PRELIMINARY REPORT

DEVELOPMENT OF DESIGN CRITERIA FOR SITING AND SPACING OF GRADE CONTROL STRUCTURES, PHASE I

Prepared for:

U.S. Army Engineer Research and Development Center
Coastal and Hydraulics Laboratory
Vicksburg, MS 39180

August 2002

Colorado State University
Engineering Research Center
Fort Collins, CO 80523

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PRELIMINARY REPORT

DEVELOPMENT OF DESIGN CRITERIA FOR SITING AND SPACING OF GRADE CONTROL STRUCTURES, PHASE I

1.0 INTRODUCTION

Grade control structures are an essential component to establishing stability in a degrading channel; however, rational engineering criteria for siting and spacing of grade control structures continue to elude the engineering community.

System-wide channel instability is often encountered as land use change or ill-conceived channel improvement projects change the fundamental relationship between the water and sediment supplied to a stream and the capacity of the stream to transport that water and sediment. As sediment is eroded at one site the channel conveyance for flood control may increase; however, as that sediment is deposited downstream, flood conveyance is diminished. In addition to channel degradation and aggradation destroying infrastructure and habitat, previously mapped floodplain relationships along these streams are disrupted, and flood control planning and zoning must be abandoned until stability is re-established in the stream.

2.0 PROJECT OBJECTIVE

Of the many tasks facing the engineering community, one of the most demanding is the stabilization of degrading channels. Channel degradation leads to the damage of transportation and drainage infrastructure, as well as planning and zoning of flood protection. The most common method of stabilizing degrading channels is the construction of in-channel grade control structures.

While the literature is sated with design criteria for the structural and hydraulic design of specific types of grade control structures, there is very little published guidance for selecting the most appropriate type of structure for the imposed conditions and the siting of the structures. As water resource projects become more and more complex, there is an increasing need for grade control structures to be utilized in a much broader sense to provide for habitat sustainability as well as erosion control. Design guidance for selecting and siting structures is essential. **The**

objective of this project is to develop and disseminate design guidance for selecting and siting grade control structures.

To accomplish the stated objective, a scope of work was developed and tasks for Phase 1 and Phase 2 are listed below.

Phase 1

1. Conduct 6 initial site visits to candidate grade control sites and make an assessment of available data.
2. Develop list of controlling factors that should be considered when selecting and siting grade control structures. This would include factors such as flood control impacts, geotechnical considerations, tributary locations, local drainage, environmental considerations, channel alignment, safety, etc.
3. Investigate applicable models for use in simulating watersheds stabilization using grade control structures.
4. Provide a brief progress report.

Phase 2

5. Conduct initial site visits to an additional 6 candidate grade control sites and make an assessment of available data.
6. Conduct detailed field investigation that will include 8 to 10 weeks of survey at selected sites (estimated to be 5 or 6 sites), with geomorphic assessment of each site. This assessment will address past and present morphology, hydrology and hydraulics of the site, as well as structure specifications, construction history, and maintenance. Sites will be selected in cooperation with sponsoring agency personnel.
7. Assess the applicability of existing methods (slope-area method, sediment routing, sediment and water continuity) for siting structures based on observed response of constructed grade control structures.
8. Conduct preliminary testing of the simulation technique selected in Phase 1.
9. Prepare a report summarizing the results of Phase 1 and Phase 2.

At the present stage of this research, Phase 1 tasks are completed.

3.0 PHASE 1 PROGRESS SUMMARY

The following sections summarize Phase 1 progress.

3.1 Initial Site Visits (Task 1)

Table 3.1 provides summary information pertaining to the grade control sites visited. Each of the eleven sites has been visited and preliminary information has been collected.

Table 3.1: Proposed Drop Structure Inspection Sites

No.	Name	Location	Watershed	Structure Type	Bed Material
1	Burney Branch	Mississippi	Suburban	Sheet Pile w/storage	Sand
2	Blue Creek	Illinois	Rural	Loose Rock	Silt/Sand
3	Blue River	Colorado	Urban	Loose Rock	Cobble/Gravel
4	Brush Creek	Missouri	Urban	Loose Rock	Sand Stone
5	Hotopha	Mississippi	Rural	Sheet Pile	Sand
6	Little Snake	Colorado	Rural	Large Boulder	Cobble/Gravel
7	McKee Creek	Illinois	Rural	Loose Rock	Silt/Sand
8	Middle Worsham	Mississippi	Rural	Sheet Pile	Sand
9	Perry Creek	Mississippi	Rural	Sheet Pile	Sand
10	Red Banks	Mississippi	Rural	Loose Rock	Sand
11	South Fork Little Snake	Wyoming & Colorado	Rural	Large Boulder	Sand

3.2 List of Important Factors in Design (Task 2)

Task 2 requires a list of factors that are important in the design and siting of grade control structures.

3.2.1 Structure Function

One important consideration is to select the function of the structure, which may either be predominately bed control or hydraulic control. There are basically two functions of grade control structures. One type of structure is designed to provide a hard point in the streambed that is capable of resisting erosive forces in the degradational zone. This is somewhat analogous to locally increasing the size of the bed material. Lanes (1955) relation would illustrate the situation by $QS^{+} \propto Q_s D_{50}^{+}$, where the increased slope (S^{+}) of the degradational reach would be offset by an increase in the bed material size (D_{50}^{+}). For this discussion, this will be referred to as a *bed control structure*. The second type of structure is designed to function by reducing the

energy slope along the degradational zone ($QS \propto Q_s D_{50}$). This will be referred to as a *hydraulic control structure*. The distinction between the processes by which these structures operate is important whenever grade control structures are considered.

Because of the complex hydraulic behavior of the flow over grade control structures, it is difficult to designate a single function that will apply without exception to each structure. For many situations, the function of a structure as either a bed control structure or hydraulic control structure is readily apparent. However, there may be circumstances where the function as either strictly a bed control or hydraulic control structure may be less evident and, in many cases, the structure may actually have characteristics of both. It also must be recognized that the hydraulic performance and, therefore, the function of the structure, can vary with time and discharge. This can occur within a single hydrograph or over a period of years as a result of upstream or downstream channel changes.

3.2.2 Type of Structure

The type of structure is a second important general decision in siting and spacing grade control structures. There are certain features that are common to most grade control structures. These include a control section for accomplishing the grade change, a section for energy dissipation, and protection of the upstream and downstream approaches. However, there is considerable variation in the design of these features. For example, a grade control structure may be constructed of riprap, concrete, sheet piling, treated lumber, soil cement, gabions, compacted earth fill, or other locally available material. Also, the shape (sloping or vertical drop) and dimensions of the structure can vary significantly, as can the various appurtenances (baffle plates, end sills, etc.). The applicability of a particular type of structure to any given situation depends upon a number of factors such as: hydrologic conditions, sediment size and loading, channel morphology, floodplain and valley characteristics, availability of construction materials, project objectives, as well as the inevitable time and funding constraints. The successful use of a particular type of structure in one situation does not necessarily ensure it will be effective in another. Some of the more common types of grade control structures used in a variety of situations are discussed in the following sections. For more information on various structure designs, the reader is referred to Neilson *et al.* (1991), which provides a comprehensive international literature review on grade control structures with an annotated bibliography.

3.2.2.1 Simple Bed Control Structures

Perhaps the simplest form of a grade control structure consists of dumping rock, concrete rubble, or some other locally available non-erodible material across the channel to form a hard point. These structures are often referred to as rock sills or bed sills. These types of structures are generally most effective in small stream applications and where the drop heights are generally less than about 2 to 3 feet. A series of rock sills, each creating a head loss of about 2 feet, was used successfully on the Gering Drain in Nebraska (Stufft, 1965). The design concept presented by Whitaker and Jaeggi (1986) for stabilizing the streambed with a series of rock sills is shown in Figure 3.1. The sills in Figure 3.1 are classic bed control structures, which are simply acting as hard points to resist the erosion of the streambed.

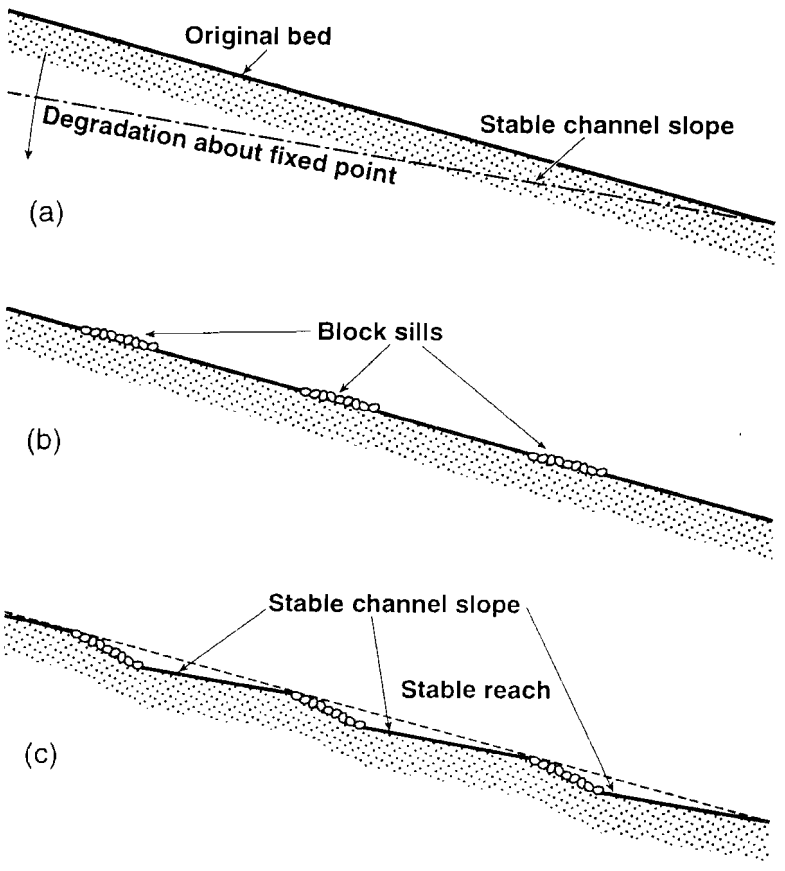


Figure 3.1: Channel Stabilization With Rock Sills (adapted from Whitaker and Jaeggi, 1986)

Construction of bed sills is sometimes accomplished by simply placing the rock along the streambed to act as a hard point to resist the erosive forces within the degradational zone. In other situations, a trench may be excavated across the streambed and then filled with rock. A critical component in the design of these structures is ensuring that there is a sufficient volume of non-erodible material to resist the general bed degradation, as well as any additional local scour at the structure. This is illustrated in Figure 3.2, which shows a riprap grade control structure designed to resist both the general bed degradation of the approaching knickpoint as well as any local scour that may be generated at the structure. In this instance, the riprap section must have sufficient mass to launch with an acceptable thickness to the anticipated scour hole depth.

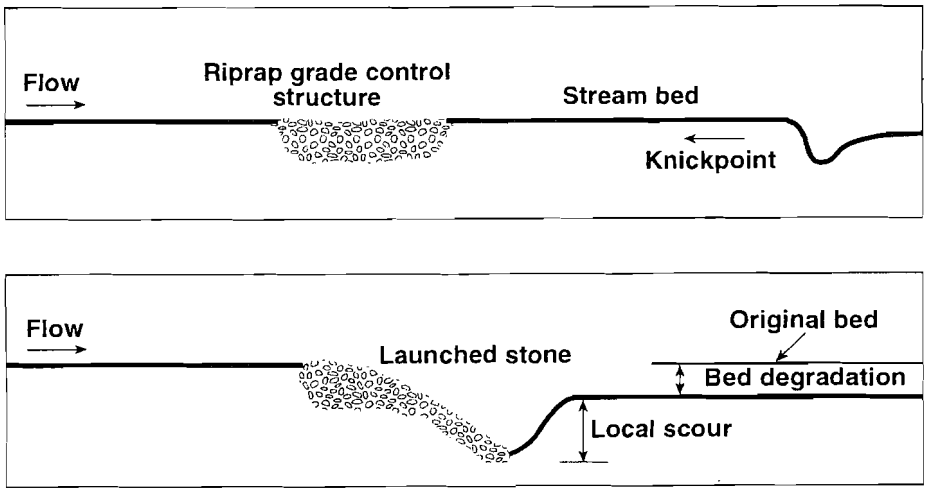


Figure 3.2: The Upper Drawing is a Built Riprap Grade Control Structure With Sufficient Launch Stone to Handle Anticipated Scour, and the Lower Drawing Indicates Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour

3.2.2.2 Structures With Water Cutoff

One problem often encountered with the above structures is the displacement of rock (or rubble, etc.) due to the seepage flow around and beneath the structure. This is particularly a problem when the bed of the channel is composed primarily of pervious material. This problem can be eliminated by constructing a water barrier at the structure. One type of water barrier consists of simply placing a trench of impervious clay fill upstream of the weir crest. This type of water barrier is illustrated in Figure 3.3. One problem with this type of barrier is the limited longevity due to susceptibility to erosion. This problem can be avoided by using concrete or

sheet piling for the cutoff wall. The conceptual design of a riprap grade control structure with a sheet pile cutoff wall is shown in Figure 3.4. In the case of the sloping riprap drop structures used by the Denver Urban Drainage and Flood Control District, an impervious clay fill is used in conjunction with a lateral cutoff wall (McLaughlin Water Engineers, Ltd., 1986). This design is illustrated in Figure 3.5.

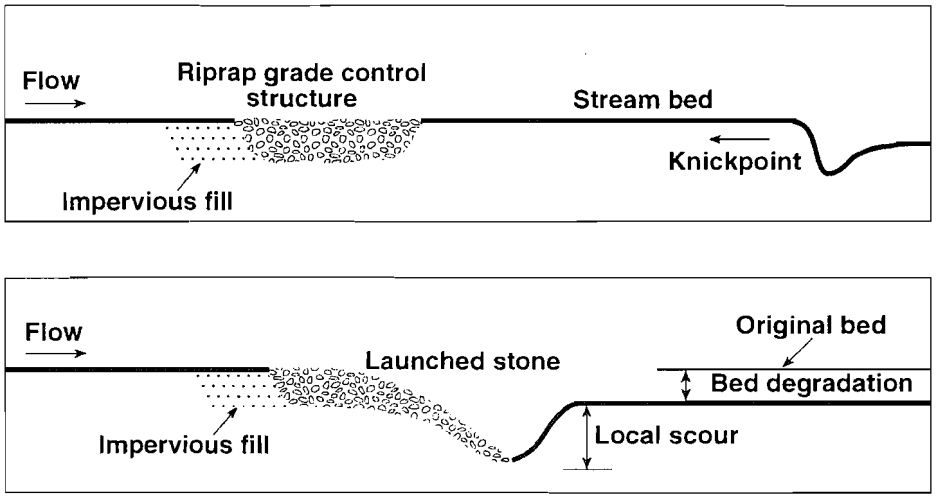


Figure 3.3: The Upper Drawing is a Built Riprap Grade Control Structure With Impervious Fill Cutoff Wall, and the Lower Drawing Indicates Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour

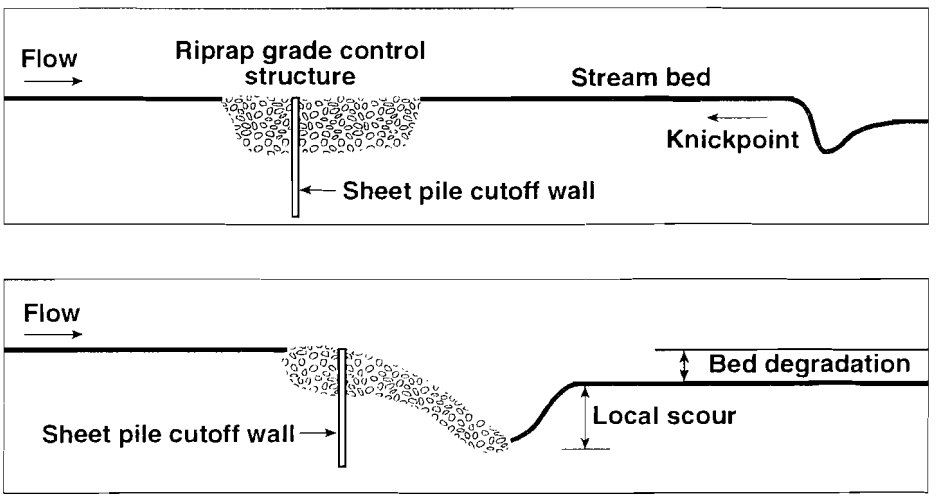


Figure 3.4: The Upper Drawing is a Built Riprap Grade Control Structure With Sheet Pile Cutoff Wall, and the Lower Drawing Indicates Launching of Riprap at Grade Control Structure in Response to Bed Degradation and Local Scour

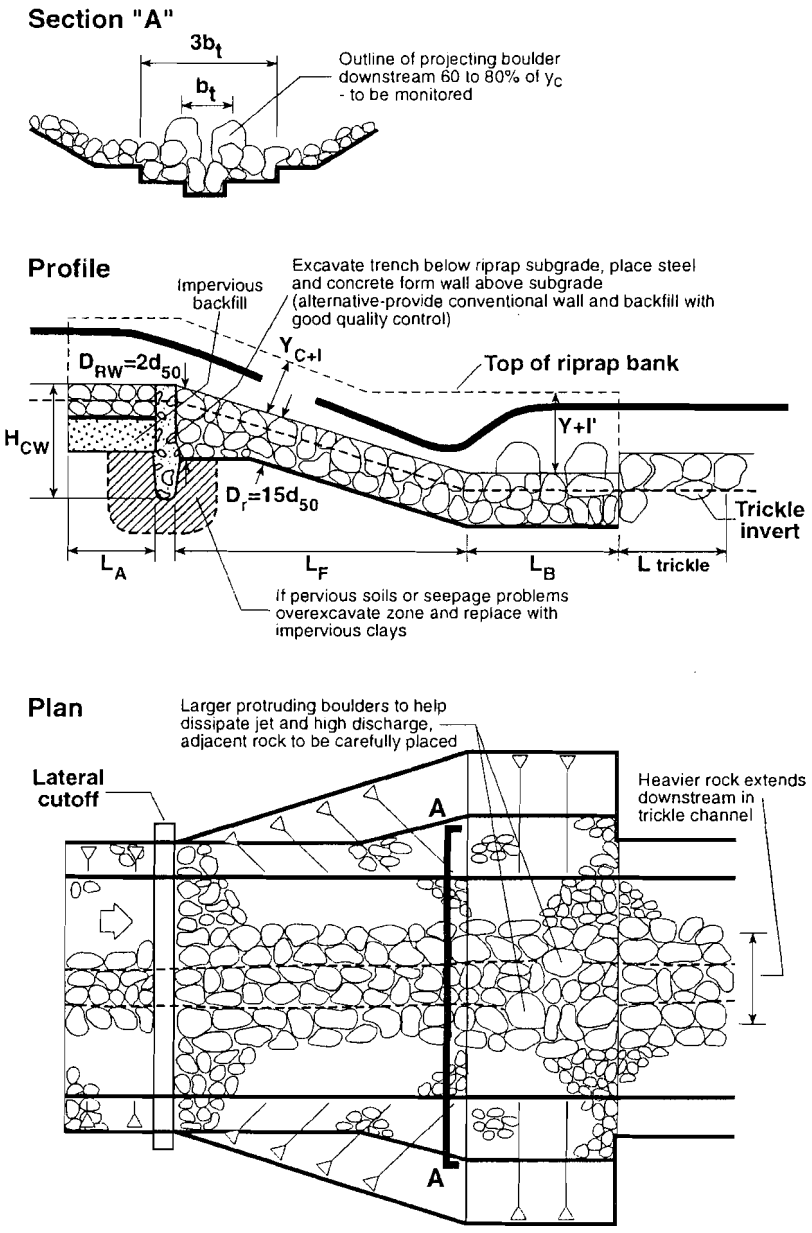


Figure 3.5: Sloping Drop Grade Control Structure With Pre-formed Riprap Lined Scour Hole (McLaughlin Water Engineers, 1986)

3.2.2.3 Structures with Pre-formed Scour Holes

A significant feature that distinguishes the sloping riprap structure of Figure 3.5 from the other structures is the pre-formed, rock protected, scour hole. A scour hole is a natural occurrence downstream of any drop whether it is a natural overfall or a man-made structure. A rock grade control structure must have sufficient launching rock to protect against the vertical scour immediately below the weir section. However, the lateral extent of the scour hole must also be considered to ensure that it does not become so large that the structure is subject to being flanked. With many simple grade control structures in small stream applications, very little, if any, attention is given to the design of a stilling basin or pre-formed scour hole. The erosion is allowed to form the scour hole. However, at higher flow and drop situations, a pre-formed scour hole protected with concrete, riprap, or some other erosion resistant materials is usually required to dissipate energy and to eliminate uncertainties in the size and location of the resultant scour hole. This scour hole serves as a stilling basin for dissipating the energy of the plunging flow. Sizing of the scour hole is a critical element in the design process, which is usually based on model studies or on experience with similar structures in the area.

The stability of rock structures is often jeopardized at low tailwater conditions due to the stability of the rock, which is often the limiting factor in determining the maximum drop height of the structure. One way to ensure the stability of the rock is to design the structure to operate in a submerged condition. This is the basis for design of the bed stabilizer shown in Figure 3.6 (U.S. Army Corps of Engineers (USACE), 1970). These structures generally perform satisfactorily when designed to operate at submerged conditions where the tailwater elevation (T) does not fall below 0.8 of the critical depth (D_c) at the crest section (Linder, 1963). Subsequent monitoring of the in place structures confirmed the successful performance in the field (USACE, 1981).

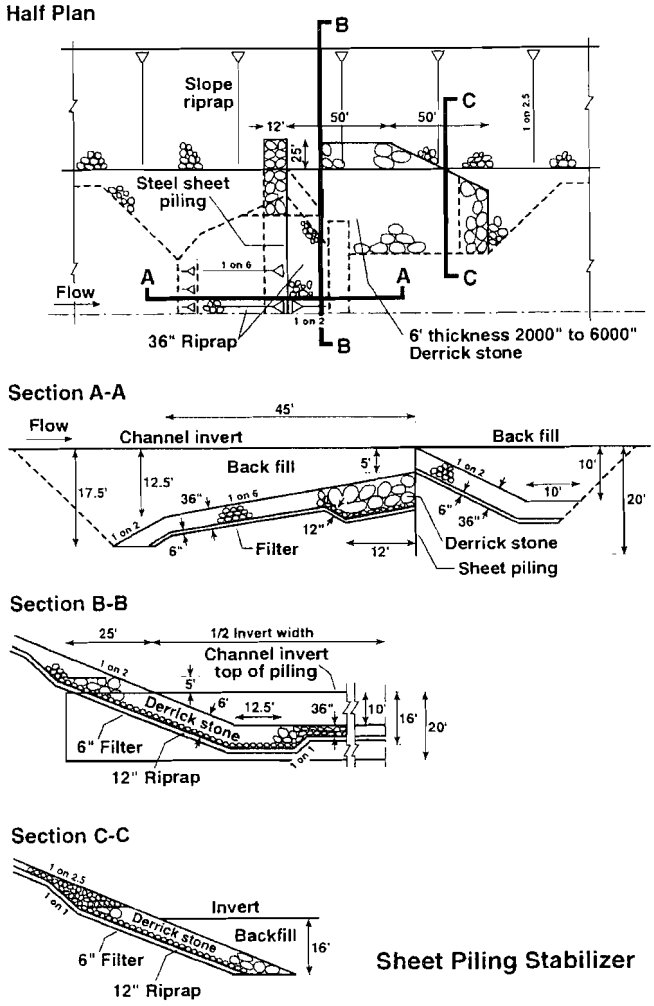


Figure 3.6: Bed Stabilizer Design With Sheet Pile Cutoff (USACE, 1970)

In many instances, energy dissipation in a grade control structure is accomplished by the plunging action of the flow into the riprap protected stilling basin. This is generally satisfactory where the degree of submergence is relatively high due to small drop heights and/or high tailwater conditions. However, at lower submergence conditions where drop heights are large or tailwater is low, additional means of dissipating the energy are often required. Little and Murphey (1982) observed that an undular hydraulic jump occurs when the incoming Froude number is less than 1.7. Consequently, Little and Murphey developed a grade control design that included an energy dissipating baffle to break up these undular waves (Figure 3.7). This structure, referred to as the ARS type low-drop structure, has been used successfully in north

Mississippi for drop heights up to about 2 meters by both the USACE and the U.S. Soil Conservation Service (SCS) (USACE, 1981). A recent modification to the ARS structure was developed following model studies at Colorado State University (Johns *et al.*, 1993; and Abt *et al.*, 1994). The modified ARS structure, presented in Figure 3.8 retains the baffle plate but adopts a vertical drop at the sheet pile rather than a sloping rock-fill section.

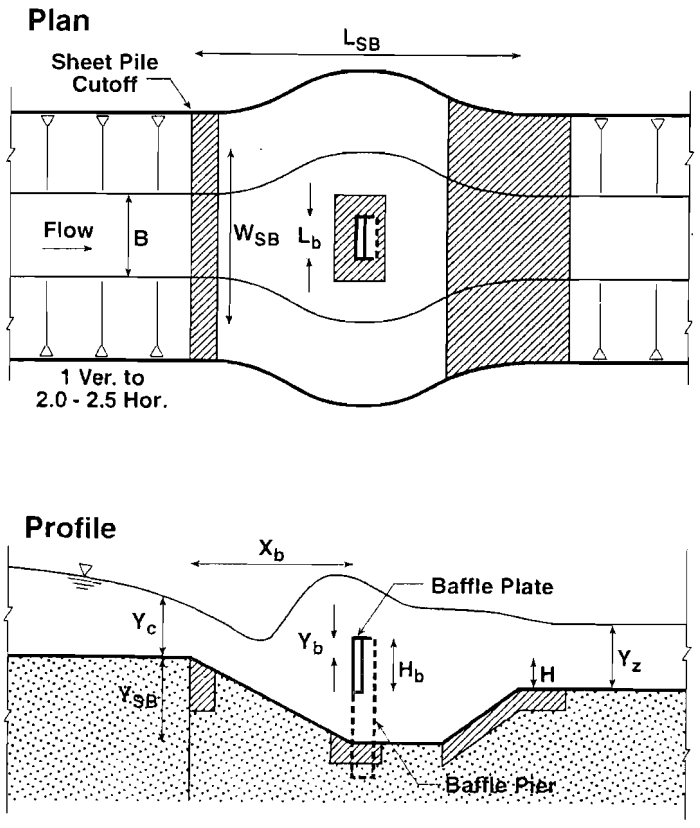


Figure 3.7: ARS-Type Grade Control Structure With Pre-formed Riprap Lined Stilling Basin and Baffle Plate (adapted from Little and Murphey, 1982)

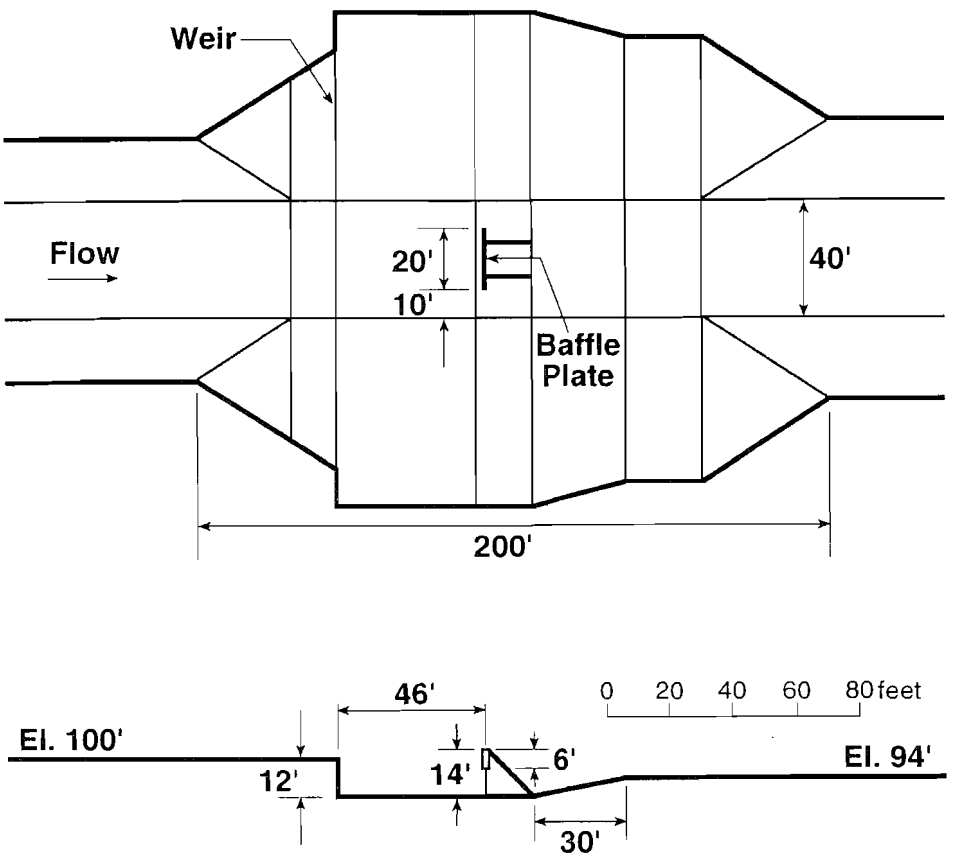
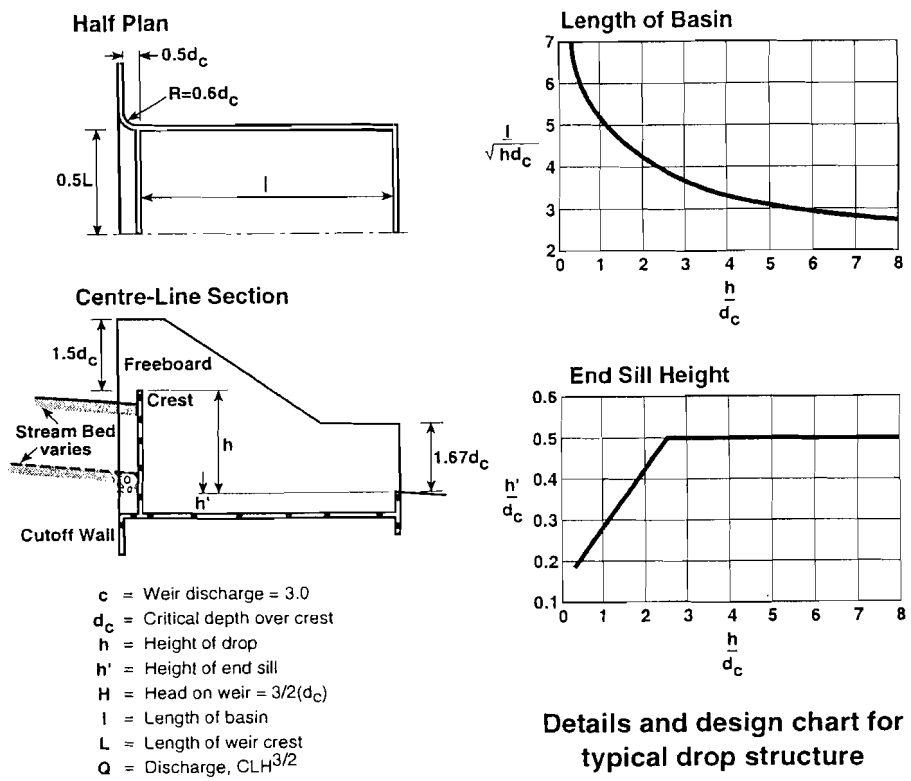


Figure 3.8: Schematic of Modified ARS-Type Grade Control Structure (Abt *et al.*, 1994)

3.2.2.4 Concrete Drop Structures

In many situations where the discharge and/or drop heights are large, grade control structures are normally constructed of concrete. There are many different designs for concrete grade control structures. The two discussed herein are the CIT and the St. Anthony Falls (SAF) structures. Both of these structures were utilized on the Gering Drain project in Nebraska, where the decision to use one or the other was based on the flow and channel conditions (Stufft, 1965). Where the discharges were large and the channel depth was relatively shallow, the CIT type of drop structure was utilized. The CIT structure is generally applicable to low-drop situations where the ratio of the drop height to critical depth is less than one; however, for the Gering Drain project this ratio was extended up to 1.2. The original design of this structure was based on criteria developed by Vanoni and Pollack (1959). The structure was then modified by model studies at the Waterway Experiment Station (WES) in Vicksburg, Mississippi, and is shown in Figure 3.9 (Murphy, 1967). Where the channel was relatively deep and the discharges

smaller, the SAF drop structure was used. This design was developed from model studies at the SAF Hydraulic Laboratory for the SCS (Blaisdell, 1948). This structure is shown in Figure 3.10. The SAF structure is capable of functioning in flow situations where the drop height to critical depth ratio is greater than one and can provide effective energy dissipation within a Froude number range of 1.7 to 17. Both the CIT and the SAF drop structures have performed satisfactorily on the Gering Drain for over 25 years.



Details and design chart for typical drop structure

Figure 3.9: CIT-Type Drop Structure (Murphy, 1967)

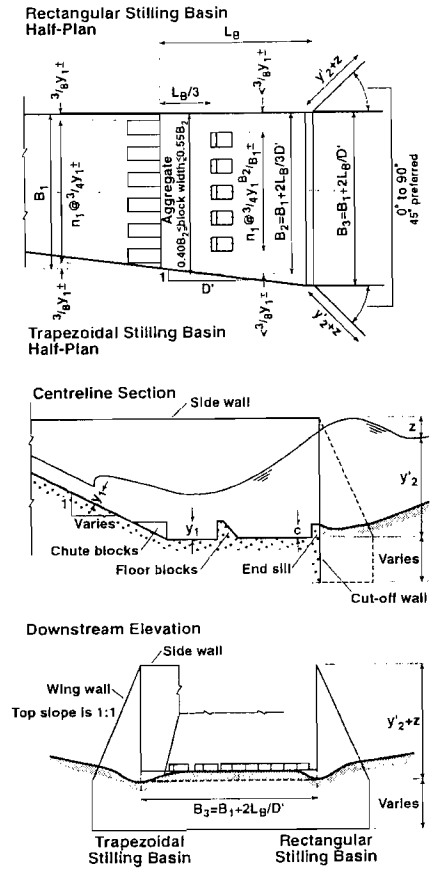


Figure 3.10: St. Anthony Falls (SAF) Type Drop Structure (Blaisdell, 1948)

3.2.2.5 Channel Linings

Grade control can also be accomplished by lining the channel bed with a non-erodible material. These structures are designed to ensure that the drop is accomplished over a specified reach of the channel, which has been lined with riprap or some other non-erodible material. Rock riprap gradient control structures have been used by the SCS for several years (SCS, 1976). These structures are designed to flow in the subcritical regime with a constant specific energy at the design discharge, which is equal to the specific energy of flow immediately upstream of the structure (Myers, 1982). Although these structures have generally been successful, there have been some associated local scour problems. This precipitated a series of model studies at the WES to correct these problems and to develop a design methodology for these structures (Tate, 1988, 1991). A plan and profile drawing of the improved structure is shown in Figure 3.11.

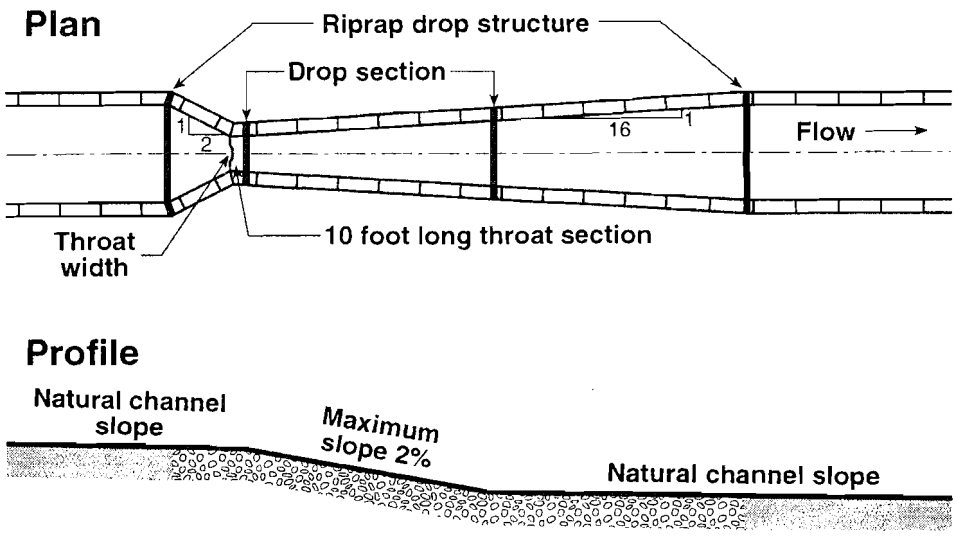


Figure 3.11: Riprap Lined Drop Structures (adapted from Tate, 1991)

3.2.3 Alternative Construction Materials

While riprap and concrete may be the most commonly used construction materials for grade control structures, there are many situations where cost or availability of materials may prompt the engineer to consider other alternatives. Gabion grade control structures are often an effective alternative to standard riprap or concrete structures (Hanson *et al.*, 1986). Guidance for the construction of gabion weirs is also provided by the USACE (1974).

Another alternative to the conventional riprap or concrete structure which has gained popularity in the southwestern United States. is the use of soil cement grade control structures. These structures are constructed of on site soil-sand in a mix with Portland Cement to form a high quality, erosion resistant mixture. Soil cement grade control structures are most applicable when used as a series of small drops in lieu of a single large-drop structure. Experience has indicated that a limiting drop height for these structures is on the order of 1 meter. Design criteria for these structures are presented by Simons and Li (1982).

3.2.4 Effectiveness of Grade Control Structures

Design considerations for improving the effectiveness of grade control structures include determination of the type, location, and spacing of structures along the stream, along with the elevation and dimensions of structures. Siting grade control structures is often considered a simple optimization of hydraulics and economics. However, hydraulics and economics alone are usually not sufficient to define the optimum spacing for grade control structures. In practice, the hydraulic considerations must be integrated with a host of other factors, which vary from site to site, to determine the final structure plan. Each of these factors should be considered in determining the effectiveness of the structures.

One of the most important steps in the siting of a grade control structure or a series of structures is the determination of the anticipated drop at the structure. This requires some knowledge of the ultimate channel morphology, both upstream and downstream of the structure, which involves assessment of sediment transport and channel morphologic processes.

The hydraulic spacing of grade control structures is a critical element of the design process, particularly when a series of structures is planned. The design of each structure is based on the anticipated tailwater or downstream bed elevation which, in turn, is a function of the next structure downstream. Heede and Mulich (1973) suggested that the optimum spacing of structures is such that the upstream structure does not interfere with the deposition zone of the next downstream structure. Mussetter (1982) showed that the optimum spacing should be the length of the deposition above the structure that is a function of the deposition slope (Figure 3.12). Figure 3.12 also illustrates the recommendations of Johnson and Minaker (1944), that the most desirable spacing can be determined by extending a line from the top of the first structure at a slope equal to the maximum equilibrium slope of sediment upstream until it intersects the original streambed. However, each of the above references implicitly includes a specific sediment supply concentration, and that concentration is necessary for rational designs.

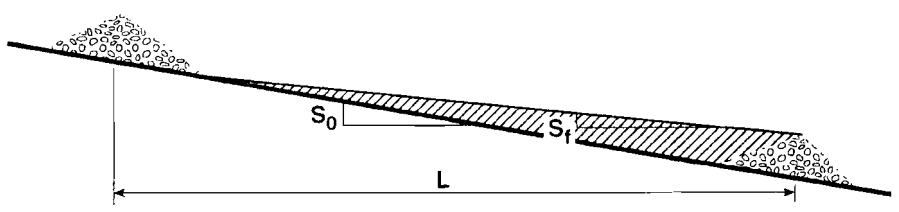


Figure 3.12: Spacing of Grade Control Structure (adapted from Mussetter, 1982)

If flow is at normal depth with the bed slope and energy slope equal, the hydraulic spacing of grade control structures is straightforward, as shown by the following equation:

$$H = (S_o - S_f)x \tag{3.1}$$

where H is the amount of drop to be removed from the reach, S_o is the original bed slope and energy, S_f is the final, or equilibrium bed and energy slope, and x is the length of the reach (Goitom and Zeller, 1989). The number of structures (N) required for a given reach can then be determined by:

$$N = H/h \tag{3.2}$$

where h is the selected drop height of the structure.

It follows from Eq. (3.1) that one of the most important factors when siting grade control structures is the determination of the equilibrium slope (S_f). Failure to properly define the equilibrium slope can lead to costly, overly conservative designs, or inadequate design resulting in continued maintenance problems and possible complete failure of the structures. Clearly, equilibrium slope (S_f) is a function of the sediment supply and is the slope required to transport the sediment supplied. The relationship between the sediment supply and the transport capacity upstream of the structure is strongly affected by the hydraulic characteristics of the structure control section, the extent of initial backwater, and the duration of flows that are controlled. Later sections of the final report will consider models for determining an equilibrium slope.

In reality, the normal depth assumption would rarely occur in an unstable, degrading channel at the time of grade control structure emplacement. As described by Chow (1959), the M2 curve is formed when the depth of flow at the downstream end of a reach decreases to less than normal depth, for example, as the flow approaches a headcut or over-steepend reach (Newbury, 2002). The upstream extent of the M2 curve approaches normal depth (Dn). In contrast to the M2, Chow (1959) described the M1 curve being formed as the flow is obstructed at the downstream extent of a reach by a grade control structure, debris, ice jam or other similar feature. The M1 or backwater curve is above normal depth of flow, and at the upstream extent the flow approaches normal depth (Dn). Both these curves are shown in Figure 3.13, taken from Newbury (2002).

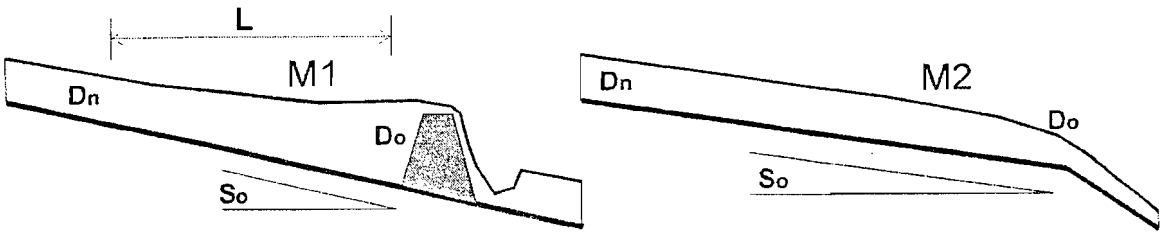


Figure 3.13: M1 and M2 Backwater Curves in Mildly Sloping Channels Caused by Changes in the Depth of Flow at the Downstream end of A Reach (from Newbury, 2002)

As the discharge increases, the flow may shift from an M1 to an M2 or to normal depth of flow at a grade control structure. If the structure is properly designed to induce upstream equilibrium, the M1 curve should occur in the lower range of flows and should persist until normal depth of flow occurs. At that normal-depth flow, the structure is no longer providing a hydraulic control and the stream energy profile is a function of channel characteristics such as bed, bank, and planform roughness. The designer must decide the ranges of flow over which an M1 curve or a normal depth will persist, and the slope at which normal depth will occur. The decisions must be based on an equilibrium of sediment transport and supply, and are tempered by the factors herein discussed.

3.2.5 Downstream Channel Response

Since grade control structures affect the sediment delivery to downstream reaches, it is necessary to consider the potential impacts to the downstream channel when grade control structures are planned. Bed control structures reduce the downstream sediment loading by preventing the erosion of the bed and banks, while hydraulic control structures have the added effect of trapping sediments. The ultimate response of the channel to the reduction in sediment supply will vary from site to site. In some instances, the effects of grade control structures on sediment loading may be so small that downstream degradational problems may not be encountered. However, in some situations such as when a series of hydraulic control structures is planned, the cumulative effects of sediment trapping may become significant. In these instances, it may be necessary to modify the plan to reduce the amount of sediment being trapped or to consider placing additional grade control structures in the downstream reach to protect

against the induced degradation. Therefore, following the hydraulic spacing of a series of grade control structures using a thorough investigation of providing a balance between supply and transport of water and sediment, the designer must utilize a long-term sediment routing model, to investigate downstream channel response.

3.2.6 Geotechnical Considerations

The above discussion focused only on the hydraulic aspects of design and siting of grade control structures. However, in some cases, the geotechnical stability of the reach may be an important or even the primary factor to consider when siting grade control structures. This is often the case where channel degradation has caused, or is anticipated to cause, severe bank instability due to exceedance of the critical bank height (Thorne and Osman, 1988). When this occurs, bank instability may be widespread throughout the system rather than restricted to the concave banks in bendways. Traditional bank stabilization measures may not be feasible in situations where system-wide bank instabilities exist. In these instances, grade control, aimed at preventing the onset of incision-triggered mass-wasting, may be the more appropriate solution.

Grade control structures can enhance the bank stability of a channel in several ways. Bed control structures indirectly affect the bank stability by stabilizing the bed, thereby reducing the length of bankline that achieves an unstable height. With hydraulic control structures, two additional advantages are apparent with respect to bank stability: (1) bank heights are reduced due to sediment deposition upstream of the structure, which increases the stability of the banks with regard to mass failure; and (2) by creating a backwater situation, velocities and scouring potential are reduced, which reduces or eliminates the severity and extent of basal cleanout of the failed bank material, thereby promoting self healing of the banks (Thorne, 1999).

3.2.7 Flood Control Impacts

Channel improvements for flood control and channel stability often appear to be mutually exclusive objectives. For this reason, it is important to ensure that any increased post-project flood potential is identified. This is particularly important when hydraulic control structures are considered. In these instances the potential for causing overbank flooding may be the limiting factor with respect to the height and amount of constriction at the structure. Grade control structures are often designed to be hydraulically submerged at flows less than bankfull so that the

frequency of overbank flooding is not affected. However, if the structure exerts control through a wider range of flows including overbank, then the frequency and duration of overbank flows may be impacted. When this occurs, the impacts must be quantified and appropriate provisions such as acquiring flowage easements or modifying structure plans should be implemented.

Another factor that must be considered when designing grade control structures is the safe return of overbank flows back into the channel. This is particularly a problem when the flows are out of bank upstream of the structure but still within bank downstream. The resulting head differential can cause damage to the structure as well as severe erosion of the channel banks depending upon where the flow re-enters the channel. Some means of controlling the overbank return flows must be incorporated into the structure design. One method is simply to design the structure to be submerged below the top bank elevation, thereby reducing the potential for a head differential to develop across the structure during overbank flows. If the structures exert hydraulic control throughout a wider range of flows including overbank, then a more direct means of controlling the overbank return flows must be provided. One method is to ensure that all flows pass only through the structure. This may be accomplished by building an earthen dike or berm extending from the structure to the valley walls that prevents any overbank flows from passing around the structure (Forsythe, 1985). Another means of controlling overbank flows is to provide an auxiliary high-flow structure which will pass the overbank flows to a specified downstream location where the flows can re-enter the channel without causing significant damage (Hite and Pickering, 1982).

3.2.8 Environmental Considerations

Projects must work in harmony with the natural system to meet the needs of the present, without compromising the ability of future generations to meet their needs. Engineers and geomorphologists are responding to this challenge by trying to develop new and innovative methods for incorporating environmental features into channel projects. The final siting of a grade control structure is often modified to minimize adverse environmental impacts to the system.

Grade control structures can provide direct environmental benefits to a stream. Cooper and Knight (1987) conducted a study of fisheries resources below natural scour holes and man-made pools below grade control structures in north Mississippi. They concluded that although

there was greater species diversity in the natural pools, there was increased growth of game fish and a larger percentage of harvestable-size fish in the man-made pools. They also observed that the man-made pools provided greater stability of reproductive habitat. Shields *et al.* (1990) reported that the physical aquatic habitat diversity was higher in stabilized reaches of Twentymile Creek, Mississippi than in reaches without grade control structures. They attributed the higher diversity values to the scour holes and low-flow channels created by the grade control structures. The use of grade control structures as environmental features is not limited to the low-gradient sand bed streams of the southeastern United States. Jackson (1974) documented the use of gabion grade control structures to stabilize a high-gradient trout stream in New York. She observed that following construction of a series of bed sills, there was a significant increase in the density of trout. The increase in trout density was attributed to the accumulation of gravel between the sills, which improved the spawning habitat for various species of trout.

Perhaps the most serious negative environmental impact of grade control structures is the obstruction to fish passage. In some cases, particularly when drop heights are small, fish are able to migrate upstream past a structure during high flows (Cooper and Knight, 1987). However, in situations where structures are impassable, and where the migration of fish is an important concern, openings, fish ladders, or other passageways must be incorporated into the design of the structure to address fish movement problems (Nunnally and Shields, 1985). The various methods of accomplishing fish movement through structures are not discussed here. Interested readers are referred to Nunnally and Shields (1985), Clay (1961), and Smith (1985) for more detailed discussions.

The environmental aspects of the project must be an integral component of the design process when siting grade control structures. A detailed study of all environmental features in the project area should be conducted early in the design process. This will allow these factors to be incorporated into the initial plan rather than having to make costly and often less environmentally effective last minute modifications to the final design. Unfortunately, there is very little published guidance concerning the incorporation of environmental features into the design of grade control structures. One source of useful information can be found in the following technical reports published by the Environmental Laboratory of the USACE, WES (Shields and Palermo, 1982; Henderson and Shields, 1984; and Nunnally and Shields, 1985).

The above discussion illustrates that the design of grade control structures is not simply a hydraulic exercise. Rather, there are many other factors that must be included in the design process. For any specific situation, some or all of the factors discussed in this section may be critical elements in the final siting of grade control structures. It is recognized that this does not represent an all inclusive list since there may be other factors not discussed here that may be locally important. For example, in some cases, maintenance requirements, debris passage, ice conditions, or safety considerations may be controlling factors. Consequently, there is no definitive cookbook procedure for designing grade control structures that can be applied universally. However, consideration of each factor in an analytic and balanced fashion, and avoiding reliance on empirical procedures, can lead to effective and intelligent use of grade control structures.

3.3 Investigation of Other Models for Siting and Design of Grade Control Models

Relationships among parameters of basin geometry, flood discharge, slope, and sedimentary characteristics can be used to develop equilibrium slope design relationships. Hack (1957) proposed an empirical relationship of mainstream length (L in miles) to drainage basin area (A in square miles):

$$L = 1.4\,A^{0.6} \tag{3.3}$$

This relationship has subsequently been shown to apply for a wide range of conditions (Ijjasz-Vasquez *et al.*, 1993), as shown in Figure 3.14 based on data measured from the Yalobusha River basin in Mississippi.

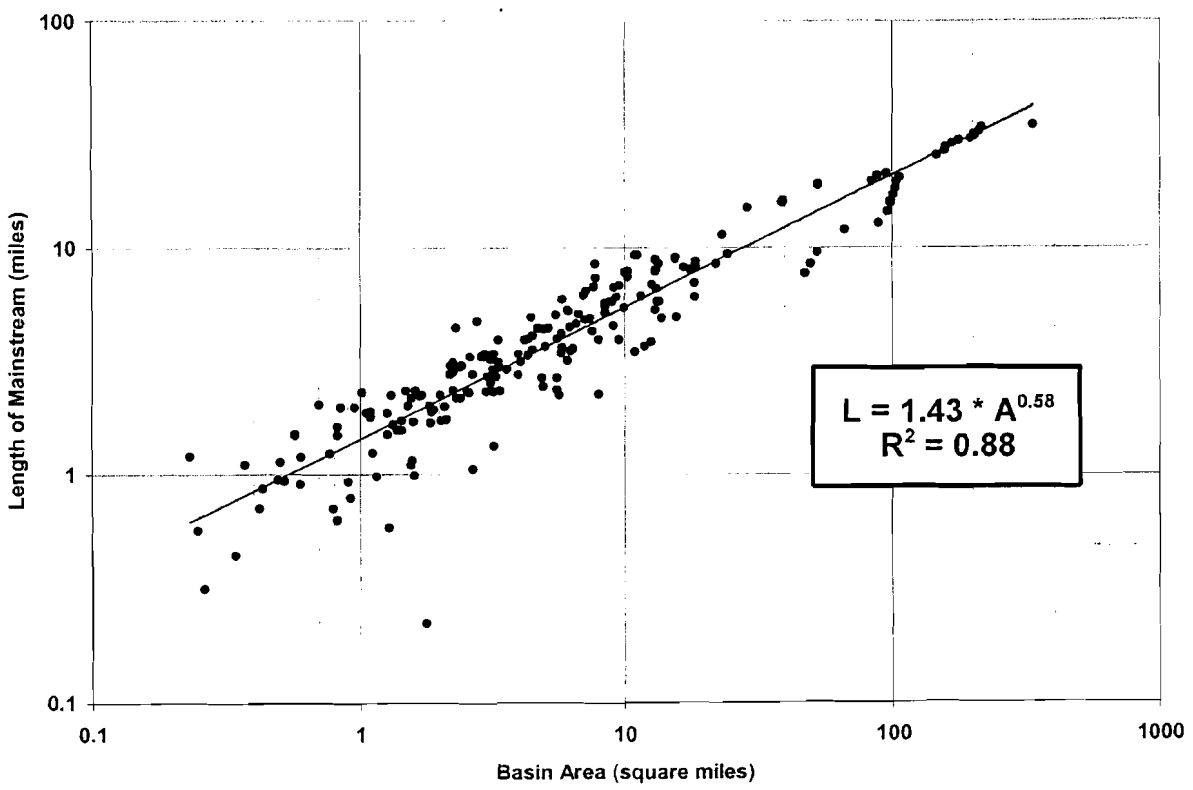


Figure 3.14: Relationship Between Mainstream Length and Basin Area in the Yalobusha Basin

Hack (1957) also proposed the following relationship relating the channel slope (S in ft/mi) to drainage area (A in square miles) and to the bed material size for which 50% are finer (d_{50} in mm) for streams in Virginia and Maryland:

$$S = a\left(\frac{d_{50}}{A}\right)^b \tag{3.4}$$

In his original paper, Hack (1957) utilized map-measured values of slope, not field-measured data, and found that $b = 0.6$. Schröder (1991) used local slope data from Hack (1957) and reported that $a = 0.0076$ and $b = 0.4$ for units of mm and square kilometers, respectively. Converting Schröder's analysis to the units originally used by Hack (1957), which are mm and square miles, the value of $a = 0.00517$ and $b = 0.4$, respectively. In addition to the Virginia and Maryland data used by Hack, Schröder added data from Germany, which when converted to original units of square miles and mm, yield $a = 0.00449$ and $b = 0.4$, respectively.

Schumm *et al.* (1984) utilized the Channel Evolution Model (CEM) to determine equlibrium reaches (Types IV and V), and Biedenharn (USACE, 1990) plotted the slope (S) as a function of drainage area (A in square miles) to develop Figure 3.15.

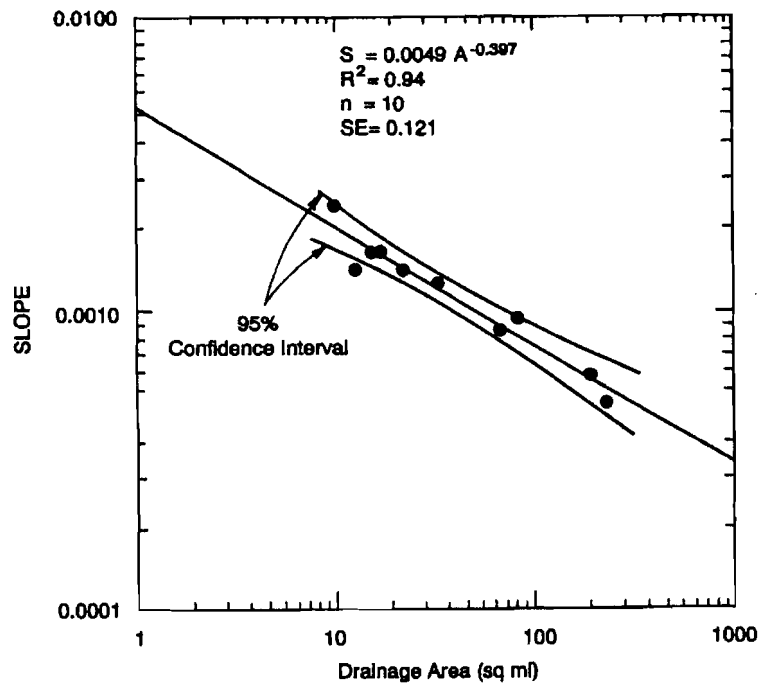


Figure 3.15: Slope-Drainage Area Relationship (from USACE, 1990)

As shown in Figure 3.15, the magnitude of the values of a and b are similar to those produced by the work of Schröder and Hack. For the figure, d_{50} is assumed to be constant at 0.3 mm.

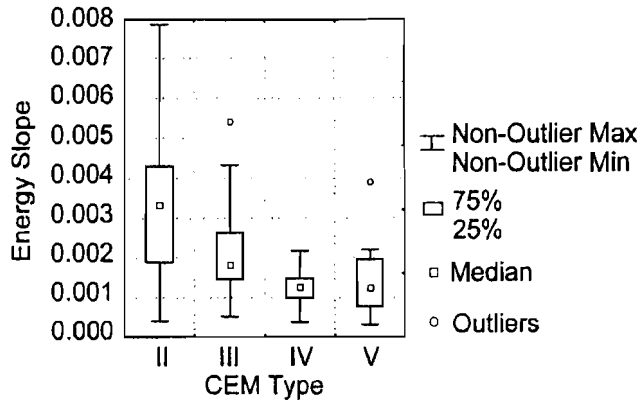
Another empirical analysis of the DEC data allows the designer to develop a level of comfort with the data, and perhaps guide the use of more complex models. The sediment transport capacity of each of the 26 Yazoo Basin study reaches was computed by Watson *et al.* (1996). These data indicate that sediment transport capacity decreases as the channel evolves to stability.

Data on hydraulic characteristics and CEM type from all streams were used to compare trends in slope, sediment transport capacity (shown as concentration), and specific stream power across evolutionary stages for a set of 26 stream reaches. Drainage area for the basins contributing to the 26 stream reaches ranged from 1.3 to 120 square miles, with a median value

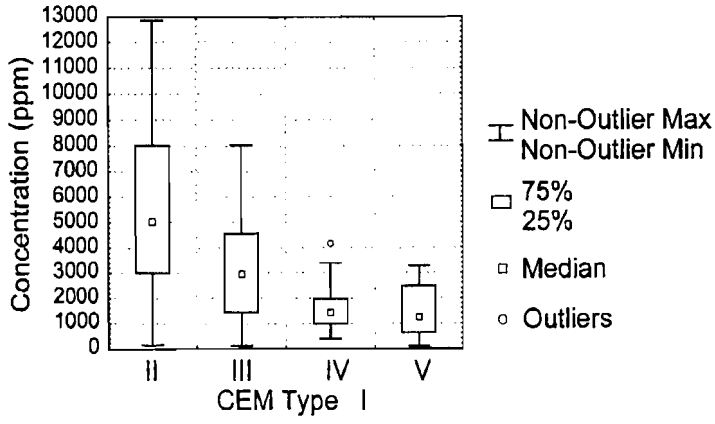
of 10.2 square miles. For these drainages, the 2-year recurrence interval discharge ranged from 6,200 cfs to 269 cfs, with a median discharge of 1,369 cfs. The assumption of uniform flow is made for this investigation, allowing the comparison of bed slopes and computed energy slopes. Box and whisker plots are shown in Figure 3.16 for the energy slope (a), the concentration (b), and the specific stream power (c) for all reaches at the 2-year recurrence interval discharge. The concentration was computed in parts per million (ppm) using the Brownlie (1981) sediment transport relationship for the 2-year recurrence interval discharge. The specific stream power was computed as the product of the specific weight of water, the 2-year recurrence interval discharge, and the energy slope, divided by the channel width. Units of specific discharge are in watts per square meter (14.56 W/m^2 = 1 ft-lb/sec/ft^2). As shown in this figure, each parameter (slope, concentration, specific stream power) decreases in value from the Type II to the Type V reaches. Box and whisker graphs indicate the median value, a box containing all the data between the 25th and 75th percentiles, and whiskers to the maximum and minimum non-outlier values. Outliers were defined as values that are outside the 25th and 75th percentile range by more than 1.5 times the difference between the 25th and 75th percentiles. This analysis represents the largest combined data set of CEM type and hydraulics analyzed to date. Although there is substantial variation in hydraulic attributes within each evolutionary stage, the expected trends are clearly present. Specific stream power appears to be an excellent predictor of channel stability, with most streams attaining relative stability at specific stream power less than 30 W/m^2 (Figure 3.16c).

The SAM procedure provides for computation of width, depth, and energy slope; whereas the slope-area method only provides slope. The SAM procedure is flexible, allowing a range of effective discharges and sediment concentrations to be utilized. An important feature of the SAM procedure is that minimum slope for a selected design concentration is computed, which means that degradation between structures and final slope following complete stream and watershed restoration can be predicted.

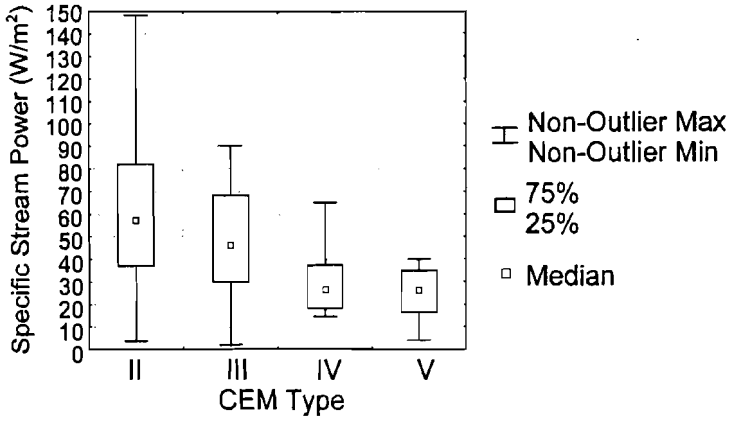
Gessler (1993) developed a sensitivity analysis of factors affecting the SAM stable channel computation. For a constant discharge and sediment concentration, he found that, in order of decreasing importance, the d_{50}, the bank angle, and Manning's roughness coefficient (n) are the most important factors in determining stable channel dimensions.



(a) Energy Slope



(b) Concentration



(c) Specific Stream Power

Figure 3.16: Box and Whisker Plots for All DEC Reaches

For preliminary design and layout of features in a watershed, an alternative approach based on the SAM model that accounts for additional parameters is suggested. The following relationships are a multi-linear power function of the stable values of slope based on Copeland's method in SAM. For channels in lower regime with sediment concentrations of 1,000 ppm or less ($R^2 = 0.999$):

$$S = 0.000112\ Q^{-0.261} * d_{50}^{0.503} * B^{0.203} * Q_s^{0.631} \tag{3.5}$$

where: S = energy slope (ft/ft);

Q = discharge (cfs);

d_{50} = sand diameter (mm);

B = bank angle expressed in horizontal component of the ratio, i.e., 1 on 3;

Q_s = sediment transport capacity (ppm); and

Manning's n = 0.035.

For channels in upper regime with sediment concentrations greater than or equal to 1,500 ppm ($R^2 = 0.916$):

$$S = 0.00241\ Q^{-0.301} * d_{50}^{0.711} * B^{0.206} * Q_s^{0.249} \tag{3.6}$$

Caution should be used in applying Eq. (3.6) since an erroneous assumption of upper regime flow could result in underestimates of channel roughness and stable channel slope.

Although the equations presented above are the result of over 500 SAM stable channel designs, caution must be used in application to avoid unwarranted extrapolation and erroneous estimates of channel roughness based on Brownlie's transitional and upper regime thresholds. The range of data used in development of Eqs. (3.5) and (3.6) is:

Q = 100 – 10,000 cfs;

d_{50} = 0.2 – 0.5 mm uniform sand;

B = 1:1, 2:1, and 3:1;

Q_s = 100 – 3,000 ppm; and

Manning's n = 0.035.

In addition to these methods, regime equations, and permissible velocity can also be used if the channel geometry and velocity data are available. One method for design would be to combine a regime relationship or permissible velocity with the results of HEC-RAS.

HEC-6 has been suggested for use in siting and designing of grade control structures. The following is a brief history of the program and of the capabilities and limitations of applying the program to design and optimization of combinations of grade control structures. William A. Thomas at the Little Rock District, USACE initiated HEC-6 development. HEC-6 evolved into Version 2.7 in 1976 during Mr. Thomas' employment at the Hydrologic Engineering Center (HEC). After leaving HEC, Mr. Thomas and his staff at WES developed the program into the Network Version of HEC-6 (sometimes referred to as TABS-1). In addition to network capability, several additional transport functions, a more complete computation of cohesive sediment suspension, and modification of the moveable bed width computation were added (USACE, 1993).

In 1986, HEC released Version 3.2 as the Library Version until replaced by Version 4.0 in 1991. Version 4.0 included the WES Network Version, code upgraded to Fortran 77 Standard, miscellaneous changes to the program output and minor error corrections. Because of these changes, Version 4.0 may produce different results than earlier versions. HEC released Version 4.1 in 1993. Version 4.1 includes sediment transport for grain sizes up to 2,048 mm, and some modification of data input capability. Version 4.1 of HEC-6 is currently available from the USACE HEC website (USACE, 2002).

In August 2000, the user's manual for HEC-6T, Version 5.13, was published by Thomas (2000), for the purpose of supplementing the 1993 HEC manual. A supplemental version from Thomas of HEC-6T was used for the computation of the Yalobusha Basin network, which included modification for the large tributary network.

Regardless of the HEC-6 version used, basic similarities and the purpose of the model remain constant. HEC-6 numerically simulates and predicts changes in river profiles resulting from scour and deposition over a period of time. Thomas (1995) listed characteristics of the program, as follows:

- one-dimensional;
- moveable boundary;
- steady-state open channel flow;
- continuous sequence of flows segmented into a series of steady flow events;
- step-backwater computation;
- sediment transport rates computed at each cross section;

- volumetric accounting of sediment in each reach;
- amount of scour or deposition computed for each reach and cross-section geometry is adjusted;
- sediment calculations are performed by grain size fraction; and
- allows for simulation of hydraulic sorting and armoring.

He also listed limitations to the program:

- one-dimensional;
- sequence of steady-state conditions;
- meandering not considered:
- formation of network computations is constrained:
  - sediment transport in distributaries is not possible,
  - flow around islands cannot be directly accommodated,
  - only one local inflow point can occur between any two cross sections;
- split flows are not provided; and
- supercritical flow is approximated by normal depth.

The 1993 manual (USACE, 1993) provides a discussion of the theoretical basis for movable boundary calculations. Both manuals provide a wealth of information; including warnings and limitations. Two of these that pertain to modeling for design of grade controls are given below:

> *The bed change calculation on the HEC-6T program was developed for large rivers in which bed changes were gradual and hydrographs rose and fell slowly with respect to time. In the small channels where the model is being applied today, the bed often fluctuates wildly during the passing of a hydrograph. Sometimes sediment will deposit during the peak flows and then erode during subsequent low flows. Since there are no checks on slope stability, the bed can become very irregular...* (p. 8-1, Thomas, 2000)

Irregular cross-section shapes have been observed to cause severe problems in simulating the DEC streams.

> *This program was designed for non-cohesive sediment transport. Some very limited cohesive theory was added for special purposes as it might relate to non-cohesive transport. This code was never intended to model cohesive*

sediment transport exclusively. However it has been used on some successful applications involving cohesive sediments by carefully posing the questions and confirming the model to prototype data. (p. F-4, Thomas, 2000)

The following paragraphs review a 1995 HEC-6 model analysis of the grade control structure by Thomas (1995). The purpose of Thomas' (1995) investigation was to develop a procedure for spacing grade control structures, and to include computed sediment continuity in that design process. The effort did not optimize the spacing of grade control structures, and only three conditions were analyzed: 1) the existing condition with DEC structures as constructed in 1995 using the 1986 to 1992 USGS gauging station data; 2) the without DEC condition using the existing condition morphology without structures and the same 1986 to 1992 hydrology as in condition 1; and 3) the future condition using the existing condition morphology (condition 1) and repeating the 1986 – 1993 USGS gauge hydrology four times to simulate a 30-year period of record. The existing condition model was run to confirm model performance. Results of the existing condition model compared well with field data points.

While no optimum design capability was developed, Thomas (1995) demonstrated a systematic procedure for analyzing the effects of grade control structures. Thomas (1995) recommended using SAM (Thomas *et al.*, 1994) or empirical methods, such as slope-area relationship (Harvey and Watson, 1986) for preliminary spacing of structures. Final design calculations for performance of structures were recommended for evaluation with HEC-6.

While each of the design tools cover a wide range of application and capability for analysis, no single model is presently available for system-wide design of equilibrium, and that can be applied with relative ease. It is critical that a suitable procedure be developed.

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