ree and Palmer (1949) method of empirically representing the functional relationships between Manning's (n) and the relevant flow parameters. Instead,

Figure 35.—Topsoil is removed from the construction area and saved, to be spread later on the finished waterway.

they derived a quasi-theoretical equation for flow and vegetation condition in a channel as follows:

$$\frac{V}{u_*} = C_1 + C_2 \ln\left(\frac{A}{A_V}\right) \qquad (44)$$

where V is the mean velocity of flow, and u_* is the shear velocity defined as $(gR_1S_1)^{1/2}$ (g represents acceleration due to gravity, R_1 is the hydraulic radius, and S_1 is the energy gradient). C_1 is a parameter that depends on the density of the vegetation, while C_2 is a parameter that depends on the stiffness of the vegetation. A is the area of the channel cross section; A_V represents the area of the vegetated part of the cross section.

The investigators could not establish design curves or tables, thus practical application is not yet feasible.

Vegetation-lined waterways require exact construction and therefore close construction supervision (fig. 36), and frequent inspections during the first treatment years. The risk, inherent to nearly all types of erosion control work, is greater for waterways at the beginning of treatment than for check dam systems. To offset this risk, in Colorado 19 percent of the original cost

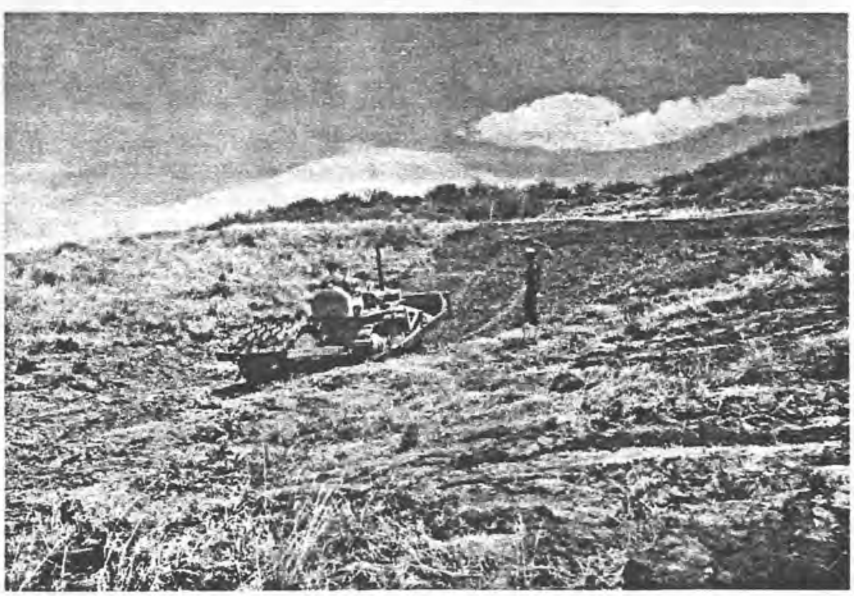


Figure 36.—A sheep-foot roller pulled by a small tractor compacts the fill in the gully. Fill was placed in layers 0.15 to 0.30 m thick.

of installation was expended for maintenance, while for the same period of time, only 4 percent was required at check dams (Heede 1968b).

Eight percent less funds were expended per linear meter of gully for construction and maintenance of grassed waterways than for check dams. This cost difference is not significant, especially if the greater involvement in waterway maintenance is recognized. In deciding on the type of gully control, one should consider not only construction costs but also risk of and prerequisites for vegetation-lined waterways.

Summary of Design Criteria and Recommendations

Spacing decreases with increasing gully gradient and increases with effective dam height (see fig. 20). Number of check dams increases with gully gradient and decreases with increasing effective dam height (see fig. 21). Expected volumes of sediment deposits increases with effective height (see fig. 22).

For practical purposes, gully gradients ranging from 1 to 30 percent do not influence volumes of sediment deposits in a treatment. On gradients larger than 30 percent, sediment catch decreases more distinctly with increasing gradient.

Rock volume requirements are much larger for loose-rock and wire-bound loose-rock dams than for fence-type dams. At effective dam heights larger than 0.6 m, treatments with double-fence dams require smallest amounts of rock (see fig. 24).

At effective dam heights larger than about 0.5 m, loose-rock and wire-bound loose-rock dams are more expensive than fence-type dams. The difference in cost increases with height (see fig. 29). Single-fence dams are less expensive than double-fence dams at effective heights up to 1.0 m.

Regardless of gradient, in a given gully, there is one effective dam height for each type of structure at which the cost of treatment is lowest (see fig. 30). For each type of treatment, rock requirements are smallest at the optimum effective dam heights for least costs (see fig. 30). The sediment-cost ratio (the value of expected sediment deposits divided by the cost of treatment) increases with effective dam height and decreases with increasing gully gradient (see fig. 31). At effective dam heights of about 0.6 m and larger, single-fence dams have a more pronounced beneficial sediment-cost ratio than loose-rock or wire-bound loose-rock dams. At effective dam heights of 1.1 m and larger, treatments with double-fence dams have the largest sediment-cost ratios (see fig. 31).

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SYMBOLS

α = angle corresponding to the gully gradient.
A = area of the channel cross section.
A_R = angle of repose of rock.
A_V = area of the vegetated part of the cross section.
B_A = breadth of loose rock or wire-bound loose-rock dams, measured at one-half of the depth of the spillway.
B_{SF} = breadth of single-fence dams, measured at one-half of the depth of the spillway.
C = discharge coefficient, taken at 1.65.
C^1 = constant whose value depends on the watershed configuration.
C_1 = parameter depending on density of vegetation.
C_2 = parameter depending on stiffness of vegetation.
c = constant whose value changes with groups of gully gradients.
c_1 = constant in Huxley's growth law.
D = depth of gully.
D_{65} = sieve size which allows 65 percent of rocks to pass through.
d = constant whose value changes with groups of gully gradients.
d^1 = constant in Huxley's growth law.
E = advancement rate of the gully.
f = constant whose value changes with groups of gully gradients.
G = gully gradient in percent.
g = acceleration due to gravity, taken as $9.81\ m/s^2$.
γ = specific weight of the fluid.
H = head of flow above weir crest.
H_D = total height of dam.
H_E = effective height of dam, the elevation of the crest of the spillway above the original gully bottom.
H_S = depth of spillway of a dam installed in a rectangular or trapezoidal gully.
H_{SV} = depth of spillway for a dam installed in a V-shaped gully.
K = constant, referring to the expected sediment gradient.
L = effective length of the weir.
L_A = average length of dam.
L_{AS} = effective length of spillway.
L_B = bottom width of the gully.
L_{BS} = bottom length of the spillway of a dam installed in a rectangular or trapezoidal gully.
L_{BSV} = bottom length of the spillway of a dam installed in a V-shaped gully.
L_{HE} = average length of dam.
L_U = width of the gully between the gully brinks.
L_{US} = length between the brinks of the spillway of a dam installed in a rectangular or trapezoidal gully.
L_{USV} = length between the brinks of the spillway of a dam installed in a V-shaped gully.
M_L = length of the wire mesh of a wire-bound dam.
M_{LB} = length of the wire mesh of the bank protection, measured parallel to the thalweg.
M_{LD} = length of the wire mesh for a double-fence dam.
M_W = width of the wire mesh of a wire-bound dam, measured parallel to the thalweg.
N_B = number of fenceposts of the bank protection work.
N_{DF} = number of fenceposts of the dam proper of a double-fence dam.
N_{SF} = number of fenceposts of the dam proper of a single-fence dam.
n = Manning's roughness coefficient.
P = wetted perimeter.
Q = rate of the peak flow in m^3/s, based on the design storm.
q = rate of the peak flow in m^3/s per unit width of spillway.
u_* = shear velocity $[(g\ R_1 S_1)^{1/2}]$.
R = constant, representing the depth of key.
R_1 = hydraulic radius.
ω = stream power per unit length of gully.
S = spacing of check dams.
S_1 = energy gradient.
τ = tractive force.
V = mean stream velocity.
V_A = volume of rock for the apron and bank protection.
V_c = critical velocity at dam crest.
V_{HC} = volume of a headcut control structure.
V_{DF} = volume of the dam proper of a double-fence dam.
V_K = volume of the key.
V_{LR} = volume of the dam proper of a loose-rock dam.
V_O = approach velocity of flow.
V_S = volume of sediment deposits above check dams.
V_{SF} = volume of the dam proper of a single-fence dam.
V_{SP} = volume of the spillway of loose rock and wire-bound loose-rock dams.

V_{SDF} = volume of the spillway of a double-fence dam.
V_{SSF} = volume of the spillway of a single-fence dam.
W = weight of rock related to D_{65}.
w = flow width.
X = size of a biological organ.
x = horizontal coordinate of a point on the trajectory, here the horizontal distance between the downstream side of the spillway and the point where the waterfall hits the apron.
y = size of an organism.
Y_c = critical depth of flow at dam crest.
z = vertical coordinate of a point on the trajectory, here the effective dam height.

Heede, Burchard H.
1976. Gully development and control: The status of our knowledge. USDA For. Serv. Res. Pap. RM-169, 42 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. 80521

Gully formation is discussed in terms of mechanics, processes, morphology, and growth models. Design of gully controls should draw on our understanding of these aspects. Establishment of an effective vegetation cover is the long-term objective. Structures are often required. The least expensive, simply built structures are loose-rock check dams, usually constructed with single- or double-wire fences. Prefabricated concrete dams are also effective. Functional relationships between dams, sediment catch, and costs, as well as a critical review of construction procedures, should aid the land manager in design and installation of gully treatments.

Keywords: Gullies, Erosion, geomorphology, erosion control, dams.

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