NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE **70**

DESIGN OF SEDIMENTATION BASINS

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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DESIGN OF SEDIMENTATION BASINS

RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

SUBJECT AREAS
- HYDROLOGY AND HYDRAULICS
- ENVIRONMENTAL DESIGN
- CONSTRUCTION

MODES
- HIGHWAY TRANSPORTATION
- PUBLIC TRANSIT
- RAIL TRANSPORTATION
- AIR TRANSPORTATION
- OTHER

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. JUNE 1980

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the National Academy of Sciences, or the program sponsors.

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The Transportation Research Board evolved from the 54-year-old Highway Research Board. The TRB incorporates all former HRB activities and also performs additional functions under a broader scope involving all modes of transportation and the interactions of transportation with society.

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PREFACE

There exists a vast storehouse of information relating to nearly every subject of concern to highway administrators and engineers. Much of it resulted from research and much from successful application of the engineering ideas of men faced with problems in their day-to-day work. Because there has been a lack of systematic means for bringing such useful information together and making it available to the entire highway fraternity, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize the useful knowledge from all possible sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series attempts to report on the various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which they are utilized in this fashion will quite logically be tempered by the breadth of the user's knowledge in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of interest to planners, engineers, designers, and contractors who must deal with the problem of sediment-laden runoff from construction sites. A comprehensive discussion of sedimentation basins, their design, and their best use is presented.

Administrators, engineers, and researchers are faced continually with many highway problems on which much information already exists either in documented form or in terms of undocumented experience and practice. Unfortunately, this information often is fragmented, scattered, and unevaluated. As a consequence, full information on what has been learned about a problem frequently is not assembled in seeking a solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of synthesizing and reporting on common highway problems. Syntheses from this endeavor constitute an NCHRP report series that collects and assembles the various forms of information into single concise documents pertaining to specific highway problems or sets of closely related problems.

All highway construction disturbs the land to some extent and thus entails the risk of excess sediment running off during storms. This report of the Transportation Research Board deals with the design, placement, and use of three types of sedimentation basins—expedient, temporary, and permanent. Each type of basin should be tailored to a specific situation and its associated risks. For example, an expedient basin might be dug one afternoon to receive runoff from a predicted night storm and then be filled in the next day.

Establishing the need for a sedimentation basin precedes its planned placement. Its basic design parameters need to be based on characteristics of local drainage, anticipated precipitation, settling rates, and so forth. This synthesis covers all these aspects plus the disposition, construction, and maintenance of dams and spillways.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researchers in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

CONTENTS

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Lawrence F. Spaine, Engineer of Design, Transportation Research Board, assisted the Special Projects Staff and the Topic Panel.

Information on current practice was provided by many highway and transportation agencies. Their cooperation and assistance were most helpful.

DESIGN OF SEDIMENTATION BASINS

SUMMARY

Sedimentation basins are useful for minimizing the effects of highway construction runoff on the environment and are often used in conjunction with other sediment-control practices. A sedimentation basin protects streams, lakes, recreation areas, and other areas that cannot tolerate sediment deposition.

There are three types of sedimentation basins—expedient, temporary, and permanent. Expedient basins are quite small, exist for only a short time (possibly only one night), and their sites are determined by the engineer during grading operations. Temporary basins remain in place for the entire duration of a construction project or until the need for them clearly has passed. Their locations are usually shown on the plans, and they are built to a higher standard than expedient basins. Permanent basins are used to intercept sediment during construction but remain after construction for other uses such as recreation, scenic enhancement, floodwater detention, or groundwater recharge.

There are two design philosophies for sedimentation basins. According to one, a basin shall not discharge during small, frequent runoffs but shall be allowed to discharge during major storms. These basins are designed to trap all sediment (except from major storms) and are cleaned out often, probably after each storm. Adherents to the second philosophy hold that a basin operates as a detention reservoir while sediment is deposited by flow moving slowly through.

Calculation of basin size requires information about the drainage area that includes erosion characteristics, surface cover and condition, and length and steepness of slopes. For the flow-detention philosophy, the designer will also need to estimate the sediment volume to be stored (using a rule of thumb or the universal soil loss equation), determine the percentage of eroded volume that will reach the basin, and estimate the efficiency of the basin in trapping sediment. Then the height of the dam can be calculated and the principal and emergency spillways can be designed.

CHAPTER ONE

INTRODUCTION

Concern about the effects of highway construction on the environment has led to the introduction of methods to limit disturbed soil (sediment) from moving onto adjacent lands or into nearby streams, lakes, and ponds. These methods include selection of the best route, use of less erodible cross-sections, temporary surface treatment, barriers that slow water flow, and sedimentation basins and traps. Many of these erosion-control practices have been described in *NCHRP Synthesis 18* (*1*). One recent manual, *NCHRP Report 221* (*2*), emphasizes surface treatment to control erosion. The material offered in this synthesis was developed from these earlier works but has as its basic assumption that a decision has been made to use sedimentation basins. Thus it deals only with sedimentation basins and their design, construction, and maintenance.

Because *NCHRP Synthesis 18* and *NCHRP Report 221* provide statements of the problem and references to legislation and various administrative decisions and regulations on implementing erosion-control practices, this background material will not be repeated here. Instead, we shall proceed directly to the role of sedimentation basins in sediment control and the design criteria for these basins.

First, how a sedimentation basin protects the environment from the effects of highway construction will be discussed; then how the need for a basin is determined and three different basin types are described. The two basic philosophies for basin design are examined in terms of their respective impacts on the design process. Sensible basin design requires estimates of sediment volume, hydrologic information, and methods for getting this information. These are all described, as are hydraulic calculations for obtaining the sizes of spillways. It was also possible to make some suggestions about the construction, maintenance and cleanout, and disposition of basins.

Each step in the design procedure, presented first in outline form, is described more fully in subsequent sections. These descriptions will assist those who wish to go directly to a particular step for design guidance.

It is assumed that those making hydrologic analyses and hydraulic computations will follow the standards and methods of their respective agencies. However, some hydrologic and hydraulic design criteria contained here as illustrative examples are cited from handbooks and other sources.

This report is based on a study of current practices in erosion and sediment control that are described in various highway agencies' specifications and records of field observations. Of the current literature reviewed for additional information, some deal with theory needed to understand sedimentation phenomena and the intelligent application of the various control methods. This material on theory appears in the appendixes where it will not interfere with the presentation of the practical design and application process. However, the theoretical portion will be referred to, as needed, to explain or to justify the use of an equation or a procedure.

DEFINITION OF A SEDIMENTATION BASIN

A sedimentation basin is a component of an erosion-control system created to protect the environment from the temporary, adverse effects of highway construction. A sedimentation basin is that part of the system that intercepts and retains soil eroded from a construction site. It must protect such sensitive resources as water supply streams and lakes, fish and other aquatic fauna, recreational facilities, and in some cases highly developed land areas subject to overflow but unable to tolerate sediment deposition.

Before construction begins, a basin can be strategically placed to intercept sediment coming from a highway construction site and thus prevent it from entering the streams and lakes below. A sedimentation basin may be the only practical way of providing protection during the clearing and grubbing phase. It is at this time that the short-term or "expedient" basin can be best used.

A sedimentation basin can also augment other sediment-control practices that have been given first consideration. These include selecting routes least disturbing to the terrain and avoiding sites most sensitive to sediment damage; choosing cross-sections with gentle slopes that minimize erosion potential; using mats, mulches, or sprayed-on materials on exposed areas as soon as possible to reduce or prevent erosion; lining channels and protecting drop structures subject to erosion and scour; and following good grading practice.

Yet, despite all these efforts to eliminate sediment production, the possibility remains that sediment will leave the construction area in greater amounts than before the area was disturbed. At this point, one must decide whether this temporary increase in sediment load can be tolerated. This decision will be influenced by current standards on and attitudes toward environmental protection, which vary from place to place and from time to time. Thus no absolute position can be taken here on what is an acceptable level of sediment increase. If an increase cannot be accepted, a sedimentation basin can be considered in order to provide the needed control.

A sedimentation basin can also be the only sediment-control device employed at a site. A comparison in Pennsylvania (*3*) of various erosion-control measures showed offstream ponds that intercept runoff from construction sites to be the most effective of all measures studied in reducing downstream suspended sediment load. What the sedimentation basin is expected to do will determine the type of basin selected to be built.

BASIN TYPES

Basins are classified as expedient, temporary, or permanent. Expedient basins exist for only a very short time, possibly as short as one night, and are constructed during the grading operation. The term "push up" has also been used to describe these basins, because the dams forming them are pushed up by a bulldozer from soil from the basin bottom. Figure 1 is a photograph of an expedient basin on a highway job in New York State.

Expedient basin dams are usually 3 to 5 ft (0.9 to 1.5 m) in height and can be built without benefit of involved design procedures. If the area draining into it is small and if the dam is built with due attention to good construction practice, an expedient sedimentation basin can provide such protection at small cost. The basin can remain until it obstructs construction operations, when it can be replaced by another.

A temporary basin is one that remains operational during the entire construction period or until the need for it has clearly passed. During the lifetime of, say, several years it will be cleaned out as needed and maintained to ensure continued satisfactory performance. Its location must not interfere with construction operations but should be shown on the plan. The temporary basin can take different forms. An excavated temporary basin is shown in Figure 2. Temporary basins, because of their longer lives and probably larger sizes, are built to higher standards than expedient basins. These standards are discussed in subsequent sections.

A permanent basin remains after construction is completed. Its initial use is to intercept sediment. Subsequent uses could be for recreation, scene enhancement, floodwater detention, or groundwater recharge. As these are non-highway uses, the construction of a permanent basin could be a cooperative venture. Most likely the basins will be off the right-of-way. Figure 3 shows a small lake in Pennsylvania that is now a recreational facility but was first a sedimentation basin during construction of the nearby interstate highway.

The design standards for a permanent basin will probably be similar to those for a temporary basin. However, if another agency is involved, their design standards may govern the design.

Figure 1. An expedient sedimentation basin during a clearing and grubbing operation.

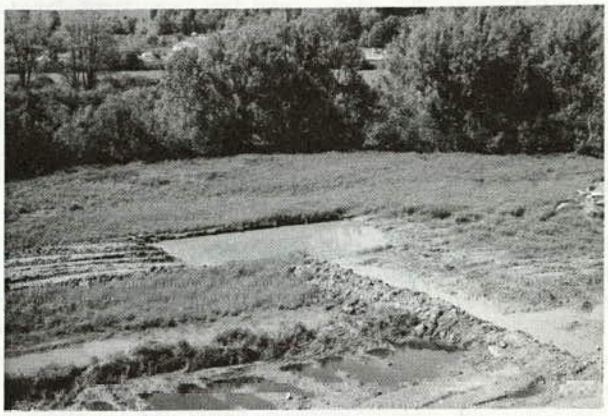


Figure 2. An excavated sedimentation basin protects a brush-lined stream in the background from sediment from highway construction.

Figure 3. This lake was originally a sedimentation basin and now is a permanent recreational facility.

CHAPTER TWO

NEED FOR A BASIN

Early in the process of locating a highway, one must seek and identify those streams, lakes, and reservoirs that, if left unprotected, could be subject to sediment loads caused by highway construction. Also to be noted are land areas that could be harmed by sediment. A broad, grassy swale being used for agriculture and conveying ephemeral flood flows but having no stream channel is a good example of a vulnerable land area. Other examples are found adjacent to streams that readily overflow and leave sediment deposits on the land. If these lands are highly developed they merit special consideration. After those sites in need of immediate protection are identified, protective works must be placed before construction starts. A city water supply susceptible to introduction of sediment is a good example; trout streams and swimming areas are others.

One protection method can be provided by a diversion dike, which requires a safe outlet for the diversion channel. Application of mats, mulches, or sprayed-on materials to excavated areas can also be considered. This might not be practical if repeated applications of surface protection will be needed during construction. If these alternatives are not practical, a sedimentation basin can provide the needed protection, and, if it can do so at a lower cost than diversion or surface protection, is the appropriate choice.

Sedimentation basins can also improve public relations. They are highly visible, tangible evidence of the efforts being made to protect the environment.

RISK

Although the need for a basin may be indicated, the costs and the benefits to be derived need to be studied before a decision is made. Cost-benefit analyses are outside the scope of this synthesis, but a few suggestions relating to this matter can be made.

The degree of protection provided has great impact on the cost-benefit ratio. For example, an expedient basin may intercept a large portion of the sediment at a small cost—a favorable cost-benefit ratio. Upgrading the basin to the temporary type may cost considerably more than the small gain in effectiveness warrants—an apparently unfavorable cost-benefit ratio. However, if the resource to be protected is a critical one, say a city water supply, and a solution must be provided, the cost analysis can take a different turn. It then becomes a matter of finding the lowest-cost solution.

In some situations a basin is needed but it is not possible to construct one. A highway paralleling a trout stream in a mountain gorge is an example. If a basin cannot be provided, the only practical solution may be to intensify other erosion control measures. The economic consequences of not providing sufficient protection have been discussed by Oscanyan and Ports (*4*).

The amount of protection provided by a sedimentation basin is a function of the storm frequency selected for its design. References made to "small storms" and "flood flows" in this report intentionally lack specificity because standards vary with geographic location. However, for small storms a 24-hr storm probably should be specified, with the frequency of occurrence being one, two, or five or more per year, depending on the rainfall history of the locale. For flood flows the selected frequency of occurrence could be 25 years or more. Only permanent basins would probably be designed for the very rare storms. Another requirement in assessing risk for expedient basins is inclusion of weather forecasts before planning for overnight protection.

LOCATION

If a sedimentation basin cannot be placed within the planned right-of-way near the sediment source, an outside site near the sediment source should be required.

The basin should be placed so that it receives runoff only from the area disturbed by highway construction. One must avoid extraneous outside drainage areas that would add runoff and thus reduce settling capability of the basin, increase spillway requirements, and add to the risk of dam failure. Therefore, because main drainageways are not suitable locations for sedimentation basins, minor tributaries as shown in Figure 4 should be used.

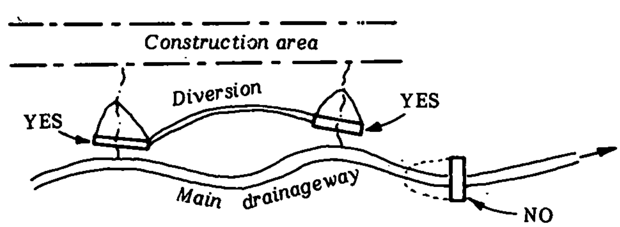


Figure 4. Good and bad locations for sedimentation basins.

The basin should be placed where it will not interfere with the movement of construction equipment or create backwater that could flood and saturate the site. Roadside ditches (Figure 5) may meet this requirement, as may unoccupied lands, old drainage channels cut off the highway, infield areas of interchanges, and wide median areas.

A basin should be accessible for easy cleanout. Further, providing an adjacent area to receive the material removed from the basin during cleanout will encourage timely cleaning and reduce the cost of this operation.

Areas to be avoided when seeking a basin site are those of archeological interest or protected wetlands. Also, if the basin is in a populated area, it may need to be fenced.

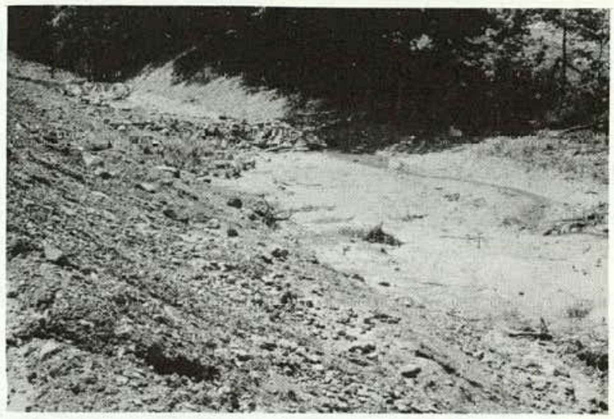


Figure 5. Filter-fabric covered rock check dams in roadside ditches create expedient sedimentation basins.

CHAPTER THREE

PREPARATION FOR DESIGN

DESIGN PHILOSOPHIES

There are two design philosophies on sedimentation basin design. The selection of a design philosophy reflects the relative importance assigned to basin components and the order in which the design steps are to be taken.

The first philosophy postulates that the basin shall not discharge during small, relatively frequent runoff events but shall be allowed to discharge major flood flows. A basin designed on this basis needs only an emergency spillway. Some provision for basin drainage may need to be added to restore capacity for runoff storage or to permit cleaning out. Estimations of trap efficiency need not be made because, except during major flood events, all sediment will be deposited in the basin. The basin should be cleaned out as needed, probably after every storm. The design approach, then, is to locate a basin site and design a dam that will impound the runoff from small, frequent storms and to provide the dam with an emergency spillway to prevent overflow during a major flood. When this philosophy governs the design of a sedimentation basin, design begins with hydrologic and hydraulic considerations that alone may be sufficient to complete the design. Basins designed under this philosophy are usually only for small drainage areas.

The second philosophy holds that the basin shall operate as a detention reservoir and that much of the sediment entering the basin shall be deposited in the basin as the flow slowly moves through it. The design process starts with an estimate of the amount and textural composition of the sediment reaching the basin site. The volume of runoff contributed by the smaller, more frequent storm is estimated by a hydrologic procedure. This volume, the size and slope of the basin, and the planned release rate are needed to calculate trap efficiency and to estimate volume of material trapped by the basin. Because the volume of sediment trapped depends on basin size, and because the basin is sized to provide storage for the sediment, this part of the design process becomes iterative. Finally, the design process is complete when flood volume storage and emergency spillway requirements are determined. These steps will be described in detail in Chapter 4.

An outline of the overall design process is shown in Figure 6.

DRAINAGE AREA

The size of the drainage area is a major determinant in the selection of an expedient or a temporary basin type, so it will probably be estimated when the basin type is selected. However, it may be necessary to characterize the drainage area in greater detail. If there are significant differences in the hydrologic and erosion characteristics of different parts of the area, the sizes of each component should be determined separately. The surface condition of each, present and projected, should be noted. The average slopes and flow lengths of each component should also be determined and recorded at the same time.

Soils must be characterized in sufficient detail to allow subsequent calculations. The data should include particle-size distribution of the soils that are expected to erode and sufficient textural description to enable the selection of a runoff coefficient or curve number.

SEDIMENT VOLUME TO BE STORED

Rule-of-Thumb Estimates

A simple and arbitrary way to estimate sediment storage volume is to use rule-of-thumb values. Examples of such values are given in Table 1, where the values are for total volume of the sedimentation basin.

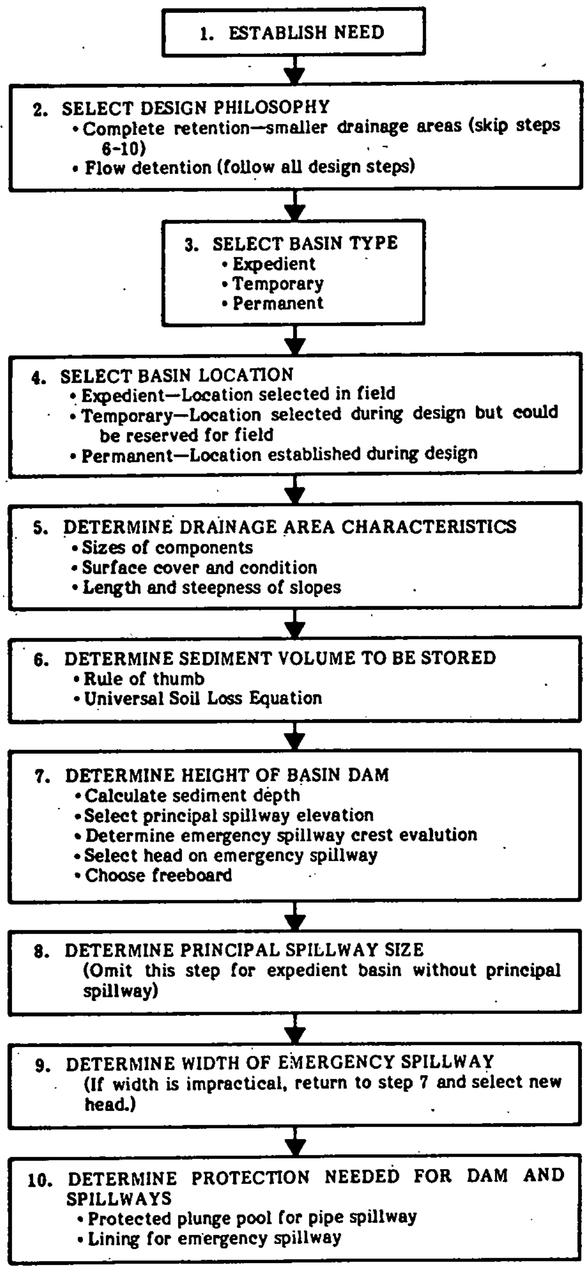


Figure 6. Design process for a sedimentation basin.

TABLE 1. VALUES FOR ESTIMATING STORAGE REQUIREMENTS

Source (reference number)	Storage Requirement[a] (watershed inches)	Location
5	0.51	Maryland
6	0.50	Maryland
7	0.61 to 1.93	Pennsylvania
8	0.50	Virginia

[a] A watershed inch is a volume equal to the watershed area multiplied by 1 in. A storage requirement of 1 watershed inch is equal to 134.4 yd^3 of storage volume per acre of drainage area.

Unit storage requirements based on experience are useful for an initial evaluation of a potential site or a preliminary estimate of basin size needed. For the smaller basins a simple rule relating the size of the basin to the size of the watershed may be satisfactory. When general guidelines for sediment storage volume are not available and experience in basin design is insufficient, estimates of the storage requirement will have to be developed from soils and climate data and from the characteristics of the conveyance system bringing the sediment to the basin.

Universal Soil Loss Equation Method

The universal soil loss equation (USLE), originally developed for application to agricultural areas by Wischmeier and Smith (*9*) and later adapted to construction sites (*10*), estimates the erosion amounts from land areas by using climate, soil, land form, and land use as parameters. This equation may be used to estimate the initial supply to the sedimentation basin. Some eroded material will not reach a downstream basin because it is intercepted en route. That portion reaching the basin, however, is estimated by applying a sediment delivery ratio to the estimated initial erosion. Finally, the trap efficiency of the reservoir will need to be estimated to establish how much of the sediment will settle in the basin. These three components—initial erosion, delivery ratio, and trap efficiency—of the procedure to estimate the sediment storage required will be treated separately.

INITIAL VOLUME ERODED

USLE in the form modified for highway construction application in *NCHRP Report 221* (*2*) is

$$A = R \cdot K \cdot LS \cdot VM \tag{1}$$

where

A = computed amount of soil loss per unit area for the time interval represented by the factor R, generally expressed as tons per acre,
R = rainfall factor,
K = soil erodibility factor,

LS = topographic factor (length and steepness of slope), and

VM = erosion-control factor (vegetative and mechanical measure).

Rainfall Factor (*R*)

The rainfall factor, R, is defined by Wischmeier and Smith (*11*) as "the number of erosion-index units in a normal year's rain. The erosion index is a measure of the erosive force of specific rainfall." Average annual values of R have been calculated from rainfall records and plotted on a map of the United States (*2*). These average annual values are not directly applicable to the highway construction situation because critical times for sediment production may be less than 1 year. Also, because basin cleanout is planned, providing for 1 year's sediment production should not be necessary. In writing about debris basins, Dodge (*12*) states: "It is usually more economical to provide debris storage adequate for a single large flood and to re-excavate the basin as often as necessary to maintain design capacity."

In this report reference is made to providing sediment storage for a small storm without specifying further. The specification for the small storm is left to the agency doing the design. However, the duration and frequency of this storm should be specified for the guidance of the designer. For purposes of illustration only, a 24-hr and 5-year frequency will be used here to define the small storm. As an example, the R factor for this storm in Payne County, Oklahoma, will be calculated to illustrate the procedure.

From the *Rainfall Frequency Atlas* (*13*), one can tabulate the rainfall amounts for the 5-year recurrence interval for the time periods as shown in Table 2. It will be assumed that the maximum 12-hr storm occurred during the maximum 24-hr storm, the 6-hr within the 12-hr, and so on. This assumption is acceptable because the order of occurrence of the various intensity periods does not affect the total storm energy. The acceptability of this assumption is reinforced by Wischmeier's (*14*) finding that "no correlation between type of storm and the unexplained residuals of soil losses could be detected in the data." He found that the relation of rainfall factor to soil loss was not significantly affected by the time of occurrence of the most intense period of the storm, whether it was at the beginning or at the end of the storm.

The third column of Table 2 shows the rainfall during the interval, which is the difference between successive rainfall totals. The fourth column gives the length of the time interval in hours, and the fifth gives the rainfall intensity in inches per hour obtained by dividing the value in the third column by the value in the fourth column. The units shown in the sixth column are calculated from the equation by Wischmeier (*15*):

$$E = 916 + 331 \log_{10} X \qquad (2)$$

where E is the kinetic energy in foot tons per acre inch and X is the rainfall intensity in inches per hour.

Equation 28 gives the energy for 1 inch of rainfall at the given intensity, so the energy value must be multiplied by the amount of rain falling during the selected interval. For example, the calculation for the first line is:

$$\text{energy units} = 1.85\ (916 + 331 \log_{10} 3.70) = 2045.$$

The rainfall factor for the storm is the total energy for the storm over 24 hr multiplied by the maximum 30-min intensity and divided by 100 (*14*):

$$R = \frac{[(\Sigma E)\ I_{30}]}{100} \qquad (3)$$

or

$$R = \frac{[(4180)\ 3.70]}{100} = 155.$$

The R value from Equation 3 is for the largest 24-hr rainfall to be expected, on the average, once every 5 years. The time of occurrence of this storm during the year is not known because seasonal occurrence is not a parameter in the precipitation data presented in the *Rainfall Frequency Atlas* (*13*). If the maximum sediment-production potential of the construction site is limited to a short period, say from initial disturbance to the establishment of temporary surface protection, and if the time of the year when this period will occur is known, consideration should be given to sizing the sedimentation basin for the probable erosion during this period. This can be done by calculating the R value for the period.

NCHRP Report 221 (*2*) provides information on the cumulative percentage distribution of the R value during the year for the various climatic areas. A sample calculation illustrates the use of this information to calculate a short-term R value for Payne County, Oklahoma. If one assumes that the construction site will be in its most disturbed state during the month of June, storage for the sediment produced during that month will be needed. The mean annual R value is 450 (*2*), which is multiplied by 1.4 (see Figure 7) to obtain the *f*-year recurrence annual value: 630. According to *NCHRP Report 221* (*2*), the accumu-

TABLE 2. CALCULATIONS OF THE *R* VALUE FOR THE 5-YR, 24-HR STORM FOR PAYNE COUNTY, OKLAHOMA

Interval Duration (hours)	Total Rain (in.)	Rain per Interval	Time Interval (hours)	Rain Intensity (in./hr)	Energy Units *
0	0				
		1.85	0.5	3.70	2045
0.5	1.85				
		0.48	0.5	0.96	437
1	2.33				
		0.42	1	0.42	332
2	2.75				
		0.35	1	0.35	267
3	3.10				
		0.55	3	0.18	367
6	3.65				
		0.60	6	0.10	350
12	4.25				
		0.75	12	0.06	382
24	5.00				
				Total	4180

lated percentages of the annual R values are 47 percent for July 1 and 27 percent for June 1; therefore 20 percent accumulated during June. Thus 20 percent of 630, or 126 units, of the annual rainfall factor will probably occur in June.

The June total for $R = 126$ is approximately equal to the annual maximum 24-hr storm value of 155 previously calculated. This is not surprising because for this particular location June is the month that has the most R units. The latter method is much simpler than the storm method for calculating a short-term R value, so designers will probably prefer it. If the month during which the greatest exposure of the site is not known, the month of most rapid accumulation of R units should be selected for the calculation of the R value. This will probably result in a basin capacity that can take the sediment volume that accumulates between cleanouts.

The point is that R can be calculated for any time interval although only average annual values have been plotted on national maps.

Soil Erodibility Factor (*K*)

The soil erodibility factor, K, represents the ability of the soil to resist erosion by rain, which is dictated by particle size and distribution, structure, void space and pore size, and organic matter. The value ranges from 0.1 to 0.7; higher values represent more erodible soils. Approximate values of K can be obtained from maps (*2*), which should be used only when analyses of the soils from the site are not available. When soils data are available, an estimating diagram of the type developed by Wischmeier (*10*) is used to determine K (see Figure 8).

Topographic Factor (*LS*)

The topographic factor, LS, can be evaluated by using the following relation developed by Foster and Wischmeier (*16*) and Wischmeier and Smith (*17, 18*):

$$LS = (l/72.6)^m\ (65.41\ s^2/s^2 + 10\ 000) = (4.56\ s/\ \sqrt{s^2 + 10\ 000}) + 0.065 \qquad (4)$$

where

l = slope length in feet,
s = slope steepness in percent, and
m = exponent varying with slope (0.2 for slopes <0.1 percent, 0.3 for slopes 1 to 3 percent, 0.4 for slopes 3.5 to 4.5 percent, and 0.5 for slopes >5 percent).

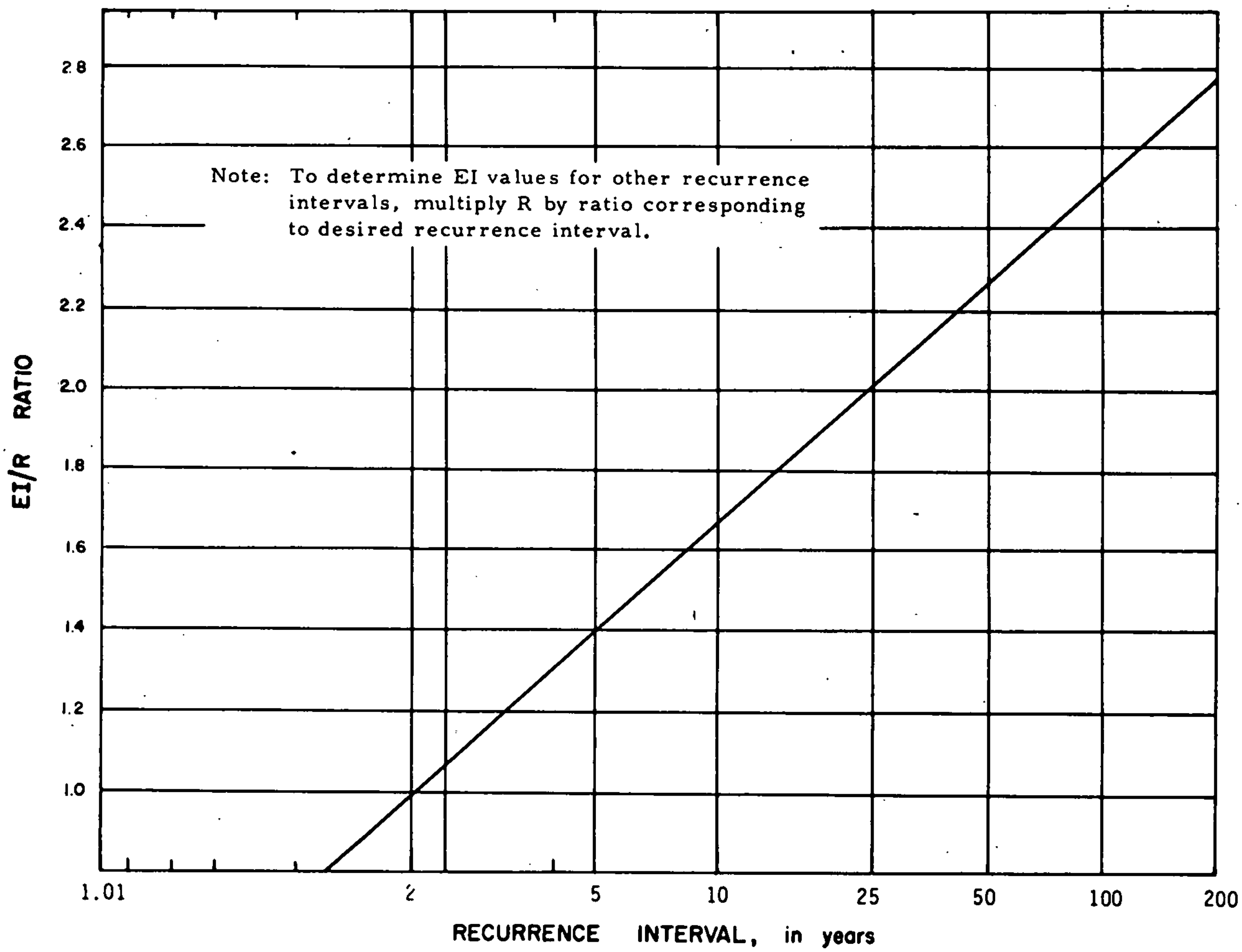


Figure 7. Relation between the EI_{30}/R ratio and the recurrence interval (2).

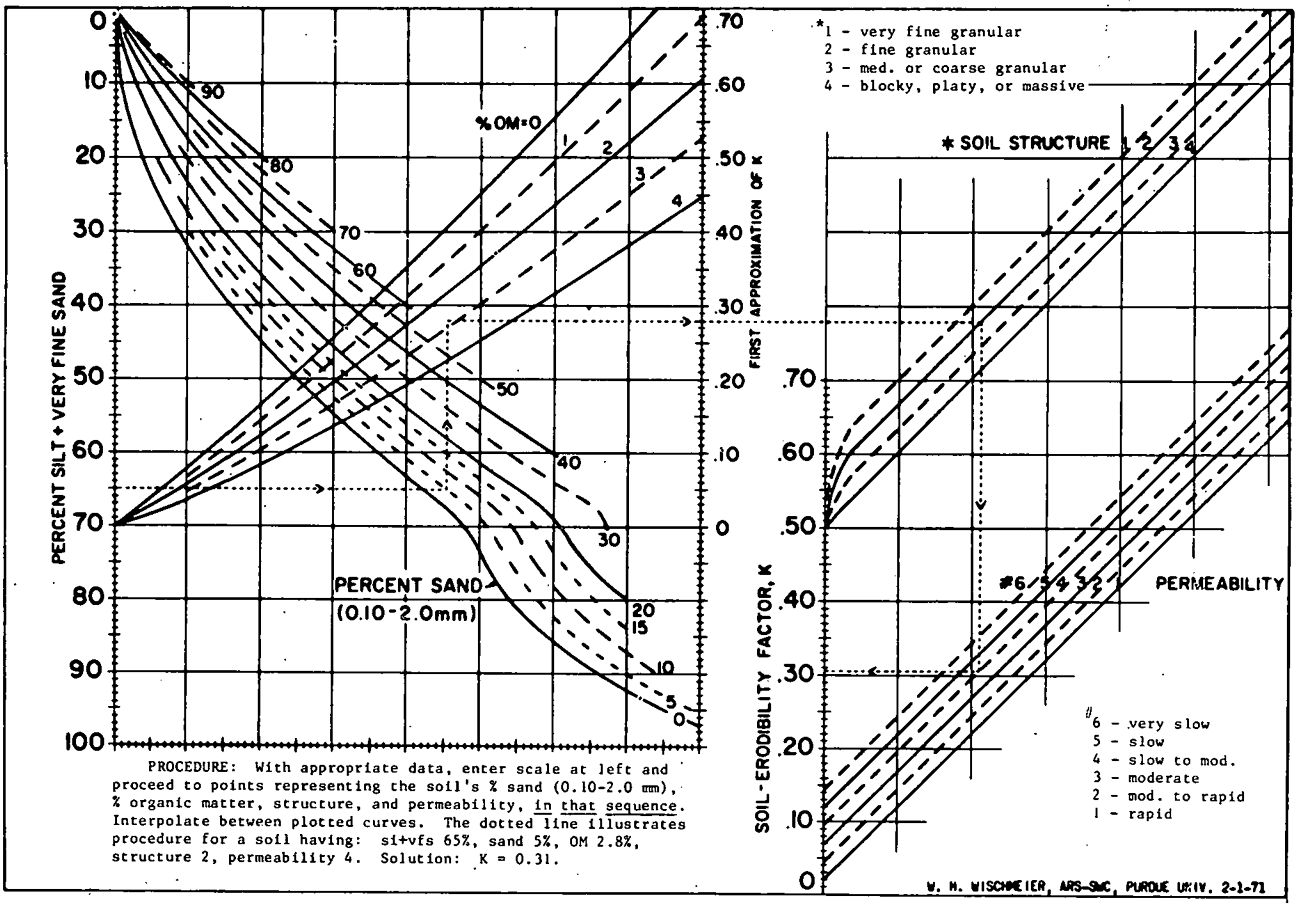


Figure 8. Nomograph for determining soil erodibility factor K (2).

Figure 9 is a graph for the solution to this equation. Multiple solutions to the equation are readily found with a programmable hand calculator, which some prefer over the graphs.

The discussion so far on the use of USLE has been concerned with a simple slope. For complex slopes, those with different degrees of steepness along the flow path, the solution is complicated by the cascading effect. One can use *NCHRP Report 221* (*2*) for estimating soil loss for these slopes.

Slope steepness and length of the highway cross-section elements are manipulated by the designer seeking the best cross-section with respect to performance, safety, economy, and low sediment-producing characteristics. Because the volume of sediment that might reach a sedimentation basin is directly affected by the *LS* factor, interaction between selection of highway cross-section and sediment basin design would be expected. However, if slopes and lengths have already been established, they will be accepted and used by the basin designer.

Erosion-Control Factor (*VM*)

The erosion-control factor, *VM*, characterizes the effect of surface condition on the sediment-producing potential of the eroding surface. Values for the *VM* factor are given in Table 3.

Because a sedimentation basin may need to provide protection before any surface treatment can be applied to the disturbed area, a *VM* value for a bare soil condition should be used in design. A conservative approach would be to use 1.71, the highest value in the table. However, as experience accumulates, a better estimate for this value may be established.

The *VM* value will drop as surface treatment is applied and becomes effective. The impact this has on the basin will be to reduce the necessary frequency of cleanout. Values of *VM* in the table for treated surfaces can be used to estimate the new sediment production rates and to assess future cleanout frequency or continued need for the basin.

Adjustments to Sediment Estimate

USLE provides an estimate of rill and sheet erosion only and does not account for gully erosion or local scour. Thus the initial estimate of sediment production should be increased if there is a possibility that such additional erosion will occur.

A sample calculation for estimating the initial quantity eroded from a 4-acre watershed follows.

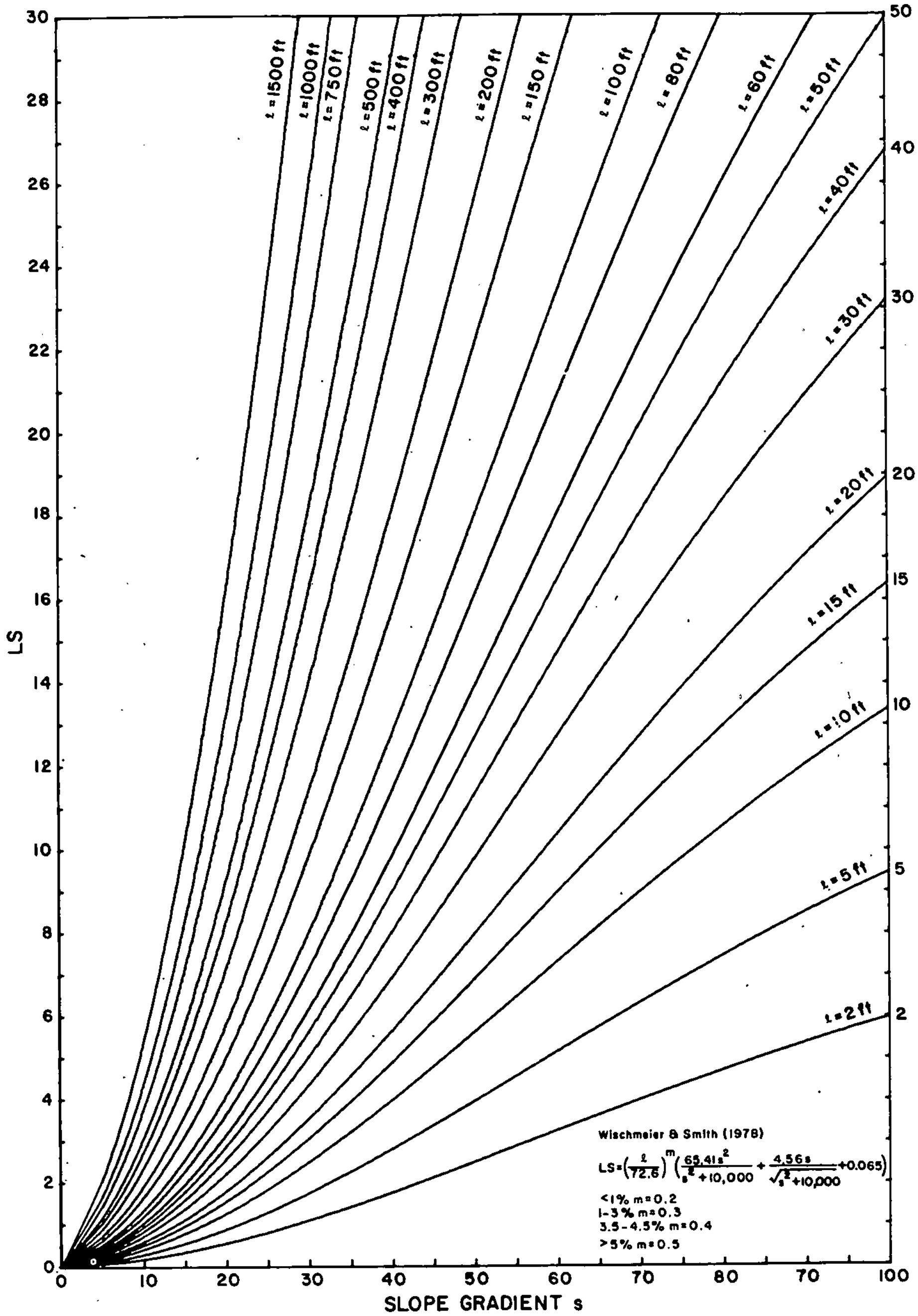


Figure 9. Graph for determining topographic factor LS of simple slopes (2).

Calculation of the Initial Volume Eroded

The rainfall factor, R, for the storm, as calculated previously ($R = 155$) for Payne County, Oklahoma, will be used in the following example of how the modified USLE is used to compute the initial volume eroded from a 4-acre watershed, half of which is disturbed by construction.

The soil erodibility index, K, will be obtained from *NCHRP Report 222* for the high erodibility area. This indicates a value of 0.40 for K. If an analysis of the soil that provided values for the various parameters used in Figure 8 had been available, this figure would have been used to obtain a better estimate of K.

The topographic factor, LS, will be obtained from Figure 9, but first the average slope length, l, and the average slope steepness, s, of the sediment-producing area must be determined. For this example a single slope will be assumed. Compound slopes require the more involved procedure described in *NCHRP Report 221*. The values used here are $l = 100$ and $s = 10$ percent; LS then equals 1.4.

The erosion control factor, VM, will be obtained from Table 3. A bare soil condition will be assumed. Much of the disturbed area will be in compacted fill so a value of 1.71 will be chosen for *VM*.

The solution to USLE yields $A = 155 \times 0.4 \times 1.4 \times 1.71 = 148$ tons per acre. The total gross erosion from the 2 acres of disturbed area is $2 \times 148 = 296$ tons.

The 2-acre undisturbed grassy part of the watershed will be ignored insofar as sediment production is concerned, because its *VM* value is very small. Table 3 shows a value of 0.01 for a permanent seeding, probably equivalent to that for a grassy area. The quantity of sediment produced by the 24-hr, 5-year storm is then estimated to be 296 tons. This weight must be converted to volume in order to calculate sediment basin size.

Converting weight to volume requires an estimate of weight per unit volume of the sediment in place in the basin. A study of data from reservoir surveys shows that the density of sediment depends on its composition. A high content of organic material reduces the unit weight; weights as low as 25 lb/ft³ have been observed. Sediments in western reservoirs are largely mineral and weigh about 60 lb/ft³ when newly deposited. This kind of sediment is probably representative of construction sediment, so a unit weight of 60 lb/ft³ is used in this example.

The volume of sediment at 60 lb/ft³ then is $(296 \times 2000)/(60 \times 27) = 365$ yd³.

SEDIMENT DELIVERY RATIO

The amount of sediment reaching the basin is estimated by applying a sediment delivery ratio, SDR, to the initial erosion estimate. Values of SDR's for the flow system of a highway construction site have not been determined so far, but it might be possible to develop estimates by calculating the sediment-transporting capability of each slope and channel in the flow system for the variously sized particles in the sediment supply. An example of this approach is the method used by Neibling and Foster (*19*).

These approaches are complex and probably beyond the requirements of this report but are mentioned for the benefit of those who want to pursue the subject of sediment delivery ratios in greater depth. Here we shall use values of sediment delivery ratios determined from studies of watersheds to estimate values for highway use.

In considering the subject of sediment delivery ratio, the American Society of Civil Engineers Task Committee on Preparation of Sedimentation Manual (*20*) wrote: "The percentage of sediment delivered from the erosion source to any specified downslope location is affected by such factors as size and texture of erodible material, climate, land use, local environment, and general physiographic position." Thus it can be expected that any portrayal of sediment delivery ratio as a function of a single, independent variable will show considerable scatter. Piest et al. (*21*) illustrated this with a plot of sediment delivery ratio versus watershed drainage area as shown in Figure 10. They explained this variation and showed the potential for improving the sediment delivery ratio method. Their developed relation for an Iowa cornfield with a 9 percent slope is reproduced here as Figure 11.

Despite the uncertainties of the sediment delivery ratio and drainage area relation, it is still a useful and practical

TABLE 3. TYPICAL *VM* FACTOR VALUES REPORTED IN THE LITERATURE (*2*)[a]

Condition	VM Factor
1. Bare soil conditions	
freshly disked to 6-8 inches	1.00
after one rain	0.89
loose to 12 inches smooth	0.90
loose to 12 inches rough	0.80
compacted bulldozer scraped up and down	1.30
same except root raked	1.20
compacted bulldozer scraped across slope	1.20
same except root raked across	0.90
rough irregular tracked all directions	0.90
seed and fertilize, fresh	0.64
same after six months	0.54
seed, fertilizer, and 12 months chemical	0.38
not tilled algae crusted	0.01
tilled algae crusted	0.02
compacted fill	1.24-1.71
undisturbed except scraped	0.66-1.30
scarified only	0.76-1.31
sawdust 2 inches deep, disked in	0.61
2. Asphalt emulsion on bare soil	
1250 gallons/acre	0.02
1210 gallons/acre	0.01-0.019
605 gallons/acre	0.14-0.57
302 gallons/acre	0.28-0.60
151 gallons/acre	0.65-0.70
3. Dust binder	
605 gallons/acre	1.05
1210 gallons/acre	0.29-0.78
4. Other chemicals	
1000 lb. fiber Glass Roving with 60-150 gallons asphalt emulsion/acre	0.01-0.05
Aquatain	0.68
Aerospray 70, 10 percent cover	0.94
Curasol AE	0.30-0.48
Petroset SB	0.40-0.66
PVA	0.71-0.90
Terra-Tack	0.66
[b]Wood fiber slurry, 1000 lb/acre fresh	0.05-0.73
[b]Wood fiber slurry, 1400 lb/acre fresh	0.01 -0.36
[b]Wood fiber slurry, 3500 lb/acre fresh	0.009-0.10
Portland cement + Latex	
1000 lbs/ac + 8 gals/ac	0.13
1500 lbs/ac + 12 gals/ac	0.006
5. Seedings	
temporary, 0 to 60 days	0.40
temporary, after 60 days	0.05
permanent, 0 to 60 days	0.40
permanent, 2 to 12 months	0.05
permanent, after 12 months	0.01
6. Brush	0.35
7. Excelsior blanket with plastic net	0.04-0.10
8. Mulch (depends on type and amount of mulch and erosion potential)	0.01-1.00

[a]Note the variation in values of VM factors reported by different researchers for the same measures. References containing details of research which produced these VM values are included in NCHRP Project 16-3 report, "Erosion Control During Highway Construction, Vol. III, Bibliography of Water and Wind Erosion Control References," Transportation Research Board 2101 Constitution Avenue, Washington, D C 20418.

[b] This material is commonly referred to as hydromulch.

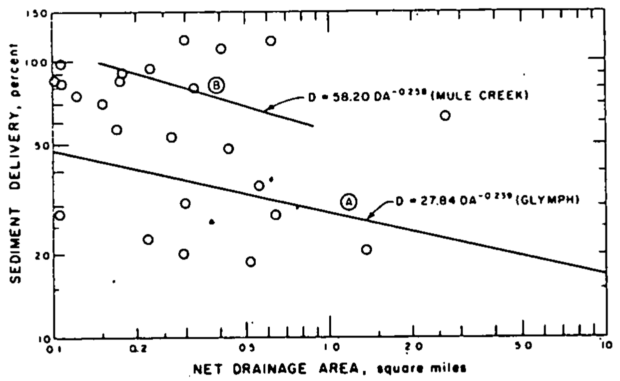


Figure 10. Comparison of sediment-delivery ratios for 24 reservoir watersheds with curves developed from reservoir data from eastern Nebraska and western Iowa (21).

tool. An early development of this relation by Roehl (*22*) is shown in Figure 12. The particle size has a major impact on the sediment delivery ratio. Reed (*3*) showed that at one location the source material (topsoil and subsoil) had a sand content of about 40 percent, whereas the flow from the construction area had only 2 percent sand. Williams (*23*) showed that the influence of particle size on sediment yield depended on travel time from source to outlet. For particles 0.4 mm in diameter and for a travel time of 7.48 hr, only 0.6 percent would reach the outlet. But, if the travel time is reduced to 0.76 hr, more than 59 percent would reach the outlet. For the shorter travel times characteristic of highway construction sites, a still greater percentage of the larger particles would be delivered to a sedimentation basin.

After a study of the various curves of sediment delivery ratio versus drainage area and of the observations on the effect of particle size, two curves are suggested to represent sediment delivery ratios. These are shown in Figure 13. They are envelope curves for the data presented in the preceding discussion. The spread between the two envelopes is so great for the larger areas that their value as estimating tools diminishes with area size. For conservative design, one should use the upper curve; for coarse particles, gentle slopes, and vegetated flow paths, the lower curve is more appropriate.

Another approach was taken by Renfro (*24*), who showed that sediment delivery ratio is a function of the relief-to-length ratio, which is total fall divided by main channel length (see Figure 14). This curve yields an SDR value of 60 percent for a relief-to-length ratio of 0.04, a main channel average slope of 4 percent. Highway construction sites, because of the steepness of their cut-and-fill slopes, probably average more than 4 percent, so values of SDR even larger than 60 percent can be expected.

The difficulty of establishing ratios for sediment delivery has inspired attempts to circumvent the need for them. Williams (*25*), for instance, developed a modified USLE in which the rainfall factor is replaced by volume of runoff and peak flow rate. The equation now provides an estimate of the sediment yield at the outlet of a watershed, or sedimentation basin location. Williams (*26*) also developed an empirical equation for estimating sediment delivery ratios based on watershed topographic and hydrologic characteristics. Nevertheless, these methods have not been developed for highway application, so no further detail is given. They are mentioned for the benefit of those who wish to study other approaches to sediment estimation.

Sediment Delivered to the Basin

The amount of the initial volume eroded that reaches the sedimentation basin will be estimated by the sediment delivery ratio. Figure 13 shows SDR's of 0.55 for sand and 1.00 for clay for 2 acres. A mean value of 0.78 will be used in this estimate. An estimated 285 yd^3 (0.78 × 365 yd^3 of erosion calculated previously) will reach the basin. The next step is to design a basin that will trap as much of the sediment as is considered practicable.

BASIN TRAP EFFICIENCY

Some of the material reaching the basin will pass through with the spillway outflow. The ratio of material retained to the total sediment load is the trap efficiency of the basin.

Trap efficiency probably has been investigated more extensively than any other aspect of sedimentation basins.

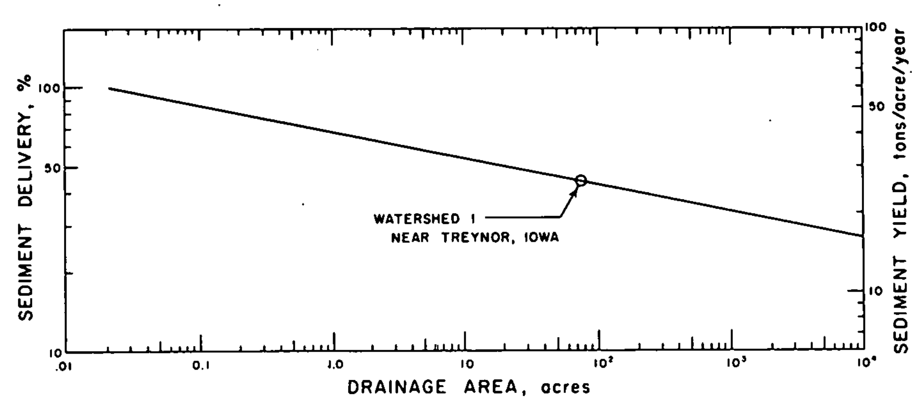


Figure 11. Probable relation of drainage area to sediment delivery and yield for sheet-rill erosion sources from Iowa cornfields that had an average 9 percent slope (21).

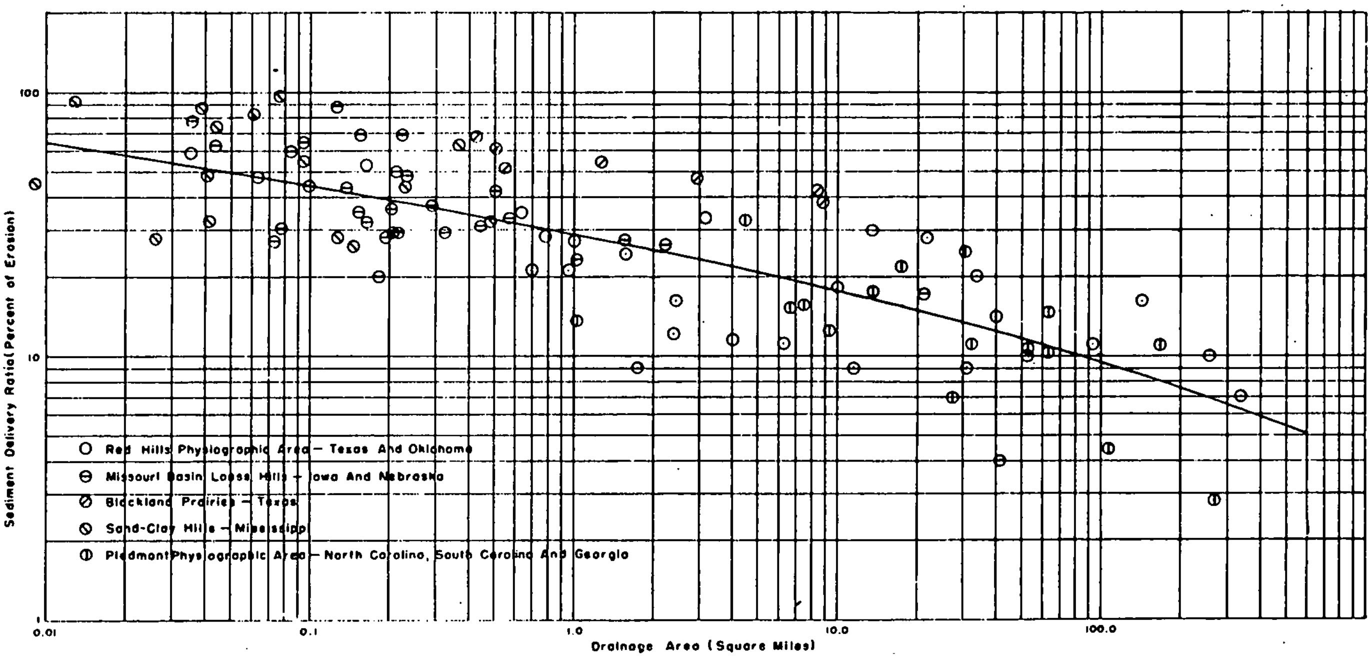


Figure 12. Sediment delivery ratio versus size of drainage area (22).

These investigations have taken two directions. One course was toward the design of settling tanks for the removal of sediments that would leave the water effluent clean enough for an intended use, such as domestic consumption or irrigation. Here high trap efficiency was the desired goal. The other course led to the design of major reservoirs and the prediction of their useful life. Here low trap efficiency was desirable.

Today there is a new need to be met, the one with which this report is concerned—environmental protection. Like settling tanks, the goal here is high trap efficiency. But no relationship or procedure has yet been found that is suitable for direct application to sedimentation basin design.

The first approach to finding a procedure for estimating trap efficiency applicable to small sedimentation basins was to seek field measurements. The data found are from reservoirs and, as such, are not directly applicable to small sedimentation basins. However, the data indicate the probable magnitude of trap efficiency and are worth examining, in that they provide a basis for making rough estimates when lengthy computational procedures may not be warrranted.

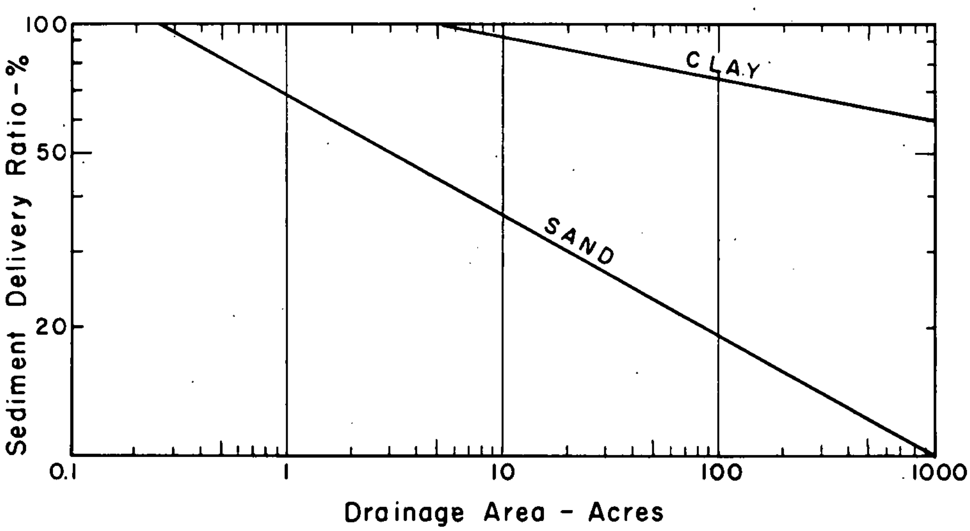


Figure 13. Sediment delivery ratios suggested for the design of sedimentation basins.

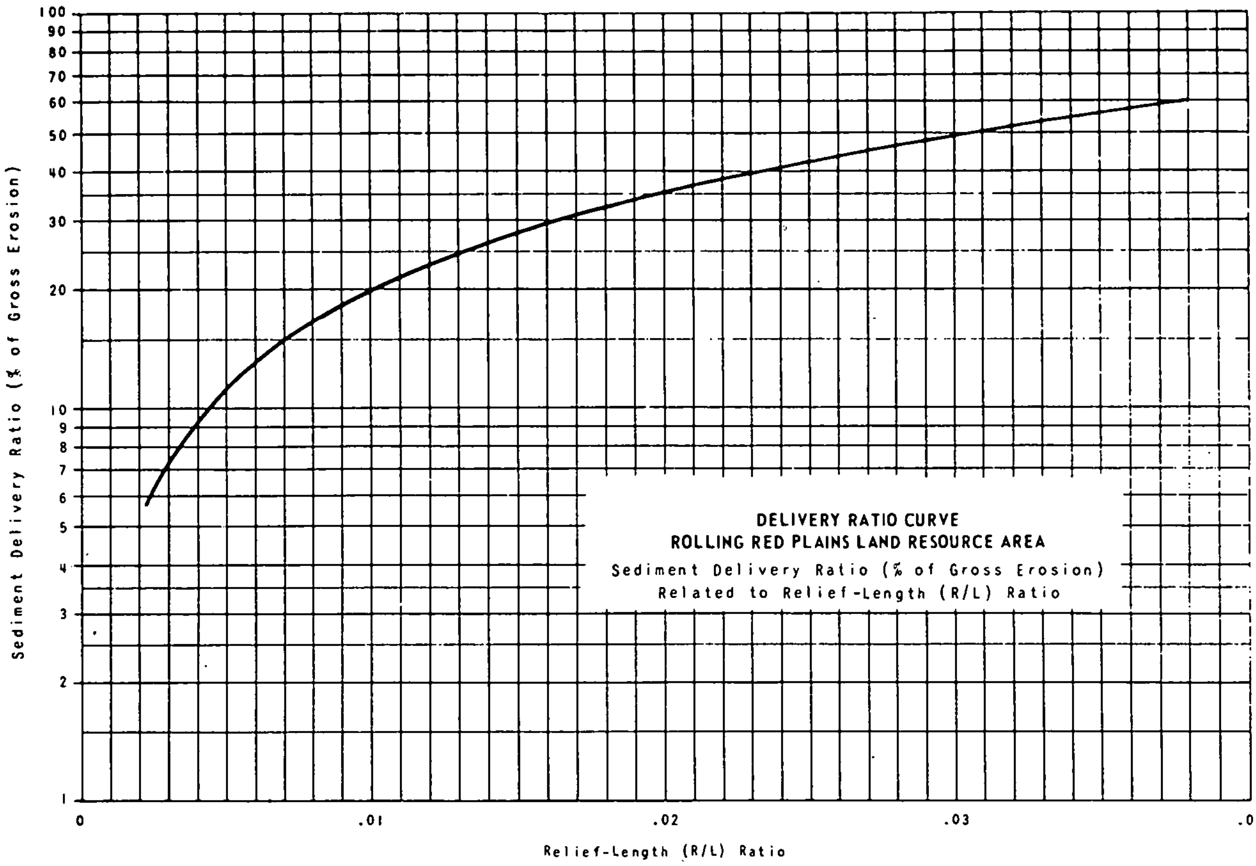


Figure 14. Sediment delivery ratio versus relief length ratio (Renfro's curve) (24).

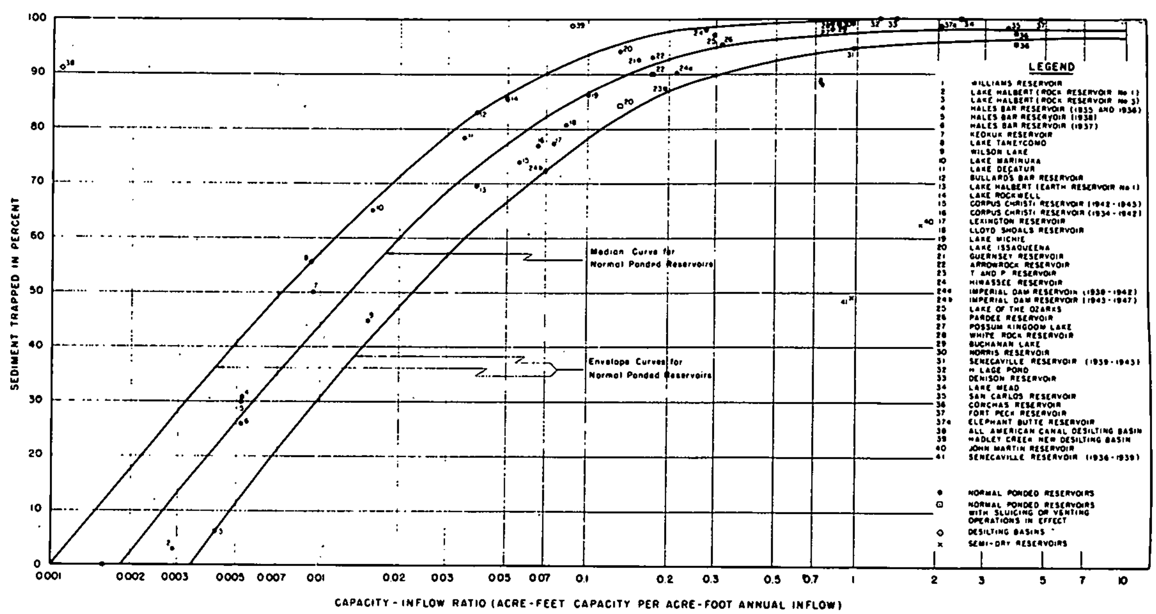


Figure 15. Detention time versus percentage of load deposited for Dardanelle lock and dam (27).

The more useful data presentations show trap efficiency as a function of a detention time. Brune (*27*) concluded that the reservoir capacity-to-inflow rate ratio was a better estimator of trap efficiency than the capacity-to-area ratio and plotted a curve of this relation (Figure 15). There is still considerable scatter to the data, so envelope curves are shown.

Livesey (*28*) related percentage of load deposited (trap efficiency) to reservoir capacity versus detention time (a ratio of reservoir capacity to discharge) and used particle size as a parameter (Figure 16). This figure shows the long time it takes to build up an appreciable deposition of silts and clays.

These various trap efficiency curves for reservoirs can be used in sedimentation basins design to obtain estimates of trap efficiency that may be suitable for initial approximation.

Figure 16 shows that in 24 hr 90 percent of the sands will settle but only 20 percent of the clays will do so. So actual trap efficiency will probably lie between these two values, depending on the particle sizes of the sediment load.

The second approach to estimating trap efficiency is analytical and involves the fall velocity of the particles, the forward flow velocity in the basin, and the length and depth of the basin. This is the design approach selected for the example in this synthesis.

Bondurant et al. (*29*) offer a design approach for determining the length of a sedimentation pond and the trap efficiency of a pond. They showed that the length of a pond required to settle a particle 1 ft is equal to the ratio of the forward flow velocity to the particle fall velocity:

$$L = V/v \qquad (5)$$

where

$L =$ length of pond required for particle to settle 1 ft,
$V =$ forward velocity of flow, and
$v =$ fall (settling) velocity of particle.

Stokes' law was used to calculate the fall velocity. The solution for this equation is shown in Figure 17. A sample calculation of trap efficiency for a given basin and a graded sediment load will illustrate the approach. The trap efficiency of the basin is estimated by considering the fall velocity of the particles reaching the basin and the geometry of the basin. The following assumptions are made:

1. A particle size distribution curve is available for the source material.
2. All particles of a diameter less than a given percentage on the size distribution curve reach the sediment basin. The percentage used is equal to the sediment delivery ratio expressed as a percentage.
3. The concentration of sediment in the flow is uniform and is equal to the total volume of sediment divided by the total volume of runoff.
4. The basin provides storage for the design storm between the principal spillway crest and the emergency spillway crest.
5. After the design storm inflow, the basin will drain to principal spillway crest within 24 hr.

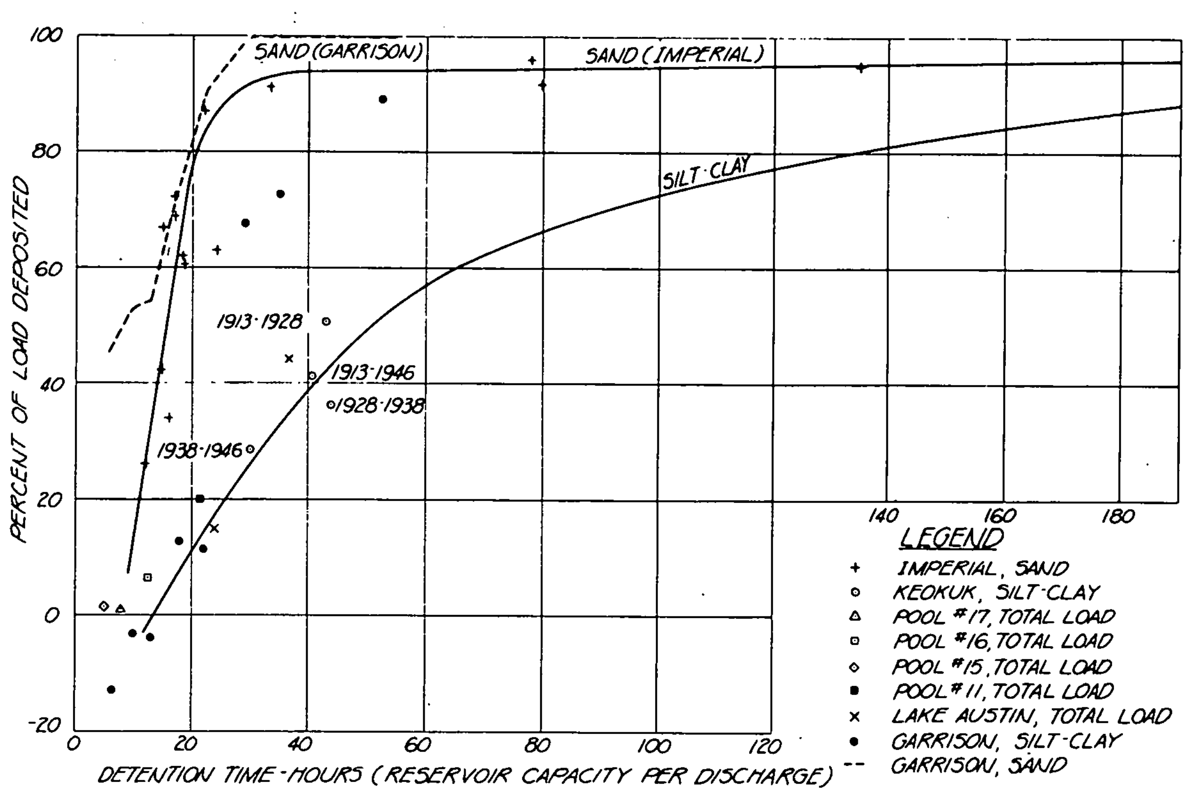


Figure 16. Trap efficiency versus ratio of capacity to inflow, type of reservoir, and method of operation (28).

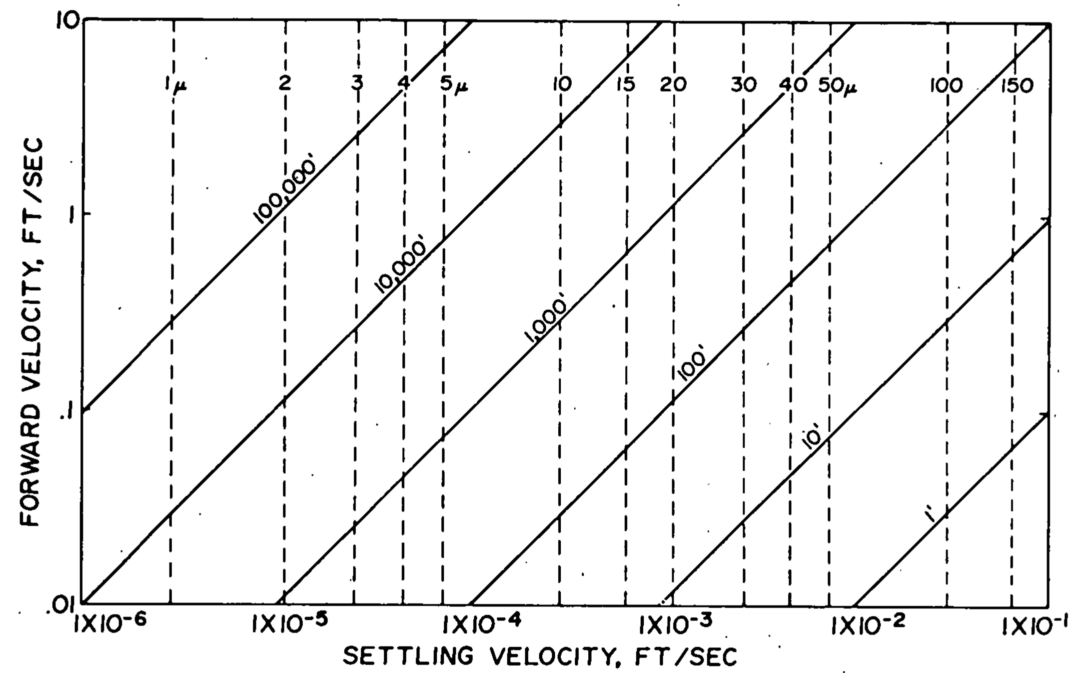


Figure 17. Pond length required for quartz particles to settle 1 ft at various forward velocities according to Stokes' law (29).

6. The outflow rate will be at a uniform rate through the pipe principal spillway. When the pipe primes, the flow rate becomes a function of the square root of the total head and does not vary much over the operating range in the sedimentation basin; therefore, the assumption is a reasonable one.
7. The particles will settle in accordance with Stokes' law.
8. When a particle reaches the basin bed it is stopped from further movement.

The particle size distribution for the soil in this example for selected particle diameters is read from the distribution curve and tabulated (Table 4).

In accordance with assumption 2, all particles of a diameter smaller than those at the 78 percent point on the size distribution curve will reach the basin. The 78 percent figure is the sediment delivery ratio used earlier. The assumption is probably not strictly correct, because some of the larger particles will reach the sedimentation basin as bed load. However, these will be halted at the very upper end of the basin and can be ignored in the subsequent calculations.

Size distribution curve for the sediment reaching the basin will be derived by dividing the "percentage smaller than" values by 0.78 to obtain the distribution values given in the third column of Table 4.

To determine how much of the 285 yd^3 calculated earlier will be deposited, the flow velocity through the basin and on the basin length must be estimated.

The runoff volume is calculated by the Soil Conservation Service (*30*) curve-number method. A curve number for the 4-acre watershed is determined by weighting the curve numbers for the disturbed and undisturbed areas. (A more detailed description of making this estimate is given in the section on estimating storm runoff in Chapter 4.) For the 5-in. rainfall of this example, the runoff is found to be 2.1 in., or a total runoff of 30 492 ft^3. If a depth of 3 ft can be provided between a principal spillway crest and an emergency spillway crest, a basin having an average surface area of 10 000 ft^2 will be required.

If this volume of runoff is to be drained off in 24 hr, the

TABLE 4. PARTICLE SIZE DISTRIBUTION OF SOURCE MATERIAL

Particle Diameter (mm)	Percentage Smaller than	Distribution of material at basin
6	100	Because the sediment delivery ratio is 0.78, these particles are assumed to have been dropped before the flow reached the basin.
2	99	
1	98	
0.6	96	
0.3	91	
0.25	78	100
0.2	75	96.2
0.1	59	75.6
0.03	35	44.9
0.01	20	25.8
0.003	12	15.4
0.001	8	10.3

average discharge rate through the principal spillway is 0.35 ft^3/sec. The average forward flow velocity is the discharge rate divided by the flow's cross-sectional area. If in this example a basin 50 ft wide and 200 ft long can be provided, the flow cross-section would be 3 ft deep by 50 ft wide, or 150 ft^2. The mean flow velocity would be 0.00233 ft/sec.

In this calculation the cross-section area below the principal spillway crest is not included in the flow cross-section because it will eventually fill with sediment. The assumption for estimating trap efficiency is on the conservative side. On the other hand, the assumption has the effect of requiring a larger basin and thus costing more. When the estimates of particle size, flow velocity, and basin length are established, the trap efficiency can be calculated by applying Stokes' law. The procedure devised by Bondurant et al. (*29*) will be followed.

The horizontal distance, L, traveled by a particle as it falls 3 ft is calculated from Equation 5 as follows:

$$L = 3\ (0.00233/v)$$

where v is the fall velocity in feet per second. Any particles for which L is less than the length of the basin (200 ft in this case) will be assumed to have settled. An analysis of the settling length requirement of the various categories of particle size is given in Table 5.

When the settling length for all particles of a given size exceeds 200 ft, the percentage of particles settling is the ratio of the basin length to the required settling length times 100. Noteworthy is the fact that, for particles smaller than 0.01 mm, the required settling length increases greatly as size is further reduced. The calculation shows that 81.6 percent of all sediment entering the basin will settle. This is the estimate of the trap efficiency of this basin.

The quantity of sediment leaving the basin with the flow will be $0.184 \times 285 = 52.4$ yd^3. If this material is uniformly mixed in the outflow, an estimate of the concentration can be made.

Concentration of sediment is commonly expressed in milligrams per liter (*31*) computed as one million times the dry weight of sediment in grams to the volume of water sediment mixture in cubic centimeters. Thus 52.4 yd^3 of sediment in 30 492 ft^3 of water has an average concentration of 44 600 mg/1. This is a high cencentration, but still much lower than the concentration of the inflow, which was about 240 000 mg/1.

If the concentration is greater than that allowed, or if the total quantity discharged is too great, consideration should be given to the use of a coagulant to hasten settling.

COAGULANTS TO SPEED SETTLING

Settling rate can be accelerated with coagulants, which, although commonly used in water-treatment plants, have not been used much in sedimentation traps. However, there are situations where it may be desirable to speed up settling. For example, outflow from a basin may persist for days and continue to pour turbid water into a stream that ordinarily would have cleared up soon after a storm. A trap in this situation might be described as being negatively effective.

TABLE 5. CALCULATION OF TRAP EFFICIENCY BY ANALYSIS OF SETTLING CHARACTERISTICS OF PARTICLES

Particle Diameter (mm)	Percent Smaller Than	Percent in Category	Fall Velocity* (fps)	Length to Settle (ft)	Percent Settling	Percent of Total Settling
0.25	100	0	0.163	0.043	100	
0.2	96.2	3.8	0.104	0.067	100	3.8
0.1	75.6	20.6	0.026	0.268	100	20.6
0.03	44.9	30.7	0.00235	2.98	100	30.7
0.01	25.6	19.3	0.000261	26.8	100	19.3
0.003	15.4	10.2	0.0000235	298.0	67	6.8
0.001	10.3	5.1	0.00000261	2680.0	7	0.4
		10.3				0
						81.6

*Calculated from Equation A-3, Appendix A, for viscosity at 60° F with $g = 32.2$ ft per sec^2, $\mu = 0.00002359$ slugs per foot-second, and $\rho = 1.94$ and $\rho_1 = 5.14$ slugs/ft^3. Substituting into Equation A-3 yields the relation

$$v = 2.61\ D^2$$

Where v = fall velocity in ft per sec

D = Diameter of particle in mm

The most common coagulant used in water treatment is alum, aluminum sulfate $Al_2\ (SO_4)_3 \cdot 18\ H_2O$, which, when added to water in the presence of calcium bicarbonate, reacts to form a gelatinous precipitate. This flocculant settles rapidly, gathering the clay particles in the water and bringing them to the bottom. The precipitate has a positive charge that neutralizes the negative charge of the clay particles and thus permits them to aggregate. If the water does not have sufficient natural alkalinity, lime is added.

The dosage of coagulant required can range from 0.2 to 5.0 grains/gal. If lime is needed to increase alkalinity, a rate of one part lime to four parts alum is suggested. The actual amount of alum needed is determined by the following jar test. Different amounts of the chemicals are added to sample jars, stirred thoroughly, and observed for formation of the floc. Then the smallest dosage that will accomplish the desired clarification of the water is prescribed.

Treatment is made after inflow has ceased. Treating during inflow requires continuous introduction and is probably not practical on most highway construction sites. However, automatically controlled chemical introduction has been used in at least one strip-mining sediment-control works.

The chemicals are not unduly costly when compared to alternative methods. For example, a triple dosage for the mixture in the example would cost \$8.46 at 1978 prices (\$158.00/ton for alum and \$72.80/ton for lime). The cost of 10 treatments during the life of the basin would be \$84.60 for the chemicals. If application costs were double the chemical costs, then the total costs of flocculating the sediments during the period the basin is in service would be about \$250.00.

Lengthening the basin to achieve the same results seems impractical. Settling particles 0.01 mm in diameter would require a basin 2680 ft long. Still, some of the material less than 0.01 mm in diameter, representing 10.3 percent of the total sediment inflow, would pass through the basin.

SCOURING VELOCITY

The final step, calculating the velocity required to move particles on the basin bottom, is done to determine whether or not the settled material will remain in place. Equation A-6 in Appendix A will be used in this calculation. The assigned values are:

$f = 0.032$, which is equivalent to a Manning n value of 0.02 for a 3-ft hydraulic radius;

$\beta = 0.04$ [Camp (*32*, p. 913) reports that, for the beginning of bed load movement, β is about 0.04];

$S = 2.65$ (quartz particles); and

$D = 0.01$ mm for the smallest particle settled.

The velocity for incipient motion, V_c, is thus calculated to be 0.132 ft/sec. However, the average mean flow velocity is only 0.00235 ft/sec, so material 0.01 mm in diameter will not be moved. Solving Equation A-6 for the diameter of a particle that can be moved by this velocity yields a value of $D = 1.04 \times 10^{-8}$ ft or $D = 3.17 \times 10^{-6}$ mm. Essentially no bed material would be moved by the flow.

CHAPTER FOUR

BASIN DESIGN

DAM HEIGHT

The height of the dam will depend on the kind of basin chosen—whether it is a natural reservoir or an excavated pit. These vary according to construction material and spillways employed. Some variations are sketched in Figures 18 and 19. For basins equipped with a pipe spillway, the following sequence of steps must be taken to determine the total depth of the basin:

1. Determination of the depth of the sediment (accumulation before cleanout).
2. Establishment of the crest elevation of the principal spillway.
3. Establishment of the crest elevation of the emergency spillway.
4. Determination of the head on the emergency spillway.
5. Determination of the freeboard required.

For basins without pipe spillways—the rockfill dam type—steps 2 and 3 are combined. The reservoir type will be discussed first.

Natural Reservoir Basins

The depth of the sediment deposited in a natural reservoir basin (Figure 18) will depend on the volume deposited and on the geometry of the reservoir site. A topographic map would be needed for an exact determination of this depth, but the required survey for such a map would probably not be warranted for many basins. Furthermore, the assumption that the deopsit has a horizontal, plane top surface is likely not true. Again, refinement in the determination of the depth-to-volume relation for the site is not justified; an approximate method for determining the depth can be used just as well. Probably many of those engaged in earthwork calculations are aware of such methods and prefer to use them. There is another method, however, that may have some usefulness.

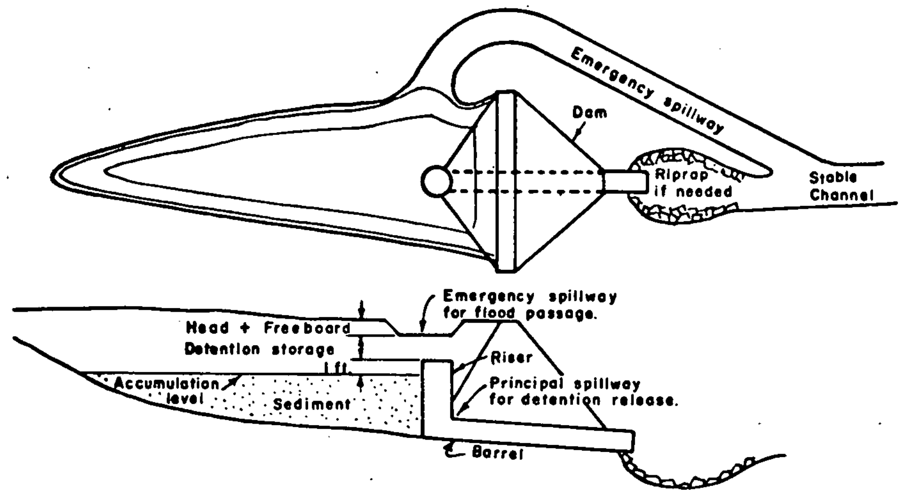
If the transverse cross-section of the reservoir site is parabolic, as many natural cross-sections are, the depth for a given volume can be estimated by using the following equation:

$$Y = [(4.5\,\overline{V}\,S\sqrt{D})/W]^{2/5} \qquad (6)$$

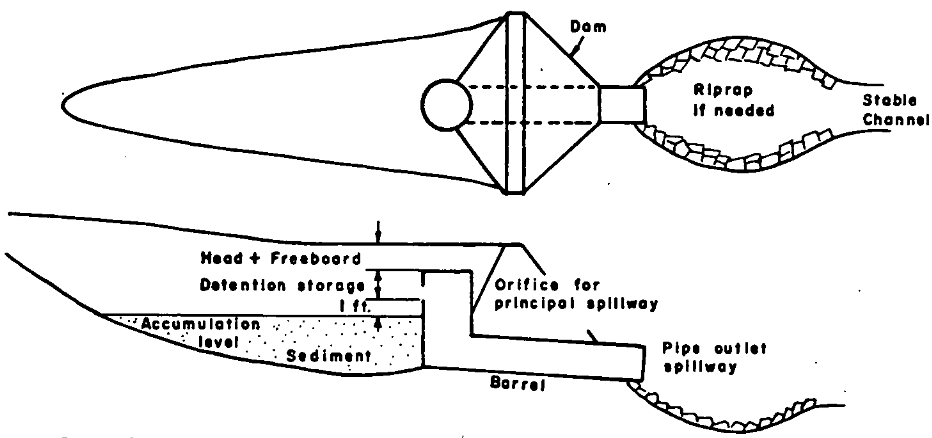
where

Y = depth in feet,
V = volume of deposition in cubic feet,
S = longitudinal floor slope of the reservoir in feet per feet,
D = depth of cross-section at the dam in feet, and
W = width of cross-section at the dam in feet.

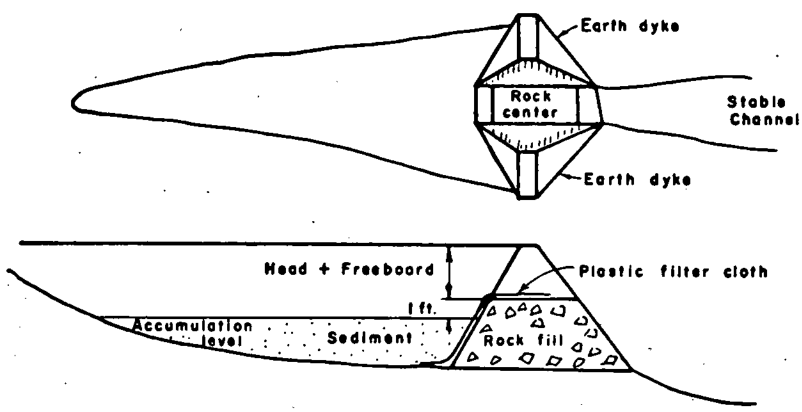
Figure 20 further defines the quantities. The dimensions D and W and the approach slope, S, are determined in the field, as is the maximum water surface elevation permissible at the location. Any pair of values for W and D can be used, but the D value that best represents the parabola should be chosen and the corresponding W value measured. With the measurement of the average lengthwise slope, S, of the reservoir bed, the three measurements determine the constants for the equation that can now be solved for reservoir depth, Y, for any given reservoir volume, V. The solution of Equation 6 is provided by the nomograph in Figure 21.



A – Sedimentation basin in natural reservoir site with pipe principal spillway and overflow weir emergency spillway.

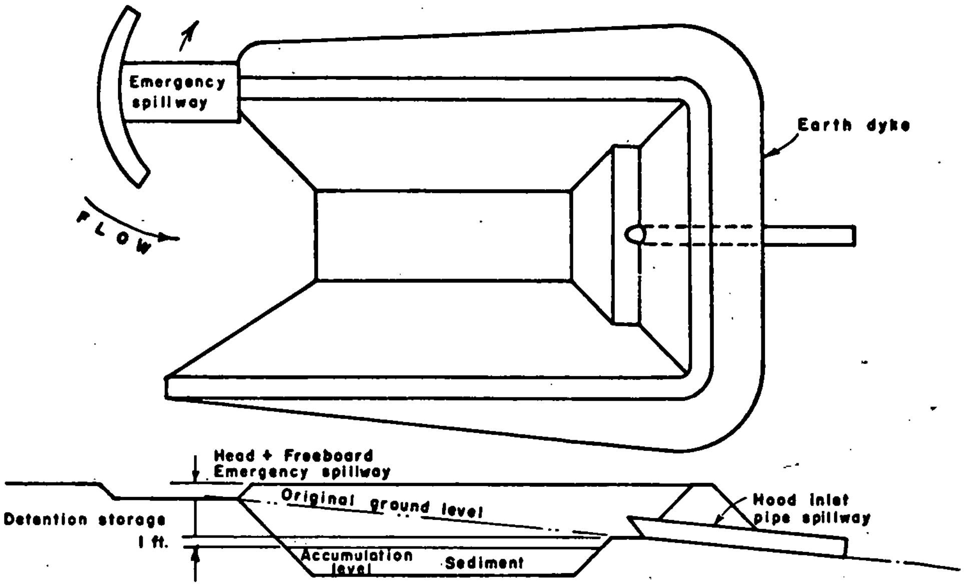


B – Sedimentation basin in natural reservoir site with pipe outlet spillway serving for principal and emergency spillways.

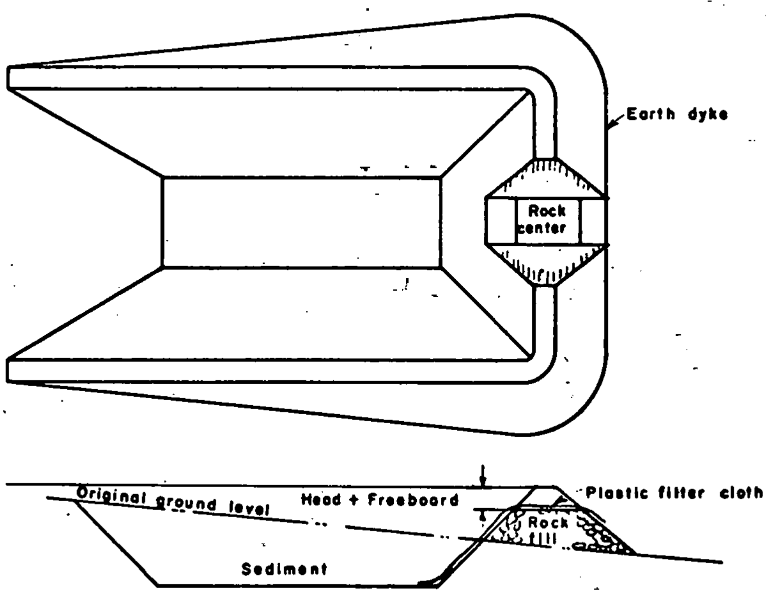


C – Sedimentation basin in natural reservoir site with porous rock fill dam for spillway.

Figure 18. Types of sedimentation basins at natural reservoir sites.



A – Excavated sediment basin with hood inlet pipe principal spillway and overflow weir emergency spillway.



B – Excavated sediment basin with porous rock fill dam for spillway.

Figure 19. Types of excavated sedimentation basins.

Basin with Pipe Spillway

It is suggested that a pipe spillway crest be set 1 ft above the expected level of the sediment deposit, although this is the designer's option. The 1 ft can be looked on as a safety factor or as a way of reducing scour of sediment deposited near the pipe.

If no runoff from the high-frequency design storm is permitted to flow over the emergency spillway, detention storage must be provided between the pipe spillway crest and the crest of the emergency spillway. The detention storage volume can equal the estimated runoff volume for a satisfactory approximation. The estimate of this volume will be covered in the section on hydrologic design.

A more accurate determination of the detention storage volume would require the routing of the inflow through the reservoir and out the principal spillway, a refinement that may not be worthwhile in many instances. This will be discussed in somewhat greater detail in the section on hydrologic design. The nomograph, Figure 21, can be used to

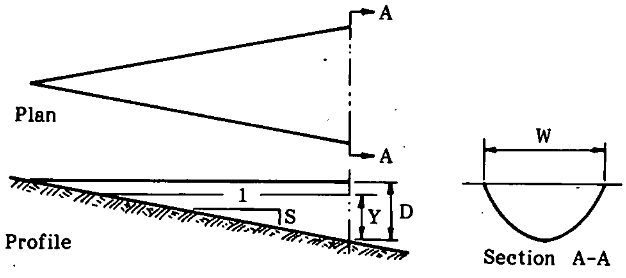


Figure 20. Dimension sketch for Equation 6.

calculate the elevation to which the pool will rise in accommodating the detention storage, if approximations are satisfactory. Otherwise, volumetric calculation procedures based on reservoir dimensions should be used. The emergency spillway crest is set at the calculated pool level.

The head on the emergency spillway may very likely be determined by the maximum water surface elevation permissible at the site. The spillway will then be sized to convey the expected design flood at this head. Estimating the design flood and sizing the spillway are discussed in subsequent sections.

Freeboard is the added height of dam to prevent overtopping by waves or to accommodate storms worse than the design storm. It is assumed that wind can cause waves when peak flow is occurring, so freeboard is added to the head. The Soil Conservation Service (*33*) suggests the following values for freeboard:

Pond length (ft)	Freeboard (ft)
600	1.0
660–1320	1.5
1320–2640	2.0

The Soil Conservation Service suggests checking state standards and specifications for local requirements. Probably 1.0 ft will be sufficient freeboard for most sedimentation basins.

The five initial design steps have been completed, and the height of the dam has been tentatively established. The next consideration is the design of the spillways.

Basin with Rockfill Dam

The permeable membrane over the face of the rockfill dam will serve as the principal spillway. The rate of flow through the membrane probably varies because of the silt

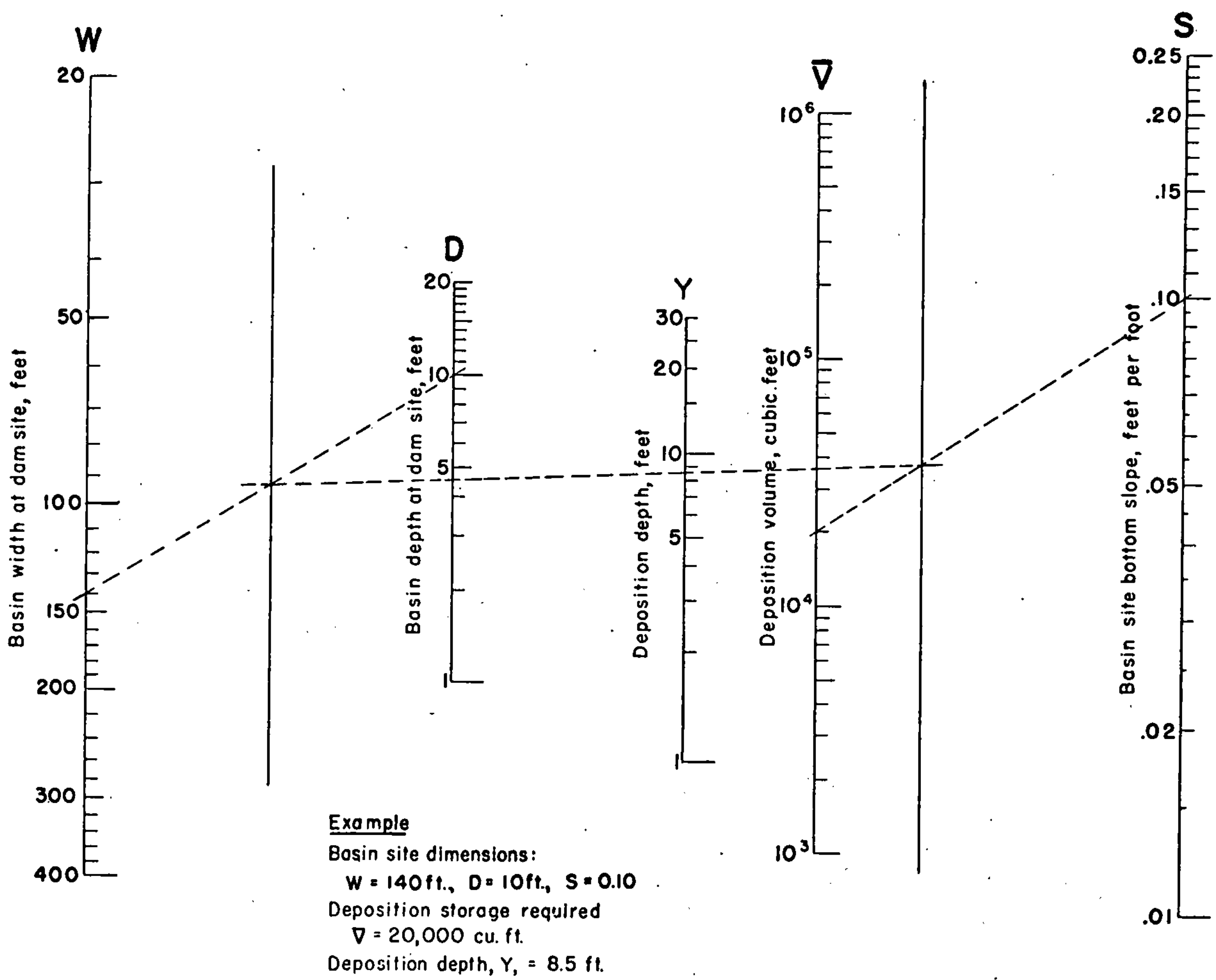


Figure 21. Nomograph for the solution of Equation 6 for storage depth in a reservoir.

that accumulates on it. However, because of the potential for some flow over the entire submerged portion of the membrane, it will be assumed that the detained runoff can be discharged in 24 hr.

Details of dam construction and membrane installation are given in Figure 22. The procedure for sizing a dam follows:

1. Add the estimated volume of sediment to the volume of runoff to be detained (the high-frequency storm).
2. Enter Figure 21 with this volume to obtain a preliminary depth figure.
3. Add 1 ft to the preliminary depth determination to fix the emergency spillway crest elevation. Head on spillway and head on freeboard are determined as before.

Excavated Pit Basins

An excavated basin (Figure 19) will generally be of simple geometry. Its volume can be calculated by using the methods employed in earthwork calculations. No nomograph is provided, nor are detailed instructions offered for these calculations. The basin's previously selected length and width will be used as a starting point. Assuming that the pipe spillway will have its invert at approximately the original ground elevation at the outlet end, the designer should then drop 1 ft below this elevation and calculate the depth to the bottom of the pit. The crest of the emergency spillway should be set to provide the required runoff storage between it and the invert of the pipe spillway. The head on the emergency spillway crest and the head on the freeboard are determined as before.

Where a permeable membrane on a rockfill serves as principal spillway, the bottom elevation of the pit is determined as previously discussed. The emergency spillway crest elevation will then be determined by adding the runoff storage depth to the ground elevation at the downstream end of the pit. The head on the emergency spillway and the freeboard will also be determined as before.

HYDROLOGIC DESIGN

Hydrologic design deals with the estimates of runoff amounts and rates that form the basis for design sizes of basins and spillways. Because the calculation of the R factor for use in the USLE was covered in the section on initial volume eroded in Chapter 3, it is not treated in this section, although it is also a hydrologic design element.

First the designer must choose the return periods. It has already been suggested that the basin be able to retain the runoff from a relatively high-frequency event and that the emergency spillway be designed for a relatively low-frequency event. By the choice of frequency, which is based on rainfall experience, the designer indicates the acceptable risk. The two risks are exceeding the storage capacity of the basin and exceeding the emergency spillway capacity.

When the designer intends to capture all the runoff so no flow occurs over the emergency spillway, a moderate risk of failure, say 50 percent, may meet the performance criterion. For a more critical situation one may want to keep the risk of failure down at 10 percent or less. For the emergency spillway, the designer may wish to set a very low risk of failure, say 1 percent. A basin cost can be calculated for each risk, and, if a benefit can be assigned, the selection of a probability may be based on a cost-benefit analysis. The cost-benefit ratio alone, however, may not be the sole criterion.

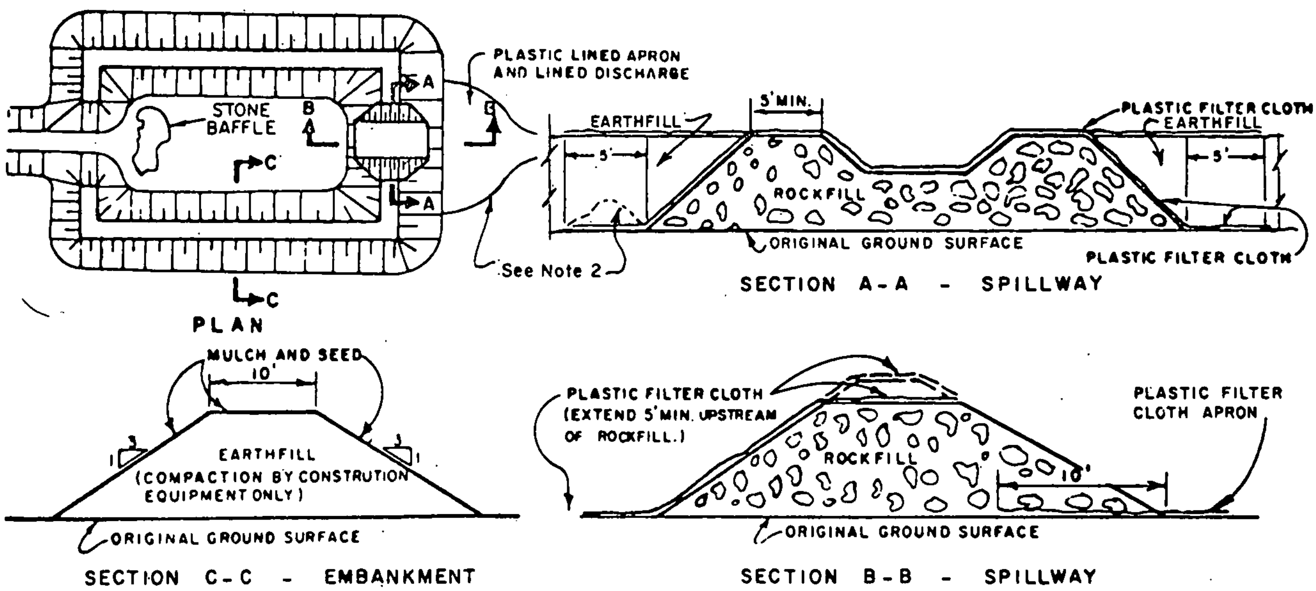


Figure 22. Suggested details for rock-dam construction and permeable membrane installations.

The probability that the return period rainfall will occur at least once during the lifetime of the sedimentation basin is given by the formula

$$P = 1 - q^n \qquad (7)$$

where

P = probability of occurring at least once during n years;
q = probability of not occurring in a particular year (for example, if the return period is 10 years, the probability of occurring in any one year is 0.1 and the probability of not occurring, q, is 0.9); and
n = lifetime of structure in years.

Return periods for various probabilities of occurrence are given in Table 6.

TABLE 6. RETURN PERIODS FOR VARIOUS PROBABILITIES OF OCCURRENCE

Probability of Occurrence	Return Period (Years) One-Year Basin	Two-Year Basin
0.01	100	200
0.05	20	39
0.10	10	19
0.20	5	9
0.50	2	3.4

The use of the table will be shown with an example. The designer, expecting the lifetime of the sedimentation basin to be 2 years, specifies that the chance of emergency spillway flow (all runoff retained or detained) will not exceed 10 percent and that the chance of exceeding the design capacity of the emergency spillway not be more than 1 percent. The table shows that a 10 percent risk requires a design return period of 19 years and a 1 percent risk requires a design return period of 200 years. The designer will use available return period data rather than use the theoretical return periods.

After choosing the return periods, the designer proceeds to the calculation of the volume of runoff to be stored and the peak flow rate to be carried by the emergency spillway. The calculation methods used will be those with which the designer is familiar or those that follow the agency's standards. Some commonly used methods will now be described.

Estimating Storm Runoff

The curve number method developed by the Soil Conservation Service (*30*) is suitable for estimating storm runoff volume. This method is based on a relation between runoff and rainfall that has curve number as a parameter. The curve number is an indicator of the runoff-producing capability of the watershed. The estimate of the curve number is based on the kind of soil, the cover or soil condition, and the antecedent moisture condition. Curve number values can be obtained from the *National Engineering Handbook* (*30*). The runoff-rainfall relation is shown in Figure 23.

Peak Flow Rate

The Rational formula is easy to use and is suitable for the design of small structures. The Rational formula is

$$Q = CiA \qquad (8)$$

where

Q = peak discharge rate in cubic feet per second;
C = a coefficient depending on the soils, cover, and topography of the contributing watershed;
i = rainfall intensity in inches per hour for a duration equal to the time of concentration and for the chosen frequency of occurrence; and
A = area of the contributing watershed in acres.

The coefficient C is numerically equal to the percentage of rainfall appearing as surface runoff, a coincidence made possible by the customary units used.

The time of concentration needed for the determination of i is estimated by measuring the length of the flow path from the most remote upstream point in the watershed to the outlet and dividing the length by the mean flow velocity. The mean velocity depends on the steepness of the slope and on the roughness of the watercourse. Values of the flow velocity (Table 7) are given in the Bureau of Reclamation's publication *Design of Small Dams* (*34*). A direct solution for time of concentration can be obtained from Figure 24.

The estimated time of concentration is the design storm duration. One can then enter a rainfall intensity-duration-frequency curve (*35*) for the locale with this duration and read the rainfall intensity for the selected return period—25 years in this case.

The coefficient C depends on the type of drainage area. Values for C suggested by Chow (*36*) are given in Table 8. A similar table with identical values is in an American Society of Civil Engineers manual (*37*). An important omission in Table 8, insofar as this report is concerned, is a C value for bare earth typical of areas disturbed by highway construction.

A table in Rouse's *Engineering Hydraulics* (*38*) gives values of C for barren areas suitable for areas disturbed by highway construction. The table is reproduced here as Table 9. A handbook on drainage (*39*) gives a range of values from 0.46 to 0.65 for impervious soils (heavy) on 1 to 2 percent slopes. These values are offered for guidance to the designer, who must make the final decision.

In the sample problem in this synthesis, a C value of 0.8 is used for the disturbed area and that of 0.2 is used for the undisturbed area (each area is 2 acres). For a watershed having different types of land use for the subareas, an average value of C should be calculated by area by weighting the C values for the subareas. Thus the average C value in this example is 0.5.

The sample watershed used to illustrate USLE will also be used to illustrate the calculation of the peak rate of the

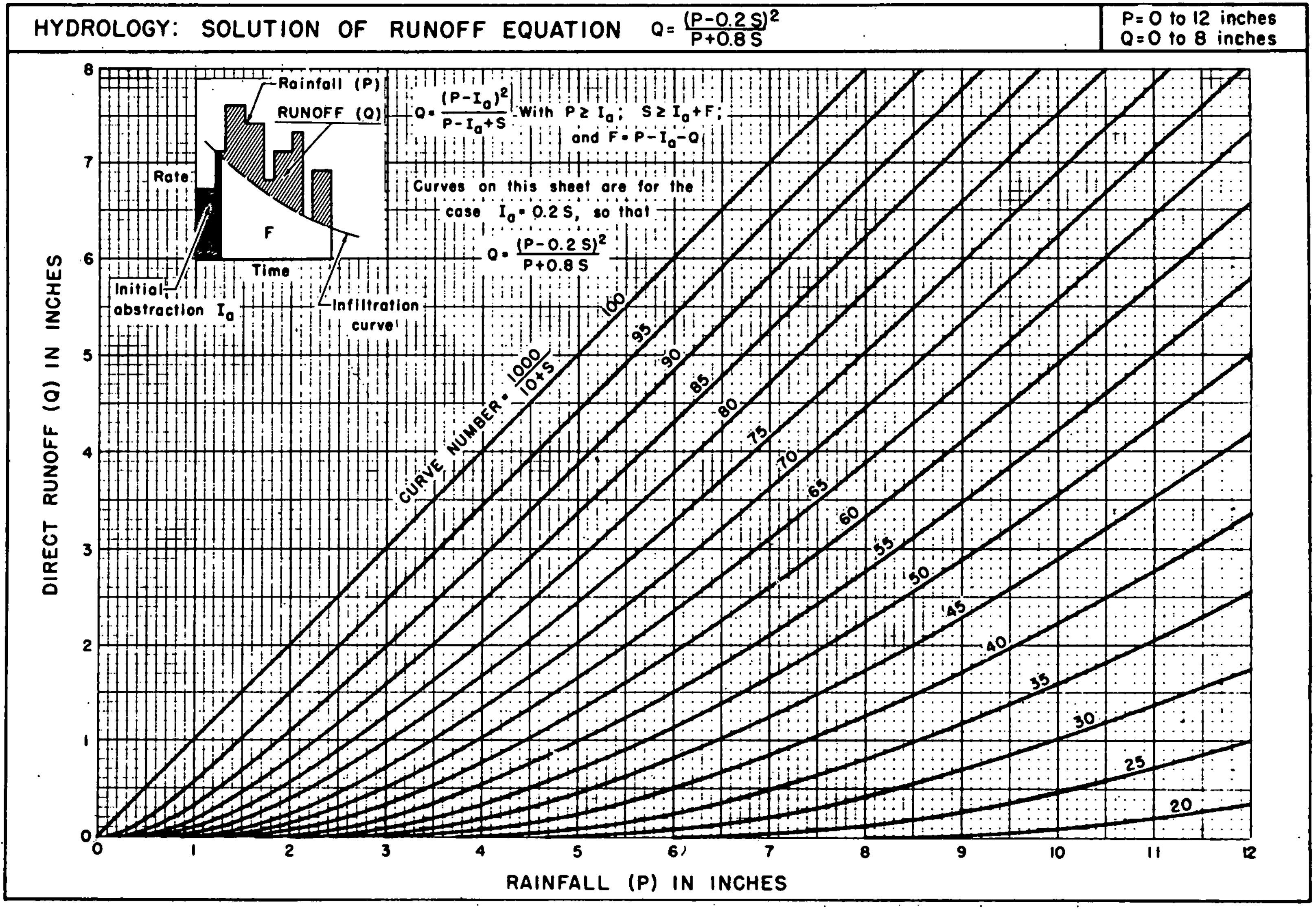


Figure 23. Solution of the runoff equation.

TABLE 7. AVERAGE FLOW VELOCITIES IN FEET PER SECOND FOR USE IN ESTIMATING TIME OF CONCENTRATION

U. S. Navy - Technical Publication Navdocks TP-PW-5 Table 8B, March 1953	
Average slope of channel from farthest point to outlet, in percent	Average velocity, feet per second
1 to 2	2.0
2 to 4	3.0
4 to 6	4.0
6 to 10	5.0

Texas Highway Department Rational Design of Culverts and Bridges, October 1946			
Slope in percent	Average velocity, feet per second		
	Woodlands (upper portion watershed)	Pastures (upper portion watershed)	Natural channel not well defined
0 - 3	1.0	1.5	1.0
4 - 7	2.0	3.0	3.0
8 - 11	3.0	4.0	5.0
12 - 15	3.5	4.5	8.0

25-year flood. The elements entering into the calculation are:

$l =$ length of flow travel (1000 ft),
$V =$ estimated mean velocity (3 ft/sec),
$T_c =$ time of concentration ($1000/3 = 333$ sec $= 5.5$ min),
$i =$ for Oklahoma City (25 yr, 5.5 min) $\times$ 7.5 in./hr,
$C = 0.5$,
$A = 4$ acres, and
$Q = 0.5 \times 7.5 \times 4 = 15$ ft^3/sec.

The discharge rate just calculated is the peak rate of the inflow hydrograph. It is not the peak discharge rate for the emergency spillway because of the storage effect of the sedimentation basin. To obtain the peak outflow rate through the emergency spillway it is necessary to route the inflow hydrograph through the sedimentation basin. This was done for an example by assuming a triangular inflow hydrograph and by preselecting an emergency spillway, in this case a 10-ft-long weir. The method used was the U.S. Geological Survey method (*40*). The routing showed that the peak discharge rate was reduced from 15 to 13.8 ft^3/sec, a reduction of 8 percent. This is the reduction for the one example only and is not necessarily the amount to be expected for other basins. However, it does indicate that for small sedimentation basins the routing calculation is hardly worth the effort.

Thus it is suggested that the peak inflow rate be used for emergency spillway design. The designer who sees a possibility for appreciable savings by using a smaller, routed design discharge may wish to do the routing calculation.

PRINCIPAL (PIPE) SPILLWAY

The principal spillway drains the floodwater temporarily stored between its crest level and the level of the emergency spillway. It is generally used on temporary and permanent sedimentation basins but not necessarily on expedient basins. However, an expedient basin may need some way of draining the ponded water so the basin can be ready for another storm or so the dam can be removed.

The designer will need to select spillway drain time. The longer the time the more efficient the basin, and the shorter the time the sooner the basin is ready for the next flood.

Pipes are normally used for principal spillways, although on occasion they are also used for emergency spillways (Figure 18*B*). Three types of pipe spillways are described.

TABLE 8. VALUES OF RUNOFF COEFFICIENT C (*36*)

Type of drainage area	*Runoff coefficient, C*
Lawns:	
Sandy soil, flat, 2%	0.05–0.10
Sandy soil, average, 2–7%	0.10–0.15
Sandy soil, steep, 7%	0.15–0.20
Heavy soil, flat, 2%	0.13–0.17
Heavy soil, average, 2–7%	0.18–0.22
Heavy soil, steep, 7%	0.25–0.35
Business:	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential:	
Single-family areas	0.30–0.50
Multi units, detached	0.40–0.60
Multi units, attached	0.60–0.75
Suburban	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial:	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets:	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
Roofs	0.75–0.95

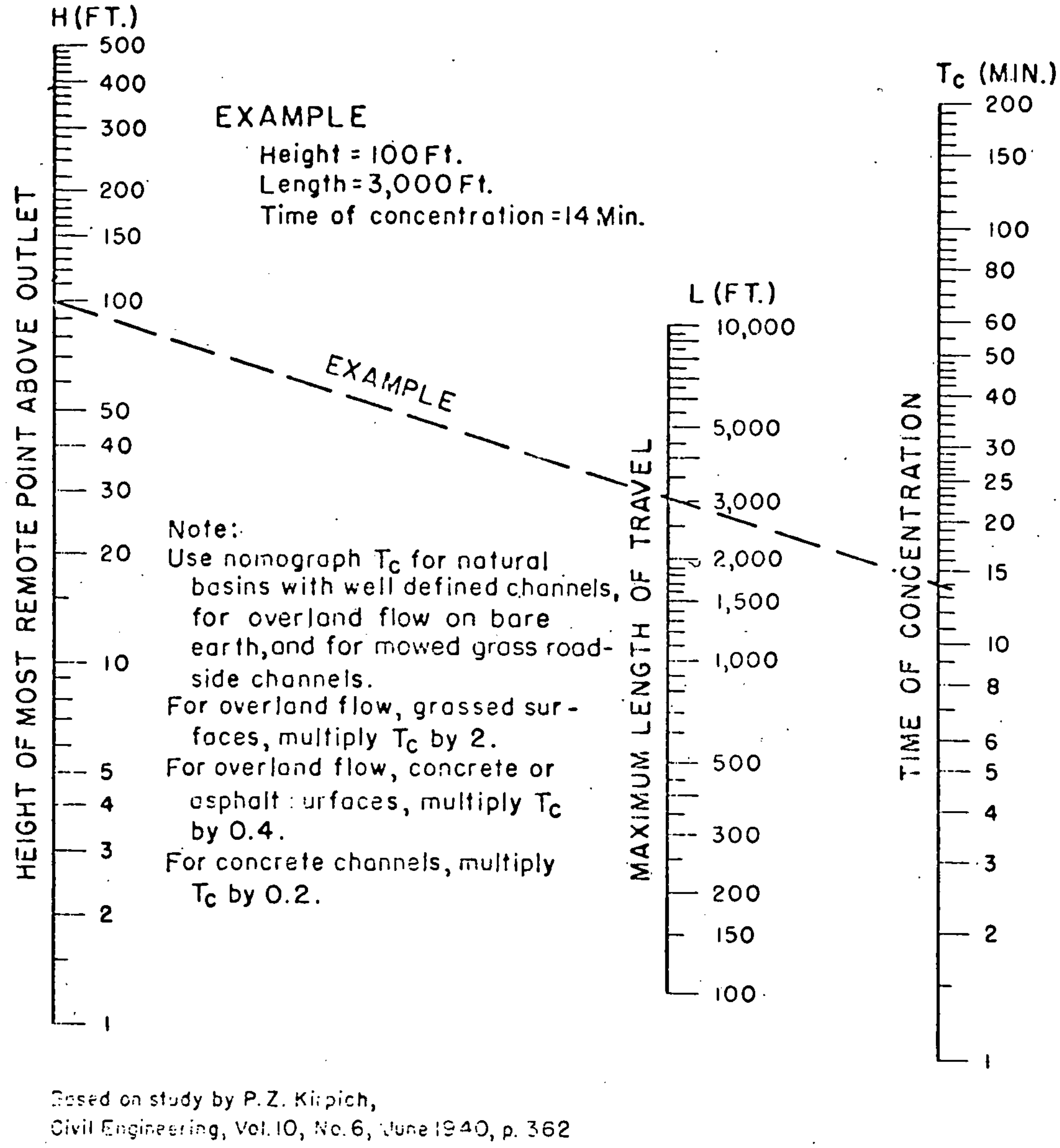


Figure 24. Time of concentration of small drainage basins.

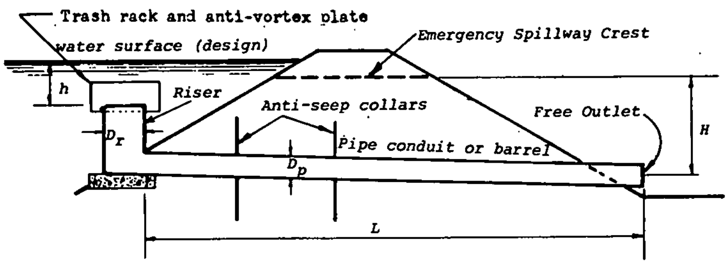
Spillway with Drop Inlet

The pipe spillway with a drop inlet entrance (Figure 25) is often used where sediment accumulation is anticipated in a reservoir. The components of this spillway, which is well suited for natural reservoir sedimentation basins, are a pipe (the barrel) extending through the dam beyond the toe of the fill and a vertical pipe entrance (the riser) connected to the upper end of the barrel. The connection of the two pipes and the pipes themselves must be watertight.

The flow rate through the spillway will depend on the flow mode, whether weir or full pipe flow. Figure 26 is a schematic rating curve for a drop inlet pipe spillway that illustrates these two modes. Other flow modes are possi-

TABLE 9. RUNOFF COEFFICIENTS FOR AGRICULTURAL AREAS *(38)*

Type of Area	Runoff Coefficient
Steep barren areas	0.90
Rolling barren areas	0.80
Rolling meadows	0.65
Timberlands	0.50
Orchards	0.40
Upland farms	0.30



H = Head on pipe spillway (pipe flow), ft. (centerline of outlet to emergency spillway crest or to design high water if no emergency spillway)
h = Head over riser crest, ft.
L = Length of pipe in ft.
D_p = Diameter of pipe conduit (barrel)
D_r = Diameter of riser

Figure 25. Pipe spillway with drop inlet entrance.

TABLE 10. RISER DIAMETERS FOR VARIOUS CONDUIT DIAMETERS

Conduit Diameter (in.)	Riser Diameter (in.)
8 - 12	18
15	21
18	24
21	30
24	30
30	36
36	48
42	54
48	60

ble (*41*), but weir and full pipe should be the only operating modes if the spillway is properly proportioned.

Inlet proportions are given in Table 10. The riser height should be 5 times the conduit diameter where the conduit slope is greater than the friction slope. For a conduit slope equal to or less than the friction slope, the required riser height is twice the conduit diameter.

The rating equation for the weir flow mode is

$$Q = CLh^{3/2} \tag{9}$$

where

$Q =$ discharge rate in cubic feet per second,
$C =$ a coefficient usually set equal to 3.1,
$L =$ crest length in feet, and
$h =$ head over crest in feet.

For full pipe flow it is

$$Q = (\pi D^2/4)\sqrt{2gH/[1 + K_e + K_b + (185.1\, n\, l/D^{4/3})]} \tag{10}$$

where

$Q =$ discharge rate in cubic feet per second,
$D =$ diameter of barrel in feet,
$g =$ acceleration of gravity (52.2 ft/sec^2),
$H =$ total head from basin highwater to outlet centerline in feet,
$K_e + K_b =$ entrance and bend loss coefficients of 1.00 for corrugated pipe and 0.65 for concrete pipe,
$n =$ Manning n, or 0.025 for corrugated pipe and 0.013 for concrete pipe, and
$l =$ length of barrel in feet.

The discharge equation is not directly solvable for D, so a trial-and-error solution is suggested in which D values are substituted for standard pipe sizes until the equation is satisfied for the given value of Q. For those who prefer tables, these are provided for full pipe flow in Appendix B.

The initial selection of the diameter of the pipe spillway barrel is based on the average discharge rate. If the spillway is in full pipe flow during most of the draw-down period, the drainage will be accomplished in approximately the time allowed. However, if the spillway is in the weir flow mode for much of the draw-down range, drainage time will be much longer than planned.

To check for such a possibility, the head-discharge curves for the spillway should be plotted. If the head at which the flow changes from weir to full pipe flow is less than the depth provided between the principal and emergency spillway crests, the design assumption of uniform flow rate is realistic and the design is adequate. If not, the spillway pipe size will need to be increased. The design process should be repeated until the draw-down time requirement is met.

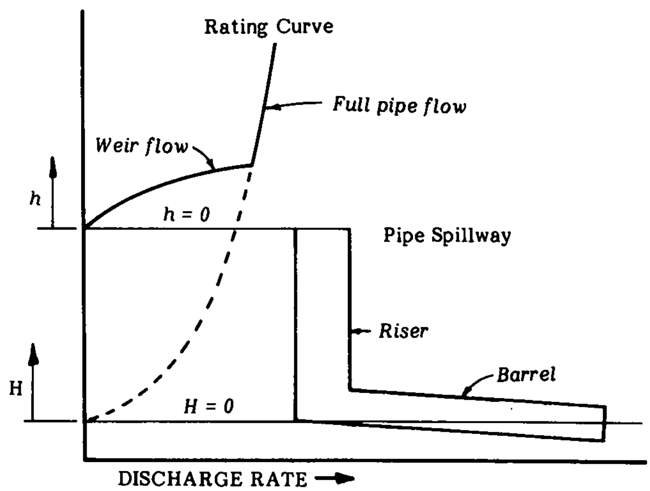


Figure 26. Schematic rating curve for a riser entrance pipe spillway that shows the two modes of operation, weir and pipe.

The riser inlet may need a trash rack if debris in the watershed could reach and plug the inlet, a possibility where trees and shrubs are being cleared away. However, if there is little debris, the rack should be omitted. The concentric trash rack shown in Figure 27 would be suitable for most installations. Dimensions are given in Appendix C. The solid skirt provided by the concentric outside pipe stops floating trash. An open rack constructed of a grid of bars is prone to becoming clogged. For information on other types of trash racks, see the study by Hebaus and Gwinn (*42*), who tested many of the different styles used by the Soil Conservation Service.

A means of draining the sedimentation basin is highly desirable because cleanout would then be much simpler. One drainage system is shown in Figure 28. However, clogging of the sand filter can occur if the sediment deposit contains considerable silt and clay. A modification of this system would be to use french drains instead of pipe drains and to rework the french drains as needed.

Perforating the riser to provide drainage has been suggested. If this is done, the perforated section of the riser should be surrounded with a sand-gravel filter pack to prevent sediment outflow through the perforations. If no drainage system is used, a small drain hole should be cut into the riser a short distance above the expected sediment deposition level.

Spillway with Principal Orifice in Riser

A riser that serves as both a principal spillway and an emergency spillway has an orifice as the principal spillway (see Figure 18*B*). The diameter of the orifice can be estimated by applying the formula for the drainage of a prismatic tank:

$$t = (2 \times \text{volume}) / \text{initial } Q \qquad (11)$$

where t is drainage time in seconds. The formula for orifice flow is

$$Q = C\pi(D^2/4)\sqrt{2gh}, \qquad (12)$$

where

C = the discharge coefficient (0.6 will be used here),
D = diameter of the orifice in feet,
h = head over the center of the orifice in feet, and
g = acceleration of gravity in feet per second per second.

A simultaneous solution of Equations 11 and 12 yields the following equation for orifice diameter, d (in inches):

$$d = (8.73\sqrt{\text{volume}})/(h^{1/4}\sqrt{t}). \qquad (13)$$

For a runoff volume of 30 492 ft³, a depth, h, of 4 ft, and a draw-down of 24 hr, the orifice size is

$$d = (8.73/4^{0.25})\sqrt{30\,492/24 \times 3600} = 3.67 \text{ in.}$$

Spillway with Hooded Inlet Entrance

A straight pipe with a hooded inlet entrance is well suited for a principal spillway for excavated sediment traps. Figure 29 shows a typical hooded inlet spillway. It is a straight pipe with the entrance cut off at an angle. The pipe is installed with the long dimension at the top. The overhang,

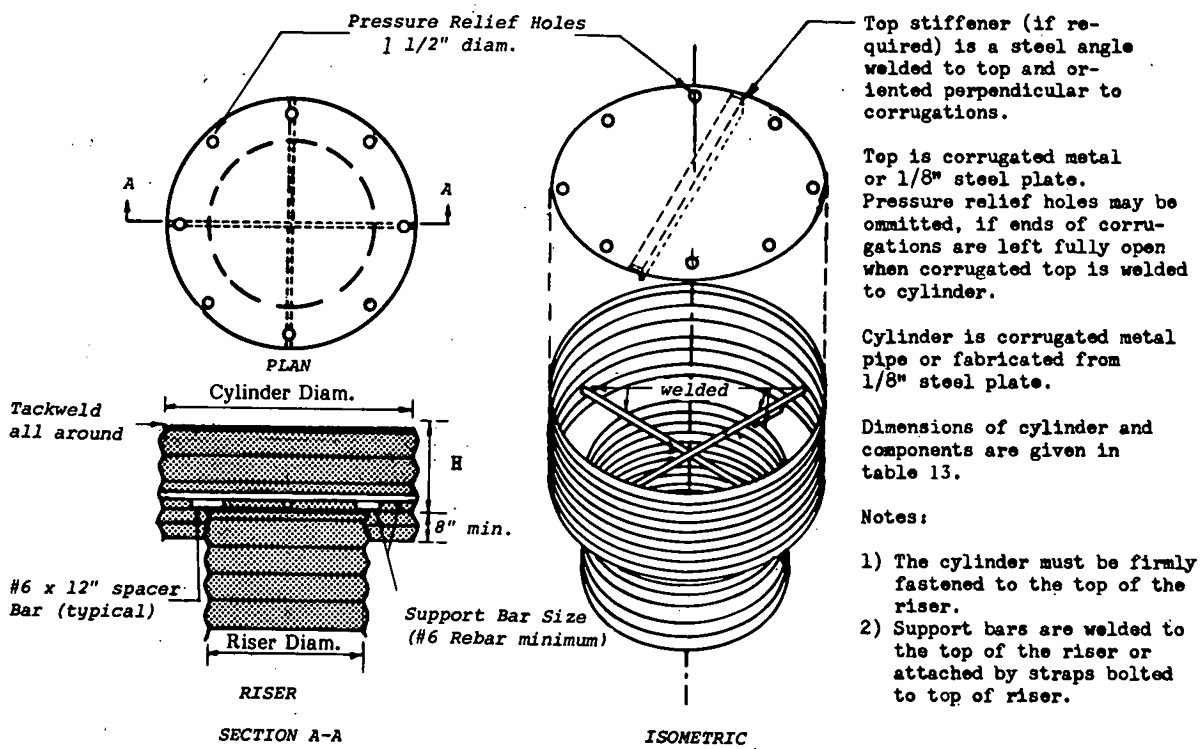


Figure 27. Concentric cylinder trash rack and antivortex plate for corrugated pipe riser.

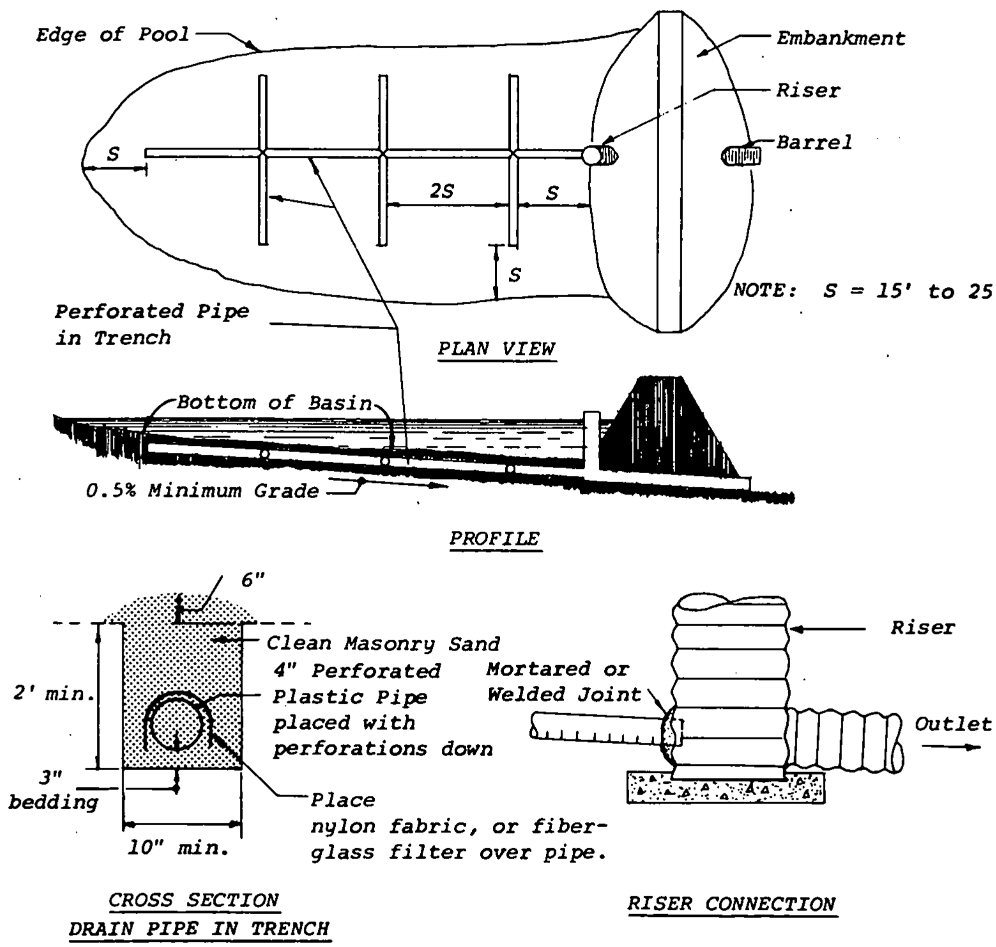


Figure 28. Subsurface drain for dewatering a sedimentation basin.

or hood, created by the miter cut causes the pipe to flow full at relatively low heads over the entrance regardless of the pipe slope. An antivortex device at the entrance is needed, however, to ensure full flow. Details of a typical hood inlet entrance are shown in Figure 30.

Velocity at the entrance to the pipe is high and may scour the nearby berm and bank. Riprap can be placed around the entrance to eliminate this scour, or the pipe entrance can be projected farther out into the pool, which will remove high velocities around the inlet away from the face of the dam. However, this can interfere with basin cleanout.

Capacities of hooded inlet spillways are given in Appendix B.

A hooded inlet must be located so that water can approach freely from both sides. A central location in the basin is therefore preferred. A trash guard should also be provided, although no special form of guard has been developed. A cage type seems most practical, but cage faces should be kept about two diameters away from the entrance to be out of the high-velocity region.

Scour Prevention Below Pipe Spillways

The discharge from a pipe spillway will create a scour hole at the outlet. Considerable sediment can be produced as the hole is formed. If the sediment from the scour hole

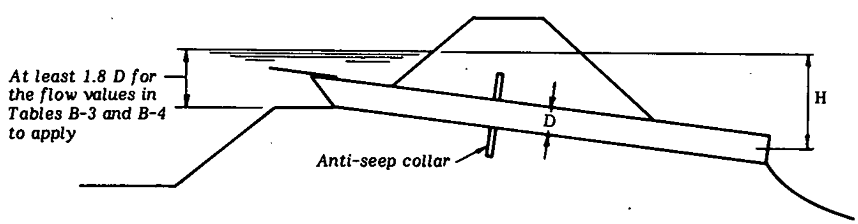
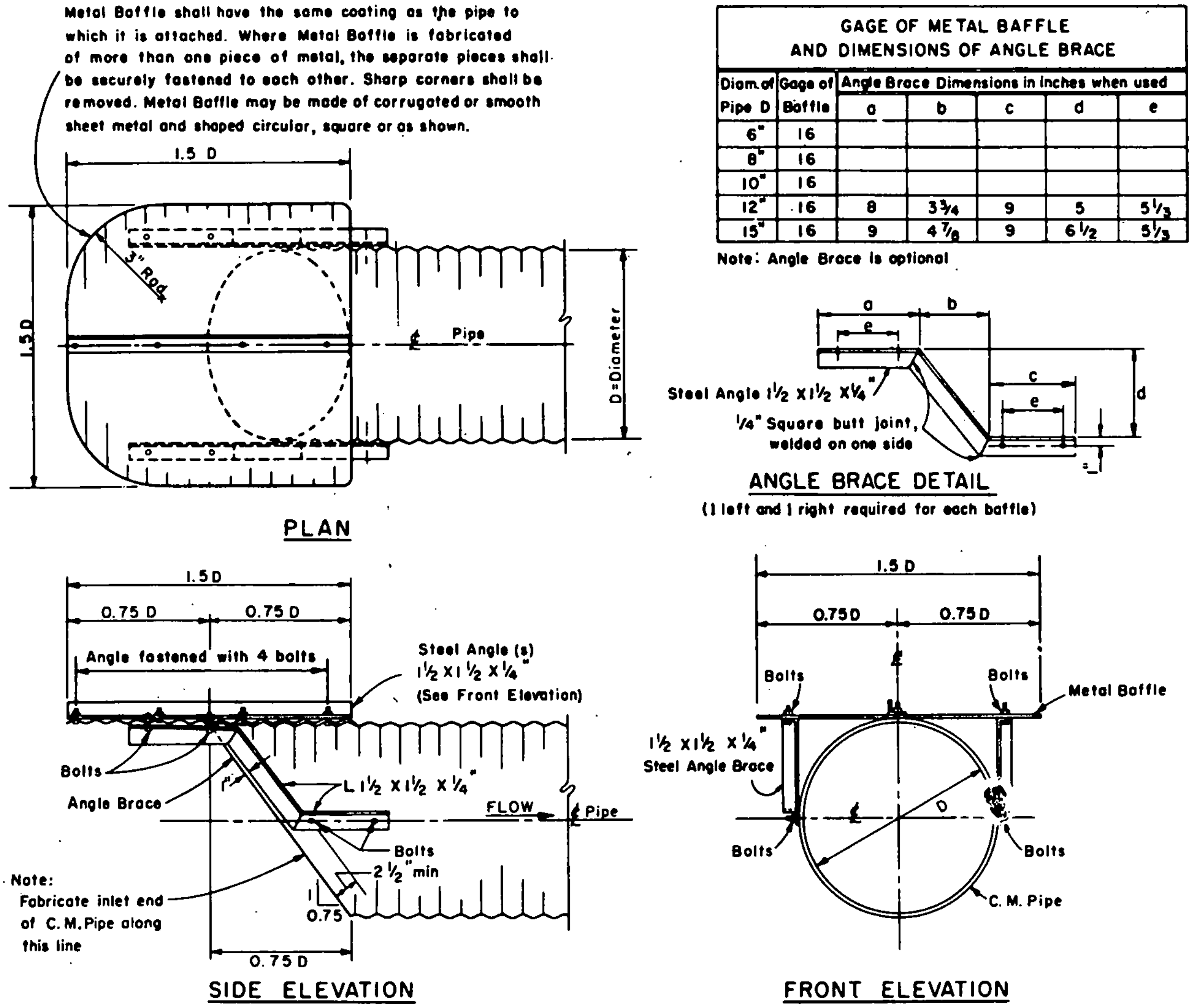


Figure 29. A straight spillway with hood inlet.



GAGE OF METAL BAFFLE AND DIMENSIONS OF ANGLE BRACE

Diam. of Pipe D	Gage of Baffle	Angle Brace Dimensions in Inches when used: a	b	c	d	e
6"	16					
8"	16					
10"	16					
12"	16	8	3¾	9	5	5⅓
15"	16	9	4⅞	9	6½	5⅓

Notes:
All bolts shall be ⅜" X 1½" with nut and split washers.
All holes for bolts shall be drilled 7/16" diameter.
All nuts, bolts, and washers shall be galvanized, cadmium plated, or stainless steel.
All cuts shall be saw or shear cuts.
Holes in the angle brace shall be spaced and located to match corrugations in pipe and baffle.
Steel angles shall be galvanized.
All galvanizing damaged by cutting, drilling or welding shall be repaired by painting with two (2) coats of zinc dust-zinc oxide primer.

Figure 30. Details of a typical hood inlet and baffle for 6- to 15-in. diameter corrugated metal pipe.

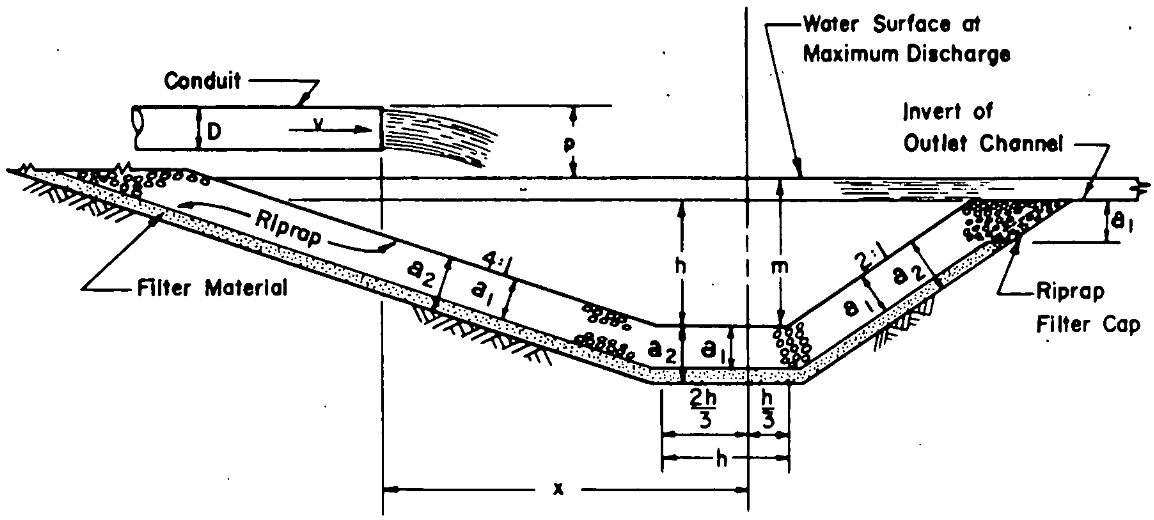
cannot be tolerated downstream, a stilling basin or an energy dissipator should be constructed. An example of a stilling basin is shown in Figure 31. The Soil Conservation Service has diagrams for the solution of the equations.

EMERGENCY WEIR SPILLWAYS

A weir spillway discharging into an open channel, and conveying the flood flow to the drainage below can provide a suitable emergency spillway. The best location for the weir is off to one side of the dam, and the crest of the spillway and the channel below should be excavated into the adjoining hillside. In this location the spillway crest is more stable against breaching than a spillway over a fill or over a dam. Where site conditions do not allow a side location, the spillway will need to be brought over the dam, but then special protection against erosion of the dam must be provided.

Regardless of where the spillway entrance is located, the hydraulics are the same unless the spillway crest material should change and influence the head-discharge relation.

In the following discussion, crest roughness will be represented by a Manning n of 0.04, which is typical of many grass-covered surfaces and some riprap linings. If the crest



Stilling Basin - Definition Sketch

NOMENCLATURE

- a ≡ thickness of riprap or total thickness of riprap and filter material, ft
- a_1 ≡ thickness of riprap, ft
- a_2 ≡ total thickness of riprap and filter material, ft
- d ≡ size of riprap of which 50 percent by weight is smaller, ft
- D ≡ inside diameter of conduit, ft
- h ≡ depth of stilling basin below invert of outlet channel, ft
- m ≡ depth of water in the stilling basin at the maximum conduit discharge, ft
- p ≡ vertical distance from the inside crown of the conduit to the water surface in the stilling basin at the maximum conduit discharge, ft
- v ≡ mean velocity in the conduit for full pipe flow at maximum discharge, ft/sec
- V_a ≡ volume between a horizontal plane at the invert of the outlet channel and a surface at a thickness = a below the exposed riprap surface, cu yds
- $\not V_{a_1}$ ≡ volume of riprap below a horizontal plane at the invert of the outlet channel exclusive of the volume in the Riprap Filter Cap, cu yds
  $= V_{a=a_1} - V_{a=o}$
- $\not V_{a_2}$ ≡ volume of filter material below a horizontal plane at the invert of the outlet channel including the volume in the Riprap Filter Cap, cu yds
  $= V_{a=a_2} - V_{a=a_1}$
- $\not V_{rfc}$ ≡ volume in the Riprap Filter Cap below a horizontal plane at the invert of the outlet channel, cu yds
- x ≡ horizontal distance from the outlet end of the conduit to the center of the stilling basin, ft

EQUATIONS

For determining the depth of the stilling basin,

$$\frac{h}{D^{1/3}} = \left[0.148\,\frac{Q}{D\,d^{1/2}} - 1.82(d)\right]^{2/3}$$

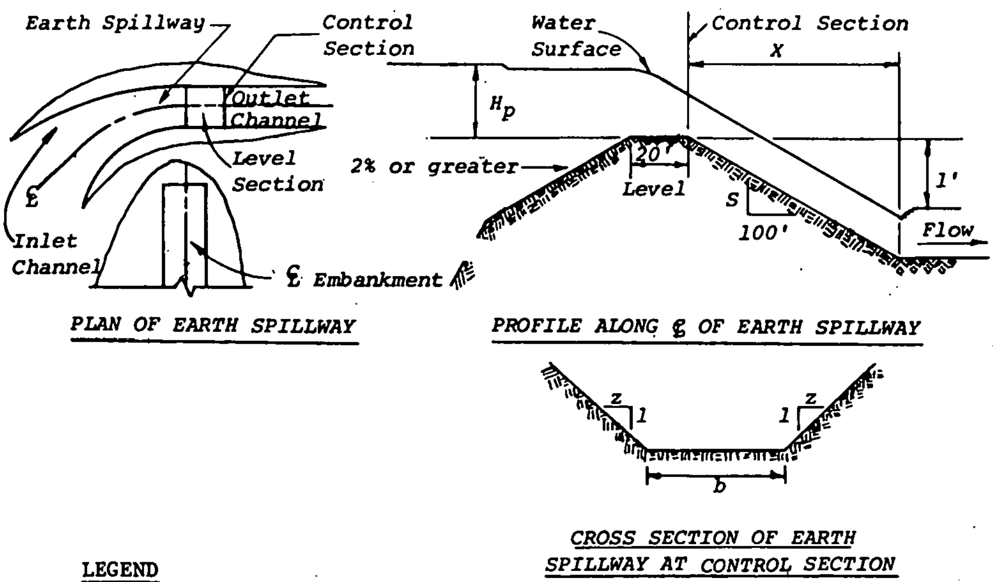
For determining the position of the stilling basin, assuming the conduit is horizontal at the outlet,

$$\frac{x}{\sqrt{p}} = \sqrt{\frac{v^2}{2g}}\left[\sqrt{1 + \frac{m}{p}} + 1 + \frac{m}{2p}\right]$$

For determining the volumes in the stilling basin,

$$V_a = 2\pi(1.167h + 1.06a)^3 - 0.029(h + 0.36a)^3$$

Figure 31. Dimensions of plunge pool stilling basin for pipe spillway cantilever outlets.



LEGEND

n = Manning's Coefficient of Roughness.
H_p = Difference in Elevation between Crest of Earth Spillway at the Control Section and Water Surface in Reservoir, in feet.
b = Bottom Width of Earth Spillway at the Control Section, in feet.
Q = Total Discharge, in cfs.
V = Velocity, in feet per second, that will exist in Channel below Control Section, at Design Q, if constructed to slope (S) that is shown.
S = Flattest Slope (S), in %, allowable for Channel below Control Section.
X = Minimum Length of Channel below Control Section, in feet.
z = Side Slope Ratio.

NOTES:

1) For a given H_p a decrease in the exit slope from S as given in the table decreases spillway discharge but increasing the exit slope from S does not increase discharge. If an exit slope (S_e) steeper than S is used, then velocity (V_e) in the exit channel will increase according to the following relationship:

$$V_e = V\left(\frac{S_e}{S}\right)^{0.3}$$

2) Data to right of heavy vertical lines on drawings should be used with caution, as the resulting sections will be either poorly proportioned or have velocities in excess of 6 ft/sec.

Figure 32. Sketch of a weir spillway and explanations for Table 11.

surface should be smoother, say if soil cement or asphalt cement were used for surface protection, the discharge would be increased for a given head. If this effect were ignored, the design would be on the conservative side in terms of capacity, but stability could be adversely affected because of higher flow velocity. However, the smoother surfaces are usually associated with protective coverings that can withstand higher velocities than a grassed surface. If these various aspects are taken into consideration, the design table, based on a Manning *n* of 0.04, will generally provide a satisfactory spillway.

A sketch of a weir spillway is shown in Figure 32 along with explanations of the accompanying Table 11. The correction factors for this table are 1.25 for $n = 0.02$, 1.15 for $n = 0.03$, and 0.85 for $n = 0.06$.

The relation between discharge rate and the head and bottom width of the spillway is affected by the roughness and length of the spillway crest. No simple weir flow equation can define this relation. It is determined by water surface profile calculations from the control section upstream into the reservoir or basin. The results are then given in tabular form.

Table 11 was prepared for a crest length of 20 ft, although this is not a rigid requirement. For expedient basins the crest length could be quite short, 10 ft or even less. For permanent basins greater crest lengths are used to gain

TABLE 11. DESIGN DATA FOR WEIR SPILLWAYS WITH 20-FT CREST, 2:1 SIDE SLOPES, VEGETATED, WITH A MANNING *N* OF 0.04

STAGE (H_p) IN FEET	SPILLWAY VARIABLES	BOTTOM WIDTH (b) IN FEET																
		8	10	12	14	16	18	20	22	24	26	28	30	32	34	36	38	40
0.5	Q	6	7	8	10	11	13	14	15	17	18	20	21	22	24	25	27	28
	V	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	S	3.9	3.9	3.9	3.9	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	X	32	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33	33
0.6	Q	8	10	12	14	16	18	20	22	24	26	28	30	32	34	35	37	39
	V	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	S	3.7	3.7	3.7	3.7	3.6	3.7	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	X	36	36	36	36	36	36	37	37	37	37	37	37	37	37	37	37	37
0.7	Q	11	13	16	18	20	23	25	28	30	33	35	38	41	43	44	46	48
	V	3.2	3.2	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
	S	3.5	3.5	3.4	3.4	3.4	5.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4	3.4
	X	39	40	40	40	41	41	41	41	41	41	41	41	41	41	41	41	41
0.8	Q	13	16	19	22	26	29	32	35	38	42	45	46	48	51	54	57	60
	V	3.5	3.5	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6	3.6
	S	3.3	3.3	3.3	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2	3.2
	X	44	44	44	44	45	45	45	45	45	45	45	45	45	45	45	45	45
0.9	Q	17	20	24	28	32	35	39	43	47	51	53	57	60	64	68	71	75
	V	3.7	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8	3.8
	S	3.2	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
	X	47	47	48	48	48	48	48	48	48	48	49	49	49	49	49	49	49
1.0	Q	20	24	29	33	38	42	47	51	56	61	63	68	72	77	81	86	90
	V	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	S	3.1	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
	X	51	51	51	51	52	52	52	52	52	52	52	52	52	52	52	52	52
1.1	Q	23	28	34	39	44	49	54	60	65	70	74	79	84	89	95	100	105
	V	4.2	4.2	4.2	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3	4.3
	S	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	55	55	55	55	55	55	55	56	56	56	56	56	56	56	56	56	56
1.2	Q	28	33	40	45	51	58	64	69	76	80	86	92	98	104	110	116	122
	V	4.4	4.4	4.4	4.4	4.4	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5
	S	2.9	2.9	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8	2.8
	X	58	58	59	59	59	59	59	59	60	60	60	60	60	60	60	60	60
1.3	Q	32	38	46	53	58	65	73	80	86	91	99	106	112	119	125	133	140
	V	4.5	4.6	4.6	4.6	4.6	4.6	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
	S	2.8	2.8	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7	2.7
	X	62	62	62	63	63	63	63	63	63	63	63	64	64	64	64	64	64
1.4	Q	37	44	51	59	66	74	82	90	96	103	111	119	127	134	142	150	158
	V	4.7	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.9	4.9	4.9	4.9	4.9	4.9	4.9	4.9
	S	2.8	2.7	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6
	X	65	66	66	66	66	67	67	67	67	67	67	68	68	68	68	68	69
1.5	Q	41	50	58	66	75	85	92	101	108	116	125	133	142	150	160	169	178
	V	4.8	4.9	4.9	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.0	5.1	5.1	5.1
	S	2.7	2.7	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.6	2.5	2.5	2.5
	X	69	69	70	70	71	71	71	71	71	71	71	72	72	72	72	72	72
1.6	Q	46	56	65	75	84	94	104	112	122	132	142	149	158	168	178	187	197
	V	5.0	5.1	5.1	5.1	5.1	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2	5.2
	S	2.6	2.6	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	72	74	74	75	75	76	76	76	76	76	76	76	76	76	76	76	76
1.7	Q	52	62	72	83	94	105	115	126	135	145	156	167	175	187	196	206	217
	V	5.2	5.2	5.2	5.3	5.3	5.3	5.3	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4	5.4
	S	2.6	2.6	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
	X	76	78	79	80	80	80	80	80	80	80	80	80	80	80	80	80	80
1.8	Q	58	69	81	93	104	116	127	138	150	160	171	182	194	204	214	226	233
	V	5.3	5.4	5.4	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.5	5.6	5.6	5.6	5.6	5.6	5.6
	S	2.5	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	80	82	83	84	84	84	84	84	84	84	84	84	84	84	84	84	84
1.9	Q	64	76	88	102	114	127	140	152	164	175	188	201	213	225	235	248	260
	V	5.5	5.5	5.5	5.6	5.6	5.6	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7	5.7
	S	2.5	2.5	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.4
	X	84	85	86	87	88	88	88	88	88	88	88	88	88	88	88	88	88
2.0	Q	71	83	97	111	125	138	153	164	178	193	204	218	232	245	256	269	283
	V	5.6	5.7	5.7	5.7	5.8	5.8	5.8	5.8	5.8	5.8	5.8	5.9	5.9	5.9	5.9	5.9	5.9
	S	2.5	2.4	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	88	90	91	91	91	91	92	92	92	92	92	92	92	92	92	92	92
2.1	Q	77	91	107	122	135	149	162	177	192	207	220	234	250	267	276	291	305
	V	5.7	5.8	5.9	5.9	5.9	5.9	5.9	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
	S	2.4	2.4	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	92	93	95	95	95	95	95	95	95	96	96	96	96	96	96	96	96
2.2	Q	84	100	116	131	146	163	177	194	210	224	238	253	269	288	301	314	330
	V	5.9	5.9	6.0	6.0	6.0	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.1	6.2	6.2	6.2	6.2
	S	2.4	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
	X	96	98	99	99	99	99	99	100	100	100	100	100	100	100	100	100	100
2.3	Q	90	108	124	140	158	175	193	208	226	243	258	275	292	306	323	341	354
	V	6.0	6.1	6.1	6.1	6.2	6.2	6.2	6.2	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3	6.3
	S	2.4	2.4	2.3	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	100	102	102	103	103	103	104	104	104	105	105	105	105	105	105	105	105
2.4	Q	99	116	136	152	170	189	206	224	241	260	275	294	312	327	346	364	378
	V	6.1	6.2	6.2	6.3	6.3	6.3	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4	6.4
	S	2.3	2.3	2.3	2.3	2.3	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2	2.2
	X	105	105	106	107	107	108	108	108	108	109	109	109	109	109	109	109	109

some safety against breaching of the spillway by scour; 50 ft is used on many Soil Conservation Service structures.

The discharge rate to be carried by the emergency spillway was determined in the section on hydrologic design earlier in this chapter.

One can enter the discharge rate in Table 11 to determine corresponding values of H_p, the spillway head, and b, the bottom width of the spillway. Site conditions may set limits on either H_p or b, but the combination that best fits the site obviously should be selected.

The Q value is to be divided by the correction factor and the corrected value entered in the table. For example, a Q value of 50 with a Manning n of 0.02 would result in corrected Q of 40. Entering Table 11 on the line with an

TABLE 12. MANNING *N* VALUES AND PERMISSIBLE VELOCITIES FOR VARIOUS MATS FOR TEMPORARY PROTECTION OF BARE SOIL SURFACES AGAINST WATER EROSION

Material	Manning n	Permissible Velocity (ft/sec)[3] sandy clay	firm loam
Jute cloth - fine mesh[1]	0.02	3.4	4.8
Jute cloth - coarse mesh[1]	0.02 to 0.06[2]	1.0	1.4
Paper fiber fabric-fine mesh[1]	0.02 to 0.03[2]	2.0	2.8
Glass fiber mat - 1" thick[1] installed in cross-wise strips	0.03	4.0	5.6
Glass fiber roving fixed with asphalt spray	0.03	2 to 3	3 to 4

(1) Mat staked to spillway surface.

(2) Varies with roughness of soil surface. The 0.02 value is for a new, smooth installation.

(3) Permissible velocity values for sandy clay obtained from tests on rolled fill and may be on conservative side for channels in original material, not loosened. The values for firm loam are estimated from a study of permissible canal velocities recommended in 1926 by ASCE.

H_p value of 1.2 shows the required spillway bottom width to be 12 ft. If the Manning *n* value had been 0.04 ($Q=50$) the required width would have been nearly 16 ft.

Table 11 is based on a grass cover on the spillway and channel surfaces. If grass is difficult to grow, impractical to obtain in time, or impossible because of site conditions, other protective measures may be needed. Soil cement has been used by the Soil Conservation Service in the southwestern states to protect emergency spillways on flood control structures where good grass linings cannot be grown. Various mats and anchored mulches have been used to provide early temporary protection to earth channels, and their use should be considered to protect against erosion that would create a new sediment source.

Some mats and mulches for temporary protection of earth spillways have been tested for their protective and hydraulic properties, permissible velocity, and Manning *n* value, respectively (*43, 44*). These findings are presented in Table 12.

Other materials have come on the market since McCool and Ree (*43, 44*) made their tests. However, data on hydraulic performance are probably not available.

CHAPTER FIVE

DAM CONSTRUCTION, MAINTENANCE, AND DISPOSITION

CONSTRUCTION

The sedimentation basin dam must not fail, because if it does greater sediment damage may occur downstream than if no dam had been built. Failure is ensured against by proper design and good construction, which are equally important.

The construction procedures given here are for small dams, generally not higher than 10 ft or perhaps 15 ft at most. For larger dams, detailed design and construction procedures, such as those published by the Soil Conservation Service or the U.S. Army Corps of Engineers, should be followed.

The specification for construction set forth in the following are suggested for permanent dams; those for temporary basins may be less stringent and for expedient basins, still less so.

Dam Top Width

The minimum top width for earth dams should be 8 ft for 10-ft height and 10 ft for 15-ft height. If the top of the dam is to serve as a roadway, its top width should not be less than 14 ft.

Face Slopes

The face slopes depend on the soil material. The more stable the fill material the steeper the side slopes may be. State and local requirements for specifications on face slopes should be checked. The generally recommended slopes are 2½ or 3 to 1 for the upstream face and 2, 2½, or 3 to 1 for the downstream face.

Clearing and Stripping

First the area under the embankment is cleared of woody and organic material. This is important for permanent dams. For temporary dams, especially those that are generally dry, the clearing can be limited to the trees and brush that would interfere with the placing of the fill.

Scarifying

If the soil surface is slick after stripping, the surface should be scarified to a depth of not more than 6 in. to improve the bond with the embankment. Scarified foundations will require compaction, which, however, should be delayed until the start of embankment construction.

Cutoff Trench

To provide a cutoff trench for a dam 10 ft high or higher, the trench along the centerline of the embankment is excavated to a depth of at least 2 ft. The ends of the trench are extended to the pipe spillway crest elevation. The bottom of the trench should be wide enough to accommodate available excavating and compacting equipment, but not less than 4 ft. The side slopes shall not be steeper than 1:1. The placing and compacting of the soil procedures shall be the same as for the embankment.

Temporary dams probably, and expedient dams certainly, will not require cutoff walls. Stability is the only concern. Leakage through a temporary sedimentation basin dam may be acceptable if piping does not occur.

The Embankment

The fill material shall be clean mineral soil free of roots, vegetation, oversized stones, rocks, or gravel. Sands and gravels shall not be used for the fill.

The fill material shall be moist enough to be formed into a ball that will not crumble. If water can be squeezed out of the ball the soil is too wet. The fill material is placed in approximately 6-in. continuous layers over the entire length of the fill, and each layer of the fill is completed by driving the hauling equipment over the fill so that the entire surface is traversed at least once by a wheel or track. If this method of compaction is used, the dam should be built 10 percent higher than design height to allow for settlement. If compactors are used on the fill the settlement allowance can be reduced to 5 percent.

Slope Protection

Immediately after completion of the earthwork, erosion protection for the embankment and the emergency spillway must be provided. Protection for the spillway has been discussed in the section on emergency spillway design. Protection for the slopes of the dam can be mats, mulches, sprays, or fast-growing vegetation. Their uses are described elsewhere (*43, 44*).

MAINTENANCE

Inspection

The sedimentation basin should be inspected periodically while construction operations are going on in its immediate vicinity so that any damage by equipment or by erosion can be repaired immediately. After each rain the basin must be inspected for erosion damage or for the need for cleanout.

Cleanout of Basin

When the sediment level comes within a foot or so of the principal spillway crest or has reached the drain hole (if

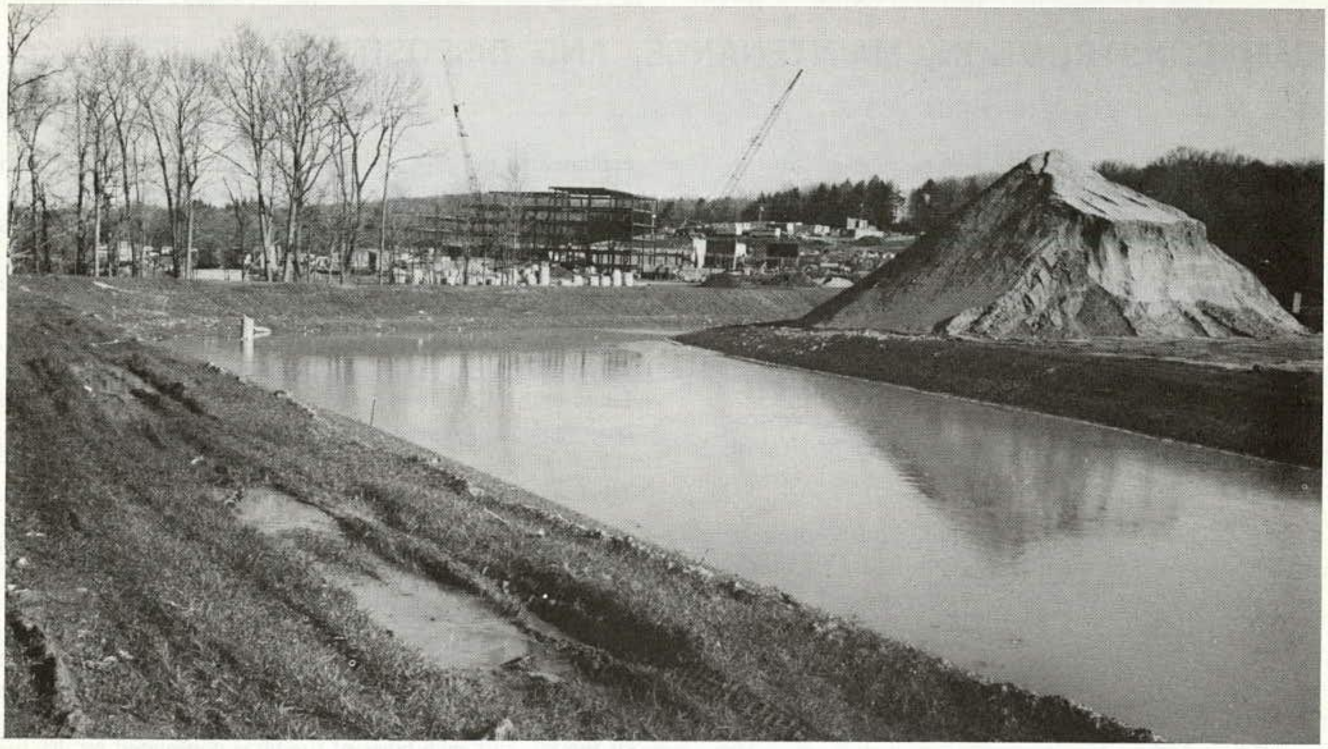


Figure 33. A sedimentation basin during construction (top) and after completion (bottom).

one is provided), the accumulated material is removed. If the basin has a dewatering system, it might be well to wait until the sediment dries before starting cleanout. However, if rain is forecast, it would be well not to delay even if a dragline must be brought in for the cleaning.

The removed sediment should be placed where it cannot re-enter the basin or the stream below.

DISPOSITION

If the basin is intended to be permanent, this will be shown on the plan. No action other than final repairs will be needed.

A good example of a permanent basin is shown in Figure 33. During construction the basin trapped sediment; after construction it became part of an attractive landscaping plan.

If the sedimentation basin is to be removed, embankment material may be needed to fill borrow areas within the reservoir. Any excess material should be hauled to a designated disposal area. An excavated or pit basin will probably require all the embankment material for fill. The site should be graded to conform to the topography of the area. If it is in a swale or draw and filling is required, the surface of the fill area should be raised enough so that surface runoff will not flow over it. The disturbed area must be seeded and covered immediately with a suitable protective medium; in some locations solid sodding might be desirable.

CHAPTER SIX

CONCLUSIONS

There has been, and continues to be, increased awareness of and concern about the consequences of erosion during construction. Measures have been taken to reduce the exposure to erosion, and seeding, sodding, and other erosion control techniques are widely used. Sediment-collecting ponds and basins are used for both short- and long-term protection of both on- and off-project facilities.

Experience and practice in the design and use of sediment-collecting basins are not consistent among state transportation agencies. There is no standard or uniform procedure for estimating the runoff and sediment that might result for any specific construction event, partly because a construction site changes from hour to hour. Soils expand and slopes and flow paths are difficult to plan for any particular time. This places much of the final responsibility for short-term sedimentation basin selection directly on the shoulders of the agency's and the contractor's field personnel. The location, type, and size of a basin are also usually of their choosing. Provisions must be included in the plans and contract that will give the necessary range of choices to meet each anticipated storm.

Although all three types of sedimentation basin (expedient, temporary, and permanent) have been widely used, there is little information available on their success or failure. Further, no correlation between sediment collected and the rainfall or drainage area (slope, condition, exposure) has yet been found. Failures are repaired immediately. Basins that have excess capacity are seldom identified.

Each construction site offers an excellent research opportunity. Valuable information on erosion losses, sediment transport, and sediment collection is readily available to be correlated with soil type, slope, and rainfall. Failures and successes should be documented.

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APPENDIX A

SETTLING THEORY

Camp (*32*) discusses the theory and design of settling tanks. The theoretical material proved helpful in developing the concept of the trap efficiency calculation.

The velocity with which a particle will fall in a fluid is a major determinant of the detention time required to settle the particle on a sedimentation basin bed. The general equation for fall velocity for a sphere is

$$v = \sqrt{(4/3)(g/C_D)[(\rho_1 - \rho)D/\rho]} \qquad \text{(A-1)}$$

where

v = fall velocity,
g = acceleration of gravity,
ρ_1 = density of particle,
ρ = density of fluid
D = particle diameter, and
C_D = a dimensionless drag coefficient.

The drag coefficient was at first assumed to be a constant until experiments showed it to vary with the Reynolds number, R. Graphs are used to portray the relation between C_D and R. Because the Reynolds number is expressed as

$$R = vd/\nu \qquad \text{(A-2)}$$

and contains the unknown value, v, being sought, a trial-and-error process is needed for the solution of Equation A-1. Camp provides a diagram that makes a direct solution possible.

Stokes' law for the settling theory of a small sphere in a viscous fluid is

$$v = (1/18)(g/\mu)(\rho_1 - \rho)\ D^2 \qquad \text{(A-3)}$$

where μ is the absolute viscosity and the other terms are as previously defined. Experimental data are in good agreement with Stokes' law for values of R from 10^{-4} to about 0.5.

Sediment particles are seldom spherical, so their drag coefficients will be different from those for spheres. Some values of drag coefficients for other shapes are given in the literature.

The fall velocity of a particle is also affected by the nearby presence of other particles in the same fluid. Camp found that, for a volumetric concentration of 10 percent of round sand grains 0.0617 cm in diameter, a correction factor of 0.68 would need to be applied to the fall velocity for a single particle.

Camp (*32*) offers a clarification theory for an ideal basin. The theory requires the following assumptions:

1. The basin is rectangular in cross-section.
2. The direction of flow is horizontal and the velocity is the same in all parts of the basin.
3. The concentration of suspended particles of each size is the same at all points in the vertical cross-section at the inlet.
4. A particle is removed from suspension when it reaches the bottom of the settling zone.

He found that the removal ratio for particles is

$$r_r = v/V_o = (b \cdot L \cdot v)/Q \qquad \text{(A-4)}$$

where

r_r = the removal ratio (analogous to trap efficiency),
b = width of basin,
L = length of basin,
v = fall velocity of particle,
Q = discharge rate, and
V_o = Q/bL, a velocity defined as the "overflow rate."

All particles having a fall velocity greater than V_o will settle and be removed from the flow. The removal of particles having a fall velocity slower than V_o is given by Equation A-4. The total removal of all particles is

$$r_r = 1 - C_{r,o} + \frac{1}{V_o}\int_o^{C_{r,o}} V\, d\,C_r \qquad \text{(A-5)}$$

where C_r is the concentration ratio of particles in a sediment suspension and $C_{r,o}$ is the concentration ratio for particles having a fall velocity less than V_o.

The two conclusions drawn from this theory are, first, that for any given discharge the removal is a function of the surface area and is independent of the depth of the basin or, the removal is a function of the overflow rate and, for a given discharge, is independent of the detention period. Second, the concentration of suspended matter at any cross-section in the settling zone increases with the depth below the surface and decreases with proximity of the cross-section to the outlet of the basin. The settling path of a particle of settling velocity, v, from the surface at the inlet end of the settling zone is a line of equal concentration, the concentration being the settling velocity analysis curve of the suspension.

The foregoing theory may not provide a direct estimate of trap efficiency for the design of sedimentation basins, but

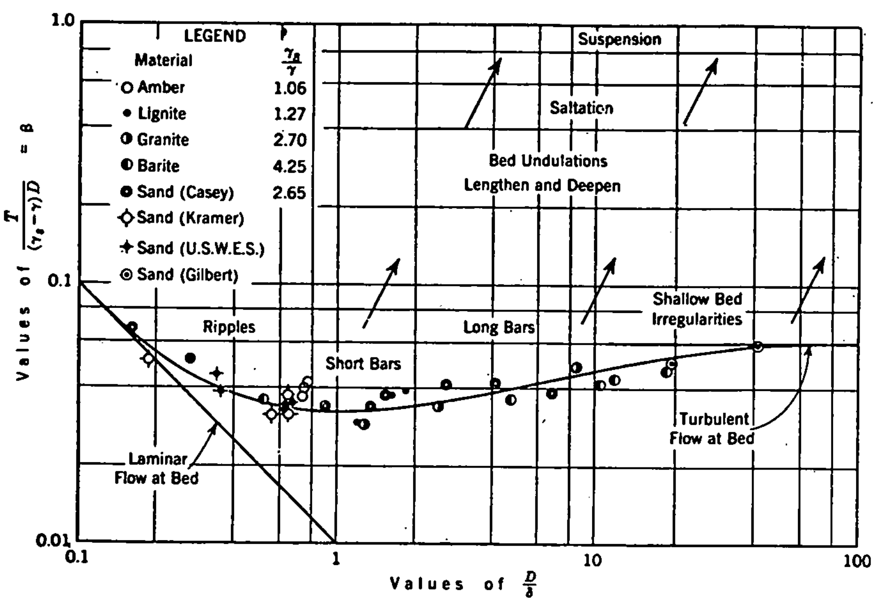


Figure A-1. Beginning a bed-load movement as a function of grain diameter and boundary layer thickness (the Shields criterion).

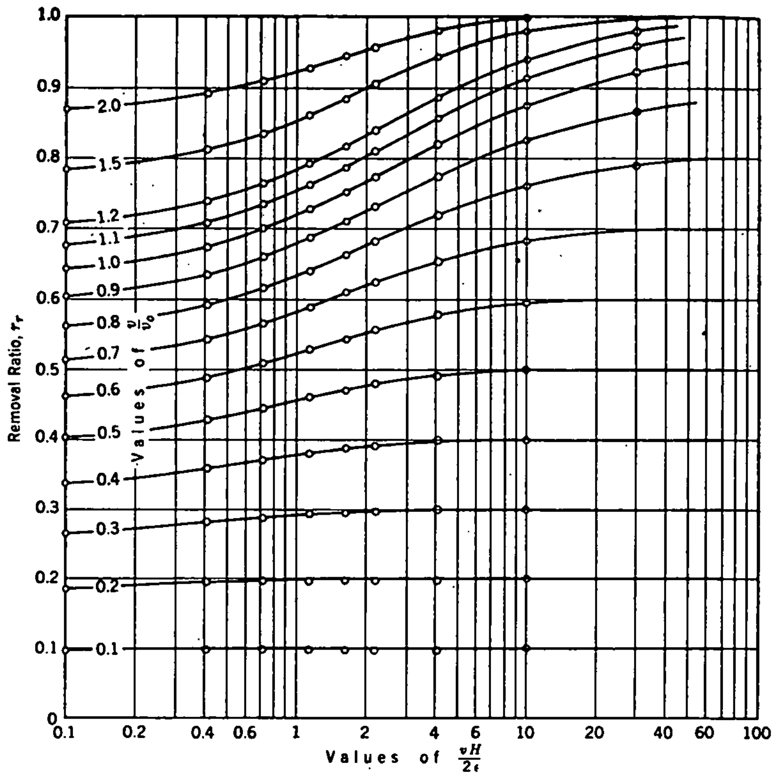


Figure A-2. Sediment removal ratio curves by Camp (32).

it does explain settling phenomena. Also, the equations, because they are physically and dimensionally correct, are useful, with appropriate correction, in models describing sedimentation processes. An example of this use is found in the work of Ward et al. (*45*).

Camp (*32*) also investigated the channel velocity required to move particles on a stream bed, and his equation is:

$$V_c = \sqrt{8/f \cdot \beta \cdot g\,(S-1)\,D} \qquad \text{(A-6)}$$

where

V_c = velocity at incipient motion;
f = Darcy-Weisbach friction factor;
β = the Shields criterion, a function of particle size and the laminar boundary layer thickness (shown in Figure A-1);
g = acceleration of gravity;
S = specific gravity of particles; and
D = particle diameter.

If this critical velocity is exceeded by the flow passing through a sedimentation basin, material previously deposited will be scoured and set in motion.

The settling theory and Equations A-3 and A-4 were based on the assumption of quiescent flow. However, flow through a basin will probably be turbulent at a Reynolds number exceeding 0.5. So the turbulent mixing process must be taken into account.

Camp, basing his work on the turbulent mixing theory, developed a diagram (Figure A-2) for the determination of the sediment-removal ratio (trap efficiency). The abscissa value is calculated with the relationship

$$vH/2\epsilon = 122\,(v/V) \qquad \text{(A-7)}$$

where

V = mean flow velocity,
H = depth of basin,
ϵ = a mixing coefficient, and
v = fall velocity of particles.

In settling tank design, Figure A-2 was used to determine the required length of tank to settle discrete particles.

TABLE B-1. PIPE FLOW CHART IN CUBIC FEET PER SECOND, $N = 0.013$, FOR DETERMINING BARREL DIAMETER FOR CONCRETE PIPE SPILLWAY

FOR REINFORCED CONCRETE PIPE INLET $K_m = K_e + K_b = 0.65$ AND 70 FEET OF REINFORCED CONCRETE PIPE CONDUIT (full flow assumed)
Note correction factors for pipe lengths other than 70 feet
diameter of pipe in inches

H, in feet	12"	15"	18"	21"	24"	30"	36"	42"	48"	54"	60"	66"	72"	78"	84"	90"	96"	102"
1	3.22	5.44	8.29	11.8	15.9	26.0	38.6	53.8	71.4	91.5	114	139	167	197	229	264	302	342
2	4.55	7.69	11.7	16.7	22.5	36.8	54.6	76.0	101	129	161	197	236	278	324	374	427	483
3	5.57	9.42	14.4	20.4	27.5	45.0	66.9	93.1	124	159	198	241	289	341	397	458	523	592
4	6.43	10.9	16.6	23.5	31.8	52.0	77.3	108	143	183	228	278	334	394	459	529	604	683
5	7.19	12.2	18.5	26.3	35.5	58.1	86.4	120	160	205	255	311	373	440	513	591	675	764
6	7.88	13.3	20.3	28.8	38.9	63.7	94.6	132	175	224	280	341	409	482	562	647	739	837
7	8.51	14.4	21.9	31.1	42.0	68.8	102	142	189	242	302	368	441	521	607	699	798	904
8	9.10	15.4	23.5	33.3	44.9	73.5	109	152	202	259	323	394	472	557	685	748	854	966
9	9.65	16.3	24.9	35.3	47.7	78.0	116	161	214	275	342	418	500	590	688	793	905	1025
10	10.2	17.2	26.2	37.2	50.2	82.2	122	170	226	289	361	440	527	622	725	836	954	1080
11	10.7	18.0	27.5	39.0	52.7	86.2	128	178	237	304	379	462	553	653	761	877	1001	1133
12	11.1	18.9	28.7	40.8	55.0	90.1	134	186	247	317	395	482	578	682	794	916	1045	1184
13	11.6	19.6	29.9	42.4	57.3	93.7	139	194	257	330	411	502	601	710	827	953	1088	1232
14	12.0	20.4	31.0	44.1	59.4	97.3	145	201	267	342	427	521	624	736	858	989	1129	1278
15	12.5	21.1	32.1	45.6	61.5	101	150	208	277	354	442	539	646	762	888	1024	1169	1323
16	12.9	21.8	33.2	47.1	63.5	104	155	215	286	366	457	557	667	787	917	1057	1207	1367
17	13.3	22.4	34.2	48.5	65.5	107	159	222	294	377	471	574	688	812	946	1090	1244	1409
18	13.7	23.1	35.2	49.9	67.4	110	164	228	303	388	484	591	708	835	973	1121	1280	1450
19	14.0	23.7	36.1	51.3	69.2	113	168	234	311	399	497	607	727	858	1000	1152	1315	1489
20	14.4	24.3	37.1	52.6	71.0	116	173	240	319	409	510	623	746	880	1026	1182	1350	1528
21	14.7	24.9	38.0	53.9	72.8	119	177	246	327	419	523	638	764	902	1051	1211	1383	1566
22	15.1	25.5	38.9	55.2	74.5	122	181	252	335	429	535	653	782	923	1076	1240	1415	1603
23	15.4	26.1	39.8	56.5	76.2	125	186	258	342	439	547	668	800	944	1100	1268	1447	1639
24	15.8	26.7	40.6	57.7	77.8	127	189	263	350	448	559	682	817	964	1123	1295	1478	1674
25	16.1	27.2	41.5	58.9	79.4	130	193	269	357	458	571	696	834	984	1147	1322	1509	1708
26	16.4	27.7	42.3	60.0	81.0	133	197	274	364	467	582	710	850	1004	1169	1348	1539	1742
27	16.7	28.3	43.1	61.2	82.5	135	201	279	371	476	593	723	867	1023	1192	1373	1568	1775
28	17.0	28.8	43.9	62.3	84.1	138	204	285	378	484	604	737	883	1041	1214	1399	1597	1808
29	17.3	29.3	44.7	63.4	85.5	140	208	290	384	493	615	750	898	1060	1235	1423	1625	1840
30	17.6	29.8	45.4	64.5	87.0	142	212	294	391	501	625	763	913	1078	1256	1448	1653	1871
L, in feet	Correction Factors For Other Pipe Lengths																	
20	1.30	1.24	1.21	1.18	1.15	1.12	1.10	1.08	1.07	1.06	1.05	1.05	1.04	1.04	1.03	1.03	1.03	1.03
30	1.22	1.18	1.15	1.13	1.12	1.09	1.08	1.06	1.05	1.05	1.04	1.04	1.03	1.03	1.03	1.02	1.02	1.02
40	1.15	1.13	1.11	1.10	1.08	1.07	1.05	1.05	1.04	1.03	1.03	1.03	1.02	1.02	1.02	1.02	1.02	1.02
50	1.09	1.08	1.07	1.06	1.05	1.04	1.04	1.03	1.03	1.02	1.02	1.02	1.02	1.01	1.01	1.01	1.01	1.01
60	1.04	1.04	1.03	1.03	1.03	1.02	1.02	1.02	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
70	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
80	.96	.97	.97	.97	.98	.98	.98	.99	.99	.99	.99	.99	.99	.99	.99	.99	.99	.99
90	.93	.94	.94	.95	.95	.96	.97	.97	.98	.98	.98	.98	.98	.99	.99	.99	.99	.99
100	.90	.91	.92	.93	.93	.95	.95	.96	.97	.97	.97	.98	.98	.98	.98	.98	.98	.99
120	.84	.86	.87	.89	.90	.91	.93	.94	.94	.95	.96	.96	.96	.97	.97	.97	.97	.98
140	.80	.82	.83	.85	.86	.88	.90	.91	.92	.93	.94	.94	.95	.95	.96	.96	.96	.97
160	.76	.78	.80	.82	.83	.86	.88	.89	.90	.91	.92	.93	.94	.94	.95	.95	.95	.96

TABLE B-2. PIPE FLOW CHART IN CUBIC FEET PER SECOND, $N = 0.025$, FOR DETERMINING BARREL DIAMETER FOR CORRUGATED METAL PIPE SPILLWAY

FOR CORRUGATED METAL PIPE INLET $K_m = K_e + K_b = 1.0$ AND 70 FEET OF CORRUGATED METAL PIPE CONDUIT (full flow assumed)
Note correction factors for pipe lengths other than 70 feet

H, in feet	diameter of pipe in inches																				
	6"	8"	10"	12"	15"	18"	21"	24"	30"	36"	42"	48"	54"	60"	66"	72"	78"	84"	90"	96"	102"
1	0.33	0.70	1.25	1.98	3.48	5.47	7.99	11.0	18.8	28.8	41.1	55.7	72.6	91.8	113	137	163	191	222	255	290
2	0.47	0.99	1.76	2.80	4.92	7.74	11.3	15.6	26.6	40.8	58.2	78.8	103	130	160	194	231	271	314	360	410
3	0.58	1.22	2.16	3.43	6.02	9.48	13.8	19.1	32.6	49.9	71.2	96.5	126	159	196	237	282	331	384	441	502
4	0.67	1.40	2.49	3.97	6.96	10.9	16.0	22.1	37.6	57.7	82.3	111	145	184	226	274	326	383	444	510	580
5	0.74	1.57	2.79	4.43	7.78	12.2	17.9	24.7	42.1	64.5	92.0	125	162	205	253	306	365	428	496	570	648
6	0.82	1.72	3.05	4.86	8.52	13.4	19.6	27.0	46.1	70.6	101	136	178	225	277	336	399	469	544	624	710
7	0.88	1.86	3.30	5.25	9.20	14.5	21.1	29.2	49.8	76.3	109	147	192	243	300	362	431	506	587	674	767
8	0.94	1.99	3.53	5.61	9.84	15.5	22.6	31.2	53.2	81.5	116	158	205	260	320	388	461	541	628	721	820
9	1.00	2.11	3.74	5.95	10.4	16.4	24.0	33.1	56.4	86.5	123	167	218	275	340	411	489	574	666	764	870
10	1.05	2.22	3.94	6.27	11.0	17.3	25.3	34.9	59.5	91.2	130	176	230	290	358	433	516	605	702	806	917
11	1.10	2.33	4.13	6.58	11.5	18.2	26.5	36.6	62.4	95.6	136	185	241	304	376	454	541	635	736	845	962
12	1.15	2.43	4.32	6.87	12.1	19.0	27.7	38.2	65.2	99.9	142	193	252	318	392	475	565	663	769	883	1004
13	1.20	2.53	4.49	7.15	12.6	19.7	28.8	39.8	67.8	104	148	201	262	331	408	494	588	690	800	919	1045
14	1.25	2.63	4.66	7.42	13.0	20.5	29.9	41.3	70.4	108	154	208	272	343	424	513	610	716	830	953	1085
15	1.29	2.72	4.83	7.68	13.5	21.2	30.9	42.8	72.8	112	159	216	281	355	439	531	631	741	860	987	1123
16	1.33	2.81	4.99	7.93	13.9	21.9	32.0	44.2	75.2	115	165	223	290	367	453	548	652	765	888	1019	1160
17	1.37	2.90	5.14	8.18	14.3	22.6	32.9	45.5	77.5	119	170	230	229	378	467	565	672	789	915	1051	1195
18	1.41	2.98	5.29	8.41	14.8	23.2	33.9	46.8	79.8	120	174	236	308	389	480	581	692	812	942	1081	1230
19	1.45	3.06	5.43	8.64	15.2	23.9	34.8	48.1	82.0	126	179	243	316	400	494	597	711	834	967	1111	1264
20	1.49	3.14	5.57	8.87	15.6	24.5	35.7	49.4	84.1	129	184	249	325	410	506	613	729	856	993	1139	1297
21	1.53	3.22	5.71	9.09	15.9	25.1	36.6	50.6	86.2	132	188	255	333	421	519	628	747	877	1017	1168	1329
22	1.56	3.29	5.85	9.30	16.3	25.7	37.5	51.8	88.2	135	193	261	341	430	531	643	765	898	1041	1195	1360
23	1.60	3.37	5.98	9.51	16.7	26.2	38.3	53.0	90.2	138	197	267	348	440	543	657	782	918	1064	1222	1390
24	1.63	3.44	6.11	9.72	17.0	26.8	39.1	54.1	92.1	141	201	273	356	450	555	671	799	937	1087	1248	1420
25	1.66	3.51	6.23	9.92	17.4	27.4	39.9	55.2	94.0	144	206	279	363	459	566	685	815	957	1110	1274	1450
26	1.70	3.58	6.36	10.1	17.7	27.9	40.7	56.3	95.9	147	210	284	370	468	577	699	831	976	1132	1299	1478
27	1.73	3.65	6.48	10.3	18.1	28.4	41.5	57.4	97.7	150	214	290	377	477	588	712	847	994	1153	1324	1507
28	1.76	3.72	6.60	10.5	18.4	29.0	42.3	58.4	99.5	153	218	295	384	486	599	725	863	1013	1174	1348	1534
29	1.79	3.78	6.71	10.7	18.7	29.5	43.0	59.5	101	155	221	300	391	494	610	738	878	1030	1195	1372	1561
30	1.82	3.85	6.83	10.9	19.1	30.0	43.7	60.5	103	158	225	305	398	503	620	750	893	1048	1216	1396	1588
L, in feet	Correction Factors For Other Pipe Lengths																				
20	1.69	1.63	1.58	1.53	1.47	1.42	1.37	1.34	1.28	1.24	1.20	1.18	1.16	1.14	1.13	1.11	1.10	1.10	1.09	1.08	1.08
30	1.44	1.41	1.39	1.36	1.32	1.29	1.27	1.24	1.21	1.18	1.15	1.13	1.12	1.11	1.10	1.09	1.08	1.07	1.07	1.06	1.06
40	1.28	1.27	1.25	1.23	1.21	1.20	1.18	1.17	1.14	1.12	1.11	1.10	1.09	1.08	1.07	1.06	1.06	1.05	1.05	1.05	1.04
50	1.16	1.16	1.15	1.14	1.13	1.12	1.11	1.10	1.09	1.08	1.07	1.06	1.06	1.05	1.05	1.04	1.04	1.04	1.03	1.03	1.03
60	1.07	1.07	1.07	1.06	1.06	1.05	1.05	1.05	1.04	1.04	1.03	1.03	1.03	1.02	1.02	1.02	1.02	1.02	1.02	1.02	1.01
70	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
80	.94	.94	.95	.95	.95	.95	.96	.96	.96	.97	.97	.97	.98	.98	.98	.98	.98	.98	.99	.99	.99
90	.89	.89	.90	.90	.91	.91	.92	.92	.93	.94	.94	.95	.95	.96	.96	.96	.97	.97	.97	.97	.94
100	.85	.85	.86	.86	.87	.88	.89	.89	.90	.91	.92	.93	.93	.94	.94	.95	.95	.95	.96	.96	.94
120	.78	.79	.79	.90	.81	.82	.83	.83	.85	.86	.87	.89	.89	.90	.91	.89	.92	.93	.93	.94	.92
140	.72	.73	.74	.75	.76	.77	.78	.79	.81	.82	.84	.85	.86	.87	.88	.86	.89	.90	.91	.91	.90
160	.68	.69	.69	.70	.71	.73	.74	.75	.77	.79	.80	.82	.83	.84	.85	.92	.87	.88	.89	.89	

TABLE B-3. PIPE FLOW CHART IN CUBIC FEET PER SECOND, $N = 0.025$, FOR CORRUGATED METAL PIPE HOODED INLET SPILLWAY

PIPE FLOW CHART (Full flow assumed)

For Hooded Inlet K_e = 1.08 and 70 feet of Corrugated Metal Pipe Conduit, n = 0.025.
Note corrections for other pipe lengths.

Dia. / H	12"	15"	18"	21"	24"	30"	36"	42"
2	2.79	4.89	7.72	11.16	15.48	26.31	40.28	57.42
3	3.41	5.99	9.46	13.67	18.97	32.32	49.34	70.34
4	3.94	6.92	10.92	15.78	21.90	37.32	56.98	81.22
5	4.40	7.74	12.21	17.64	24.48	41.72	63.70	90.80
6	4.82	8.47	13.37	19.32	26.82	45.70	69.77	99.45
7	5.21	9.16	14.45	20.88	28.97	49.37	75.38	107.45
8	5.57	9.78	15.44	22.31	30.97	52.77	80.57	114.85
9	5.91	10.38	16.38	23.61	32.85	55.98	85.47	121.83
10	6.23	10.94	17.26	24.95	34.62	59.00	90.09	128.41
11	6.53	11.48	18.11	26.17	36.32	61.90	94.50	134.70
12	6.82	11.99	18.91	27.33	37.93	64.64	98.69	140.67
13	7.10	12.48	19.69	28.45	39.49	67.29	102.73	146.44
14	7.37	12.95	20.43	29.52	40.97	69.83	106.61	151.96
15	7.63	13.40	21.15	30.56	42.41	72.27	110.34	157.28
16	7.88	13.84	21.84	31.56	43.80	74.64	113.96	162.44
17	8.12	14.27	22.51	32.53	45.15	76.94	117.46	167.44
18	8.36	14.68	23.17	33.48	46.46	79.17	120.88	172.31
19	8.59	15.08	23.80	34.39	47.73	81.34	124.19	177.02
20	8.81	15.47	24.42	35.28	48.97	83.45	127.41	181.61
21	9.03	15.86	25.02	36.16	50.18	85.52	130.57	186.12
22	9.24	16.23	25.61	37.00	51.36	87.52	133.62	190.46
23	9.45	16.59	26.19	37.84	52.52	89.49	136.64	194.77
24	9.65	16.95	26.69	38.65	53.64	91.42	139.57	198.95
25	9.85	17.30	27.30	39.45	54.75	93.30	142.45	203.05
L	Correction Factors For Other Lengths							
40	1.23	1.21	1.19	1.18	1.16	1.13	1.12	1.10
50	1.14	1.13	1.12	1.11	1.10	1.09	1.08	1.07
60	1.06	1.06	1.05	1.05	1.04	1.04	1.04	1.03
70	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
80	0.95	0.95	0.95	0.96	0.96	0.96	0.97	0.97
90	0.90	0.91	0.91	0.92	0.92	0.93	0.94	0.94
100	0.86	0.87	0.88	0.89	0.89	0.90	0.91	0.92

TABLE B-4. PIPE FLOW CHART IN CUBIC FEET PER SECOND, $N = 0.010$, FOR SMOOTH PIPE HOODED INLET SPILLWAY

PIPE FLOW CHART (Full flow assumed)

For Hooded Inlet K_e = 1.08 and 70 feet of smooth pipe conduit, n = 0.010. Note corrections for other lengths.

Dia. / H	10"	12"	14"	15"	18"	21"
2	3.20	4.85	6.85	7.99	11.92	16.64
3	3.92	5.94	8.38	9.79	14.60	20.39
4	4.53	6.85	9.68	11.31	16.86	23.54
5	5.06	7.66	10.82	12.64	18.85	26.32
6	5.54	8.39	11.86	13.84	20.64	28.83
7	5.99	9.07	12.81	14.96	22.30	31.15
8	6.40	9.69	13.69	15.99	23.84	33.29
9	6.79	10.28	14.52	16.96	25.29	35.31
10	7.16	10.84	15.31	17.87	26.65	37.22
11	7.51	11.36	16.05	18.74	27.95	39.03
12	7.83	11.87	16.77	19.58	29.20	40.77
13	8.16	12.36	17.46	20.41	30.39	42.45
14	8.47	12.82	18.11	21.15	31.54	44.05
15	8.77	13.27	18.75	21.89	32.64	45.59
16	9.06	13.71	19.36	22.61	33.72	47.08
17	9.33	14.13	19.96	23.31	34.75	48.53
18	9.61	14.54	20.54	29.99	35.76	49.94
19	9.87	14.94	21.10	24.64	36.74	51.31
20	10.12	15.33	21.65	25.28	37.69	52.64
21	10.38	15.71	22.19	25.91	38.63	53.95
22	10.62	16.07	22.70	26.51	39.53	55.21
23	10.86	16.44	23.24	27.11	40.42	56.45
24	11.09	16.79	23.72	27.69	41.29	57.67
25	11.32	17.14	24.21	28.26	42.14	58.86
L	Correction Factors for Other Lengths					
40	1.11	1.09	1.08	1.08	1.06	1.05
50	1.07	1.06	1.05	1.05	1.04	1.03
60	1.03	1.03	1.02	1.02	1.02	1.02
70	1.00	1.00	1.00	1.00	1.00	1.00
80	0.97	0.97	0.98	0.98	0.98	0.98
90	0.95	0.95	0.96	0.96	0.96	0.97
100	0.93	0.93	0.94	0.94	0.95	0.96

APPENDIX C

DIMENSIONS OF CONCENTRIC CYLINDER TRASH RACKS AND ANTIVORTEX PLATES FOR STEEL PIPE RISERS

Riser Diameter (in.)	Cylinder Diam. (in.)	Cylinder Thick (gage)	H (in.)	Minimum Size Support Bar	Minimum Top Thickness	Minimum Top Stiffener
12	18	16	6	#6 Rebar	16 ga.	-
15	21	16	7	"	"	-
18	27	16	8	"	"	-
21	30	16	11	"	"	-
24	36	16	13	"	14 ga.	-
27	42	16	15	"	14 ga.	-
36	54	14	17	#8 Rebar	12 ga.	-
42	60	14	19	"	"	-
48	72	12	21	1-1/4" pipe or 1-1/4-1/4x1/4 angle	10 ga.	-
54	78	12	25	"	"	-
60	90	12	29	1-1/2" pipe or 1-1/2x1/2x1/4 angle	8 ga.	-
66	96	10	33	2" pipe or 2x2x3/16 angle	8 ga w/stiffener	2x2x1/4 angle
72	102	10	36	"	"	2-1/2x2-1/2x 1/4 angle
78	114	10	39	2-1/2" pipe or 2x2x1/4 angle	"	"
84	120	10	42	2-1/2" pipe or 2-1/2x2-1/2x1/4 angle	"	2-1/2x2-1/2x 5/16 angle

Note: The criterion for sizing the cylinder is that the area between the inside of the cylinder and outside of the riser is equal to or greater than the area inside the riser. Therefore, the above table is invalid for use with concrete pipe risers.

APPENDIX D

SPECIFICATIONS AND PLANS

This appendix includes copies of specifications and standards for sedimentation basins used by various agencies and suggestions made by individuals and organizations. These are offered to the designer seeking ways to comply with the rules imposed by the variety of conditions encountered in the field. One may get some ideas but should be advised that not all of the ways shown are endorsed in this report. Included are

- A suggestion on the use of diked areas.
- Suggestions and plans for sedimentation traps and basins by the Urban Institute, American Society of Civil Engineers, and National Association of Home Builders.
- Pennsylvania Department of Transportation sedimentation pond designs and specifications on sedimentation ponds.

DIKED AREA

An untested idea but one that might have merit in some situations is offered here for consideration. Its purpose is to divert the sediment-laden flow onto a diked, level area. All sediment should be allowed to accumulate during the life of the construction and the establishment of vegetation. When the need for the sediment trap is over and the dikes have been bladed down, uneven sediment deposition is leveled and then turned under with a deep turning plow. This brings the original ground back to the surface.

In California, sediment deposits as deep as 6 ft (2 m) have been turned under and the original ground brought back to the top. One advantage of this scheme would be to raise the level and to improve the drainage of a low-lying area. A sketch of this proposal is shown in Figure D-1.

TRAPPING

Sediment-trapping measures generally are used where channelized flows contain sediment in greater than acceptable amounts. Sediment traps and sedimentation basins are differentiated by their size and method of design and construction.

Traps are relatively small installations used for small drainage areas; they can be inexpensive to construct and comparatively simple to maintain. They are useful in areas unsuited to larger sedimentation basins. Several traps can often be substituted for a single, larger sedimentation basin if an area is divisible into small subwatersheds.

Sedimentation basins are relatively large and frequently expensive to design, construct, and maintain. They can be relatively efficient sediment-removal devices, but that efficiency often is obtained at high cost. Accordingly, sedimentation basins should be considered a last resort and used when other approaches to site planning, erosion prevention, or trapping are inadequate to reduce off-site sedimentation to acceptable levels.

SEDIMENT TRAPS

Sediment traps are small, temporary detention structures used to intercept runoff and trap sediment. They rarely are practical for drainage areas larger than about 5 acres. They are usually installed by excavation and/or embankment; embankments usually should not exceed 5 ft in height. Depending on climate, sediment trap volume customarily is about 1800 ft^3/acre of drainage area, which seems to provide adequate trapping efficiency and sediment storage. The volume of storage required depends on the amount and

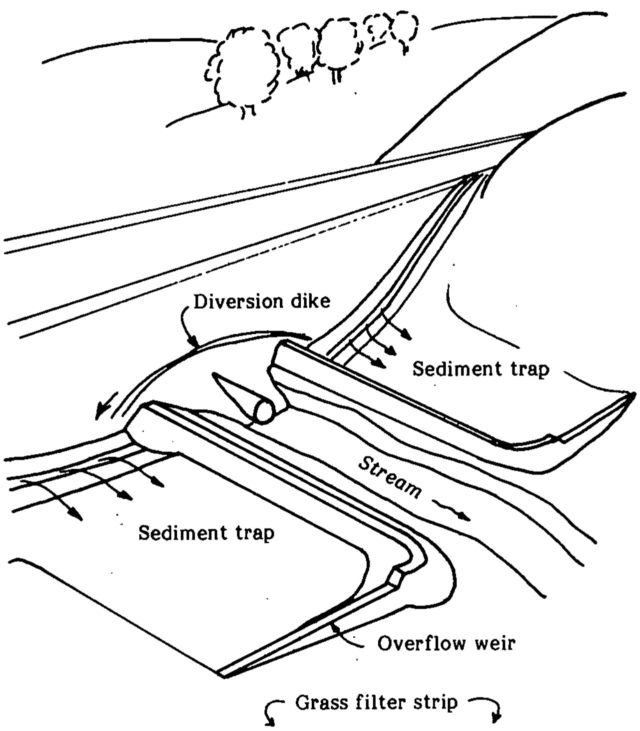


Figure D-1. Sediment trap that uses diked areas.

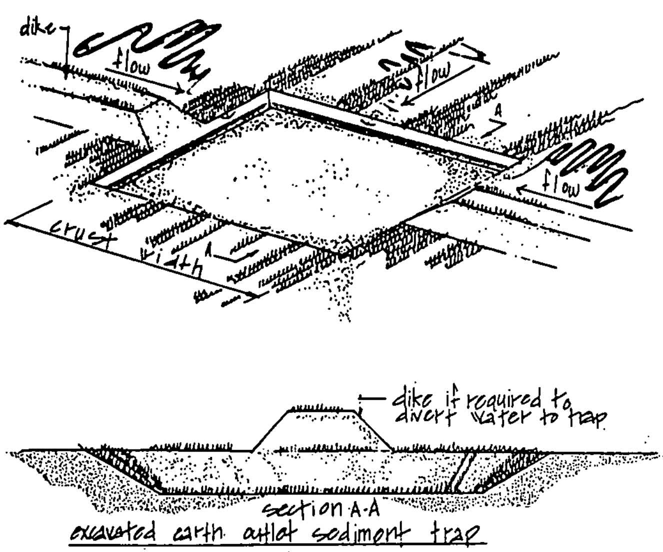


Figure D-2. Earth outlet sediment trap.

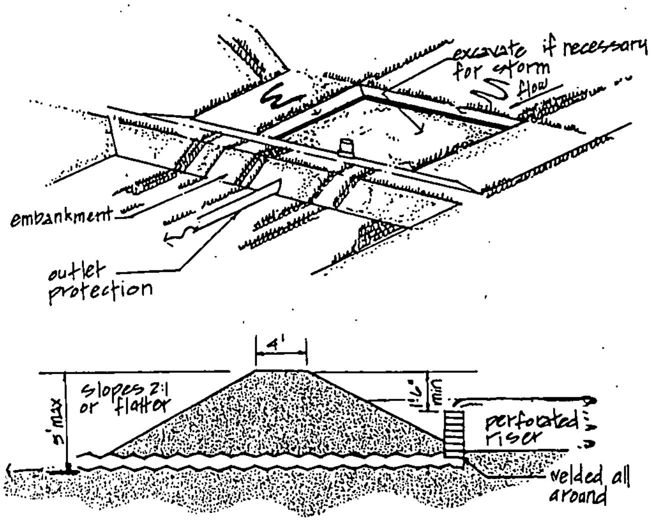


Figure D-3. Embankment section through riser.

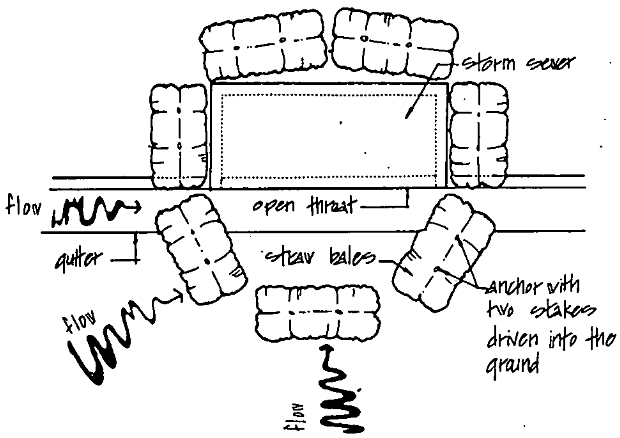


Figure D-4. Storm inlet sediment trap.

intensity of expected rainfall and on estimated quantity of sediment. Traps should be cleaned when accumulated sediments equal about one-half of trap storage capacity.

Sediment traps may be constructed with earth, pipe, or stone outlets, or they may be installed at storm drain inlets. Outlet selection is based on construction costs.

Earth outlet sediment traps (Figure D-2) discharge over or onto natural ground.

Pipe outlet sediment traps (Figure D-3) have outlets consisting of a piped riser that functions as a skimming weir, and they discharge through the embankment. The diameter of the riser should be larger than that of the discharge pipe.

Storm inlet sediment traps (Figure D-4) consist of a basin formed either by excavation or a natural depression in the ground adjacent to and upstream from storm sewer inlets. Water may be discharged (and often filtered) through an opening into a storm drain inlet structure. The opening may be either the inlet opening or a temporary opening made by omitting bricks or blocks in the inlet.

Sandbag sediment barriers consist of bags filled with soil or stone, stacked at regular intervals along the ditch upstream of a storm drain inlet or culvert, for trapping coarse sediment particles.

SEDIMENTATION PONDS

The operation of sedimentation ponds is based on retaining the water in a quiet condition for a period of time such that the sediment load carried by the stream will settle in a given distance, thus allowing sediment-free water to flow over the outlet. This retention time will depend on the material being transported by the stream.

In the design of sedimentation ponds and basins it is understood that, under normal conditions, the device can have a capacity of 2200 ft^3 for each acre of cleared and grubbed area tributary to it, with the capacity adjusted upward to 7000 ft^3/acre for extreme cases where an extremely high rate of erosion is anticipated.

The inlet and outlet structures shall be designed to pass a minimum flow based on a 5-yr storm frequency and a 5-min storm duration for temporary facilities, and a 10-yr storm frequency with a 5-min duration for permanent facilities.

Sedimentation ponds of type 1 consist of a dam created to impound water with or without an excavated storage area and limited to a maximum depth of 15 ft. A riser pipe connected to a pipe spillway and an emergency spillway are used to convey water through the dam. The riser pipe cross-sectional area must be at least 1.5 times the cross-sectional area of the pipe spillway.

Sedimentation ponds of type 2 consist of an excavated storage area with controlled inlet and outlet areas. The pond will have defined side slopes and should be limited to approximately 12 acres of project area tributary to it.

Pennsylvania's approaches to types 1 and 2 and illustrations of these types of sedimentation ponds follow.

SECTION 859

SEDIMENTATION POND — TYPE 1

859.1 DESCRIPTION—This work shall consist of constructing a dam to impound water in a storage area limited to a maximum height of fifteen (15) feet, with a controlled outlet and an emergency spillway in accordance with these specifications and within reasonably close conformity to the design and dimensions shown on the drawings.

859.2 MATERIALS—

(a) Borrow Excavation. . . . . Section 205

(b) Embankment. Section 206

(c) Class A Cement Concrete. . . . . . Section 704.1(g)

(d) Corrugated Metal Pipe. . . . . Section 707

(e) Reinforcement Steel. . . . . Section 709

859.3 CONSTRUCTION REQUIREMENTS—The entire storage area and embankment foundation area shall be cleared and grubbed in accordance with Section 201.3.

The key trench shall be excavated to the dimensions shown and so that it extends for the full length of the dam.

The emergency spillway shall be excavated in natural ground at the location and to the width shown on the drawings.

The pipe spillway, with the anti-seep collar attached as shown, shall be installed at locations indicated on the drawings. The contractor will **not** be required to construct the embankment, and trench, prior to placing the pipe. Coarse aggregate is **not** to be used as embankment material around the pipe.

The trash rack, anti-vortex device, and riser pipe with its concrete foundation, may be constructed either before or after the embankment material is placed.

The embankment material is to be placed in accordance with the requirements of Section 206.

When the need for the sedimentation pond no longer exists, the contractor shall recondition the site by filling in all excavated areas, removing embankments, riser pipe assemblies, corrugated metal pipe, and anti-seep collar, and restore the area in accordance with Section 205.1(f), unless otherwise directed. Salvageable items shall become the property of the contractor.

859.4 METHOD OF MEASUREMENT—

(a) **Excavation.** Excavation will be measured as Class 1 Excavation in accordance with Section 203.4(a)2.

(b) **Riser Pipe Assembly.** Riser Pipe Assembly will be measured as a unit, acceptably completed.

(c) **Corrugated Metal Pipe.** Corrugated Metal Pipe will be measured by the linear foot along the centerline of the pipe.

(d) **Anti-Seep Collar.** Anti-Seep Collar will be measured as a unit, acceptably completed.

(e) **Embankment.** Embankment will be measured in accordance with Section 206.4.

(f) **Borrow Excavation.** Borrow Excavation will be measured in accordance with Section 205.4(a), 205.4(b) or 205.4(c), whichever is applicable.

859.5 BASIS OF PAYMENT—

(a) **Clearing and Grubbing.** No separate payment will be allowed for clearing and grubbing since this work will be considered incidental to the other items of work.

(b) **Excavation.** Excavation will be paid for at the contract unit price per cubic yard for Class 1 Excavation as specified in Section 203.5(a).

(c) **Riser Pipe Assembly.** Riser Pipe Assembly will be paid for at the contract lump sum price, complete in place as specified, which price will include the riser pipe, concrete foundation, trash rack, and anti-vortex device.

(d) **Corrugated Metal Pipe.** Corrugated Metal Pipe will be paid for at the contract unit price per linear foot for the type and size specified, complete in place.

(e) **Anti-Seep Collar.** Anti-Seep Collar will be paid for at the contract unit price each, complete in place as specified.

(f) **Embankment.** Embankment construction is not payable directly; however, compensation shall be as specified in Section 206.5.

(g) **Borrow Excavation.** Borrow Excavation will be paid for at the contract unit price per cubic yard as specified in Section 205.5(a), 205.5(b) or 205.5(c), whichever is applicable.

(h) No separate payment will be allowed for the complete reconditioning of the site, since that cost will be considered incidental to the other items of the work.

SECTION 860

SEDIMENTATION POND – TYPE 2

860.1 DESCRIPTION—This work shall consist of constructing a sediment collecting pond in accordance with these specifications and within reasonably close conformity to the design and dimensions shown on the drawings.

860.2 MATERIALS—

(a) **Rock.** Rock shall be hard and angular in shape; resistant to weathering; reasonably free from soil, shale, and organic materials; and shall meet the size requirements tabulated below. Neither width nor thickness of any single rock shall be less than one third its length. Rounded rock or boulders are not permitted. Rock with shale seams is not acceptable.

Rock Size	*Maximum Percent of Total Weight Smaller than Given Size*
24	100
18	40
3	20

The minimum specific gravity of the rock shall be 2.5 as determined in accordance with AASHTO T85 (PTM 506), bulk-saturated, surface-dry basis.

(b) Borrow Excavation. . . . . Section 205

(c) Embankment. . . . . Section 206

860.3 CONSTRUCTION REQUIREMENTS—The sedimentation pond shall be constructed at the location shown on the drawings or as directed.

The area shall be cleared and grubbed in accordance with Section 201.3.

The pond shall be constructed by excavating, forming embankments in accordance with Section 206, and providing rock lining at the outlet end.

The use of borrow excavation for the formation of the embankment shall be subject to the requirements of Section 205.1(a).

When the need for the sedimentation pond no longer exists, the contractor shall recondition the site by filling in all excavated areas, remove embankments and rock linings, and restore the area in accordance with Section 205.1 (f), unless otherwise directed.

860.4 METHOD OF MEASUREMENT—

(a) **Excavation.** Excavation will be measured as Class 1 Excavation in accordance with Section 203.4(a).

(b) **Embankment.** Embankment will be measured in accordance with Section 206.4.

(c) **Rock Lining.** Rock Lining will be measured as specified in Section 850.4.

(d) **Borrow Excavation.** Borrow Excavation will be measured in accordance with Section 205.4(a) or 205.4(b), whichever is applicable.

860.5 BASIS OF PAYMENT—

(a) **Clearing and Grubbing.** No separate payment will be allowed for clearing and grubbing since this work will be considered incidental to the other items of work.

(b) **Excavation.** Excavation will be paid for at the contract unit price per cubic yard for Class 1 Excavation in accordance with Section 203.5(a).

(c) **Embankment.** Embankment construction is not payable directly; however, compensation shall be as specified in Section 206.5.

(d) **Rock Lining.** Rock Lining will be paid for at the contract unit price per square yard, complete in place, as specified in Section 850.5.

(e) **Borrow Excavation.** Borrow Excavation will be paid for at the contract unit price per cubic yard as specified in Section 205.5(a) or 205.5(b), whichever is applicable.

(f) No separate payment will be allowed for the complete reconditioning of the site since that cost will be considered incidental to the other items of work.

SECTION 861

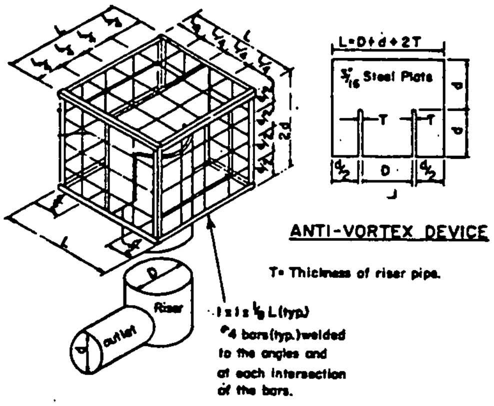
CLEANING SEDIMENTATION STRUCTURES

861.1 DESCRIPTION—This work shall consist of the proper cleaning and disposal of sediment deposited in the erosion and sedimentation control devices in accordance with these specifications.

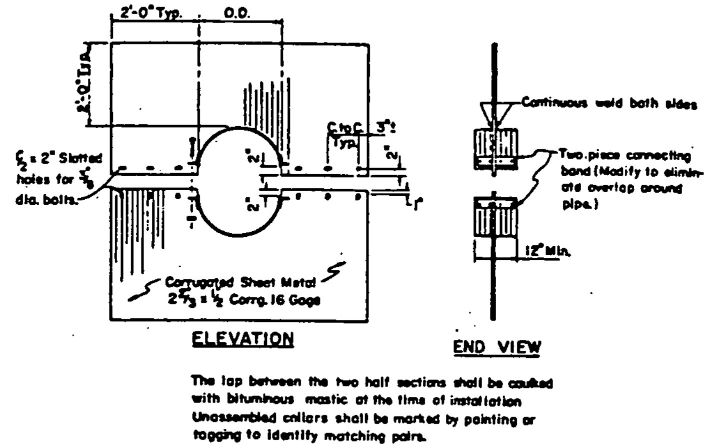
861.3 CONSTRUCTION REQUIREMENTS—When the accumulation of sediment in the Sedimentation Structure has reached a point of 1/3 the depth of the sediment structure, the sediment shall be removed and disposed of in such locations that the sediment will not again erode into the construction areas or into natural waterways. The removal of the sediment shall be done in such a manner so as not to damage the sedimentation structure.

861.4 METHOD OF MEASUREMENT—The removal of sediment from sedimentation structures will be measured by determining the quantity by load count times the rated capacity of the hauling equipment. The cross-sectional method shall be used for large quantities and where trucks are not used.

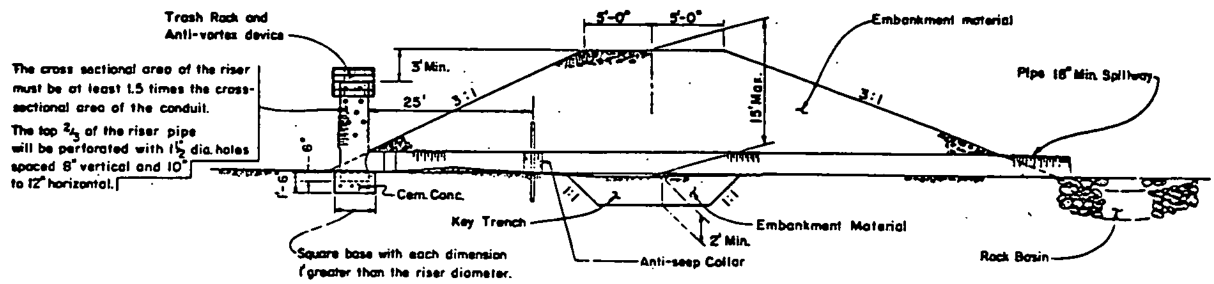
861.5 BASIS OF PAYMENT—Cleaning Sedimentation Structures will be paid for at the contract unit price per cubic yard complete in place, which price will include removal and disposition of the sediment in a location where it will not erode into construction areas or water courses.



TRASH RACK AND ANTI-VORTEX DEVICE



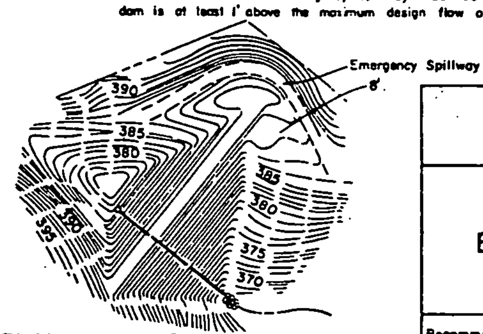
DETAIL OF ANTI-SEEP COLLAR



SEDIMENTATION POND - TYPE I

NOTES:

- An emergency spillway with a min. bottom width of 8' must be provided for every Sedimentation Pond - Type I.
- The emergency spillway must be placed in undisturbed ground and cannot be placed in embankment areas. The emergency spillway can go over the embankment if Rock Lining is used.
- The elevation of the emergency spillway must be such that the dam is at least 1' above the maximum design flow of the spillway.

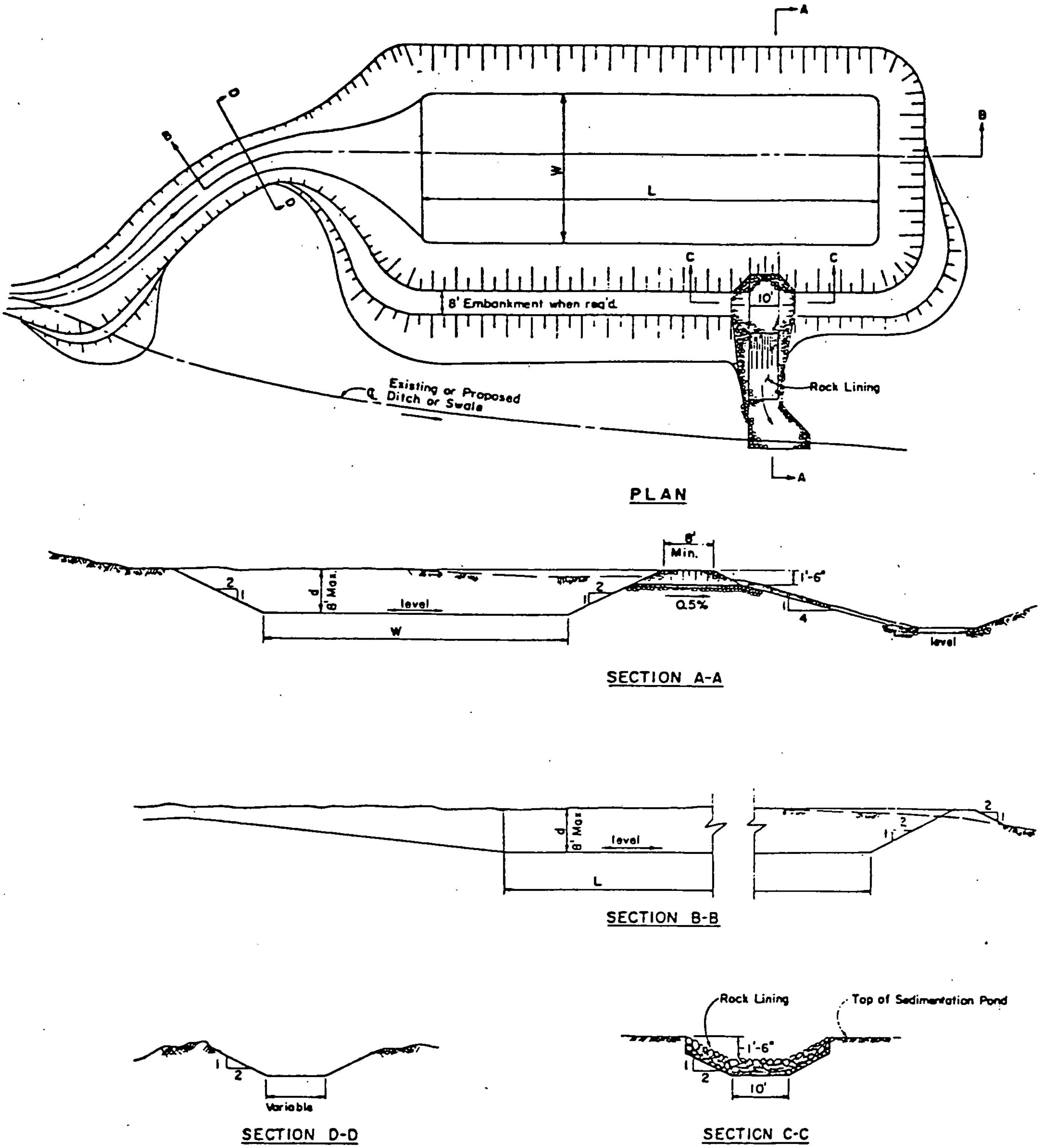


PLAN VIEW OF AN EMERGENCY SPILLWAY CUT INTO EXISTING GROUND

Commonwealth of Pennsylvania
DEPARTMENT OF TRANSPORTATION
BUREAU OF DESIGN

EROSION & SEDIMENT CONTROL

Recommended June 1, 1976	Approved June 1, 1976	Sht. 3 of 4
Director, Bureau of Design	Deputy Chief Hwy. Engr.	RC-7(



SEDIMENTATION POND - TYPE 2

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