

SYMPOSIUM 2.--EROSION AND SEDIMENT CONTROL

Soil Erosion Concepts and Misconceptions--	
L. D. Meyer, D. G. DeCoursey, and M. J. M. Romkens.....	2-1
Comparative Costs of Erosion and Sedimentation	
Control Measures--	
K. H. Ateshian.....	2-13
Soil Ripping Treatments for Runoff and Erosion Control--	
Earl F. Aldon.....	2-24
Soil and Water Loss From Impoundment Terrace Systems--	
John M. Laflen and H. P. Johnson.....	2-30
Excavated Sediment Traps Prove Superior to Dammed Ones--	
Charles P. Tryon, Bruce L. Parsons, and Mack R. Miller	2-42
Urban-Area Sediment Yield--Effects of Construction-Site	
Conditions and Sediment-Control Methods--	
Thomas H. Yorke and William J. Herb.....	2-52
Erosion and Sediment Control on Reshaped Land--	
L. D. Meyer and M. J. M. Romkens.....	2-65
Defining the Sediment Trapping Characteristics of a	
Vegetative Buffer. Special Case: Road Erosion--	
Coryell A. Ohlander.....	2-77
Geotechnical Management of Forest Lands in Granitic	
Terrane--	
B. G. Hicks.....	2-83
Evaluation of an Extensive Sediment Control Effort in	
the Los Angeles River Basin--	
Earl C. Ruby.....	2-91

3rd FISC

	Page
Sediment Control at Imperial Dam--	
T. H. Moser and W. D. Sears.....	2-103
Determining Erosion from Hawaiian Agricultural Lands--	
Gary W. Frasier, John A. Replogle, Keith R. Cooley, and S. A. El-Swaify.....	2-124
The Universal Soil Loss Equation as Adapted to the Pacific Northwest--	
D. K. McCool, R. I. Papendick, and F. L. Brooks.....	2-135
Techniques of Researching Information for the Development of a Manual to Vegetate Soils of Low Productivity Disturbed by Construction Activities--	
Francis Wm. Bennett and Roy L. Donahue.....	2-148
The Dilemmas of Setting Sediment Standards--	
J. Gessler.....	2-158
Design and Performance of Rock Revetment Toes--	
Walter M. Linder.....	2-168
Control of Turbidity at Construction Sites--	
E. J. Carlson.....	2-180

SYMPOSIUM 2
EROSION AND SEDIMENT CONTROL

SOIL EROSION CONCEPTS AND MISCONCEPTIONS

By L. D. Meyer, Agricultural Engineer, D. G. DeCoursey, Hydraulic Engineer, and M. J. M. Romkens, Soil Scientist, USDA Sedimentation Laboratory, Oxford, Mississippi

ABSTRACT

Current concepts of the process of soil erosion by water are presented. Their implications as a basis for continued improvements in erosion prediction and control are discussed. Research deficiencies are indicated, and promising new approaches are explored.

INTRODUCTION

The complementary processes of soil erosion and sedimentation involve a complex interrelationship of many factors that influence detachment, transportation, and deposition of soil particles. Recent research has provided knowledge useful for developing improved concepts concerning these processes and for allaying certain misconceptions about them.

THE PROCESSES OF SOIL EROSION AND SEDIMENTATION BY WATER

Soil erosion is the detachment of soil particles from the soil mass and their transport by rainfall and runoff. Eroded soil* in transport is sediment, and sedimentation occurs when sediment is deposited. Soil movement is often intermittent, with detachment, transport, and deposition recurring repeatedly as the soil material moves through a watershed system. Concepts of these processes have been explored by Bennett (1939), Horton (1945), Ellison (1947), Stallings (1953), Smith and Wischmeier (1962), Robins and Neff (1963), Guy (1970), Piest (1970), Partheniades (1971), Bennett (1974), and others.

Detachment Both raindrop impact and shear from flowing runoff are major forces causing soil detachment. The effects of raindrop impact are uniformly distributed over the land surface. Several trillion raindrops annually bombard each hectare (2.5 acres) of land in the humid United States at impact velocities of up to 9 m/sec (30 ft/sec). Unless the soil surface is protected by vegetation or mulch, these raindrops can detach tremendous quantities of soil for transport by splash or runoff.

Detachment by runoff occurs primarily on the small percentage of the land surface where the flow concentrates and exceeds the critical tractive force for the existing soil condition. The rills that result from detachment by runoff are much more apparent than erosion on interrill areas between rills, although the erosion rates for a watershed may be of the same order of magnitude for both types.

* When used hereafter in this context, "soil" refers to soil particles, either primary or aggregated.

Transport Eroded soil is transported downslope by raindrop splash and overland flow. Net downslope movement by splash increases as the slope steepens (Ekern, 1950). The rate of downslope transport by splash is usually quite small relative to that by runoff (Young and Wiersma, 1973; Lattanzi et al., 1974), but considerable soil may be splashed short distances to locations where concentrated runoff can transport it, if the soil surface is unprotected.

Most eroded soil is transported downslope by concentrated runoff, but runoff does not begin until the soil surface storage is filled and rainfall intensity exceeds the infiltration rate of the soil. The quantity and size of material that can be transported by runoff is a function of runoff velocity and turbulence, and these increase as the slope steepens and the flow increases.

Either the availability of detached soil or the transport capability of the erosive agents can limit soil erosion (Meyer and Wischmeier, 1969; Rowilson and Martin, 1971). Only soil particles that have been detached from the soil mass will be eroded, no matter how great the transport capability (detachment limiting), or only soil particles that can be transported by the flow will be eroded, no matter how much is detached and available for transport (transport limiting). Erosion control practices that are designed to reduce whichever of these two subprocesses is limiting will usually be more effective than those designed to reduce the other subprocess that is in excess.

Deposition Some of the sediment being transported by runoff may deposit when the transport capability of runoff is reduced by vegetation or by decreased slope steepness. Sedimentation is a segregating process with the largest and most dense materials settling first, whereas colloidal material may remain in suspension indefinitely. Therefore, the size distribution of the eroded material is important in determining the portion of the sediment load that is deposited and the size distribution of the remaining sediment load.

Rill and Interrill Erosion Separation of the source of soil eroded from upland areas into that from small channels, called rills, and that from between rills, the interrill areas, is helpful both conceptually and in studying the soil erosion process. Rill erosion occurs on only a limited part of the land surface and is caused primarily by runoff, whereas interrill erosion results primarily from raindrop impact and is relatively uniform over the remainder of the surface. Rill patterns usually develop because of tillage marks, surface roughness, or topographic variations that affect flow patterns of runoff. The tendency for rills to meander decreases as the slope steepens. Once rill erosion begins, rilling typically progresses upslope by a series of intensely erodible headcuts or knickpoints (Meyer et al., 1974).

The term "sheet erosion" has often been used to include all erosion that can be obliterated by tillage practices, thus including sizeable rills. Actual sheet flow and sheet erosion occur only before runoff has appre-

ciable detaching capacity. Use of the terms "interrill" and "rill" erosion eliminates the need for the term "sheet", since interrill erosion is literally sheet erosion and rill erosion implies the nonsheet, locationally selective removal caused by runoff detachment.

Interrill areas are not rilled because the thin-film flow has not reached a tractive force sufficient to detach soil particles and/or the transport capability of interrill flow is fulfilled with sediment detached by rainfall. In the latter case, no capability remains for detachment by runoff. The start of rilling is evidence that the tractive force of the flow has exceeded the critical level and that the sediment load capacity of runoff is greater than the available detached material. The interrelationship of sediment transport capacity, sediment load, and detachment rate of runoff is discussed by Foster and Meyer (1972b). Horton's (1945) "belt of no erosion" apparently referred to the area near the top of a slope where no rill erosion was visible but considerable interrill erosion could occur.

The relative contributions of rill and interrill erosion vary with the hydrologic and soil conditions. The interrill erosion rate is relatively constant all over an area of sloping land when the soil, cover, etc. are uniform. Even slope steepness affects interrill erosion relatively little (Lattanzi et al., 1974). However, rill erosion varies from none near the top of a slope, where all sediment is from interrill areas, to major rates downslope, where concentrated flow occurs. Thus, the interrill contribution to the total upland erosion may vary from 100% on short slopes where rilling is negligible to a small percentage of the total for slopes where rill erosion is greatly dominant.

Although interrill detachment and splash transport from raindrop impact are similar for all interrill areas on a slope, the detachment and transport capabilities of concentrated runoff increase downslope as the flow acquires greater mass and velocity. Generally, cohesive soil is more easily transported than detached by rill flow (Meyer and Wischmeier, 1969), so rills soon have more transport capability than there is material available. As a result, transport capability seldom limits the rate of rill erosion once rilling begins, unless the soil is noncohesive and very readily detached or until downslope cover or reduced steepness reduces the transport capability of the flow.

IMPROVED EROSION PREDICTION AND CONTROL

Improved concepts of the soil erosion and sedimentation processes not only increase our understanding of these processes but they also provide useful clues for developing better erosion prediction techniques and erosion-control practices.

Universal Soil Loss Equation Much of the current technology for erosion prediction and control is based on the Universal Soil Loss Equation (USLE) developed by Wischmeier and Smith (1965) and Wischmeier (1974).

It includes greater detail in several aspects than the Musgrave (1947) equation and other predecessors. The USLE identifies six major factors that affect upland soil erosion by water: rainfall erosiveness, soil erodibility, slope length, slope steepness, cropping and management practices, and supporting conservation practices. Empirical relationships, based on statistical analyses of available data, were used for evaluating these factors. Although the model that was developed is widely used, the potential exists for significantly improving various USLE factor values, relationships, and interrelationships as more knowledge and improved concepts of the erosion process become available.

Various misconceptions about the use of the USLE seem to have developed. Like all models, it should only be used for its intended purpose--estimating long-term average annual soil losses for upland slopes where deposition is negligible. Erosion from gullies and stream channels or deposition on or below upland slopes is not considered by the USLE. It is applicable for seasonal periods with some increase in the error margin, and it is least accurate for individual storms. It can be adapted to predict erosion rates during extreme years using appropriate factor values for selected probabilities of occurrence (Wischmeier and Smith, 1965). Since the USLE has been so widely adopted for erosion prediction and a continuing need exists for such a model, improving its factors and developing improved models are important research goals.

Soil and Topographic Relationships Improved relationships to describe the effects of soil erodibility, slope length, and slope steepness on upland erosion seem feasible by considering the separate contributions of rill and interrill erosion, instead of using only the lumped response of an area as has been done previously. An example is the USLE soil erodibility factor, K, defined as the erosion rate for a specific slope length (22.1 m or 72.6 ft) and steepness (9%). All research data collected at other lengths or steepnesses must be adjusted to this base condition, and all erosion predictions made for other lengths and steepnesses must be adjusted from this condition using established topographic relationships (Wischmeier and Smith, 1965). These relationships do not take into account how soils differ in their extent of rill and interrill erosion at the base condition. Neither do they account for differences in the susceptibilities of different soils to rill and interrill erosion at other topographic conditions. Instead, upland erosion is expressed as simply proportional to an exponential power of slope length (Zingg, 1940; Musgrave, 1947; Wischmeier and Smith, 1965). Most groups of data used to derive these relationships included only 3 lengths with the longest less than 45 m (150 ft). For slope steepness, a similar exponential relationship was used (Zingg, 1940; Musgrave, 1947) until a quadratic expression (Wischmeier and Smith, 1965) was found superior. Slopes studied ranged from only 3 to 20%.

Recent analyses (Roster and Meyer, 1972a) have suggested that the susceptibility of a given soil condition to rill erosion relative to its susceptibility to interrill erosion may greatly influence the effect of slope length or slope steepness on total erosion. They indicated that

the exponent of slope length in the USLE relationship effectively increased as rill erosion increased (Meyer et al., 1974), rather than remaining constant as is commonly assumed. Since the amount of rill erosion relative to that of interrill erosion generally increases with both slope length and slope steepness, erosion will increase more with slope length on steep slopes than on lesser slopes. Therefore, a larger exponent will be appropriate for the steeper slopes. Foster, et al (1973) showed how a soil's susceptibility to rill and to interrill erosion might be separated into K_r and K_i instead of lumping both effects into one factor as done for the USLE K-factor.

Experimental data (Meyer et al., 1972a) from a soil with two types of prior management illustrate how different susceptibilities to rill and interrill erosion can affect the erosion rates of different soils at different slope lengths. These two conditions, R-3 and R-4, had essentially identical interrill erosion rates and no rilling at interrill lengths of 0.6 m (2 ft). However, combined rill plus interrill erosion for field plots of 10.7 m (35 ft) length was more than three times greater for soil condition R-3 than for R-4. These data were fitted to a relationship that separately considered rill and interrill erosion characteristics (Meyer et al., 1974). The resulting equations showed that the relative erosion of these two conditions increased with slope length from no difference at short lengths, where there was no rilling, to a maximum ratio of more than three times between 10 and 15 m (Table 1). Extrapolation of the data to a 40 m length showed that relative erosion decreased to 2.5 times there, and the limit for very long slopes approached 2.0 times. The relative contributions of rill and interrill

Table 1. Soil Losses for Two Conditions that had Different Susceptibilities to Rilling

Slope Length, L m	Condition R-3 ^a			Condition R-4 ^b			G_T Ratio, R-3/R-4
	Interrill Eros, G_I	Rill Eros, G_R	Total, G_T	Interrill Eros, G_I	Rill Eros, G_R	Total, G_T	
	-----kg/hr-----			-----kg/hr-----			
1	7	0	7	7	0	7	1.0
3	21	8	29	21	0	21	1.4
5	35	32	67	35	0	35	1.9
10	70	162	232	70	4	74	3.1
15	105	392	497	105	49	154	3.2
20	140	722	862	140	144	284	3.0
30	210	1680	1890	210	484	694	2.7
40	280	3040	3320	280	1030	1310	2.5

$$^a G_T = G_I = 7L, L < 1; G_T = G_I + G_R = 7L + 2(L - 1)^2, L > 1$$

$$^b G_T = G_I = 7L, L < 8; G_T = G_I + G_R = 7L + 1(L - 8)^2, L > 8$$

These equations were derived using techniques in Meyer et al., (1974) and data from Meyer et al., (1972a).

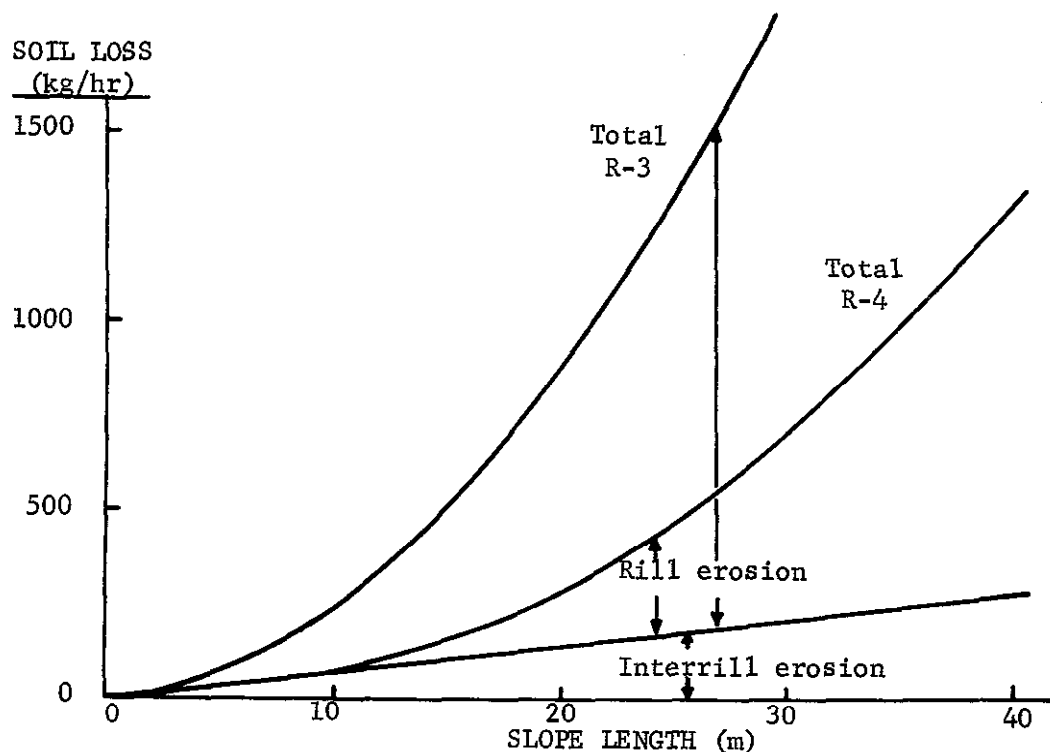


Figure 1. Relative contributions of rill and interrill erosion as affected by slope length for two conditions. R-3 and R-4 were equally susceptible to interrill erosion, but R-3 was much more susceptible to rilling.

erosion to the totals are indicated in Figure 1. These data suggest that the relative erodibility of these two soil conditions depends on the slope length at which they are tested and that the effect of slope length on erosion is not adequately expressed by an exponential relationship.

Additional research is needed to test these implications, but they suggest that extrapolation of current slope length and slope steepness relationships beyond the limited range of experimental data may lead to erroneous estimates of erosion rates. Furthermore, the effects of other erosion-influencing factors like rainfall intensity, surface cover, and type of management on rill and interrill erosion may be as important as those of soil type and topography. Such effects have not been included in past erosion models.

Limiting or Critical Erosion Conditions The realization that major rill erosion does not begin until runoff reaches a critical condition, called "breakdown", suggests a useful concept for improving erosion prediction and erosion control techniques. A study (Meyer et al., 1972b) of several mulches showed that such a breakdown caused a major increase in the erosion rate per unit of erosion hazard (center curves of Figure 2). The

results also indicated that improved cover can significantly increase the slope length at which major rilling begins for given soil and slope conditions (Meyer et al., 1972a). Vegetation and mulches decrease the erosiveness of runoff by (a) maintaining greater infiltration rates and thus reducing the amount of runoff, (b) increasing the hydraulic roughness and thus slowing runoff velocity, and (c) absorbing much of the shear stress of the runoff and thus reducing that exerted on the soil itself.

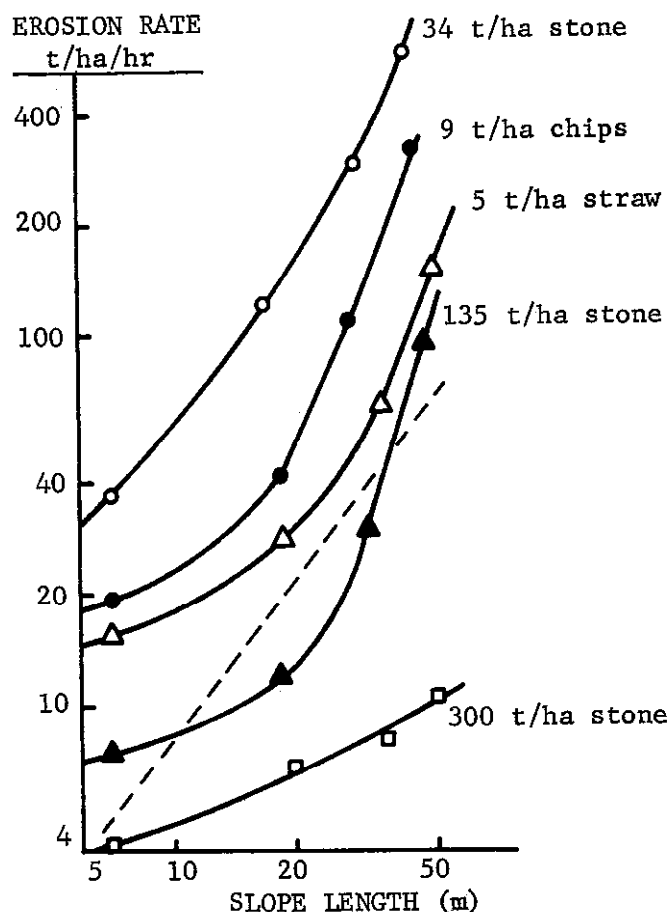


Figure 2. Effect of mulch type and rate on erosion rate for several slope lengths. No rilling occurred at any of the lengths studied for the bottom curve. Rilling began before the shortest length of the top curve. The lengths where the other three curves steepened were those at which serious rilling began. A straight line fitted to such data (illustrated by dashed line) may indicate erroneous erosion rates when extrapolated well beyond the range of data.

By selecting design criteria for erosion control practices to prevent appreciable rill erosion, soil losses will remain at the lower rate on the erosion hazard versus erosion rate curve (Figures 1 and 2). Once rilling begins, erosion increases more rapidly per unit of increased erosion hazard than before rilling starts. Soil surface covers that decrease rilling tendencies cause the point where the line curves upward to be shifted farther to the right.

The failure to recognize that the erosion trend changes when serious rilling begins may lead to misinterpretation of erosion data. For example, the effect of slope length on erosion rate is usually expressed

as an exponential relationship, as discussed earlier. Such relationship plots as a straight line on logarithmic paper, yet a straight line does not properly express data like some of that in Figure 2. Thus, a straight line that is fitted to data with more than one trend may lead to serious inaccuracies, particularly if extrapolated beyond the range of data.

The concept of limiting or critical conditions is quite common. Critical tractive force is often considered in sediment-transport hydraulics. Threshold wind velocity is an important parameter in wind erosion. Hudson (1971) found a critical value of intensity at which rain begins to become erosive. Although limiting conditions have seldom been used as a basis for developing erosion-control design criteria, they seem to have merit if the conditions at which breakdown begins can be established.

Transportability of Eroded Soil Eroded soil particles are often detached as aggregates, thus the textural characteristics of a soil may be a poor indicator of the transportability of its eroded material. Much of the sediment that texturally consists mostly of silt and clay may have transportability characteristics more like those of sand because of aggregation. (The slightly lesser density of aggregates affects transportability much less than their increased size.) Also, the transportability of soil eroded from interrill areas may differ from that from rills. Sediment from interrill areas may be finer than that from rills, because (a) raindrop impact breaks aggregates into smaller sizes than those detached by rill flow and (b) large particles are not as readily transported from the interrill areas. In contrast, material eroded from rills is generally larger and less affected by selective sorting than interrill sediment.

The transportability of eroded soil is particularly important when runoff passes through locations of potential deposition. The larger the size distribution and density of the transported material, the more sediment will deposit and the greater the effect on the subsequent size distribution of the outflow material. Practices that protect the soil surface from raindrop impact will both decrease the amount of detached soil available for transport and will reduce the breakdown of aggregates so that the remaining material is less transportable, especially on interrill areas.

Sediment Delivery Ratios The ratio of the sediment yield to gross erosion for a watershed is known as the sediment delivery ratio. Thus, this ratio is greatly dependent on the transportability of sediment and the opportunities for sediment deposition within the watershed. The sediment delivery ratio is an example of a concept that is necessary at the present state of knowledge, although lumped together in it are various aspects of sediment movement and deposition.

Methods for evaluating sediment delivery ratios are generally less well developed than those for estimating upland erosion. Field data are rarely available, so sediment delivery ratios are usually estimated from

correlations with watershed area, stream length, relief, or other factors (Roehl, 1962; Maner, 1963) even though the sediment delivery ratio is largely a function of the deposition of sediment eroded from the uplands. The major factors influencing sediment deposition--particle size distribution, flow velocity, and topography--are seldom considered.

Since estimates of sediment yield will continue to be essential for wise soil and water management, efforts toward improved evaluations of sediment delivery are warranted. Considerable potential for improved sediment routing exists by using basic concepts of sediment movement, deposition, and other physical processes involved (Renfro, 1972).

RESEARCH APPROACHES

With new knowledge about soil erosion and a better understanding of the erosion process, past misconceptions about research approaches have become apparent. Thus, examination of research techniques in light of current knowledge can be used to improve them and the interpretation of research data.

Size of Erosion-Research Areas Erosion data from cups, small pans, and infiltrometer plots may provide useful estimates of splash and interrill erosion if water is applied at an energy and intensity comparable to natural rainfall, but they cannot be meaningfully extrapolated to provide estimates of field erosion. Recent advances in separating the causes and rates of rill and interrill erosion showed that if realistic rates of field erosion are to be measured, experimental research areas must be at least large enough to accumulate enough runoff to transport detached particles and to allow rilling to develop. Yet, even runoff-plot studies that evaluate both rill and interrill erosion may be inadequate on sites where gully or streambank erosion is the dominant sediment source or where deposition is significant. Experimental research must be designed to include evaluations of all major sediment sources and sinks.

Data Collection Techniques The lack of erosion and sediment yield data for many land-use conditions and the high cost of developing research projects to collect needed data often tempts those planning research to take shortcuts. Typical suggested shortcuts include collecting only sporadic grab samples of runoff instead of sampling runoff events systematically, omitting replication of treatments, and conducting studies on available sites regardless of unsuitable or confounding features. Such approaches often provide misleading data that may be misinterpreted and lead to incorrect conclusions. Erosion rates vary appreciably throughout each storm, among different storms, at different locations, and for relatively similar conditions. Systematic sediment sampling over a range of conditions provides a better average and also indicates the variability that may occur among similar conditions. Grab samples, unreplicated treatments, and other shortcuts may lead to conclusions based on extremes and give little indication of the confidence range of the results.

Experimental Research vs Analytical Research Experimental research involves collecting data from experiments. Such data are used to compare the conditions under study and to suggest numerical trends. Extrapolation of trends indicated by experimental research beyond the range of data is risky.

Analytical research as defined here is based on the use of basic physical concepts to derive fundamental relationships. If the basic concepts are validated and properly used, the resultant relationships can be confidently applied over a wide range of conditions. Generally, much less experimental data are required to verify analytically derived relationships than are necessary to develop conclusive relationships using the data alone. This is an important consideration, since an extensive experimental effort is costly in time, money, and personnel.

Much past research has been predominantly experimental or predominantly analytical. Yet a parallel, complementary effort for both aspects has great potential for effective research. A strong analytical effort provides sound fundamental concepts and guidance in research planning. A well-planned experimental effort provides appropriate "checkpoint" evaluations for analytical expressions and tests of the analytical implications. Either of them alone or both of them independently may be inefficient or misleading.

Until recent years, inadequate analytical effort toward developing sound concepts was a common shortcoming of soil and water conservation research. Recently, broad conceptual models have been proposed, but the necessary experimental data needed for their validation are not available. Thus, insufficient experimental data pertinent to model validation seems to have become a major deficiency of current research. Since experimental knowledge often takes longer to obtain than the development of new conceptual models, sound and usable experimental research data are becoming an urgent need.

Development of Mathematical Models Empirical erosion models, such as the USLE and its predecessors, have been very useful for many field applications. However, they were developed from statistical analyses of available experimental data and thus are not sound bases for developing fundamental erosion models.

Fundamental models are now feasible because of recent advances in understanding basic soil erosion principles and processes. They are urgently needed for assessing subtle differences in watershed response that result from changes in land use, with due consideration given to the various elements that interact within a watershed system. Such models may be more detailed than necessary for predictive uses, but they are essential for evaluating land-use alternatives. They are a high priority research need in the field of erosion and sedimentation. The analytical and experimental research conducted during development of these fundamental erosion models also will provide information for refining the empirical models.

REFERENCES

- Bennett, H. H. 1939. Soil Conservation, McGraw-Hill Book Co., Inc. New York, 993 p.
- Bennett, J. P. 1974. Concepts of Mathematical Modeling of Sediment Yield, *Water Resources Research* 10:485-492.
- Ekern, P. C. 1950. Raindrop Impact as the Force Initiating Soil Erosion. *Soil Sci. Soc. Amer. Proc.* 15:7-10.
- Ellison, W. D. 1947. Soil Erosion Studies, *Agric. Engin.* 28:145-146, 197-201, 245-248, 297-300, 349-351, 402-205, 442-444.
- Foster, G. R. and Meyer, L. D. 1972a. Mathematical Simulation of Upland Erosion Using Fundamental Erosion Mechanics, *Proc. of the Sediment Yield Workshop*, Oxford, Miss. ARS-S-40:190-207.
- Foster, G. R. and Meyer, L. D. 1972b. A Closed-Form Soil Erosion Equation for Upland Areas, Sediment (Einstein), Chapter 12.
- Foster, G. R., Meyer, L. D., and Onstad, C. A. 1973. Erosion Equations Derived from Modeling Principles. Paper 73-2550. ASAE Winter Meeting, Chicago, Ill.
- Guy, H. P. 1970. Fluvial Sediment Concepts, U. S. Geological Survey Techniques of Water-Resources Investigations, Book 3, Chapt. C1.
- Horton, R. E. 1945. Erosional Development of Streams and Their Drainage Basins - Hydrophysical Approach to Quantitative Morphology, *Geological Soc. of Amer. Bull.* 56:275-370.
- Hudson, N. 1971. Soil Conservation, Cornell University Press, Ithaca, New York. Chapter 4.
- Lattanzi, A. R., Meyer, L. D., and Baumgardner, M. F. 1974. Influences of Mulch Rate and Slope Steepness on Interrill Erosion, *Soil Sci. Soc. of Amer. Proc.* 38:946-950.
- Maner, S. B. 1963. Geology in Sediment Delivery Ratios. Proceedings of the Fed. Inter-Agency Sedimentation Cnfr., USDA Misc. Publ. 970:108-110.
- Meyer, L. D., Foster, G. R., and Nikolov, S. 1974. Effect of Flow Rate and Canopy on Rill Erosion, ASAE Paper 74-2035 (mimeographed), accepted for publication in *Trans. ASAE*.
- Meyer, L. D., Foster, G. R., and Romkens, M. J. M. 1972a. Source of Soil Eroded by Water from Upland Slopes, *Proc. of Sediment Yield Workshop*, Oxford, Miss. ARS-S-40:177-189.
- Meyer, L. D., Johnson, C. B., and Foster, G. R. 1972b. Stone and Woodchip Mulches for Erosion Control on Construction Sites. *Jour. Soil and Water Conserv.*, 27:264-269.
- Meyer, L. D. and Wischmeier, W. H. 1969. Mathematical Simulation of the Process of Soil Erosion by Water, *Trans. ASAE* 12:754-758, 762.
- Musgrave, G. W. 1947. The Quantitative Evaluation of Factors in Water Erosion, A First Approximation, *Jour. of Soil and Water Conserv.* 2:133-138.
- Partheniades, E. 1971. Erosion and Deposition of Cohesive Materials. River Mechanics, Vol. II, Chapter 25.
- Piest, R. F. 1970. Sediment Sources and Sediment Yields. *J. Hydraulics Div., ASCE*: 96(HY 6) 1283-1329.

- Renfro, G. W. 1972. Use of Erosion Equations and Sediment Delivery Ratios for Predicting Sediment Yield, Proc. of the Sediment Yield Workshop, Oxford, Miss. ARS-S-40:33-45.
- Robins, J. S. and Neff, E. L. 1963. Principles of the Erosion Process, Proc. Forest Watershed Mgmt. Symp., Oregon St. Univ., 127-139.
- Roehl, J. W. 1962. Sediment Source Areas, Delivery Ratios, and Influencing Morphological Factors, Internatl. Assoc. for Hydrol. Publ., 59:202-213.
- Rowlison, D. L. and Martin, G. L. 1971. Rational Model Describing Slope Erosion, J. Irrig. and Drainage Div., ASCE 97(IR1):39-50.
- Smith, D. D. and Wischmeier, W. H. 1962. Rainfall Erosion. Advances in Agronomy, Academic Press, New York, V. 14, 109-148.
- Stallings, J. H. 1953. Mechanics of Water Erosion, USDA Soil Conservation Service TP-118. 18 pages.
- Wischmeier, W. H. 1974. New Developments in Estimating Water Erosion, Proceedings, 29th Annual Meeting, SCSA, Syracuse, NY:179-186.
- Wischmeier, W. H. and Smith, D. D. 1965. Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains, USDA Agric. Handbook 282, 48 pages.
- Young, R. A. and Wiersma, J. W. 1973. The Role of Rainfall Impact on Soil Detachment and Transport, Water Resour. Res. 9:1629-1636.
- Zingg, A. W. 1940. Degree and Length of Land Slope as it Affects Soil Loss in Runoff, Agric. Engin. 21:59-64.

COMPARATIVE COSTS
OF EROSION AND SEDIMENTATION
CONTROL MEASURES

by

K. H. Ateshian

Technical Director, Irrigation/Agriculture
Engineering-Science, Inc., Berkeley, California

ABSTRACT

Cost information on erosion and sediment control measures for 25 methods in current widespread use are presented in a manner to provide a sound basis for estimating local costs. Using three principal cost elements: materials, labor and equipment, the detailed unit costs presented can also be up-dated to reflect inflationary trends in conjunction with an accepted cost index such as the ENR Construction Cost Index.

The simplified procedures developed for estimating rainfall erosion index permit the prediction of soil loss (using the Universal Soil-Loss Equation) for selected watersheds. The cost data, when applied to soil losses, will reflect comparative costs per ton of soil retained. Control effectiveness and economic life of each method can then be used to determine comparable annual cost figures. With this final step, the process of engineering decision-making and selection can be greatly simplified.

INTRODUCTION

Natural erosion processes are accelerated when existing protective cover is removed before or during construction activities. Increased exposure of the soil mantle to the full kinetic energy of rain and overland and channelized flow, plus the dynamic mechanical action of men and machines as they move over the site, cause increased movement and loss of soil particles. While this erosion of soil frequently causes on-site construction problems, it is even more common for the sediment removed to create many undesirable conditions in downstream areas (Hotes 1973).

Most of these adverse results of man's construction activities can be reduced by proper and timely use of both structural and nonstructural measures of various types. Selection of the proper measures to use in any specific situation requires not only the availability of technical information but also comparative and up-to-date cost information on the various methods.

This paper presents in a condensed form cost data and information obtained in the course of a research investigation by Engineering-Science, Inc. for the Environmental Protection Agency, under the direction of Mr. Robert E. Thronson, Project Officer.

Costs of erosion control practices and sediment removal work are presented in tabular form in terms of the three elements: materials, labor and equipment. In order that contractors and government agencies

may use the data more readily, mobilization plus overhead and profit margins are included in the costs. Due consideration should, of course, be given to local conditions and inflationary trends in estimating for selected watersheds at a given point in time for budgetary purposes.

The simplified procedures developed for estimating average annual rainfall erosion index, especially for areas west of the Rockies, permit the prediction of potential average annual soil loss (using the Universal Soil-Loss Equation). The other parameters involved in the equation can be evaluated from field measurements.

The cost data of several candidate methods of erosion and sediment control, when applied to soil losses estimated by the Universal Soil-Loss Equation, will reflect comparative unit costs per ton of soil retained. Introduction of control effectiveness and economic life as additional parameters can assist engineering decision-making by converting the unit costs of each of the candidate methods into comparable annual cost figures. With this final step, selection of the method to be implemented can be greatly simplified.

COST OF EROSION AND SEDIMENT CONTROL MEASURES

Erosion control methods fall basically into one of two broad categories. These are designated as structural and vegetative measures. With the introduction of chemicals for soil stabilization, a separate category is indicated. However, because they are usually applied to support and ensure emergence of vegetation, chemicals can be appropriately included in the vegetative category. On the other hand, removal of sediments from downstream areas has to be considered either separately or in conjunction with erosion control measures for the simple reason that no erosion and sediment control measure is totally effective, and moreover, because of the law of diminishing returns, it is economically unjustifiable to aim for an absolute level of effectiveness. In actual field practice, a combination of structural and vegetative measures suited to the particular site is employed together with some provision for removal of sediments from downstream areas on a maintenance basis.

With this field practice in mind, Table 1 presents the comparative costs of 16 structural and 9 vegetative measures together with 3 sediment removal methods (Hotes 1973). Where costs are affected by the size of the structure, areal extent of vegetative practice or volume of sediment removed, these are clearly identified. Hence, it is possible to develop several alternative combinations and to cost these out for a given particular site. Alternatively, Table 1 can be utilized to arrive at the cost of erosion control and compare it with the cost of various methods of correcting erosion-type damage by removal of sediments.

For maximum usefulness, the contractor cost-estimating approach has been used as the principal basis for the costs presented in Table 1, using three main cost elements: materials, labor and equipment. Details of material unit costs, labor rates and equipment rental rates used in arriving at the comparative costs are listed in Table 2. Therefore,

TABLE 1
COMPARATIVE COSTS OF EROSION AND SEDIMENT CONTROL MEASURES FOR CALIFORNIA, JANUARY 1973
(after Hotes [1973])

Category	Procedure/Method	Size	Unit	Total Cost	Cost Breakdown			Remarks/Comments
					Materials	Labor	Equip't	
STRUCTURAL MEASURES "Construction Activities"	1) Gravel & Earth Check Dam	1 x 5 2 x 10 2 x 15	cu. ft. " "	\$ 1.84 1.72 0.83	0.16 0.12 0.22	1.68 1.60 0.48	- - 0.13	height x width " "
	2) Rock Riprap Check Dam	2 x 5 3 x 10 4 x 15 5 x 20	cu. ft. " " "	7.00 6.71 6.80 8.17	0.62 0.63 0.62 0.63	6.38 6.08 5.59 6.73	- - 0.59 0.82	height x width " " "
	3) Concrete Check Dam	2 x 5 x 4 6 x 10 x 8 5 x 18 x 14 7 x 20 x 20	cu. yd. " " "	598 288 261 217	150 85 82 73	388 191 168 134	59 12 11 10	height x width x length
	4) Concrete Chute	6 x 40	sq. ft.	5.40	1.88	2.80	0.73	3' bottom width
	5) Diversion Dike	15	cu. yd.	12.93	-	8.86	4.07	material at site
	6) Erosion Check - Jute	4 x 152	lin. ft.	3.43	0.25	3.18	-	width x length
	7) Filter Berm	30	cu. ft.	10.63	7.55	3.08	-	truck tailgates
	8) Flexible Down Drain	24"Ø x 300'	lin. ft.	7.34	7.03	0.31	-	
	9) Flexible Erosion (VSL) Control Mats	4" x 25' x 1,320'	sq. ft.	1.18	0.84	0.30	0.04	thickness x width x length
	10) Gabions	10 100 1,000	sq. yd. " "	30.10 15.50 12.67	8.80 6.88 6.51	21.30 6.79 5.35	- 1.82 0.81	
	11) Level Spreader	15 44 78	lin. ft. " "	3.80 1.90 1.63	- - -	2.33 1.25 1.06	1.47 0.66 0.57	
	12) Sandbag Barrier	180	sacks	3.10	0.41	2.69	-	
	13) Sectional Down Drain	24"Ø x 40' 24"Ø x 234'	lin. ft. "	14.55 10.91	2.68 2.58	10.93 7.80	0.95 0.53	
	14) Sediment Retention Basin	6 x 30 7 x 30 8 x 40	cu. ft. " "	13.78 12.88 10.51	0.98 1.51 0.98	8.64 7.76 6.08	4.16 3.61 3.45	dam height x length
	15) Storm Sewer Inlet Prot.	straw	Bale	7.86	2.90	4.96	-	
	16) Gravel Weir	8" x 4' x 6'	weir	10.44	4.64	5.80	-	height x width x length
VEGETATIVE MEASURES "Highway and Landscape"	17) Chemical Soil Stabilizer	10	acres	1,300	1,040	125	141	Petroset SB
	18) Excelsior Mat	1	acre	12,200	2,173	9,651	362	Steep slopes
	19) Hydroseeding	1 10 30	acre acre acre	858 427 344	235 235 235	260 88 47	363 104 62	
	20) Fumigation - Meth Br.	10	acres	2,344	2,178	88	78	Heavily infested areas prior to hydroseeding
	21) Jute Mesh (Ludlow)	1	acre	7,700	2,545	4,798	363	Steep slopes
	22) Sodding	1	acre	14,800	6,564	2,508	5,678	
	23) Sod Plugs - 4" sq.	1	acre	11,300	847	4,994	5,491	Plugs @ 12" c/c
	24) Straw or Hay	10	acre	1,200	245	343	575	4 tons/acre
	25) Woodchips - 3" thick 3/4" thick	1 1	acre acre	8,000 3,100	2,015 598	3,713 1,401	2,248 1,114	with seed & fert.

TABLE 1 (Continued)

COMPARATIVE COSTS OF EROSION AND SEDIMENT CONTROL MEASURES FOR CALIFORNIA, JANUARY 1973

Category	Procedure/Method	Size	Unit	Total Cost	Cost Breakdown			Remarks/Comments
					Materials	Labor	Equip't	
REMOVAL OF SEDIMENTS "Maintenance and Municipal"	26) Excavation/Removal							
	a) of sediments from streets		cu. yd.	8.00	-	5.04	2.96	
	b) of sediments from basement		"	77.00	-	59.75	17.25	
	c) with bucketline from sewers		"	144.00		111.00	33.00	
	d) with vacator from sewers		"	68.00		40.00	28.00	
	e) of sediments from							
	(1) Reservoirs	0 - 0.5	"	1.40	-	0.28	1.12	using trucks
	(capacity in millions)	0.5 - 1	"	1.15	-	0.23	0.92	" "
		1 - 2	"	2.40	-	0.36	2.04	" conveyors
		2 - 3	"	2.00	-	0.30	1.70	" "
		3 - 4	"	1.75	-	0.26	1.49	" "
		4 - 6	"	1.55	-	0.23	1.32	" "
		6 - 10	"	1.30	-	0.20	1.10	" "
		10+	"	1.25	-	0.19	1.06	" "
	(2) Debris Basins	0 - 0.02	"	2.55	-	0.51	2.04	using trucks
		0.02 - 0.04	"	2.00	-	0.40	1.60	" "
		0.04 - 0.06	"	1.60	-	0.32	1.28	" "
		0.06 - 0.08	"	1.40	-	0.28	1.12	" "
		0.08 - 0.10	"	1.25	-	0.25	1.00	" "
	27) Dredging	200 yd ³ /hr	cu. yd.	0.40	-	0.30	0.10	23 hrs./day
		400 " "	"	0.33	-	0.25	0.08	" "
		1,500 " "	"	0.17	-	0.13	0.04	" "
	28) Water Treatment							
	Filtration	1	10 ⁵ gal.	87.00				Capital & Operation
		5	"	50.00				" "
		10	"	38.00				" "
		50	"	23.00				" "
	Coagulation & Filter	1	"	295.00				" "
		5	"	180.00				" "
		10	"	145.00				" "
		50	"	87.00				" "

NOTES:

Included in above costs are the following:

a) MOBILIZATION & DEMOBILIZATION:

- 1) Small Jobs (up to \$500): 1/2 day for all labor and equipment.
- 2) Large Jobs: 1 day for all labor and equipment.
- 3) Highway & Landscape Works: 2 days for all labor and equipment.

b) OVERHEAD & PROFIT:

- 1) Small Jobs: 45% of direct costs (M + L + E)
- 2) Large Jobs & Highway and Landscape Works:
 - i) Labor overhead: 18% of labor for Contractor's labor overhead
 - ii) Plus overhead and profit: 25% of direct costs (M + L + E).

TABLE 2
DETAILS OF MATERIAL UNIT COSTS, LABOR RATES AND EQUIPMENT RENTAL RATES
USED IN ARRIVING AT COMPARATIVE COSTS OF TABLE 1
(after Hotes [1973])

Description	Unit	Unit Cost	Remarks
<u>MATERIALS:</u>			
1) Earth	cu. yd.	\$ 2.50	
Gravel	"	10.00	Small quantities - less than 2
	"	5.20	Large quantities
2) Rock Riprap	"	8.10	Small quantities
	"	6.00	Large quantities
Grout Material	"	5.40	
3) Formwork	sq. ft.	0.30	
Re-Steel	lb.	0.10	
Concrete	cu. yd.	20.00	
Metalwork	lb.	0.50	Plus \$0.30/lb for labor
Timber	b. ft.	0.20	Plus \$0.10/b. ft. for labor
4) Gunite	sq. ft.	1.50	
6) Jute Mesh	Roll	45.00	4' x 225' long
8) Flexible Down Drain	lin. ft.	4.85	24"φ; max. length 300'
9) Flexible Erosion Mat	sq. ft.	0.38	Plus \$0.30 for grout & stakes
10) Gabions	cage	7.60	6 x 3 x 1 ft. deep gabions
	"	10.40	9 x 3 x 1 ft. deep gabions
	"	12.80	12 x 3 x 1 ft. deep gabions
12) Sand	ton	4.20	
Sacks	sack	0.20	
13) Sectional Pipe	lin. ft.	2.00	Half-round 24"φ
14) Corrugated Metal Pipe	lin. ft.	1.45	6"φ
	"	3.17	8"φ
	"	3.85	10"φ
	"	5.60	15"φ
15) Straw	ton	30.00	(20 bales/ton)
Bale	unit	2.00	Complete with 1 pr. stakes
16) All-in Gravel	cu. yd.	5.50	
17) Chemical Soil Stabilizer	gallon	2.50	1:7 dillution rate with H ₂ O
18) Excelsior Mat	roll	16.00	3' x 150' long
19) Fibermulch	ton	150.00	1500 lb/acre
Seed (75% rye, 25% barley)	100 lb.	26.00	200 lb/acre
Fertilizer (11:8:4)	ton	72.00	15 lb./1000 sq. ft.
20) Methyl Bromide	sq. ft.	0.04	Specialty Subcontract
21) Jute Mesh - Ludlow	roll	30.00	787 sq. ft. effect area/roll
Staples	1000	15.00	225 staples/roll
22) Sodding - 16" strips	sq. ft.	0.16	Complete with precultivator
23) Sod plugs - 4" sq.	unit	0.055	Complete with precultivator
24) Straw/Hay	ton	30.00	4 tons/acre in two layers
25) Woodchips	cu. yd.	4.00	

TABLE 2 (Continued)
 DETAILS OF MATERIAL UNIT COSTS, LABOR RATES AND EQUIPMENT RENTAL RATES
 USED IN ARRIVING AT COMPARATIVE COSTS OF TABLE 1

Description	Unit	Unit Cost	Remarks
<u>LABOR:</u>			
Forman	hr.	9.00	
Laborer	"	8.00	
Equipment/Tractor Operator	"	11.00	
Mason	"	10.00	
Carpenter	"	10.00	
Truck Driver	"	9.00	
Hydroseeder Operator	"	10.00	
Fumigator Operator	"	10.00	
<u>EQUIPMENT:</u>			
D4 Dozer	hr.	10.00	
Front End Loader	"	14.00	
Backhoe	"	10.00	
Truck - 6 cu. yd.	"	12.00	
Loader	"	9.00	
Rammer	"	2.00	
Concrete Pump	"	10.00	
Hydroseeder	"	30.00	
Fumigator	"	20.00	
Cultipacker	"	10.00	
Blower	"	25.00	
Dray Bucket	day	60.00	
Vactor 800	"	240.00	

knowing the present local conditions of a particular site and with material costs, labor rates and equipment rental rates significantly different from those listed in Table 2, appropriate adjustments or modifications and additions can be made to the comparative cost under question. Thus differences in the three cost elements and inflationary trends can be taken care of with a few simple proportionate calculations.

ESTIMATION OF SOIL LOSS

In addition to the cost of erosion control measures, it is necessary to know the potential soil loss under a specified set of circumstances for a given geographic location. Furthermore, the amount of sediment retained on site by the use of one or a combination of several applicable control measures should be estimable. The amount retained may be estimated as the difference between estimated potential soil loss without the protective measure and the estimated loss with the measure (Hotes 1973).

A number of formulas have been developed for estimation of potential soil loss. None of these is completely satisfactory to experts in the soil conservation field, and efforts continue toward development of better expressions for relationships among the many complex factors which must be taken into account.

The Universal Soil-Loss Equation was developed by the Agricultural Research Service and reflects considerable research data, especially for areas east of the Rockies. This semiempirical predictive relationship between the mass of soil loss per unit area and all major factors known to influence rainfall erosion is described in detail (Wischmeier 1965) in USDA Handbook No. 282. It has the form:

$$A = RKLSCP$$

Equation (1)

where, on an average annual basis, A = soil loss in tons per acre per year; R = rainfall erosion index; and K, L, S, C and P are soil erodibility, slope length, slope gradient, cropping management and erosion control practice factors, respectively. For detailed information on the use of this equation, refer to USDA Handbook No. 282. It should be noted, however, that although the last five factors: K, L, S, C and P can be evaluated from simple field measurements (and Wischmeier, 1971, greatly simplifies the evaluation of K) the iso-erodent map for estimating the rainfall erosion index covers only the continental United States east of the Rockies (104° longitude), leaving out the 11 western states.

ESTIMATION OF RAINFALL EROSION INDEX

One limitation of the Universal Soil-Loss Equation has been the difficulty of calculating the average annual rainfall index. It has required not only a minimum of 22 years of continuous recorded rain-gage information but also a more than generous amount of labor input to go through the tedious effort of arriving at the erosion index for

each single storm, a requirement that in effect has kept the equation from being truly "universal" (Ateshian 1974_a).

Because rainfall energy is the principal criterion for the rainfall erosion index parameter, an analytic attack on this aspect was undertaken with the objective of developing a methodology that could tie together both east and west (Ateshian 1974_a). Efforts in that direction yielded the following:

(1) Hydrologically speaking, there are two distinct zones in the United States:

Zone I - Coastal side of the Sierra Nevada and Cascade Mountains in California, Oregon and Washington; the Hawaiian Islands; Alaska.

Zone II - Remaining United States; Puerto Rico; Virgin Islands.

(2) The 2-yr 6-hr rainfall is the critical rainfall for estimating average annual rainfall erosion index. The equations developed are as follows:

$$\text{Zone I: } R = 16.55 P^{2.2} \quad \dots \text{Equation (2)}$$

$$\text{Zone II: } R = 27.00 P^{2.2} \quad \dots \text{Equation (3)}$$

Subsequent to the finalization of the EPA report, further independent research by the author resulted in the following significant developments:

(1) As a first approximation, the USWB charts for 2-yr 6-hr rainfall were utilized in conjunction with the above equations to develop an iso-erodent map for the continental United States west of the Rockies, covering the 11 western states excluded from Handbook 282. Figure 1 is such a development (Ateshian 1974).

(2) There exist simple relationships between the annual rainfall erosion index for an average year and those for 5-, 10- and 20-yr frequencies (Ateshian 1974). Indiscriminate substitution of 6-hr rainfall of a particular frequency--other than that of 2-yr--in the above equations does not necessarily yield the correct corresponding rainfall erosion index.

Work is in progress to develop a single relationship for estimating erosion indices for various return periods using only the 2-yr 6-hr rainfall and frequency as the independent parameters.

The simplified procedure for estimating rainfall erosion index by use of Figure 1 permits the prediction of soil loss (using the Universal Soil-Loss Equation) for any geographical area in the 11 western states.

COST OF TYPICAL EROSION CONTROL MEASURE

In order to demonstrate the method of application of the comparative unit costs presented in this paper, a typical example of hydroseeding is prepared for an area 10 acres in size with maximum uninterrupted

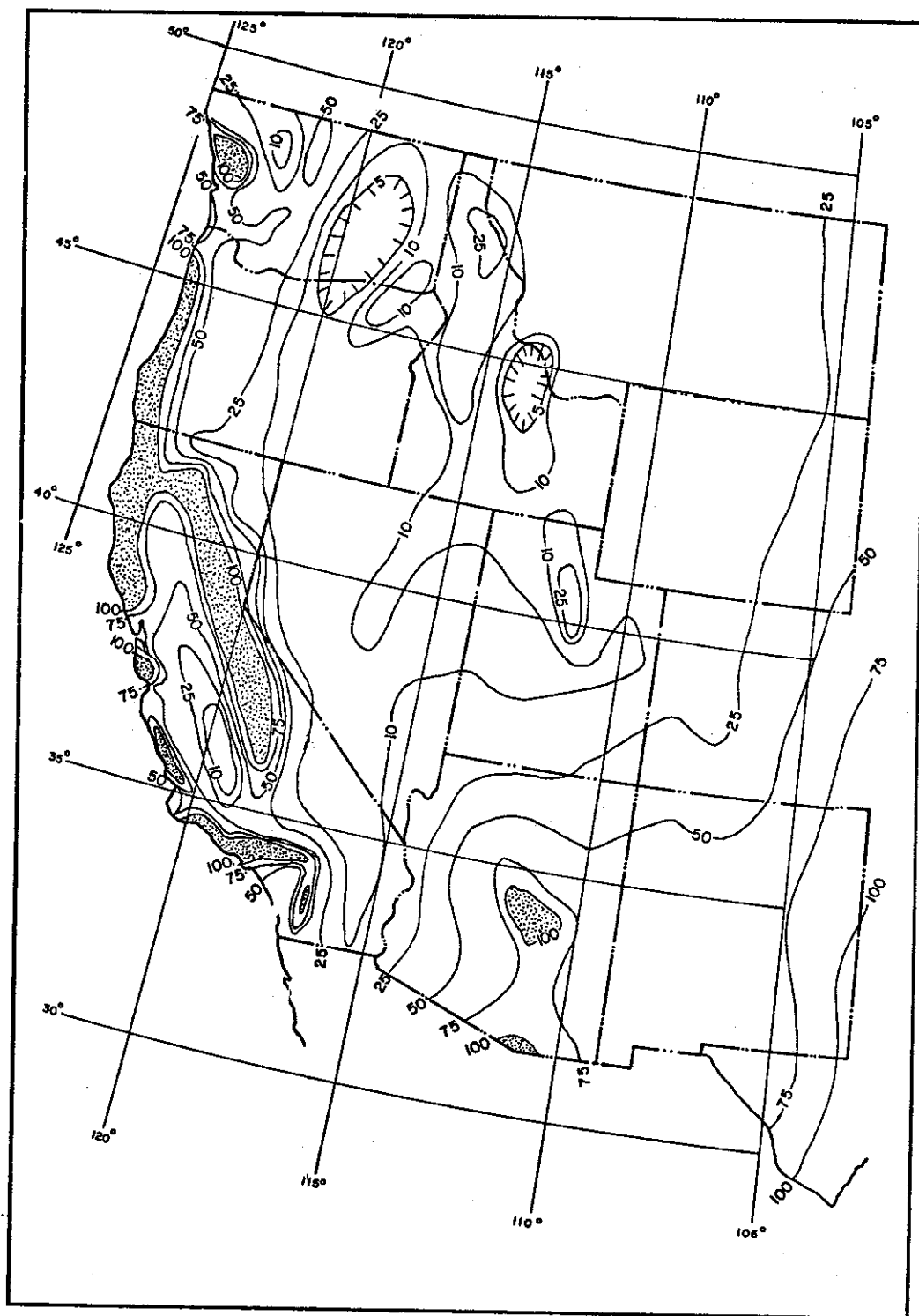


Figure 1. Average Annual Rainfall Erosion Index

length of 220 feet in the direction of a uniform 10 percent slope. One of these is assumed to be located in the Occoquan Basin, Virginia and the other in the Walnut Creek Basin, California. The accepted control effectiveness of hydroseeding is 95 percent.

Occoquan Basin

The average annual erosion index, as given by the USDA Handbook No. 282 iso-erodent map, is nearly 200. For a soil with a K-factor of 0.40, the soil retained is as follows:

$$A' = RKLS (0.95)$$

where $R = 200$; $K = 0.40$; and $LS = 2.0$ (from USDA Handbook No. 282).

$$\begin{aligned} \therefore A' &= 200(0.40)(2.0)(0.95) \\ &= 152 \text{ tons/acre/yr} \end{aligned}$$

Initial capital cost in Virginia is 86.5 percent of that in California because of lower labor rates and lower material costs.

$$\text{Initial Capital Cost} = (0.865)(427) = \$370/\text{acre}$$

With an annual maintenance cost of \$250/acre (assumed), an economic life of 10 years, and an annual interest rate of 8 percent:

$$\begin{aligned} \text{Total Annual Cost} &= (0.1490)(\$370) + \$250 \\ &= 55 + 250 \\ &= \$305/\text{acre} \end{aligned}$$

$$\begin{aligned} \text{Cost/Ton Retained} &= \$305 \div 152 \\ &= \$2/\text{ton} \end{aligned}$$

Walnut Creek Basin

The 2-yr 6-hr rainfall for the Walnut Creek Basin, located west of the Sierra Nevada, is 1.50 inches. Soil K-factor is 0.25.

$$\begin{aligned} \therefore R &= 16.55 P^{2.2} \\ &= 16.55 (1.5)^{2.2} \\ &= 40 \text{ (Figure 1 yields a similar figure)} \end{aligned}$$

Soil retained is as follows:

$$\begin{aligned} A' &= 40(0.25)(2.0)(0.95) \\ &= 19 \text{ tons/acre/yr} \end{aligned}$$

$$\text{Initial Capital Cost} = \$427/\text{acre (Table 1)}$$

With an annual maintenance cost of \$350/acre (to allow for summer irrigation requirements), an economic life of 10 years and an annual interest rate of 8 percent:

$$\begin{aligned} \text{Total Annual Cost} &= (0.1490)(\$427) + \$350 \\ &= \$63 + \$350 \\ &= \$414/\text{acre} \end{aligned}$$

$$\begin{aligned} \text{Cost/Ton Retained} &= \$414 \div 19 \\ &= \$22/\text{ton} \end{aligned}$$

Thus for this example the cost per ton of soil retained in the Walnut

Creek Basin area is 11 times that in the Occoquan Basin area. If these two areas were under consideration by a governmental agency for implementation of the erosion control measure of hydroseeding, then it is obvious that the Occoquan Basin area would be the choice.

REFERENCES

- Ateshian, K.H., 1974, Paper presented at International Erosion Control Association Conference V held at Sacramento, California on 22 February 1974.
- Ateshian, K.H., 1974_a, "Estimation of Rainfall Erosion Index," Journal of Irrigation and Drainage Division, ASCE, IR-3, pp 293-307.
- Hotes, F. L., et al., Comparative Costs of Erosion and Sediment Control, Construction Activities, U.S. Environmental Protection Agency (R.E. Thronson Project Officer), EPA-430/9-73-016, 205 pp.
- Wischmeier, W.H., and Smith, D.D., 1965, Predicting Rainfall Erosion Losses from Croplands East of the Rocky Mountains, Guide for Selection of Practices for Soil and Water Conservation, USDA, Agricultural Research Service in cooperation with Purdue Agricultural Experiment Station, Agriculture Handbook No. 282, 47 pp.
- Wischmeier, W.H., et al., 1971, "A Soil Erodibility Nomograph for Farmland and Construction Sites," Journal of Soil and Water Conservation, pp 189-193.

SOIL RIPPING TREATMENTS FOR RUNOFF AND EROSION CONTROL

by Earl F. Aldon, Principal Hydrologist, Rocky Mountain Forest and Range Experiment Station, Albuquerque, New Mexico

ABSTRACT

Soil ripping, a form of deep, wide plowing, effectively reduces runoff and erosion from semi-arid watersheds. Surface runoff was still reduced 85 percent and erosion 31 percent 3 years after treatment. Treatment effectiveness declined after 3 to 5 years depending on amounts and intensities of summer thunderstorms. Opening old rips that were sealed over with sediment was as effective in controlling runoff as ripping between old rips. Reripping by either method reduced runoff by two-thirds compared with untreated check plots. Ripping caused a favorable shift in forage production from galleta to alkali sacaton. Ripping effects on runoff are short-lived, but forage production patterns may persist for 10 years. Ripping must be done carefully to avoid subterranean channel formation (erosion by soil piping).

INTRODUCTION

Soil ripping is a form of deep plowing to break up soil for increased moisture penetration. A large crawler-type tractor pulls a plow-like device, consisting of two chisels about 2.2 m (7 feet) apart, along the contour. As the device is pulled through the ground, each tooth cuts a furrow about 10 cm (4 inches) wide and 71 cm (28 inches) deep. Two vertical triangular plates mounted near the top of the teeth open the furrow to about 38 cm (15 inches) at the surface.

This treatment has been used by land managers on highly erosive soils in the Rio Puerco drainage of New Mexico to hold moisture where it falls, thereby increasing plant growth and decreasing erosion.^{1/} The study reported in this paper was done on and near the San Luis Watersheds on the Rio Puerco drainage, 93 km (58 miles) northwest of Albuquerque, New Mexico, in the transition zone between woodland and semi-desert grassland. The area is comprised of mesas or uplands, steep rocky breaks, and alluvial grasslands.

The headwaters of the 7.8 sq km (3 square mile) San Luis Watersheds originate on mesas that break off into steep, rocky slopes. These breaks give way to rolling foothills that merge with the alluvial bottoms. A layer of Mesa Verde Sandstone overlies Mancos shale.

^{1/} Research reported here was conducted in cooperation with the Bureau of Land Management, U.S. Department of the Interior, Albuquerque, New Mexico.

The sandstone breaks and underlying shales form the parent soil material, the texture of which varies from sandy loams to silty clays. The study areas range in elevation from 1828 to 2133 m (6,000 to 7,000 feet), and is typical of the larger semi-arid area in northwestern New Mexico.

The principal perennial grasses on the study sites are alkali sacaton (*Sporobolus airoides* (Torr.) Torr.), galleta (*Hilaria jamesii* (Torr.) Benth.), and blue grama (*Bouteloua gracilis* (H.B.K.) Lag.). Shadscale saltbush (*Atriplex confertifolia* (Torr. and Frem.), other saltbushes (*Atriplex* spp.) and big sagebrush (*Artemisia tridentata* Nutt.) are the most common shrubs. Some pinyon pine (*Pinus edulis* Engelm.) and juniper (*Juniperus* spp.) trees and cholla cactus (*Opuntia* spp.) are scattered over the area.

Grazing on the San Luis Watershed area has been overwinter only (November–April) since 1958. Numbers are adjusted to achieve 55 percent utilization of alkali sacaton (Aldon and Garcia, 1967).

Average annual precipitation is close to 25 cm (10 inches). Average growing-season precipitation (May 1–November 1) is around 15 cm (6 inches). High-intensity convective thunderstorms characterize the summer storm period (July–August).

PLOT STUDIES

To evaluate the effect of ripping on runoff and erosion, Hickey and Dortignac set out 32 specially designed runoff plots (Garcia et al, 1962) and followed the effects of precipitation on ripping for 3 years (Hickey and Dortignac, 1964). Peak 3-minute intensities of storms producing runoff during this study varied from 36 to 114 mm (1.4 to 4.5 inches) per hour. Lower intensity storms produced runoff when they were of sufficient duration to wet the soil amply. Small storms of greater intensity tended to seal the soil surface and thus produced runoff.

Runoff from untreated plots averaged 60 percent of precipitation.

Treatment was 100 percent effective in reducing runoff for the first storm. Overall effectiveness was 96 percent at the end of the first year. Only one storm occurred on the site during the second year, a 63 mm (2.48-inch) rain with a peak intensity of 36 mm (1.4 inches) per hour. Runoff was reduced 69 percent. Third-year effectiveness averaged about 85 percent. Over the three-year period, effectiveness of ripping declined approximately 15 percent.

The amount and duration of rainfall in excess of 6.4 mm (0.25 inch) per hour were significantly related to surface runoff.

Reduction in erosion due to ripping was 100 percent for the first rain, and averaged 85 percent for the first year. Reduction for the only storm the second year was 65 percent. Third year of study ended with 31 percent reduction in erosion.

The authors concluded that soil ripping is highly effective in reducing surface runoff, and that ripping is effective in reducing erosion on silt-loam soils, but not as effective on clay soils.

Because of the risk of increasing soil piping, ripping is not a suitable land treatment practice on a shale soil that is so shallow that the ripping operation penetrates to the parent material.

WATERSHED STUDY

Two of the San Luis Watersheds were paired for studying the effect of ripping over a large area (Aldon, 1966). Ripping was done on the contour on alluvial bottoms and other workable areas (slopes less than 5%) on one 191 ha (471 acre) watershed. The other 225 ha (555 acre) watershed was used as a control. During the 3 pre-treatment years, the watershed to be ripped consistently produced more runoff than the control watershed. For the first year after the ripping, the treated watershed yielded no measureable runoff, while runoff from the control was about average. Runoff from the treated watershed was about half that from the control the second year. These differences could not be analyzed statistically, but they suggested the ripping treatment effectively reduced runoff. Treatment effectiveness has diminished gradually, as expected, since the third year.

EFFECTS ON VEGETATION

Ground cover has been improving on the treated watershed since grazing management was started (Aldon and Garcia, 1967). Alkali sacaton and galleta occurred about the same number of times on the watershed. The ripping treatment caused a temporary 28 percent decline in perennial grass cover. This decline was about equal for each of the three principal species (Aldon, 1966).

Total forage production has also been improving on the areas since grazing management was started (Aldon and Garcia, 1967) in spite of variable precipitation. When the percent contribution of each species to total forage was graphed, a marked response to the ripping treatment was apparent (Aldon and Garcia, 1972). Prior to ripping, galleta provided over half of the total forage production. The untreated watershed shows about this same relationship. Since treatment, alkali sacaton has produced over half of the forage. The trend since treatment, however, is to revert back gradually to the original relationship.

Blue grama has contributed about 10 percent of the total forage throughout the 15 years of study.

Individual alkali sacaton plants produced more forage as a result of treatment. The treatment's effectiveness for runoff and erosion control was short-lived on this watershed, but its effect on plants--a favorable shift in forage importance from galleta to alkali sacaton--has continued for 7 years. It may take another 3 to 5 years until the original relationship is re-established.

RE-RIPPING EFFECTS

Since ripping is a short-lived treatment, a test was conducted to determine if effectiveness of old ripped areas could be restored by re-ripping. The question to be answered was: would runoff and erosion be reduced as much if old rips were re-opened (re-ripping with lower horsepower requirements), or should new rips be made in the spaces between old rips?

The study site had been ripped 5 years prior to re-treatment. Three blocks consisting of five runoff plots in each block were constructed. Each block contained three treatments randomized within blocks as follows: two plots where the old rip was opened (re-ripped), two plots where new rips were made between old rips, and one check plot where no new ripping was done. This design would detect a 30 percent difference in mean treatment responses. Runoff plot construction has been described.

Re-ripping was highly effective in reducing surface runoff and erosion (Aldon, 1972). Runoff was reduced by at least two-thirds on the treated plots compared with the untreated check plots. Storm events the first year had intensities less than 6.4 mm (0.25 inch) in 15 minutes, and were in amounts less than 16.5 mm (0.65 inch), which may account for the lack of runoff the first year. Since check plots failed to produce runoff also, initial ripping may still have had some effectiveness. In the second year, only one storm event, the only storm with an intensity exceeding 6.4 mm (0.25 inch) in 15 minutes, produced runoff. In the third year, all four runoff events occurred in weeks where sizable amounts of precipitation fell, and when storm intensities were in excess of 6.4 mm (0.25 inch) in 15 minutes.

The ripping treatment reduced sediment from the larger storms by two-thirds.

SOIL SUBSTRATE

The following characteristics of soils from the treated areas may help others to better judge the applicability of these study findings to other areas.

The sites are generally classed as alluvial land with a pH (paste) of 8.0, saturated conductivity of 0.96 mmho/cm, exchangeable sodium of 2.7%, organic matter of 2.8%, sand 47%, silt 36%, and clay 17% (Campbell, 1968). Some Christianburg clay soil type also is present, with a pH of 7.8, saturated conductivity of 0.61 mmho/cm, exchangeable sodium 2.1%, organic matter 1.2%, sand 29%, silt 34%, and 37% clay, classed as a clay loam.

The surface layer of alluvial land commonly is about 25 cm (10 inches) thick. It is very pale brown, contains some free lime, and has granular structure. The texture ranges from loamy fine sand to loam. The subsoil is light yellowish brown, structureless, and slightly more compact than the surface layer. It consists of stratified loam, sandy loam, and clay loam derived from alkaline shale and sandstone and deposited by streams (Folks and Stone, 1968).

The surface layer of Christianburg series soils consists of grayish-brown silty clay to clay that has granular structure. It contains little lime. The subsoil is a very thick layer of grayish-brown clay that has a coarse prismatic structure. Coarser textured strata occur in some places. This layer is hard and dense, and in many places has pronounced vertical cracks resulting from shrinkage (Folks and Stone, 1968).

DISCUSSION

Soil ripping is an effective but short-lived treatment to reduce runoff and erosion on semi-arid watersheds. Re-ripping old rips can extend the treatment effectiveness a few more years. There is an initial loss of vegetation, but the treatment can result in a favorable shift in forage production from galleta to alkali sacaton. These changes may persist for 10 years.

When these studies were conducted, ripping cost between \$8 and \$10 per ha (\$4 and \$5/acre). Costs are higher now, and the need for fuel conservation may preclude largescale use of this treatment on any but high-value areas. Since the treatment is short-lived, management practices to enhance vegetation establishment and recovery are needed to permanently reduce runoff and erosion from semi-arid areas. There is always a high risk in these areas that ripping may result in soil piping, and precautions should be taken to avoid this type of subterranean channel formation.

REFERENCES

- Aldon, Earl F. 1966. Deferred grazing and soil ripping improves forage on New Mexico's Rio Puerco drainage. N. Mex. Stockman 31(11): 44-46.
- Aldon, Earl F. 1972. Reactivating soil ripping treatments for runoff and erosion control in the Southwest. Annals of Arid Zone 11(3-4): 154-160.
- Aldon, Earl F. and George Garcia. 1967. Summer deferred grazing can improve deteriorated semi-desert ranges. U.S. Forest Serv. Res. Note RM-95, 3 p. Rocky Mt. Forest and Range Exp. Stn., Fort Collins, Colo.
- Aldon, Earl F. and George Garcia. 1972. Vegetation changes as a result of soil ripping on the Rio Puerco in New Mexico. J. Range Manage. 25: 381-383.
- Campbell, R. E. 1968. Production capabilities of some upper Rio Puerco soils of New Mexico. U.S. Forest Serv. Res. Note RM-108, 7 p. Rocky Mt. Forest and Range Exp. Stn., Fort Collins, Colo.
- Folks, J. J. and W. B. Stone. 1968. Soil Survey, Cabezon Area, New Mexico. U.S. Soil Conserv. Serv., Albuquerque, New Mexico. 44 p.
- Garcia, George, Wayne C. Hickey, Jr., and E. J. Dortignac. 1963. An inexpensive runoff plot. U.S. Forest Serv. Res. Note RM-12, 8 p. Rocky Mt. Forest and Range Exp. Stn., Fort Collins, Colo.
- Hickey, Wayne C., Jr. and E. J. Dortignac. 1964. An evaluation of soil ripping and soil pitting on runoff and erosion in the semiarid Southwest. Internatl. Union Geodesy and Geophys., Internatl. Assoc. Sci. Hydrol., Land Erosion, Precipitation, Hydrometry, Soil Moisture Comm. Pub. 65: 22-23.

SOIL AND WATER LOSS FROM IMPOUNDMENT TERRACE SYSTEMS

By John M. Laflen, Agricultural Engineer, ARS, USDA, and H. P. Johnson, Professor, Agricultural Engineering, Iowa State University, Ames, Iowa.

ABSTRACT

Impoundment terrace systems greatly reduce sediment discharge from agricultural watersheds. Graphs are presented for selecting orifice size and predicting sediment discharge. Construction and operation problems with impoundment terraces are described, as well as methods for solving these problems.

INTRODUCTION

Terraces effectively control sediment loss from agricultural lands. They are usually classified according to alignment, cross-section, grade, or outlet. Terrace outlets for controlling runoff water are surface channels, subsurface conduits, or infiltration and percolation to underlying strata. The impoundment terrace (also called "tile-outlet terrace") outlets are subsurface conduits, and infiltration and percolation to underlying strata. Figure 1 is a sketch of an impoundment terrace.

Our objective is to present data needed to design impoundment terraces that will drain in a specified time with a predictable sediment discharge. Some common design, construction, and operational problems observed with impoundment terraces are described and their solutions are discussed.

DISCHARGE OF WATER FROM IMPOUNDMENT TERRACES

In the impoundment terrace system, runoff water from the area between terraces is stored temporarily in pondage areas around risers that release the water to underground conduits. Several pondage areas from terraces constructed at different elevations usually drain into the same underground conduit. Understanding the hydraulics of water discharge from impoundment terraces is necessary to design systems that drain soon enough so as not to impede farm machinery operation and to avoid damage to crops and the system components.

Contribution from North Central Region, Agricultural Research Service, U. S. Department of Agriculture and Journal Paper J-8255 of the Iowa Agriculture and Home Economics Experiment Station, Ames, Iowa. Project 2058.

The water discharge system is usually designed so that no underground conduit flows under pressure. The underground conduit is usually a conventional tile line. Water pressure gradients causing high velocities in the conduit may damage the conduit, and water may flow from upper terraces into lower terraces. The American Society of Agricultural Engineers (ASAE)(1975) recommended orifices to control flow rates from individual terraces when two or more pondage areas drain into the same conduit. Orifices are sized to discharge the maximum design rate from the drainage area (usually 2.5 to 5.0 cm/day, 1 to 2 in/day) at the water depth in the pondage area when the design runoff volume is stored. The design runoff volume is usually that runoff volume expected for a selected recurrence interval. Rochester and Busch (1974) have presented alternative means of selecting design storms. Rochester and Busch (1975) have also simulated conduit flow conditions for two impoundment terraces without orifice control.

The orifice plate which controls flow into the conduit is laid horizontally below the lowest slot (opening) in the riser. If the riser slots do not restrict flow, and the orifice plate is at the bottom of the pondage area, discharge from the pondage area through the orifice can be expressed as

$$Q = CD^{0.5} \quad [1]$$

where C is a coefficient (a function of orifice area, orifice discharge coefficient, and gravity), and D is depth of water above the orifice plate.

The time required to drain a pondage area is needed to minimize crop damage within the pondage area. This time can be computed by integrating the equation

$$A \frac{dD}{dt} = -Q - IA \quad [2]$$

where A is ponded surface area, D is water depth, t is time, Q is discharge rate, and I is infiltration rate.

The ponded surface area can be expressed as

$$A = aD^b \quad [3]$$

where a and b are constants. Laflen (1972) found a values of 1930, 2830, and 6870, with corresponding b values of 1.29, 1.11, and 1.73 for three impoundment terraces in Iowa. Water surface areas at 0.5 m (1.6 ft) depth at the riser were 790, 1310, and 2070 m² (8500, 14100, and 22300 ft²). Rochester and Busch (1974) found 1.77 was a good b value for a pondage area; they had hypothesized that the pondage area volume was approximately described by an inverted, three-sided pyramid, b would then be 2.00. Haan and Johnson (1967) found that equation 3 expressed the relationship between depth and area for depressions in Iowa.

Substituting equation 3 into equation 2 and rewriting,

$$\frac{dD}{dt} = - \frac{CD^{0.5}}{aD^b} - I. \quad [4]$$

If infiltration is zero, integration of equation 4 yields

$$T_f = \left[\frac{a}{C} \right] \frac{D_o^{(b+0.5)}}{(b+0.5)} \quad [5]$$

where T_f is time required to drain the temporarily stored water and D_o is the initial depth of water. Equation 5 gives the maximum time required for a pondage area to drain after inflow ceases.

Equation 4 is not easily integrated for most conditions when infiltration rate is not zero. Numerical or analog methods can be used to obtain required solutions. Equation 4 is easily integrated when b is 1.5. Then, time to drain can be written as

$$T_f = \frac{D_o}{I} - \frac{C}{aI^2} \ln \left[1 + \frac{a}{C} I D_o \right]. \quad [6]$$

Initial depth can be written as

$$D_o = [V(b+1)/a]^{1/(b+1)} \quad [7]$$

where V is initial volume of water ponded.

Time required to drain the water pondage area as a function of initial depth is shown in Figure 2 for a wide range of ratios of $a:C$, for infiltration rates of 0 and 0.64 cm/hr (0.25 in/hr) and for $b = 1.5$. Figure 2 shows that, as discharge decreases relative to volume stored (a/C increases), the effect of infiltration on duration of ponding increases. For example, for an initial depth of 1 m (3.3 ft) an a value of 2,000, and a C value of 40 ($a/C = 50$), the pondage area drains in about 21 hours with 0.64 cm/hr (0.25 in/hr) infiltration rate, and in about 25 hours with no infiltration. If a/C equals 200, a remains the same, and C equals 10, the times required to drain would be 57 and 100 hours. In both instances, the initial volume stored is about 800 m³ (28000 ft³) with a surface area of 2,000 m² (21500 ft²) at a depth of 1 m (3.3 ft). Discharge at maximum depth is 40 m³/hr (1400 ft³/hr) for $a/C = 50$, but is only 10 m³/hr (350 ft³/hr) for $a/C = 200$.

Infiltration of ponded water within the pondage area can significantly reduce the volume of water discharged. As shown in Figure 3, with 200 m³ (7000 ft³) initially stored, an infiltration rate of 0.64 cm/hr (0.25 in/hr) a C of 51 and an a of 5520, about 16% of the initial volume is infiltrated. Doubling of the infiltration rate results in infiltration of 27% of the initial volume of water. Holding the infiltration rate at 0.64 cm/hr (0.25 in/hr) but reducing

C to 13 would result in infiltration of 42% of the initial volume of water. The data in Figure 3 were generated by using numerical integration methods, and were based on the assumptions that infiltration rate was constant with no additional inflow after discharge began. Infiltration is expressed as a percentage of the volume in the pondage area when discharge begins.

The assumption that riser slots do not restrict flow rates is not always valid. Laflen (1972) derived an expression for flow through a series of equal area slots spaced a distance, L, apart along a riser pipe. This expression can be written as

$$Q_s = \frac{2}{3} C_s \sqrt{2g} D^{0.5} a_s \frac{D}{L} \quad [8]$$

where Q_s is the sum of flow through all slots in the riser pipe, C_s is a common discharge coefficient for the slots, g is gravity, D is water depth, and a_s is the area of each slot. Flow through the slots must be greater than flow through the orifice if the riser does not restrict flow. Equation 1 can be rewritten as

$$Q_o = C_o a_o \sqrt{2g} D^{0.5} \quad [9]$$

where Q_o is flow through the orifice, C_o is a discharge coefficient for the orifice, and a_o is orifice area. If flow is not restricted by the riser pipe, Q_o must be less than Q_s . Assuming that $C_o = C_s$, slot area, a_s , and spacing, L, must be selected so that

$$\frac{2}{3} \frac{a_s}{a_o} \frac{D}{L} \geq 1 \quad [10]$$

or flow will be restricted by the riser pipe. The depth, D, to use in equation 10 should be less than 1/2 of D_o to assure that the riser pipe will not control flow rates over an appreciable period of the total discharge time. When slots are of equal area but are not distributed uniformly along the riser, the average spacing of slots along the riser can be used with little error.

SOIL LOSS FROM IMPOUNDMENT TERRACES

Few field measurements of soil loss from impoundment terraces have been made. Laflen et al. (1972) reported measured soil losses from four terrace systems in Iowa. They used the Universal Soil-Loss Equation (Wischmeier and Smith, 1965) to estimate erosion between terraces and determined the ratio of soil discharged from the terrace system to soil eroded between terraces (soil-loss ratio). They reported an average ratio of 0.045. Soil-loss ratios for the most severe storm at each location ranged from 0.02 to 0.12. Average annual soil loss did not exceed 900 kg/ha (800 lb/A) at any of the

four locations. Owen and Wittmuss (1969) reported a loss of 903 kg/ha (806 lb/A) from a terraced watershed in Nebraska in 1967, compared with 10,700 kg/ha (9550 lb/A) from a comparable watershed without terraces. In 1971 in Nebraska (Fisher and Lane, 1973), soil losses of 67 and 252 kg/ha (60 and 225 lb/A) were measured from two impoundment terraced areas. A nearby conventionally terraced area lost 460 kg/ha (410 lb/A), and an area without treatment lost 24,021 kg/ha (21434 lb/A).

Laflen (1972) developed a method for mathematically modeling sedimentation in water pondage areas. The model defines the elevation within the ponded water of the uppermost sediment particles of a particular size, for each sediment size of interest, with time. Stokes Law is used to compute the fall velocity of the sediments. The model is designed so that the concentration of a particular size of sediment in the discharged water is equal to its initial concentration until the uppermost sediment of that size reaches the bottom of the pondage area. Also assumed is that, at some initial time, there is a homogeneous sediment-water mixture in the pondage area and no inflow to the terrace storage after that time. Laflen, Johnson, and Hartwig conducted studies to test the model^{1/} that was used to predict the effect on sediment discharge of factors like the depth-discharge relationship in a pondage area, shape of the pondage area, initial volume of runoff water, and size distribution of the sediment eroded to the pondage area^{1/}. The relationship of these factors to sediment discharge is shown in Figures 4a to 4f.

Graphs presented in Figure 4 show the reasoning involved in estimating the effects of various factors on sediment discharge from impoundment terraces. Sediment discharge is expressed in Figure 4 as a delivery ratio, D_r , the ratio of sediment discharged from the pondage area to sediment delivered to the pondage area, expressed as a percentage. Also, sediment sizes are assumed to be log-normally distributed. The parameters used, μ and σ , are, respectively, the natural log of the size (microns) for which 50% of the material is finer, and the difference between μ and the natural log of the size for which 15.9% of the material is finer. As shown in Figure 4a, when water discharge rates are high relative to volume of water stored (C is large), sediment discharge is increased. If the ponded water is shallow because of the shape of the pondage area (a is large), sediment discharge is reduced (Figures 4d and 4e). Discharge rate affects the loss of fine materials but has little effect on loss of material in the large silt-sized range (Figure 4c). Infiltration rate affects sediment discharge, but the effect is interrelated with the other factors. Nearly all very fine sediments will discharge with the water. Coarse materials settle rapidly; hence, size distribution has a pronounced effect on sediment discharge (Figure 4f).

^{1/} Unpublished data.

Data shown in Figure 4 can be used to predict sediment delivery from impoundment terraces. For example, if a pondage area has been surveyed and it has been determined that $a = 10000$ and $b = 1.5$, assuming infiltration is zero, initial volume is 500 m^3 (17700 ft^3), and C is 25, then, entering Figure 4e along a vertical line above $a = 10000$, a delivery ratio of between 6 and 7% would be estimated. Since a/C is about 400, and initial depth is about 0.4 m (1.3 ft), time to drain would be about 39 hours (equation 5 or Figure 2). If C were doubled, D_r would be increased to about 8 or 9% (by interpolating along vertical line above $a = 10000$ in Figure 4e) and time to drain would decrease by half. If initial volume were raised to 1000 m^3 (35000 ft^3) (and $C = 25$), D_r would be about 5% (vertical line in Figure 4d from $V = 1000$ to line where $a = 11040$, $I = 0$, $C = 26$). If infiltration rate were 1.27 cm/hr (0.5 in/hr) rather than zero, D_r would be less than 3%.

In some cases the assumption that runoff occurs during short periods (storage area fills instantaneously) may be seriously violated. In regions where most erosion is caused by short-duration, high-intensity thunderstorms, Figure 4 may be used with little error.

IMPOUNDMENT TERRACE PROBLEMS

Several problems with impoundment terraces have been observed. Most of these problems can be overcome by good design and construction.

Terraces may overtop due to runoff exceeding the design runoff which will cause some damage during overtopping. Terraces should be repaired as soon as possible after overtopping.

Earth slides have been observed on steep backslopes of impoundment terraces after prolonged heavy rainfall periods. The ASAE (1975) recommends that the backslope of terraces be no greater than 2:1 and that they be grassed.

A problem frequently observed is piping along the conduit beneath the terrace. Generally, this happens because the fill around the conduit was not packed sufficiently. Proper compaction and soil moisture control will reduce the probability of piping. Sheet metal cutoff walls are seldom used but would help to control seepage.

Plugging of the slots in the riser, or plugging of the orifice have been observed. We have found narrow riser slots (0.3 by 2.5 cm) (1/8 by 1 in) plugged with fine trash. Loose cattle hair has plugged orifices. The solution recommended for the plugging problem is larger slots in the riser and occasionally checking orifices for plugging. Plugging or partial plugging of orifices or riser slots results in crop loss within the pondage area.

Underground conduit failure due to high pressure in the conduit is uncommon but has been observed. The ASAE recommended (1975) using an orifice to control conduit pressure by controlling flow. Properly selecting orifice sizes to provide timely drainage and sizing the conduit to carry expected flow will eliminate these failures unless the underground conduits plug. Few of these failures have been observed, even for systems that have no orifices, because usually the slots in the riser pipe have restricted flow to rates below that of the conduit design capacity. In one instance in Iowa where inlet pipes were constructed with large slots with no orifices, there was a conduit blowout (tile line). Lower terraces overflowed while upper terraces drained. Measured flow rates were about 4 to 5 times greater than expected. Conventional placement of circular orifices would have been difficult after construction was completed; the problem was solved by inserting a circular plate of smaller diameter than the inlet pipe below the lowest slot normal to the longitudinal axis of the riser. This plate was held in position by a steel rod welded to the plate and hooked over the top of the riser pipe. Water flows between the horizontal plate and the riser pipe and flow rates were controlled at near design levels.

Pondage areas that drain too slowly, or that are constructed on soils with a low permeability may cause crop damage or considerably delay farming operations. Sometimes, better orifice sizing may reduce the problem or tile drainage of the pondage area may be required.

Roberts (1972) described problems with impoundment terraces in potato-growing areas of New Brunswick, Canada. Ponding of water on some soils made farm machinery operation difficult and resulted in a complete loss of a potato crop within the pondage area. Water soaked potatoes were a source of infection for other potatoes during storage, and potato losses were substantial. He reported that impoundment terraces perform best when soils were deep and uniform in texture, and crops were on stalks above the water level, and harvested under dry conditions.

OFFSITE EFFECTS

Sediment discharge from agricultural lands can be significantly reduced when impoundment terraces are installed. Reduced sediment loss will reduce sediment deposition on other lands and in private and public water bodies. Public agencies have partly funded the construction of impoundment terraces on some small watersheds in Iowa to protect public reservoirs. McCarty (1967) described one case.

The effect of impoundment terraces on reducing peak flow rates has received little attention. Generally, the design discharge rate for an impoundment terrace is less than $25 \text{ m}^3/\text{hr}/\text{ha}$ ($0.1 \text{ cfs}/\text{Acre}$). For

small areas, this will be 1 to 10% of the peak rate without impoundment terraces. The effect on peak rate of runoff from larger areas may also be significant, but predicting the effect requires extensive analysis.

SUMMARY

Impoundment terraces can significantly reduce sediment delivery from watersheds. Benefits from reduced peak runoff rates can be expected.

Hydraulic design of the water-discharge system is required if the system is to function properly. Orifices should be used to control flow; underground conduit size should be based on flow through the orifices. Slots in the riser pipe should be large enough so that the orifice controls discharge from the terrace, but small enough to perform some screening.

Impoundment terrace problems are prevented when systems are properly constructed and maintained on well-drained soils. Graphs are presented to enable prediction of sediment discharge and duration of water pondage for most field conditions encountered.

REFERENCES

- American Society of Agricultural Engineers. 1975. Design, layout, construction, and maintenance of terrace systems - ASAE recommendation R268.1. In: Agricultural Engineers Yearbook, 576-579.
- Fisher, W. F., and D. E. Lane. 1973. Till planting. In: Proc. of Conservation Tillage Conference, 187-194.
- Haan, C. T., and H. P. Johnson. 1967. Geometrical properties of depressions in north central Iowa. Iowa State J. Sci. 42(2):149-160.
- Laflen, J. M. 1972. Simulation of sedimentation in tile-outlet terraces. Ph.D. Thesis. Iowa State Univ.
- Laflen, J. M., H. P. Johnson, and R. C. Reeve. 1972. Soil loss from tile-outlet terraces. J. Soil Water Conserv. 27(2):74-77.
- McCarty, C. A. 1967. Development of a terrace program in the Pine Lake Watershej. In: Soil and America's Future. Proc. 22nd Annual Meeting. 139-141. Soil Conserv. Soc. Am.
- Owen, W. J. and H. D. Wittmuss. 1969. Hydrologic effects of soil conservation practices. Paper 69-224, Am. Soc. Agric. Eng.
- Roberts, J. A. 1972. Evaluation of parallel terraces with tile outlets. Engineering research and development in agriculture. Engr. Res. Serv., Canada Agriculture, 17:9-10.
- Rochester, E. W. and C. D. Busch. 1974. Hydraulic design for impoundment terraces. Trans. Am. Soc. Agric. Eng. 17(4):694-696, 700.
- Rochester, E. W. and C. D. Busch. 1975. Digital simulation of impoundment terrace hydraulics. Paper 75-2038, Am. Soc. Agric. Eng.
- Wischmeier, W. H. and D. D. Smith. 1965. Predicting rainfall-erosion losses from cropland east of the Rocky Mountains. U. S. Dept. of Agric., Agric. Handb. 282.

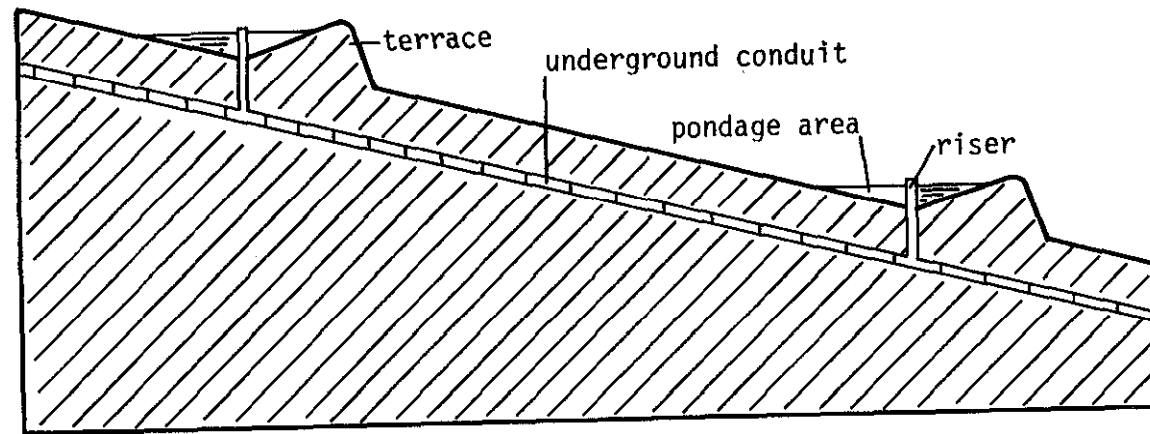


Figure 1. Sketch of impoundment terrace system.

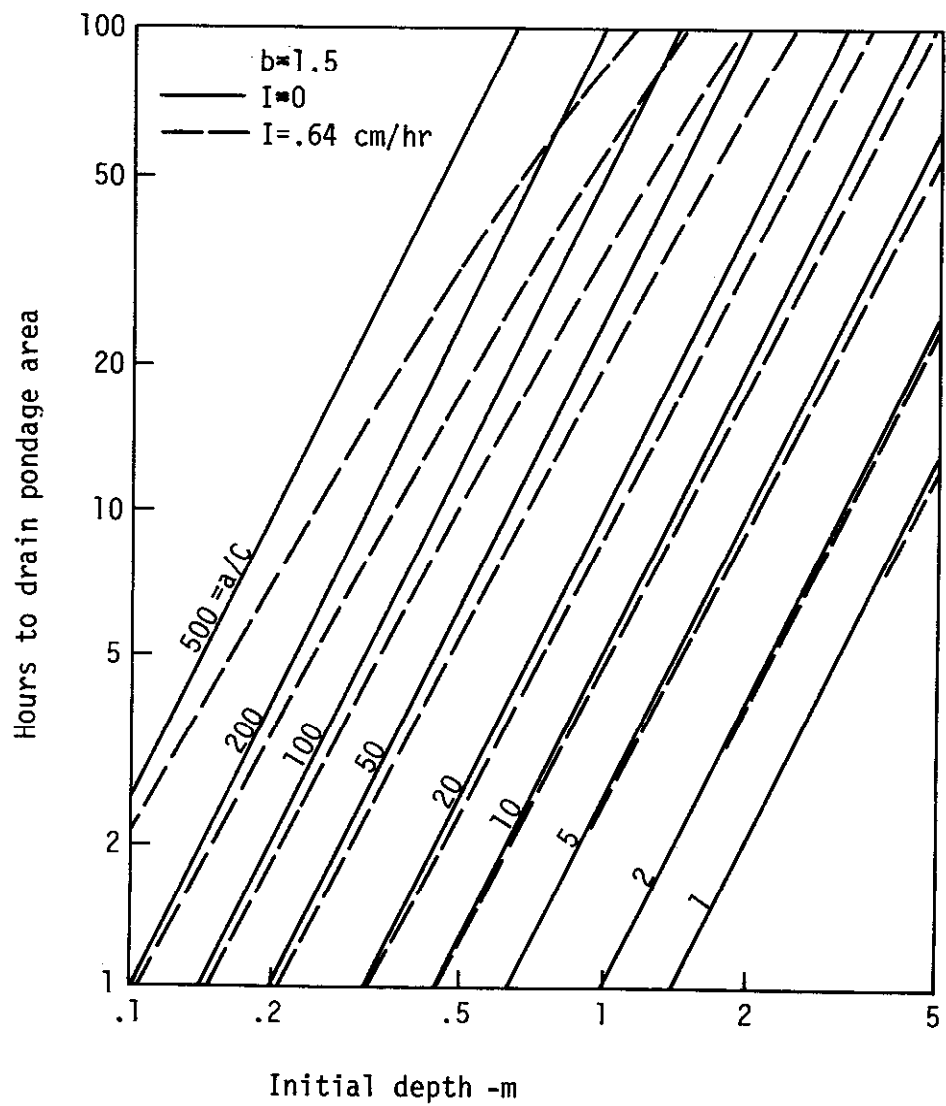


Figure 2. Time required to drain water pondage area for given initial water depths.

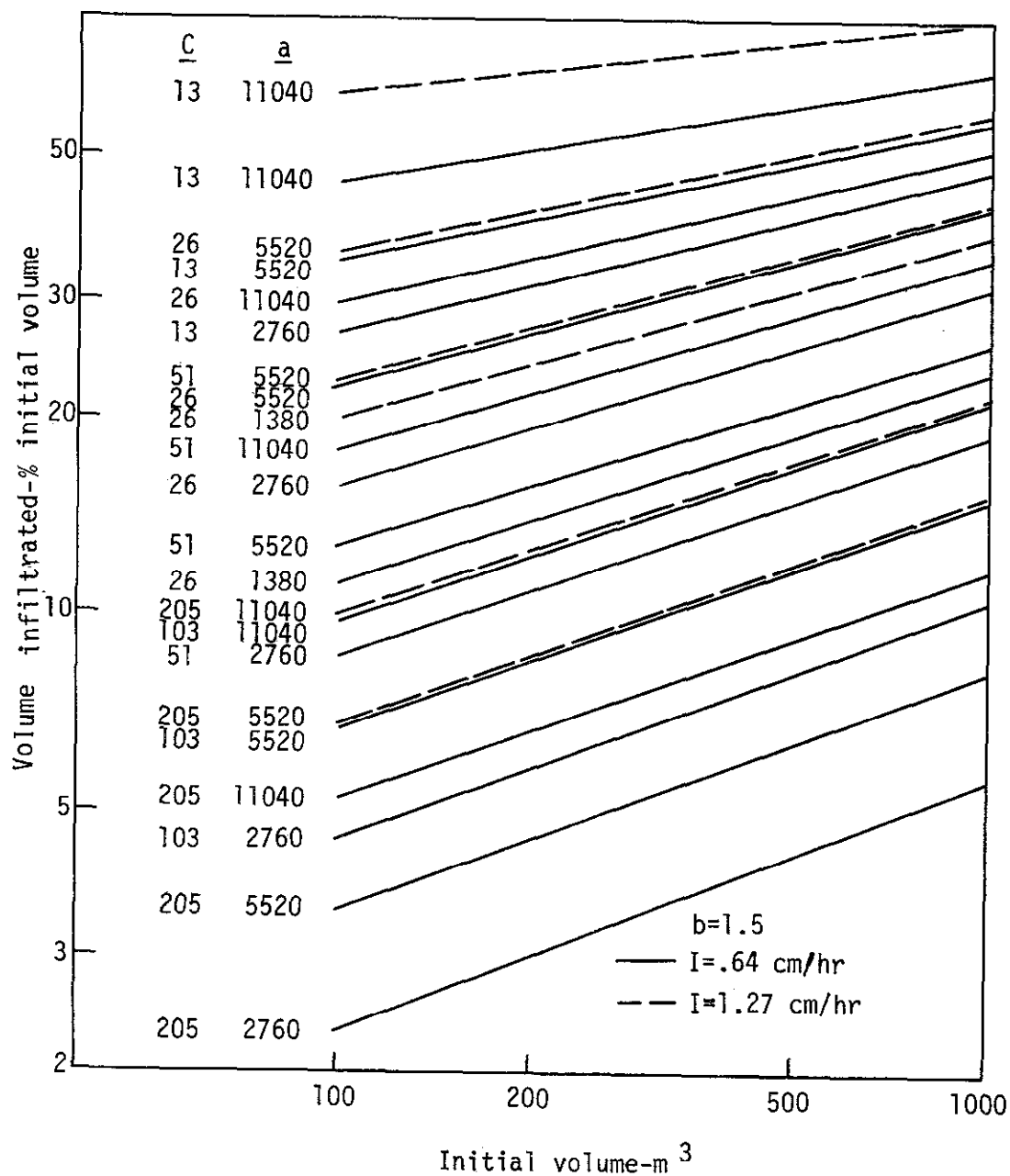


Figure 3. Relationships of water volume infiltrated to initial volume of water stored, infiltration rate, and shape of pondage area.

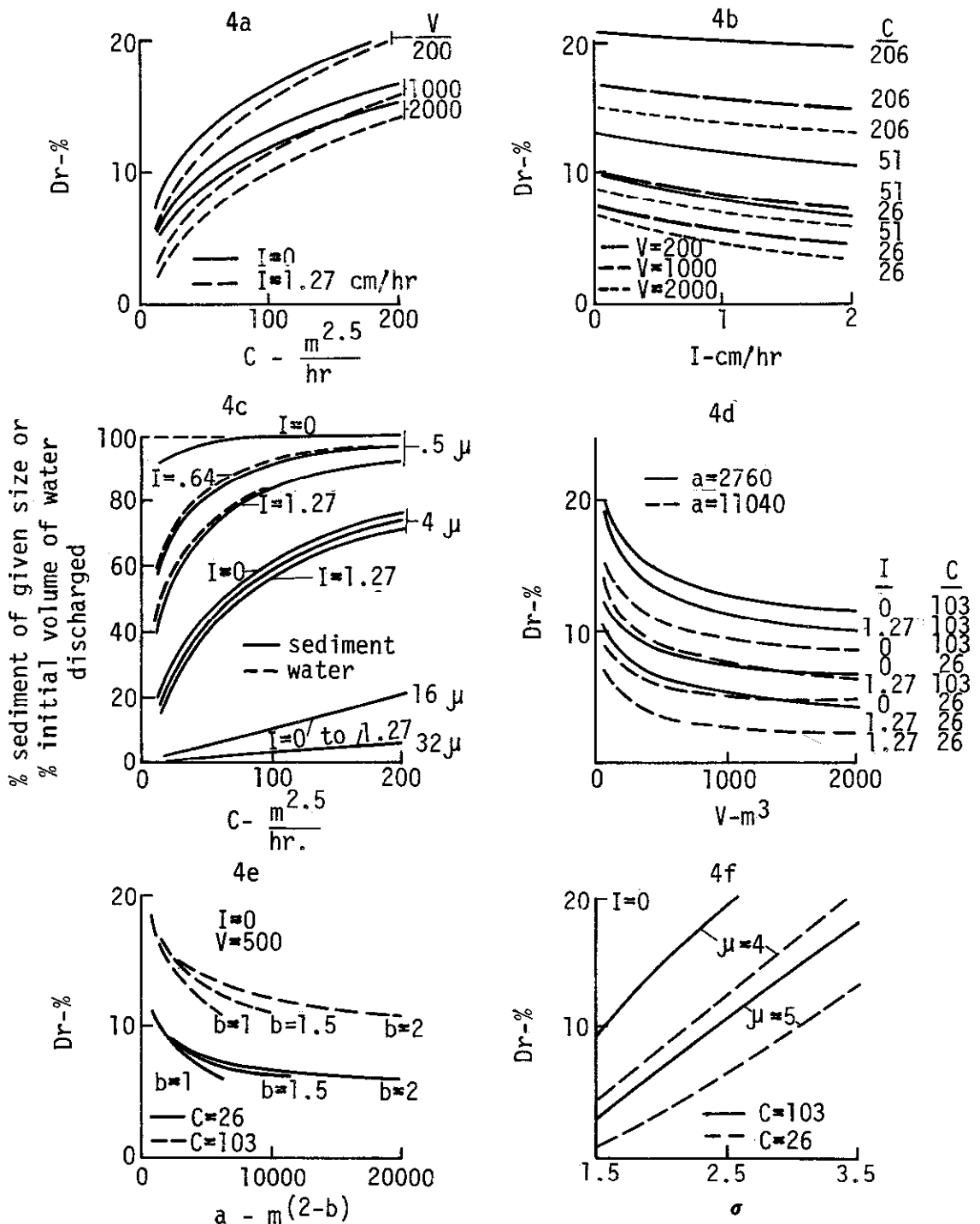


Figure 4. Relationship among hydraulic and terrace geometry factors and sediment discharge from impoundment terraces. Unless otherwise stated, $\mu \approx 4$, $\sigma \approx 2$, $b \approx 1.5$, and $a \approx 2760$. Units are consistent and given as legends above.

EXCAVATED SEDIMENT TRAPS PROVE SUPERIOR TO DAMMED ONES

By Charles P. Tryon, Watershed Scientist, Bruce L. Parsons, Forest Engineer, and Mack R. Miller, Soil Scientist, U. S. Forest Service, National Forests in Missouri, Rolla, Missouri 65401.

ABSTRACT

A decade of experience on large earth moving jobs in the rugged Missouri Ozarks has shown that excavated sediment traps are incomparably superior to small detention dams in terms of cost, industry acceptance, and sediment trap efficiency. This "how to" article is a working man's guide to excavated sediment trap design and construction.

INTRODUCTION

Stilling basins and various other kinds of small impoundments have been used to trap eroded sediment for many years, and numerous publications, people, and agencies seek to increase their use by extolling their merits. Virtually nowhere, however, is the idea of excavated -- rather than impounded -- sediment traps even mentioned, let alone recommended as a superior method of accomplishing the sediment control task.

BACKGROUND

During the 1960's, the world's largest lead mining area was being developed in the heart of Missouri's Clark National Forest. Eight new lead mines, six new ore concentration mills, two new smelters, 44 miles of new railroad, and many miles of new highways and transmission lines were constructed, all involving large areas of earth moving and resultant erosion and sedimentation. Where National Forest lands were involved, the U. S. Forest Service required that the construction agencies and contractors install small dams in the drainageways below earth moving sites to trap sediment before it reached the major streams.

Historically, small sediment entrapment dams have been built of a wide variety of materials -- logs, railroad ties, earth, loose rock, rock-filled gabions, stump and root wads, brush and wire, automobile tires, and concrete, just to name a few. Spillways for these dams have ranged from none at all to simple notches in the crest to more elaborate designs. Some structures have performed quite well, some have not, and many have performed well initially but deteriorated over the years and ultimately failed.

With few exceptions, the small dams built in Missouri's Lead Belt in the 1960's were composed of dumped rock ranging in size from coarse gravel to boulders the size of a desk, with varying amounts of finer soil materials mixed in. The rock usually had to be trucked several miles to the

site, and rough roads had to be pushed in from the construction right-of-way so the trucks could dump their loads directly on the dam site. The rock was then shaped (but not compacted, for semi-permeability) with a bulldozer or front end loader, and a small notch was cut in the top center of the fill to serve as a spillway. Usual dam dimensions were 9-15 meters (30-50 feet) long and $1\frac{1}{4}$ - $2\frac{1}{2}$ meters (4-8 feet) high. Contractors bid them in at \$300-\$800 each, which amounted to a very sizeable expense when many of them were required. Much of the construction industry was very unhappy over the Forest Service's insistence on this added cost.

In addition to the contractor's basic cost and profit margin considerations, at least two other factors probably had a bearing on these high bid prices. One is that few contractors had ever been required to build sediment traps before; they weren't sure what they were getting into, so they padded the price a little for self-protection. The other is that, from both the construction agency's and contractor's perspectives, the traps were perceived as an entirely different job in addition to, rather than an integral part of, the main road, mine, or mill construction job.

By 1970, the shoe was on the other foot. If the Forest Service could demand that others spend their money building sediment traps, it was only fair that the Forest Service also build them at federal expense on their own heavy construction jobs. In addition, it was becoming increasingly evident that the Forest Service's sediment trap design wasn't trapping sediment very well, which was more than a little embarrassing. Dams constructed entirely of coarse material were letting much of the sediment filter on through, and dams constructed of finer materials were breaching. Although they were only intended to function for a year or so until the sediment source areas could be stabilized by revegetation, many of the dams were filling to capacity and overflowing long beforehand. Thus motivated, Forest Service soil scientists, water scientists, and engineers began looking for a better way.

EXCAVATED SEDIMENT TRAPS --- PHILOSOPHY AND DESIGN

The answer was excavated, rather than impounded, sediment traps. Stated in the most meticulous scientific terms, an excavated sediment trap is defined as a "hole gouged out of the ground." Persons involved in design, contract preparation, and construction supervision may find this definition somewhat offensive to their sense of professionalism, but to define them more conventionally defeats their purpose of maximum sediment retention at least cost.

The initial suggestion of excavated, rather than dammed, sediment traps was not immediately accepted by everyone. To gain their acceptance, the first obstacle that had to be overcome was the seemingly inherent human instinct that a dam is somehow more of an "accomplishment" than a hole in the ground, even though the hole in the ground is functionally superior to the dam in every respect. Talk of building dams, even tiny ones, stirs the hearts of men to soaring heights; talk of digging holes

in the ground only stirs visions of backaches. A dam is a definite "something;" a hole in the ground is "nothing" surrounded by dirt. The importance of functional performance often comes out second best to emotional impact.

The second obstacle was some of our engineer's and contractors' preoccupation with symmetry, smoothness, and perfection. Excavated sediment traps require none of these attributes to perform adequately, and insisting on them only increases cost. Mankind is often a creature of habit, and our people had some trouble getting used to the idea that work they would ordinarily consider sloppy was, in fact, professionally superior to their conventional way of thinking.

The working theory of sediment traps, of course, is that they decrease water velocity enough to cause sediment particles of various, sometimes predetermined, sizes to settle out of suspension within a given detention time, and that they have sufficient storage capacity to store the anticipated inflow of settleable particles. The inter-relationships between particle size, velocity, basin length and width, storage requirements, and other factors have been worked out in detail by others (Vetter, 1940; Pemberton, 1971; and Hansen, 1973 and 1975). Although Hansen's articles tell how to design basins in perennially flowing trout streams to trap the sediment after it gets there, the techniques for designing basins on construction jobs to trap the sediment before it gets to the trout streams are exactly the same, and his articles are required reading for the fledgling sediment trap designer. The calculation of exact basin dimensions will not be discussed in detail here.

In Missouri, excavated sediment traps are usually built with straight passes of a bulldozer, merely gouging out a trench one dozer blade wide with vertical sidewalls. Although the spoil can sometimes be used as fill on the main construction job, it is just as often wasted upslope of the sediment trap or in such a way that most of it will end up back in the trap if it erodes. Whether or not the vertical sidewalls cave in after the bulldozer operator gets out of the hole is of no functional significance, although sloping of the sidewalls may be a desirable safety or aesthetic consideration in some locations. Exhibit I illustrates the basic construction concept.

DAMMED AND EXCAVATED SEDIMENT TRAP COMPARISONS

Whereas contractors were bidding in the small dams at \$300-\$800 each, the cost of the excavated traps was \$50-\$150 each.

For most practical purposes, dams can only be built in drainageways; holes can be gouged out of the ground almost anywhere.

Dams require fill material to build them from; holes in the ground require no construction materials at all.

Dams inevitably have more exacting construction requirements than holes gouged out of the ground, thus requiring more construction supervision and resultant higher cost.

In the Missouri Ozarks, where sediment source composition averaged about 20 percent gravel and larger, 30 percent sand, and 50 percent silt and clay (smaller than 0.02 mm.), efficiency of the excavated sediment traps ranged from 75 to 99 percent. In contrast, 75 percent was absolute tops for the dams, and most were in the range of 30-50 percent before breaching.

Whereas many of the dams ultimately breached and lost all or part of their sediment accumulations, there is nothing about a properly located hole in the ground that can fail. Naturally occurring hillslopes and drainageway gradients are made by Mother Nature, not by accident. A dam constitutes an interruption of the natural gradient, and Mother Nature's hydraulic forces will always try to breach it and return the channel to its original configuration, in which case man's sediment entrapment objective is lost. A hole in the ground is also an interruption of the natural hillslope or channel gradient, and Mother Nature's hydraulic inclination is to fill it back in. In this case, however, both man and Mother Nature can have their own way, and everybody is happy.

EXCAVATED TRAP CONSTRUCTION TIPS

Unless it is absolutely unavoidable, traps should never butt directly against the sediment source area. If the source is similar to a steep highway fill, the trap will rapidly fill with very coarse material which rolls as much as it washes downslope, and much retention capacity will be lost unnecessarily. It's better to place the trap some slight distance away from the edge of the source area so that those particles large enough to settle out unassisted on the natural ground slope will do so before reaching the trap.

Excavated traps should empty their overflow onto undisturbed ground in a wide shallow sheet rather than in concentrated channel-type flow. Constructed spillways are usually unnecessary for properly located traps.

Excavated sediment traps can be located on all but the very steepest of slopes by constructing them on the contour where necessary.

For maximum trap efficiency, sediment traps should be located where they will intercept only short duration storm runoff, rather than perennial or other non-sediment laden flow. A trap full to overflowing with water at the beginning of a storm will pass much of the incoming sediment -- particularly the silt and clay fractions -- on through. In contrast, traps which have emptied themselves of water by evaporation and subsurface seepage since the last storm will often retain 100 percent of the sediment laden runoff from the next storm. Up to the point where the cost differential becomes too great, several small traps on the hillside

are more effective than one large trap in a frequently flowing drain-
ageway.

To illustrate how excavated sediment traps can be included in construc-
tion contracts, Exhibits II, III, and IV show construction plans and
specifications from an actual Forest Service road project. For this
contract, excavated sediment traps were administratively divided into
"excavated traps" and "sediment basins" (they could just as easily have
been called roses and raspberries), the former being located far enough
from the construction site that the excavated material was to be wasted,
and the latter being near enough that the excavated material was to be
incorporated into the roadbed. Actual 1971 bid prices were \$50-\$150
for the "excavated traps" and \$50-\$100 for the "sediment basins."
Obviously, individual construction agencies have their own way of doing
things, and there is nothing sacred about the specific words and
sketches in these exhibits.

REFERENCES

- Hansen, E. A., 1973, In-channel Sedimentation Basins -- A Possible Tool
for Trout Habitat Management, *The Progressive Fish Culturist*, 35 (3),
138-142.
- Hansen, E. A., 1975, Sedimentation Basin Design, USDA Forest Service,
Engineering Technical Information System Field Notes, 7(1), 1-5.
- Pemberton, E. L., and Lara, J. M., 1971, A Procedure to Determine
Sediment Deposition in a Settling Basin, USDI Bureau of Reclamation,
Sedimentation Investigations Technical Guide Series, Section E, Part
2, 8 p.
- Vetter, C. P., 1940, Technical Aspects of the Silt Problem on the
Colorado River, *Civil Engineering*, 10, 698-701.

EXHIBIT I

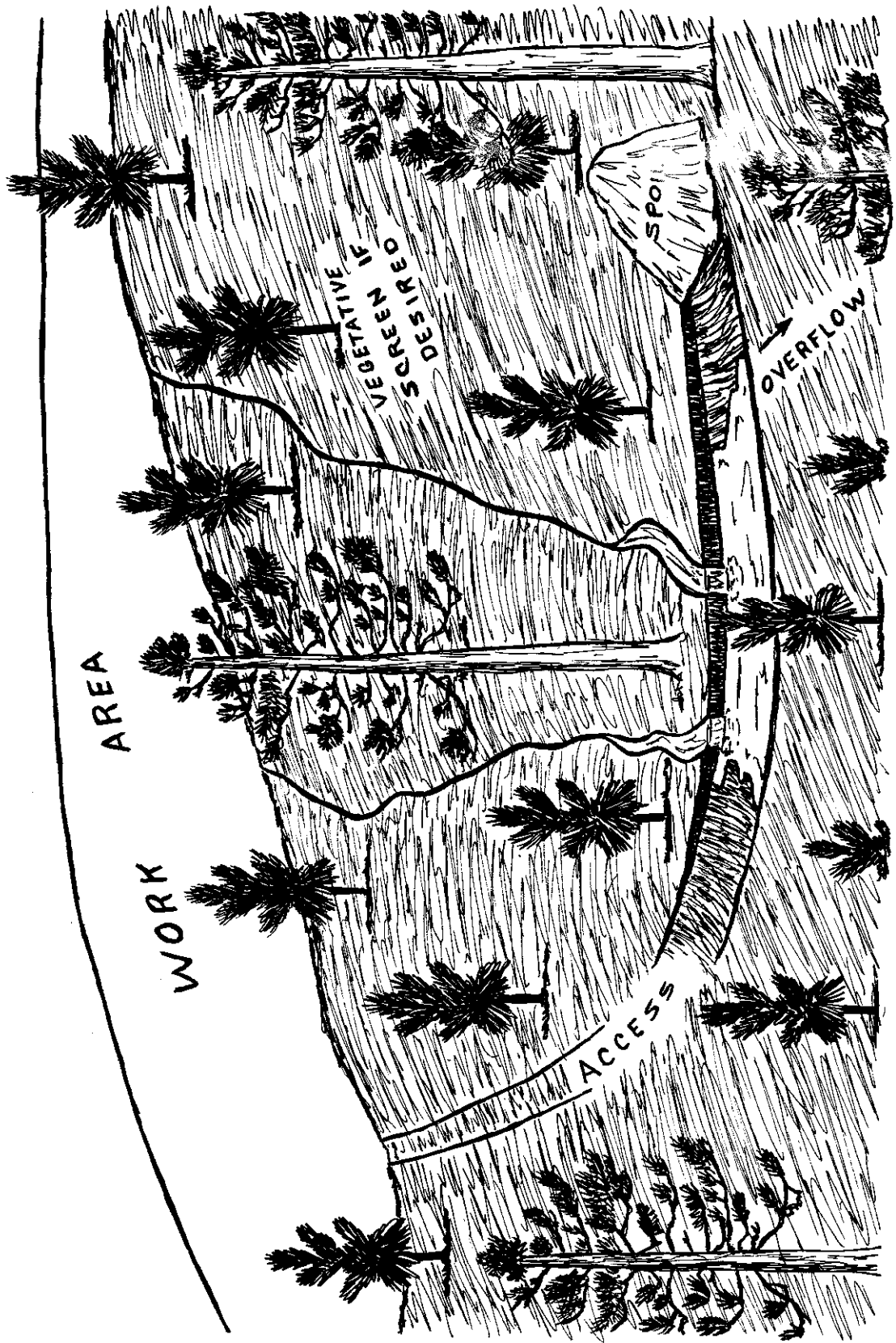


EXHIBIT II

CONSTRUCTION SPECIFICATIONS

Sediment Control Excavation

Item 701

Description

- 1.1 Sediment control excavation in this item refers to excavation for the purpose of retaining sediment eroded from within the construction limits during construction.

The term "sediment basin" shall apply to sediment control excavation adjacent to the roadway. The term "excavated trap" shall apply to sediment control excavation in major drainages a considerable distance from the construction site.

Constructions Methods

- 3.1 Sediment control excavation shall conform to the size, dimensions, and general shapes shown on the plans. Sediment control excavation will be done at locations shown on the plans.
- 3.2 Sediment control excavation shall be done as soon as clearing operation is completed and before grading begins.
- 3.3 Sediment control excavation adjacent to roadway will be utilized within the roadbed.
- 3.4 Sediment control excavation not adjacent to roadway will be stockpiled on the uphill side, but out of the direct flow of drainage to the trap.

Method of Measurement

- 4.1 Measurement of sediment control excavation will consist of the actual number of sediment basins and/or excavated traps constructed.

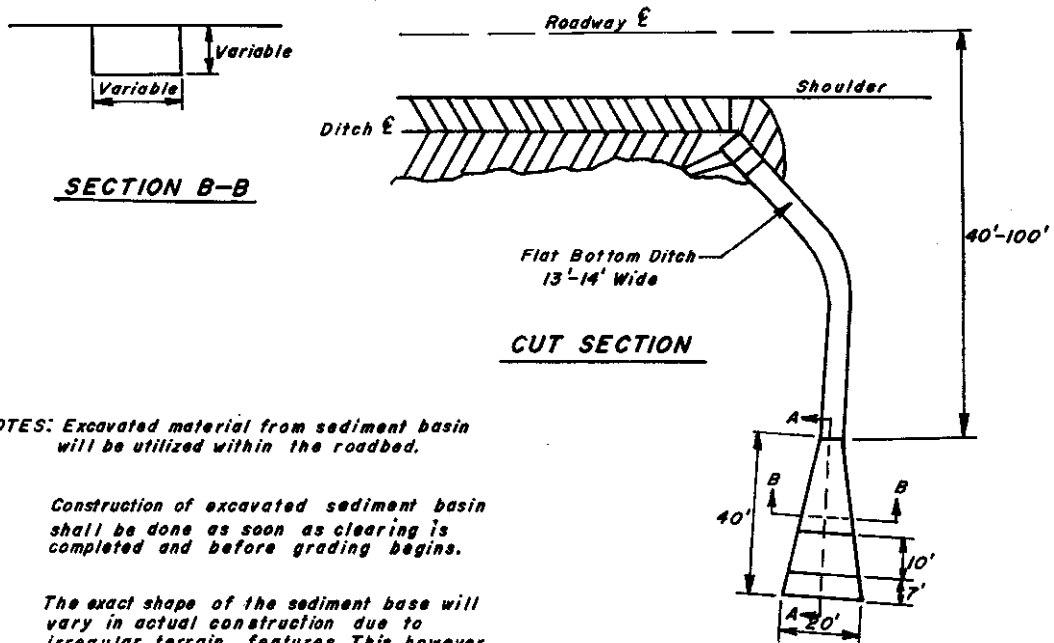
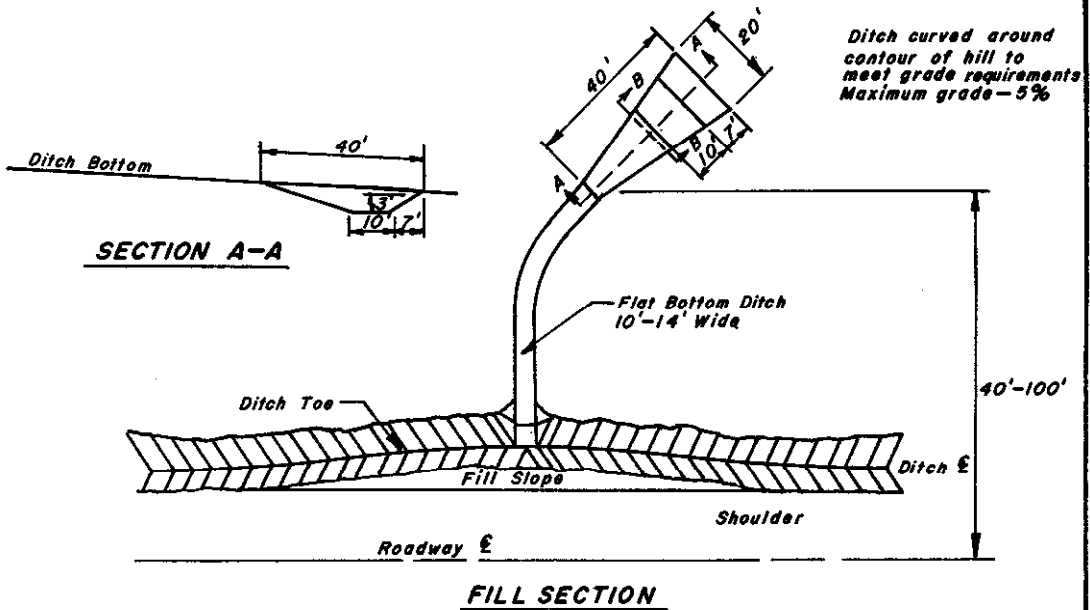
EXHIBIT II - Continued

Basis of Payment

- 5.1 Payment for sediment control excavation will be made on a lump sum basis for each sediment basin and each excavated trap. Such payment will be full compensation for all labor, equipment, clearing, grubbing, excavation, and incidentals necessary to complete the work in this specification.

<u>Pay Item and Name</u>	<u>Unit of Measurement</u>
701-1 Excavated Trap	Lump Sum (Each)
701-2 Sediment Basin	Lump Sum (Each)

EXHIBIT III



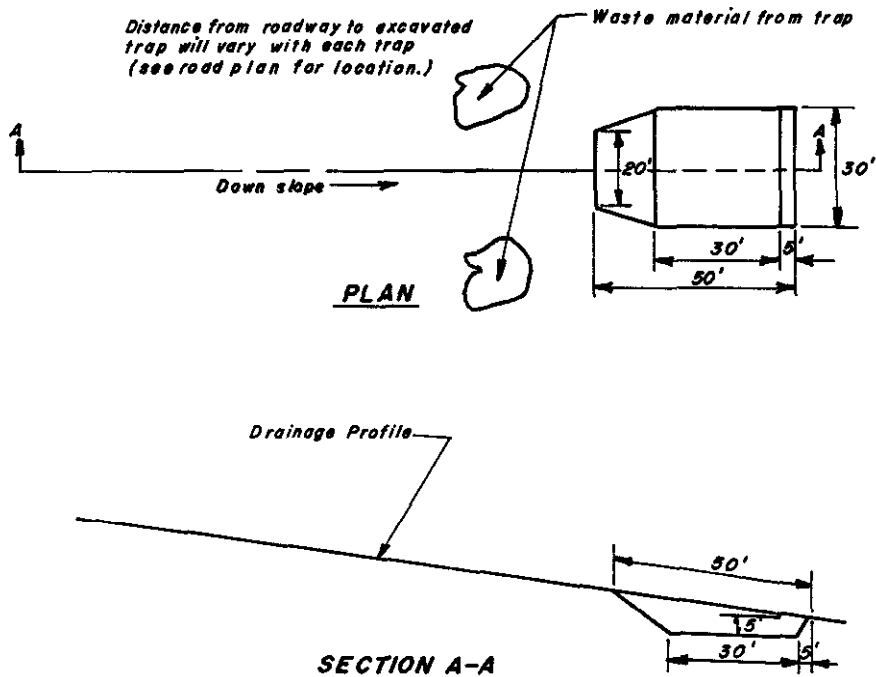
NOTES: Excavated material from sediment basin will be utilized within the roadbed.

Construction of excavated sediment basin shall be done as soon as clearing is completed and before grading begins.

The exact shape of the sediment base will vary in actual construction due to irregular terrain features. This however will not affect the quantities of payments.

TYPICAL SEDIMENT BASIN

EXHIBIT IV



NOTE: Construction of excavated traps shall be done as soon as clearing is completed and before grading begins.

The exact shape of excavated traps will vary in actual construction due to irregular terrain features. This however will not effect the quantities of payment.

The material from the excavated traps will be stockpiled on the uphill side but out of the direct flow of drainage to the traps.

TYPICAL EXCAVATED TRAP

URBAN-AREA SEDIMENT YIELD--EFFECTS OF CONSTRUCTION-SITE CONDITIONS AND SEDIMENT-CONTROL METHODS

By Thomas H. Yorke and William J. Herb, Hydrologists
U. S. Geological Survey, College Park and Parkville, Maryland

ABSTRACT

Suspended-sediment discharge and land use were monitored in eight small drainage basins in Montgomery County, Maryland, between 1963 and 1974. Five of these basins, located on the fringe of the rapidly expanding Washington, D. C., metropolitan area, were subjected to continuous construction activity as woodlots, pastures, and cultivated fields were replaced by houses, apartments, and shopping centers. Construction-site sediment yields ranged from 16 to 226 tonnes per hectare per year (7 to 101 tons per acre per year). The degree of sediment control and the slope conditions on construction sites were the most significant factors affecting sediment yields; the proximity of construction to stream channels and the length of time that surface soils were unprotected affected yields to a lesser extent. As a result of improvements in grading practices and control measures, construction-site sediment yields were reduced 60 to 80 percent between 1966 and 1974.

INTRODUCTION

Each year thousands of hectares of pastures, cultivated fields, and woodlots are replaced by housing subdivisions, apartment complexes, and shopping centers in the Washington, D. C. metropolitan area. Increased sedimentation resulting from these construction activities places a tremendous stress on local streams, lakes, and estuaries. Sediment yields ranging from 8,800 to 42,000 t/km² (tonnes per square kilometre) or 25,000 to 120,000 tons/mi² (tons per square mile) were reported for urban construction sites in Baltimore and Washington (Guy and Ferguson, 1962; Wolman, 1964; Guy, 1965; and Vice, Guy, and Ferguson, 1969). After 1965, local government agencies instituted several programs to control sediment eroded from urbanizing areas. This paper evaluates the effects of sediment-control measures and other factors on construction-site sediment yields.

The study was made in the headwaters of the Rock Creek and Anacostia River basins in Maryland (fig. 1). Eight streams with drainage areas ranging from 0.91 to 25.2 km² (0.35 to 9.73 mi²) were monitored for streamflow and suspended-sediment discharge (table 1). Daily records of streamflow were available for 6 of the 8 streams; sediment data were available for most significant storms in the basins. A network of 9 recording rain gages and a varying number of nonrecording gages was used to measure the areal distribution of precipitation. Land use was analyzed and compiled from aerial photographs obtained annually.

The study basins are in the eastern division of the Piedmont physiographic province. The variation of bedrock has created a diversified topography

Table 1.--Summary of streamflow-sediment stations used in the study.

Station	Drainage Area (km ²)	Land Use ²	Period of Sediment Record
Williamsburg Run near Olney	5.83	Urban	Oct. 1966 to Sept. 1974
North Branch Rock Creek near Norbeck	25.2	Rural- urban	Nov. 1966 to Sept. 1974
Manor Run near Norbeck	2.62	Urban	Nov. 1966 to Aug. 1974
Northwest Branch Anacostia River at Norwood	6.35	Rural	Mar. 1967 to Aug. 1974
Nursery Run at Cloverly	.91	Rural	Oct. 1966 to Aug. 1974
Batchellors Run at Oakdale ¹	1.22	Rural	Aug. 1967 to Aug. 1974
Bel Pre Creek at Layhill	4.38	Urban	Mar. 1963 to Aug. 1974
Lutes Run at Lutes ¹	1.22	Urban	Feb. 1963 to Apr. 1970; June 1973 to Sept. 1974

¹Partial-record streamflow site.

²Land use classified as urban if more than 25 percent of the basin area was occupied by residences, shopping centers, and schools. Rural-urban category indicates less than 25 percent urban, but significant amounts of urban development between 1966 and 1974.

which is characterized by gently rolling hills on the upland and by steep slopes (25 percent or greater) adjacent to stream channels. Silt loam and silty clay loam soils of the Manor-Chester-Glenelg soil association predominate in the basins. These soils generally are uniform with one major exception--the subsoils of the Manor and Glenville series are poorly developed, while the subsoils of the other series in the association are well developed with moderate to strong blocky structures (U. S. Soil Conservation Service, 1961). The weak structure of the Manor and Glenville subsoils makes them highly susceptible to erosion during construction when surface soils are removed by rough grading for rights-of-way and foundations.

The climate of the study area is temperate and rather humid; mean annual temperature is about 13°C and mean annual precipitation is about 1,070 mm (millimetres) or 42 in (inches). Precipitation is evenly distributed throughout the year, ranging from 71 mm (2.8 in) in February to 124 mm (4.9 in) in August. Summer precipitation is characterized by short, high-intensity rains from convective storms, and winter precipitation is mostly low-intensity rainfall from frontal storms. Annual runoff averages about 360 mm/yr (14 in/yr). Monthly runoff ranges from 15 mm (0.6 in) in September to 43 mm (1.7 in) in March. Suspended-sediment discharge generally follows the distribution of rainfall and runoff. It is high in early spring when runoff is high and in summer when intense rains cause extensive erosion.

Land use varied considerably in the study basins. Batchellors Run remained essentially rural throughout the period. The Nursery Run and

Northwest Branch Anacostia River basins experienced some urbanization at the beginning of the study, but the land use did not change appreciably between 1966 and 1974. The other basins underwent urban development. During the peak construction period, as much as 15 percent of the Bel Pre Creek basin and 20 percent of the Lutes Run basin were under construction. Urban land ranged from 0 percent of the Batchellors Run basin to 60 percent of the Lutes Run basin in 1974.

URBAN-AREA SEDIMENT YIELDS

Sediment yields of the study basins were typical of urbanizing areas in the suburbs of Washington, D. C. They were representative of basins in which the land use was in a constant state-of-change because of construction activities. In order to evaluate construction-site sediment yields it was first necessary to isolate periods of equal amounts of construction in each basin and then separate the construction-site component from the total basin yield for the selected periods.

Average Annual Sediment Yields

Average annual sediment yields were computed for periods of equal land use in the study basins using the flow-duration sediment-transport curve method described by Miller (1951). The number of yields computed for each basin varied, depending on land use changes in the basin. For example, six average annual yields were computed for the Williamsburg Run basin because of changes in the amount and location of construction within the basin. In the rural basins, only one yield was determined since the land use was essentially constant during the study period.

The initial selection of periods for which average annual sediment yields were computed was based on land-use data. Periods of equal construction activity were determined by examining aerial photographs, field notes, and grading and paving permits furnished by the Montgomery County Department of Public Works. Because of constantly changing conditions on construction sites, it was not possible to define exact periods of equal construction activity, but periods were selected so that the land use depicted by aerial photographs would be representative of average conditions during the period. An effort was made to identify periods based on the location as well as the amount of construction within the basins so that a range of physical conditions on construction sites could be evaluated.

Once the periods were selected, sediment-transport curves were made for the periods having adequate storm data. The curves were developed from logarithmic plots of daily sediment discharges and the corresponding daily water discharges. Particular attention was given to the medium and high range of water and sediment discharge because most of the sediment load was transported during large storms. Curves for several periods were not used because the high ends of curves were poorly defined or because a disproportionate number of storms were sampled during either the growing or dormant season. Transport curves were used in conjunction with the respective flow-duration curves of the basins to compute average annual sediment yields adjusted to runoff conditions between 1966 and 1974. Twenty-seven

yields were determined for the eight basins. They ranged from 0.61 t/hm² (tonne per square hectometre) or 0.27 ton/acre in the Bel Pre Creek basin during a period of no construction to 25.3 t/hm² (11.3 tons/acre) in the Manor Run basin when 11 percent of the basin was under construction.

Sources of Sediment

The sediment yields computed for each basin represented the combined yield from forests, croplands, construction sites, and all other land-cover categories in the basins. In order to isolate the construction-site part of each basin yield, it was necessary to estimate the yield from some sources and compute others by successively eliminating estimated yields from total basin yields. Sediment yields for low-yield land covers (forest, grass, rural residential, and public-commercial) were estimated as indicated below:

Forest--Most of this category was composed of upland, oak-hickory hardwood forests. An estimated annual sediment yield of 0.07 t/hm² (0.03 ton/acre) was based on sediment yields of forested areas of 0.07 t/hm² (0.03 ton/acre) in Pennsylvania (Williams and George, 1968), 0.04 t/hm² (0.02 ton/acre) in Mississippi (Ursic and Dendy, 1965), and 0.07 t/hm² (0.03 ton/acre) in the Potomac River basin (Wark and Keller, 1963).

Grass--The grass category included well-managed turf on golf courses, abandoned fields, or lightly grazed pastures. An annual yield of 0.45 t/hm² (0.2 ton/acre) was selected to approximate the yield of 0.27 t/hm² (0.12 ton/acre) for abandoned fields in Mississippi (Ursic and Dendy, 1965).

Rural residential--This category represented residential property with lots greater than 0.2 hm² (0.5 acre) and generally greater than 0.4 hm² (1 acre). An annual sediment yield of 1.1 t/hm² (0.5 ton/acre) was assumed because a fairly large part of this land area was used for large gardens and paddock areas for horses. Also, many of the roads were dirt and the roadside ditches were unimproved, adding to the erosion potential.

Commercial-public--This category included some shopping areas, but was mostly playing fields of public schools. An annual sediment yield of 0.7 t/hm² (0.3 ton/acre) was assumed because of accelerated erosion in the stream channels immediately downstream from these areas.

Sediment yields for high-yield land use categories (urban residential, cropland, and construction areas) were determined by subtracting the assumed yields for forest, pasture, rural residential, and public-commercial from the total sediment yields of each basin. For example, sediment yields for

cropland were determined for four basins during periods when there were no urban residential or construction areas. The computed annual yields for cropland were:

	Tonnes/Hectare
Batchellors Run	1.5
Bel Pre Creek	6.1
Nursery Run	9.6
Northwest Branch Anacostia River	7.4

Similarly, the annual yield for urban residential was determined by subtracting other known or assumed yields from the total yields of the basins during periods with no construction or only minimal construction activities. Sediment yields of 8.1 and 8.7 t/hm² (3.6 and 3.9 tons/acre) were computed for urban residential in the Lutes Run and Manor Run basins, respectively. Most of the yield from urban sites apparently resulted from stream-channel erosion. Channel erosion averaged 6.7 t/yr (7.4 tons/yr) for each hectare developed in the Manor Run basin between 1967 and 1974 (unpublished data of the U. S. Geol. Survey).

Nineteen of the twenty-seven values for average annual sediment yield estimated by the sediment-transport flow-duration curve method were representative of periods of significant amounts of construction. Construction-site yields computed for these periods ranged from 16.1 to 226 t/hm²/yr (7.2 to 101 tons/acre/yr) and averaged 73.3 t/hm²/yr (32.7 tons/acre/yr).

FACTORS AFFECTING CONSTRUCTION-SITE SEDIMENT YIELDS

A number of construction-site factors could affect construction-site sediment yields. Slope conditions on the sites, proximity of sites to stream channels, buffer zones of natural vegetation, ratio of area of largest development to total area under construction in the basin, and the percentage of construction area with sediment controls were factors considered for analysis. Other factors were probably significant, but could not be evaluated properly. Soils, particularly their relative erodibility, could cause substantial variation of sediment yields; however, the soil conditions were generally uniform except for one or two sites. The density of housing could affect sediment yields, but again the conditions in the basins were too uniform to detect significant differences.

Construction-site sediment yields and the corresponding site factors selected for study are summarized in table 2. Each factor, except for the amount of sediment control in each basin, was determined from aerial photographs corresponding to the periods of the transport curves. The sediment-control factor was based on the average amount of construction with adequate sediment controls as determined by field surveys of the Montgomery County Sediment Control Task Force in 1968 and 1970, and the Montgomery County Sediment Control Section in 1972 and 1974. Adjustments to the county-wide averages determined by the field surveys were made for study basins where observations by project personnel were available.

The multiple effects of site factors was evaluated with a linear multiple-regression model. The logarithm of construction-site sediment yield was the dependent variable and the site factors were the independent variables.

Table 2. Construction-site sediment yields and related factors.

Basin and Period	Construc- tion area	Average- annual sediment yield	Average slope of construc- tion sites	Construc- tion area within 61 m of channel	Ratio of area of largest develop- ment to total con- struction	Construc- tion area without 61 m buf- fer zone	Area under construc- tion with adequate sediment control
	hm ²	t/hm ²	percent	percent	percent	percent	percent
Williamsburg Run							
Oct. 1966 - Mar. 1968 (1)	44	46.2	5.45	25.3	99.3	96.6	10
Apr. 1968 - Mar. 1969 (2)	28	54.7	5.75	20.4	83.3	85.2	36
Apr. 1969 - Mar. 1971 (3)	18	51.1	5.97	26.1	39.8	52.0	42
Apr. 1971 - Dec. 1971 (4)	30	32.3	4.30	20.0	24	88.0	49
Jan. 1972 - Dec. 1973 (5)	36	19.3	5.15	17.4	39	75.6	60
¹ North Branch Rock Creek							
Oct. 1966 - Mar. 1968 (1)	35	104	6.70	52.1	45.1	88.7	5
Apr. 1969 - Mar. 1971 (2)	24	141	7.64	20.3	74.3	81.4	36
Apr. 1971 - Dec. 1971 (3)	25	28.0	7.34	29.2	58.3	91.7	49
Jan. 1972 - Dec. 1973 (4)	21	33.9	6.94	7.0	49.2	52.8	60
Manor Run							
Mar. 1967 - Sept. 1968 (1)	30	216	6.38	36.6	87.2	81.7	10
Jan. 1969 - Dec. 1969 (2)	25	226	6.72	41.7	88.9	100.	15
Apr. 1970 - Dec. 1970 (3)	19	122	5.90	37.5	80.	87.5	27
Feb. 1971 - Dec. 1972 (4)	12	47.5	7.28	22.2	46.1	88.6	52
Bel Pre Creek							
Jan. 1966 - Dec. 1967 (1)	61	47.3	5.72	19.3	88.1	65.1	0
Jan. 1968 - Dec. 1969 (2)	46	59.2	5.92	32.4	92.4	93.8	35
Jan. 1970 - Dec. 1971 (3)	40	37.0	5.56	34.8	83.0	67.8	45
Jan. 1973 - Sept. 1974 (4)	57	16.1	5.58	25.0	91.7	97.0	68
Lutes Run							
Oct. 1966 - Dec. 1967 (1)	12	71.3	5.42	55.0	35.0	75.0	0
Jan. 1968 - Dec. 1970 (2)	5	38.1	6.58	64.8	66.7	78.5	38

¹North Branch Rock Creek near Norbeck excluding Williamsburg Run.

Equations 1, 2, and 3 illustrate the results of the regression analysis.

$$\log S_y = 2.08 - .010 C, \text{ S.E.} = .24 \text{ log units} \quad (1)$$

$$\log S_y = 1.21 + .143 S - .010 C, \text{ S.E.} = .22 \text{ log units} \quad (2)$$

$$\log S_y = 1.02 + .143 S + .002 B - .010 C, \text{ S.E.} = .22 \text{ log units} \quad (3)$$

where S_y = construction site sediment yield, in t/hm²/yr.

C = percent of total construction area with adequate sediment control.

S = average slope of construction sites, in percent.

B = percent of total construction area without a buffer zone of natural vegetation.

S.E. = standard error of estimate.

The equations indicate that sediment controls and the slope of the construction sites were the two most significant factors affecting construction-site sediment yields. The amount of construction with sediment controls explained 45 percent of the variation of sediment yield between construction-sites. Sediment controls and slope explained 59 percent. Other factors which were analyzed did not appear to play a significant role in explaining variations of sediment yields between sites. This could be due to the limited number of observations or to the effect of factors not analysed in the regression analysis.

The increase in effective sediment control during the investigation resulted from several resolutions and ordinances passed by the Montgomery County Council between 1965 and 1971. In 1965, sediment control was the stated policy of Montgomery County; developers were encouraged to institute sediment controls, but there was no regulation requiring their use. A county ordinance, passed in 1967, required land developers to submit sediment-control plans with preliminary subdivision plans. Sediment control plans had to be approved by the Montgomery County Soil Conservation District, but there were no provisions to insure that the plans were implemented in the field. The most significant sediment-control legislation was a county ordinance enacted in 1971 that established the Sediment Control Section within the County's Department of Environmental Protection. This unit was charged with reviewing sediment-control plans and inspecting construction sites to insure that sediment-control measures were properly installed and adequately maintained during construction.

As a result of the ordinances and training sessions for developers, engineers, and grading contractors, the design and maintenance of sediment controls improved markedly. An example of the improvements in sediment basin design is illustrated by figure 2. Basin A is typical of those

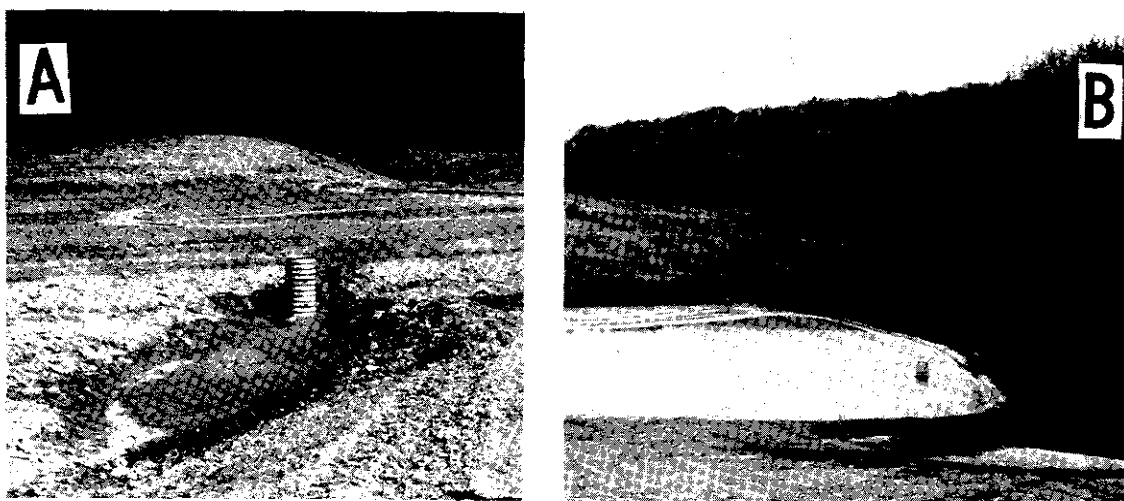


Figure 2.--Sediment basins typical of those used before (A) and after (B) 1968.

used in 1966, which were usually low embankments placed across drainage swales at the terminus of construction sites. The sediment-storage area of these basins was often completely filled after one or two storms and the basins were ineffective thereafter. In contrast, many of the basins used in later years (fig. 2B) were sediment-stormwater management basins that were large enough to trap significant quantities of sediment before it could enter the streams. Other control measures were used more frequently as sediment control became accepted practice; limited grading, temporary seeding, and mulching were used to reduce the amount of exposed soil; and diversion berms and stabilized waterways were used to reduce erosion on critical slopes. Frequent visits by trained inspectors

also assured that control measures damaged by construction traffic were repaired and maintained as required.

The significance of land slope on construction-site sediment yields is indicated by slope conditions on construction sites in the Bel Pre Creek and Manor Run basins in 1967 (fig. 3). The average slope on construction

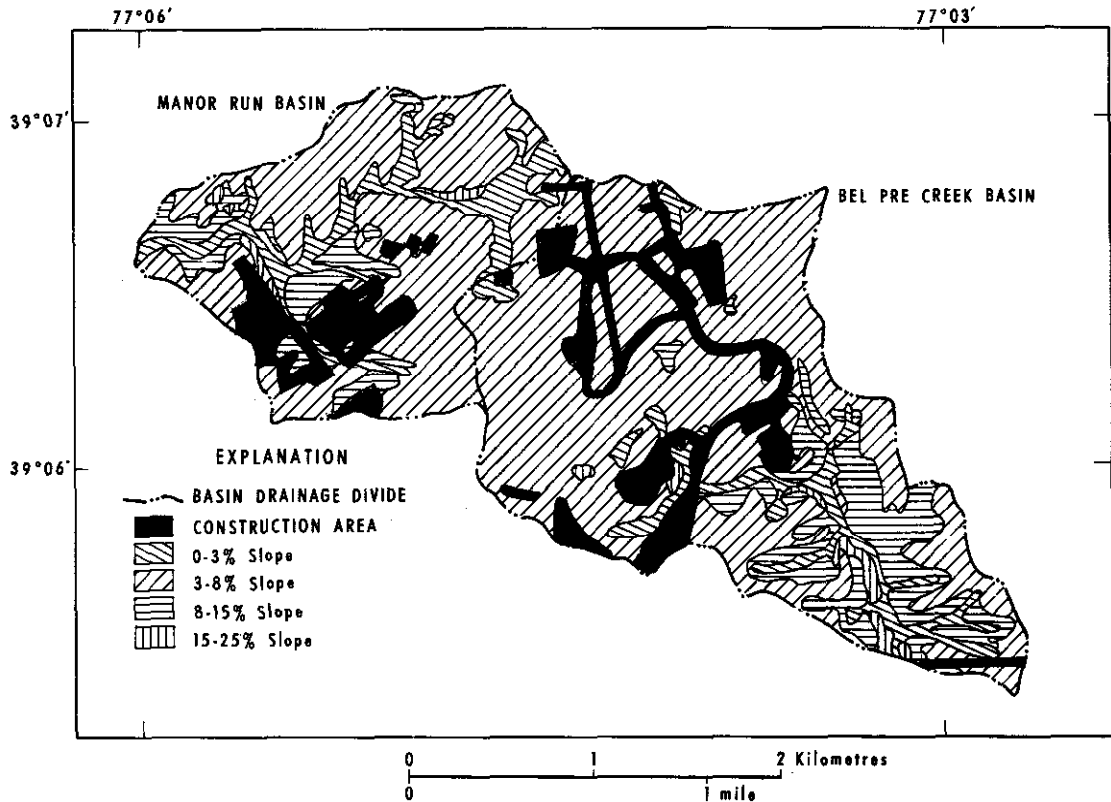


Figure 3.--Location of construction sites and slope conditions, Bel Pre Creek and Manor Run basins, June 1967. (Slope classification adapted from Soil Survey of Montgomery County, Maryland, U. S. Soil Conservation Service, 1961.)

sites in Bel Pre Creek was 5.72 percent and the average annual sediment yield was 47.3 t/hm² (21.1 tons/acre) between January 1966 and December 1967. In contrast, the average slope on the Manor Run construction sites was 6.38 percent and the sediment yield was 216 t/hm²/yr (96.4 tons/acre/yr) between March 1967 to September 1968. Virtually all the construction in Bel Pre Creek was on slopes ranging from 3 to 8 percent while part of the construction area in Manor Run was on slopes between 8 and 15 percent.

Some sites had slopes greater than 15 percent. The photograph in figure 4 shows the severity of erosion on the long, steep slopes in the Manor Run basin. Extensive rill and gully erosion on slopes greater than 10 percent apparently contributed to the high sediment yield from this basin.

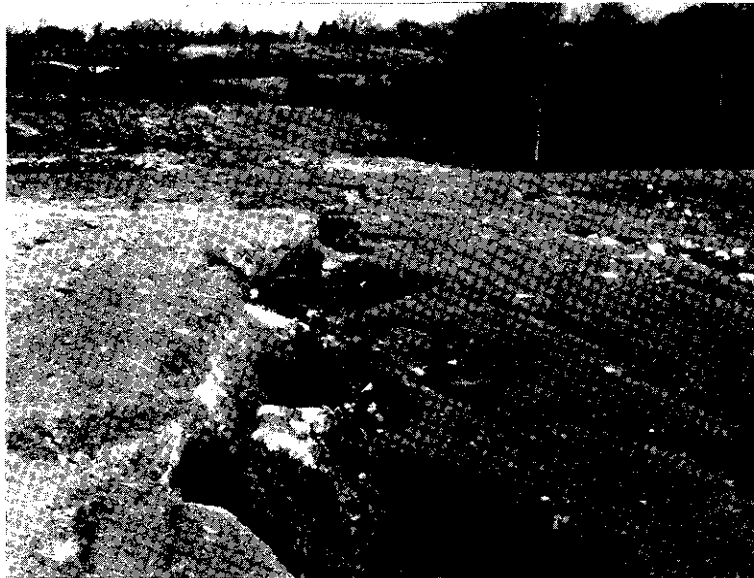


Figure 4.--Extensive erosion on long steep slopes in the Manor Run basin.

To further examine the relation between construction-site sediment yields and various site factors, the yields computed from observed data and the yields estimated with equation 2 were plotted as shown in figure 5. The scatter about the line of equality was significant, reflecting the large standard error of estimate of the regression equation. The data points with the greatest departure from the line of equality were assumed to be representative of conditions not evaluated by the regression analysis. For example, the low computed sediment yields in the Lutes Run basin, where the percentage of construction area with sediment controls was lower than average and the slope of construction sites was higher than average, probably resulted from these sites being open long enough for natural stabilization to develop. One 5.2-hectare (13-acre) site in this basin was rough-graded prior to 1963 and remained undeveloped through 1973. A similar condition existed in the North Branch Rock Creek basin. A site was first opened in 1966 and was not completed until 1972. The low yield computed in 1971 (period 3) apparently reflected the length of time the site was open.

ESTIMATED ANNUAL SEDIMENT YIELD, IN TONS PER ACRE

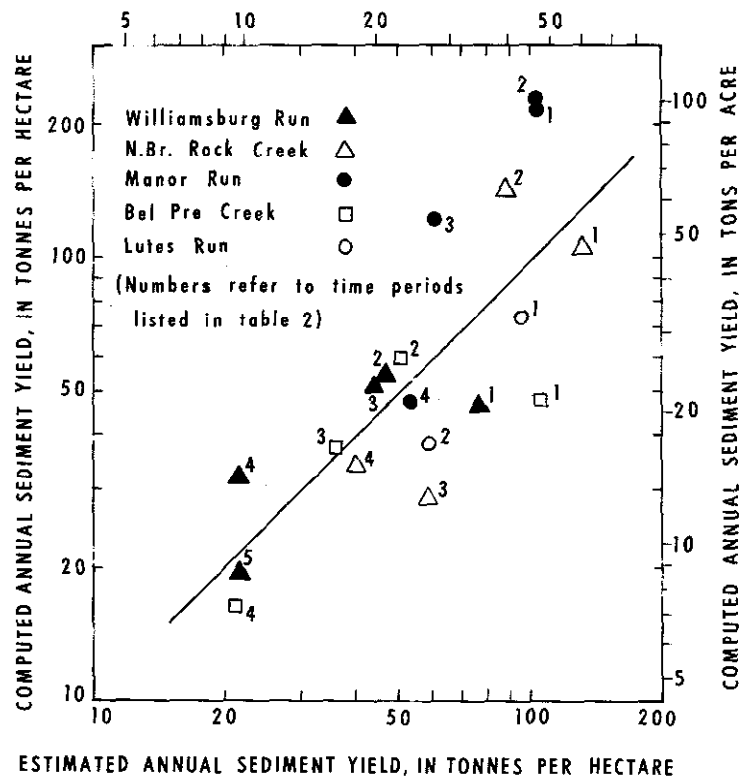


Figure 5.--Relation between construction-site sediment yields estimated by Equation 2 and those computed from observed data.

The effect of the proximity of construction sites to stream channels is also shown in figure 5. The computed sediment yields for Manor Run, periods 1-3, plot well above the line of equality. Construction within 61 m (200 ft) of stream channels averaged 39 percent during these periods. In contrast, the computed yields for period 1 in Williamsburg Run and Bel Pre Creek plot well below the line of equality as the construction within 61 m (200 ft) averaged 25 and 19 percent, respectively. Similar results were obtained in an earlier study in the same basins, where sediment yields from summer storms increased as the amount of construction within 30 and 91 m (100 and 300 ft) of stream channels increased (Yorke and Davis, 1972). The lower sediment yields from construction sites located farther from stream channels illustrate the filtering effects of vegetation on sediment-laden runoff waters. As runoff transporting sediment from a construction site passes through a heavily vegetated zone, its velocity is reduced and some of the water infiltrates into the soil. This reduces the capacity of the water to transport sediment, and much of it is deposited before it enters the stream.

SUMMARY

Average annual sediment yields of eight small drainage basins in Montgomery County, Md., ranged from 0.61 to 25.3 t/hm² (0.27 to 11.3 tons/acre), depending on land-use conditions in the basins. Cultivated fields, urban residential areas, and construction sites were the major sources of sediment. Annual sediment yields attributable to cultivated fields and residential areas averaged 6.2 and 8.4 t/hm² (2.8 to 3.7 tons/acre), respectively. Construction-site sediment yields ranged from 16.1 to 226 t/hm² (7.2 to 101 tons/acre) and averaged 73.3 t/hm² (32.7 tons/acre).

Construction-site sediment yields were reduced 60 to 80 percent between 1966 and 1974 in each of the study basins where active construction occurred. Some of the reduction was attributed to reducing slopes on construction sites and increasing distances between construction sites and stream channels; however, most of the reduction was the result of improvements in the design and implementation of sediment-control measures. These included limiting grading to reduce the amount of land open at any one time, planting of temporary vegetation and application of mulch to protect exposed soils, construction of diversion berms and stabilized waterways to reduce erosion on critical slopes, and use of large sediment basins to trap eroded sediment on site.

REFERENCES

- Guy, H. P., and Ferguson, G. E., 1962, Sediment in small reservoirs due to urbanization: Am. Soc. Civil Engin., Jour. Hydraulics Div., v. 88, no. HY2, p. 27-37.
- Guy, H. P., 1965, Residential construction and sedimentation at Kensington, Maryland, in Federal Inter-Agency Sedimentation Conference Proc., 1963: U. S. Dept. of Agriculture Misc. Pub. 970, p. 30-37.
- Miller, C. R., 1951, Analysis of flow-duration, sediment rating curve method of computing sediment yield: U. S. Dept. of Interior, Bureau of Reclamation, 55 p.
- Ursic, S. J., and Dendy, F. E., 1965, Sediment yields from small watersheds under various land use and forest covers, in Federal Inter-Agency Sedimentation Conference Proc., 1963: U. S. Dept. of Agriculture Misc. Pub. 970, p. 47-52.
- U. S. Soil Conservation Service, 1961, Soil Survey, Montgomery County, Maryland: U. S. Dept. of Agriculture, Soil Conservation Service, 107 p., 54 maps.
- Vice, R. B., Guy, H. P., and Ferguson, G. E., 1969, Sediment movement in an area of suburban highway construction, Scott Run basin, Fairfax County, Virginia, 1961-64: U. S. Geol. Survey Water-Supply Paper 1591-E, 41 p.

- Wark, J. W., and Keller, F. J., 1963, Preliminary study of sediment sources and transport in the Potomac River basin: Interstate Comm. Potomac River Basin Tech. Bull., 1963-11, 28 p.
- Williams, K. F., and George, J. R., 1968, Preliminary appraisal of stream sedimentation in the Susquehanna River basin: U. S. Geol. Survey open-file rept., 73 p.
- Wolman, M. G., 1964, Problems posed by sediment derived from construction activities in Maryland: Rept. to the Maryland Water Pollution Control Commission, Annapolis, Md., 125 p.
- Yorke, T. H., and Davis, W. J., 1972, Sediment yields of urban construction sources, Montgomery County, Maryland: U. S. Geol. Survey open-file rept., 39 p.

EROSION AND SEDIMENT CONTROL ON RESHAPED LAND

By: L. D. Meyer, Agricultural Engineer, and M. J. M. Römken, Soil Scientist, USDA Sedimentation Laboratory, Oxford, Mississippi.

ABSTRACT

Recent research concerning soil erosion and sediment control on land reshaped for nonagricultural purposes is discussed. Included are: adaptation of the Universal Soil Loss Equation, variability of rainfall erosiveness, evaluations of soil erodibility and topographic effects, influence of mulch rates and types, and enhancement of rapid revegetation. Implications of this knowledge and areas of research needs are indicated.

INTRODUCTION

Man has developed machines to reshape and use land more extensively than past generations ever dreamed possible. Unfortunately, these machines also enable him to abuse valuable land and water resources unless their use is accompanied by proper erosion control and runoff management practices. The tremendous sediment yields that often originate from reshaped land have particularly severe consequences in the densely populated urban environment. Growing public concern and restrictive legislation regarding sediment problems make development of modern erosion-control techniques imperative.

Several million acres in the United States are denuded and topographically reshaped each year for nonagricultural purposes. Our growing population and high standard of living require more houses, shopping centers, highways and waterways, and they require mining more coal, metals and other minerals. Most technology for erosion control on such land has been adapted from knowledge obtained by agricultural research. Yet, the conditions encountered often differ greatly from those found in agriculture. Agricultural research has been conducted primarily on surface soils, yet reshaping may expose soil a meter or more below the surface. Agricultural research has studied slope steepnesses up to 20%, yet construction slopes up to 50% are common and surface-mine spoil often exists at the soil material's angle of repose. Cultural and mechanical conservation practices that control erosion on agricultural land have not been studied under the more extreme conditions usually found on reshaped land.

Erosion prediction and erosion-control technology are still in their infancy for application to reshaped land. However, in recent years, research has begun to provide important new knowledge on soil erosion and sediment control for reshaped areas such as those for highway construction, housing developments, waterway and reservoir projects, and surface mining.

RESEARCH KNOWLEDGE

Universal Soil Loss Equation (USLE) Probably the most useful tool for erosion prediction on reshaped land has been the Universal Soil Loss Equation (Wischmeier and Smith, 1965; Wischmeier, 1973). Although based on data from agricultural conditions and developed for agricultural use,

the USLE has been applied to a wide range of conditions that are far beyond its intended use.

The Universal Soil Loss Equation computes the average rate of upland erosion per unit of land area as the product of factors for rainfall erosiveness, soil erodibility, slope length, slope steepness, cultural procedures, and mechanical conservation practices. When applying the USLE to reshaped land, some factor values are used as evaluated for agricultural conditions and others have been estimated for reshaped land (Wischmeier and Meyer, 1973; Chen, 1974; Meyer, 1974; Ports, 1975). The USLE does not account for erosion from gullies and stream channels or for deposition of eroded soil at the base of a slope, in vegetation, or elsewhere below the upland area. These aspects must be considered separately to evaluate the net sediment yield.

If the predicted soil loss is greater than is tolerable, the USLE may be used to evaluate alternative, less erodible construction procedures until a tolerable rate is predicted. For example, rainfall erosiveness can be reduced by scheduling the more erodible phases of a project to avoid seasons of greatest erosion potential or by decreasing the duration of exposure. Soil erodibility may be reduced by exposing a depth of the soil profile that is less erodible. Slope lengths may be reduced by diversion terraces, or slope steepnesses may be decreased by reshaping. Cultural practices may be improved by mulching, roughening the land surface, or rapid revegetation. Supplemental practices such as contour ripping or leaving contour strips of vegetation may be used. The factors in the USLE are multiplied, so relatively small changes in several factors can substantially affect the erosion-rate product.

Either as an erosion-prediction equation or for comparing the effectiveness of various erosion-control practices, the USLE is a valuable tool in land management planning. For conditions where its factors have been evaluated or can be satisfactorily estimated, the USLE will provide useful predictions of upland soil erosion.

Erosive Potential of Rain The erosiveness of rainfall has been expressed as a function of its intensity and impact energy (Wischmeier, 1962; Wischmeier and Smith, 1965). The average annual erosion potential of rainfall east of the Rocky Mountains varies as shown in Figure 1. The erosive potential of rainfall also varies throughout the year as illustrated for two areas in Figure 2. For short-term land reshaping projects, the average potential for a season or month may be computed by using the appropriate distribution curve (Wischmeier and Smith, 1965) and the average annual erosion potential (Figure 1) for the locality of interest. For instance, Figure 2 shows that the monthly erosion potentials for Memphis, Tennessee vary little throughout the year, whereas those for Rochester, Minnesota peak in late spring and summer. Because of these different distributions, the erosion potential during the summer months is slightly greater in Rochester, although the annual potential at Rochester is only half that at Memphis.



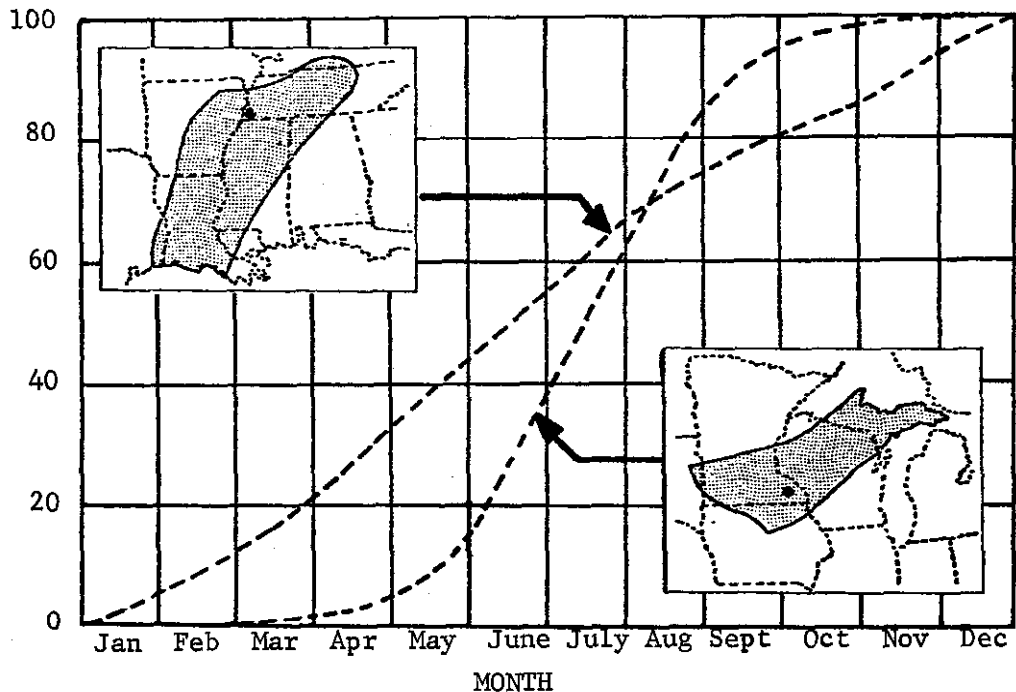
Figure 1. Average annual rainfall erosion potentials east of the Rocky Mountains (from Wischmeier and Smith, 1965).

In addition to varying within a year, the erosive potential of rainfall varies from year to year. The probabilities for above-average annual erosion potentials are also available for many locations (Wischmeier and Smith, 1965, Table 11). Even the erosion potential for individual storms is indicated for recurrence intervals up to once in 20 years (Wischmeier and Smith, 1965, Table 12). These data show that many locations will occasionally have a year with nearly twice the average annual erosion potential and that a single storm with more than half the average annual potential is possible at many locations.

Soil Erodibility A soil's susceptibility to erosion is affected by its water intake characteristics and its ability to resist detachment and transport by rainfall and runoff. These soil properties usually change with depth, so they may vary considerably as different soil horizons are exposed during land reshaping. Soil erodibility, known as the K-factor in Agriculture Handbook 282 (Wischmeier and Smith, 1965), is defined as the rate of soil erosion per unit of erosion potential from a unit plot, which is tilled up and down slope and has been kept in fallow condition for at least 2 consecutive years.

The complexity and variability of soils have thwarted attempts to derive a predictive relationship between soil erodibility and soil properties that is universally applicable. Wischmeier and Mannering (1969) proposed a 24-term linear regression equation, derived from data on 55 mostly medium-textured surface soils of the Corn Belt. It related soil erodibility to various individual and combined physical soil properties. Although this equation accounted for 98 percent of the variation in observed erodibility, it was too cumbersome for practical use. Also,

PERCENT OF ANNUAL
RAINFALL EROSION POTENTIAL



	Memphis, Tennessee (Annual Potential-300 units)		Rochester, Minnesota (Annual Potential-150 units)	
Month	Percent of Annual	Units during Month	Percent of Annual	Units during Month
Jan.	6	18	<1	1
Feb.	7	21	<1	1
Mar.	9	27	1+	2
Apr.	11	33	4	6
May	11	33	10	15
June	12	36	22	33
July	11	33	24	36
Aug.	8	24	21+	32
Sept.	6	18	10	15
Oct.	6	18	4	6
Nov.	7	21	1+	2
Dec.	6	18	<1	1
	<u>100</u>	<u>300</u>	<u>100</u>	<u>150</u>

Figure 2. Distribution of annual rainfall erosion potential for two sections of the United States (from Wischmeier and Smith, 1965).

the analysis did not include data for subsoils commonly found on reshaped land. Similar analyses have been performed, with less success, on agricultural soils mostly from the Southeastern United States by Barnett et al. (1966) and Barnett and Rogers (1966).

The most useful tool currently available for evaluating soil erodibility is a nomograph (Figure 3) by Wischmeier et al. (1971). This nomograph permits graphical computation of soil erodibility from five readily identifiable parameters: two textural parameters, organic matter content, and indices for structure and for permeability. These parameters can be obtained from routine laboratory analyses and standard soil profile descriptions as defined in the USDA Soil Survey Manual (1951). The nomograph, based on data from Corn Belt surface soils earlier examined by Wischmeier and Mannering (1969), represents a relatively simple technique of evaluating soil erodibility with reasonable accuracy. Moreover, the nomograph also accurately predicted erodibilities of the C-horizons of a Miami loam and Wingate silt loam and of subsoils exposed by mechanically desurfacing a Shelby loam and Marshall silt loam (Wischmeier et al., 1971).

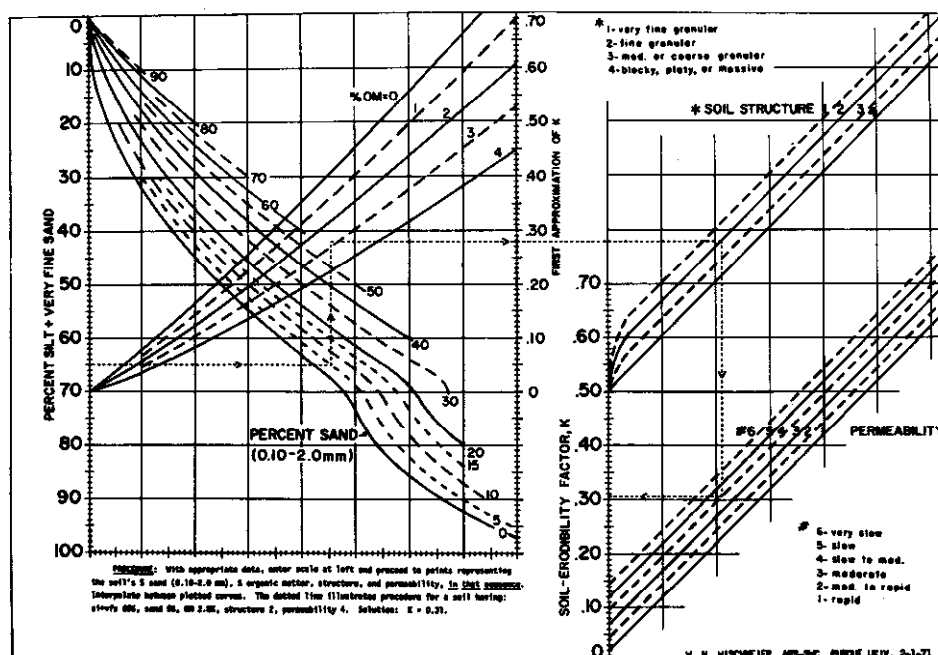


Figure 3. Soil erodibility nomograph for determining soil K-factors (from Wischmeier, et al., 1971).

More recent erodibility studies on selected high-clay subsoils, using procedures similar to those of Wischmeier et al. (1971), yielded soil erodibility K-factors substantially different from nomograph predictions in most cases (Table 1). The largest differences were observed for well-aggregated, high-clay subsoils (Romkens et al., 1975) and were attributed to the clod stability of these well-aggregated soils, which

maintained hydraulic roughness and relatively high infiltration rates. K-factors obtained for the poorly aggregated, high-clay subsoils agreed better with nomograph predictions. K-factors computed for scalped-only surfaces, using techniques similar to those used for the tilled subsoils, also differed substantially from nomograph estimates (Table 1). The largest discrepancies again were obtained for well-aggregated soils. Hence, if scalped surfaces were chosen instead of tilled surfaces as the basis for soil erosion predictions for high-clay subsoils, soil erodibility factors determined from tilled surfaces would be multiplied by about 2 for poorly aggregated subsoils and by about 16 for well-aggregated subsoils.

Table 1--Soil erodibility K-factors for selected high-clay subsoils as predicted by the soil-erodibility nomograph and as measured experimentally

Soil Series	K _{nomo} ^a	K _{scalped} ^b	K _{tilled} ^b
Mayberry ^c	0.37	0.67	0.04
St. Clair	0.34	0.48	0.31
Pawnee	0.29	0.45	0.24
McGary ^c	0.43	0.36	0.17
Wymore ^c	0.34	0.49	0.03
Portageville ^c	0.21	0.05	0.01

^a According to Wischmeier et al., 1971.

^b Soil erodibility determinations on scalped only and scalped-then-tilled surfaces, respectively (Romkens et al., 1975).

^c Well-aggregated subsoils.

To improve predictions of K-values for high-clay subsoils, chemical and mineralogical properties were determined and included in multiple regression analyses. The result was an equation involving one textural parameter of the nomograph by Wischmeier et al. (1971) and the percentage of iron plus aluminum that was extractable by CDB (citrate-dithionite-bicarbonate) (Jackson, 1971). This relationship accounted for 90% of the variance in erodibility (Romkens et al., 1974). Although the equation is based on limited data from finer textured subsoils in the North-Central United States, it logically includes inorganic constituents in lieu of organic matter as the "soil binding agent" term in erodibility predictions. To improve soil erodibility predictions for reshaped land, additional research should include subsoils with a wider range of texture, structure, and permeability.

Because of leaching and weathering, most subsoils, particularly in the humid regions, have different physical, chemical, and mineralogical characteristics in their various layers or horizons. Sedimentation and erosion also may form profiles with layers of different textural and mineralogical compositions. Land reshaping often exposes several layers or soil horizons. Soil loss predictions with the USLE can be adapted to

these conditions by summing the soil lost from each exposed horizon in a manner described by Foster and Wischmeier (1974), using separate erodibility factors for each horizon.

Present soil erodibility evaluations are based on soil loss data from experimental areas of less than 0.01 ha (0.01-0.02 acre) subjected to natural or simulated rainstorms with no distinction among ongoing processes of detachment and transport by rainfall and by runoff. Current research is directed toward evaluating erodibility as it relates to the particular subprocess of rainfall or runoff. This approach may provide more meaningful data, since the relative contributions from these subprocesses change as the area and topography of reshaped land change.

Topography The relationships currently used to express the effect of slope length and slope steepness on erosion were derived from limited data on mostly agricultural soils of moderate steepnesses and lengths. They may not be appropriate for the topographic extremes often found on reshaped land, so additional research is needed.

Analytical reasoning (Foster and Meyer, 1972) and limited data (Meyer et al., 1972a) indicate that erosion increases differently on steep or long slopes than is predicted by the relationships used in the USLE. On such slopes, rill erosion becomes dominant relative to interrill erosion, and the exponent in the expression that relates erosion to slope length will increase (Meyer et al., 1974) rather than remaining constant as in the presently used slope-length formula. Similar analyses (Foster and Meyer, 1972) showed that a quadratic relationship as used to express the slope steepness effect in the USLE may be appropriate but that the relative susceptibility to rill and interrill erosion may affect the coefficients. Experimental research to verify or modify the analytical implications is needed.

During massive land reshaping, the shape of a slope's final surface profile may greatly affect subsequent sediment yields (Onstad et al., 1967; Meyer and Kramer, 1969; Young and Mutchler, 1969; Foster and Wischmeier, 1974). Several shape options are illustrated in Figure 4. A convex slope is more erodible than a uniform slope, because it is steepest near the toe where runoff is greatest. A uniform slope will yield more sediment than a concave one, because the concave slope is steepest where the flow is least and because some of the sediment eroded from upper portions of the concave slope may deposit as it flattens near the toe. If a concave shape is not acceptable, because of safety problems on cut or fill sideslopes along a roadway or because of disharmony on building sites, shaping to a complex slope may be a satisfactory solution. A complex slope that is convex along its upper portion and concave along its lower portion will generally field less sediment than a uniform slope. A flat section at the toe of a slope will also reduce the sediment yield. Proper slope shaping during massive topographic modification can significantly reduce downslope sediment problems.

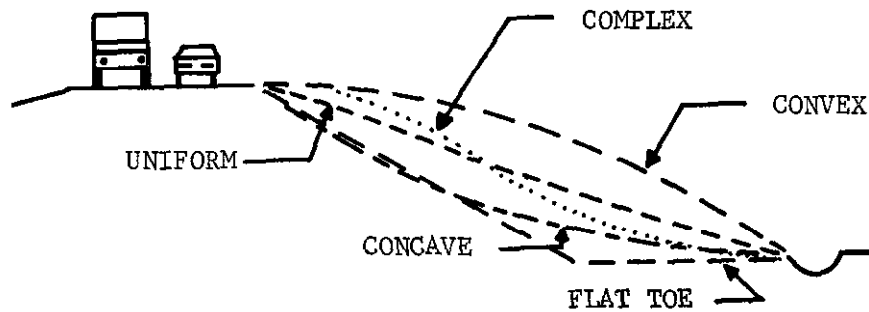


Figure 4. Several slope shapes to which land may be formed.

A technique for evaluating the erosion rates for irregularly shaped slopes has recently been developed by Foster and Wischmeier (1974). Slopes are divided into segments so that the steepness and soil type of each can be considered uniform. Losses from the several segments are summed, and a combined factor is obtained for the slope length, slope steepness, and soil erodibility factors in the USLE.

Surface Condition, Mulches, and Revegetation Soil erosion on agricultural land is affected by cultural factors such as soil surface condition, surface cover, and density of vegetation. On reshaped land, different construction practices produce various surface conditions, and mulches or rapid revegetation are frequently used to reduce the resultant erosion.

Erosion-plot research using intense simulated rainstorms was conducted to provide data on soil and water losses from typical construction conditions (Meyer et al., 1971). Three storms totaling about 12.8 cm (5 in) of rain at an intensity of 6.4 cm/hr (2.5 in/hr) were applied to scalped subsoil of 12% steepness under six surface treatments. As shown in Figure 5, three conditions were very erodible: the scalped-only surface, the scalped and scarified surface, and the 0.6-m (2-ft) depth of bulldozer-packed fill. Each yielded about 110 t/ha (50 tons/acre) from plots that were only 10.7 m (35 ft) long. Two other conditions, 0.6-m depth of loose fill and 10 cm (4 in) of topsoil over scalped subsoil, lost about 70 t/ha (30 tons/acre). However, scalped subsoil with 2.2 t/ha (1 ton/acre) of straw mulch lost only 22 t/ha (10 tons/acre). Runoff ranged from 70% of the applied rain on the loosely filled subsoil to more than 80% on the compact fill. The 110-t/ha losses suggest that the three most erodible conditions would yield about 30,000 cubic meters (40,000 cubic yds) of deposited sediment per 100 ha (250 acres) from 12.8 cm (5 in) of intense rain on moderately steep, denuded slopes with lengths of 50 to 100 m (150 to 300 ft).

Surface mulches of small-grain straw are a very common erosion-control practice on reshaped slopes. Results from studies that have been conducted to determine the effectiveness of several rates of straw mulch are summarized in Figure 6. For short slopes of 3 to 5% on moderately

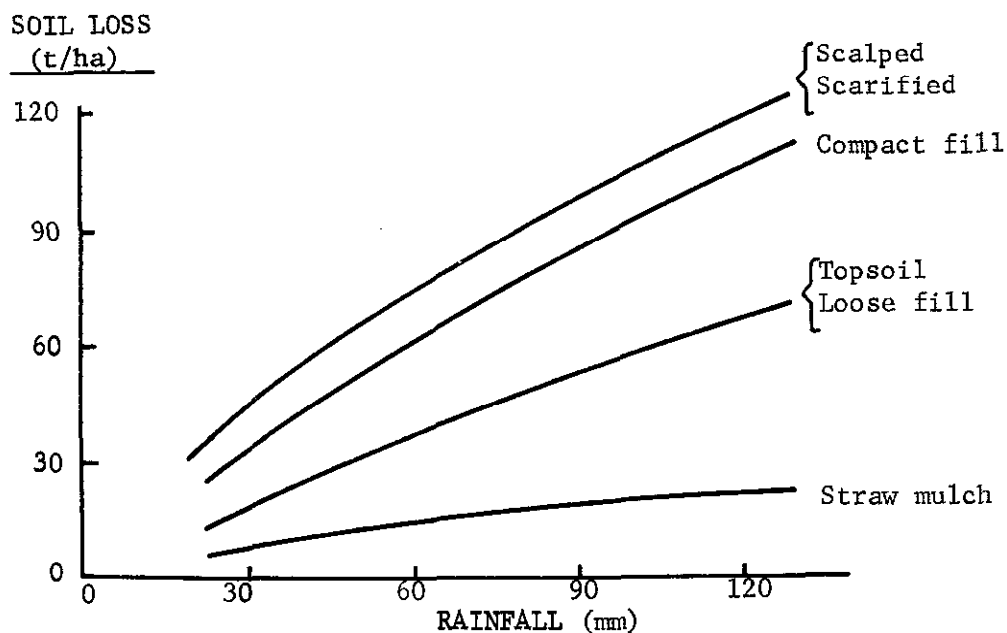


Figure 5. Erosion for several typical construction conditions during intense rainstorms. Miami subsoil, 12% slope.

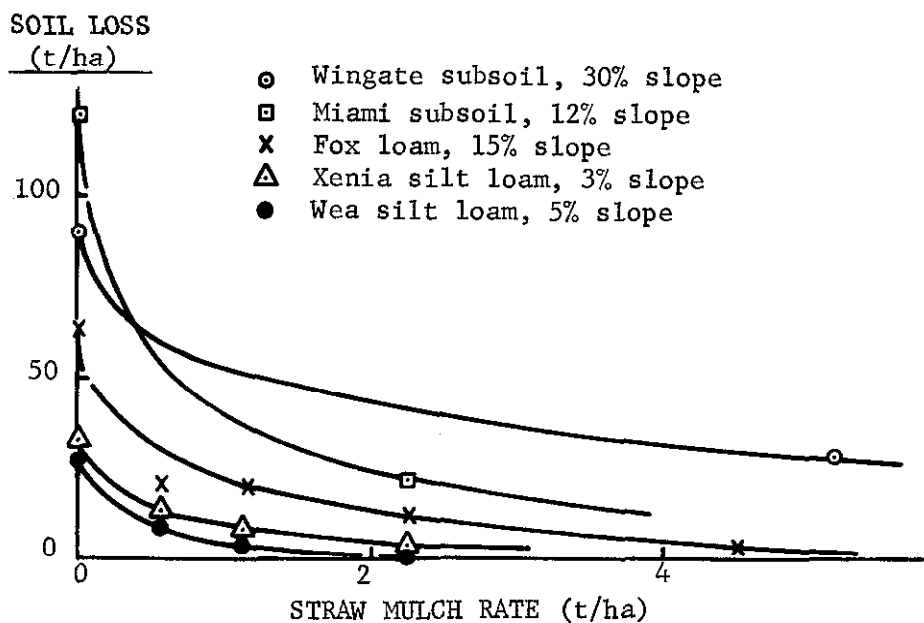


Figure 6. Effect of straw mulch rate on erosion for different erosion hazards.

erodible soils, 1 to 2 t/ha (1/2 to 1 ton/acre) of straw prevented serious erosion (Meyer et al., 1970). However, 4+ t/ha (2 tons/acre) were required to control erosion on 15% slopes, and this rate did not adequately control erosion on a very erodible soil at 20% slope (Meyer et al., 1972b). Some mulch rates that are effective on short slopes may lose their effectiveness on longer slopes, just as some that were effective on small steepnesses lost their effectiveness on steep slopes.

Mulches of road stone and woodchips have important potential for erosion control on reshaped slopes where more conventional mulches are unsatisfactory (Meyer et al., 1972b). As indicated by Figure 7, moderate rates of stone and woodchips were as effective as 5 t/ha (2+ tons/acre) of straw mulch and were much less erodible than bare soil on 20% borrow-pit sideslopes. Higher applications of stone at 300 t/ha (135 tons/acre) and greater or woodchips at about 56 t/ha (25 tons/acre) stabilized 20% slopes up to 45 m (150 ft) long, and they were less than one-tenth as erodible as the straw-mulched slopes. Stone and woodchip mulches are less easily blown away by wind than straw. Also, they provide immediate erosion control, whereas vegetation takes time to become effective. Stone mulch will not wash away, and it can permanently stabilize slopes that cannot be revegetated because of unfavorable soil chemical or physical characteristics.

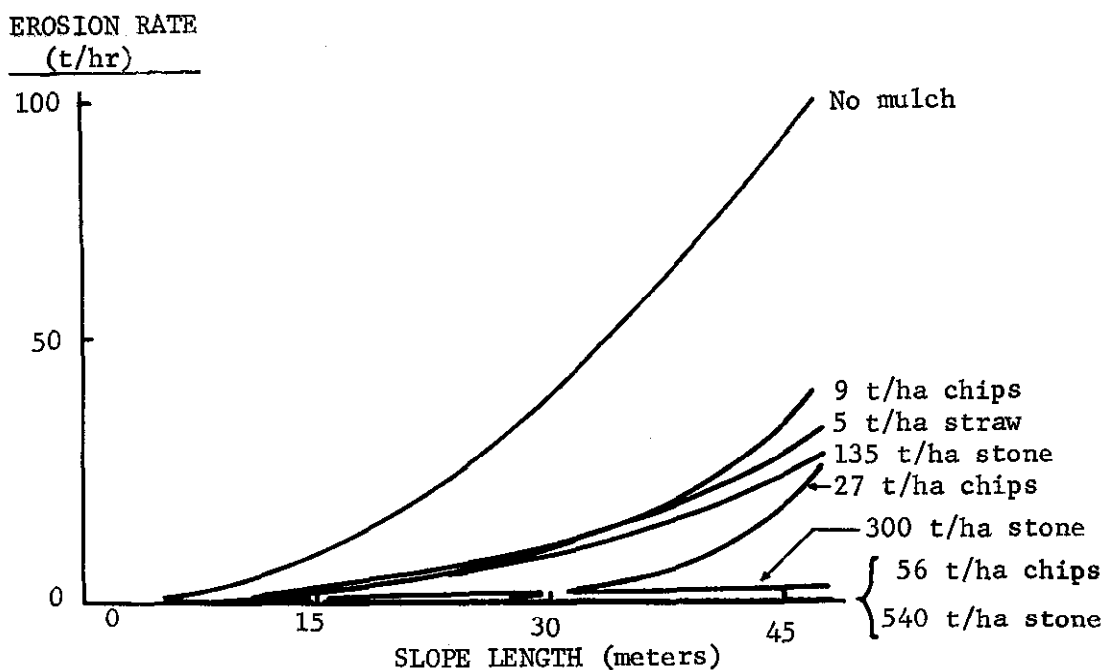


Figure 7. Effect of mulch type, mulch rate, and slope length on erosion rate per 100-m width during intense rainstorms. Wingate subsoil, 20% slope.

Rapid revegetation of reshaped land is important for erosion control. Mulching greatly enhances rapid seed germination and establishment of good stands (Meyer et al., 1971). Other practices that encourage revegetation include topsoil applied over subsoil, shallow tillage to improve the seedbed, fast-growing grasses in the seed mixture, and fertilization to start and maintain healthy growth. Straw mulches are most common, but stone and woodchips also improve the chances for rapid revegetation of subsoils as compared to no mulch (Meyer et al., 1972b).

Sediment basins Upland erosion-control practices such as mulching, revegetation, and topographic modification are generally preferable to downstream sediment control practices. However, when upland measures are not sufficient, sediment basins may be used to trap soil eroded from the upland slopes. Recent techniques (Boysen, 1974; Chen, 1974) enable planners to coordinate upland practices with sediment basin designs to achieve specified levels of downstream sediment reduction.

REFERENCES

- Barnett, A. P., and Rogers, J. S., 1966, Soil Physical Properties Related to Runoff and Erosion from Artificial Rainfall. Trans. ASAE 9:123-125, 128.
- Barnett, A. P., Tyson, B. L., and McGinnis, J. T., 1966, Soil Erodibility in Relation to Soil and Site Properties. First Pan Amer. Soil Conserv. Congr. Proc., Sao Paulo, Brasil, pp. 83-91.
- Boysen, S. M., 1974, Predicting Sediment Yield in Urban Areas. Proceedings of the National Symposium on Urban Rainfall and Runoff and Sediment Control. UKY BU106:199-203.
- Chen, Chang-Ning, 1974, Evaluation and Control of Soil Erosion in Urbanizing Watersheds. Proceedings of the National Symposium on Urban Rainfall and Runoff and Sediment Control. UKY BU106:161-172.
- Foster, G. R., and Meyer, L. D., 1972, Mathematical Simulation of Upland Erosion Using Fundamental Erosion Mechanics. Proceedings of the Sediment Yield Workshop, Oxford, Mississippi, ARS-S-40:190-207.
- Foster, G. R., and Wischmeier, W. H., 1974, Evaluating Irregular Slopes for Soil Loss Prediction. Trans. ASAE 17:305-309.
- Jackson, M. L., 1973, Soil Chemical Analysis - Advanced Course. 2nd Ed. 1st Printing. Dept. of Soil Sci., Univ. of Wisc., Madison.
- Meyer, L. D., 1974, Overview of the Urban Erosion and Sedimentation Processes. Proceedings of the National Symposium on Urban Rainfall and Runoff and Sediment Control. UKY BU106:15-23.
- Meyer, L. D., Foster, G. R., and Nikolov, S., 1974, Effect of Flow Rate and Canopy on Rill Erosion. ASAE Paper No. 74-2035 (mimeographed), accepted for publication in Trans. ASAE.
- Meyer, L. D., Foster, G. R., and Romkens, M. J. M., 1972a, Source of Soil Eroded by Water from Upland Slopes. Proceedings of Sediment Yield Workshop, Oxford, Mississippi, ARS-S-40:177-189.
- Meyer, L. D., Johnson, C. B., and Foster, G. R., 1972b, Stone and Wood chip Mulches for Erosion Control on Construction Sites. Jour. Soil and Water Cons., 27:264-269.
- Meyer, L. D., and Kramer, L. A., 1969, Erosion Equations Predict Land Slope Development. Agric. Engineering, 50, 522-523.

- Meyer, L. D., Wischmeier, W. H., and Daniel, W. H., 1971, Erosion, Runoff and Revegetation of Denuded Construction Sites. Trans. ASAE 14:138-141.
- Meyer, L. D., Wischmeier, W. H., and Foster, G. R., 1970, Mulch Rates Required for Erosion Control on Steep Slopes. Soil Sci. Soc. Amer. Proc. 34:928-931.
- Onstad, C. A., Larson, C. L., Hermsmeier, L. F., and Young, R. A., 1967, A Method for Computing Soil Movement Throughout a Field. Trans. ASAE 10:742-745.
- Ports, M. A., 1975, Urban Sediment Control Design-Criteria and Procedures ASAE Paper No. 75- , 1975 ASAE Winter Meeting, Chicago, IL.
- Romkens, M. J. M., Nelson, D. W., and Roth, C. B., 1975, Soil Erosion on Selected High Clay Subsoils. J. Soil & Water Cons. (In Press)
- Romkens, M. J. M., Roth, C. B., and Nelson, D. W., 1974, Erodibility of Selected High-Clay Subsoils as Related to Physical and Chemical Parameters. Agronomy Abstracts, p. 169.
- U. S. Department of Agriculture, 1951, Soil Survey Manual. Agr. Handbook No. 18. U. S. Govt. Printing Office, Washington, D. C.
- Wischmeier, W. H., 1962, Rainfall Erosion Potential, Agric. Engineering 43:212-215, 225.
- Wischmeier, W. H., 1973, Upslope Erosion Analysis, Environmental Impact on Rivers (Colo. State Univ., Ft. Collins, Colo.) Chapter 15.
- Wischmeier, W. H., Johnson, C. B., and Cross, B. V., 1971, A Soil Erodibility Nomograph for Farmland and Construction Sites. Jour. Soil and Water Cons. 26:189-193.
- Wischmeier, W. H., and Mannering, J. V., 1969, Relation of Soil Properties to Its Erodibility. Soil Sci. Soc. Amer. Proc. 33:131-137.
- Wischmeier, W. H., and Meyer, L. D., 1973, Soil Erodibility on Construction Areas. Highway Research Board Spec. Rept. 135:20-29.
- Wischmeier, W. H., and Smith, D. D., 1965, Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains. Agr. Handbook No. 282. U. S. Govt. Printing Office, Washington, D. C.
- Young, R. A., and Mutchler, C. K., 1969, Soil Movement on Irregular Slopes. Water Resources Research, 5:1084-1089.

DEFINING THE SEDIMENT TRAPPING
CHARACTERISTICS OF A VEGETATIVE
BUFFER. SPECIAL CASE: ROAD EROSION

by Coryell A. Ohlander, Staff
Hydrologist, River Basin Surveys and
Cooperative Watershed Programs,
State and Private Forestry, Region 2,
U. S. Forest Service, Denver, Colorado,
(303/234-4321)

Abstract

This paper discusses the amount of sediment trapped by a vegetative buffer that may exist below a road drainage outlet. The buffer characteristics are defined in relationship to slope percent, soil erodibility, and a soil-cover complex index called the Runoff Curve Number (RCN). A Standard Buffer is defined as a particular length of site capable of trapping 953 kilograms of sediment per year:

$$\begin{aligned} \text{Standard Buffer, meters} = & \\ & (16.16 + 17.69 \text{ Soil K} + 1.34 \text{ Slope percent}) \\ & \times (31\text{RCN}) / (6900 - 69 \text{ RCN}) \end{aligned}$$

Where: Standard Buffer is the slope distance needed for filtering out the sediment, soil "K" is the soil erodibility index from the universal soil loss equation, and RCN is the Runoff Curve Number from the Soil Conservation Service Engineering Handbook.

The ratio of actual buffer distance to the Standard Buffer distance times 953 kilograms equals the yearly sediment trapped.

The relationship of Road Erosion minus Sediment Trapped has implications that can be used as a first approximation of some land use effects on water quality.

Note: The relationship was created from a synthesis of existing research and prediction equations. The correlation "r" of 0.78 merely compares the results of two predictive equations.

Introduction

There has been a need in land use planning to determine as reliably as possible the sediment reduction benefits resulting from filtering sediment laden drainage water through vegetation before it is discharged to a water course. For the special case of road drainage, a

small amount of research is available for some parts of the country (Packer 1967), but nothing particularly suitable for general application. Rules of thumb are available such as "50 feet plus 4 times slope percent" (Trimble 1957), but these are not much help when definitive answers are needed.

The need for an improved definition of the Road Erosion-Sediment Trapped - Water Quality relationship gains emphasis from the current drives for improving water quality (Environmental Protection Agency 1973). One way to improve water quality is to leave an undisturbed vegetative buffer below road drainage outlets and allow the sediment to settle out. The problem is to build the buffer definition in such a way that the effects can be used with a broad level multi-objective evaluation process and yet allow stepping down into an implementation phase if the opportunity arises.

A correlary problem is that since current land use planning makes use of many different methods and levels of evaluation, a buffer evaluation method has to account for at least the major impacts and, if possible, use the currently accepted methods of the project or lead agency.

In the field of USDA river basin hydrologic studies and PL-566 watershed projects, the Soil Conservation Service is the lead agency. Their methods of determining the soil loss and hydrology of agricultural lands are well researched and generally known to practicing hydrologists. These methods form the basis of my procedures (Ohlander 1973,1975) that combine Paul Packer's sediment control guides (Packer 1967,1970) with the universal soil loss equation (Soil Conservation Service 1972) and Runoff Curve Number hydrology evaluation procedures (Soil Conservation Service 1963).

Development of the Standard Buffer Equation

The Standard Buffer equation was developed to predict the annual sediment reduction benefits obtainable for specific vegetation and soil conditions. The Standard Buffer is defined as the downstream length of a site necessary to annually filter out 953 kilograms (1.05 tons) of sediment from a point source of discharge.

$$\text{Standard Buffer, meters} = (16.16 + 17.69 \text{ Soil K} + 1.34 \text{ Slope percent}) \times (31\text{RCN}) / (6900 - 69 \text{ RCN})$$

Where Soil K is the soil erodibility index from the Universal Soil Loss Equation.

Slope percent is for the average side slope of the buffer.

The $(31\text{RCN}) / (6900 - 69 \text{ RCN})$ ratio compares different potential infiltration rates as indexed by the Runoff Curve Number methodology.

It is therefore a ratio of the standard infiltration rate (4.49"/hr., RCN=69) to that rate for the buffer under consideration. The assumption here is that as the infiltration rate decreases the buffer length has to increase proportionately.

To develop the equation, the Universal Soil Loss Equation was applied to the specific road sections (6 percent grade) used in Packer's Sediment Guides Table 2 (Packer 1970). This exercise resulted in an average of 953 kilograms of soil loss from the road. Next an RCN was assigned to the buffer that best approximated the conditions described for terrain, obstructions, and cover. Packer's Herbaceous Vegetation obstruction type was chosen and an RCN of 69 (Fair condition range, Soil Group B) was assigned. Thus, under the annual average rainfall assumptions used in the Universal Soil Loss Equation the Standard Buffer was able to trap and hold 953 kilograms of sediment.

Validation

The buffer equation is only one part of the following basic land relationship:

Road Erosion = Sediment Trapped + Sediment lost to stream.

To try to build on-site data to statistically validate each factor in the equations would be a nightmare, if not impossible. However, in the summer of 1973 some field plots collected for the North Platte Type IV USDA River Basin Study did lend themselves to a straight forward analysis. Road erosion, gully erosion, road drainage patterns, and buffer characteristics were sufficiently distinct on 79 field plots to allow analysis. Of these, 23 were particularly good for looking at the sediment trapped - road erosion relationship. The remaining 56 plots had several additional road or buffer features that complicated the analysis. A further overall complication was the lack of data on rainfall intensity and frequency available for the 12 years the roads were in place; an annual average situation was assumed for use in computing road erosion.

Measurements of slope percent, soil erodibility index, Runoff Curve Numbers, and actual sediment flow distances through the buffer were used to compute Sediment Trapped. A range of Runoff Curve Numbers from 26 to 81 was encountered within a mixture of vegetative types. Simple linear regressions comparing Sediment Trapped and Road Erosion (assigned the role of an independent variable) are shown in Table 1. Perfect agreement would require an intercept = "0", a slope of "1.0", and a correlation value, $r=1.0$. Please note that the correlation values merely compares the results of two predictive equations, both of which have unknown error. The statistical inferences from these correlation values are imaginary.

The first comparison in Table 1, approximates the scope of values in the secondary roads that Packer (1970) used. For the 9 plots having less than 953 kg. Road Erosion, the comparison is good and suggests to me that the Sediment Trapped-Road Erosion relationship is adequately defined. Comparisons 2 and 5 deal with Road Erosion values from 953 kg. up to 12,519 kg. and includes the extra complications of soil loss from gully and ditch erosion. For Road Erosion computations greater than 4535 kg. (5 tons) we predict greater Road Erosion than we have Sediment Trapped. As time goes by though, downed logs and branches will rot away and perhaps release sediment to move down hill again. The use of Runoff Curve Numbers helps to reflect a mature buffer rather than one with temporary benefits from downed logs.

Table 1 Five Different Comparisons of Soil Trapped
and Road Erosion Computations
for 79 North Platte, Wyo. Survey Plots

Linear Regression ($y=a+b(x)$)	Remarks
1. Soil Trapped = 0.0641 + 0.9596 Road Erosion Correlation $r = 0.78$	9 plots less than 953 kg (1.05 Ton) Road Erosion and non-complicated buffer analysis. Range in Road Erosion 84 kg-872 kg (0.0924 Tons - 0.9609 Tons)
2. Soil Trapped = 1.8217 + 0.4735 Road Erosion Correlation $r = 0.64$	14 plots over 953 kg (1.05 Ton) Road Erosion and non-complicated buffer analysis. Range in Road Erosion 1542 kg-12519 kg (1.7 Tons - 13.8 Tons)
3. Soil Trapped = 0.8864 + 0.6650 Road Erosion Correlation $r = 0.77$	All 23 "best" plots together
4. Soil Trapped = 1.2612 + 0.3884 Road Erosion Correlation $r = 0.58$	All 79 plots together with complicated Road Erosion and Buffer analysis.
5. Soil Trapped = 1.5099 + 0.3566 Road Erosion Correlation $r = 0.53$	All 44 plots over 953 kg (1.05 Ton) Road Erosion Range 998 kg-12519 kg (1.1 - 13.8 Ton)

CONCLUSION

My conclusion is that the relationship cannot be proved - or disproved - and that a planner is left with the feeling that the equation is - or is not - suitable for the job at hand. "To what extent can I afford to be wrong?" Seems to be the operable question.

As for me, I use it. I am comfortable with the necessary assumptions. They don't seem to be any more outlandish than many other assumptions used in River Basin planning. It is well to point out that the primary purpose of this exercise is not really basic research, but an attempt to make better use of existing procedures and improve our response to the dictates and needs of new planning criteria such as the Water Resource Council's "Principles and Standards" (USDA 1974).

Literature Cited

- Ohlander, C. A. 1975. Expose' in river basin planning - soil and water quality worksheets - North Platte Study. U. S. Forest Service, State and Private Forestry, Area Planning and Development Denver, mimeo.
- Ohlander, C. A., and C. Conklin. 1973. Some basic land relationships for a small computer. U. S. Forest Service, State and Private Forestry, Area Planning and Development, Denver, mimeo.
- Packer, P. E. 1967. Criteria for designing and locating logging roads to control sediment. Forest Service, Vol. 13, p. 107.
- Packer, P. E., and G. Christensen. 1970. Guides for controlling sediment from secondary logging roads. Intermountain Forest and Range Experiment Station.
- Trimble, G. R., Jr., and R. S. Sartz. 1957. How far from a stream should a logging road be located? Journal of Forestry, Vol. 55, p. 339-341.
- U. S. Department of Agriculture. 1974. USDA procedures for planning water and related land resources.
- U. S. Environmental Protection Agency. 1973. Processes, procedures, and methods to control pollution resulting from silvicultural activities. EPA 430/9-73-010 October 1973. Office of Air and Water Programs.
- U. S. Soil Conservation Service. 1963. National Engineering Handbook, Sec. 4, Supplement A, Hydrology. Third edition.
- U. S. Soil Conservation Service. 1972. Procedure for computing sheet and rill erosion on project areas. Technical Release No. 51. Engineering Division Sept. 1972.
- Wischmeier, W. H., and D. D. Smith. 1965. Predicting rainfall - erosion losses from cropland east of Rocky Mountains. U. S. Dept. of Agriculture Handbook No. 282.

GEOTECHNICAL MANAGEMENT OF FOREST LANDS IN GRANITIC TERRANE

By B. G. Hicks¹

ABSTRACT: Weathered granitic rocks have historically created problems for forest land management. Specialists are working to minimize the environmental damage resulting from road construction and logging impacts. Problems result when entry is made into steep granitic terrane with high rainfall where road construction and logging is planned. The damage is largely due to liquefaction type failures which produce greatly accelerated mass wasting as a result of debris landslides and flows. This geotechnical scheme is a unique plan for forest land management which produces a quantitative program for an environmentally safe operation and will yield an economically feasible project. The format of the scheme is: 1. Select a granitic area with stability problems, prepare detailed topographic map and obtain color and color infrared photography. 2. Hydrologic monitoring. 3. Engineering geologic study: mapping, drilling, sampling, geophysical surveys, field testing for stability parameters. 4. Soil engineering: lab and model test representative samples and compute stability parameters. 5. Correlate field properties with lab and model tests. 6. Prepare stability unit map. 7. Develop logging and road construction project using stability map and road design data.

INTRODUCTION

Weathered granitic rocks of the Sierran and Pacific Northwest batholiths have historically created problems for forest land management. Specialists of all types from various public and private organizations are working to attempt to minimize the environmental damage resulting from road construction and logging impacts. However, the problem has been compounded by (1) the difficulty in getting research applied to activities on the ground, as well as (2) the problems of integrating the input from the various specialists required to produce viable management schemes for forest lands. The scheme described in this report is the result of an intense effort to bridge this gap between research and field work, and to develop a rapid method for geotechnically oriented road construction and logging.

The major problems result when entry is made into granitic terrane with high rainfall where full bench construction is required and silvicultural practices call for clear-cutting or removal of large volumes of timber. These terrane impacts cause the maximum stress to any slope. In weathered granitics the damage is compounded by the potential for accelerated mass wasting resulting from debris flows and avalanches.

¹

Engineering Geologist, U. S. Forest Service, Rogue River National Forest, Medford, Oregon

For the first project, the major input will be from engineering geologists and soil engineers with only auxiliary data from soil scientists, hydrologists and silviculturists. It is probable that a better scheme could be developed by a program which more fully integrated all of the specialty fields, including ecologists.

The need for such a program is obvious to anyone familiar with the unnecessary damage and degree of accelerated mass wasting occurring in granitic terrane. Not only are unnecessary costs of road reconstruction occurring, but downstream damage to property and water supply facilities has become almost commonplace. The acreage lost to production by landsliding, increase in water temperatures, etc., are additional factors. However, their direct costs are not as readily established. Waste in any form at this time in history must be viewed as ignorance and our best efforts are required to reduce or eliminate waste throughout the system.

In addition to the above, the unnecessary energy consumption required to rebuild roads, replace bridges and culverts, clean reservoirs, etc., is a clear signal that the problem must be solved. This paper yields a direct method to do that for granitic terrane, as well as produce the framework for developing quantitative management schemes (geotechnical planning, it can be called) for any terrane. Additional work is required to develop a similar program for volcanic, metamorphic, sedimentary or any distinct geologic/geomorphic units producing clearly definable stability characteristics.

Granitic terrane as a specific material upon which to develop this quantitative geotechnical plan, was selected almost accidentally. The importance of this fortunate circumstance only became obvious during the final stages of the initial work (mid-1973). Stability solutions in the non-cohesive material (sandy, granular, permeable) produced by granitic rock weathering are readily amenable to simple analyses and calculations. By contrast, failures in cohesive materials require considerable more data and field input for solution. Once it was determined that granitic (non-cohesive) failures, whether they are roadfills or natural ground, are easily analyzed (liquefaction and flow, at least initially), the rationale for design of fills, stability in and beneath logging areas was established. Surface erosion is the only other contributor, but this too can be easily solved (empirically if not quantitatively). The problem then resolves itself into the building of roads which cannot fail when saturated or by erosion and developing logging schemes which do not produce sufficient additional ground water to cause failure. Empirical solutions are produced by a geotechnical study of existing failures correlated with lab and model tests. Field tests are devised from lab testing keyed to easily definable field properties (e.g., density, mechanical analysis, etc.). These data can then be extrapolated from areas of failure to non-failed areas to produce stability zonation maps.

The following is a program in progress at this time. The pressure of early completion, plus the obvious simplifications that the field data will undoubtedly yield will allow short circuiting of some of these steps. It remains the goal of full research to complete a long term investigation, clearly defining all properties and parameters in the accepted academic approach.

GRANITIC TERRANE GEOTECHNICAL STUDY

GENERAL

This study uses an area of major debris slides and fill failures in granitic terrane as a quantitative base upon which to develop logging impact and road design criteria for future operations in similar unstable areas. The study is based upon well established techniques and methods of engineering geology and soil engineering. Specific assumptions are required. However, they do not detract from the utility of the study. The granitic batholith involved is located on the Ashland District, Rogue River National Forest, Jackson County, Oregon. The study contains the following general steps described in detail later.

PHASE I - TOPOGRAPHIC MAPPING, AERIAL PHOTOGRAPHY AND HYDROLOGIC MONITORING

PHASE II - ENGINEERING GEOLOGIC STUDY

PHASE III - EXPLORATION PROGRAM AND FIELD TESTING (INSITU STABILITY PARAMETERS)

PHASE IV - SOIL ENGINEERING INPUT

- Lab testing
- Model studies
- Stability analyses
- Test correlation (Lab Properties versus Insitu Values)

PHASE V - "STABILITY UNIT" MAP PREPARATION

PHASE VI - NEW PROJECT (TIMBER SALE) DEVELOPMENT

- Logging and road design contract

DETAILS OF STUDY

The following outline summarizes the approach needed for the quantitative engineering geologic study and soil engineering investigation. The geologic study and soil investigation must be integrated into a timber sale contract for safe impact and stable construction in granitic terrane. The program will require specific reorientation and undoubtedly simplification as the study progresses.

PHASE I - TOPOGRAPHIC MAPPING, AERIAL PHOTOGRAPHY, AND HYDROLOGIC MONITORING

Topographic mapping: Maps with a scale of 1:6000 to 1:12000 and three to six meters (20- to 40-foot) contour interval is necessary. (Selection of scale and C.I. is function of slope and size of area.)

Features significant to the delineation of stability criteria are not apparent at the scale of the existing standard topographic mapping; i.e., 1:62500 and 25-meter (80-foot) contour interval. The detailed topography is also required to zone the impacts and to develop a usable stability unit map.

Aerial photography: Matched color and color infrared at a scale sufficient for detailed delineation of terrane units (e.g., 1:12000). High level NASA color infrared photography should also be obtained.

Hydrologic and Sedimentation Monitoring: Monitoring in the form of critically located precipitation gages, suspended load stream samplers (continuous or near-continuous), and stream flow measurements. The above monitoring is not required specifically for this study, but is necessary to separate man-caused versus natural events and to enable public land management practices to be compared with private land management practices. Measurements of failure volumes, bed load volumes and material stored in stream channels are also required for this latter portion.

PHASE II - ENGINEERING GEOLOGIC STUDY

Detailed engineering geologic study of area including (a) aerial photo interpretation, (b) hand and/or portable auger drilling and seismic surveying of failures (fills, cuts and blocks) and stable features, (c) field testing of auger holes for pertinent soil properties, (d) quantitative geologic/geomorphic stratification of failed and unfailed granitic terrane, (e) preparation of engineering geologic map interrelating the above preliminary quantitative slope stability parameters (i.e., preliminary stability unit map.)

This phase of the study involves determining general geologic and foundation engineering criteria for stability characteristics throughout the study area. However, data on water tables, depth versus density and relative permeabilities will be determined.

Engineering geologic studies must include quantitative geologic/geomorphic treatment of the granitic terrane landforms. Factors (e.g., slope, aspect, structure, fabric, rock type, etc.) must be developed so that the present landforms and the forces creating it are understood. The quantification of these factors will allow statistical and mathematical prediction of slope-forming processes. Computer-derived slope, aspect, isometric perspective and contour perspective maps will be generated for this study. The integration of the soil engineering data with the quantitative geologic/geomorphic input yields a dynamic approach to landform processes.

PHASE III - EXPLORATION PROGRAM AND FIELD TESTING (INSITU STABILITY PARAMETERS)

Phase III will allow representative sites to be selected. The sites selected for detailed analysis must then have detailed topographic maps prepared so that the failure geometry can be determined. Failure and non-failure sites should be selected to establish control.

The representative sites (logging area, natural cut slopes and fills) must be drilled and sampled to produce material for testing to yield data for stability analyses. Both disturbed and undisturbed samples are required. Exploration is being done by hand auger or portable rotary drill, with seismic surveying used to delineate between drill holes and to extrapolate beyond drill holes.

During drilling the following methods will be used to determine the field stability parameters to be correlated with the laboratory testing data.

Strength: Menard Pressure Cell or equivalent (strength parameters may be modeled from back-calculating of liquefaction failure and model studies.)

Density and Moisture: Nuclear Moisture/Density Probe.

Permeability: Above water table - Falling Head Permeameter

Below water table - Probably Pump Out Test
(depends on permeability)

Piezometers will be installed at depths determined later by the study to yield the necessary data for stability calculations. The piezometers must be installed to reflect the

effect of logged and non-logged areas versus failed and non-failed terrane. The influence of the different depth/permeability relationships in the stability units is being investigated. For the first project, a short cut will be used which involves the use of empirical back-calculation of failures and model data.

PHASE IV - SOIL ENGINEERING INPUT

Lab Testing: A soil lab testing program is planned on the samples from the exploration program. All pertinent stability parameters needed for input to the stability analyses will be obtained.

Model Studies: A model using natural materials will be designed to yield piezometric head versus failure potential.

Stability Analyses: Calculations will be made which yield the various critical stability factors for the stability units (e.g., the critical piezometric pressure). This data integrated with the quantitative geologic/geomorphic phase will allow delineation of the terrane susceptible to failure by the impact of logging. Appropriate scaling procedures will yield the safe logging volumes necessary for stability. The final product will be a map zoning the amount, location and type of impact that will yield acceptable results.

The samples from slope failures and failed versus stable fills will yield data for analyzing the methods needed to safely proceed with construction. Careful site selection and field testing are required. Undisturbed sampling will be difficult; therefore, the field tests such as density and permeability become particularly important.

Test Correlation (Lab Properties and Model Studies versus Insitu Values: The field parameters obtained from the insitu testing program must be correlated with their respective lab and model values used in the stability analyses. Thus the critical field values will be obtained. It is these limiting field values which will be investigated when the program is used in new project areas.

PHASE V - STABILITY UNIT MAP PREPARATION

With the critical parameters identified from the study area, a project to develop a new timber sale in the granitic terrane can be initiated. A lead time of at least one year is required in order to determine some relative precipitation and ground water movement data. The insitu strength parameters

developed during the above study are used in conjunction with quantitative geologic/geomorphic data derived from field investigation to develop a "stability unit" map for the new area. If anomalous terrane is encountered, it is felt that only minor lab testing will be necessary.

Once the stability unit map is complete, the preparation of a timber sale should be accomplished by a team effort. The ideal team would be composed of a timber specialist, engineering geologist, soil engineer and soil scientist. It is essential to have a timber specialist and an engineering geologist.

PHASE VI - NEW PROJECT (TIMBER SALE) DEVELOPMENT

The above study was designed to yield road design and construction costs. Once the timber sale appears feasible, road construction input should be made. The soil engineering portion of the study yields data for fill design, road surfacing, cutslope design, etc.

Input from the engineering studies is designed to yield safe, acceptable impact levels. The combination of the stability unit map and road costs will determine if an economic sale exists.

REFERENCES

- Gonsior, M. J., and Gardner, R. B. 1971. Investigation of Slope Failures in the Idaho Batholith. U.S.D.A., Forest Service Research Paper INT-97, Intermountain Forest and Range Experiment Station, Ogden, Utah.
- Hicks, B. G., and Collins, T. K. 1970. Use of Engineering Geology to Reduce the Impact of Road Construction and Logging on the Forest Environment. U.S.D.A. Forest Service Rept., Klamath National Forest, Yreka, California.
- Savina, M. 1974. Ashland Watershed. Geology and Slope Stability. U.S.D.A. Forest Rept., Rogue River National Forest, Medford, Oregon.
- Swanston, D. 1970. Mechanics of Debris Avalanching in Shallow Fill Soils of Southeast Alaska. U.S.D.A. Forest Service Res. Pap. PNW-103, Pacific Northwest Forest and Range Experiment Station, Portland, Oregon.
- Wilson, S. A. and Hicks, B. G. 1975. Slope Stability and Mass Wasting in the Ashland Creek Watershed. U.S.D.A. Forest Service Rept., Rogue River National Forest, Medford, Oregon.

EVALUATION OF AN EXTENSIVE SEDIMENT CONTROL

EFFORT IN THE LOS ANGELES RIVER BASIN

By: Earl C. Ruby, Hydrologist

Stanislaus National Forest

ABSTRACT

The Los Angeles River Flood Prevention Project, authorized by Congress in 1941, included sediment control. One component of the plan was to construct 852 small check dams, 24 to 30 feet in height, in key channels, using reinforced concrete cribbing. The small dams were located above sediment trap basins, for the purpose of reducing the volume of sediments which must be excavated from the basin. The dams are designed to function at maximum efficiency after they are full of sediments.

The study uses single regression analysis to relate the sediment yield of treated channels, for several years before treatment, to a norm. The treated channel is then related to the same norm to determine its post treatment performance to the amount of sediment expected without treatment. A second regression uses annual runoff, in place of the norm, to analyze the influence of check dams. The results of these two regression studies are similar. In order to evaluate and support the conclusions reached by the regression analysis, three other studies were done, using the sediment yield records, regressions, and t-tests. The conclusions of the study are:

1. There is a temporary reduction in the sediment yield to the basin during the time that the check dam system fills to mature debris cones.
2. Once the system is full, the sediment yield at the basin will return to normal, or perhaps slightly higher than normal.
3. The check dam systems do not influence the rate of sedimentation relative to runoff.
4. A burned watershed, or a major flood, can induce a treated channel to unload the temporary sediment storage in the system. This can be a small volume, or a catastrophic amount, depending on how many years have elapsed in which to store sediments.
5. The check dam systems can be breached by large boulders in major floods.

Field observations are included to help explain the results of the data analysis. An economic study concludes that the check dam systems return negative returns on the investment, based on a 45 percent sample of the total number of dams constructed.

INTRODUCTION

History

The Los Angeles River Flood Prevention Project was begun in 1941, as authorized by Congress in House Document 426, 77th Congress. One component of the treatment plan was to construct a system of low check dams, in key channels, for sediment control. In the mid-1950's, a concrete crib-type dam was developed as a pilot project. The first few crib dam systems appeared to be highly beneficial as debris control structures. In 1959, a treatment plan to install 852 of the crib dams was initiated. By 1972, more than 300 had been constructed on both Forest Service and private lands.

Purpose

A massive sediment control system has been constructed in the L. A. River Basin. More than 90 large sediment trap basins, with storage capacities of from 5 to 500 acre feet, have been installed on every major channel. Each basin was originally located above a residential development, to trap and desilt the flood flows. The desilted waters are then released to a concrete-lined channel which flows to the ocean. Each of the sediment basins must be excavated annually, at a cost of from \$1.50 to more than \$5.00 per cubic yard, in order to maintain a safety margin. (Ruby, 1974) The primary purpose of the check dams is to permanently reduce the volume of sediment reaching the control basin each year, thereby reducing the maintenance costs. The average annual yield is about 5,000 cubic yards/sq.mi., with extremes of from 1,200 to 62,000 cubic yards/sq.mi.

Original Design Assumptions

Many studies were conducted to estimate the benefits and functions of crib dam systems. The optimum dam height was determined to be 24 feet to 30 feet above stream bed. Each dam was expected to fill with sediment and reach maximum efficiency when the sediment cone was fully mature. A major function expected was to allow the unstable sideslopes of the channels to roll out onto the sediment cone and attain a stable angle of repose. Several benefits were expected:

1. The pit above each dam was designed to store from 5,000 to 15,000 cubic yards of sediments.
2. The sidewall and channel bed were expected to be stabilized, which would stabilize approximately 37 percent of the available sediments, and prevent their transport to the basin.
3. The sediment cone was expected to mature with a surface gradient which would be 7/10th that of the original channel gradient. Each sediment cone would reach the sill of the next higher dam, which would form a stable system.
4. Each mature sediment cone was expected to provide a stable channel bed which would eliminate down cutting, spread flood flows, reduce flood velocities, retard sediment movement, store water in the interspace, and control the direction of flood flows.
5. The primary direct benefit for each check dam system is the value of reduced cleanout costs at the basin.

DESCRIPTION OF THE STUDY

The study of these check dam systems is a post-construction evaluation to compare their actual performance to that which was expected. The data base is the existing records of sediment basin cleanout. There are some functions of check dam systems which cannot be evaluated by sediment yield records. Therefore, it is also necessary to use such tools as photographic history, field observations, and the accumulated experience of people, to supplement the analysis of sediment records. This evaluation includes four studies of the sediment records, an economic evaluation, and a record of field observations.

Regression Analysis

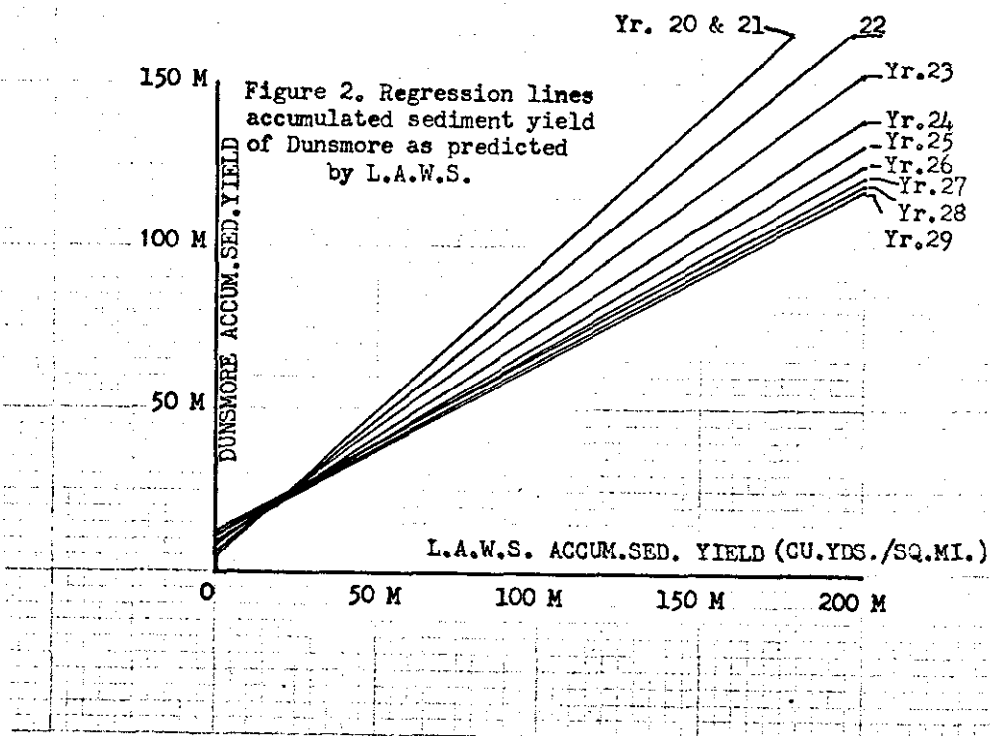
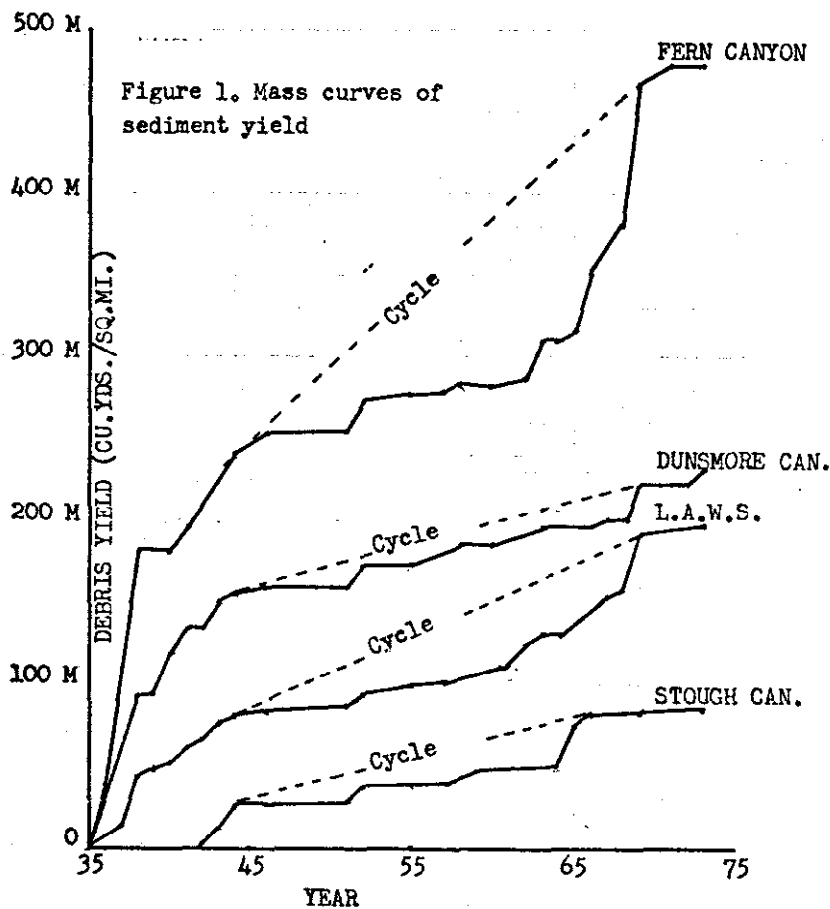
Two regression studies were done. The first (Ruby, 1972) used sediment records from Dunsmore Canyon watershed, as the dependent variable, and the Los Angeles Watershed System (L.A.W.S.) as the independent variable. The data was arrayed as sediment yield, per square mile, per year, for both variables. In the process of data analysis, it was discovered that there was a weather-related sediment cycle in the mass sediment yield curves. Figure 1 illustrates the sediment cycle for 3 selected watersheds and L.A.W.S. Dunsmore was chosen for evaluation because:

1. It is neither an extremely high, nor low sediment producer.
2. Within the sediment cycle, Dunsmore has 21 years of records before treatment and 9 years after treatment. This is the best set of records available.
3. All 9 check dams were installed in one work season, 1964.
4. The dams were located in an old alluvial fan, where the reduction in annual sediment yield was expected to be significant.

Dunsmore is an 0.84 square mile watershed, and L.A.W.S. is the average of all sediment basins in the L.A. Watershed. Seventy-three percent of the watersheds in L.A.W.S. are less than 1 square mile in area. The mass sediment yield data is correlated by regressing Dunsmore on L.A.W.S. for the sequence of years, 1944-1964, then again for 1944-1965, continuing to add one year of data each year until 1944-1973. Each regression line is plotted in Figure 2. The regression lines indicate the performance of Dunsmore in relation to the performance of the entire watershed system. The analysis indicates that:

1. For the first 21 years of the sediment cycle, Dunsmore correlated strongly with L.A.W.S., (correlation coefficient 0.97).
2. In 1965, (year 22) there was a significant shift downward in the slope of the regression line, and a continued additional shift downward for the next 7 years.

The definite change in the slope of the regression line during the first year after the check dams were installed indicates that the check dam system did reduce the sediment yield. This analysis is the first on record that graphically illustrates the functions of the check dam



systems. The systems had always been designed and evaluated by a series of assumptions here-to-fore. The indicated reduction in sediment was tested further to ascertain whether it could be expected to be a permanent or temporary condition. If the reduction is due entirely to the accumulation of sediment above the dams, in sediment cones, then the inflow to the basin can be expected to return to the pre-treatment normal. The volume of sediments above each check dam can be estimated from the measurements taken in 1964 (as built) 1966, 1971, and 1973. The accumulated sediment storage is estimated by the computer program DAMCAP and added to the record as though it had been yielded instead of stored. The regression analysis is then repeated as above. The change in the slope of the regression line (b value) is plotted in Figure 3. This analysis supports the following conclusions:

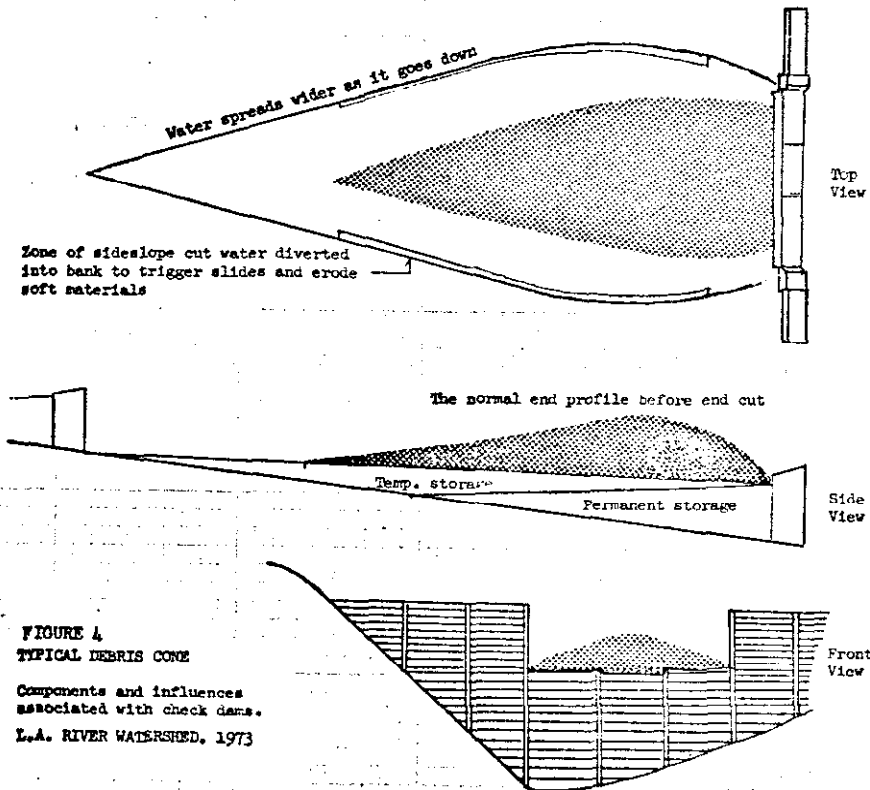
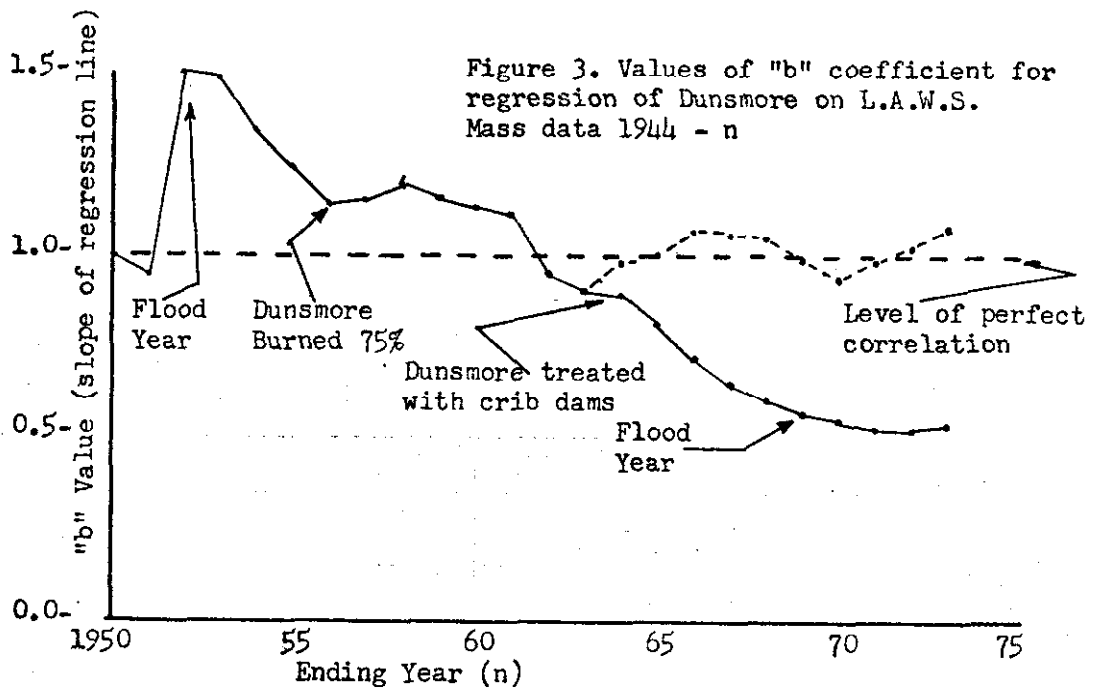
1. Using only the sediment basin records, there is an apparent 52 percent reduction in the annual sediment yield.
2. The apparent reduction can be entirely accounted for as storage in the check dam system by including the volume of each sediment cone in the yield data.
3. The apparent reduction is therefore a temporary condition which can be expected to last until the check dam system fills. The sediment yield at the basin can be expected to return to the pre-treatment normal.
4. There has been a slight, but statistically insignificant, increase in the sediment yield resulting from check dam construction.

Field observations in Dunsmore supplement the above study, in that 4 of the 9 dams have over stored sediment by from 9 percent to 155 percent more than the designed amount. The overload of sediments is a mound in the middle of the cone, which may be yielded to the basin in the next high magnitude flood. The lower 5 of the 9 dams have not yet filled to maturity.

The second regression analysis (Ruby, 1973) used annual sediment yield from Santa Anita Canyon (10.8 sq.mi.) as the dependent variable and total annual runoff as the independent variable. The sediment cone volumes for each of the 53 check dams were added to the annual yield record. In order to eliminate all other events which might also influence the sediment record, an estimated volume was subtracted to compensate for a large fire of 1953.

The period of comparability is 1927-1972 for both sets of data. The mass curves are analyzed as sediment yield regressed on runoff for the years 1927-1958 (without treatment) and 1958-1972 (with treatment). The study is designed to estimate the influence of the 53 check dams on the sediment yield into Santa Anita Reservoir. The conclusion of the study:

1. The 53 check dams installed in Santa Anita Canyon have not influenced the rate of sedimentation in the watershed relative to total annual runoff.



2. The effect of the check dams has been to store sediments in cones above the structures, without producing any significant change in the annual sediment yield.

This study also indicated that there was a slight, but not statistically significant, increase in sediment yield as a result of check dam construction.

Response of Check Dams to a 100 Percent Burned Watershed (Ruby, 1973)

A fire swept through portions of L.A.W.S. on 3/16/64, burning several metered watersheds, 10 of which were 100 percent burned. We are fortunate in that one of the watersheds, Brand, is the first to ever be treated with check dams in the L.A. Basin. The dams were built in the late 1930's and filled with sediment by 1952. Several good studies had been previously made of Brand Canyon and the adjacent Sunset Canyon, as paired watersheds. This present study is a continuation of the previous, using the same data. The evaluation includes four studies of the basin cleanout records:

1. Analysis of Brand (treated) and Sunset (untreated) as paired watersheds, using t-tests, for their entire 28 year history and the 8 years after the burn.
2. Analysis of the sediment unload response to the burn of all watersheds.
3. Comparison of the yield of each watershed to a norm (L.A.W.S.).
4. Analysis of the sediment yield for all burned areas in response to the 1969 flood, (a 30 year flood).

All of the studies were designed to evaluate the performance of the treated watershed in relation to the untreated watersheds. The study concluded that:

1. The treated watershed, (Brand), yielded significantly less than the untreated, paired watershed, (Sunset), until they were burned. In the post-burn sediment unload phase, Brand caught up with Sunset in accumulated sediment yield per square mile. There is no significant difference between Brand and Sunset for either the 8 years after the burn, or their 28 year history.
2. The check dams in Brand Canyon temporarily stored the sediments, as an overload on the sediment cones, then released them under the influence of the burned watershed.
3. The treated canyon yielded significantly more sediment in response to the burn than 8 of the 9 untreated watersheds.
4. In relation to all of the burned areas, the treated watershed yielded significantly more sediment than the norm, in response to the burn, but less than the norm in response to the flood of 1969. This may indicate that the burn had already stimulated a sediment unload prior to the flood.
5. There is no apparent change in sediment yield that can be credited to the check dam systems.

6. There is a slight, but statistically insignificant, increase in sediment yield which can be credited to check dam construction.
7. A benefit can be claimed for the temporary storage of sediments in the treated canyon, because final cleanout and disposal was delayed for at least 15 years.

Response of Check Dam Systems to a Major Flood (Ruby, 1973)

The winter rains of 1969 brought two major floods, of about a 30 year return interval. Most of the check dam systems had not filled to a level of mature sediment cone. The check dam systems should be filled in order to evaluate their most important function, which is their expected reduction in sediment yield after filling. There were 6 check dam systems that had been in place for 4 or more years, although only 1 had filled up to the design amount. Using the sediment yield of the 1969 flood, three comparisons were made. Treated channels, with 4 or more years of records, were compared to a selected group of similar untreated channels, with their own history, and with a norm, (L.A.W.S.). The analysis concludes that:

1. There was no significant decrease in sediment yield, in response to the 1969 flood, due to check dam systems.
2. The check dam systems did not make any significant contribution towards reducing other effects of the flood.
3. There are indications that the treated channels yielded more than they would have been expected to yield without treatment, although the data would need further study to prove the increase to be a statistically significant amount. The field observations indicate that the source of increased sediments may be the unload of excess storage on the sediment cone, and sidewall erosion.
4. Many of the crib dams in the larger watersheds were breached, by large boulders, or otherwise failed during the flood.
5. The treated canyons seemed to be highly effective sediment reduction projects until this large flood occurred.

The following photographs illustrate the process of overload/unload that seems to be typical of check dam systems. The process is also graphically illustrated in Figure 4.

Evaluation as Paired Watersheds (Ruby, 1973)

This is a study of several treated channels, as a group, to their own history of sediment yield with and without check dams. Regressions and t-tests are used to compare the data as mass sediment yield per sq.mi., and relative to a norm, (L.A.W.S.). The study is not an exhaustive analysis, but is done to improve the confidence of the previous studies. The conclusions are:

1. It is possible to illustrate the decrease in sediment while the system fills to maturity, but there are no indications of a permanent reduction in sediment yield at the basin.
2. Each check dam system seems to return to the normal, pre-treatment sediment yield after the dams fill.

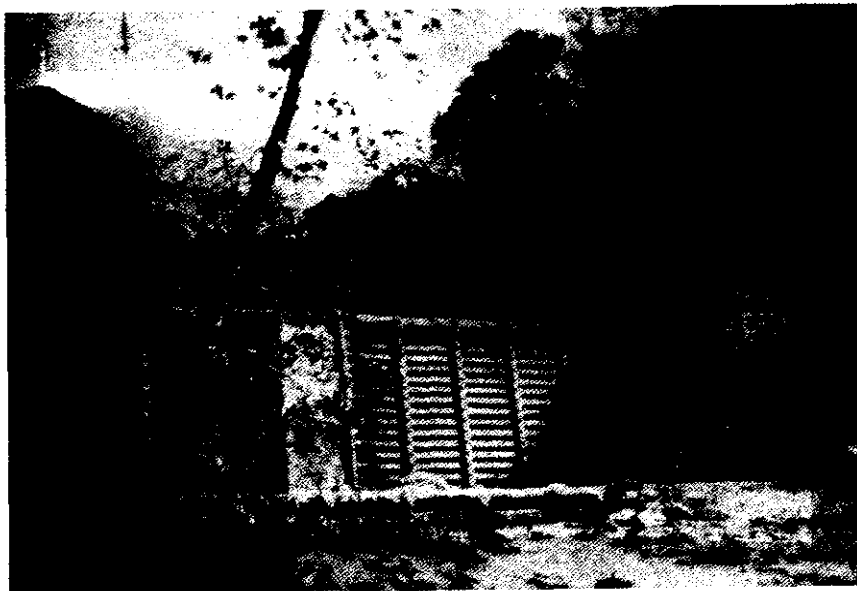


Photo #1. Taken September, 1966. Looking upstream to newly constructed dam number M-3, in the main stem of Sawpit Canyon.

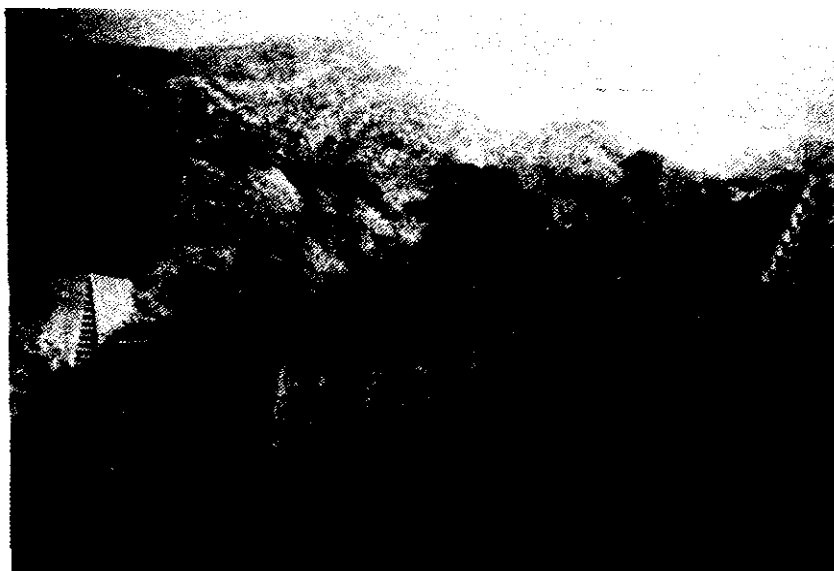


Photo #2. Taken November, 1968. The sediment cone is mature, and overloaded with a mound of sediments in mid-channel. The sediment cone is now ready to unload the excess storage in a high flood. Subsequent flows are now directed into the channel banks on either side. The spillway is constricted and can no longer pass the design flood.



Photo #3, taken March, 1969, after a major flood event, (50-year return). Looking upstream to the same check dam as the two previous photos. The mound of sediments has been washed off of the spillway, along with all of the vegetation that was growing on it. The sediment cone is now a flat plain, with only a shallow drift of sediments on the right. The typical overload/unload process may now start over with the deposition of a new sediment mound above the spillway, and the accumulation of temporary sidewall sediment storage. Note that the base flow is diverted over to the left, due to the meander which has been induced by the change in channel geometry. The next flood will be directed into the soft channel bank on the left of the photo. The sill has been damaged, although the gunite cap is still holding. The sediment cone of the next lower dam should have "toed out" on this sill, according to the design assumptions. Note in the photo that there is a gully where the sediment cone of the next lower dam should be.

Field Observations (Ruby, 1974)

After the sediment yield was analyzed, the check dam systems were examined to attempt some reasonable explanation for the results of the study. Listed below are those observations:

1. In deep alluvial deposits, where the channel has not yet reached bedrock, the check dams effectively stop downcutting. The flow of the floods must be directed to the center of the channel, perhaps with a V-notch spillway, or the sediments will drift up in mid-channel and direct flows into the sides.
2. The canyon walls may roll out onto the sediment cones and become stable for up to 15 years. A flood of greater than 15 year frequency may lower the level of the cone and leave the sidewall deposit suspended as a vertical bank. Once the bank begins to erode away, it will progress to the ridgeline or the extent of the deposit.
3. There is still a question on whether flood velocities are reduced, or increased. The change of the channel roughness from a boulder field to a fine-grained sediment cone may increase the velocity.
4. The check dam systems seem to reduce channel gradient, but this also introduces meander to the stream.
5. The normal, pre-treated channel is typically scoured out to a fairly wear-resistant channel due to the frequent floods. Above the scour line is a typical zone of sidewall sediment accumulation, which is out of reach of a "normal" flood. When the channel bed is raised by a sediment cone 30 feet deep, the future floods are elevated to the level of this ancient sediment storage. The meander of the stream, and diverting of flows by mid-channel debris drifts, are the two factors which cause the erosion of sidewall deposits.
6. The sediment cones are good dissipaters of mud flows.
7. At no time will the systems trap over 65 percent of the sediment yield, even if they are newly constructed.
8. In future floods the treated channels may be expected to yield moderate to large volumes of sediments which have been stored as an overload in the system. The most recent high magnitude flood did not unload catastrophic volumes, probably because the systems had not been in place long enough to accumulate the overload. The indications are that the time in which to store large volumes is the only component necessary to create the catastrophic event.

Economic Evaluation

There are three areas of direct benefits used in the economic evaluation, sediment storage, permanent and temporary, and floodwater control. A secondary benefit is the storage of useable water in the interspace of each sediment cone. The permanent storage is calculated. Temporary storage is assumed to accumulate as an overload in the system for 15 years, then unload in 5 years. The value of sediment storage is the actual cost of cleaning out the sediment basin below.

Floodwater benefits are claimed be assuming that floods are delayed due to spreading of the flow, and partially stored in the cones, which can be thought of as improving the capacity of the basin. The value is set at \$75./year for each system of dams, as an average value.

Water yield improvement assumes that each sediment cone will store 33 percent of its volume in the interspace. The value is \$35./ acre foot.

Three costs are considered, the initial investment, an annual maintenance cost of \$50./dam/year, and administrative costs of \$50./dam/year. Each cost and benefit are discount back to present worth, using the computer model, INVEST III, at four interest rates. The interest rates are the normal 6%, 5.5%, 10%, and the rate used in the pre-construction analysis. Tabulated below is a summary of the evaluation at the original interest rate, which was from 2 7/8% to 4%.

<u>SUMMARY ECONOMIC ANALYSIS DATA</u>					
Canyon Name	Net Pres. Worth Ben	Net Pres. Worth Cost	Net Pres. Worth Proj	Annual Equiv. Value	Ben Cost Ratio
Cooks	\$ 128,411	\$ 228,592	-\$100,181	-\$ 4,080	\$0.56
Dunsmore	\$ 392,753	\$ 441,963	-\$ 49,209	-\$ 1,869	\$0.89
Halls	\$ 684,804	\$ 649,531	+\$ 35,273	+\$ 1,382	\$1.05
Santa Anita	\$1,456,608	\$2,188,604	-\$731,997	-\$32,786	\$0.67
Sawpit	\$ 881,472	\$1,263,399	-\$381,927	-\$14,844	\$0.70
Ward	\$ 126,656	\$ 119,873	+\$ 6,784	+\$ 274	\$1.06
Sums	\$3,670,704	\$4,891,962	-\$457,035	-\$51,950	\$0.75

These six canyons represent 45% of all of the check dams constructed to date. The conclusion is that the check dam systems are of negative value for sediment control, even though two systems show marginal benefit/cost ratios.

REFERENCES

- Ruby, Earl C., 1972, Sediment Trend Study L.A. River Flood Prevention Project.
 Ruby, Earl C., 1973, Evaluation of Check Dams For Sediment Control.
 Ruby, Earl C., 1974, Review Report, Los Angeles River Flood Prevention Project.

SEDIMENT CONTROL AT IMPERIAL DAM

By T. H. Moser, Project Manager, and W. D. Sears, Supervisor, Imperial Dam, Yuma Projects Office, Bureau of Reclamation, Yuma, Arizona

ABSTRACT

The construction of major dams and reservoirs on the Colorado River has reduced the chance of major floods and generally stabilized the river. However, along much of the lower Colorado River, channel and bank stabilization work is still required to reduce the sediment transport to a practical minimum within multi-purpose concepts for control and usage of the river facility. In the 238 kilometer (148-mile) meandering river channel from Parker Dam to Imperial Dam, sediment is continuing to be transported downstream in large quantities. The amount transported is largely contingent on the flow in the river and the degree of stabilization. Some reaches of the river have received stabilization treatment while other reaches have not. Since about 1963 water has been released from storage at Parker Dam only in the amount required for downstream irrigation and domestic uses and for scheduled deliveries of Treaty water to Mexico. As a result of these conditions, an annual average of about 726,000 metric tons (800,000 U.S. tons) of sediment have passed Imperial Dam in recent years. In addition, however, there has been a continued buildup of sediment in the reservoir.

In the design of Imperial Dam, quite different sediment control facilities were provided for the two canal diversions. The All-American Canal desilting facilities consist of three double basins with 12 large rotating scrapers in each half basin. On the Gila Gravity Main Canal a flushing-type settling basin was used.

Sediment from the control facilities is returned to the river below Imperial Dam where it is sluiced downstream to Laguna Settling Basin. At this location sediment is periodically removed by dredge. Dredging is also required periodically to keep a channel open to the Gila Canal headworks.

INTRODUCTION

In the design concepts for Imperial Dam, it was recognized that large quantities of sediment would have to be removed from the water in order to provide relatively silt-free flow in the All-American Canal and the Gila Gravity Main Canal. Historically, the Colorado River had carried vast quantities of sediments picked up along the 2,400

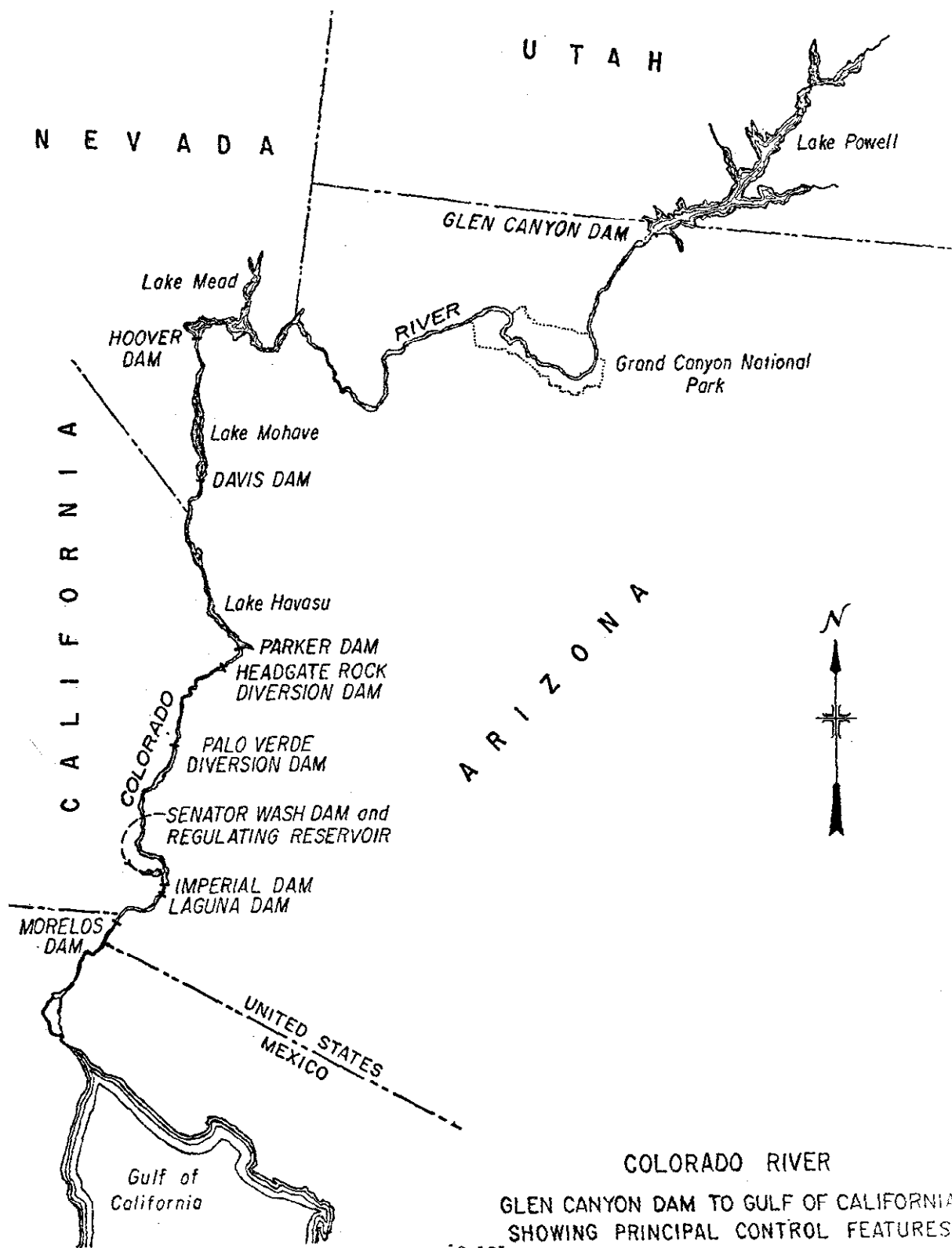
kilometer (1,500-mile) course from its headwaters in Colorado and Wyoming to the Gulf of California. Prior to the construction of Hoover Dam in the Black Canyon of the Colorado River, some 480 kilometers (300 miles) upstream from Imperial Dam, the river flowed uncontrolled through deep canyons and wide alluvial valleys. Ram-paging floods occurred during the spring snow melt and then the flow often diminished to a mere trickle during the late fall months.

Estimates of sediment, based on information and data collected prior to the closure of Hoover Dam, indicated that the average sediment load being transported past the damsite was in excess of 145,152,000 metric tons (160,000,000 U.S. tons) annually. (Borland and Miller, 1960) Below the site of Hoover Dam, the river generally flowed through desert-type terrain. In the valley flood plains, the river meandered and changed course frequently, scouring and depositing sediment as it progressed downstream.

With the closure of Hoover Dam in 1935, extensive changes of river regime were expected and have occurred. Trapping the large amount of sediment-laden water in Lake Mead and releasing clear water in regulated quantities to the channel downstream initiated river channel adjustment. These channel adjustments were increased and complicated by the completion of Parker and Imperial Dams (1938), Davis Dam (1950), and, to some extent, by Headgate Rock Diversion Dam (1942), and Palo Verde Diversion Dam (1958). (See Figure 1) The clear water releases from each dam picked up a sediment load from channel banks and bed causing degradation below each dam and aggradation in the backwater area upstream of each reservoir. These river adjustments caused problems in each reach of the river by either lowering or raising the water surface elevation.

Imperial Dam, located about 29 kilometers (18 miles) northeast of Yuma, Arizona, was designed and constructed as a diversion dam to raise the water surface so that irrigation diversions could be made to the All-American Canal on the California side of the river and to the Gila Gravity Main Canal on the Arizona side. The diversions each year amount to approximately 7,400,000,000 cubic meters (6,000,000 acre-feet) of water to irrigate over 485,650 hectares (1,200,000 acres) of rich farmland in the southwestern United States and northwestern Mexico.

The beginning capacity of the All-American Canal is 429 cubic meters per second (15,155 cubic feet per second (ft^3/s)). It serves the Imperial and Coachella Valleys in California and the Yuma Project in both California and Arizona, comprising approximately 271,150 hectares (670,000 acres) of irrigated lands. A large portion of the water delivered to Mexico pursuant to the 1944 Water Treaty is transported



COLORADO RIVER
GLEN CANYON DAM TO GULF OF CALIFORNIA
SHOWING PRINCIPAL CONTROL FEATURES

through the canal, being returned to the river upstream of Morelos Dam, Mexico's principal diversion point. The beginning capacity of the Gila Gravity Main Canal is 62.3 cubic meters per second (2,200 ft³/s). It serves five irrigation districts in Arizona, comprising approximately 46,540 hectares (115,000 acres) of irrigated lands.

During the first few years, the Bureau of Reclamation operated all facilities at Imperial Dam. Since 1952, however, the All-American Canal headworks and desilting basins have been operated by Imperial Irrigation District. The remaining facilities, including the Gila Canal headworks and settling basin and the river sluice gates, have remained under the operation of the Bureau of Reclamation, as well as the overall sediment disposal program.

Although the flows and sediment load arriving at Imperial Dam are not as great as originally anticipated, large quantities still prevail. In recent years an annual average of about 726,000 metric tons (800,000 U.S. tons) of sediment have passed Imperial Dam. In addition, there has been a continued buildup of sediment in the reservoir. The All-American Canal desilting basins and the Gila Canal settling basin have performed quite satisfactorily, although the means of disposing of the sediment removed in these facilities is vastly different from the design concept since large quantities of sluicing flows are no longer available.

WATER CONTROL OPERATIONS

The amount of sediment that is picked up and transported in the Colorado River is largely contingent on the flow in the river and measures undertaken to stabilize the banks and bed of the river. In the years after Imperial Dam was constructed, the flows were much greater than in recent years because of generally greater runoff, a lesser amount of up-river reservoir storage capacity, and lesser upstream water requirements. Since about 1963, the flows in the lower Colorado River below Hoover Dam have been regulated as a water conservation measure under the criteria that water will be released from storage only in the amount needed to meet downstream irrigation and domestic requirements in the United States and scheduled deliveries to the Republic of Mexico pursuant to the 1944 Water Treaty. These are scheduled for release from Parker Dam, the last major storage dam on the river, at a mean daily rate necessary to meet requirements at Imperial Dam and at other locations between Parker and Imperial. Within the mean daily releases at Parker Dam, however, the hourly releases can fluctuate to provide the most effective use of the water for power generation, utilizing the ability of the hydro-electric plant to meet peak power loads. To some extent the releases at Hoover Dam and Davis Dam can be varied to meet weekly or seasonal power peaks as well.

Travel time for the water to flow the 238 kilometers (148 miles) from Parker Dam to Imperial Dam is approximately 3 days. Therefore, the mean daily releases at Parker Dam are for requirements at Imperial Dam 3 days later. Scheduling the daily releases also involves consideration of diversions for the Colorado River Indian Reservation at Headgate Rock Dam, for the Palo Verde Irrigation District at Palo Verde Dam, and for other minor diversions upstream of Imperial Dam, as well as considerations of anticipated return flows from each irrigated area, transportation losses, and changes in channel storage.

To meet the water diversion requirements, some of which have changed in the 3-day interim of travel time, the operations at Imperial Dam are coordinated with those of Laguna Dam and Senator Wash Dam and Regulating Reservoir.

Laguna Dam, located 8 kilometers (5 miles) downstream from Imperial Dam, was the original permanent diversion structure on the lower Colorado River. Although the previous diversions at this structure are now served by Imperial Dam, it still has a vital role in water conservation and sediment control. Senator Wash Dam and Regulating Reservoir, an off-channel facility 3.2 kilometers (2 miles) upstream from Imperial Dam on the California side of the river, serves as a pump-back storage reservoir for reregulation of flows arriving at Imperial Dam.

Although Imperial Reservoir presently has practically no storage capacity, less than 1,233,500 cubic meters (1,000 acre-feet), regulation and utilization of the flows arriving at Imperial Dam have been extremely efficient. For the past several years operational waste, represented by deliveries to Mexico in excess of Treaty requirements, has been generally less than 0.5 percent. Practically the only waters arriving at the mouth of the Colorado River in the Gulf of California are from infrequent intense rain storms along the lower reaches of the river.

As a result of the high operational efficiency in recent years, the total annual flows have been reduced from previous years and the peak flows have been reduced. (Annual flows are listed in Table 1.) Sediment reduction attributable to reduced flows has not fully materialized due to recent river management activities that have temporarily increased sediment movement in some reaches of the river between Parker and Imperial Dams and also because of other unstable reaches on which no work has been performed.

SEDIMENT INFLOW TO IMPERIAL RESERVOIR

Sediment control at Imperial Dam is influenced by the condition of the various reaches of the river below Parker Dam. Under the Bureau of

TABLE 1
ALL-AMERICAN CANAL - SUSPENDED SEDIMENT LOAD

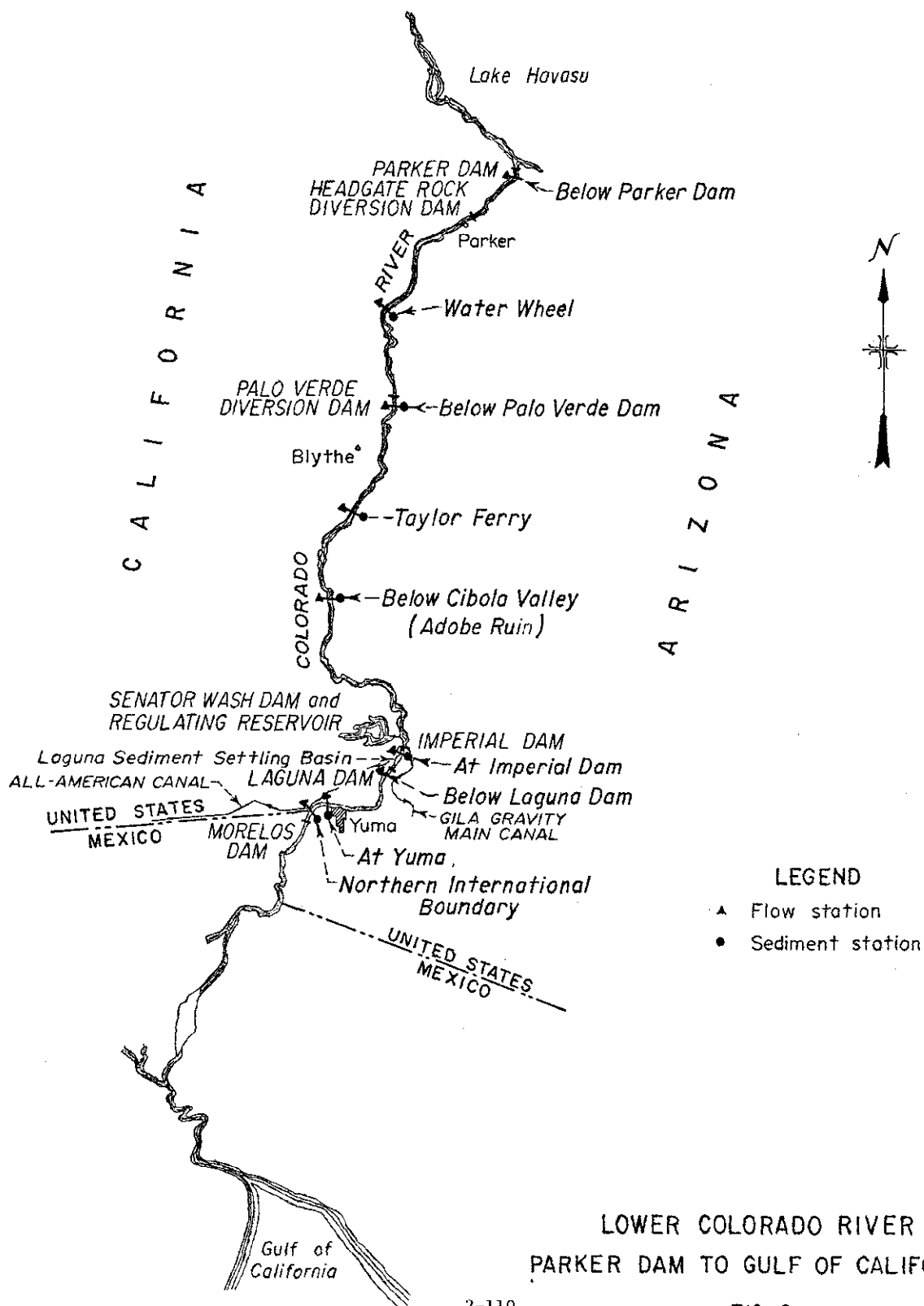
Cal. Yr.	Flow Arriving At Imperial Dam (1,000 AF)	Total Incoming Load at Headworks (1,000 U.S. Tons)	Load to All- American Canal (1,000 U.S. Tons)	Desilting Basins Sludge Return to River (1,000 U.S. Tons)	Percent of Sediment Returned to River
1957	7,261	1,098	372	726	66
1958	10,270	2,540	834	1,706	67
1959	7,500	425	296	129	30
1960	7,015	326	209	117	36
1961	6,294	363	199	164	45
1962	6,432	501	212	289	58
1963	6,517	505	276	229	45
1964	5,894	535	205	330	62
1965	5,714	746	238	508	68
1966	5,825	992	354	638	64
1967	5,594	642	234	408	64
1968	5,749	584	202	382	65
1969	5,608	500	192	308	62
1970	5,691	831	352	479	58
1971	5,832	669	230	439	66
1972	5,795	638	202	436	68
1973	5,856	657	183	474	72
1974	6,220	817	205	612	75
Average	6,393	743	278	465	63

Reclamation's river management program, several reaches of the river have been stabilized or partially stabilized while others have not. The objective of the river management program is to provide a stable confined channel which will efficiently carry normal riverflows and safely pass floodflows. Many other benefits are derived from this program, such as water salvage, sediment reduction, improved channel for navigation by small boats, preservation and enhancement of fish and wildlife values, development of recreational facilities, and enhancement of land values.

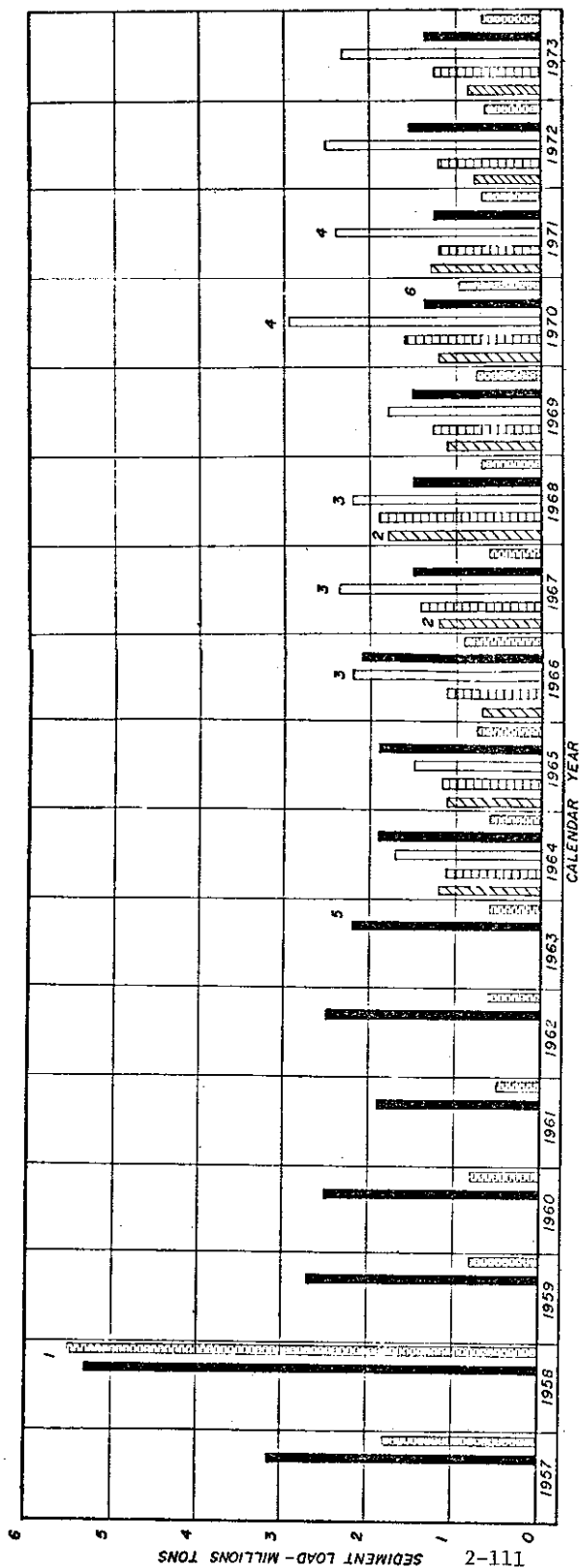
On the map of the Colorado River from Parker Dam to the Gulf of California (Figure 2) are shown the stations for the measurement of flow and sediment. Figure 3 is a bar graph of the amount of sediment being carried in the river at various locations between Parker Dam and Imperial Dam. Notes on the graph indicate the causes of the principal changes.

Except for some recent sediment removal by dredge immediately upstream from Imperial Dam, no stabilization or channel improvement work has been performed in the 77.2 kilometer (48-mile) reach of the river immediately above the dam, identified as the Imperial Division. When Imperial Dam construction was completed in 1938, the water level of the river was raised 7 meters (23 feet), creating a 104,847,500 cubic meter (85,000 acre-foot) impoundment. As anticipated, the reservoir largely filled with sediment within a few years, flattening the river gradient throughout the Imperial Division and creating many bays, lakes and side channels. As this segment of the river has aggraded, much of the sediment has deposited in the side bars which formed low natural levees and have helped to maintain the integrity of the channel. Continuing aggradation and periods of higher flow have, from time to time, breached the natural levees and deposited sediment into the backwater areas. Continuation of these processes will reduce the size and depth of backwater areas and will also result in a reduction of sediment trap efficiency.

Trap efficiency is defined as the difference in sediment inflow and outflow divided by the sediment inflows. For many years after construction of Imperial Dam, the trap efficiency of the Imperial Division was over 70 percent. From 1965 through 1969 the efficiency was in the range of about 60 percent, but decreased in 1970 to 33 percent and was 43 percent in 1971. This sudden decrease in trap efficiency with a corresponding increase in sediment at Imperial Dam resulted largely from the opening in March 1970 of a new dredge channel in the reach of the river immediately upstream of the Imperial Division. The new cut excavated to a depth somewhat below design grade, acted to some extent as a trap and caused a temporary degradation of the channel in the



LOWER COLORADO RIVER
PARKER DAM TO GULF OF CALIFORNIA



CAUSES OF PRINCIPAL CHANGES OF SEDIMENT LOAD

1. Increase due to high peak and total annual flows.
2. Increase due to construction of bankline structures upstream in Parker Division.
3. Increase due to construction of bankline structures in Palo Verde Division.
4. Increase due to opening of new dredge-constructed channel in Mar. 1970 in Cibola Division.
5. From 1963, releases from storage at Parker Dam limited to downstream requirements for irrigation and domestic use. No large sluicing flows after this time. Beginning in 1966 peak and total flows reduced further by additional river regulation provided by Senator Wash Dam. Had the effect of reducing trap efficiency of Imperial Division.
6. Opening of Cibola Cut and subsequent degradation of upper reaches of Imperial Division further reduced trap efficiency of Imperial Division.

NOTE: FOR YEARLY FLOW AT IMPERIAL DAM, SEE TABLE 1.
FOR LOCATION OF SEDIMENT STATIONS, SEE FIGURE 2.

SEDIMENT LOAD - LOWER COLORADO RIVER
FIGURE 3

upper portion of the Imperial Division. The degraded channel caused the suspended sediment to pass through this reach without spilling over the natural levees. The trap efficiency was 56 percent in 1972 and 49 percent in 1973.

SEDIMENT CONTROL AT IMPERIAL DAM

At the time Imperial Dam was designed, much of the flow of the river was not used for irrigation. This flow carried very large quantities of sediment. It was estimated that great quantities of silt, in the magnitude of many millions of cubic yards per year, would continue for many years. Although it was recognized that future irrigation developments within the Colorado River basin would ultimately utilize a large share of the total water supply of the river, it was contemplated that there would remain sizeable quantities of surplus water that would be available for transporting downstream the sediments removed at the desilting works of the All-American Canal and Gila Gravity Main Canal.

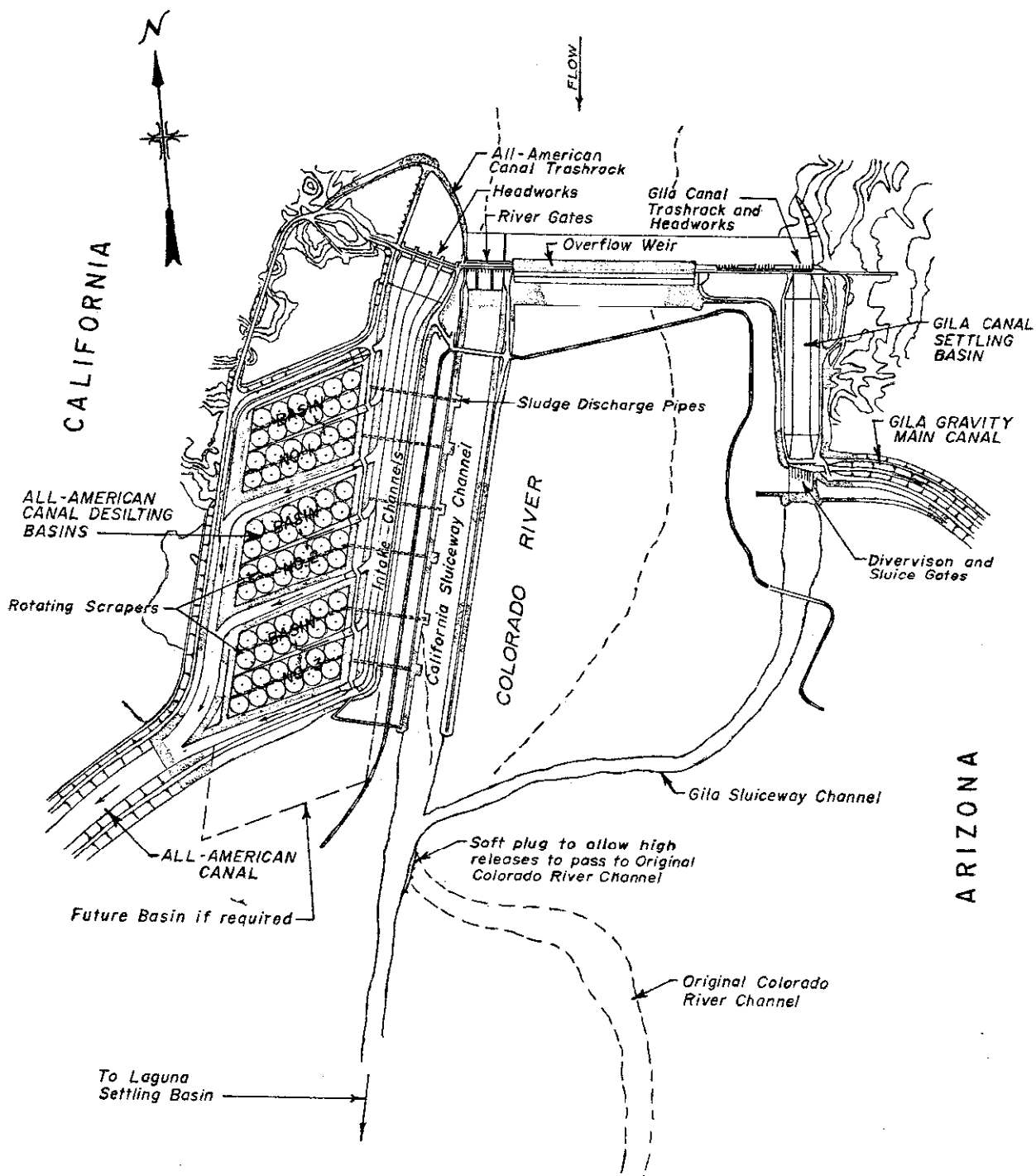
Based on these assumptions, it was necessary to design the most effective type of desilting facilities that would prevent all but the very fine material from entering the canal systems. A large number of model tests to develop a design were conducted in the Denver, Colorado, hydraulic laboratories of the Bureau of Reclamation. Tests were also conducted at a field hydraulic laboratory at Montrose, Colorado.

In this report we will cover the features of Imperial Dam only to the extent necessary to explain the functioning of the desilting works. Complete design and construction details can be found in the Bureau of Reclamation's publication of final reports of "Imperial Dam and Desilting Works." ^{1/} We will, however, explain the performance of the desilting works, covering the operation, maintenance, costs and major problems.

ALL-AMERICAN CANAL DESILTING BASINS

The principal features of Imperial Dam are shown on Figure 4. Water is routed to the All-American Canal through the trashrack into four intake channels of approximately equal capacity, each controlled by a 22.9 meter (75-foot) roller gate. The intake channels, designed for a nonsilting velocity of about 1.8 meters per second (6 feet per second)

^{1/} RECLAMATION, BUREAU OF, Denver, Colorado, "Imperial Dam and Desilting Works-Final Reports, Part IV-Design and Construction, Bulletin 6, Boulder Canyon Project" 1949.



PRINCIPAL FEATURES OF IMPERIAL DAM

deliver water into the influent channels and thence to the basins. The influent channels are also designed to maintain a non-silting flow and to allow increments of water to escape into the desilting basins at regular intervals through 7.6 centimeter (3-inch) vertical slots. In the basins the velocity of flow is reduced to 7.6 centimeters per second (0.25 feet per second) allowing the sediment to deposit. The clear water is discharged over full-length weirs into the effluent channels, which combine to form the All-American Canal. The sediment on the floor of the basins is moved by 38.1 meter (125-foot) diameter scrapers, which revolve continuously to push the sediment to collection troughs in the center. The sediment is then flushed back to the main river channel below the dam. Periodically, the river sluice gates in the dam are opened and the sediment is sluiced downstream to the Laguna Settling Basin.

The All-American Canal desilting basins were designed on the basis of a river flow of 595 cubic meters per second (21,000 ft³/s) and diversions to the canal of 340 cubic meters per second (12,000 ft³/s) with the maximum daily sediment loads reaching the headworks estimated at 81,600 metric tons (90,000 U.S. tons). Of this, it was intended that the basins would remove approximately 63,500 metric tons (70,000 U.S. tons).

The plan adopted for the All-American Canal desilting works provided for construction of three basins, each designed for a flow of 113.3 cubic meters per second (4,000 ft³/s). The plan also provided for a future addition of a fourth basin without interfering with existing facilities, at a location shown in Figure 4. The basins were laid out in the form of parallelograms with the influent and effluent channels at an angle of 60 degrees with the intake channels on the east and with the upper end of the All-American Canal on the west. The 60-degree angle was selected to provide the best fit of the rotating scrapers with the least amount of dead area not controlled by the scrapers. Each of the basins was divided in half by an influent channel. The dimensions of each half-basin are approximately 82.3 by 234.7 meters (270 by 770 feet), measured along the sides of the parallelogram. The levees, forming the long sides of the basins, act as skimming weirs for the discharge of clear water into adjacent effluent channels and thence into the All-American Canal. Bypass gates were provided to allow the flow to pass directly from the intake channels to the effluent channels in the event of an emergency that would make the basins inoperable.

In each half-basin are two rows of six scraper mechanisms. Each mechanism consists of two diametrically opposed scraper arms revolving about a central pedestal on which the motor drive is mounted. Each

scraper sweeps a 38.1 meter (125-foot) circular area. Diagonal scraper blades move the silt toward the central pedestal collector trough to which the sludge discharge pipes are connected. The sludge discharge pipes in turn are connected to an interceptor line which extends lengthwise under the center of each half-basin. The six interceptor lines discharge to the California Sluiceway Channel below the dam. Valves are installed in the pedestals to regulate the volume of flow necessary to hold the silt to a desired concentration, assumed in the design to be 10 percent.

The influent channels are unique in configuration. They were designed to create an even distribution of water to the basins. Each of the converging side walls is actually a series of separate short walls about 2.4 meters (7 feet) long, separated by a succession of influent slots, totaling 110 for each side. Each short wall is parallel to the channel centerline, but is offset inwardly about 6.4 centimeters (2-1/2 inches) from the immediately upstream preceding one in such a manner that at the last pair of slots the distance between the two walls has been reduced from 13.7 meters (45 feet) to slightly less than 12.7 centimeters (5 inches). The slots are about 7.6 centimeters (3 inches) wide at the throat and 3.7 meters (12 feet) high.

Although Imperial Dam was completed in 1938, the All-American Canal was not placed in complete operation until 1941. By this time the large reservoir created by the construction of the dam had largely filled with sediment. Sediment in the water entering the All-American Canal didn't reach objectionable levels, however, until 1945. At that time the desilting works were placed in operation. For the most part the basins have been operated as envisioned by the designers with only a few minor changes. Seldom, if ever, however, have the basins been operated with the sediment return to the river near the proposed 10 percent concentration. This heavy concentration could not be handled effectively without clogging the pipes. The general manner of operation has been to regulate the water discharge valves to release approximately 1.4 cubic meters per second ($50 \text{ ft}^3/\text{s}$) from each half-basin, which results in a sediment concentration generally less than one percent. It was also found in actual operation that the influent channel slots were not causing an even distribution of flow to the basins, especially on the south side. To correct this problem, metal slot deflectors were installed on all but the last four slots on the south side and on the last three slots on the north side.

The desilting basins are considered to be performing an efficient role in protecting the All-American Canal from sediment problems. An annual centerline profile and selected cross-sections of the canal have indicated little change from the original condition. The amount

of sediment entering the canal is of such small particle size that it stays in suspension and is generally less than 70 grams per cubic meter (0.1 ton per acre-foot). This indicates an efficiency considerably better than anticipated in the design. Primarily, the greater efficiency has resulted from substantially less quantities of sediment being transported in river flows than was contemplated. The annual quantities of suspended sediment diverted to the All-American Canal desilting basins and the proportions returned to the river and carried into the canal are shown in Table 1.

Operation of the basins requires a two-man crew around the clock to perform routine functions, such as unplugging valves, keeping the slots clear of moss and other trash, and general housekeeping. Total operating costs for the basins, including electric power, labor and all general operating costs, was about \$85,000 for Calendar Year 1974. This indicated an operating cost of about \$0.017 per acre-foot of water diverted to the basins.

Routine maintenance of the basins consists of repair of electric motors and gears, replacement of valves, general repairs of the pedestals and scrapers, and the painting of metalwork. Imperial Irrigation District has installed a complete cathodic protection system on all metalwork, which has reduced the maintenance costs by such a large degree that similar installations have been made on the entire All-American Canal system and the remaining portions of Imperial Dam. Total maintenance costs for the basins during 1974 was about \$21,000. This cost, however, varies considerably from year to year depending on the amount of major maintenance work on the basins. Because of the variance, it is difficult to establish an average cost per acre-foot, but probably it would be something less than \$0.01.

The major maintenance problem in the basins has been with the sludge discharge pipes to the river. These pipes range in size from the 20.3 centimeter (8-inch) diameter discharge from each pedestal to 91.4 centimeter (36-inch) diameter for the main discharge pipes to the river. About 1961, leaks began developing along the bottom of the pipes. Various methods of patching were tried with the best results obtained from fiberglass sheets cemented with epoxy. It was determined that the leaks occurred because of erosive action of the sediment. Apparently, the velocity is not great enough to keep the sediment in suspension.

As leaks occurred more frequently, all of the pipes that were accessible were rotated 180 degrees. This was only successful for 3 to 4 years before leaks again occurred. In 1970, all of the sludge pipes were lined with cement mortar. For all accessible sections of straight

pipe, the mortar was machine-applied at a uniform thickness of 1.0 centimeter (3/8-inch). At other locations the mortar was hand-placed. An inspection in the winter of 1972-73 revealed that some sections of the mortar lining showed appreciable deterioration, the lining either being washed out or worn through. This condition required additional hand patching at that time and a planned program of inspection and patching as necessary each winter season. Eventually, complete replacement will be necessary. The last segment of pipe between the sludge gallery and the river for the north half of Basin No. 2 has already been replaced, since it was not feasible to rotate this buried pipe.

GILA CANAL SETTLING BASIN

In the design of the Gila Canal settling basin, control of sediment into the Gila Gravity Main Canal was not considered as critical as for the All-American Canal, primarily because of the difference in water diversions and the more sandy soil characteristics of land irrigated from this canal. Because of this, a flushing-type settling basin was selected as the most desirable (See Figure 4). It was contemplated that with a 56.6 cubic meters per second (2,000 ft³/s) diversion from the river the settling basin would remove approximately 80 percent of the sediment.

It was also assumed that water would be available for near-continuous flushing of sediment from the basin if necessary. In the initial operation of the basin, rather large sediment loads entered and it was necessary to flush the basin several times each year. In addition, a continuous flow of approximately 85.0 cubic meters per second (3,000 ft³/s) was diverted through the basin for about 2 to 6 weeks each year to move the sediment downstream and to maintain the integrity of the Gila Sluiceway Channel. Then a shift of the main channel in the reservoir feeding this canal caused the sediment to settle out before entering the settling basin, and for the past several years, it has not been necessary to flush the basin more than once a year. For this sluicing operation, a flow of 56.6 cubic meters per second (2,000 ft³/s) is generally used for a period of 2 to 3 hours. The sediment deposited in the reservoir has been removed by dredge.

The basin was designed for desilting a flow of 56.6 cubic meters per second (2,000 ft³/s), the original capacity of the Gila Gravity Main Canal, and it was planned to construct two additional basins for an ultimate canal capacity of 170 cubic meters per second (6,000 ft³/s). With the passage of the Gila Reauthorization Act of 1947, the maximum capacity of the canal was set at 62.3 cubic meters per second (2,200 ft³/s). It was considered that a 10 percent increase over original design capacity would not materially reduce the effectiveness of the basin. This has proven to be true.

With the requirement to sluice the basin only once each year and because of limited quantities of sluicing water, a build-up of sediment occurred downstream of the basin. To correct this situation, the sluiceway channel was realigned in 1970 so that it now discharges into the California sluiceway channel. With this modification, the sediment is now carried to the Laguna Settling Basin, where it is later removed by dredge.

Maintenance of the concrete-lined Gila Canal settling basin has been minimal except for one occasion following World War II when the U.S. Army was using the basin and sluiceway for the testing of pontoon bridges. A rapid change in water elevation caused considerable damage to the basin from uplift of the floor slab. Since that time, strict criteria for draining and refilling the basin have been in effect. The basin is drained and refilled at a rate not to exceed 1.2 to 1.5 meters (4 to 5 feet) elevation change each hour as a precaution in case the tile drains beneath the floor slab will not have capacity to relieve back pressure or have become plugged.

Water from the reservoir enters the basin through three radial headgates. The diversion works at the downstream end has two sets of gates, arranged with one set directly above the other so that the flow may be either taken from the top of the basin into the Gila Gravity Main Canal or passed under the canal at sluicing velocities capable of sweeping the accumulated silt from the basin. There are eight pairs of fixed wheel gates for the diversion and sluiceway structures.

In recent years it has been necessary to take the Gila Gravity Main Canal out of service for approximately one week each fall for maintenance work. During the outage, one diversion and one sluiceway gate are replaced with a spare set of gates that have been overhauled and painted. Even by replacing the gates every 8 years, considerable maintenance work has been necessary to replace broken hoisting stems, sheared pins, and hoisting nuts. Maintaining proper alinement of this type of gate in the gate guides has also been a problem. Maintenance of the headgates can be accomplished at any time by lifting the gates clear of the water.

The Gila Canal settling basin has been effective in keeping the canal free of sediment. At each annual sluicing operation about 18,100 to 27,200 metric tons (20,000 to 30,000 U.S. tons) of sediment are removed from the basin. The amount of sediment entering the canal averages less than 70 grams per cubic meter (0.1 U.S. ton per acre-foot) and the total quantities are reducing each year. Total sediment load that entered the canal during 1974 was only 26,310 tons (29,000 U.S. tons) with a total volume of water of about 1,160,700,000 cubic meters (941,000 acre-feet).

Total operation and maintenance costs for the Gila Canal headworks, settling basin and diversion works for Calendar Year 1974 were \$59,900 of which \$40,700 was for operation. The cost per acre-foot was about \$0.06. Considering that this cost includes all operation and maintenance costs of the headworks and diversion works as well as the settling basin, the desilting cost is probably comparable with the cost of operating the All-American Canal desilting basins. Differences in the accounting procedures, however, do not allow direct comparison.

SEDIMENT REMOVAL FROM IMPERIAL RESERVOIR

As Imperial Reservoir filled with sediment, well-defined channels developed upstream that flowed directly to the two canal headworks. As the sediments continued to deposit in the reservoir and as flows reduced from the water conservation efforts in the last few years, large sand bars formed just above the dam and became stabilized with vegetative growth. Since the major flow has been to the All-American Canal, the main flow of the river shifted to the California side. After that the main supply to the Gila Canal headworks was across the face of the dam. Because of slower velocities as this portion of the water moved laterally, a large part of the sediment load to the Gila Canal headworks was deposited in the reservoir. This deposit immediately above the dam has restricted the flow to such an extent that on three occasions it has been necessary to place a dredge in the reservoir to open up the channel. Approximately 1,080,000 metric tons (1,190,000 U.S. tons) of sediment have been moved by dredging. During 1974 the cost of this dredging, utilizing a 10-inch cotterhead dredge, was \$0.58 per metric ton (\$0.63 per U.S. ton). This appears to be a continuing problem and studies are being conducted to determine if changes in channel location or configuration would lessen the problem.

CONTROL OF SEDIMENT IN THE RIVER BELOW IMPERIAL DAM

During the early years of operation of Imperial Dam, large flows of surplus water were used to carry the silt removed by the desilting works down the Colorado River. During the early 1950's sluicing flows up to 850 cubic meters per second (30,000 ft³/s) were used to clear the channel and transport the sediments. About 370,050,000 cubic meters (300,000 acre-feet) of water were used each year for that purpose. While this was effective in moving the sediment downstream, it created a serious sediment problem for Mexico. In 1950, Morelos Dam, Mexico's principal diversion structure located about 1.6 kilometers (one mile) downstream of the Northerly International Boundary, was completed. This structure diverted water into Mexico's Alamo Canal and much of the sediment passing down the river flowed into the canal. Frequent dredging was required to remove it. Because of

limited space, the dredge spoil was placed between the canal bank and the river channel. In time, much of the dredge material found its way into the river channel, which deteriorated badly since only minimum flows were available at this location to transport the sediment further downstream. These sediment deposits in the river channel were endangering the integrity of the levee system that protected the United States and Mexican lands during floods. In addition, Mexico was protesting that it was receiving a disproportionate share of the sediment load.

Therefore, discussions and planning were started on alternate means of handling sediment. Also, by this time it had become apparent that large sluicing flows would not be available in the future. The discussions were culminated in a Memorandum of Understanding, dated August 28, 1961, between the Bureau of Reclamation and the U.S. Section of the International Boundary and Water Commission (IBWC). The Bureau of Reclamation agreed to attempt to reduce the amount of sediment arriving at Morelos Dam to the same proportion as that contained in the flow at the first sediment measuring station upstream of Imperial Dam. This station is officially designated as "Below Cibola Valley", but is also known as "Adobe Ruin." If this reduction was found to be infeasible, the United States would pay Mexico the costs of disposing of the excess sediment. In return, Mexico agreed to dispose of the sediment removed from the Alamo Canal in such manner that it would not enter the river channel.

Various methods of collecting and disposing of the sediment were studied in detail. The most practical plan was the construction of a large settling basin between Imperial and Laguna Dams. This would allow the sediment accumulated at Imperial Dam to be sluiced into the settling basin, designated Laguna Settling Basin, where it could be removed by dredge. By utilizing the storage capacity of Laguna Reservoir, this sluicing operation could be accomplished without wasting water. The particular location for the settling basin has the advantage of adequate room for disposal of the sediment for many years.

To facilitate construction of Laguna Settling Basin, a 30 centimeters (12-inch) cotterhead dredge was purchased in 1963. It dredged a basin 137 meters (450 feet) wide, 914 meters (3,000 feet) long and 7.6 meters (25 feet) deep. Inlet and outlet channels were constructed from the end of the California Sluiceway Channel into the basin and from the basin to Laguna Reservoir. The inlet channel was designed for a gradient that produces a non-silting velocity so that the sediments are conveyed to the settling basin. All construction work was completed by 1965 and the basin was placed in operation.

In order to perform the sluicing operations at Imperial Dam without waste of water, it is necessary to drain water from Laguna Reservoir before the sluicing is begun. This reservoir, having a storage capacity of 1,973,600 cubic meters (1,600 acre-feet), is drawn down by releasing greater flows to the river below Laguna Dam. These flows are utilized as part of the scheduled Treaty flows to Mexico. Water is then released from Imperial Dam through the California Sluiceway Channel in sufficient quantity to flush accumulated sediment downstream to the settling basin. Usually the sluicing flows are 226 cubic meters per second (8,000 ft³/s) for a short duration or about one-quarter that amount for a longer period. The sediments settle out in the basin as the velocity slows and the sluicing water flows into Laguna Reservoir where it is stored for later release. Generally about 5 to 20 sluicing operations are required each month, the number depending on the sediment load and availability of water.

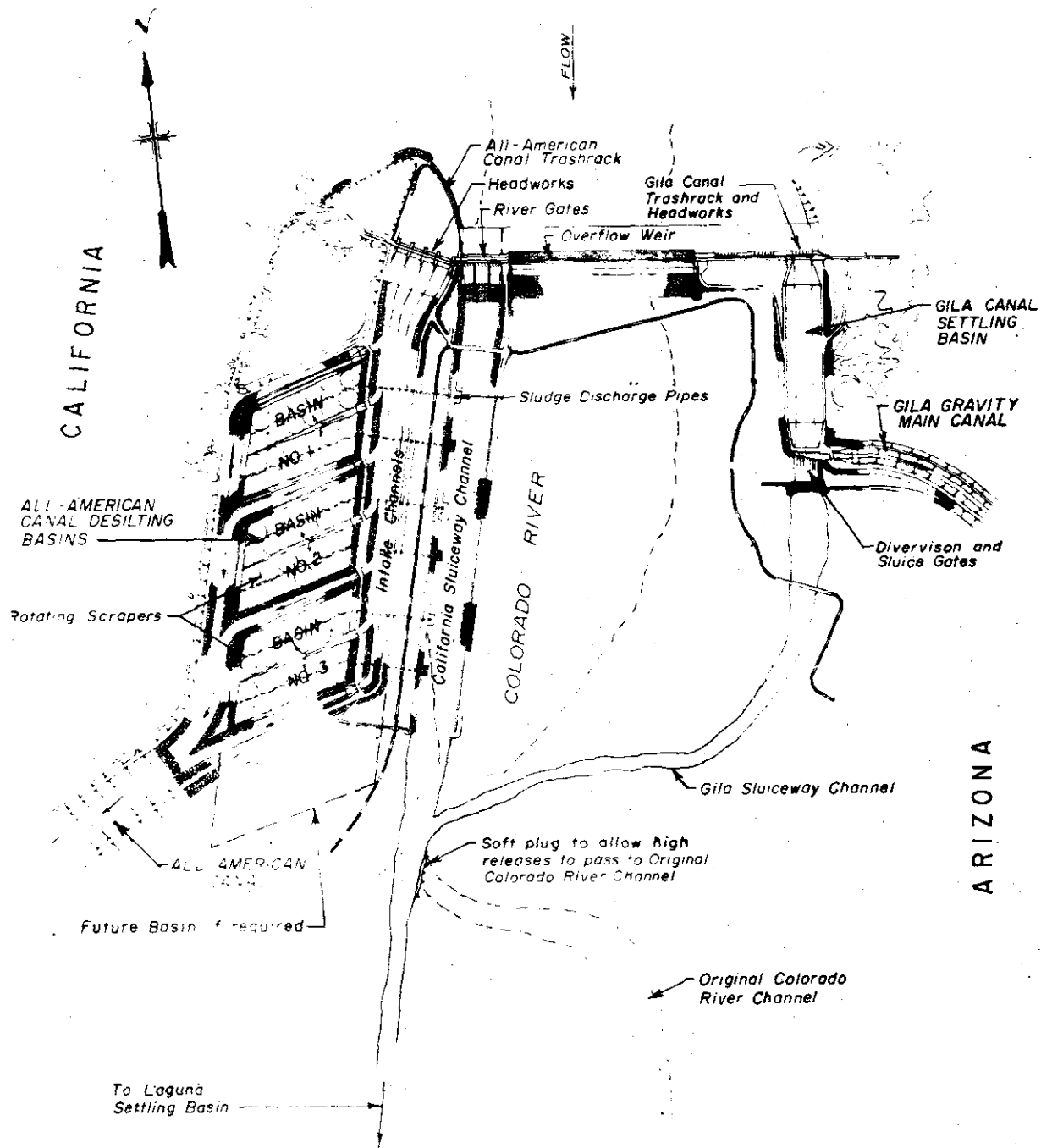
Since the basin was placed in operation nearly 6,305,000 metric tons (6,950,000 tons) of sediment have been dredged from it at an average cost during 1974 of \$0.36 per metric ton (\$0.40 per U.S. ton). To maintain the effectiveness or trap efficiency of the basin, it is necessary to remove the deposited material by dredge about every 3 years.

Operation of the settling basin has been very satisfactory with the amount of sediment arriving at Morelos Dam below that stipulated in the agreement with the U.S. Section, IBWC. It is noteworthy to mention that the Mexican Government has had little need to remove sediment from the Alamo Canal since Laguna Settling Basin was placed in operation.

Looking to the future, there is a need to complete the stabilization of the lower Colorado River to a satisfactory degree so that the sediment control measures at Imperial Dam will continue to be effective and to allow for the greatest multi-purpose utilization of the river.

REFERENCES

1. BORLAND, Whitney M., and MILLER, Carl R., "Sediment Problems on the Lower Colorado River." Proceedings paper 2452, Journal of the Hydraulics Division, ASCE, Vol. 86, No. HY-4, April 1960.
2. DOMINY, Floyd E., "Design of Desilting Works for Irrigation Systems," Proceeding paper 4498, Irrigation and Drain Div., ASCE, December, 1966, Vol. 92, No. IR-4.
3. MOSER, T. H. and SEARS, W. D., "Sediment Control Imperial Dam," Ninth Congress on Irrigation and Drainage, Moscow, 1975, International Commission on Irrigation and Drainage.
4. PEMBERTON, Ernest L., "Sediment Problems as They Relate to Irrigation Structure Operation and Maintenance," Engineering and Research Center, Bureau of Reclamation, Denver, Colorado, January 1972.
5. RECLAMATION, BUREAU OF, Boulder City, Nevada, "Annual Reports of Flows and Diversion at Imperial Dam and All Actions Affecting the Coordination Thereof," (prepared by Supervisor, Imperial Dam, Yuma Projects Office, Yuma, Arizona).
6. RECLAMATION, BUREAU OF, Denver, Colorado, "Imperial Dam and Desilting Works, Final Reports Part IV-Design and Construction, Bulletin 6, Boulder Canyon Project," 1949.
7. RECLAMATION, BUREAU OF, Boulder City, Nevada, "Report of Task Force, Review of the River Management Program, Colorado River Front Work and Levee System," July 1971.
8. RECLAMATION, BUREAU OF, Boulder City, Nevada, "Report on Comprehensive Plan, Colorado River Channelization, Palo Verde and Cibola Valley Divisions, Colorado River Front Work and Levee System," December 1959.
9. RECLAMATION, BUREAU OF, Boulder City, Nevada, "Reports on River Control Works and Investigations, Lower Colorado River Basin, Colorado River Front Work and Levee System."
10. RECLAMATION, BUREAU OF, Boulder City, Nevada, "Status Report, The Laguna Settling Basin, Colorado River Front Work and Levee System," February 1971.



PRINCIPAL FEATURES OF IMPERIAL DAM

DETERMINING EROSION FROM HAWAIIAN AGRICULTURAL LANDS ^{1/}

By Gary W. Frasier and John A. Replogle, Research Hydraulic Engineers, Keith R. Cooley, Hydrologist, U. S. Water Conservation Laboratory, 4331 East Broadway, Phoenix, Arizona; and S. A. El-Swaify, Soil Scientist, Department of Agronomy and Soil Science, University of Hawaii, Honolulu, Hawaii.

ABSTRACT

Techniques, instrumentation, and preliminary results of studies to determine the natural rainfall runoff and associated soil erosion from six small watersheds within fields of sugarcane and pineapple in Hawaii are presented. The initial results indicate that with current cultural practices, rainfall runoff and soil erosion are negligible from fields with a crop cover, but may be significant from roads within the cropped area, and during and after harvest.

INTRODUCTION

In Hawaii, erosion became a major concern when soil sediments were found deposited on the coral reefs surrounding the islands at the outfall of the many streams and on the oyster beds in Pearl Harbor. To prevent further sedimentation damage, the source and potential quantities of the sediments must be identified so that effective erosion control measures can be prescribed.

Methods such as the Universal Soil Loss Equation (Wischmeier and Smith, 1960) are used in much of the contiguous United States for estimating soil erosion and sediment movement. However, the elements necessary for such estimates have not been verified for the volcanic-derived soils, crops, or climatic conditions in the Hawaiian Islands. In 1971, the U.S. Water Conservation Laboratory, Agricultural Research Service, U.S. Department of Agriculture; the University of Hawaii; and the Soil Conservation Service, USDA; began joint studies to obtain data needed for quantitative evaluation of these elements. Of immediate interest were natural rainfall runoff and associated erosion losses from representative soils supporting the major crops in the State. Results of such studies would then be used as the basis for translating other independent estimates of erosion hazard (such as from rainfall simulation for erodibility predictions) to more realistic field situations. This paper describes the techniques, instrumentation, and preliminary results of rainfall-runoff and soil erosion studies on six small watersheds within pineapple and sugarcane fields in Hawaii.

^{1/} Joint contribution from the Agricultural Research Service, U.S. Department of Agriculture and the University of Hawaii Agricultural Experiment Station (Journal Series No. 1907).

DESCRIPTION OF WATERSHEDS

In 1971, five watersheds, three in sugarcane fields and two in pineapple fields, were instrumented to measure rainfall, runoff, and erosion. In 1974, a sixth watershed in a pineapple field was instrumented. The sugarcane sites, designated according to geographical location, are Laupahoehoe and Honokaa (on the Island of Hawaii), and Waialua Sugar (on the Island of Oahu). The pineapple sites, all on the Island of Oahu, are designated Waialua Pineapple, Mililani, and Kunia. Table 1 lists the climatic, soil, and crop settings originally associated with each of the areas.

The watersheds range from about 0.8 hectares (2 acres) for Waialua Pineapple to 2.8 hectares (7 acres) for Kunia. Average annual precipitation ranges from 760-1270 millimeters (30-50 inches) at Kunia to 3800-5100 millimeters (150-200 inches) at Laupahoehoe. The soils are generally fine textured and well structured, with a runoff potential classified as slow to medium and a corresponding permeability classified as moderate to rapid. Land slope varies within each watershed but is classified in the 5-15 percent range. Further details on the classification and characteristics of these soils are given in the Soil Survey Reports on Hawaii (Foote, Hill, Nakamura, and Stevens, 1972; Sato et al., 1973).

In mid-1972, the Mililani field was removed from pineapple production and remained idle until the spring of 1973. During this period with no cropping, the area became covered with a vigorous growth of weeds and small brush. In 1973, the field was cleared and planted to potatoes. Since then, the field has been alternately planted to various short-term crops and left uncultivated with heavy weed cover. In mid-1974 the Waialua Pineapple study area was converted to sugarcane, and in November 1974 the Kunia watershed was instrumented to provide continued runoff and erosion data from a pineapple culture.

INSTRUMENTATION AND PROCEDURES

Critical depth flumes with waterstage recorders were used to provide a continuous record of the runoff from the watersheds. Precipitation was measured with recording raingages located on or adjacent to each watershed. A proportionate sample of the runoff water was collected for sediment analysis with specially designed and calibrated sediment samplers at the outlet of each flume. The flumes were installed in June 1971, the waterstage recorders and raingages installed in February 1972, and the sediment-sampling equipment installed in December 1972.

Critical depth flumes.

The critical depth flumes were specifically designed and constructed as described by Replogle (1971, 1974). Flume heights were limited to 0.5 meter (1.5 feet) to minimize possible interference with the farm machinery and were designed to measure flow rates of 0.003 to 0.5 cubic

meters per second (0.1 to 20 cubic feet per second) representing flow depths of 0.03 to 0.5 meters (0.1 to 1.5 feet), respectively. The flumes, constructed from aluminum sheeting, were trapezoidal in cross section with side slopes of 2 to 1. Originally, the flumes had a throat section 0.91 meter (3 feet) long with a bottom width of 0.3 meter (1 foot); an entrance section 0.91 meter (3 feet) long with a bottom width of 0.91 meter (3 feet); and a transitional section 0.86 meter (2.83 feet) long connecting the entrance and throat sections.

Observations indicated that during some runoff events, sediments up to 7.5 centimeters (3 inches) deep were deposited in the flume entrance section. Analysis of the flow characteristics in the flume showed that the average water velocity through the entrance section was less than 30 centimeters per second (1 foot per second) for flow depths of less than 0.15 meter (0.5 foot). This velocity was insufficient to maintain the sediments in suspension through the entrance section.

The flumes were modified by installing an insert which changed the cross-sectional area and increased the flow velocity sufficiently to maintain acceptable sediment transport through the flume. The modified flume had an entrance section with a 0.3-meter (1-foot) bottom width and a "V"-shaped throat section. The entrance section was 0.03 meter (0.1 foot) above the throat elevation. These modifications increased the average flow velocity of water in the entrance section to 50 centimeters (1.6 feet) per second at flow depths of 0.15 meter (0.5 foot). To date, the modifications have reduced the sedimentation problem. To spread the flow for sediment sampling, a 0.9-meter (3-foot) long, 0.3-meter (1-foot) bottom width, trapezoidal extension was added to the exit end of the flume.

Sediment sampler.

Although several commercial samplers are marketed for sampling pollution in streams, most of these samplers are specialized equipment, requiring electricity for operation, and are generally poorly suited for aliquot sampling of both suspended and bedload materials (Anon., Federal Inter-agency Sedimentation Project, 1969; Wallings and Teed, 1971). A two-stage sampling system was designed and built that required no electrical power, could operate unattended for up to a week, handle peak flow rates of 0.5 m³ per second (20 cfs) from the flume, and reduce the total quantity of runoff containing both suspended and bedload materials to a representative aliquot of less than 75 liters (20 gallons). A rack of slotted flow splitters modified from the ones used by Barnes and Frevert (1954) were used for the primary sampler. The splitters, 1.2 meters (4 feet) long, were installed below the nappe from the flume on 12.7-centimeter (5-inch) intervals parallel to the flow direction, and sloping downstream at about a 10-degree angle to help clear trash. A 1/40th aliquot from the nine splitters on each flume was conveyed to a secondary sampler for further reduction. The secondary sampler operated on the principle of the Coshocton wheel sampler (Parsons, 1954, 1955; Replogle, 1963; Wang,

1971). A theoretical analysis and laboratory tests showed that the sampler could be redesigned to overcome deficiencies of rotational variations and splashout of portions on the sample to assure an accurate aliquot of water, suspended sediment, and bedload material. Power for this sampler was derived from small vanes on the rotating wheel.

The entire sediment-sampling system described collects approximately one part in 3750 of the total runoff. This water was stored in a 120-liter (32-gallon) plastic container which was collected at weekly intervals for sediment analysis.

Data recorders.

Eighteen months (November 1974) after original instrumentation, the data collection system was changed to record both the waterstage and raingage data on a single 30-day strip chart (minimum time division = 5 minutes). Tipping bucket raingages accurate to ± 0.25 millimeter (0.01 inches) were used to measure precipitation. The waterstage readings were recorded by a continuous drag pen accurate to ± 1.0 millimeter. The advantage of having both instruments record on the same chart was partially offset by the necessity of moving some of the raingages close to the recorders, resulting in a minimum fetch for optimum performance on some storms.

For data processing, the portions of the recorder charts that indicated rainfall had occurred were date-time noted and the data transferred to a computer-compatible tape by the use of an analog-to-digital chart reader. Possible operator error of $\pm 2\%$ can occur during the chart reading but this is within the accuracy of the flumes at low flows.

Originally the data were collected on recorders which digitized and punched separate paper tapes at 5-minute intervals. After about 50,000 cycles, representing a period of 6 months, these recorders became worn and failed to operate properly. During processing of this data, several other problems were encountered that proved more difficult than anticipated in the equipment selection. One was the matching of rainfall and runoff to a common time scale. When starting the two recorders separately, each with a punch interval of 5 minutes, the time difference between the two recorders could be as much as 10 minutes. This made it virtually impossible to match the true time relation between rainfall and runoff events. Also, problems of the recorders overpunching caused difficulty in matching the data because there was no way of keeping track of the elapsed time on the data tapes. The second problem not fully realized initially was the impact of processing all data tapes, even if there were only small quantities of rainfall and no runoff.

Sediment analysis.

A representative 6-liter portion of storm runoff and sediment collected in the 120-liter (32-gallon) containers was transferred to the University of Hawaii's Department of Agronomy and Soil Science for analysis. Total sediment content was determined for all storms and particle-size distributions were determined for storms where sample size was sufficient. For high sediment concentrations ($> 1\text{--}2\text{ g/liter}$), the amount of sediment was determined gravimetrically after flocculation by a few drops of 1 N HCl, decanted of supernatant liquid and dried at 105°C . For low sediment concentrations, turbidimetric techniques utilizing standard curves of turbidity vs. sediment concentration were used.

PRELIMINARY RESULTS

Sediment losses.

Table 2 shows representative data on sediment losses from three Oahu watersheds. Further analysis of these data will be required (particularly for detailed storm events) to determine the contribution of each soil loss factor and eventually assess the inherent susceptibility of each of the watershed soils to erosion. Tentatively, however, it appears that the contrasting cultural and cropping factors to which one (Wahiawa) soil (Waialua Pineapple and Mililani sites) was subjected affected the runoff and soil losses. The site at Mililani with occasional heavy weed growth and variable cropping (and slightly less slope gradients) was less subject to erosion under similar storms than was the site at Waialua Pineapple, which was originally under pineapple then changed to sugarcane. Rainfall runoff was correspondingly affected. The Ultisol (Paaloa soil on Waialua Sugar) was less susceptible to erosion than the Wahiawa soil (Dangler et al., 1975). All soil losses monitored during 1974 were very low and represented little environmental hazard. Furthermore, visual inspection indicated that erosion from the pineapple fields was primarily from the roads around the subfields. That a less-than-full canopy of sugarcane can effectively reduce soil losses was clearly demonstrated at the Laupahoehoe area. The sugarcane on that field was about 2 feet high when the flume was installed. During the next 24 months, the water received in the sediment collection tank as well as water in the stilling well remained clear even though there were several runoff events.

Runoff results.

Rainfall-runoff data from the watersheds have been tabulated from February 1972 through March 1975. More detailed data analysis will be performed after each field goes through one or more cropping cycles with complete instrumentation.

Preliminary analysis of the runoff data indicates that runoff from the watersheds is negligible once the crop is tall enough to provide complete

cover. The standard cultivation practice of placing large borders around each pineapple subfield also tends to assist in holding the water within the planting area of the crop. There is an interval during and after harvest when significant runoff can occur. This is illustrated in some preliminary data shown in Figure 1 and Figure 2 for the Laupahoehoe study area. Figure 1 shows the 15-minute rainfall intensity and resulting runoff when the crop was at full maturity on 19 November 1973. A total of 183 millimeters (7.2 inches) of rain fell during an 18-hour period. Total runoff was 7 millimeters (0.26 inch) or about 4% of the precipitation. The sugarcane was harvested on 20 January 1974, leaving the field bare. On 23 January 1974, 104 millimeters (4.1 inches) of rain produced 98 millimeters (3.8 inches) of runoff with a substantially increased rate of runoff (Figure 2).

SUMMARY AND CONCLUSIONS

Rainfall-runoff and associated soil erosion were measured on some agricultural lands in Hawaii. Small watersheds within existing fields of sugarcane and pineapple were instrumented with proportional sediment samplers, critical depth flumes, and stripchart waterstage raingage recorders. Preliminary results indicate that standard cultural practices may affect the quantity of runoff and soil erosion. While the crop is growing, runoff and erosion are minimal, and come primarily from the roads within the fields. Runoff and erosion may be significant during and after harvest periods when the fields are relatively bare.

REFERENCES

- Anon., 1969, A Study of Methods Used in Measurement and Analysis of Sediment Loads in Streams, Federal Interagency Sedimentation Project, Cooperative Project sponsored by the Subcommittee on Sedimentation, Water Resources Council, St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minn., revised June 1970, 57 pp.
- Barnes, Kenneth K., and Frevert, R. K., 1954, A Runoff Sampler for Large Watersheds, Part I Laboratory Studies, *Agr. Eng.*, 35(2), 88-90.
- Dangler, E. W., El-Swaify, S. A., Ahuja, L. R., and Barnett, A. P., 1975, Erodibilities and Related Infiltration Characteristics of Selected Hawaii Soils by Rainfall Simulation, ARS-W-Series, USDA. In press.
- Foote, D. E., Hill, E. L., Nakamura, S., and Stevens, S., 1972, Soil Survey of Islands of Kauai, Oahu, Maui, Molokai, and Lanai, State of Hawaii. USDA, Soil Conservation Service, in cooperation with University of Hawaii, 130 pp.
- Parsons, D. A., 1955, Coshocton-Type Runoff Samplers, Supplementary Laboratory Investigations, USDA, SCS-TP-124, Washington, D.C., 16 pp.
- Parsons, D. A., 1955, Coshocton-Type Runoff Samplers, Supplementary Laboratory Investigation at the Stillwater, Oklahoma, Outdoor Hydraulic Laboratory, USDA, ARS-41-2, Washington, D.C., 11 pp.
- Replogle, John A., 1971, Critical-Depth Flumes for Determining Flow in Canals and Natural Channels, *Trans. ASAE*, 14(3), 428-433, 436.

- Replogle, John A., 1974, Tailoring Critical-Depth Measuring Flumes, in Flow--Its Measurement and Control in Science and Industry, Proc. Instrument Soc. Amer. Symposium, Pittsburgh, Pa., May 1971, pp. 123-132.
- Replogle, John A., and Johnson, L. C., 1963, Improved Measuring and Sampling Equipment for Sediment-Loaded Runoff, Trans. ASAE, 6(3), 259-261.
- Sato, H., Ikeda, W., Paeth, R., Smyth, R., and others, 1973, Soil Survey of the Island of Hawaii, State of Hawaii, USDA, Soil Conservation Service, in cooperation with University of Hawaii, 115 pp.
- Walling, D. E., and Teed, A., 1971, A Simple Pumping Sampler for Research into Suspended Sediment Transport in Small Catchments, J. Hydrology, 13, 325-337.
- Wang, J. Y., Hermanson, R. E., and Hendrick, J. G., 1971, Modification of H-Flume and Coshocton Wheel System, Trans. ASAE, 14(6), 1015-1017.
- Wischmeier, W. H., and Smith, D. D., 1960, A Universal Soil-Loss Equation to Guide Conservation Farm Planning, 7th International Congress of Soil Science, Madison, Wisconsin.

Table 1 Description of study watersheds

Characteristic	Watershed					
	Laupahoehoe	Honokaa	Waialua Pineapple	Waialua Sugar	Mililani	Kunia
Island	Hawaii	Hawaii	Oahu	Oahu	Oahu	Oahu
Crop	Sugarcane	Sugarcane	Pineapple	Sugarcane	Pineapple	Pineapple
Size, in hectares	0.86	2.16	0.80	2.51	2.09	2.84
Average annual precipitation, in millimeters	3800-5100	1800-2500	1000-1500	1800-2300	1000-1500	750-1300
Soil series	Kaiwiki	Kukaiiau	Wahiawa	Paaloa	Wahiawa	Kawaihapai
Field texture	Silty clay loam	Silty clay loam	Silty clay	Silty clay	Silty clay	Clay loam
Soil Order	Inceptisols	Inceptisols	Oxisols	Ultisols	Oxisols	Mollisols
General soil slope, in percent	10-20	12-30	3-8	3-12	3-8	2-6
Erosion hazard	Slight	Moderate	Slight	Slight to moderate	Slight	Slight
Runoff potential	Slow	Medium	Slow	Slow to medium	Slow	Slow
Permeability	Rapid	--	Moderately rapid	Moderately rapid	Moderately rapid	Moderate

Table 2 Representative data on sediment losses for selected storms during 1974

Watershed	Sampling date, Julian		Storm (sampling) totals				
	Start	Stop	Rain (mm)	Runoff (mm) %		Sediment g/l kg/hectare	
Mililani	22	32	117	6	5	0.48	27
	72	87	86	1	1	0.06	2
	106	113	137	1	1	0.16	2
	148	161	69	> 1	> 1	0.51	4
	190	203	64	> 1	> 1	0.24	2
Waialua Pineapple	8	22	74	17	23	2.08	350
	133	148	76	7	9	6.89	478
	190	203	122	11	9	1.30	137
Waialua Sugar	133	141	71	> 1	> 1	1.28	9
	161	176	43	> 1	> 1	3.42	4
	190	203	173	2	1	0.44	7

Note: 1 inch = 25.4 millimeters

1 kg/hectare = 4.45×10^{-4} tons/acre

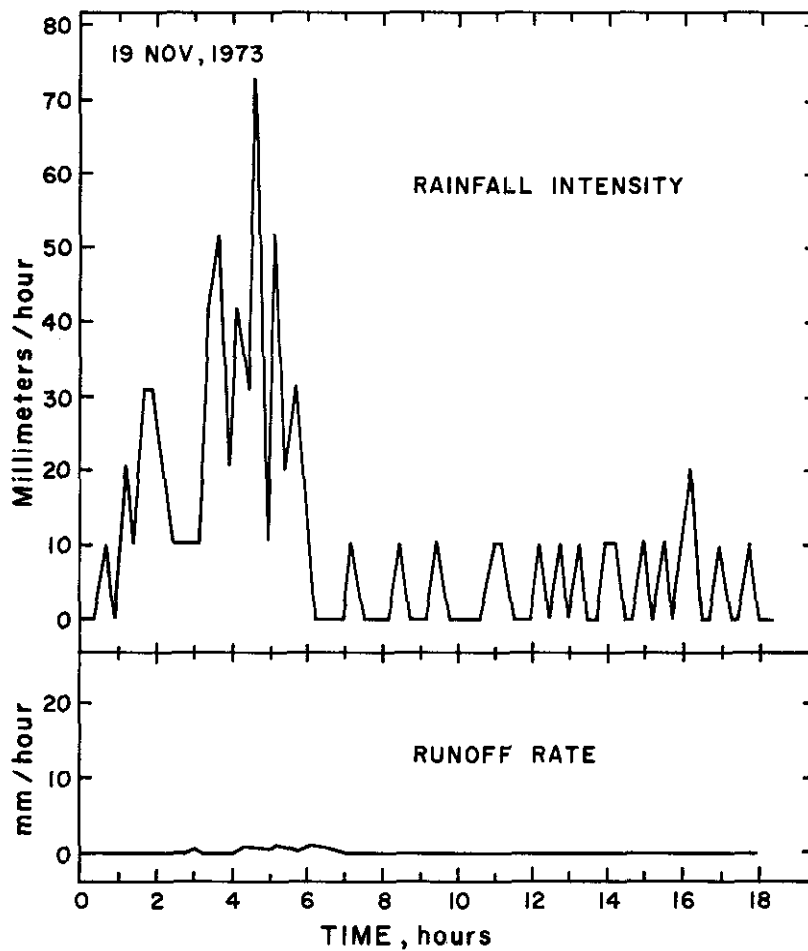


Figure 1. Runoff rate and rainfall intensity for 15-minute time interval from Laupahoehoe watershed before sugarcane harvest.

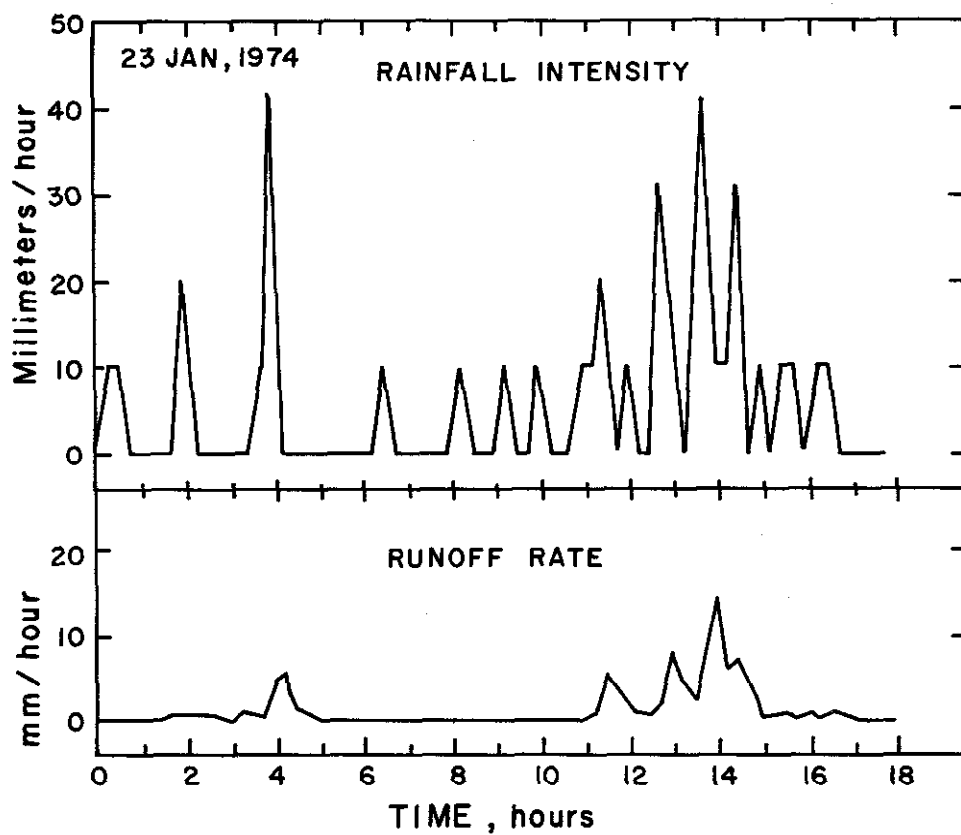


Figure 2. Runoff rate and rainfall intensity for 15-minute intervals from Laupahoehoe watershed after sugarcane harvest.

THE UNIVERSAL SOIL LOSS EQUATION AS ADAPTED TO THE PACIFIC NORTHWEST

By D. K. McCool, Agricultural Engineer, and R. I. Papendick, Soil Scientist, Agricultural Research Service, Washington State University, Pullman, Washington; and F. L. Brooks, Agronomist, Soil Conservation Service, Portland, Oregon.

ABSTRACT

Use of the Universal Soil Loss Equation for soil loss prediction in the Pacific Northwest has been limited by inapplicable relationships for the factors of the equation. A first-generation adaptation of the equation was made by developing new relationships to fit Pacific Northwest conditions. These included (a) a slope length-steepness relationship to account for steeper slopes (> 20 percent) and runoff-induced erosion; (b) a runoff-rainfall erosion factor that accounts for effects on soil loss of climatic phenomena peculiar to the region, i.e., low intensity rainfall, snowmelt, and rain or snowmelt on frozen ground; and (c) first-approximation crop management factors to fit the region. The Soil Conservation Service is field testing the adaptation.

INTRODUCTION

Soil erosion from water is a critical problem in the Pacific Northwest dry-land grain region, a unique physiographic area encompassing more than 8 million acres of cropland in eastern Washington, northeastern Oregon, and northwestern Idaho (Fig. 1) (Horner, Oveson, Baker, and Pawson, 1960). Estimated annual soil loss in eastern Washington alone is in excess of 40 million tons (Kaiser, 1972). The most severe erosion occurs in the Palouse area of eastern Washington and adjacent Idaho where average erosion rates on croplands range from 2 to 25 T/A; 15 to 40 percent is delivered off-farm and the highest average soil loss rate is associated with the highest sediment delivery ratio. Runoff and erosion rates are highly variable, depending on precipitation characteristics, temperature, topography, soil characteristics, and crop management practices. On-site soil losses sometimes exceed 100 T/A in critical areas with inadequate protection. Annual runoff varies from 0.5 to 4.5 inches. For the area with the 40 percent delivery ratio, the estimated annual runoff is 1.5 inches. This yields an average off-farm concentration of 59,000 ppm total sediment load in the drainage waters.

A critical need for the Pacific Northwest is a predictive method for assessing the effects of climate, topography, soils, crop management practices, and erosion control practices on soil loss. This would be a powerful tool for planning crop management systems and supporting practices urgently needed to control erosion on croplands for resource conservation and environmental protection. Past attempts at erosion prediction in the Pacific Northwest have met with little or no success. Most efforts have been with the Universal Soil Loss Equation (USLE), using relationships developed from climatic and topographic conditions east of the Rocky Mountains. Applying the USLE with these relationships to the Pacific Northwest gives unrealistic soil loss estimates because these relationships are invalid for the Pacific Northwest.

Revised relationships are needed to adjust for differences in these factors between the two regions. This paper reports a first-generation adaptation of the USLE to the Pacific Northwest.

DESCRIPTION OF THE PACIFIC NORTHWEST DRYLAND GRAIN REGION

The Pacific Northwest differs considerably from other parts of the nation in both climate and topography. The climate is Mediterranean-like with humid winters and dry summers in contrast to the continental-type climate prevalent elsewhere. Annual precipitation ranges from 8 to 25 inches in different parts of the region (Horner, Starr, and Patterson, 1957). Precipitation timing and other climatic factors make the region ideally suited for winter wheat production. Winter wheat/summer fallow rotation is necessary in the lower rainfall areas, but winter wheat recrop or rotations involving spring crops are entirely feasible in the higher rainfall areas. Use of the winter wheat/summer fallow rotation with excessive tillage and low surface residue rates in the intermediate to higher precipitation areas creates a serious erosion problem. Approximately two-thirds of the precipitation occurs during November through March when ground cover is minimal. Fortunately, most precipitation falls at low intensity. The 10-year return interval, 24-hour duration precipitation value ranges from approximately 1.5 to 2.5 inches (Meteorology Committee, 1969). Rainfall intensities of most winter storms are less than 0.15 inch/hr (Horner et al., 1957). The percentage received as snow varies widely, depending mainly on elevation but generally increasing in a north-easterly direction across the region. Although occasional high intensity summer storms produce severe local erosion, the total soil loss on a regional basis is inconsequential compared with that during the winter period. Annual precipitation and seasonal distribution are highly variable from year to year.

Frozen soil contributes to erosion in much of the area by limiting water infiltration. Alternate freeze-thaw periods are common and during extended cold periods frost may penetrate to 3 feet (Phillips, 1965, 1970) but more commonly to depths of a few inches to 1 foot. Frequently, these extended cold periods are followed by snow changing to rain with subsequent high runoff and soil loss.

As with climate, the landscape features of the region greatly accentuate the erosion problem and add difficulty to erosion prediction. Slopes in some areas are as much as 2,500 to 3,000 ft long, with steepness from 15 to 40 percent. In other areas, lengths are shorter, 100 to 250 ft, but steepness may reach 55 percent (Kaiser, Starr, and Johnson, 1961). The steepest areas occur mainly on north and northeast slopes, and most are cropped using conventional methods applied to other parts of the field. These steep areas are susceptible to large soil slips as well as severe rill erosion because of excessive soil moisture from melting of large snow drifts deposited by prevailing westerly or southwesterly winds.

Conventional erosion-control practices used east of the Rocky Mountains have met with little success in the region. Much of the area is too steep to terrace effectively--the close vertical spacing is incompatible with 30- to 60-ft tillage equipment widths, the terraces frequently drift full of snow and overflow during runoff, and grassed outlets are difficult to maintain

because of the steep grade and sustained low flows. Much of the area is too irregular for conventional contour strips. The substitute most acceptable to farmers has been the divided slope system, with the slopes divided into upper and lower portions but with tillage only approximately on the contour. Improved conservation tillage techniques offer the best opportunity for reducing the enormous soil losses sustained with the present farming techniques and for meeting water quality requirements of the Federal Water Pollution Control Act Amendments of 1972 (PL 92-500).

THE UNIVERSAL SOIL LOSS EQUATION

The Universal Soil Loss Equation (USLE) resulted from analysis of the data from several erosion research stations established in the United States in the early 1930s. Sufficient data had been collected by 1940 that attempts were made to determine the effect of slope length and steepness on erosion (Zingg, 1940). In 1954, a data analysis center was established to evaluate data from these stations and to develop a generalized relationship for the various factors influencing soil loss. The result was the USLE (Wischmeier and Smith, 1965):

$$A = RKLSCP \quad [1]$$

where A = the computed soil loss per unit area

R = the rainfall factor

K = the soil-erodibility factor

L = the slope-length factor

S = the slope-gradient factor

C = the crop-management factor

P = the erosion-control practice factor

Relationships and numerical values for each of the six factors, which differ from one field or location to another, have been determined from research data from the area east of the Rocky Mountains. The USLE has since been widely used by the Soil Conservation Service and other agencies in planning crop-management and erosion-control practices in that portion of the United States. Data from the Palouse Conservation Field Station near Pullman, WA, the only erosion station west of the Rocky Mountains, were not used in the development of the USLE because they did not fit the pattern of the data from the rest of the country.

ADAPTING THE USLE TO THE PACIFIC NORTHWEST

This project marked the first serious effort to adapt the USLE for use in the Pacific Northwest. The east-of-the-Rocky Mountain R factor consists of the number of rainfall erosion index (EI) units, defined as the product of total kinetic energy of rain and maximum 30-minute intensity, in a normal year's rain. The measured rainfall erosion-index of 9 at Pullman, WA, seemed unrealistically low, considering the large soil losses. Thus, available soil loss data were used in developing a new relationship to evaluate climatic influences on soil loss.

Relationships between the soil-erodibility factor, K, and soil mechanical properties, organic matter content, structure, and permeability as determined

under conditions east of the Rocky Mountains were assumed applicable to the Pacific Northwest. The validity of this assumption will be examined in future studies.

The slope-length factor, L , is generally expressed as $(\lambda/72.6)^m$ where λ is the field slope length in feet and m is a constant of variation (Zingg, 1940). The exponent m is determined from field data and has been found to generally increase with slope steepness. There is meager data from slopes steeper than 20 percent on which to base exponents for the steep slopes typical of the Pacific Northwest. Moreover, the type of erosional agent, predominantly runoff as opposed to combined rainfall, splash, and runoff, may affect the relationship between slope length and soil loss. A new relationship was developed for the Pacific Northwest.

Extrapolation of the relationship developed by Wischmeier and Smith (1965) for slope-steepness factor, S ,

$$S = (0.43 + 0.30s + 0.043s^2)/6.613 \quad [2]$$

(s = field-slope gradient in percent) to the 40 percent slopes commonly cultivated gave soil loss values that, in comparison to those from lesser slope steepnesses, seemed unrealistically high. Therefore, available soil loss data were analyzed and new relationships proposed.

Crop management factors were also of concern, because C factors used in the small grain areas of the Great Plains do not take into account the large differences in antecedent soil moisture (which influences infiltration rate) at fall-planting between continuous cropping and wheat after fallow in the Pacific Northwest. The former practice is frequently used in the higher precipitation areas and in those areas is effective in reducing erosion as compared with the 2-year winter wheat/summer fallow rotation.

Erosion-control practice factors from the Midwest, determined from study of fields of low slopes, approach unity for the steep slopes commonly cultivated in the Pacific Northwest. Additionally, no P -factor reduction is currently given to chiseled stubble or mulch strips which alternate with seeded grain in 2-year rotations in the Pacific Northwest and are believed to reduce soil loss by increasing infiltration and hence reducing runoff volume.

The first generation relationships for the adaptation of the USLE to the Pacific Northwest consisted of determining new slope length and steepness relationships and altering the R factor to account for both rainfall and snowmelt runoff. The basic assumptions and form of the USLE were not altered. Field data and observations of soil loss from rills, collected from erosion seasons 1958-59 through 1971-72 by Verle Kaiser of the Soil Conservation Service (Kaiser, 1971), provided the data for most of the modifications. Fields that were observed year after year were the most useful. Data from the Palouse Conservation Field Station (Horner, McCall, and Bell, 1944) were also utilized in the alterations. Details of the alterations are reported elsewhere (McCool, Wischmeier, and Johnson, 1974). It was necessary to determine the new slope length and steepness relationships before the R factor could be calculated from the field soil loss data. These alterations will be discussed in the order of development rather than the order of

appearance in the USLE. A first approximation of C factors for some of the more common rotations and management practices was also attempted.

Length and Steepness Factors

Length and steepness effects on soil loss are commonly combined into the product of the L and S factors. This method was most convenient for examining the data from the Pacific Northwest. Data from four of the fields Kaiser observed were used for modifying the length and steepness relationships. The fields were in winter wheat after summer fallow in 1967-68, 1969-70, and 1971-72. Field notes gave soil loss per acre from rilling alone from specific slopes on Class IIIe (7 to 25 percent slope), IVe (25 to 40 percent slope), and VIe (40 to 55 percent slope) land. The data were adjusted for sheet erosion losses by use of data from plots at the Palouse Conservation Field Station, C factors were assigned, and all losses were adjusted to base plot conditions. The data were subjected to regression analysis and the following relation was obtained:

$$LS = (\lambda/72.6)^{0.08}(s/9)^{0.89} \quad [3]$$

where LS = product of slope length and steepness factors

λ = field slope length, in feet

s = field slope gradient, in percent

The relationship gave lower soil loss values for the steeper slopes than that determined by extrapolation from the east-of-the-Rocky Mountain relationships. Because the Kaiser data were somewhat subjective, a relationship only slightly less severe than that for the eastern United States was proposed for the steeper slopes

$$LS = (\lambda/72.6)^{0.3}(s/9)^{1.3} \quad [4]$$

For slopes less than 9 percent, the following relationship was suggested:

$$LS = (\lambda/72.6)^{0.3}(0.43 + 0.30s + 0.043s^2)/6.613 \quad [5]$$

A plotting of the LS relationships of equations [4] and [5] is shown in Fig. 2.

Rainfall and Runoff Erosion Factor

The term to quantify the climatic forces causing water erosion might be termed a rainfall and runoff erosion factor. It would consist of an equivalent winter erosion-index factor including the combined effects of both splash and runoff detachment and movement from rain on thawing ground, rain on snow, and rain on unfrozen ground without snow cover, to be added to the measured erosion index (EI) for spring, summer, and fall.

$$R_T = R + R_s \quad [6]$$

where R_T = rainfall and runoff erosion factor

R = measured erosion index (EI) for spring, summer, and fall

R_s = equivalent erosion index for winter months

The value of R_S for a specific area of eastern Washington was determined by the use of Kaiser's observations of soil loss from seven fields for 1958-59 through 1971-72. By using the tentative LS relationships, equations [4] and [5], a mean R_S of 27 was determined. This R_S value represents the equivalent erosion index for the months of October through March for the seven fields that centered on an area slightly west of Pullman, WA.

The measured EI for the spring through late fall months of April through September for Pullman is approximately 5; so a benchmark rainfall and runoff erosion index value, R_T , for the Pullman vicinity is approximately 32.

The Pullman soil loss data from the hard, bare fallow plot (Horner et al., 1944) indicated a strong correlation between soil loss and the December through March precipitation. In other parts of the United States, including some areas west of the Rocky Mountains where snowmelt is not a serious erosion hazard, it has been determined that the EI or R is closely related to the 2-year return interval, 6-hour duration precipitation by the relationship (Wischmeier, 1974):

$$EI = 27 P^{2.2} \quad [7]$$

where EI = erosion index

P = 2-year return interval, 6-hour duration precipitation in inches.

For the dryland, grain-growing region of the Pacific Northwest, a factor determined from correlation of precipitation to soil loss is added to the right-hand side of equation [7]

$$R_T = 27 P^{2.2} + 1.5 P_{(D-M)} \quad [8]$$

where R_T = rainfall and runoff erosion factor

P_T = 2-year return interval, 6-hour duration precipitation in inches

$P_{(D-M)}$ = December through March precipitation in inches.

The coefficient of 1.5 applied to the December through March precipitation was chosen to give an R_T value approximately the same as the benchmark value for the Pullman vicinity: $R_T = 20 + (1.5 \times 10.4) = 36$. The components of equation [8] do not correspond individually to R and R_S in equation [6] but merely total R_T . This relationship leads to a geographic rainfall and runoff erosion factor distribution for the dryland, grain-growing region of the Pacific Northwest as shown in Fig. 3.

The distribution of the rainfall and runoff erosion factor throughout the year is necessary for determining average crop-management factors for a given crop-management system. The total actual rainfall erosion index value of 9 for Pullman was distributed the same as the average annual distribution for Spokane, Pullman, and Pendleton (Wischmeier, personal communication, March 21, 1974). The Pullman soil loss data from the hard, bare fallow plot showed much more severe erosion from December through March (Horner et al., 1944), so the actual rainfall erosion index was used for April through November. The R_S was decreased by the October and November erosion index, and the remainder was distributed evenly through the months of December through March. The results are plotted in Fig. 4. Approximately 83 percent of the total rainfall

and runoff erosion factor, R_T , occurs from October through March, the approximate time period after winter wheat is planted and normally offers little ground cover until spring growth begins.

Crop-Management Factor

First-approximation crop management factors for three typical rotations under both conventional and conservation tillage were determined by starting from assumed residue levels following harvest and using residue disappearance coefficients for each tillage operation (USDA, 1969). The amount of surface residue at a particular crop stage was used with mulch effect information (Wischmeier, 1973) and an estimate of clod size to determine the C factor for a particular period. The C factors for the period after planting winter wheat and for the mature crop and post-harvest stage were determined from Agricultural Handbook 282 (Wischmeier and Smith, 1965). The resulting C factors for the annual winter wheat and winter wheat/pea rotations seemed unrealistically high when compared to the C factors for the winter wheat/summer fallow rotation. Hence, modifications were made to account for the differences in soil moisture prior to planting winter wheat following different crops using a correction factor that ranged from 0.5 to 1.0. The validity of this assumption is currently being investigated. The resulting tentative C factors for three of the prevalent crop rotations are given in Table 1, along with details of the tillage operations assumed for each management level.

APPLICATION OF THE USLE ADAPTATION

Use of the USLE adaptation is best illustrated by an example. Assume a field near Pullman, WA in the 18 to 21-inch precipitation zone with a 150-ft slope length and a 12 percent slope. The soil is a Palouse silt loam ($K = 0.32$). Assume a tolerable soil loss, T , of 5 T/A. We wish to find the required C factor and possible crop rotation and management if no supporting conservation practice is applied ($P = 1$).

From Fig. 3, the R_T for Pullman vicinity is 35. For a 150-ft long, 12 percent slope, the LS factor is 1.8 (from Fig. 2 or equation [4]), and the required C factor is

$$C = T/RKLSP = 5/(35 \times 0.32 \times 1.8 \times 1.0) = 0.25 \quad [9]$$

This C factor could be met by conservation tillage and either annual winter wheat or a winter wheat/pea rotation (see Table 1).

If the slope steepness was increased to 25 percent, the LS factor would be 4.7, and the required C factor would be 0.12. These conditions would require considerably more stringent management than for the less steep area. None of the sample rotations and managements given in Table 1 could be used. A rotation including a grass and legume seeding or very rough tillage would be necessary to maintain soil loss within the tolerable limit.

REFERENCES

- Horner, G. M., McCall, A. G., and Bell, F. G., 1944, Investigations in Erosion Control and the Reclamation of Eroded Land at the Palouse Conservation Experiment Station, Pullman, Washington, 1931-42, USDA, SCS, Technical Culletin 860.
- Horner, G. M., Oveson, M. M., Baker, G. O., and Pawson, W. W., 1960, Effect of Cropping Practices on Yield, Soil Organic Matter and Erosion in the Pacific Northwest Wheat Region, Agr. Expt. Stations of Idaho, Oregon, and Washington, and the Agricultural Research Service, USDA, Bulletin 1.
- Horner, G. M., Starr, W. A., and Patterson, J. K., 1957, The Pacific Northwest Wheat Region, Yearbook of Agriculture, pp. 475-481.
- Kaiser, Verle G., 1971 mimeo handout updated by telephone conversation.
- Kaiser, Verle G., 1972, Major Problem Areas for Soil and Water Conservation in the Dryfarmed Grainlands of the Columbia-Snake-Palouse Area of the Pacific Northwest, USDA, SCS, State of Washington.
- Kaiser, Verle G., Starr, W. A., Johnson, S. Burkett, 1961, Types of Topography as Related to Land Use in Whitman County, Washington, Northwest Science, 25, 69-75.
- Meteorology Committee, Pacific Northwest River Basins Committee, 1969, Climatological Handbook, Columbia Basin States, Precipitation, Vol. 2.
- McCool, D. K., Wischmeier, W. H., and Johnson, L. C., 1974, Adapting the Universal Soil Loss Equation to the Pacific Northwest, ASAE 74-2523, Presented at the Winter Meeting of ASAE, Chicago, IL., December 10-13.
- Phillips, E. L., 1965, Washington Climate for Adams, Lincoln, Spokane, and Whitman Counties, Agr. Ext. Service, Washington State University, Bulletin No. EM-2545.
- Phillips, E. L., 1970, Washington Climate for Asotin, Benton, Columbia, Franklin, Garfield, and Walla Walla Counties, Agr. Ext. Service, Washington State University, Bulletin No. EM-3127.
- U.S. Department of Agriculture, Soil Conservation Service, 1969, Residue Management in Stubble Mulch Farming, Unnumbered Technical Note, Ref. No. M-2430-227.
- Wischmeier, W. H., 1973, Conservation Tillage to Control Water Erosion, Proceedings of the National Conservation Tillage Conference, Des Moines, IA., March 28-30, pp. 133-141.
- Wischmeier, W. H., 1974, New Developments in Estimating Water Erosion, Proceedings of the 29th Annual Meeting of the Soil Conservation Society of America (Land Use: Persuasion or Regulation), August 11-14, Syracuse, NY, pp. 179-186.
- Wischmeier, W. H., and Smith, D. D., 1965, Predicting Rainfall-Erosion Losses from Cropland East of the Rocky Mountains, USDA, ARS, Agricultural Handbook No. 282.
- Zingg, A. W., 1940, Degree and Length of Land Slope as it Affects Soil Loss in Runoff, Agricultural Engineering, 21, 59-64.

Table 1. Tentative Crop Management Factors for Three Prevalent Rotations, Dryland Grain Region of the Pacific Northwest.

Crop Rotation	C Factor	
	Conventional Tillage ¹	Conservation Tillage ²
Winter wheat-summer fallow ³	0.40	0.30
Annual winter wheat ⁴	.27	.14
Winter wheat-pea rotation ⁵	.30	.20

¹Conventional tillage includes use of a moldboard plow as the primary tillage implement followed by several operations that reduce clod size.

²Conservation tillage includes use of a deep chisel or other implement that leaves a high percentage of residues on the surface followed by a minimum number of secondary operations.

³Medium yield with medium growth before winter.

⁴Medium yield with late seeding or late emergence before winter.

⁵Medium yield with late seeding or late emergence of wheat before winter.

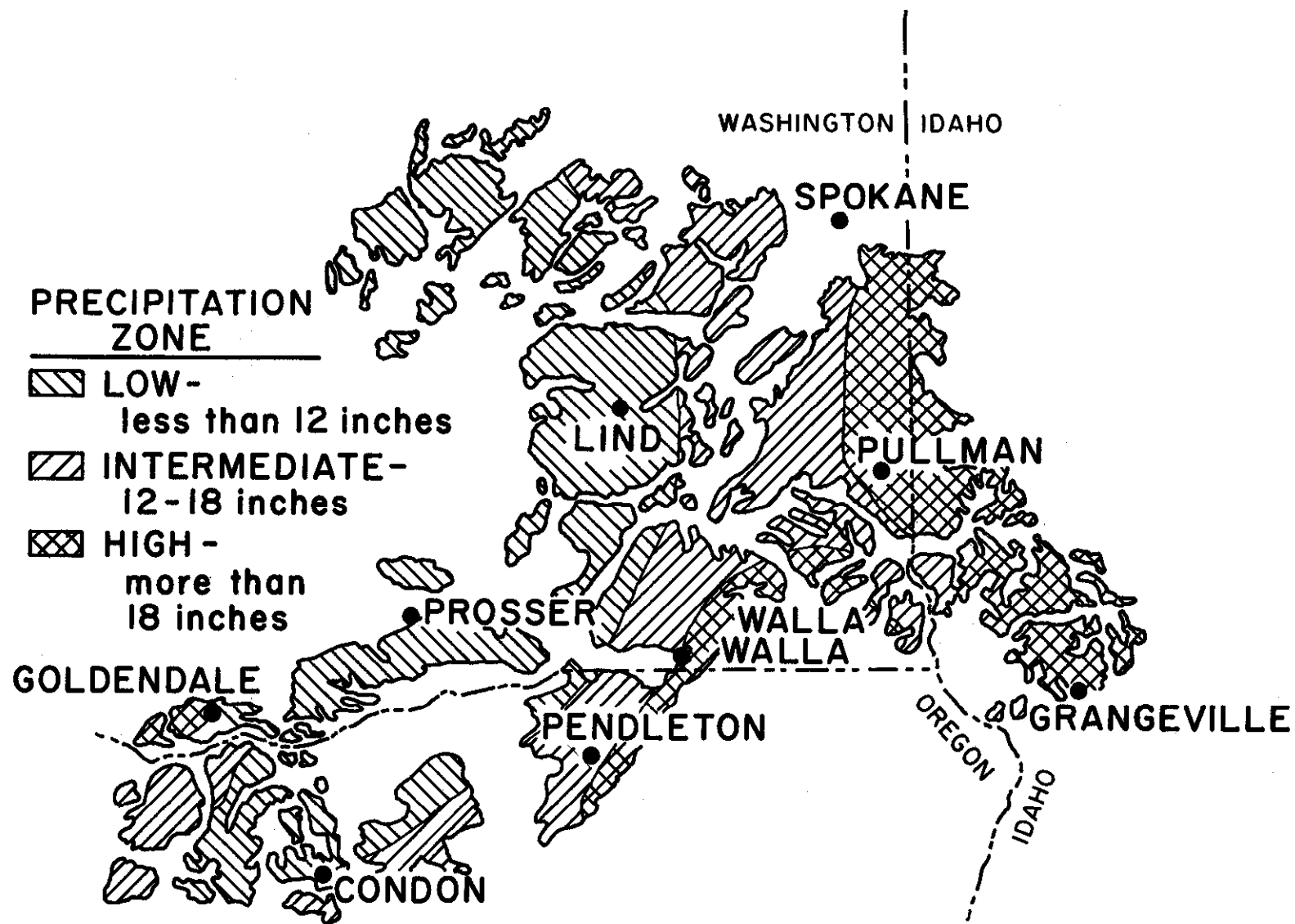


Fig. 1. Dryland Grain Region of the Pacific Northwest

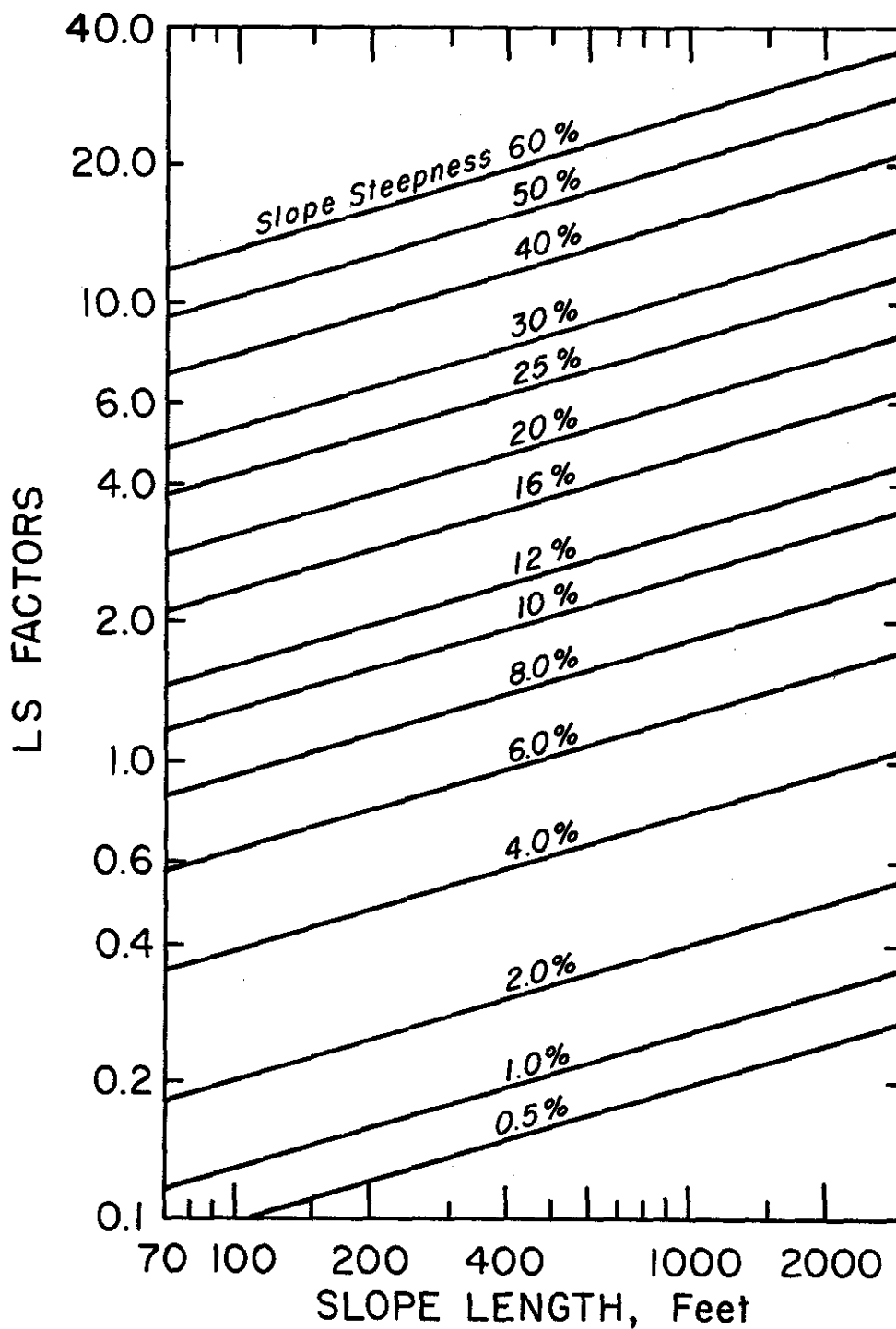


Fig. 2. Tentative LS Factors for Dryland Grain Region of the Pacific Northwest.

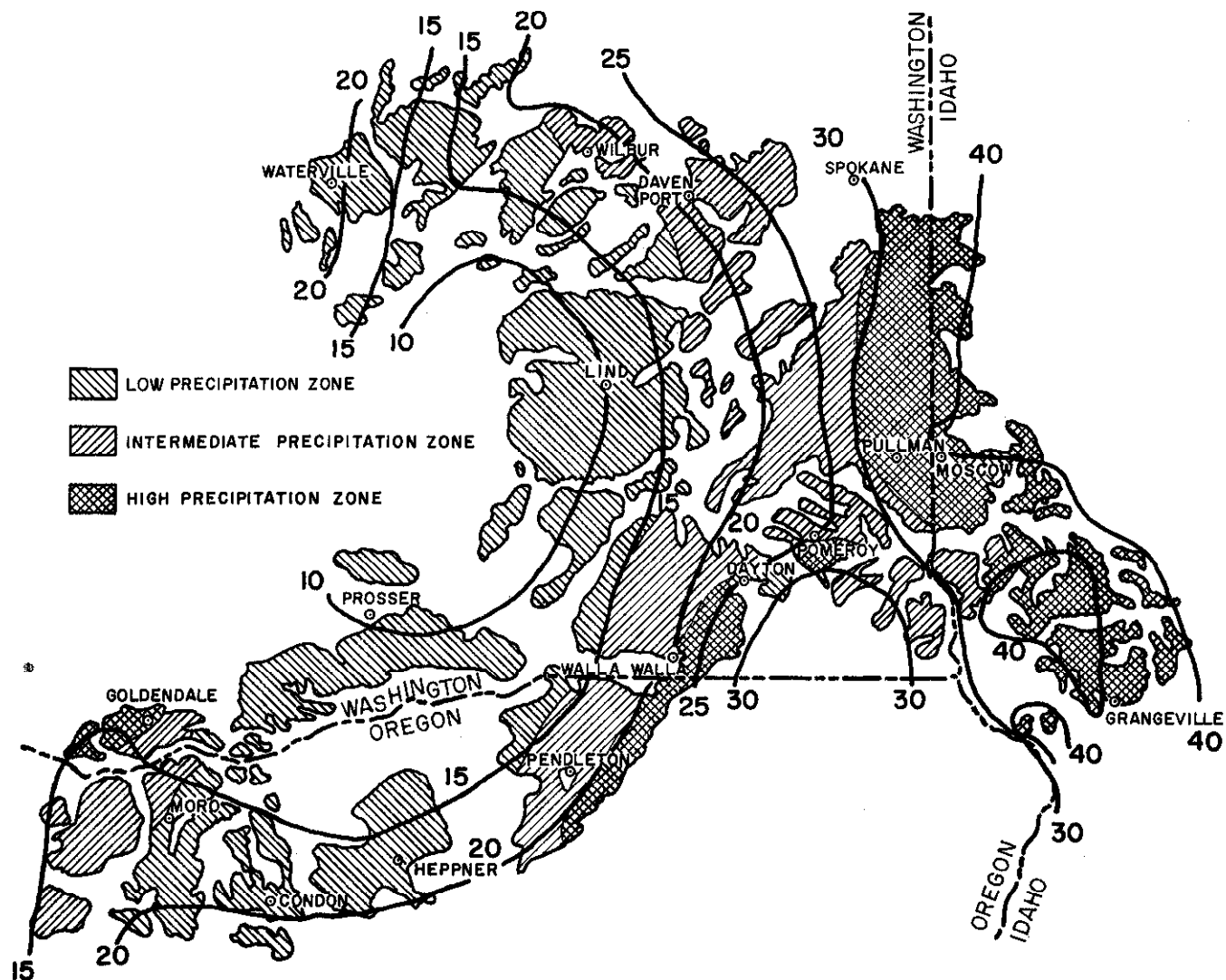


Fig. 3. Rainfall and Runoff Erosion Factor for Dryland Grain Region of the Pacific Northwest.

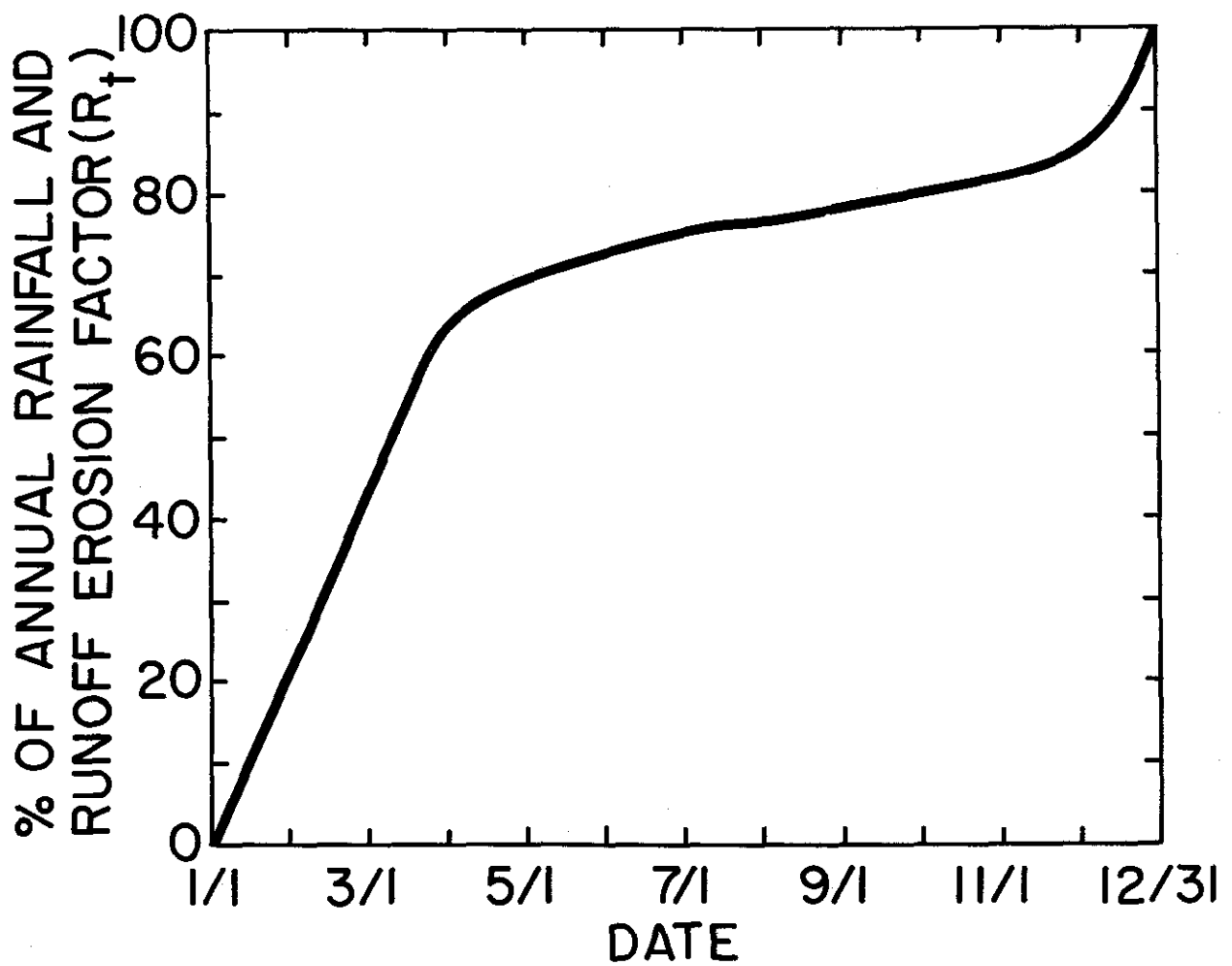


Fig. 4. Rainfall and Runoff Erosion Factor Distribution for Dryland Grain Region of the Pacific Northwest.

TECHNIQUES OF RESEARCHING INFORMATION FOR THE DEVELOPMENT
OF A MANUAL TO VEGETATE SOILS OF LOW PRODUCTIVITY
DISTURBED BY CONSTRUCTION ACTIVITIES

Francis Wm. Bennett
Senior Engineer, for Agriculture and Environment
Midwest Research Institute, Kansas City, Missouri

Dr. Roy L. Donahue
Consultant on Soils, Agriculture, and Environmental Sciences
Midwest Research Institute, Kansas City, Missouri

ABSTRACT

This paper explains the strategy, planning procedure, resource personnel used, documentation, and special problems encountered by the Midwest Research Institute in the process of developing a manual under contract with the United States Environmental Protection Agency on vegetating soils of low productivity disturbed by construction activities. The manual emphasizes the control of soil erosion and the reduction in emission of sediment. Persons, agencies, and institutions contacted for resource information totalled 203, and more than 1,000 documents were studied, 420 of which were considered sufficiently useful to be included in the list of references.

I. Introduction

A. Overview

The infinite combinations of factors causing low soil productivity induced this serious remark from one of our resource persons: "How can you write a manual for the fifty states on how to vegetate soils of low productivity that have been disturbed by construction activities when every foot is different?" With "encouragement" such as this, we began to compile our "impossible manual." Our reply was: "We have an EPA contract to do this work and we intend to fulfill the contract. Will you please send the reference material that we requested? We will put your name on a mailing list so that you may read how we reconciled your question. Thank you. We may be asking you for further information."

Low soil productivity may have its origin naturally or anthropically. Natural causes of low productivity include long and steep natural slopes, a low supply of all available plant nutrients as in some beach sands, unbalanced plant nutrients such as a severe deficiency of nitrogen, low or uneven distribution of rainfall, temperatures too cold or too hot

for optimum plant growth, or a shallow root bed caused by large stones, bedrock, or dense clay, or droughty and deep sands. Anthropogenic sources of low soil productivity are long and steep construction slopes and resultant unstable soils, coarse, cloddy, or stony slopes, smooth and compacted soil surfaces, and clay soils made too dense for plant growth because of compaction when wet by heavy machinery.

B. Strategy

To reduce the major infinite combinations, we decided to select extensive soils of low productivity by using the United States system of soil taxonomy that was adopted officially in 1965 by the National Cooperative Soil Survey, led by the Soil Conservation Service. This system consists of classifying soils into six groups: orders, suborders, great groups, subgroups, families, and series. An available soil map of the entire United States at the lowest category is one that delineates the 225 soil Great Groups. A formative element in the name of each Great Group implies relative soil productivity.

The most extensive Great Groups whose names contained a connotation of low soil productivity were selected for this research study as shown in Table 1.

For simplification of the study, nine of the ten demonstration sites were selected on the basis of soil Great Groups with low productivity and the tenth site was chosen because of intense soil acidity created by weathering of iron disulfide (pyrite) in shallow geologic sediments. In the aggregate, the ten sites chosen represent about 38% of the land area of the fifty states.

Techniques for researching this information on soil Great Groups consisted principally of using previous contacts (and from these, making new ones) in the Soil Conservation Service in Washington, D.C., many of the State Soil Scientists, and personnel in forty of the cooperating Land-Grant Universities.

II. Techniques of Researching

A. Planning

Prescribing a comprehensive procedural plan for writing the manual and a time schedule for its execution, commanded a major segment of the energy expended in writing the project proposal for the United States Environmental Protection Agency. Based on MRI experience and a little crystal ball gazing on the side, we set up a 9-month schedule.

Table 1. GREAT SOIL GROUP, FORMATIVE ELEMENT, AND CONNOTATION
OF NAME INDICATING LOW SOIL PRODUCTIVITY

<u>Soil Great Groups</u>	<u>Formative element</u>	<u>Connotation of low soil productivity</u>
<u>Cryaquepts</u> <u>Cryumbrepts</u>	<u>Cry</u>	Cold
<u>Dystrochrepts</u>	<u>Dystr</u>	Few plant nutrients
<u>Fragiudalfs</u>	<u>Frag</u>	Brittle pan
<u>Haplargids</u> <u>Haplorthods</u> <u>Hapludults</u>	<u>Hapl</u>	Least advanced horizon
<u>Haplargids</u> <u>Paleudults</u> <u>Quartzipsamments</u>	<u>Id</u> <u>Pale</u> <u>Psamm</u>	Dry Old, highly weathered Sand
<u>Quartzipsamments</u> <u>Hapludults</u> <u>Xerorthents</u>	<u>Quartz</u> <u>Ult</u> <u>Xer</u>	High content of quartz Ultimate in weathering Dry

The Program Plan was divided into seven tasks which included a review of the first draft by MRI consultants and peer reviewers selected by the Midwest Research Institute. (Figure 1.) These critiques would be the basis for writing the first draft submitted for review by the EPA at the end of 5-1/2 months.

These seven tasks were expanded, subdivided, and ordered during the nine-month period into a Program Schedule. (Figure 2.) After the award of the contract, the Program Plan and Program Schedule served as a directional and chronological guide during all phases of development of the manual.

B. Staffing

In the subject program, this group consisted of a project staff of two, a research manager, and three consultants who worked on assigned aspects of the program. The research manager was educated as a chemist and had worked as a chemist, an environmental scientist, an aerospace scientist, and as a research and development manager. The two co-principal investigators are basically an agricultural engineer and a soil scientist. Their experience is collectively varied and extensive: design and use of earth-moving equipment; agronomy; forestry; ecology, soil and environmental sciences; overseas transfer of agricultural technology such as crop development, irrigation, soil testing, food production technologies, machinery development, farm mechanization, grain storage, and applied soil science to the developing countries of Asia, Africa, Europe, and South America; systematic evaluation of pollution control in forestry; and textbook writing. The three special consultants consisted of two agronomists and a soil scientist with field experience in the arts and sciences of vegetating disturbed soils on the west coast, central United States, and on the east coast, respectively.

Peer reviewers of the entire draft manual consisted of relevant personnel of the Soil Conservation Service, the Forest Service, and the three short-term consultants. Persons who reviewed segments of the first draft included civil engineers, agronomists, soil scientists, horticulturists, hydrologists, botanists, and foresters. These reviewers were employed in all regions of the United States. Their contributions consisted often of contrasting opinions that had to be reconciled by the principal investigators. The principal investigators also had strong differences on some subjects. Hours of person-to-person discussion, however, harmonized the diversities.

C. Resource Documents

Our resource documents included computer bank printouts, current bulletins, recent books, and historic works that had been by-passed because of changing priorities and fads in research, and Xeroxed and unpublished

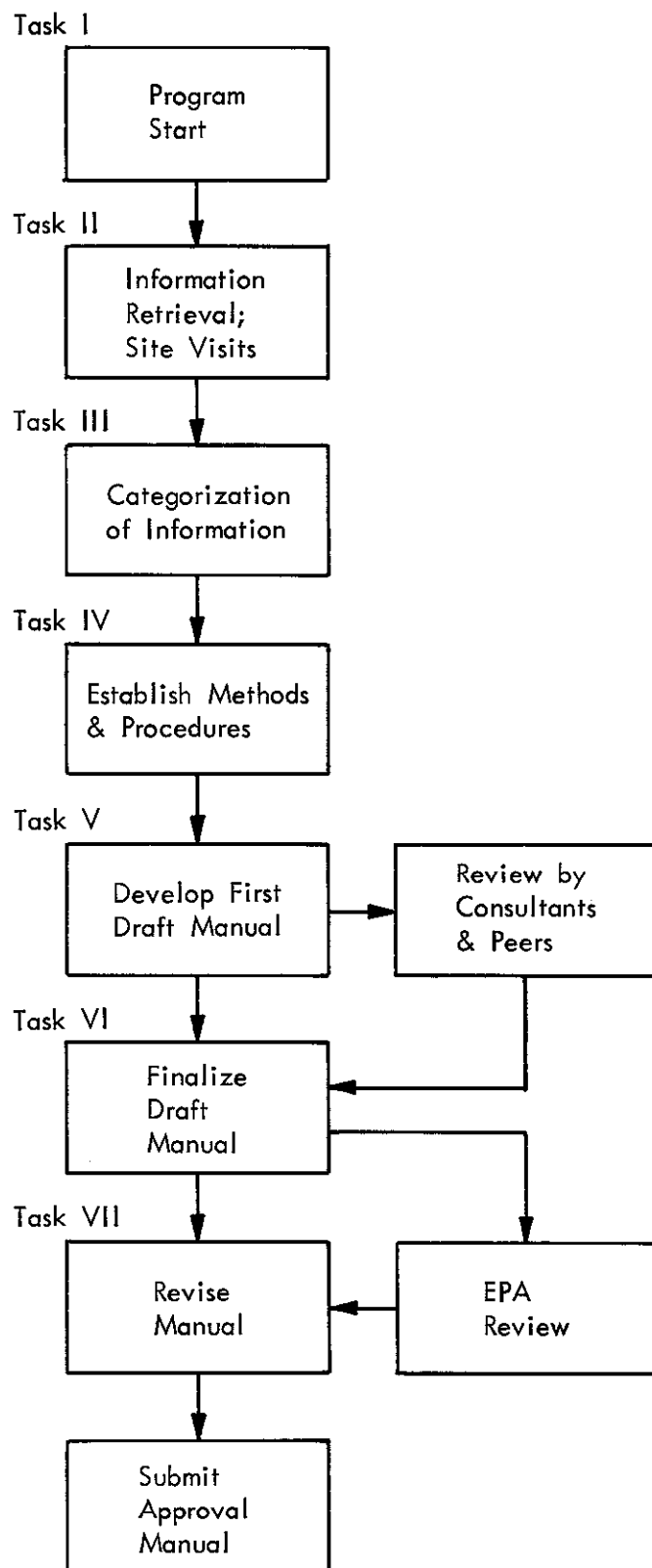


Figure 1 - Program Plan

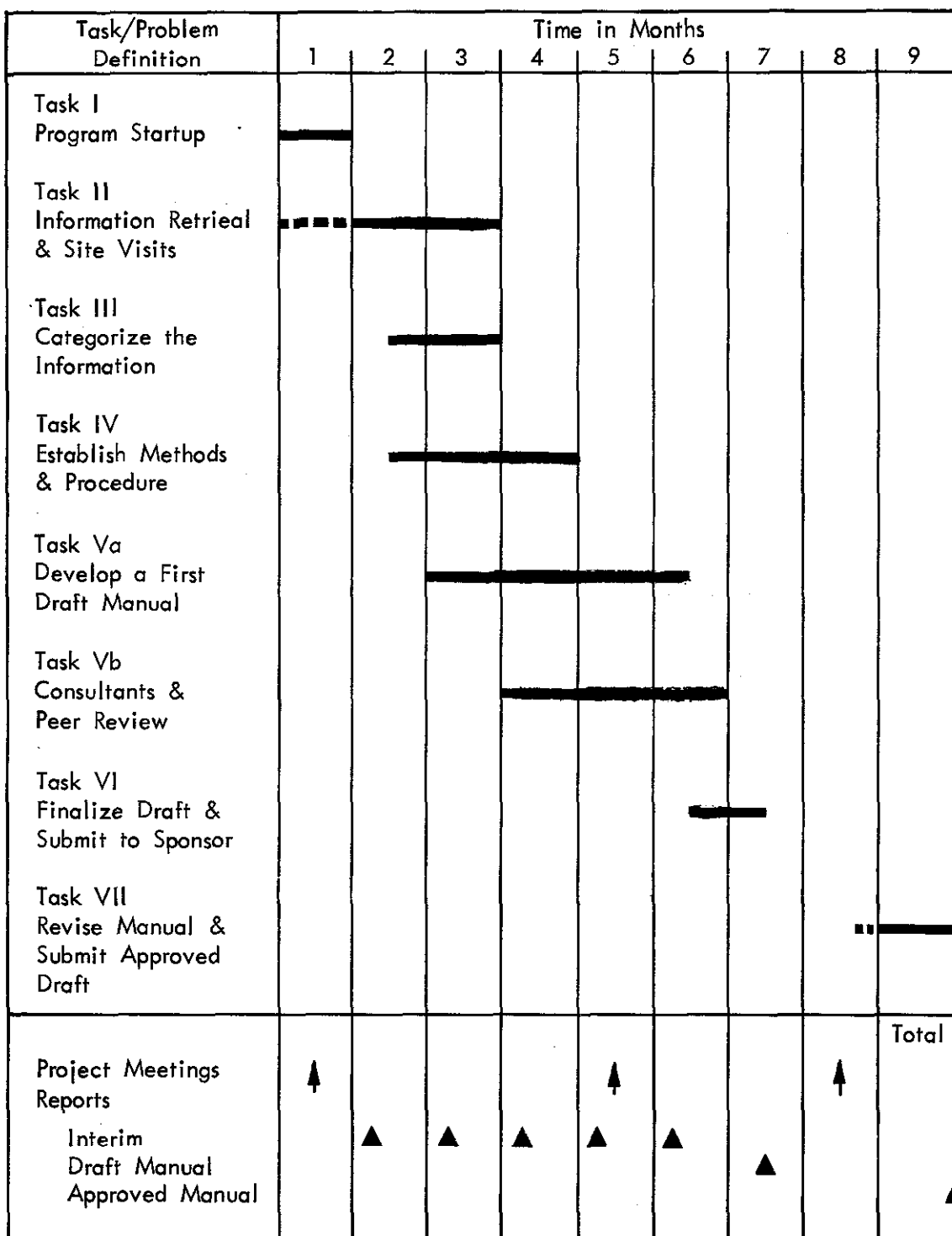


Figure 2 - Program Schedule

research reports just completed. Concurrent to the literature search, we made several hundred telephone calls in an effort to search out where research was in progress and where groups or committees were working to write documents on related subjects that were soon to be released.

Information documented on the technology employed to establish vegetation at each of the sites came from personal visits to the sites, the employing of three consultants for assistance on five of the sites, the use of officials in the State Highway Departments, at the Land-Grant Universities, the Soil Conservation Service, published bulletins and books, as well as unpublished documents that we were privileged to use, and from our prior experience.

In total, more than 1,000 documents were studied with 420 of them listed in the final draft of the manual and 220 of these were specifically cited. By telephone, letter, and personal visits, the following resource persons, agencies, and organizations were contacted in our techniques for researching information:

61 State Agencies

43 Corporate Industries

40 Land-Grant Universities

23 Professional Individuals

19 Branches of Federal Agencies

10 National Organizations

5 Private Firms

2 Libraries, including the Linda Hall Library, a national repository, and the library at Midwest Research Institute.

There is another dimension to information retrieval which amazes us every time we conduct a study of this type. It has to do with trust. Mostly from telephone conversations explaining our need for particular information, we received and reviewed dozens of manuscripts from persons we have never met, who asked only that we return the material to them. We are very grateful for their confidence, goodwill, and willingness to be helpful. On several occasions, the one-of-a-kind document was misplaced and not returned when promised. A telephone call followed with our apology.

D. Demonstration Sites

The use of consultants working directly in the field in their respective regions substantially expanded the team effort to document practical and successful experience in the art of establishing vegetation on difficult and contrasting sites.

To document current technology on how to establish vegetation on soils of low productivity that have been disrupted and intermixed by construction activities, ten sites throughout the United States were selected. Criteria of selection included:

1. An extensive soil Great Group with a name connotating low productivity.
2. Within each Great Group a specific soil series or group of soil series was researched to determine if vegetation had been successfully established on a construction site. In some areas this detail was not feasible and in one area an acid-forming geologic formation was selected because of repeated failure to establish grass even after as much as fifty tons of lime per acre was applied.
3. Every important detail concerned with establishing and maintaining vegetation was documented and placed in a separate section in the manual. These details include the slope grade, the condition of the soil surface, soil nutrient testing, fertilizing and liming, selection of seeds or plants, mulching, seeding, or planting, and successful maintenance procedures.

III. Special Problems

The over-riding problem encountered in developing this practical research manual was trying to predict the multidisciplinary and interdisciplinary users. We are aware that persons who design and build roads are primarily civil engineers, but often the civil engineer is not the ultimate decision-maker nor the ultimate implementer of the decisions.

At the time that we were researching information, the Federal Highway Administration was requesting from each state their policy on environmental impact, in order for the highways to be assisted with Federal funds. Such policies were being developed by personnel from the State Highway Departments, assisted by personnel from other agencies. The timeliness of this request encouraged the states to take more interest in the development of our manual but also limited the time of the concerned personnel available to us.

Contrasts in meanings of words and terms recognized by the various disciplines was another problem that had to be faced. What is soil? What is sand? What is a terrace? What is erosion? Is erosion a part of sedimentation? These were some of the difficulties encountered; their solution led to multiple definitions in the "Definition of Terms."

The best resource materials were current unpublished research reports that had not yet been officially cleared for release. Some of such reports were denied to us; others took uncommon persuasion to obtain. Libraries were of little value except for historical documents. Least valuable were the five-to ten-year-old bulletins and books whose technology was outdated.

Our final draft contained 32 black and white photographs that were interspersed throughout the manual and keyed to the text. Obtaining high-quality photographs on the right subjects and on time was one of the most patience-trying activities.

Since the manual was written for use in all fifty states, one prerequisite was that the presentation had to be made with no real or apparent regional or state bias. Such a criterion was approximated by selecting three consultants as widely separated as possible, working for a Research Institute located near the center of the United States, and by having Principal Investigators that have lived and worked in most regions of the United States and on all continents, except Australia, for periods aggregating eighteen years.

IV. References

Soil Conservation Service, "Soil Taxonomy," Soil Survey Staff, U.S. Department of Agriculture, 330 pages, October 1973.

U.S. Geological Survey, "Distribution of Principal Kinds of Soils: Orders, Suborders, and Great Groups. National Cooperative Soil Survey Classification of 1967," U.S. Geological Survey, Washington, D.C., Sheet No. 86. Also available in: "National Atlas of the United States," U.S. Geological Survey, 417 pages, page 86, 1970.

Environmental Protection Agency, "Methods of Quickly Vegetating Soils of Low Productivity, Construction Activities," Written under contract to Midwest Research Institute. In press, 1975.

THE DILEMMAS OF SETTING SEDIMENT STANDARDS

By J. Gessler, Associate Professor, Department of Civil Engineering, Colorado State University, Fort Collins, Colorado.

ABSTRACT

Society has a multitude of interests in the quality of our rivers as it relates to the movement of sediment. Some of these interests are directly contradictory to each other. Reducing suspended sediment loads in the interest of water quality as related to domestic and industrial use or as related to recreational purposes may result in off-setting the delicate equilibrium of the river's biota due to changes of the food supply or may lead to drastic changes in the river's geomorphic structure. With the biotic and geomorphic characteristics changing from river to river sediment standards cannot have the form of absolute upper and lower limits for the total sediment transport. They must be tied to the past history of the individual river.

Standards without enforcement are meaningless. This will require monitoring the total sediment transport at strategic locations. Selection of such locations again leads to a dilemma. Deviations in sediment transport from some base line data must be investigated and their cause determined, whether it is natural or man-induced. In order to do so the deviations must be relatively strong and the number of possible sources relatively small. This demand puts the monitoring stations into the far upstream regions of the watersheds. But this will require a large number of stations which makes the monitoring at these locations economically unfeasible.

Ideas for possible solutions are suggested. Special emphasis is on areas in which knowledge is insufficient for reaching any conclusions and will require extensive research efforts.

INTRODUCTION

There is ever increasing pressure from society at large to protect all aspects of our environment from man's interference. This has led already to numerous restrictive regulations largely in the area of air and water pollution. The tighter controls on water pollution has certainly resulted in considerable improvements in the quality of some of our streams and rivers. Yet, an area of water quality remaining with little explicit regulation is the amount of sediment carried by streams and rivers. Man's activity in the watershed has led to drastic changes in the sediment load: excessive sediment erosion due to construction activities or poor farm land management has resulted in severe damage to the river biota, or elimination of most sediment transport in a river as the result of a dam closure has caused geomorphic changes in the riverbed, completely destroying the delicate equilibrium which was reached over centuries. Man has acted irresponsible in many situations concerning sediment in rivers and it is then perfectly logical that attempts are made to set sediment standards in order to improve the quality of the streams and rivers.

But sediment as a pollutant is distinctly different from most other pollutants which leads to considerable difficulties in setting such standards. It is the purpose of this paper to discuss some of the problems associated with setting such standards.

CHARACTERISTICS OF STANDARDS

Most standards developed out of concern about pollution of our rivers by pollutants truly foreign to the river environment. When their effect on the environment became unfavorable it was a relatively simple matter to limit the amount of pollutant which could be discharged into the river. The regulation may specify that the concentration of a pollutant in the river cannot exceed a set standard. Control of such a regulation again is relatively straight forward since the pollutant is foreign to the river. Any traces measured are due to man's interference.

The situation has become more difficult with the identification of pollutants which are naturally present in the rivers. The term pollutant now obtains a somewhat broader meaning: any "matter" which is carried by the river on an "unfavorable" concentration level. The term "unfavorable" calls for a definition: is unfavorable equivalent to unnatural or can a pristine river be polluted? Does "matter" carried by a river become a pollutant as soon as there are concentration changes due to man's interference? Salinity and heat pollution are two examples for pollutants which fall into this category and with which one is quite familiar. And, certainly sediment belongs to this group of pollutants as well. In fact, after close examination one detects a surprising similarity between heat as a pollutant and sediment as a pollutant.

Setting standards and controlling them becomes difficult where the pollutant is not foreign to the pristine river. The river environment is adjusted to the pollutant. Its total elimination does not necessarily lead to an improvement. If the system can tolerate (or needs a certain amount of) the pollutant can it adjust to higher (man induced) concentrations? In regard to controlling standards the situation is quite complicated because the concentration level in the pristine river is likely to be a random variable. The situation with man-induced pollutant cannot be directly compared with a situation as it would have existed without human interference.

Heat pollution may serve as an example. If, at some given point, a regulation would permit the increase of the water temperature by say 5°F (e.g., due to a series of nuclear power plants further upstream), control of such a regulation would be difficult since the temperature under conditions without the power plants (but all other parameters the same) remains unknown. From the point of view of control an absolute standard (an upper limit which cannot be exceeded) is to be preferred. But then the amount of pollutant which can be released into the river becomes a random variable. From an operational point of view this is highly undesirable.

PURPOSE OF SEDIMENT STANDARDS

Concerns over sediment as a pollutant developed largely over a situation where man induced excessive amounts of sediment into the stream and river system. But the lack of sediment load can be just as serious a problem. Comparison

with the river temperature is again appropriate. Due to release practices at the foot of large dams some rivers below such dams show an almost constant temperature throughout the year. The lack of higher summer temperature can lead to situations where certain species of the river biota cannot complete their life cycle and become extinct. Other species, not affected by the temperature change, may depend on the extinct species as food supply and then become indirectly affected. Such a chain reaction may affect the entire biota.

A lack of sediment moved in a river may pose serious problems at two fronts: nutrient transport in a river is closely related to sediment transport. Reduction of the sediment load may then result in insufficient food supply for the biota; and a lack of sediment load may lead to excessive erosion in the riverbed resulting in extensive damage to the river channel. Sediment transport in a river must not be "low", or "high", it must be "right", i.e., fall into a certain interval. But considering the natural variation within the biota and of the geomorphic characteristics, specification of an interval is not easy.

One may say that the purpose of sediment standards is to maintain a viable river biota and geomorphic equilibrium. Considering the random aspects of sediment transport this may mean to insure that changes in the sediment load of streams and rivers due to human activity will not result in adjustments in the river environment which are more significant than those adjustments and variations which would occur naturally.

If standards oriented toward these purposes do not suffice for sufficient water quality for industrial and/or domestic use, more rigorous standards (smaller sediment concentrations) can only be implemented if this does not upset the biotic and geomorphic equilibrium. Recreational aspects of sediment standards have a very low priority. Society may have to learn that a river with clear water is not necessarily "more beautiful". Among certain segments of society this may well require a change in the value system - a very slow process indeed.

SEDIMENT SOURCES - MEASURING POINTS

By the time the sediment becomes a pollutant in streams and rivers it has already extensively moved through the environment by various modes of transportation. But none of these modes permits easy measurement of the sediment flux. Only after the sediment has reached a more or less continuously running stream does it become possible to measure certain aspects related to sediment transport.

Turbidity is a characteristic presently used in defining sediment standards and it is quite easy to measure. Suspended sediment load can be determined without too much difficulty, yet such measurements require a lot of time. Bed load is usually not measured. It would take very expensive equipment to do so. Any measurements before the sediment reaches a stream is much more difficult, if not totally impossible.

The sediment sources and the measuring points are far apart; far in distance and far in time. If the sediment standards of a particular river get violated it will be necessary to locate the excessive source. Yet, the further apart source and measuring point the more difficult the task of locating the source.

A typical example for a source is sediment erosion due to rain and wind from cultivated land. There is no feasible way of directly measuring such loss. The detachment process is intermittent (stochastic), the initial transport phase is intermittent, there is temporary storage in the intermittently running creeks before the sediment reaches the streams which continuously carry water and sediment, and which permit some measurement of the sediment flux. By the time the sediment flux measurements indicate a problem, the conditions at the source may have changed again.

The measured sediment flux (assuming it can in fact be measured) is the integration of sediment production and transport from all sources upstream from the measuring point. And, if excessive sediment flux is observed, it may not even be an excessive source in the watershed but could be due to geomorphic changes within the stream system, totally unrelated to the sediment production of the land. A typical example would be the changes and adjustments in the meander pattern of a stream where large amounts of sediment may be transported over relatively short distances. Such changes could well be within the natural adjustments of a riverbed.

The chances of pin pointing excessive sediment sources (and this must be the objective of monitoring sediment flux at some station) are better the smaller the possible number of sources, or the smaller the watershed. An effective control system would need numerous measuring points at the small streams. Such a requirement is in clear contradiction to the demand for an economical surveillance system.

CONTROL AT THE SOURCE

If the first point for possible measurement of sediment flux in the system is already too far from the source in order to locate the source, it appears logical to control sediment production at the source. In regard to soil losses from farm land or forests (true non-point sources) such control would have to specify the management practices employed.

Point sources could now be defined as a source where direct measurement of the sediment production is possible. For instance, at a construction site of a river crossing measurements directly upstream and downstream of the site would permit to determine the strength of the source. Control at such sources would be relatively simple, though possibly quite expensive.

Certainly the land users would consider specification of the management techniques as an undue interference. Nevertheless, society at large has a clear interest in the movement of sediments in the environment. As long as the land user employs "standard practice" as spelled out in the regulations, he would not be questioned in regard to sediment production. If he would like to use other than standard practice he would have to prove that such techniques would not produce "excessive" amounts of sediment.

HOW TO DERIVE STANDARDS FOR A RIVER

In the previous paragraph it was mentioned that the sediment flux should be controlled at the small stream, but that this results in an economically unacceptable surveillance system. One could then take the attitude that

standards be only developed and set as the needs on specific rivers and streams develop. But by the time the need is discovered too much damage may have been done without the possibility of legal steps against the owner of the source. Furthermore, in order to develop standards, the conditions of the pristine river (or as nearly as possible pristine) should be known. Such conditions clearly would be lost by the time the needs for standards are discovered.

An alternative would be to select and survey certain rivers, which for one reason or another are of considerable importance. These rivers would be studied in great detail and standards would be set for them one by one according to their geomorphic and biotic characteristics, and possibly other factors could be included as well. Other rivers would not have specified standards. The assumption is that such uncontrolled streams eventually discharge into rivers with standards. And, if an uncontrolled stream shows excessive sediment transport it would eventually show up in the controlled stream. The fact that individual streams may become highly unstable is not at all much different from what can occur under pristine conditions. Yet, again from a legal point of view, such a procedure is not desirable, even though it may achieve the goal of a generally stable river environment. The important question is whether legal steps can be taken to correct an unsatisfactory condition in a small stream if no specific standards are set. Like in the previous paragraph, the answer is most likely a no.

The question then becomes one of how to set standards for each river and stream in an economically feasible way, keeping in mind that two pristine rivers of similar hydraulic characteristics may show widely differing sediment fluxes. Two main components of setting standards have been identified: the biotic aspects and the geomorphic aspects. If rivers and streams are classified according to their geomorphic aspects one would not expect very drastic changes as one moves from one watershed to the next adjacent one. One could then map the geomorphic characteristics of rivers with actually surveying only relatively few, carefully selected rivers. Much of the same argument applies to the biotic characteristics. Again, these could be mapped, without actually looking at each river. Then these two maps could be superimposed and the combination of a certain geomorphic class with a certain biotic class would determine the applicable sediment standards.

Whether more than the two classes mentioned need to be mapped may remain, at this stage, an unanswered question. One could think of mapping of hydrologic or climatologic characteristics. Yet, at this time it is felt that either one would at least indirectly be reflected by the two classes first mentioned.

Such a mapping procedure does not say anything on what constitutes acceptable limits for sediment transport, or in other words, a reasonable interval for sediment transport. It merely suggests a possible procedure which appears economically more feasible than the alternatives.

WHAT STANDARDS

It was mentioned above that at present turbidity is the only sediment related standard of considerable importance. But turbidity is certainly not a good indicator for geomorphic stability. It depends very much on chemical-physical

characteristics of the suspended material which may or may not be related to the material close to or on the riverbed, i.e., material which has more immediate influence on geomorphic stability. Furthermore, turbidity is, generally speaking, not a good indicator for the food supply for the biota or the biota itself. The only direct information provided is on light penetration which has no direct influence on geomorphic stability and is only one of several parameters which influence the biota.

The ideal standard would certainly be one which relates to the total sediment transport characteristics of the stream, i.e., to the sediment transport of each grain size, as a function of water discharge, or stage. This information is usually not available at present time, even at rivers which are extensively surveyed. Data collection is now largely oriented toward discharge and suspended sediment concentration. This has a simple explanation: determination of water discharge is easy after a stage-discharge relationship is measured at a stable cross section; and collection of water samples from various depths poses little difficulties beyond the fact that it is time consuming. Of course this latter aspect is the reason why such suspended sediment transport measurements are very limited and concentrated at the larger rivers.

Why is such information insufficient? The development in time of the longitudinal profile of a river or stream is much more affected by the bed load transport than the suspended transport. Many rivers carry many times their bed load as suspended load. Yet, significant river bed changes can only occur if the bed gets highly mobile. Periods of extensive bed load transport are at many rivers very short and limited to flood stages (especially in the smaller rivers). But in larger rivers bed load transport may be almost continuously in process throughout the year. What situation prevails depends largely on the size distribution of the bed forming sediment. If one would like to understand the geomorphic changes in a river, one must understand the influence of grain size distribution on the bed load transport.

Grain size distribution of the bed material also is an important parameter for the biota since it determines the geometric shape of the boundary. It, furthermore, affects the renewal rate of the top few inches or so of the riverbed which for the biota is an important part of the system.

The grain size distribution of the suspended material may be of lesser significance for the geomorphic characteristics of the stream or river. But for the biota it appears to be of great importance. Inorganic suspended sediment transport is likely to be heavily correlated with the organic sediment transport, which forms the food supply for the biota. And, finally, as mentioned above, the suspended material (including its grain size distribution) determines turbidity, i.e., light penetration, which directly affects the biota.

Sediment standards then should take into account the following aspects: The full spectrum of grain sizes in streams and rivers is of significant importance for the biota. At least the grain sizes of the bed material controls the geomorphic equilibrium. Inorganic sediment standards need to be set for that full spectrum of grain sizes. Standards for organic sediment may be set separately to insure appropriate food supply; yet the two may be so heavily correlated that a separation is impossible and one would have to rely on the assumption that maintaining standards for the inorganic matter insures adequate

supply of organic matter.

One important aspect which to this point remained undiscussed, is how much the standards should relate to the status quo or how much of past damage has to be corrected. If such correction is necessary, the main problem, of course, is what constituted pristine conditions. But, perhaps a very pragmatic position can be considered. It is the purpose of standards for sediment transport in rivers and streams to maintain the geomorphic and biotic equilibrium, and one should realize that a true geomorphic equilibrium does never exist and that at a time scale of decades a river slowly adjusts. Nor is there a true biotic equilibrium, which probably shifts at a much shorter time scale. As long as a river shows conditions fairly close to the long-term equilibrium in both areas, the standards could be based on the status quo, no matter whether the status quo represents more or less pristine conditions. "Fairly close" should be interpreted to mean "within the range of natural fluctuations". Yet, if the equilibriums are off-set drastic measures are needed to reach new equilibriums which again do not necessarily have to coincide with the equilibrium of the pristine river.

In summary, we can now conclude:

Standards must be set for all streams including the very small ones, else it becomes impossible to locate and take legal steps against excessive sediment sources. Such standards need to be very sophisticated and must relate to the full grain size distribution of the moving sediment for geomorphic and biotic reasons. But extensive measuring programs in which data is collected on a more or less continuous basis are then economically impossible.

In order to insure the quality of our streams it is suggested that standard land management techniques are developed for the various regions of the nation which insure acceptable soil erosion levels. But this does not eliminate the needs for standards in the streams. Such standards must take into account the highly individual characteristics of each stream. Since it is an impossible task to survey each stream and set its own standards, such standards must be related to at least two key parameters: the geomorphology and the biota of the river. Each of these factors may not vary significantly within one region. Instead of mapping each stream it may be sufficient to map regions. Any combination of geomorphic and biotic characteristics of the region would then determine the standards applicable to the region's streams.

RESEARCH NEEDS

Based upon the previous discussion one immediately recognizes that before the task of setting standards can be considered considerable research has to be undertaken.

Research Needs as Related to Land Management

It is fully appreciated that this is not directly part of the overall discussions as defined by the title of this paper. But, because of the previously mentioned dichotomy - sediment source/measuring point - it must be included.

The universal soil loss equation is a well established tool. Nevertheless, it

tells very little about the temporary storage of eroded soil between its original location and the point where it reaches the permanently established water course. This storage is important for two reasons: (1) it controls the time scale of the sediment movement between source and stream and (2) it may offer a last chance to keep the sediment from reaching the streams by changing such temporary storage into permanent storage. In the overall analysis of the sediment movement in the environment, this storage adds another highly stochastic component. At present this component enters into the calculations in form of a mean "efficiency". Yet, its statistical distribution in time and space appears to be of significance.

Research Needs as Related to River Morphology

Field measurements of bed load are an absolute necessity in order to learn more about overall bed stability. Yet a continuous survey program collecting such data at many locations is not possible because of the financial consequences.

The use of remote sensing for almost instantaneous measurement of suspended sediment load along dozens of miles of a river should be investigated. Such measurements would need to be calibrated against a number of actual concentration measurements made at the same time the river's length was flown. With relatively small manpower input, a very substantial and accurate piece of information could be obtained. Such data would introduce a new concept of instantaneous survey over many miles, where in the past one obtained data at one point, more or less continuous in time.

It was pointed out that two closely related aspects in river morphology need much more attention: bed load and grain size distribution. Bed load research would be of particular interest when related to the corresponding suspended load. Such research cannot be done under laboratory conditions; they must occur in the field. Because of the large effort required, only very few sites can be considered and need to be carefully selected, most likely on small streams. After one has gained new insight into the process one can draw the appropriate conclusions for the development of standards and will discontinue to collect more bed load data.

At the same time one needs to observe the grain size distribution curves of the moving material. Only one sediment transport equation in wide use predicts grain size distribution. Yet it was developed only for rivers with sandy beds and it never claimed to be accurate in respect to the grain size distribution. It is not at all an overstatement to say that there is no tool predicting the grain size distribution of the sediment transported in a stream or river. Closely related to the problem of grain size distribution is the one of natural sorting processes in rivers including the armoring of riverbeds and its subsequent destruction in the following flood. Again, almost no information is available; yet, clearly this is directly related to the transition from stable to unstable bed configurations.

Research Needs as Related to the Biota

The greatest research need appears to be in a description of the biota and its

distribution in the water and river bed in terms of the geomorphic characteristics of the river. A good deal is known about land management and its affect on soil loss, as well as about river mechanics even though certainly not enough to answer all questions related to setting standards. But about correlation between geomorphology and the biota very little is known.

The efforts should concentrate on the small streams. If the small streams can be kept in order (and of course there are many more miles of small streams than of big rivers), the large rivers will more or less automatically be taken care of.

The stream biologists seem to have very little to offer in specific knowledge at this time. This should not be surprising; their problem has a multitude more boundary conditions than all the other problems. Local climatological considerations, the hydrology of the stream, the geomorphology of the stream, etc., all need to be understood before only the boundary conditions of the biota are defined. Any research on the river biota must be done in close cooperation with the other specialists, in particular the river morphologist if the final goal is to set sediment standards.

A number of specific research topics may be mentioned. In order to maintain a biota, a certain number of "degree days" is necessary. Whether the minimum number is met by means of a few relatively warm days or an extended period of not so warm days frequently does not matter. When asking what do high sediment concentrations do to the biota it may again not be the absolute value of such concentration but rather the product of concentration and time. This opens very interesting aspects in regard to sediment standards. Specification of a concentration level never to be exceeded is possibly far from an optimum solution. The process in the pristine river is highly stochastic. Simply limiting the mean appears totally insufficient.

It was mentioned above that an important component of the biota lives in the top few inches or so of the riverbed. This layer may be, geomorphically speaking, very stable over most of the year, particularly in the small stream. But during flood stages it will get completely worked over. What is the significance of this renewal for the biota?

A third aspect of research closely related to sediment and biota is the one on food supply. In terms of transport processes there is considerable similarity between organic particles and sediment (inorganic) particles, despite the difference in specific weights. If there is a strong correlation between suspended sediment transport at a river cross section and the available food supply this would impose additional limits on the range of possible standards.

One of the major problems in regard to all research of this nature just mentioned is that it cannot be done in the laboratory but must be done in the field. Again, of major importance is the site selection.

Research Needs as Related to the Stochastic Aspects

Throughout this paper reference was made to the fact that one deals with a highly stochastic system. Hydrology is largely stochastic; sediment transport is a stochastic process, so is the river eco system. The timing of "major" (i.e.,

usually rare) events is absolutely crucial for the biota. And not only is the timing important but these rare events, e.g., a 100 year flood are necessary to maintain a healthy biota and possibly necessary to maintain the long-term geomorphic equilibrium.

The title of this paragraph is not to imply that more and new techniques are needed in the analysis of stochastic processes but all researchers should keep in mind that they deal with a stochastic system, one in which the distribution of extreme values might be of great importance. The statisticians have developed a great deal of tools which can and must be applied.

CONCLUSIONS

A number of dilemmas make it exceedingly difficult to set sediment standards which will achieve the goal of a healthy equilibrium of the biotic and geomorphic characteristics of the stream and river system. "Clean" water does not imply a healthy river - it may even work the opposite way. The spacial separation of sediment source and possible measuring point for the sediment flux imposes extreme economical limitations on what can be enforced as far as standards are concerned. The problem of time scales make any monitoring - and for that matter research - very difficult. A workable solution might be to:

- a) regulate the land management practice,
- b) set standards by classifying watersheds according to a geomorphic scale and a biotic scale, the combination of the two determining the applicable standards.

Nevertheless, before such classification can take place much has to be learned about the interaction between biota and river morphology.

ACKNOWLEDGMENT

This paper constitutes a summary of a paper (Gessler, 1975) which was compiled in connection with a workshop held at Colorado State University in 1974. These activities were supported (in part) by funds provided by the United States Department of the Interior, Office of Water Resources Research, as authorized by the Water Resources Research Act of 1964, and pursuant to Grant Agreement No. 14-31-0001-4006.

REFERENCES

- Gessler, J., 1975, Research Needs as Related to the Development of Sediment Standards in Rivers, Environmental Resources Center, Colorado State University, Completion Report Series No. 60.

DESIGN AND PERFORMANCE OF ROCK REVETMENT TOES

by Walter M. Linder, Chief, Hydrologic Engineering
Branch, U.S. Army, Corps of Engineers, Kansas City District

ABSTRACT

One of the major causes of failure of rock riprap revetments is undercutting or erosion of bank material below the base of the revetment. When this occurs, rock from the side slope migrates downward and exposes areas of unprotected bank, subjecting the entire revetment to progressive destruction. The author has been involved in several model and prototype tests of the performance of various shapes of revetment toes. This paper describes these investigations and presents conclusions that can be drawn from the studies. The studies indicate that while shape is important, the volume of reserve rock provided in the revetment toe is probably more significant. Rock from a revetment toe generally migrates downward on approximately a 1Vertical on 2Horizontal slope to form a protective layer one to two rock diameters in thickness. Protection will continue as long as there is sufficient rock remaining in the toe structure to prevent separation of the side-slope protection

INTRODUCTION

Many methods of bank protection have been developed and used depending upon the availability of labor, materials, economics, and maintenance requirements. One of the more common methods of bank protection is a layer of broken stone or riprap. Failure of the riprap placed on the upper slope can be avoided by using rock that is sufficiently large to prevent its being transported by the force of the moving water and by the use of a properly designed filter between the riprap and the bank in order to prevent the loss of fine grained bank material through the rock. One of the major causes of failure is due to erosion undermining the base of the revetment. When this occurs, rock from the side slope migrates downward and exposes areas of unprotected bank. The entire revetment then becomes subject to progressive destruction.

Undermining of bank protection can result from a general degradation of the bed; impingement of high velocity currents against the bank; and concentration of flow along a concave bank. Undercutting can be prevented by extending the revetment down to the maximum expected depth of erosion. However, this often requires deep underwater excavation and rock placement which is very costly. An alternative is to provide a reservoir of rock or a toe structure at the existing bed grade. Rock from this reservoir will slough down and continue to protect the bank as the bed degrades. Various toe geometries have been developed and used by the various organizations involved in bank stabilization and flood control. Design manuals suggest various toe geometries, but little guidance is given as to what shape works best under a given condition or how much rock should be provided for the anticipated depth of scour.

This paper describes model tests of various toe geometries and the prototype performance of the shapes previously model tested. The model and prototype tests did not yield exact design criteria, but conclusions can be drawn which should lead to better design and reduced chances for failure.

PRELIMINARY MODEL TESTS

A brief investigation of the performance of revetment toe shapes was conducted by the author at the University of Iowa hydraulic laboratories in 1962. Three different toe geometries, each having equal cross sectional area below the design bed grade, were tested. These were (1) extension of the side slope protection to the prototype equivalent of 10 feet below bed grade; (2) an enlarged section at the bottom of the side slope with its base located the equivalent of $5\frac{1}{2}$ feet below bed grade; and (3) a $3\frac{1}{2}$ foot thick by 19 foot wide horizontal blanket with the top of the blanket at bed grade. In prototype quantities the volume of rock in each toe would be about $2\frac{1}{2}$ cubic yards (yd^3) per linear foot.

Two separate tests were run. The first test was made using the slope extension on one side of the channel and the enlarged section at the bottom of the slope on the opposite side. In the second test sections of the slope extension and the horizontal blanket were placed on one side of the channel and the enlarged section at the bottom of the slope was used on the opposite side. In each test the bed was allowed to progressively degrade until there was complete failure of the revetment. Depth of degradation averaged about 0.35 ft. ($17\frac{1}{2}$ ft. prototype) in the first test and 0.40 ft. (20 ft. prototype) in the second test.

During the first test the enlarged toe section showed evidence of distress as soon as the base of the toe was undercut. However, rock from the toe sloughed downward and continued to protect the side slope. As the bed continued to degrade, the revetment with the slope extension showed no evidence of damage until the bottom of the revetment was undercut. When this occurred, the revetment separated at various locations as rock from the side slope migrated downward. Failure then progressed rapidly until the entire revetment was destroyed. Rock from the enlarged toe section continued to slump downward and protect the bank as degradation progressed. At the end of the test the bank line was somewhat uneven but the entire side slope was still covered with a layer of rock.

During the second test the enlarged toe section again showed some evidence of distress as soon as the base of the toe was undercut, and eventually some localized failure occurred. Rock was immediately washed from the surface of the horizontal blanket and continued to be removed until there was no longer an effective reservoir of rock. Progressive failure of that portion of the revetment then developed. The revetment with the slope extension also progressively failed when the base of the riprap was undercut. The enlarged toe section generally performed better than the other two types, but the difference in performance was not as clearly defined as in the first test.

Although the tests were very brief and the results of the two tests were somewhat inconsistent, they did serve to show that a reservoir of rock at the base of a riprapped side slope could provide protection against degradation to a substantial depth below the base of the toe.

MODEL STUDIES BY THE UNIVERSITY OF NEBRASKA

In 1968 the Omaha District of the Corps of Engineers contracted with the University of Nebraska to conduct hydraulic model studies of various toe geometries. The primary purpose of this contract was to extend the previous studies conducted at the University of Iowa. At the same time the Kansas City District was in the process of designing bank protection for the outlet channel at Milford Dam which is located on the Republican River in north-central Kansas. This channel was originally excavated to a 100 ft. bottom width with 1V on 3H side slopes through highly erodible fine silts and sands. Riprap had been placed on the side slopes for a distance of approximately 1,000 ft. downstream of the stilling basin. Within a short period of time the remaining 7,000 ft. length of this channel had eroded to a width of about 200 ft. and was threatening to encroach into a proposed recreation area. The proposed design was to retain the then existing 200 ft. bottom width, grade the banks to a 1V on 3H side slope, and pave the banks with a layer of rock having a median weight of 25 pounds (lbs.) and a maximum weight of 150 lbs. The rock was to be placed over a 6 in. thick filter blanket in order to prevent loss of the fine grained bank material through the riprap. It was anticipated that 7 to 10 ft. of degradation could occur after the banks had been stabilized. Extending the side slope revetment to that depth would have required up to 10 ft. of underwater excavation. Past experience with these cohesionless soils had shown that 2 to 3 ft. was the maximum depth of excavation and placing of a controlled thickness of rock that could be accomplished without dewatering. The estimated cost at 1967 price levels of dewatering and placing the rock to the expected depth of degradation was approximately \$900,000. In view of this very high cost, the Kansas City District proposed to place a horizontal blanket of rock on the bed at the base of the side slopes. The volume of rock in this blanket was to be two times the volume required if the slope protection were extended to the expected depth of degradation. The estimated cost using the rock blanket was slightly over \$400,000 or less than half the cost of extending the slope protection. Since the amount of rock actually required to provide sufficient protection as the bed degraded was not known, it offered an opportunity to use the model tests to see if additional savings might be possible.

The study was conducted at the Mead Hydraulics Laboratory, a facility jointly used by the University of Nebraska and the Missouri River Division, located at the University of Nebraska Field Laboratory near Mead, Nebraska. Ten different toe geometries were tested. These included 6 different horizontal blankets with their base at bed grade; an extension of the side slope revetment; two versions of an enlarged section at the base of the side slope; and an inverted trapezoid with the top surface at bed grade. These toe shapes are illustrated in Figure 1.

After construction of the model channel for each test shape, a discharge of about 4.6 cubic feet per second (c.f.s.) was pumped through the basin. In order to maintain a nearly constant cross sectional area of flow and velocity, the water surface elevation was periodically lowered to compensate for the loss of bed material by degradation. After several hours of operation, the flow was stopped and performance characteristics of the revetment recorded. This cycle was repeated from 3 to 5 times for each individual test. Runs were continued until there was substantial loss of bank protection. Data collected included discharge and discharge distribution, water surface and bed profiles, cross sections, measurements of rock position, extent of rock coverage, and photographs.

Details of the model studies have been described in the contract report and in an unpublished Master's thesis (Roumph, 1969-1970). A summary of test results as presented by Roumph is given in table 1.

Results of the tests were similar to the findings of the earlier tests at Iowa City. Except for the thin blanket in test No. 6, the various horizontal blankets all performed equally well. The thin blanket in test No. 6 provided a much sparser slope coverage and rock tended to scatter over the bed to a greater extent than it did for the other horizontal blankets. The extension of the slope protection performed well until it was undercut. When that occurred, a rapid and progressive failure of the entire slope protection developed. The two thickened sections at the base of the slope used in runs 8 and 9 had comparable performance. In both cases a weak spot developed at the point where the steeper slope at the back of the toe section intersected the upper side slope. The lower bank slopes that formed as a result of degradation were all very close to 1V on 2H, and in all cases the layer of rock that formed on this lower slope was seldom more than one rock thick. Frequently a narrow horizontal apron formed at the base of this lower slope. The width of this apron appeared to be dependent upon the height of the dunes in the bed of the model channel. In some cases this apron was several rock diameters thick and in general the thicker the horizontal blanket, the thicker the apron. Displacement of the toe rock was normally downward and channelward. There was very little movement of rock parallel to the channel.

The model tests confirmed that the horizontal blanket proposed as toe protection for the Milford outlet channel revetment would perform as expected. They also confirmed that the volume of rock in the toe could be substantially reduced and still provide adequate protection.

PROTOTYPE PERFORMANCE

Placing of riprap on the banks of the Milford outlet channel started in August 1968 and was completed early in 1969. Based on the results of the model tests, the volume of rock in the horizontal toe section was reduced approximately 25 percent from that required by the original design. The toe sections actually constructed were 3 ft. thick and the width varied between 12 and 17 ft. This provided a quantity of rock approximately $1\frac{1}{2}$ times the volume that would have been required to extend the slope protection to the expected depth of degradation. Four special toe test

sections, two on each bank and each 200 ft. long, were constructed at the downstream end of the outlet channel. Two different horizontal blankets were placed on the left bank; one a wide thin blanket and the other a thick narrow blanket. On the right bank one test section was essentially an extension of the slope protection except the lower 4 ft. was placed on a 1V on $1\frac{1}{2}$ H slope. The other test section was an enlarged section at the base of the slope except it also was placed on a 1V on $1\frac{1}{2}$ H slope. It was intended that the toe test sections would have equal cross sectional areas. What was actually built in the field is not definitely known as detailed surveys were obtained of only one test section at the time of the initial construction. The other three test sections were not surveyed until after several feet of degradation had occurred.

Several periods of prolonged high releases from Milford Lake have occurred since completion of the outlet channel bank protection. A discharge of at least 500 c.f.s. has occurred approximately 50 percent of the time interval between January 1969 and December 1974. In 1973 releases between 8,000 and 12,000 c.f.s. occurred for a period of 44 days. As a result of these flows, the bed of the outlet channel has degraded an average of 5 to 6 feet.

In December 1974 the outlet channel was inspected for the purpose of evaluating the overall performance of the revetment toes. It was found that the revetment toes have performed their intended function extremely well. The toe rock has moved downward as the bed degraded and covered the bank with a layer of rock that would average about two rock diameters in thickness. Approximately half of the toe rock is still in place at the base of the revetment.

The four special test sections were inspected in detail in order to compare the performance of the various toe geometries. A surveyed cross section which extended from the top of the revetment to the bed and a short distance out into the channel was obtained for each test section. The bed was probed at the base of the slope to locate the outer extremity of the rock. A small amount of rock was removed from each test section to locate the original elevation of the base of the toe and to estimate the thickness of the rock layer formed by the downward movement of the rock.

COMPARISON OF PROTOTYPE TOE TEST SECTIONS

Test Section No. 1. This toe was constructed as an extension of the upper slope protection except the lower 4 feet was placed on a 1V on $1\frac{1}{2}$ H slope. The base of the toe was to have been located 39 inches below the bed grade at the time of construction. Three to $3\frac{1}{2}$ feet of degradation below the base of the toe had occurred. The surveyed cross section indicated that the entire riverward face of the toe had receded several feet as the rock moved downward and adjusted to the degradation. The toe rock had covered the lower slope quite well, but there was some evidence of separation in the vicinity of the original base of the toe. Since there is little or no reserve of rock remaining in this toe, progressive failure of the lower slope protection may occur if there is

significant additional degradation. A surveyed cross section and the estimated original toe section are shown on Figure 2a.

Test Section No. 2. This toe was constructed as an enlarged section at the bottom of the revetment. The base of the toe was to have been located 16 in. below the bed grade. Both the front and back of the toe section were placed on a 1V on $1\frac{1}{2}$ H slope. The break in slope between the toe section and the 1V on 3H bank slope was located 18 in. above the original bed grade. Approximately $4\frac{1}{2}$ ft. of degradation below the original base of the toe had occurred. Surveys showed that the riverward face of this toe had also receded several feet as degradation occurred. Rock had covered the lower slope to a thickness of approximately 2 rock diameters. There appeared to be some evidence of separation along the original base elevation of the toe as a line of exposed bedding could be seen along the entire length of the test section. A surveyed cross section and the estimated original toe section are shown in Figure 2b.

Test No. 3. This toe was constructed as a 12 inch thick horizontal layer placed on the channel bed. The original width of the blanket was estimated at 18 ft. instead of 12 ft. as shown in the contract plans. The channel bed in the vicinity of this toe had degraded approximately 6 ft. The rock had moved downward to cover the lower slope. However, the thickness would average somewhat less than one rock diameter as small areas of exposed sand could be seen over the entire face of the toe. There is a substantial reserve of rock remaining in the toe. However, if additional degradation occurs too rapidly, partial failure could occur due to separation of the thin blanket. High flows for an extended period of time could also cause severe leaching of bank material through the sparse blanket. On the other hand, it is possible that recession of the bank by leaching could cause additional rock to move downward from the remaining portion of the toe and form a thicker bank covering. A surveyed cross section and the estimated original toe section are shown in Figure 3a. A photograph showing its appearance is presented in Figure 4.

Test Section No. 4. This toe was constructed as a $2\frac{1}{2}$ ft. thick horizontal layer of rock placed on the channel bed. The original width of the blanket was estimated at 10 ft. instead of 7 ft. as shown in the contract plans. Approximately 5 ft. of degradation had occurred in the vicinity of this test section. There was a uniform blanket of rock on the lower slope that would average at least two rock diameters thick. There was no evidence of rock separation or areas of exposed bank material. The riverward edge of the rock was essentially at the base of the slope with very little migration of rock out into the channel. A substantial reserve of rock still remained which could provide material for additional degradation. A surveyed cross section and the estimated original section are shown in Figure 3b. A photograph showing its appearance is presented in Figure 5.

CONCLUSIONS

The performance of the prototype test sections generally confirmed the results observed in the model tests. An extension of the slope protection

shows evidence of stress soon after the base of the revetment is undercut. The enlarged section at the base of the bank protection performs reasonably well. However, it appears to have a tendency to eventually separate at the point where the steeper back slope of the toe intersects the flatter slope of the bank protection. A thin horizontal blanket does not release rock at a rate sufficient to provide an adequate thickness of coverage. The performance of the thicker and narrower horizontal blanket was clearly superior to that of the other 3 test sections. The model tests by the University of Nebraska indicated that only a single layer of rock would form as the toe rock moved downward. The model tests also indicated that some channelward movement of rock would occur and form a horizontal apron at the base of the slope. The prototype performance of the thick horizontal blanket indicates that a layer several rock diameters thick can develop under actual field conditions. There was also much less movement of rock channelward at the base of the slope than indicated by the model test. Apron formation in the model may have been the result of the relatively large dune pattern in the model bed. In the model tests at both Iowa City and Mead, the height of these dunes would approximate one-fourth of the water depth. In the Iowa City model studies, the author observed that the turbulence associated with the movement of these dunes was very severe and would remove and scatter some of the rock at the base of the revetment. In the Milford outlet channel, the bed at the time of the inspection was essentially flat and even during high flows it is doubtful if dunes of significant height existed. Final side slopes in the model were approximately 1V on 2H for all toe geometries. In the prototype test sections the slope below the horizontal toes was approximately 1V on 2H while that of the slope extension and enlarged base was about 1V on $1\frac{1}{2}$ H.

For conditions of parallel flow, as in the Milford outlet channel, the horizontal blanket toe geometry provided an economical and efficient method of protecting the revetment against damage by undercutting. It appears that a volume of rock equal to $1\frac{1}{2}$ times the volume required to extend the slope protection to the expected depth of degradation is sufficient for parallel flow conditions. The thickness of a horizontal blanket should probably be at least twice the maximum rock diameter. One cannot definitely conclude that toe protection of this type will work as well under all flow conditions. In a situation where the flow is directly against the bank with severe scour occurring at the base of the slope, the local degradation might be too rapid for the reservoir of toe rock to form an effective blanket. However, if the toe rock is sufficiently heavy so that it is not removed by the flow and relatively large quantities of rock are released, it may provide sufficient protection to prevent complete failure of the revetment. Model and prototype tests under such conditions could provide valuable additional information.

REFERENCES

Roumph, Robert F., August 1970, "Performance of Riprap Toe Structures with Channel Degradation," unpublished Master's Thesis, University of Nebraska Department of Civil Engineering.

TABLE 1

SUMMARY OF DATA ANALYSIS
UNIVERSITY OF NEBRASKA MODEL STUDIES

Run Number	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Running Time, hours	14.5	0.35	0.42	0.53	-----	-----	-----	5.10	90	39	6.56	70	Good	Poor coverage at the bottom of the slope in the upstream reach of the channel-downstream slope coverage was good
Average Maximum Degrad., ft.	0.44	0.50	0.65	-----	-----	-----	0.0189	7.48	90	18	8.06	83	Good	Good coverage on the entire slope throughout the length of the channel
Dist. to Lowest Prot., ft.	0.69	0.78	0.97	1/1.99	-----	-----	0.0189	7.48	100	57	11.77	64	Very Poor	Severe weakening of coverage first appeared at the original bed elevation-as degradation continued, the strip of pool coverage enlarged downward.
Water Surface Slope, ft./ft.	0.20	0.32	-----	-----	-----	-----	0.0209	9.51	43	0	4.12	100	58	Coverage was weak on the entire slope
Lower Bank Slope, ft./ft.	0.73	0.81	0.87	1/1.96	-----	-----	0.0209	9.51	82	2	8.01	98	76	Poor protection on the bottom of the slope in the upstream reach and top of the slope in the downstream reach.
Maximum Degradation, ft.	0.58	0.70	0.75	1/2.00	-----	-----	0.0237	7.26	100	75	12.68	57	55	Initial weakening of coverage occurred at the change in slope-as degradation continued, middle of slope coverage became weak
Distance to Lowest Protection, ft.	0.69	0.82	0.92	1/2.02	-----	-----	0.0250	7.48	99	55	11.50	64	83	Weakening of coverage first occurred along the steepened back side of the toe.
Lower Bank Slope, ft./ft.	0.68	0.80	0.84	1/2.00	-----	-----	0.0228	7.52	61	78	10.45	44	77	Poor coverage on the upper slope at about the original bed elevation.
Water Surface Slope, ft./ft.	0.72	0.81	0.93	9.93	-----	-----	-----	8.56	100	33	11.34	75		

The data for these runs were not analyzed.

DEFINITION OF COLUMN HEADINGS

Column (1)	Run Number	Column (8)	Toe Area - Cross-section area of the toe before degradation.
Column (2)	Running Time - Duration of time the model was operated during the run.	Column (9)	Toe Area Used - Percent of the toe area consumed in the degradation process.
Column (3)	Average Degradation - Distance between the average bed elevation and the original bed elevation.	Column (10)	Slope Riprap Used - Quantity of slope riprap consumed in the degradation process expressed as a percentage of the toe area.
Column (4)	Maximum Degradation - Distance between the lowest bed elevation and the original bed elevation.	Column (11)	Total Riprap Used - Sum of the toe area used and slope riprap used.
Column (5)	Distance to Lowest Protection - Distance between bottom of slope coverage and the original bed elevation.	Column (12)	Riprap Supplied by Toe - Ratio of slope riprap used to total riprap used.
Column (6)	Lower Bank Slope - Slope (y/x) of the portion of the bank affected by degradation.	Column (13)	Weakest Slope Coverage - Weakest coverage on any area of the bank larger than several square inches.
Column (7)	Water Surface Slope - Average slope of the water surface during the run as obtained from the piezometric tubes.	Column (14)	Location of weakest slope coverage.

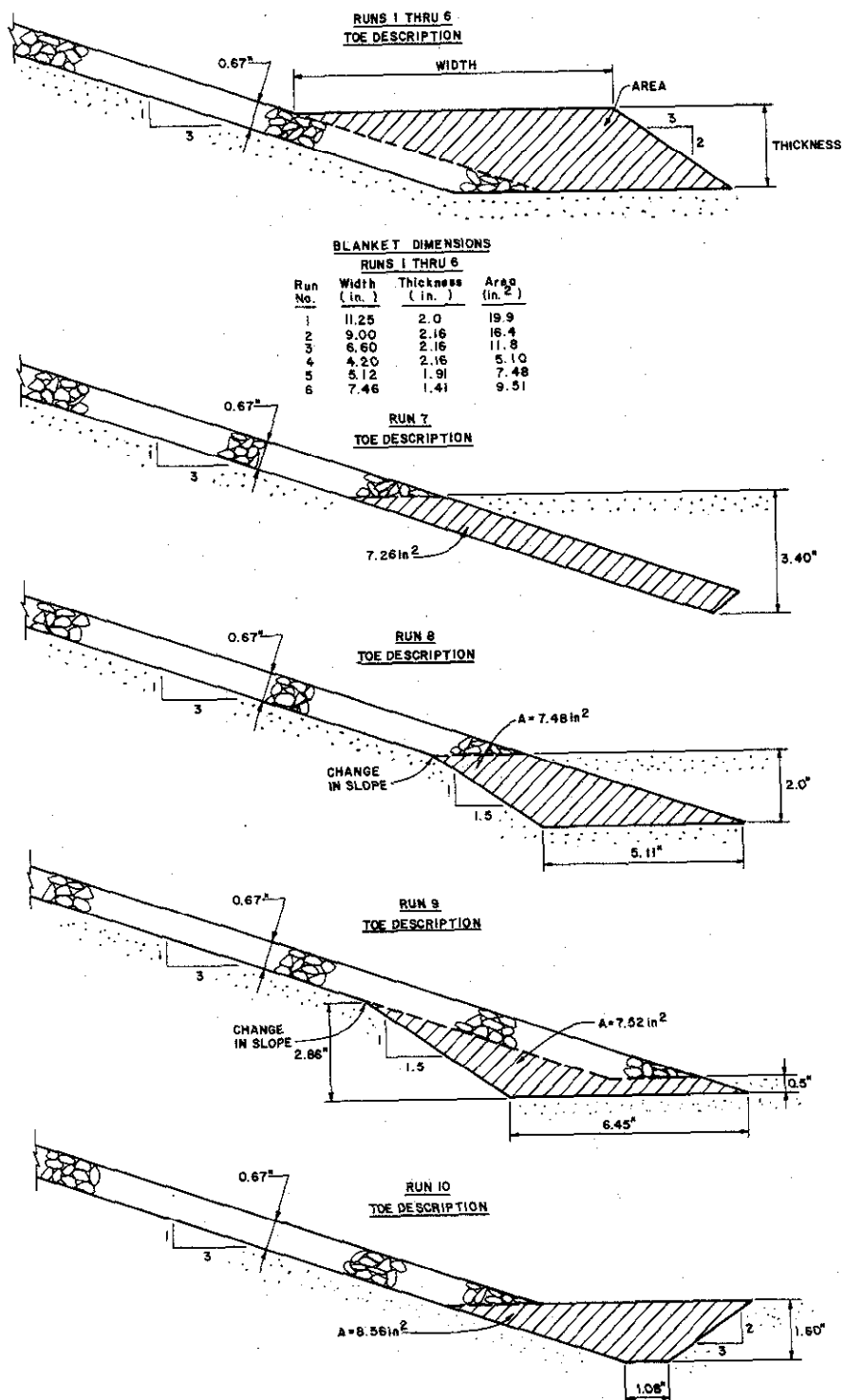


FIGURE 1—TOE GEOMETRIES MODEL TESTED BY THE UNIVERSITY OF NEBRASKA

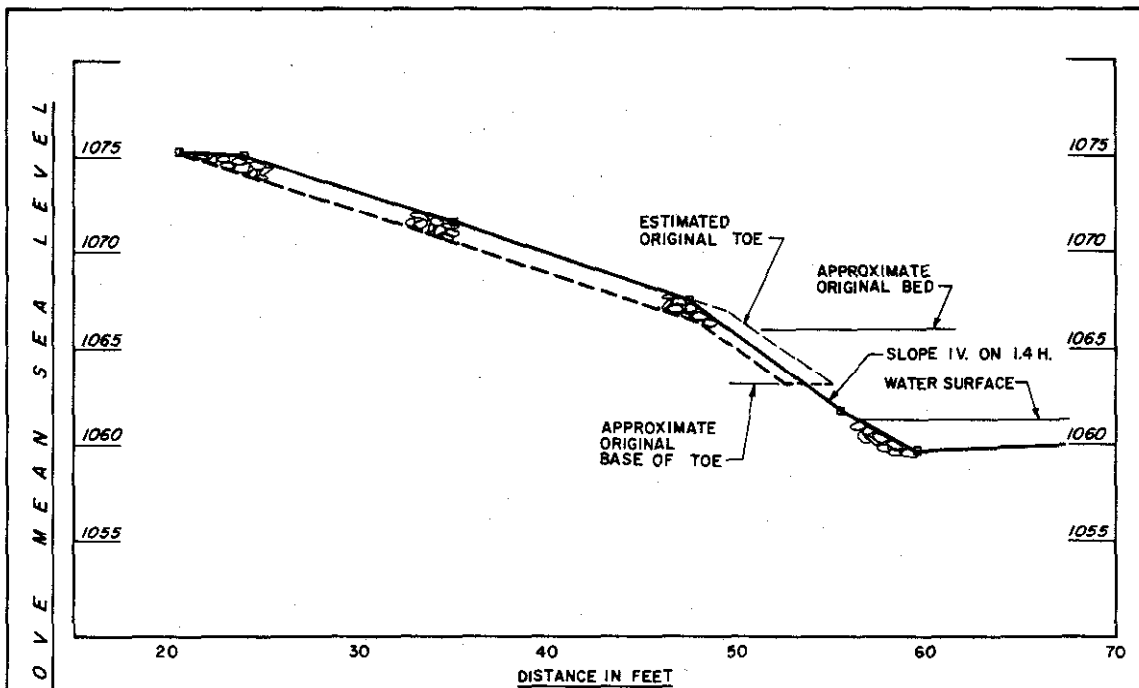


FIGURE 2a-REVTMENT TOE TEST SECTION NO. 1 STA. 80+00 R.

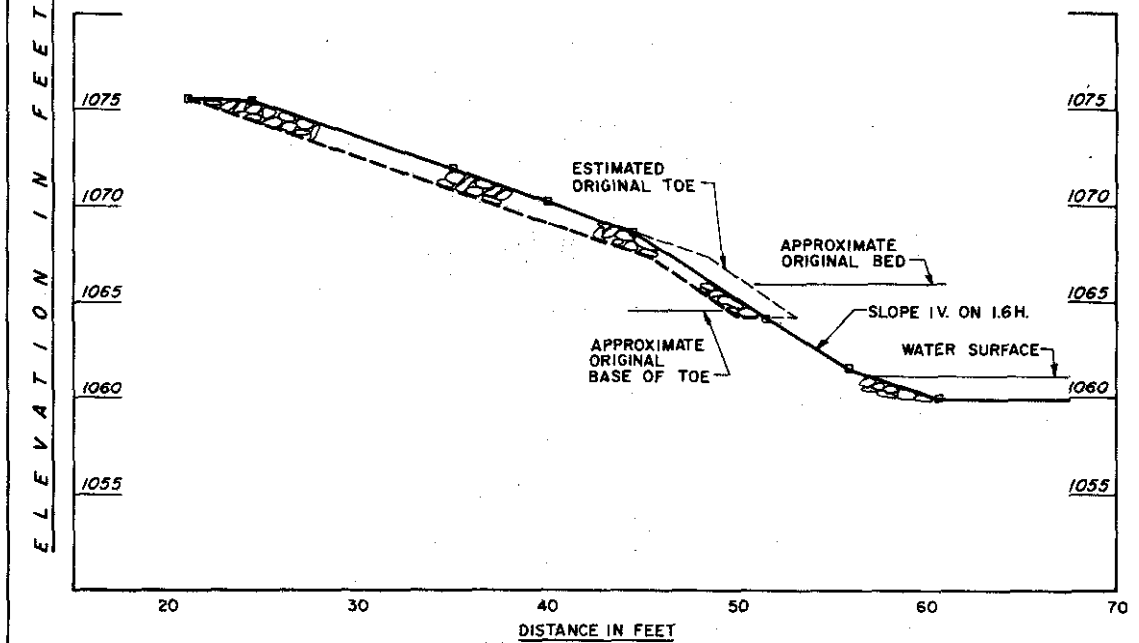


FIGURE 2b-REVTMENT TOE TEST SECTION NO. 2 STA. 83+00 R.

FIGURE 2 - MILFORD OUTLET CHANNEL PROTOTYPE
REVTMENT TOE TEST SECTIONS

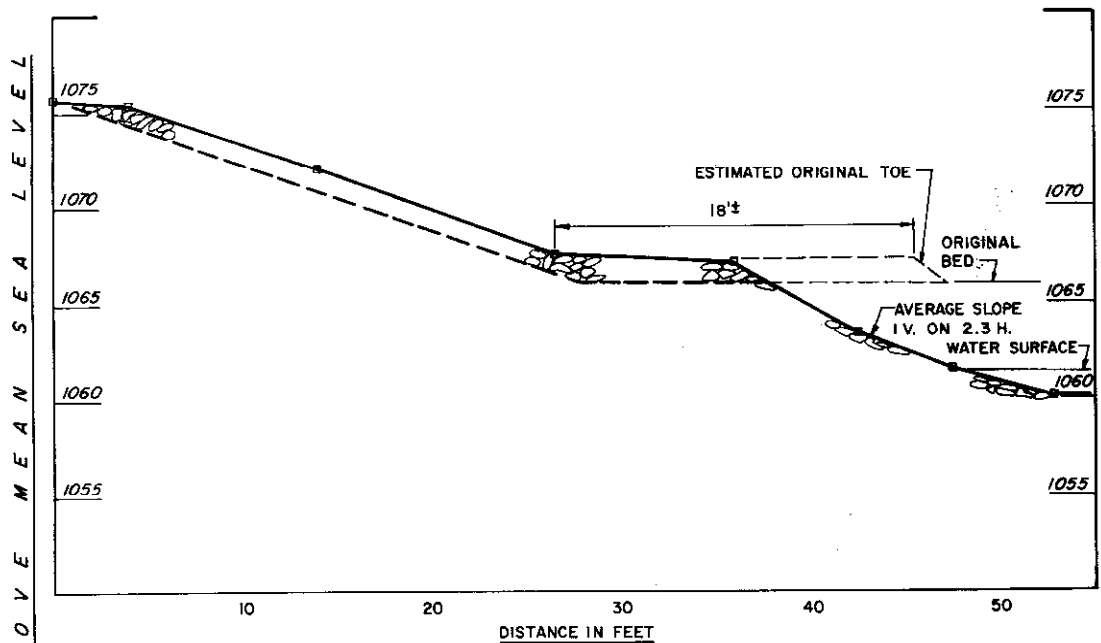


FIGURE 3a-REVTMENT TOE TEST SECTION NO. 3 STA. 80+00 L.

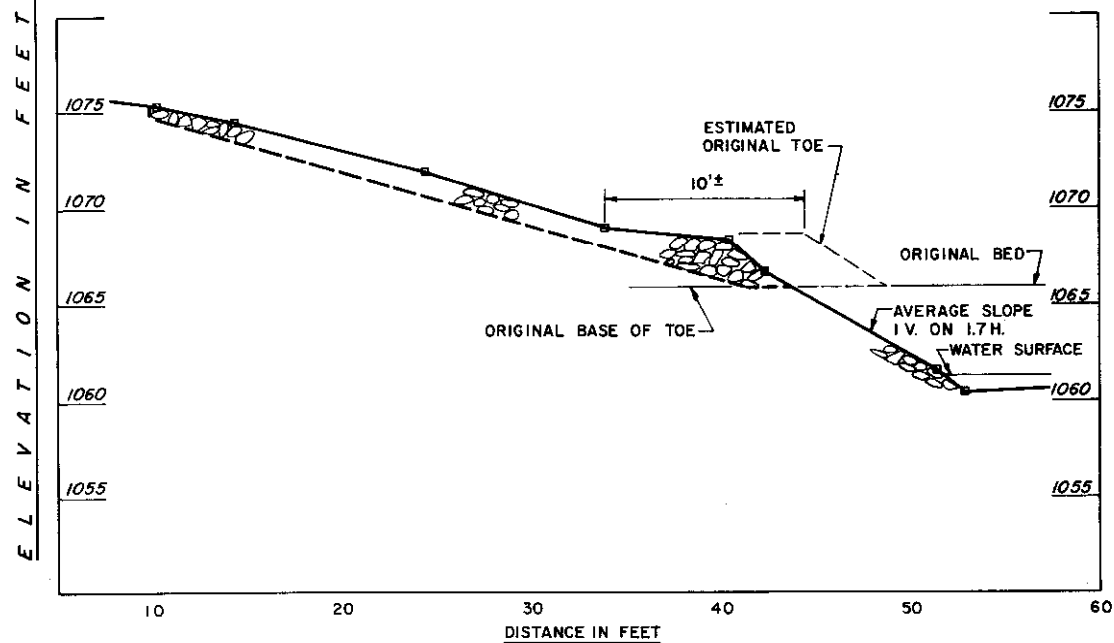


FIGURE 3b-REVTMENT TOE TEST SECTION NO. 4 STA. 83+00 L.

FIGURE 3 - MILFORD OUTLET CHANNEL PROTOTYPE
REVTMENT TOE TEST SECTIONS

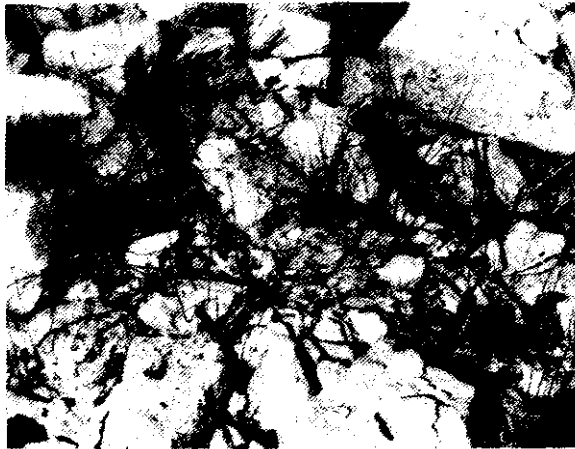


FIGURE 4
SLOPE PROTECTION FORMED BELOW THE THIN
HORIZONTAL BLANKET. NOTE SPARSE ROCK
COVERAGE WITH AREAS OF EXPOSED SAND.



FIGURE 5
VIEW ALONG SLOPE PROTECTION BELOW THE
THICK HORIZONTAL BLANKET. SURVEY ROD
SHOWS LOCATION OF THE BASE OF THE TOE

CONTROL OF TURBIDITY AT CONSTRUCTION SITES

By E. J. Carlson, Research Hydraulic Engineer, U.S. Bureau of Reclamation, Division of General Research, Engineering and Research Center, Denver, Colorado

ABSTRACT

A study team was organized at the Bureau of Reclamation Engineering and Research Center, Denver, Colorado, to collect and disseminate information on Control of Turbidity at Construction Sites following the requirements of Public Law 92-500. A summary of most of the areas in which information was assembled is included in the report. A statement of policy on water pollution control in the United States by the Board of Control of the Water Pollution Control Federation is given. Public Law 92-500 and two Executive Orders from the Office of the President on pollution control are reviewed. The National Pollution Discharge Elimination System (NPDES) is described. Turbidity measurement and causes of turbidity at construction sites are described. An example of analysis of turbidity data obtained with typical turbidity measuring instruments and the accuracy obtained is given. Some typical variations of bid prices for control of turbidity at typical Bureau of Reclamation projects are listed. The variation of turbidity standards in the Western United States is discussed.

INTRODUCTION

Over the past several years national concern has been focused on the problem of pollution of all our national resources. A review of construction as related to control of turbidity in effluents to streams from construction activities is particularly timely now during the early stages of this general awareness. Water pollution control is being sought by various state and Federal agencies and by the profession working with water resources development within the United States. This new public awareness is felt in the areas of communication media, state and Federal regulations, and the political arena. Water is one of our most precious and irreplaceable resources, and it is under close scrutiny.

The following statement of policy on water pollution control in the United States was adopted by the Board of Control of the Water Pollution Control Federation on October 3, 1971. 12/* It gives the extent to which the public has become interested in control of water pollution including turbidity and for that reason it is included here.

* Numbers refer to references at the end of the paper.

STATEMENT OF POLICY ON WATER POLLUTION CONTROL IN THE U.S.
Adopted by the Board of Control of the Water Pollution Control
Federation - October 3, 1971

This Statement of Policy was adopted originally in 1960. Since then, revisions have been made to keep abreast of expansion and changes in the water pollution control field.

Water pollution means water quality damage and consequent interference with beneficial use of a vital resource - clean water.

Pollution of the Nation's inland surface waters, coastal waters, and ground waters is a continuing threat to the national health, esthetic enjoyment, safety, and economic welfare. National survival, in terms of future urban, industrial, and commercial growth and prosperity, dictates the protection of all water resources from any acts, such as the discharging of harmful substances which cause unreasonable impairment of water quality and adversely affect their highest level of usefulness. While considerable progress has been made in pollution control by municipalities and industries, many water resources are being degraded, impaired, and damaged by such discharges and acts, and they will be further adversely affected by the degree and pattern of population growth, industrial processing, commercial expansion, chemical usages, agricultural developments, and other technological advancements.

The Water Pollution Control Federation is pledged to provide leadership and guidance to all constructive efforts that contribute to the control of water pollution. Its pledge is summarized by the following points:

1. The discharge of all wastewater into the waters of the Nation must be controlled.
2. The objectives of water pollution control must include preservation of high quality waters for protection of public health; for industrial, agricultural, and recreational uses; for fish and wildlife propagation; and for the maintenance of an esthetically desirable environment.
3. The responsibilities for the adequate treatment and control of wastewater must be assumed individually and jointly by industry and local, state, interstate, and Federal governments.
4. The administration of water pollution control must be firm and effective and should remain in the hands of state and interstate water pollution control agencies. Regulatory agencies must be supported by adequate budgets and fully staffed by competent engineers, scientists, and supporting personnel.
5. Federal, state, and local laws and practices must reflect the changing needs in order to obtain and maintain the most economical

and effective means for financing the construction, management, operation, and maintenance of wastewater collection systems and treatment works.

6. The public must be made fully aware of the consequences of water pollution and the costs of its control. Only in this way can the public be prepared to sponsor and support sound water pollution control measures.

7. Basic and applied research by competent personnel must be encouraged by broad efforts to develop new knowledge that will solve water pollution problems.

8. Wastewater represents an increasing fraction of the Nation's total water resources and should be reclaimed for beneficial reuse. To this end the development and application of methods for wastewater reclamation must be accelerated.

9. Mandatory certification or licensing of adequately trained and properly compensated personnel must be encouraged as a requirement for maximum effectiveness of treatment facilities.

PUBLIC LAW 92-500

Public Law 92-500, 15/ 92d Congress, S. 2770, October 18, 1972, which is the amendment to the Federal Water Pollution Control Act, states * * * "It is the national goal that the discharge of pollutants into navigable waters be eliminated by 1985:". Navigable waters are deemed to mean all streams for the purpose of this report. In the interim, before 1985, the Environmental Protection Agency (EPA) and the states have established standards of turbidity that an effluent can have which is passed into a stream. The states are reviewing their laws and going through the process of updating their standards to be in line with Federal Government recommendations.

Public Law 92-500 is complex and is difficult to get implemented because it requires many changes of present practices that are causing pollution in streams, including discharge of sediment from construction operations. Implementation of Public Law 92-500 has caused much discussion and many meetings because of its effect on so many operations and people. The cost of making the changes to comply with the law will require very large expenditures much of which must come from Federal funds. The procedures for obtaining the funds and doing the work in compliance with the law are not completely clear. The art and its application have not, in the main, begun to approach public expectations. Revisions of the art are needed and changes in the manner of its administration are required. It seems that there is a need to utilize more effectively, in harmony with Federal responsibility, the knowledge and experience of state and local officials and particularly of members of the engineering profession.

PRESIDENTIAL DOCUMENTS

Two Presidential documents under Title 3 - The President, were issued in the area of pollution control. Executive Order 11738 8/ provides for administration of the Water Pollution Control Act with Respect to Federal Contracts, Grants, or Loans; and Executive Order 11752 9/ concerns Prevention, Control, and Abatement of Environmental Pollution at Federal Facilities.

The goals of Public Law 92-500 and the Executive Orders above are consistent with the desires of the public. The means of obtaining these goals in an orderly and responsible manner will require long and arduous efforts on the part of private and public business, individuals, and the courts.

TURBIDITY 10/

Turbidity in water is caused by the presence of suspended matter such as clay, silt, very fine sand, finely divided organic and inorganic matter, plankton, and other microscopic organisms. Turbidity should be clearly understood to be an expression of the optical property of a sample which causes light to be scattered and absorbed rather than transmitted in straight lines through the sample. Turbidity should be distinguished as entirely different from a weight concentration of suspended matter. Although attempts have been made to correlate turbidity with weight concentration of suspended matter, these correlations are for the most part impractical as size, shape, color, and refractive index of particulate materials are of great importance optically, but bear little direct relationship to the concentration by weight and specific gravity of the suspended matter. Instruments have been made that can indicate turbidity of grab samples very quickly and that can indicate turbidity of the stream continuously at a much less cost than determining weight concentrations of suspended solids. For this reason, some turbidity reading instruments (nephelometric, light scatter method) have been adopted as standard instruments for determining turbidity. It should be emphasized that instruments based on the light scatter principle do not reflect a weight concentration of suspended solids.

The standard method for determination of turbidity has been based on the Jackson Candle Turbidimeter. However, the lowest turbidity value which can be measured directly on this instrument is 25 Jackson turbidity units. With turbidity requirements of the states falling very near or below this value, indirect secondary methods have been required to estimate turbidity on such samples. The Jackson Candle Method is rather cumbersome and difficult to use particularly in field tests. Consequently, many companies have developed instruments which are indirect methods for estimating turbidity. Unfortunately, no instrument has been devised which will duplicate the results obtained on the Jackson Candle Turbidimeter for all samples. Owing to fundamental differences in optical systems, the results obtained with different types of secondary

instruments will frequently not check closely with one another even though the instruments are all precalibrated against a candle turbidimeter.

A further cause of discrepancies in turbidity analysis is the use of suspensions of different types of particulate matter for the preparation of instrumental calibration curves. Prepared suspensions have different optical properties depending upon the particle size, distribution, shapes, colors, and refractive indices. Most commercial turbidimeters available for measuring low turbidities give comparatively good indications of intensity of light scatter in one particular direction predominantly at right angles to the incident light. Since there is no direct relationship between Jackson Candle Turbidity and the intensity of light scatter at 90° , there is no valid basis for calibrating the 90° turbidimeters in terms of candle units.

Some turbidimeters have been developed that use a forward light-scattering technique to give indications of turbidity. A sealed submerged sensor head contains two separate light sources and detectors. The intensity of a direct collimated beam of white light directed through the sample or continuous stream is measured by the direct beam detector. The light scattered in the forward direction by turbidity in the sample is measured by the scattered beam detector. Output signals from the two detectors are then electronically ratioed to give the turbidity measurement. Manufacturers of the forward scatter instrument claim that it eliminates the effects of color in the sample, source light, and sample bottle or continuous-tube characteristics providing accurate and repeatable turbidity measurements. The forward scatter instrument is a much more complicated design and consequently more costly than the 90° light scatter instruments. Forward scatter instruments have not come into as much use as the 90° light scattering instruments for measuring turbidity caused by suspended sediments. Descriptions of the visual method (Jackson Candle turbidimeter) and the nephelometric method (light scatter in one direction) are given in several references. 2/ 3/ 7/ 11/ 13/ 16/

ANALYSIS OF TURBIDITY DATA - AN EXAMPLE

Grab samples were collected over a period of several weeks from the Arkansas River at Pueblo damsite. The samples were analyzed by three methods to determine turbidity and concentration of suspended sediment. Two commercial turbidity meters were used. One was calibrated with a Formazin standard to read in Jackson Turbidity Units. The second, a so-called Infra Red Direct Reading instrument, was calibrated to read in milligrams per liter (p/m). The third method of analysis was to filter the suspended solids and weigh the dried residue to determine the suspended sediment concentration in milligrams per liter (p/m). The drying and weighing method was taken as the standard in comparing results obtained from the two turbidity measuring instruments. The three different methods of measurement were made from the same sample in every case. Sixty-two samples were used from which the measurements and the analyses were made. Three samples which had extra large concentrations were excluded because of the great effect

they would have on the analysis and because the majority of turbidity measurements were less than 100 milligrams/liter. The three extra large concentrations were 367 to 1,172 milligrams per liter.

Linear least squares fit were made using the dried weight method as the independent variable and the two data sets from the turbidity instruments measurements as dependent variables.

The standard deviation was computed for each set of turbidity meter data. This is our indication of the accuracy and repeatability of the turbidity reading instruments for samples taken from one stream in one general location. A standard deviation of 6.0 milligrams per liter was computed for the first meter calibrated to read in Jackson Turbidity Units. For the second, direct reading, instrument a standard deviation of 10.4 milligrams per liter was computed. Standard deviations values were computed for different groupings of the data along the X axis. Higher and lower values than those for all data were computed, but the average values were very close to the standard deviations computed with all 62 data values.

Confidence limits were computed for the least squares fit lines for the turbidity meter data. Computations were then made to determine the number of samples that would be required to obtain accuracies of plus or minus 2, 3, and 5 milligrams per liter. For example it was shown that seven samples would be required to obtain an average within ± 5 milligrams per liter at a confidence level of 95 percent. The number of samples required rose to 31 for an average accuracy within ± 2 milligrams per liter at a confidence level of 95 percent.

This analysis gives some indication of the accuracy of two specific turbidity meters when compared to measurement of low concentrations of sediment by the drying and weighing method.

A survey made in 1973 showed there were over 25 companies making turbidity measuring instruments which varied in price from about \$525 to over \$4,000. Three typical continuous reading turbidity meters and one grab sample meter are presently being tested in the Hydraulics Laboratory of the Bureau of Reclamation at Denver, Colorado.

CONSTRUCTION ACTIVITIES THAT CAUSE TURBIDITY PROBLEMS

A list of construction activities that may cause turbidity problems are shown here. There may be many other items for other types of construction. For Bureau of Reclamation projects the list covers the major turbidity producing activities.

1. Excavation for the foundation of diversion, earth fill, and concrete dams, including handling of wet earth materials
2. Downstream channelization and excavation of river valley materials

3. Construction of diversion tunnels for dams
4. Processing natural or quarry materials for concrete aggregate.
5. Construction of tunnels for water conveyance systems (treatment of tunnel water and mud, hauling of waste materials, etc.)
6. Operation of concrete mixing plants
7. Construction wastewater for concrete dams (water for washing, curing, and cleanup)
8. Grouting
9. Construction of canals, temporary and permanent roads, and construction camps
10. Construction of siphons or other water conveyances across drainage channels
11. Reservoir clearing

These activities are separated into specific work processes that may require different technologies for turbidity or sediment control. For example, it may be desirable to utilize settling ponds or Clar-flocculators to settle out fines during excavation for the foundation of the dam. However, a similar technology would not be used when a reservoir area is cleared or a road built into a construction site.

NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM (NPDES)

The Federal Water Pollution Control Act, as amended by Public Law 92-500, enacted October 18, 1972, requires a permit to discharge a pollutant into a waterway from one or more point sources. Federal departments, agencies, and instrumentalities are also subject to these requirements.

There are two sets of (NPDES) application forms to be used. Short forms (A-D) 5/ and standard forms (A-C).6/ The owner and operator of any activity or wastewater system, publicly or privately owned, which discharges waste into a waterway must obtain a permit for such discharges.

To ensure that the administering office has sufficient time to examine applications from new sources of discharge of pollutants, a complete NPDES application must be filed (1) no less than 180 days in advance of the date on which it is desired to commence the discharge of pollutants, or (2) in sufficient time prior to commencement of the discharge of pollutants to insure compliance with the requirements of the act.

For a contractor-owner arrangement of a construction project, the contractor is usually the operator during the construction period after the

contract has been awarded. However, in preparation of plans and specifications for a construction project, the owner should consult with appropriate EPA and state offices to assure that water pollution control specifications paragraphs provide adequate guidance for the contractor to meet the pertinent water quality standards in the area which will be affected by the contractor's activities. It may be appropriate for the owner to apply to the EPA for a (NPDES) permit. The application assumes that the construction will create a definite pollutant discharge. Instructions printed in the Federal Register (Vol. 37, No. 247 - Friday, December 22, 1972) indicate the permit to discharge could be transferred from the owner to the contractor during the actual construction period. Additional paragraphs of explanation in the specifications may be necessary as the particular situation requires.

For contracts and construction that would make 180 days notice in applying for a permit to discharge impossible, a temporary permit or verbal permission could possibly be obtained from the EPA or state director as the case dictates.

FLOCCULANTS AND COAGULANT AIDS

Various chemicals have been used for many years to assist in removing suspended solids from water. They have been used in connection with sanitary water supply and sewage treatment systems. In recent years many chemical companies have developed many different flocculants and coagulant aids that can assist in removing fine sediments from water. Some of them are made from polymer-type chemicals and are specified for use depending on whether the water and sediment is ionic or cationic.

The Environmental Protection Agency compiled a list of companies and the flocculants and coagulant aids that they make. EPA emphasizes that its findings bear only on the health aspects of the use of the chemicals in drinking water treatment and do not endorse nor indicate effectiveness for the proposed use. The list comprises 206 separate product names made by 46 companies.

TURBIDITY STANDARDS IN WESTERN STATES

To administer Public Law 92-500 each of the states must have standards by which they will allow owners and operators to discharge pollutants into streams. The standards are set by the states to be equal or better than recommendations made in Public Law 92-500 and administered by the Environmental Protection Agency.

An investigation to determine the pollution control standards in streams was made in the Western part of the United States, the area of operation of the Bureau of Reclamation. Most of the states had some standards for discharging into their streams before passage of Public Law 92-500. To come within the recommendations of EPA, the Western states are generally

changing their standards. Some states specify different standards for different streams, lakes, and in some cases bays in the ocean. The differences in standards between states are so great they are too extensive to list within the space allowed here. Whenever an owner or contractor plans construction in a different state from where he is familiar, a thorough investigation should be made of the turbidity standards for the particular state where work is planned.

TURBIDITY CONTROL DURING CONSTRUCTION ON TYPICAL BUREAU OF RECLAMATION PROJECTS

Several Bureau projects have been started and some have been completed since Public Law 92-500 was enacted on October 18, 1972. In the specifications for each construction project, paragraphs are provided on "Prevention of Water Pollution and Control of Turbidity." The paragraphs give information that is adequate for contractors to bid this item and plan construction and satisfy the requirements for monitoring and maintaining pollution control. In most cases suggested methods for pollution control of effluents returning to streams and also methods for monitoring the streams are given in the specifications. The contractor is always directed to comply with applicable Federal and state laws that apply to pollution control. A contingency plan is required in the case of an emergency spill or discharge of a highly turbid effluent.

Some contractors have been very innovative in devising methods of controlling water pollution. As experience is gained under the new law it is expected that costs for controlling turbidity at construction sites will be reduced. Bid prices for the line item, Prevention of Water Pollution and Control of Turbidity, over the past few years has varied widely. Some of the variations for typical Bureau projects are given here:

Bid Variations for Prevention of Water Pollution and Control of Turbidity Typical Projects

Project	No. of bidders	Variation of bids	
		Low	High
Teton Dam, Idaho	6	\$150,000	\$ 925,000
Pueblo Dam, Colorado	8	200,000	1,000,000
Auburn Dam, California			
Excavation and Foundation Treatment	9	10,000	150,000
Crystal Dam Diversion and Foundation			
Tunnels	10	50,000	396,000
Cunningham Tunnel	9	100,000	547,112

The wide variation is due to the fact that in most cases these jobs are the first time the contractor has bid an item for control of turbidity and water pollution. It is expected that the bids will generally be in line and somewhat reduced when experience is gained and pollution control equipment becomes more readily available.

LIST OF REFERENCES

1. American Public Health Association, "Standard Methods for Examination of Water and Water Waste," 13th Edition, 1971
2. ASTM Designation: D1889-71, "Standard Methods of Test for Turbidity of Water"
3. ASTM Designation: D1888-67, "Standard Methods of Test for Particulate and Dissolved Matter in Industrial Water"
4. Environmental Protection Agency, "Clean Water, Methods for Chemical Analysis of Water and Wastes, 1971
5. Environmental Protection Agency, "National Pollutant Discharge Elimination System, Application for Permit to Discharge - Short Form 7550-8 (1-73)"
6. Environmental Protection Agency, "NPDES - Standard Form 7550-23A (7-73)"
7. Eichner, Don, and Hach, Clifford C., "The Absolute Turbidity of Pure Water," Hach Chemical Company, Ames, Iowa, 1971
8. Federal Register, Vol. 30, No. 176, September 12, 1973, Executive Order 11738
9. Federal Register, Vol. 38, No. 243, December 19, 1973, Executive Order 11752
10. Hach, Clifford C., "Principles of Surface Scatter in Light Scattering (Nephelometry)," Hach Chemical Company, Ames, Iowa, 1971
11. Hach, Clifford C., "Introduction to Turbidity Measurement," Hach Chemical Company, Ames, Iowa, 1971
12. Journal of Water Pollution Control Federation, Volume 45, No. 3, March 1973, "Yearbook"
13. Monitor Technology, Inc., "Monitek Industrial Turbidimeters," 303 Convention Way, Redwood City, California

14. McCullough, Charles A., and Nichlew, Robert R., "Control of Water Pollution During Dam Construction," Journal of Sanitary Engineering Division, SAI, ASCE, February 1971
15. 92d Congress, Public Law 92-500, S2770, October 18, 1972, cited as "Federal Water Pollution Control Act Amendments of 1972"
16. Tool, H. Raymond, "Suspended Solids Determinations Using a Filter Photometer," Hach Chemical Company, Ames, Iowa, 1971
17. U.S. Bureau of Reclamation, Hydraulic Laboratory Report No. Hyd-405, "Progress Report No. 1: Preliminary Investigations in Hydraulic Laboratory of Flocculating Agent No. 1," December 1955
18. U.S. Bureau of Reclamation, Hydraulic Laboratory Report No. R-Hyd-6, "Progress Report No. II - Preliminary Investigations in Hydraulic Laboratory of Flocculating Agents"
19. Water Newsletter, March 6, 1972, Volume 14, No. 5