Highways in the River Environment

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Welcome to HIRE - Highways in the River Environment

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Introduction

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The purpose of this chapter is to lay the groundwork for application of the concepts of open-channel flow, fluvial geomorphology, and river mechanics to the design, maintenance, and related environmental problems associated with highway crossings and encroachments.

Basic definitions of terms and notations adopted for use herein have been presented in the preceding section for easy use and rapid reference. Additionally, these important terms and variables are defined and explained as they are encountered.

1.1 Classification of River Crossings and Encroachments

There is a wide variety of types of rivers, river crossings and encroachments. Encroachment is any occupancy of the river and floodplain for highway use. The objective herein is to consider the fluvial, hydraulic, geomorphic and environmental aspects of highway encroachments, including bridge and culvert locations, alignments, longitudinal encroachments, stabilization works and road approaches. Encroachments usually present no problems during normal stages but require special protection against floods. Flood protection requirements vary from site to site.

Some bridges and culverts must accommodate the passage of livestock and farm equipment underneath during periods of low flow. Other bridges require low embankments for aesthetic appeal, especially in populated areas. Still other bridges require short spans with long approaches and numerous piers for economic reasons. All of these factors, and many more, contribute to the difficulty in generalizing the design for all highway encroachments.

A classification of encroachments based on prominent features is helpful. Classifying the regions requiring protection, the possible types of protection, the possible flow conditions, the possible channel shapes, and the various geometric conditions aids the engineer in selecting the design criteria for the conditions he has encountered.

1.1.1 Types of Encroachment

In the vicinity of rivers, highways generally must impose a degree of encroachment. In some instances, particularly in mountainous regions or in river gorges and canyons, river crossings can be accomplished with absolutely no encroachment on the river. The bridge and its approaches are located far above and beyond any possible flood stage. More commonly, the economics of crossings require substantial encroachment on the river and its floodplain, the cost of a single span over the entire floodplain being prohibitive. The encroachment can be in the form of earth fill embankments over the floodplain or into the main channel itself, reducing the required bridge length; or in the form of piers and abutments or culverts in the main channel of the river.

There are also longitudinal encroachments not connected with river crossings. Floodplains often appear to provide an attractive low cost alternative for highway location, even when the extra cost of flood protection is included. As a consequence, highways, including interchanges, often encroach on a floodplain over long distances. In some regions, river valleys provide the only feasible route for highways. This is true even in areas where a floodplain does not exist. In many locations the highway must encroach on the main channel itself and the channel is partly filled to allow room for the roadway. In some instances this encroachment becomes severe, particularly as older highways are upgraded and widened. There is also often the need to straighten a stretch of the river, eliminating meanders, to accommodate the highway.

1.1.2 Geometry of Bridge Crossings

The bridge crossing is the most common type of river encroachment. The geometric properties of bridge crossings illustrated in Figure 1.1.1 are commonly used depending on the conditions at the site. The approaches may be skewed or normal (perpendicular) to the direction of flow, or one approach may be longer than the other, producing an eccentric crossing. Abutments used for the overbank-flow case may be set back from the low-flow channel banks to provide room to pass the flood flow or simply to allow passage of livestock and machinery, or the abutments may extend up to the banks or even protrude over the banks, constricting the low-flow channel. Piers, dual bridges for multi-lane freeways, channel bed conditions, spur dikes and guide banks add to the list of geometric classifications.

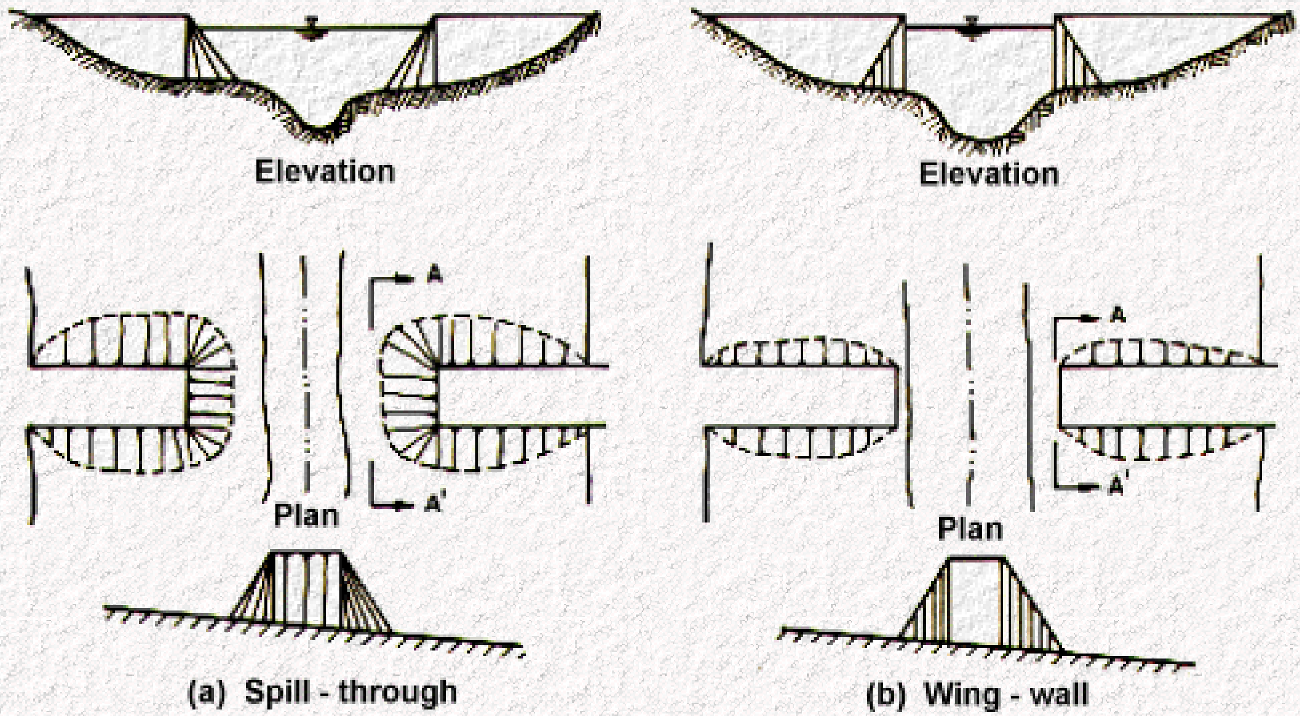

Figure 1.1.1. Geometric Properties of Bridge Crossings

The design procedures have been derived from laboratory and field observations of bridge crossings. The design procedures include allowances made for the effects of skewness, eccentricity, scour, abutment setback, channel shape, submergence of the superstructure, debris, spur dikes, wind waves, ice, piers, abutment types, and flow conditions. These design procedures take advantage of the large volume of work that has been done by many people in describing the hydraulics and scour characteristics of bridge crossings.

1.2 Dynamics of Natural Rivers and Their Tributaries

Frequently, environmentalists, river engineers, and those involved in transportation, navigation, and flood control mistakenly consider a river to be static; that is, unchanging in shape, dimensions, and pattern. However, an alluvial river generally is continually changing its position and shape as a consequence of hydraulic forces acting on its bed and banks. These changes may be slow or rapid and may result from natural environmental changes or from changes by man's activities. When an engineer modifies a river channel locally, this local change frequently causes modification of channel characteristics both up and down the stream. The response of a river to man-induced changes often occurs in spite of attempts by engineers to keep the anticipated response under control.

The points that must be stressed are that a river through time is dynamic, that man-induced change frequently sets in motion a response that can be propagated for long distances, and that in spite of their complexity all rivers are governed by the same basic forces. The highway engineer must understand and work with these natural forces. It is absolutely necessary for the design engineer to have at hand competent knowledge about: (1) geological factors, including soil conditions; (2) hydrologic factors, including possible changes in flows, runoff, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream, including the probable geometric alterations that will be activated by the changes his project and future projects will impose on the channel; and (4) hydraulic characteristics such as depths, slopes, and velocity of streams and what changes may be expected in these characteristics in space and time.

1.2.1 Historical Evidence of the Natural Instability of Fluvial Systems

In order to emphasize the inherent dynamic qualities of river channels, evidence is cited below to demonstrate that most alluvial rivers are not static in their natural state. Indeed, scientists concerned with the history of landforms (geomorphologists), vegetation (botanists), and the past activities of man (archaeologists), rarely consider the landscape as unchanging. Rivers, glaciers, sand dunes, and seacoasts are highly susceptible to change with time. Over a relatively short period of time, perhaps in some cases as long as man's lifetime, components of the landscape may be relatively stable. Nevertheless stability cannot be automatically assumed. Rivers are, in fact, the most actively changing of all geomorphic forms.

Evidence from several sources demonstrate that river channels are continually undergoing changes of position, shape, dimensions, and pattern. In Figure 1.2.1 a section of the Mississippi River as it was in 1884 is compared with the same section as observed in 1968. In the lower 6 miles of river, the surface area has been reduced approximately 50 percent during this 84-year period. Some of this change has been natural and some has been the consequence of river development work.

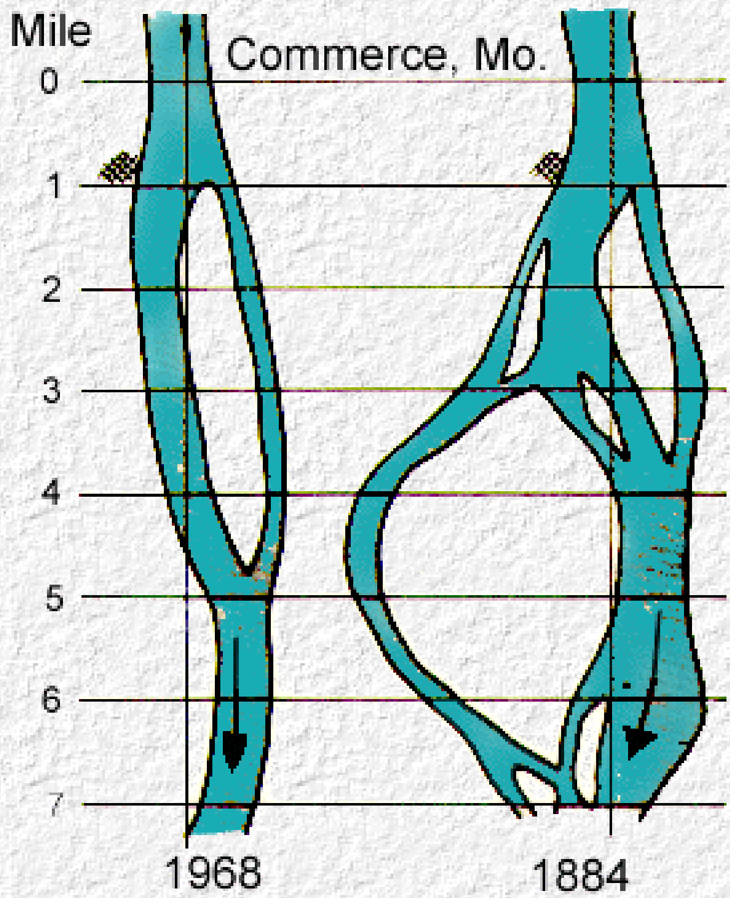

Figure 1.2.1. Comparison of the 1884 and 1968 Mississippi River Channel near Commerce, Missouri

In alluvial river systems, it is the rule rather than the exception that banks will erode, sediments will be deposited and floodplains, islands, and side channels will undergo modification with time. Changes may be very slow or dramatically rapid. Fisk's (1944) report on the Mississippi River and his maps showing river position through time are sufficient to convince everyone of the innate instability of the Mississippi River. The Mississippi is our largest and most impressive river and because of its dimensions it has sometimes been considered unique. This is, of course, not so. Hydraulic and geomorphic laws apply at all scales of comparable landform evolution. The Mississippi may be thought of as a prototype of many rivers or as a much larger than prototype model of many sandbed rivers.

Rivers change position and morphology (dimensions, shape, pattern) as a result of changes of hydrology. Hydrology can change as a result of climatic changes over long periods of time, or as a result of natural stochastic climatic fluctuations (droughts, floods), or by man's modification of the hydrologic regime. For example, the major climatic changes of recent geological time (the last few million years of earth history) have triggered dramatic changes in runoff and sediment loads with corresponding channel alteration. Equally significant during this time were fluctuations of sea level. During the last continental glaciation, sea level was on the order of 400 feet lower than at present, and this reduction of base level caused major incisions of river valleys near the coasts.

In recent geologic time, major river changes of different types occurred. These types are deep incision and deposition as sea level fluctuated, changes of channel geometry as a result of climatic and hydrologic changes, and obliteration or displacement of existing channels by continental glaciation. Climatic change, sea level change, and glaciation are interesting from an academic point of view but are not considered as cause of modern river instability. The movement of the earth's crust is one geologic agent causing modern river instability. The earth's surface in many parts of the world is undergoing continuous measurable change by upwarping, subsidence or lateral displacement. As a result, the study of these ongoing changes (called neotectonics) has become a field of major interest for many geologists and geophysicists. Such gradual surface changes can affect stream channels dramatically. For example, Wallace (1967) has shown that many small streams are clearly offset laterally along the San Andreas fault in California. Progressive lateral movement of this fault on the order of an inch per year has been measured. The rates of movement of faults are highly variable, but an average rate of mountain building has been estimated by Schumm (1963) to be on the order of 25 feet per 1000 years. Seemingly insignificant in human terms, this rate is actually 0.3 inches per year or 3 inches per decade. For many river systems, a change of slope of 3 inches would be significant. (The slope of the energy gradient on the Lower Mississippi River is about 3 to 6 inches per mile).

Of course, the geologist is not surprised to see drainage patterns that have been disrupted by uplift or some complex warping of the earth's surface. In fact, complete reversals of drainage lines have been documented. In addition, convexities in the longitudinal profile of both rivers and river terraces (these profiles are concave under normal development) have been detected and attributed to upwarping. Further, the progressive shifting of a river toward one side of its valley has resulted from lateral tilting. Major shifts in position of the Brahmaputra River toward the west are attributed by Coleman (1969) to tectonic movements. Hence, neotectonics should not be ignored as a possible cause of local river instability.

Long-term climatic fluctuations have caused major changes of river morphology. Floodplains have been destroyed and reconstructed many times over. The history of semi-arid and arid valleys of the western United States is one of alternating periods of channel incision and arroyo formation followed by deposition and valley stability which have been attributed to climatic fluctuations.

It is clear that rivers can display a remarkable propensity for change of position and morphology in time periods of a century. Hence rivers from the geomorphic point of view are unquestionably dynamic, but does this apply to modern rivers? It is probable that during a period of several years, neither neotectonics nor a progressive climate change will have a detectable influence on river character and behavior. What then causes a river to appear relatively unstable from the point of view of the highway engineer or the environmentalist? It is the slow but implacable shift of a river channel through erosion and deposition at bends, the shift of a channel to form chutes and islands, and the cutoff of a bend to form oxbow lakes. Lateral migration rates are highly variable; that is, a river may maintain a stable position for long periods and then experience rapid movement. Much therefore depends on flood events, bank stability, permanence of vegetation on banks and the floodplain and watershed land use. A compilation of data by Wolman and Leopold shows that rates of lateral migration for the Kosi River of India range up to approximately 2500 feet per year. Rates of lateral migration for two major rivers in the United States are as follows: Colorado River near Needles, California, 10 to 150 feet per year; Mississippi River near Rosedale, Mississippi, 158 to 630 feet per year.

Archaeologists have also provided clear evidence of channel changes that are completely natural and to be expected. For example, the number of archaeologic sites of the floodplains decreases significantly with age because the earliest sites are destroyed as floodplains are modified by river migration. Lathrop (1968), working on the Rio Ycayali in the Amazon headwaters of Peru, estimates that on the average a meander loop on this river begins to form and cuts off in 5000 years. These loops have an amplitude of 2 to 6 miles and an average rate of meander growth of approximately 40 feet per year.

A study by Schmuddle (1963) shows that about one-third of the floodplain of the Missouri River over the 170-mile reach between Glasgow and St. Charles, Missouri, was reworked by the river between 1879 and 1930. On the Lower Mississippi River, bend migration was on the order of 2 feet per year, whereas in the central and upper parts of the river, below Cairo, it was at times 1000 feet per year (Kolb, 1963). On the other hand, a meander loop pattern of the lower Ohio River has altered very little during the past thousand years. (Alexander and Nunnally, 1972).

Although the dynamic behavior of perennial streams is impressive, the modification of rivers in arid and semi-arid regions and especially of ephemeral (flowing occasionally) stream channels is startling. A study of floodplain vegetation and the distribution of trees in different age groups led Everitt (1968) to the conclusion that about half of the Little Missouri River floodplain in western North Dakota was reworked in 69 years.

Historical and field studies by Smith (1940) show that floodplain destruction occurred during major floods on rivers of the Great Plains. As exceptional example of this is the Cimarron River of Southwestern Kansas, which was 50 feet wide during the latter part of the 19th and first part of the 20th centuries (Schumm and Lichty, 1957). Following a series of major floods during the 1930's it widened to 1200 feet, and the channel occupied essentially the entire valley floor. During the decade of the 1940's a new floodplain was constructed, and the river width was reduced to about 500 feet in 1960. Equally dramatic changes of channel dimensions have occurred along the North and South Platte Rivers in Nebraska and Colorado as a result of man's control of flood peaks by reservoir construction. Natural changes of this magnitude due to changes in flood peaks are perhaps exceptional, but emphasize the mobility of rivers and their ability to adapt to changing conditions.

Another somewhat different type of channel modification which testifies to the rapidity of fluvial processes is described by Shull (1922, 1944). During a major flood in 1913, a barge became stranded in a chute of the Mississippi River near Columbus, Kentucky. The barge induced deposition in the chute and an island formed. In 1919, the island was sufficiently large to be homesteaded, and a few acres were cleared for agricultural purposes. By 1933, the side channel separating the island from the mainland had filled to the extent that the island became part of Missouri. The island formed in a location protected from the erosive effects of floods but susceptible to deposition of sediment during floods. For these reasons the channel filling was rapid and progressive. It cannot be concluded that islands will always form and side channels fill at such rapid rates, but island formation and side-channel filling appear to be the normal course of events in any river transporting moderate or high sediment loads regardless of the river size.

In summary, archaeological, botanical, geological, and geomorphic evidence supports the conclusion that most rivers are subject to constant change as a normal part of their morphologic evolution. Therefore, stable or static channels are the exception in nature.

1.2.2 Introduction to River Hydraulics and River Response

In the previous section it was established that rivers are dynamic and respond to changing environmental conditions. The direction and extent of the change depends on the forces acting on the system. The mechanics of flow in rivers is a complex subject that requires special study which is unfortunately not included in basic courses of fluid mechanics. The major complicating factors in river mechanics are: (a) the large number of interrelated variables that can simultaneously respond to natural or imposed changes in a river system and (b) the continual evolution of river channel patterns, channel geometry, bars and forms of bed roughness with changing water and sediment discharge. In order to understand the responses of a river to the actions of man and nature, a few simple hydraulic and geomorphic concepts are presented here.

River forms are broadly classified as straight, meandering, braided or some combination of these classifications, but any changes that are imposed on a river may change its form. The dependence of river sinuosity on the slope which may be imposed independent of the other river characteristics is illustrated schematically in Figure 1.2.2. By changing the slope, it is possible to change the river from a meandering one that is relatively tranquil and easy to control to a braided one that varies rapidly with time, has high velocities, is subdivided by sandbars and carries relatively large quantities of sediment. Such a change could be caused by a natural or artificial cutoff. Conversely, it is possible that a slight decrease in slope could change an unstable braided river into a meandering one.

The significantly different channel dimensions, shapes, and patterns associated with different quantities of discharge and amounts of sediment load indicate that as these independent variables change, major adjustments of channel morphology can be anticipated. Further, if changes in sinuosity and meander wavelength as well as in width and depth are required to compensate for a hydrologic change, then a long period of channel instability can be envisioned with considerable bank erosion and lateral shifting of the channel before stability is restored. The reaction of a channel to changes in discharge and sediment load may result in channel dimension changes contrary to those indicated by many regime equations. For example, it is conceivable that a decrease in discharge together with an increase in sediment load could actuate a decrease in depth and an increase in width.

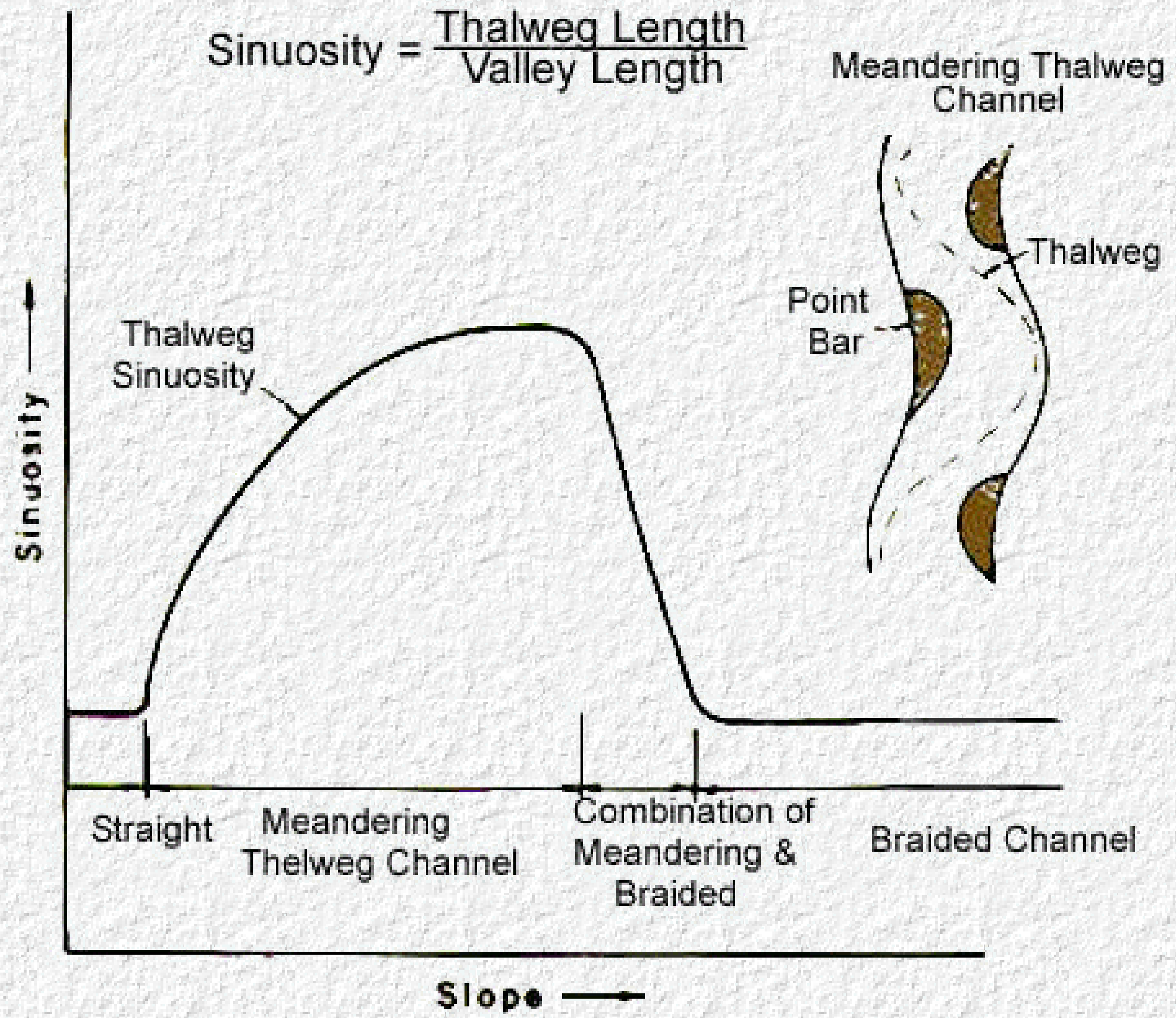

Figure 1.2.2. Sinuosity vs. Slope with Constant Discharge

Changes in sediment and water discharge at a particular point or reach in a stream may have an effect ranging from some distance upstream to a point downstream where the hydraulic and geometric conditions will have absorbed the change. Thus, it is well to consider a channel reach as part of a complete drainage system. Artificial controls that could benefit the reach may, in fact, cause problems in the system as a whole. For example, flood control structures can cause downstream flood damage to be greater at reduced flows if the average hydrologic regime is changed so that the channel dimensions are actually reduced. Also, where major tributaries exert a significant influence on the main channel by introducing large quantities of sediment, upstream control on the main channel may allow the tributary to intermittently dominate the system with deleterious results. If discharges in the main channel are reduced, sediments from the tributary that previously were eroded will no longer be carried away and serious aggradation with accompanying flood problems may arise.

An insight into the direction of change, the magnitude of change, and the time involved to reach a new equilibrium can be gained by studying the river in a natural condition; having knowledge of the sediment and water discharge; being able to predict the effects and magnitude of man's future activities; and applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers.

The current interest in ecology and the environment have made people aware of the many problems that mankind can cause. Previous to the present interest in environmental impact, very few people interested in rivers ever considered the long-term changes that were possible. It is imperative that anyone working with rivers, either with localized areas or entire systems, have an understanding of the many factors involved, and of the potential for change existing in the river system.

Two methods of predicting response are employed. They are the physical and the mathematical models. Engineers have long used small scale hydraulic models to assist them in anticipating the effect of altering conditions in a reach of a river. With proper awareness of the large scale effects that can exist, the results of hydraulic model testing can be extremely useful for this purpose. A more recent and alternative method of predicting short-term and long-term changes in rivers involves the use of mathematical models. To study a transient phenomenon in natural alluvial channels, the equations of motion and continuity for sediment laden water and the continuity equation for sediment can be used as discussed in Chapter 3 and Chapter 4.

1.3 Effects of Highway Construction on River Systems

Highway construction can have significant general and local effects on the geomorphology and hydraulics of river systems. Hence, it is necessary to consider induced short-term and long-term responses of the river and its tributaries, the impact on environmental factors, the aesthetics of the river environment and short-term and long-term effects of erosion and sedimentation on the surrounding landscape and the river. The biological response of the river system should also be evaluated and considered.

1.3.1 Immediate Responses

Let us consider a few of the numerous and immediate responses of rivers to the construction of bridges, training and channel stabilization works and approaches.

In the preceding paragraphs we indicated that local changes made in the geometry or the hydraulic properties of the river may be of such a magnitude as to have an immediate impact upon the entire river system. More specifically, contractions due to the construction of encroachments usually cause contraction and local scour, and the sediments removed from this location are usually dropped in the immediate reach downstream. In the event that the contraction is extended further downstream, the river may be capable of carrying the increased sediment load an additional distance but only until a reduction in gradient and a reduction in transport capability is encountered. The increased velocities caused by encroachments may also affect the general lateral stability of the river downstream.

In addition, the development of crossings and the contraction of river sections may have a significant effect on the water level in the vicinity and upstream of the bridge. Such changes in water level upstream of the bridge are called backwater effects. The highway engineer must be in a position to accurately assess the effects of the construction of crossings upon the water surface profile.

To offset increased velocities and to reduce bank instabilities and related problems, one ends up, in many instances, with stabilizing or channelizing the river to some degree. When it is necessary to do this, every effort should be made to do the channelization in a manner which does not degrade the river environment, which includes the river's aesthetic value.

As a consequence of construction, many areas become highly susceptible to erosion. The transported sediment is carried from the construction site by surface flow into the minor rills, which combine within a short distance to form larger channels leading to the river. The water flowing from the construction site is usually a consequence of rain. The surface runoff and the accompanying erosion can significantly increase the sediment yield to the river channel unless careful control is exercised. The large sediment particles transported to the main channel may reside in the vicinity of the construction site for a long period of time or may be slowly moved away. On the other hand, the fine sediments are easily transported and generally pollute the whole cross section of the river. The fine sediments are transported downstream to the nearest reservoir or to the sea. As will be discussed later, the sudden injection of the larger sediments into the channel may cause local aggradation, thereby steepening the channel, increasing the flow velocities and possibly causing instability in the river at that site.

The suspended fine sediments can have very significant effects on the biomass of the stream. Certain species of fish can only tolerate large quantities of suspended sediment for relatively short periods of time. This is particularly true of the eggs and fry. This type of biological response to development normally falls outside of the competence of the engineer. Yet his work may be responsible for the discharge of these sediments into the system. If he is unable to cope with the problem, the engineer should utilize adequate technical assistance from experts in fisheries, biology, and other related areas to overcome the consequences of sediment pollution in a river. Only with such knowledge can he develop the necessary arguments to sell his case that erosion control measures must be exercised to avoid significant deterioration of the stream environment not only in the immediate vicinity of the bridge but in many instances for great distances downstream.

Another possible immediate response of the river system to construction is the loss of the recreational use of the river. In many streams, there may be an immediate drop in the quality of the fishing due to the increase of sediment load, or other changed hydraulic characteristics within the channel. Some natural rivers consist of a series of pools and riffles. Both form an important part of the environment from the viewpoint of fisheries. The introduction of larger quantities of sediment into the channel and changes made in the geometry of the channel may result in the loss of these pools and riffles. Along the same lines, construction work within the river may cause a loss of food essential to fish life and often it is difficult to get the food chain reestablished in the system.

<u>Construction and operation of highways in water-supply watersheds present very real problems and require special precautionary designs to protect the water supplies from highway residue.</u> These residues may be largely sedimentary and may increase the turbidity of the water. There have been instances, however, where other unwelcome materials such as asphalt distillates and deicing salts which have been traced to highway operations.

The preceding discussion is related to only a few immediate responses to construction along a river. However, they are responses that illustrate their importance to design and the environment.

1.3.2 Delayed Response of Rivers to Development

In addition to the example of possible immediate responses discussed above, there are important delayed responses of rivers to highway development. As part of this introductory chapter, consideration is given to some of the more obvious effects that can be induced on a river system over a long time period by highway construction.

Sometimes it is necessary to employ training works in connection with highway encroachments to favorably align the flow with bridge or culvert openings. When such training works are used, they generally straighten the channel, shorten the flow line, and increase the local velocity within the channel. Any such changes made in the system that cause an increase in the gradient may cause an increase in local velocities. The increase in velocity increases local and contraction scour with subsequent deposition downstream where the channel takes on its normal characteristics. If significant lengths of the river are trained and straightened, there can be a noticeable decrease in the elevation of the water surface profile for a given discharge in the main channel. Tributaries emptying into the main channel in such reaches are significantly affected. Having a lower water level in the main channel for a given discharge means that the tributary streams entering in that vicinity are subjected to a steeper gradient and higher velocities which cause degradation in the tributary streams. In extreme cases, degradation can be induced of such magnitude as to cause failure of structures such as bridges, culverts or other encroachments on the tributary systems. In general, any increase in transported materials from the tributaries to the main channel causes a reduction in the quality of the environment within the river. More specifically, as degradation occurs in the tributaries, bank instabilities are induced and the sediment loads are greatly increased. Increased sediment loads usually result in a deterioration of the environment.

1.4 The Effects of River Development on Highway Encroachments

Some of the possible immediate and delayed responses of rivers and river systems to the construction of bridges, approaches, culverts, channel stabilization, longitudinal encroachments, and the utilization of training works have been mentioned. It is necessary also to consider the effects of highway encroachments on river development works. These works may include, for example, water diversions to and from the river system, construction of reservoirs, flood control works, cutoffs, levees, navigation works, and the mining of sand and gravel. It is essential to consider the possible or probable long-term plans of all agencies and groups as they pertain to a river when designing crossings or when dealing with the river in any way. Let us consider a few typical responses of a crossing to different types of water resources development.

Cutoffs may develop naturally in the river system or cutoffs can be constructed by man. The general consequence of cutoffs is to shorten the flow path and steepen the gradient of the channel. The local steepening can significantly increase the velocities and sediment transport. Also, this action can induce significant instability such as bank erosion and degradation in the reach. The material scoured in the reach affected by the cutoff is probably carried only to an adjacent downstream reach where the gradient is flatter. In this region of slower velocities the sediment drops out rapidly. Deposition can have significant detrimental effect on the downstream reach of river, increasing the flood stage in the river itself and increasing the base level for the tributary stream, thereby causing aggradation in the tributaries.

Consider a classic example of a cutoff that was constructed on a large bend in one of the tributaries to the Mississippi. Along this bend, small towns had developed and small tributary streams entered the main channel within the bendway. It was decided to develop a cutoff across the gooseneck to shorten the flow line of the river, reduce the flood stage and generally improve poor hydraulic conditions in that location. Several interesting results developed.

In the vicinity of the cutoff, the bankline eroded and degradation was initiated. Within the bendway, the small tributaries continued to discharge their water and sediment. Because of the flat gradient in the bend, this channel section could not convey the sediment from the small systems through it and aggradation was initiated. Within a short period of time sufficient aggradation had occurred so as to jeopardize water intakes, sewage outfalls and so forth. As a consequence of the adverse action in the vicinity of the cutoff and within the bendway itself, it was finally decided that it would be more beneficial to restore the river to its natural form through the bend. This action was taken and the serious problems were alleviated.

In such a haphazard program of river development, the highway engineer would be hard pressed to maintain and plan for his highway system along and over this reach of river.

Another common case occurs with the development of reservoirs for storage and flood control. These reservoirs serve as traps for the sediment normally flowing through the river system. With sediment trapped in the reservoir, essentially clear water is released downstream of the dam site. This clear water has the capacity to transport more sediment than is immediately available. Consequently the channel begins to supply this deficit with resulting degradation of the bed or banks. This degradation may significantly affect the safety of bridges in the immediate vicinity. Again, the degraded or widened main channel causes steeper gradients on tributary streams in the vicinity of the main channel. The result is degradation in the tributary streams. It is entirely possible, however that the additional sediments supplied by the tributary streams would ultimately offset the degradation in the main channel. Thus, it must be recognized that downstream of storage structures the channel may either aggrade or degrade and the tributaries will be affected in either case.

There are important responses induced upstream of reservoirs as well as downstream. When the stream flowing into a reservoir encounters the ponded water, its sediment load is deposited forming a delta. This deposition in the reservoir flattens the gradient of the channel upstream. The flattening of the upstream channel induces aggradation causing the bed of the river to rise, threatening highway installations and other facilities. For example, Elephant Butte Reservoir, built on the Rio Grande, has caused the Rio Grande to aggrade many miles upstream of the reservoir site. This change in bed level can have very significant effects upon bridges, other hydraulic structures and all types of training and stabilization works. Ultimately the river may be subjected to a flow of magnitude sufficient to overflow existing banks, causing the water to seek an entirely new channel. With the abandonment of the existing channel there would be a variety of bridges and hydraulic structures that would also be abandoned at great expense to the public.

The clear-water diversion into South Boulder Creek in Colorado is another example of river development that affects bridge crossings and encroachments as well as the environment in general. Originally the North Fork of South Boulder Creek was a small but beautiful scenic mountain stream. The banks were nicely vegetated; there was a beautiful sequence of riffles and pools which had all the attributes of a good fishing habitat. Years ago, water was diverted from the Western Slope of the Rockies through a tunnel to the North Fork of South Boulder Creek. The normal stage in that channel was increased by a factor of 4 to 5. The extra water caused significant bank erosion and channel degradation. In fact, the additional flow gutted the river valley, changing the channel to a straight raging torrent capable of carrying large quantities of sediment. Degradation in the system had reached as much as 15 to 20 feet before measures were taken to stabilize the creek.

Stabilization was achieved by flattening the gradient by constructing numerous drop structures and by reforming the banks with riprap. The system has stabilized but it is a different system. The channel is straight, much of the vegetation has been washed away, and the natural sequence of riffles and pools has been destroyed. The valley may never again have the natural form and beauty it once possessed. It is necessary for us to bear in mind that diversions to or from the natural river system can greatly alter its geometry, beauty and utility. The river may undergo a complete change, giving rise to a multitude of problems in connection with the design and maintenance of hydraulic structures, encroachments and bridge crossings along the affected reach.

In the preceding paragraphs possible immediate and long-term responses of river systems to various types of river development have been described. Nothing has been indicated about how to determine the magnitude of these changes. This important aspect of response of rivers to development will be treated more objectively in later chapters.

1.5 Technical Aspects

Effects of river development, flood control measures and channel structures built during the last century have proven the need for taking into account delayed and far-reaching effects of any alteration man makes in a natural alluvial river system.

Because of the complexity of the processes occurring in natural flows and the erosion and deposition of material, an analytical approach to the problem is very difficult and time consuming. Most of our river process relations have been derived empirically. Nevertheless, if a greater understanding of the principles governing the processes of river formation is to be gained, the empirically derived relations must be put in the proper context by employing the analytical approach. In that way the distinct limitations of the empirical relations can be removed.

Mankind's attempts at controlling large rivers has often led to the situation described by J. Hoover Mackin (1937) when he wrote:

> "the engineer who alters natural equilibrium relations by diversion or damming or channel improvement measures will often find that he has the bull by the tail and is unable to let go . . . as he continues to correct or suppress undesirable phases of the chain reaction of the stream to the initial 'stress' he will necessarily place increasing emphasis on study of the genetic aspects of the equilibrium in order that he may work with rivers, rather than merely on them."

Through such experiences, man realizes that, to prevent or reduce the detrimental effects of any modification of the natural processes and state of equilibrium on a river, he must gain an understanding of the physical laws governing them, and become knowledgeable of the far-reaching effects of any attempt to control or modify a river's course.

1.5.1 Variables Affecting River Behavior

Variables affecting alluvial river channels are numerous and interrelated. Their nature is such that, unlike rigid boundary hydraulic problems, it is not possible to isolate and study the role of any individual variable.

Major factors affecting alluvial stream channel forms are: (1) stream discharge, temperature, viscosity; (2) sediment load; (3) longitudinal slope; (4) bank and bed resistance to flow; (5) vegetation; (6) geology, including types of sediments; and (7) the works of man.

The fluvial processes involved are very complicated and the variables of importance are difficult to isolate. Many laboratory and field studies have been carried out in an attempt to relate these and other variables to the present time. The problem has been more amenable to an empirical solution than an analytical one.

In an analysis of flow in alluvial rivers, the flow field is complicated by the constantly changing discharge. Significant variables are, therefore, quite difficult to relate mathematically. It is desirable to list measurable or computable variables which effectively describe the processes occurring and then to reduce the list by making simplifying assumptions and examining relative magnitudes of variables, striving toward an acceptable balance between accuracy and limitations of obtaining data. When this is done, the basic equations of fluid motion may be simplified (on the basis of valid assumptions) to describe the physical model.

It is the role of the succeeding chapters to present these variables, define them, show how they interrelate, quantify their interrelations where feasible, and show how they can be applied to achieve the successful design of river crossings and encroachments.

1.5.2 Basic Knowledge Required

In order for the engineer to cope successfully with river engineering problems, it is necessary that he has an adequate background in engineering with an emphasis on hydrology, hydraulics, erosion and sedimentation, river mechanics, soil mechanics, structures, economics, the environment and related subjects. In fact, as the public has demanded more comprehensive treatment of river development problems, the highway engineer should further improve his knowledge, and the application of it, by soliciting the cooperative efforts of the hydraulic engineer, hydrologist, geologist, geomorphologist, meteorologist, mathematician, statistician, computer programmer, systems engineer, soil physicist, soil chemist, biologist, water management staff and economist. Professional organizations requiring these talents should be encouraged to work cooperatively to achieve the long range research needs and goals relative to river development and application of knowledge on a national and international basis. Through an appropriate exchange of information between scientists working in these fields, opportunities to do a better job with all aspects of river development should be greatly enhanced.

1.5.3 Data Requirement

Large amounts of data pertaining to understanding the behavior of rivers have been acquired over a long period of time. Nevertheless, data collection efforts to date have been sporadic and unfocused. Agencies should take a careful look at present data requirements needed to solve practical problems along with existing data. A careful analysis of data requirements would make it possible to more efficiently utilize funds to collect data in the future. The basic type of information that is required includes: water discharge hydrograph, sediment discharge hydrograph, the characteristics of the sediments being transported by streams, the characteristics of the channels in which the water and sediment are transported, and the characteristics of watersheds and how they deliver water and sediment to the stream systems. Environmental data is also needed so that proper assessment can be made of the impact of river development upon the environment and vice versa. The problem of data requirements at river crossings is of sufficient importance that it is treated in greater detail in Chapter 6.

1.6 Future Technical Trends

When considering the future, it is essential to recognize the present state of knowledge pertaining to river hydraulics and then identify inadequacies in existing theories and encourage further research to help correct these deficits of knowledge. In order to correct such deficits there is need to take a careful look at existing data pertaining to rivers, future data requirements, research needs, training programs and methods of developing staff that can apply this knowledge to the solution of practical problems.

1.6.1 Adequacy of Current Knowledge

The basic principles of fluid mechanics involving application of continuity, momentum and energy concepts are well known and can be effectively applied to a wide variety of river problems. Considerable work has been done on the hydraulics of rigid boundary open channels and excellent results can be expected. The steady-state sediment transport of nearly uniform sizes of sediment in alluvial channels is well understood. There is good understanding of stable channel theory in non-cohesive materials of all sizes. The theory is adequate to enable us to design stable systems in the existing material or, if necessary, designs can be made for appropriate types of stabilization treatments for canals and rivers to have them behave in a stable manner. There have been extensive studies of the fall velocity of non-cohesive sediments in static fluids to provide knowledge about the interaction between the particle and fluid so essential to the development of sediment transport theories. The use of computers and the development of computer programs has greatly helped the hydraulic engineer to solve problems on highway crossings and encroachments. These programs are the FHWA WSPRO or the Corps of Engineers HEC 2 used to solve the water surface elevation and the velocity of the flow in a river. The FHWA BRISTARS program to determine flow of water and sediment through a bridge crossing. The Corps of Engineers HEC 2 program to route the water through a river system is also very valuable. This latter program is of particular importance in routing water from an upstream gaging station downstream to a bridge taking into account any increase or decrease in flow that might occur in between the station and the bridge.

Thus, available concepts and theories which can be applied to the behavior of rivers are extensive. However, in many instances only empirical relationships have been developed and these are pertinent to specific problems only. Consequently, a more basic theoretical understanding of flow in the river systems needs to be developed.

With respect to many aspects of river mechanics, it can be concluded that knowledge is available to cope with the majority of river problems. On the other hand, the number of individuals who are cognizant of existing theory and can apply it successfully to the solution of river problems is limited. Particularly,the number of individuals involved in the actual solution of applied river mechanics problems is very small. There is a specific reason for this deficit of trained personnel. Undergraduate engineering educators in the universities in the United States, and in the world for that matter, devote only a small amount of time to teaching hydrology, river mechanics, channel stabilization, fluvial geomorphology, and related problems. It is not possible to obtain adequate training in these important topics except at the graduate level, and only a limited number of universities and institutions offer the required training in these subject areas. There is great need for adequately trained people to cope with river problems.

1.6.2 Research Needs

As knowledge of river hydraulics is reviewed, it becomes quite obvious that many things are not adequately known. Research needs are particularly urgent and promise a rather quick return. Stabilization of rivers and bank stability of river systems need further consideration. Also, the study of bed forms generated by the interaction between the water and sediment in the river systems deserves further study. The types of bed forms have been identified but theories pertaining to their development are inadequate. Simple terms have been used to describe the characteristics of alluvial material of both cohesive and non-cohesive types; a comprehensive look at the characteristics of materials is warranted.

Other important research problems include the fluid mechanics of the motion of particles, secondary currents, two-dimensional velocity distributions, fall velocity of particles in turbulent flow and the application of remote sensing techniques to hydrology and river mechanics. The physical modeling of rivers followed by prototype verification, mathematical modeling of river response followed by field verification, mathematical modeling of water and sediment yield from small watersheds and studies of unsteady sediment transport are areas in which significant advances can be made.

A primary research need is the collection of field data on the flow variables and depth of scour at bridges, embankments and at river control structures. In addition, laboratory studies are needed to improve the equations for estimating total scour at piers and abutments. For example, laboratory studies to determine methods to predict scour depths and widths at piers and abutments when pressure flow occurs (pressure flow occurs when a bridge is over topped); methods to predict scour depth and width when a footing or pilecap is exposed to the flow; the effect of debris on scour depths and widths; the effect of angle of attack on a row of cylinders; the design of structures placed ahead or around piers to decrease scour; to determine the length of pier that is no longer a factor in the angle of attack, scour depth and length of pier relations; the riprap size needed to protect piers or abutments as a function of depth below the average streambed depth; the distribution of contraction scour in the cross-section when a bridge is over or downstream of a stream bend and abutment scour research with a more realistic flow distribution in the overbank area. In abutment scour research we should use discharge in the overbank area instead of approach abutment length.

Operational research on decision making, considering cost and risk criteria to determine the hydrologic and hydraulic design of highway structures and project alternatives, is another pressing research area. Insufficient data is frequently a problem of river mechanics analysis. A comprehensive study on information theory is needed to cope with such difficulties.

Finally the results of these efforts must be presented in such a form that it can be easily taught and easily put to practical use.

1.6.3 Training

It has been pointed out that engineering training is somewhat inadequate in relation to understanding the developments of rivers. There is need to consider better ways to train engineers to disseminate existing knowledge in this important area. The training of individuals could be accomplished by conducting seminars, conferences or short courses in institutions in the spirit of continuing education. There should be an effort to improve the curriculum of university education made available to engineers, particularly at the undergraduate level. At the very minimum such a curriculum should strive to introduce concepts of fluvial geomorphology, river hydraulics, erosion and sedimentation, environmental considerations and related topics.

Manuals, handbooks and reference documents should be prepared for practicing engineers in order to overcome the deficit of knowledge. Publications of material pertaining to rivers should be encouraged. This material can be and is being published to some degree in the proceedings of conferences, in journals and textbooks. Better use of informative films and video tapes could be made, a technique that would be of assistance in teaching effectively, efficiently and economically. Similarly, television and video tapes can be economically prepared and utilized in instructional situations. Television cameras are available that enable the teacher to record and take field situations directly into the classroom for class consideration.

Formal training should be supported with field trips and laboratory demonstrations. Laboratory demonstrations are an inexpensive method of quickly and effectively teaching the fundamentals of river mechanics and illustrating the behavior of structures. These demonstrations should be followed by field trips to illustrate similarities and differences between phenomena in the laboratory and in the field.

Finally, larger numbers of disciplines should be involved in the training programs. Cooperative studies should involve research personnel, practicing engineers and people from the many different disciplines with an interest in rivers.

Go to Chapter 2 (Part I)

Chapter 2 : HIRE
Open Channel Flow
Part I

Go to Chapter 2, Part II

2.1 Introduction

In this chapter the fundamentals of rigid boundary open channel flow are described. In open channel flow, the water surface is not confined; surface configuration, flow pattern and pressure distribution within the flow depend on gravity. In rigid boundary open channel flow, no deformations of the bed and banks are considered. Mobile boundary hydraulics refers to flow which can generate deformation of the boundary through scour and fill. Mobile boundary hydraulics will be discussed in later chapters. In this chapter, we restrict ourselves to a one-dimensional analysis of rigid boundary open channel flow: velocity and acceleration are large only in one direction and are so small as to be negligible in all other directions.

Open channel flow can be classified as: (1) uniform or nonuniform flow; (2) steady or unsteady flow; (3) laminar or turbulent flow; and (4) tranquil or rapid flow. In uniform flow, the depth and discharge remain constant with respect to space. Also, the velocity at a given depth is the same everywhere. In steady flow, no change occurs with respect to time at a given point. In laminar flow, the flow field can be characterized by layers of fluid, one layer not mixing with adjacent ones. Turbulent flow on the other hand is characterized by random fluid motion. Tranquil flow is distinguished from rapid flow by a dimensionless number called the Froude number, Fr. If $Fr < 1$, the flow is subcritical; if $Fr > 1$, the flow is supercritical, and if $Fr = 1$, the flow is called critical.

Open channel flow can be nonuniform, unsteady, turbulent and rapid at the same time. Because the classifying characteristics are independent, sixteen different types of flow can occur. These terms, uniform or nonuniform, steady or unsteady, laminar or turbulent, rapid or tranquil, and the two dimensionless numbers (the Froude number and Reynolds number) are more fully explained in the following sections.

2.1.1 Definitions

Velocity: The velocity of a fluid particle is the time rate of displacement of the particle from one point to another. Velocity is a vector quantity. That is, it has magnitude and direction. The mathematical representation of the fluid velocity is a function of the increment of length ds during the infinitesimal time dt; thus,

$$v = \frac{ds}{dt} \qquad (2.1.1)$$

Streamline: An imaginary line within the flow which is everywhere tangent to the velocity vector is called a streamline.

Acceleration: Acceleration is the time rate of change in magnitude or direction of the velocity vector. Mathematically, acceleration a is expressed by the total derivative of the velocity vector or

$$a = \frac{dv}{dt} \qquad (2.1.2)$$

The vector acceleration a has components both tangential and normal to the streamline, the tangential component embodying the change in magnitude of the velocity, and the normal component reflecting the change in direction

$$a_s = \frac{dv_s}{dt} = \frac{\partial v_s}{dt} + \frac{1}{2}\frac{\partial \left(v_s^2\right)}{\partial s} \qquad (2.1.3)$$

$$a_n = \frac{dv_n}{dt} = \frac{\partial v_n}{\partial t} + \frac{v^2}{r} \qquad (2.1.4)$$

The first terms in Equation 2.1.3 and Equation 2.1.4 represent the change in velocity, both magnitude and direction, with time at a given point. This is called the local acceleration. The second term in each equation is the change in velocity, both magnitude and direction, with distance. This is called convective acceleration.

Uniform flow: In uniform flow the convective acceleration terms are zero.

$$\frac{\partial v_s}{\partial s} = 0 \; and \; \frac{v^2}{r} = 0 \qquad (2.1.5)$$

Nonuniform flow: In nonuniform flow, the convective acceleration terms are different from zero.

$$\frac{\partial v_s}{\partial s} = 0 \; or \; \frac{v^2}{r} \neq 0 \qquad (2.1.6)$$

Flow around a bend ($v^2/r = 0$) and flow in expansions or contractions ($\partial v / \partial s = 0$) are examples of nonuniform flow.

Steady flow: In steady flow, the velocity at a point does not change with time

$$\frac{\partial v_s}{\partial t} = 0 \; and \; \frac{\partial v_n}{\partial t} = 0 \qquad (2.1.7)$$

Unsteady flow: In unsteady flow, the velocity at a point varies with time

$$\frac{\partial v_s}{\partial t} \neq 0 \; or \; \frac{\partial v_n}{\partial t} \neq 0 \qquad (2.1.8)$$

Examples of unsteady flow are channel flows with waves, flood hydrographs, and surges. Unsteady flow is difficult to analyze unless the time changes are small.

Laminar flow: In laminar flow, the mixing of the fluid and momentum transfer is by molecular activity.

Turbulent flow: In turbulent flow the mixing of the fluid and momentum transfer is related to random velocity fluctuations. The flow is laminar or turbulent depending on the value of the Reynolds number (Re = $\rho VL/\mu$), which is a dimensionless ratio of the inertial forces to the viscous forces. Here ρ and μ are the density and dynamic viscosity of the fluid, V is the fluid velocity, and L is a characteristic dimension, usually the depth (or the hydraulic radius) in open channel flow. In laminar flow, viscous forces are dominant and Re is relatively small. In turbulent flow, Re is large; that is, inertial forces are very much greater than viscous forces. Turbulent flows are predominant in nature. Laminar flow occurs very infrequently in open channel flow.

Tranquil flow: In open channel flow, the free surface configuration, in response to changes in channel geometry depends on the Froude number (Fr = V/ $\sqrt{gL}$), which is the ratio of inertial forces to gravitational forces. The Froude number is also the ratio of the flow velocity v to the celerity (c = $\sqrt{gL}$) of a small gravity wave in the flow (this concept is detailed in Section 2.4). When Fr < 1, the flow is subcritical (or tranquil), and surface waves propagate upstream as well as downstream. The boundary condition that controls the tranquil flow depth is always located at the downstream end of the subcritical reach.

Rapid flow: When Fr > 1, the flow is supercritical (or rapid) and surface disturbances can propagate only in the downstream direction. The control section of rapid flow depth is always at the upstream end of the rapid flow region. When Fr = 1.0, the flow is critical and surface disturbances remain stationary in the flow.

2.2 Basic Principles

2.2.1 Introduction

The basic equations of flow in open channels are derived from the three conservation laws. These are: (1) the conservation of mass; (2) the conservation of linear momentum; and (3) the conservation of energy. The conservation of mass is another way of stating that (except for mass-energy interchange) matter can neither be created nor destroyed. The principle of conservation of linear momentum is based on Newton's second law of motion which states that a mass (of fluid) accelerates in the direction of and in proportion to the applied forces on the mass.

In the analysis of flow problems, much simplification can result if there is no acceleration of the flow or if the acceleration is primarily in one direction, the accelerations in other directions being negligible. However, a very inaccurate analysis may occur if one assumes accelerations are small or zero when in fact they are not. The concepts explained in this chapter assume one-dimensional flow and the derivations of the equations utilize a control volume. A control volume is an isolated volume in the body of the fluid, through which mass, momentum, and energy can be convected. The control volume may be assumed fixed in space or moving with the fluid.

2.2.2 Conservation of Mass

Consider a short reach of river shown in Figure 2.2.1 as a control volume. The boundaries of the control volume are the upstream cross-section, designated section 1, the downstream cross-section, designated section 2, the free surface of the water between sections 1 and 2, and the interface between the water and the wetted perimeter (banks and bed).

The statement of the conservation of mass for this control volume is

$$\begin{bmatrix} \text{Mass flux} \\ \text{out of the} \\ \text{control volume} \end{bmatrix} - \begin{bmatrix} \text{Mass flux} \\ \text{into the} \\ \text{control volume} \end{bmatrix} + \begin{bmatrix} \text{Time rate of change} \\ \text{in mass in the} \\ \text{control volume} \end{bmatrix} = 0$$

Mass can enter or leave the control volume through any or all of the control volume surfaces. Rainfall would contribute mass through the surface of the control volume and seepage passes through the interface between the water and the banks and bed. In the absence of rainfall, evaporation, seepage and other lateral mass fluxes, mass enters the control volume at section 1 and leaves at section 2.

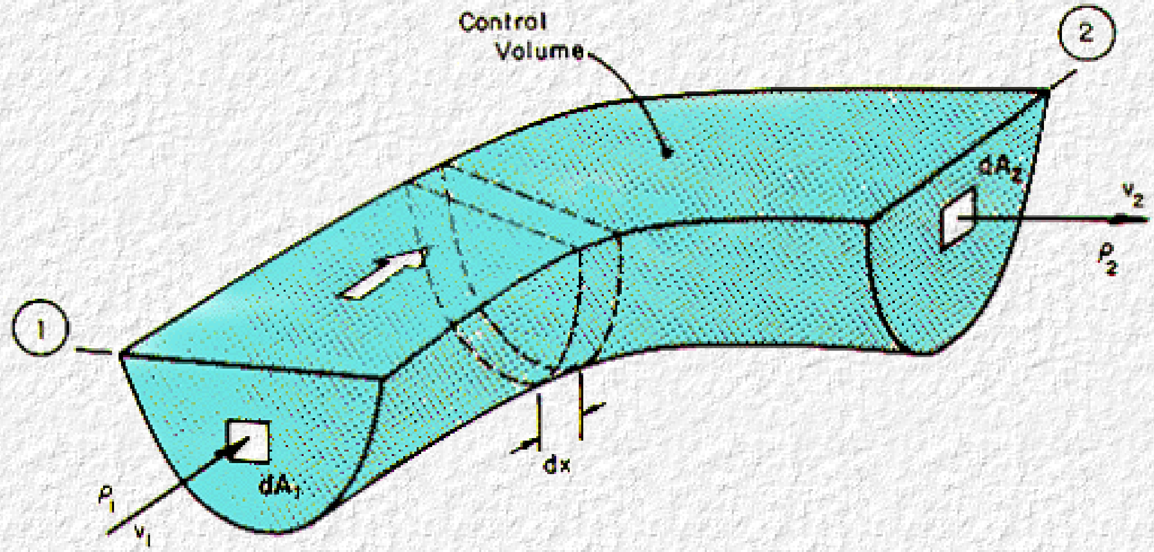

Figure 2.2.1. A River Reach as a Control Volume

At section 2, the mass flux out of the control volume through the differential area dA_2 is $\rho_2\ v_2\ dA_2$. The values of ρ_2 and v_2 can vary from position to position across the width and throughout the depth of flow at section 2. The total mass flux out of the control volume at section 2 is the integral of all $\rho_2\ v_2\ dA_2$ through the differential areas that make up the cross-section area A_2, and may be written as

Mass flux out of the control volume $= \int_{A_2} \rho_2 v_2 dA_2$

Similarly

Mass flux into the control volume $= \int_{A_1} \rho_1 v_1 dA_1$

The amount of mass inside a differential volume $d\forall$ inside the control volume is $\rho\ d\forall$ and

Mass inside the control volume $= \int_{\forall} \rho\ d\forall$

The statement of conservation of mass for the control volume calls for the time rate of change in mass. In mathematical notation,

Time rate of change in mass in the control volume $= \dfrac{\partial}{\partial t}\int_{\forall} \rho \, d\forall$

For the reach of river, the statement of the conservation of mass becomes

$$\int_{A_2} \rho_2 v_2 dA_2 - \int_{A_1} \rho_1 v_1 dA_1 + \frac{\partial}{\partial t}\int_{\forall} \rho \, d\forall = 0 \qquad (2.2.1)$$

It is often convenient to work with average conditions at a cross-section, so we define an average velocity V such that

$$V = \frac{1}{A}\int_{A} v \, dA \qquad (2.2.2)$$

The symbol v represents the local velocity whereas the velocity V is the average velocity at the cross-section.

Because water is nearly incompressible the density ρ of the fluid is considered constant, $\rho_1 = \rho_2 = \rho$. When the flow is steady

$$\frac{\partial}{\partial t}\int_{\forall} \rho d\forall = 0 \qquad (2.2.3)$$

and Equation 2.2.1 reduces to the statement that inflow equals outflow or

$$\rho V_2 A_2 - \rho V_1 A_1 = 0$$

That is, for steady flow of incompressible fluids

$$V_1 A_1 = V_2 A_2 = Q = VA \qquad (2.2.4)$$

where Q is the volume flow rate or the discharge.

Equation 2.2.4 is the familiar form of the conservation of mass equation for steady flow in rivers. It is applicable when the fluid density is constant, the flow is steady and there is no significant lateral inflow or seepage.

2.2.3 Conservation of Linear Momentum

The curved reach of the river shown in Figure 2.2.1 is rather complex to analyze in terms of Newton's Second Law because of the curvature in the flow. Therefore, as a starting point, the differential length of reach dx is isolated as a control volume.

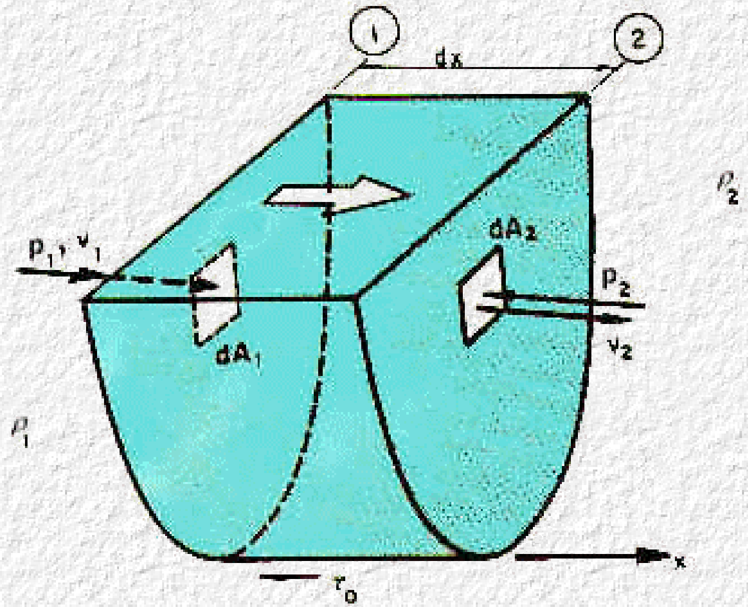

Figure 2.2.2. The Control Volume for Conservation of Linear Momentum

For this control volume, shown in Figure 2.2.2, the pressure terms p_1 and p_2 are directed toward the control volume in a direction normal to the sections 1 and 2. The shear stress τ_o is exerted along the interface between the water and the wetted perimeter and is acting in a direction opposite to the axis x. The statement of conservation of linear momentum is

$$\begin{bmatrix}\text{Flux of momentum}\\ \text{out of the}\\ \text{control volume}\end{bmatrix} - \begin{bmatrix}\text{Flux of momentum}\\ \text{into the}\\ \text{control volume}\end{bmatrix} + \begin{bmatrix}\text{Time rate of change}\\ \text{of momentum in}\\ \text{the control volume}\end{bmatrix} = \begin{bmatrix}\text{Sum of the forces}\\ \text{acting on the fluid}\\ \text{in the control volume}\end{bmatrix}$$

The terms in the statement are vectors so we must be concerned with direction as well as magnitude.

Consider the conservation of momentum in the direction of flow (the x-direction in Figure 2.2.2). At the outflow section (section 2), the flux of momentum out of the control volume through the differential area dA_2 is

$$\rho_2 v_2 \, dA_2 \, v_2$$

Here $\rho_2 \, v_2 \, dA_2$ is the mass flux (mass per unit of time) and $\rho_2 v_2 \, dA_2 \, v_2$ is the momentum flux through the area dA_2.

Flux of mometum out of the control volume $= \int_{A_2} \rho_2 v_2 dA_2 v_2$

Similarly, at the inflow section (section 1),

Flux of mometum into the control volume $= \int_{A_1} \rho_1 v_1 dA_1 v_1$

The amount of momentum in the control volume is $\int_{\forall} \rho v \, dV$ so

Time rate of change of mometum in the control volume $= \frac{\partial}{\partial t}\{\int_{\forall} \rho v \, dV\}$

At the upstream section, the force acting on the differential area dA_1 of the control volume is $p_1 \, dA_1$ where p_1 is the pressure from the upstream fluid on the differential area. The total force in the x-direction at section 1 is $\int_{A_1} p_1 \, dA_1$. Similarly, at section 2, the total force is $-\int_{A_2} p_2 \, dA_2$.

There is a fluid shear stress τ_o acting along the interface between the water and the bed and banks. The shear on the control volume is in a direction opposite to the direction of flow and results in a force $-\tau_o P \, dx$ where τ_o is the average shear stress on the interface area, P is the average wetted perimeter and dx is the length of the control volume. The term P dx is the interface area.

The body force component acting in the x-direction is denoted F_b and will be discussed in a subsequent section. The statement of conservation of momentum in the x-direction for the control volume is

$$\int_{A_2} \rho_2 v_2^2 \, dA_2 - \int_{A_1} \rho_1 v_1^2 \, dA_1 + \frac{\partial}{\partial t}\int_{\forall} \rho v \, dV = \int_{A_1} p_1 \, dA_1 - \int_{A_2} p_2 \, dA_2 - \int_L \tau_o P \, dx + F_b \quad (2.2.5)$$

Again, as with the conservation of mass equation, it is convenient to use average velocities instead of point velocities. We define a momentum coefficient β so that when average velocities are used instead of point velocities, the correct momentum flux is considered.

The momentum coefficient for incompressible fluids is

$$\beta = \frac{1}{V^2 A} \int_A v^2 dA \qquad (2.2.6)$$

For steady incompressible flow, Equation 2.2.5 is combined with Equation 2.2.6 to give

$$\rho\beta_2 V_2^2 A_2 - \rho\beta_1 V_1^2 A_1 = \int_{A_1} p_1 dA_1 - \int_{A_2} p_2 dA_2 - \int_L \tau_0 P\, dx + F_b \qquad (2.2.7)$$

The pressure force and shear force terms on the right-hand side of Equation 2.2.7 are usually abbreviated as ΣF_x so

$$\Sigma F_x = = \int_{A_1} p_1 dA_1 - \int_{A_2} p_2 dA_2 - \int_L \tau_0 P\, dx + F_b \qquad (2.2.8)$$

The conservation of momentum equation becomes

$$\rho\beta_2 V_2^2 A_2 - \rho\beta_1 V_1^2 A_1 = \Sigma F_x \qquad (2.2.9)$$

for steady flow with constant density. With Equation 2.2.4 the steady flow conservation of linear momentum equation takes on the familiar form

$$\rho Q(\beta_2 V_2 - \beta_1 V_1) = \Sigma F_x \qquad (2.2.10)$$

2.2.4 Conservation of Energy

The First Law of Thermodynamics can be written

$$\dot{Q} - \dot{W} = \frac{dE}{dt} \qquad (2.2.11)$$

Where

$\dot{Q}$ = the rate at which heat is added to a fluid system

$\dot{W}$ = the rate at which a fluid system does work on its surroundings

E = the energy of the system

Then dE/dt is the rate of change of energy in the system.

The statement of conservation of energy for a control volume is then

$$\begin{bmatrix}\text{Flux of energy}\\ \text{out of the}\\ \text{control volume}\end{bmatrix} - \begin{bmatrix}\text{Flux of energy}\\ \text{into the}\\ \text{control volume}\end{bmatrix} + \begin{bmatrix}\text{Time rate of change}\\ \text{of energy in the}\\ \text{control volume}\end{bmatrix} = \dot{Q} - \dot{W}$$

The choice of a control volume is arbitrary. To illustrate the procedure the control volume is reduced to the size of a streamtube connecting dA_1 and dA_2 as shown in Figure 2.2.3. The streamtube is bounded by streamlines through which no mass or momentum enters.

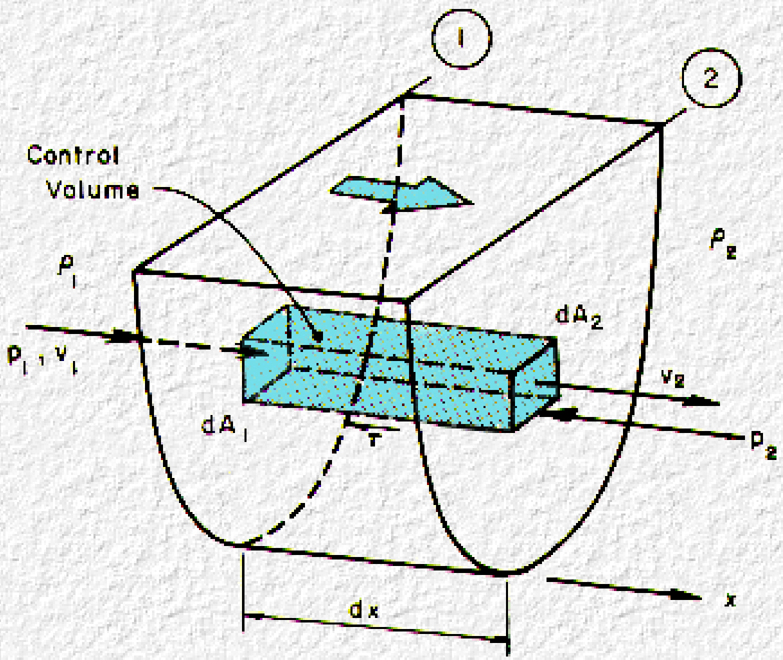

Figure 2.2.3. The Streamtube as a Control Volume

For steady flow of an incompressible fluid in the streamtube

Flux of energy out of the control volume $= \rho_2 e_2 dA_2 v_2$

Flux of energy into the control volume $= \rho_1 e_1 dA_1 v_1$

and

Time rate of change of energy in the control volume $= 0$

Here e is the energy per unit mass. Accordingly, the total energy E in a control volume is

$$E = \int_{\forall} \rho e \, d\forall \tag{2.2.12}$$

Unless one is concerned with thermal pollution, evaporation losses, or problems concerning the formation of ice in rivers, the rate at which heat is added to the control volume can be neglected; that is

$$\dot{Q} \simeq 0 \tag{2.2.13}$$

The work done by the fluid in the control volume on its surroundings can be in the form of pressure work W_p, shear work W_τ, or shaft work (mechanical work) W_s. For the streamtube shown in Figure 2.2.3, no shaft work is involved ($W_s = 0$).

The rate at which the fluid pressure does work on the control volume surrounding through the boundary dA_1 in Figure 2.2.3 is

$$- p_1 dA_1 v_1$$

and on boundary dA_2, the rate of doing pressure work is

$$p_2 dA_2 v_2$$

At the other boundaries of the streamtube, there is no pressure work because there is no fluid motion normal to the boundary. Hence, for the streamtube

$$\dot{W}_p = p_2 dA_2 v_2 - p_1 dA_1 v_1 \tag{2.2.14}$$

Along the interior boundaries of the streamtube there is a shear stress resulting from the condition that the fluid velocity inside the streamtube may not be the same as the velocity of the fluid surrounding the streamtube. The rate at which the fluid in the streamtube does shear work on the control volume is

$$\dot{W}_\tau = \tau P \, dx \, v \tag{2.2.15}$$

where τ is the average shear stress on the streamtube boundary, P is the average perimeter of the streamtube, dx is the length of the streamtube and v is the fluid velocity at the streamtube boundary. The product P dx is the surface of the streamtube subjected to shear stresses.

Then for steady flow in the streamtube, the statement of the conservation of energy in the streamtube shown in Figure 2.2.3. is

$$\rho_2\, e_2 v_2 dA_2 - \rho_1\, e_1 v_1 dA_1 = p_1 v_1 dA_1 - p_2 v_2 dA_2 - \tau P v\, dx \qquad (2.2.16)$$

The conservation of mass for steady flow in the streamtube is (according to Equation 2.2.4)

$$v_2 dA_2 = v_1 dA_1 = dQ \qquad (2.2.17)$$

Now Equation 2.2.16 reduces to

$$(\rho_1\, e_1 + p_1)\, dQ - (\rho_2\, e_2 + p_2)\, dQ = \tau P v\, dx \qquad (2.2.18)$$

The energy per unit mass e is the sum of the internal, kinetic and potential energies or

$$e = u + \frac{v^2}{2} + gz \qquad (2.2.19)$$

where

u = the internal energy associated with the fluid temperature

v = the velocity of the mass fluid

g = the acceleration due to gravity

z = the elevation above some arbitrary reference level.

This expression for e is substituted in Equation 2.2.18 to yield

$$u_1 + \frac{v_1^2}{2} + gz_1 \frac{p_1}{\rho} = u_2 + \frac{v_2^2}{2} + gz_2 \frac{p_2}{\rho} + \frac{\tau P v dx}{\rho dQ} \qquad (2.2.20)$$

By dividing through by g and defining the head loss h_ℓ as follows

$$h_\ell = \frac{u_2 - u_1}{g} + \frac{\tau P v dx}{\rho dQ} \qquad (2.2.21)$$

The energy equation for the streamtube becomes

$$\frac{v_1^2}{2g} + \frac{P_1}{\gamma} + z_1 = \frac{v_2^2}{2g} + \frac{P_2}{\gamma} + z_2 + h_\ell \qquad (2.2.22)$$

If there is no shear stress on the streamtube boundary and if there is no change in internal energy ($u_1 = u_2$), the energy equation reduces to

$$\frac{v_1^2}{2g} + \frac{P_1}{\gamma} + z_1 = \frac{v_2^2}{2g} + \frac{P_2}{\gamma} + z_2 = \text{const.} \qquad (2.2.23)$$

which is the Bernoulli Equation.

Generally, there is not sufficient information available to do a differential streamtube analysis of a reach of river, so appropriate changes must be made in the energy equation. A reach of river such as that shown in Figure 2.2.1 can be pictured as a bundle of streamtubes. We know the statement of the conservation of energy for a streamtube. It is Equation 2.2.22 which can be written

$$\left(\frac{v_1^2}{2g} + \frac{P_1}{\gamma} + z_1\right) v\,dA = \left(\frac{v_2^2}{2g} + \frac{P_2}{\gamma} + z_2\right) v\,dA + h_\ell\, v dA \qquad (2.2.24)$$

because $v_1\, dA_1 = v_2\, dA_2 = vdA$ for the streamtube.

The common form of the energy equation used in open channel flow is derived by integrating Equation 2.2.24 over the cross-section area

$$\int_A \left(\frac{v^2}{2g} + \frac{p}{\gamma} + z\right) v dA = \left\{\frac{\alpha v^2}{2g} + \frac{\bar{p}}{\gamma} + \bar{z}\right\} Q \qquad (2.2.25)$$

Where α is the kinetic energy correction factor defined by the expression

$$\alpha = \frac{1}{V^3 A} \int_A v^3 dA \qquad (2.2.26)$$

to allow the use of average velocity V rather than point velocity v. The average pressure over the cross-section is $\bar{p}$, defined as

$$\bar{p} = \frac{1}{VA} \int_A p\, v dA \tag{2.2.27}$$

The term $\bar{z}$ is the average elevation of the cross-section defined by the expression

$$\bar{z} = \frac{1}{VA} \int_A z v dA \tag{2.2.28}$$

and Q is the volume flow rate or the discharge. By definition

$$Q = \int_A v dA \tag{2.2.29}$$

Also

$$H_L = \frac{1}{VA} \int_A h_\ell\, v dA \tag{2.2.30}$$

In summary, the expression for conservation of energy for steady flow in a reach of river is written

$$\frac{\alpha_1 v_1^2}{2g} + \frac{\bar{p}_1}{\gamma} + \bar{z}_1 = \frac{\alpha_2 v_2^2}{2g} + \frac{\bar{p}_2}{\gamma} + \bar{z}_2 + H_L \tag{2.2.31}$$

The tendency in river work is to neglect the energy correction factor even though its value may be as large as 1.5. Usually it is assumed that the pressure is hydrostatic and the average elevation head $\bar{z}$ is at the centroid of the cross-sectional area. However, it should be kept in mind that Equation 2.2.26, Equation 2.2.27 and Equation 2.2.28 are the correct definitions of the terms in the energy equation. Appendix 2 has a solved example problem illustrating the calculation of α and β for a stream.

2.2.5 Hydrostatics

When the only forces acting on the fluid are pressure and fluid weight, the differential equation of motion in an arbitrary direction x is

$$\frac{\partial}{\partial x}\left(\frac{p}{\gamma} + z\right) = \frac{a_x}{g} \tag{2.2.32}$$

In steady uniform flow (and for zero flow), the acceleration is zero and we obtain the equation of hydrostatics

$$\frac{p}{\gamma} + z = \text{Constant} \tag{2.2.33}$$

However, when there is acceleration, the piezometric head varies in the flow field. That is, the piezometric head is not constant in the flow. This is illustrated in Figure 2.2.4. In Figure 2.2.4a the pressure at the bed is hydrostatic and equal to γy_o whereas in the curvilinear flow (Figure 2.2.4b) the pressure is larger than γy_o because of the acceleration resulting from a change in direction.

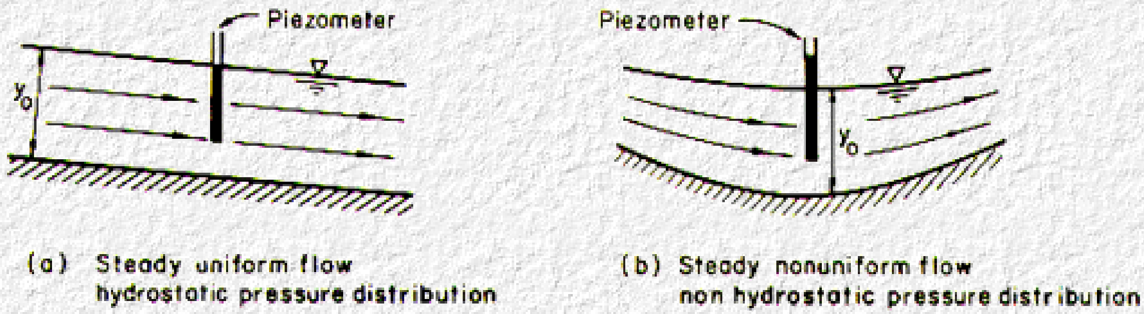

Figure 2.2.4. Pressure Distribution in Steady Uniform and in Steady Nonuniform Flow

In general, when fluid acceleration is small (as in gradually varied flow) the pressure distribution is considered hydrostatic. However, for rapidly varying flow where the streamlines are converging, expanding or have substantial curvature (curvilinear flow), fluid accelerations are not small and the pressure distribution is not hydrostatic.

In Equation 2.2.33, the constant is equal to zero for gage pressure at the free surface of a liquid, and for flow with hydrostatic pressure throughout (steady, uniform flow or gradually varied flow) it follows that the pressure head p/γ is equal to the vertical distance below the free surface. In sloping channels with steady uniform flow, the pressure head p/γ at a depth y below the surface is equal to

$$\frac{p}{\gamma} = y\cos\theta \tag{2.2.34}$$

Note that y is the depth (perpendicular to the water surface) to the point, as shown in Figure 2.2.5. For most channels, θ is small and $\cos\theta \simeq 1$.

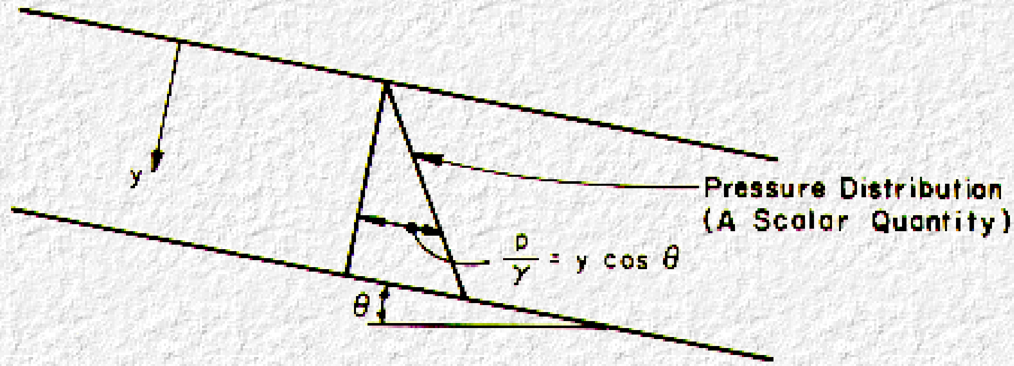

Figure 2.2.5. Pressure Distribution in Steady Uniform Flow on Steep Slopes

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2.3 Steady Uniform Flow

In steady, uniform open channel flow there are no accelerations, streamlines are straight and parallel, and the pressure distribution is hydrostatic. The slope of the water surface S_w and the bed surface S_o and the energy gradient S_f are equal. Consider the unit width of channel shown in Figure 2.3.1 as a control volume. According to Equation 2.2.31, the conservation of energy for this control volume is

$$\frac{\alpha_1 v_1^2}{2g} + \frac{\bar{p}_1}{\gamma} + \bar{z}_1 = \frac{\alpha_2 v_2^2}{2g} + \frac{\bar{p}_2}{\gamma} + \bar{z}_2 + H_L \qquad (2.2.31)$$

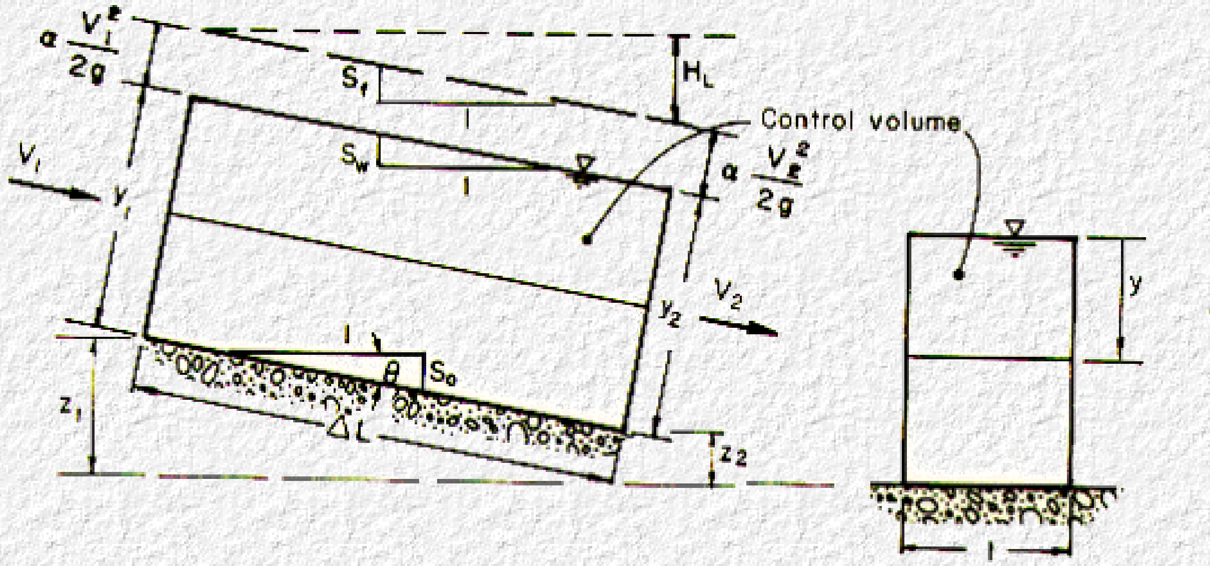


Figure 2.3.1. Steady Uniform Flow in a Unit Width Channel

The pressure at any point y below the surface is y cosθ. Then according to Equation 2.2.27

$$\bar{p}_1 = \frac{1}{V_1 y_1} \int_0^{y_1} \gamma y \cos\theta \, v_1 \, dy$$

Assuming only small variations in the point velocity v with y we have

$$\bar{p}_1 \simeq \frac{\gamma y_1 \cos\theta}{2}$$

Similarly,

$$\bar{p}_2 \simeq \frac{\gamma y_2 \cos\theta}{2}$$

Also according to Equation 2.2.28

$$\bar{z}_1 \simeq z_1 + \frac{y_1 \cos\theta}{2}$$

and

$$\bar{z}_2 \simeq z_2 + \frac{y_2 \cos\theta}{2}$$

With the above expressions for $\bar{p}_1, \bar{p}_2, \bar{z}_1$ and $\bar{z}_2$ the energy equation for this control volume reduces to

$$\frac{\alpha_1 V_1^2}{2g} + \frac{y_1 \cos\theta}{2} + z_1 + \frac{y_1 \cos\theta}{2} = \frac{\alpha_2 V_2^2}{2g} + \frac{y_2 \cos\theta}{2} + z_2 + \frac{y_2 \cos\theta}{2} + H_L \qquad (2.3.1a)$$

or

$$\frac{\alpha_1 V_1^2}{2g} + y_1 \cos\theta + z_1 = \frac{\alpha_2 V_2^2}{2g} + y_2 \cos\theta + z_2 + H_L \qquad (2.3.1b)$$

For most natural channels θ is small and y cosθ ≃ y. The velocity distribution in the vertical is normally a log function for which $\alpha_1 \simeq \alpha_2 \simeq 1$. Then the energy equation becomes

$$\frac{V_1^2}{2g} + y_1 + z_1 = \frac{V_2^2}{2g} + y_2 + z_2 + H_L \qquad (2.3.1c)$$

and the slopes of the bed, water surface and energy grade line are respectively

$$S_o = \sin\theta = \frac{(z_1 - z_2)}{\Delta L} \qquad (2.3.2)$$

$$S_w = \frac{(z_1 + y_1) - (z_2 + y_2)}{\Delta L} \qquad (2.3.3)$$

and

$$S_f = \frac{H_L}{\Delta L} = \frac{\left(\frac{V_1^2}{2g} + y_1 + z_1\right) - \left(\frac{V_2^2}{2g} + y_2 + z_2\right)}{\Delta L} \qquad (2.3.4)$$

Steady uniform flow is an idealized concept for open channel flow and is difficult to obtain even in laboratory flumes. For many applications, the flow is steady and the changes in width, depth or direction (resulting in nonuniform flow) are so small that the flow can be considered uniform. In other cases, the changes occur over such a long distance the flow is a gradually varied flow.

Variables of interest for steady uniform flow are: (1) the mean velocity V; (2) the discharge Q; (3) the velocity distribution v(y) in the vertical; (4) the head loss H_L through the reach; and (5) the shear stress, both local τ and at the bed τ_o. These variables are interrelated.

2.3.1 Shear Stress and Velocity Distribution

Shear stress τ is the internal fluid stress which resists deformation. The shear stress exists only when fluids are in motion. It is a tangential stress in contrast to pressure, which is a normal stress.

The local shear stress at the interface between the boundary and the fluid can be determined quite easily if the boundary is hydraulically smooth; that is, if the roughness at the boundary is submerged in a viscous sublayer as shown in Figure 2.3.2.

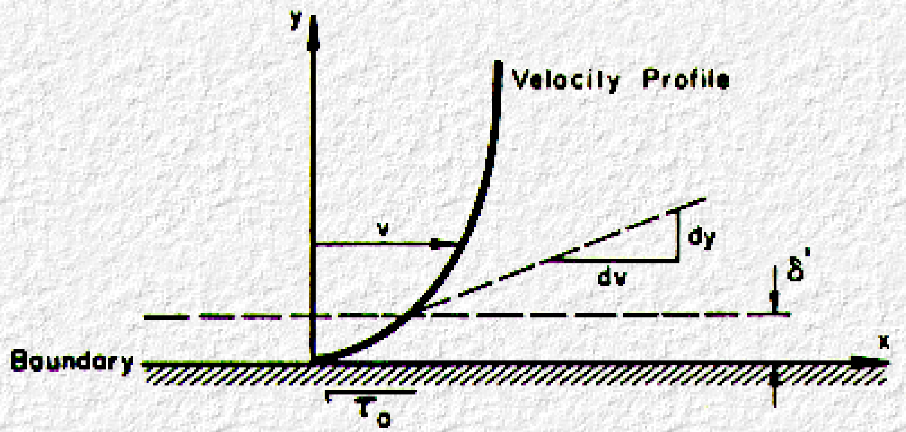


Figure 2.3.2. Hydraulically Smooth Boundary

Here, the thickness of the laminar sublayer is denoted δ'. In laminar flow, the shear stress at the boundary is

$$\tau_0 = \mu\left(\frac{dv}{dy}\right) \text{at } y = 0 \qquad (2.3.5)$$

The velocity gradient is evaluated at the boundary. The dynamic viscosity μ is the proportionality constant relating boundary shear and velocity gradient in the viscous sublayer.

When the boundary is hydraulically rough, the thickness of the laminar sublayer is very small compared to the roughness height. The path of fluid particles in the vicinity of the boundary are shown in Figure 2.3.3.

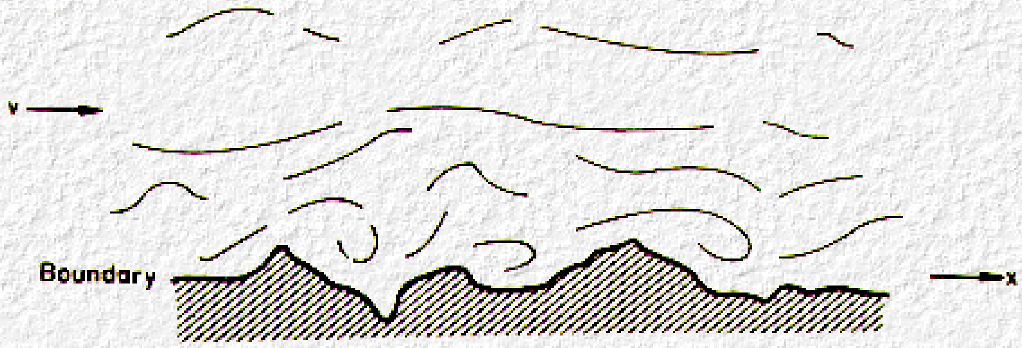
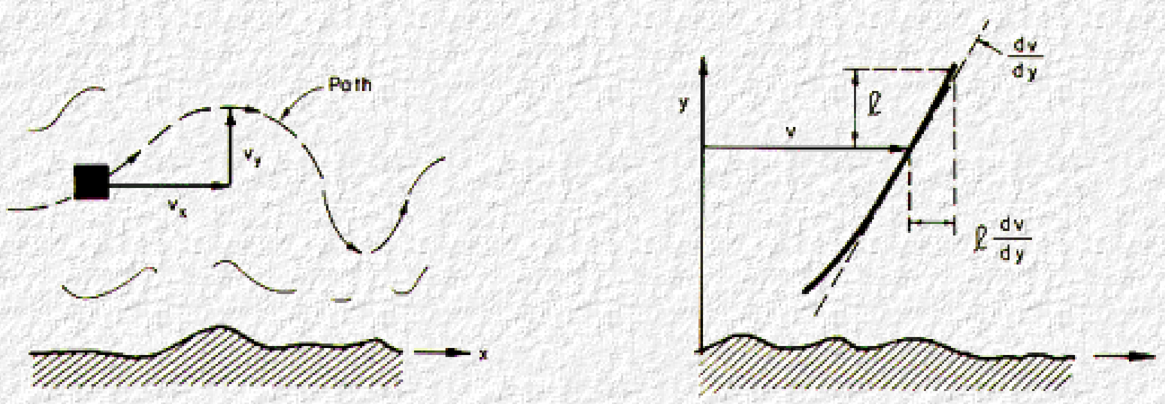


Figure 2.3.3. Hydraulically Rough Boundary

The velocity at a point near the boundary fluctuates randomly about a mean value. The random fluctuation in velocity characterizes turbulent flows. As shown in Figure 2.3.4a, the particle has a vertical component of velocity v_y as well as a horizontal component v_x.



(a) Path of the particle (b) Velocity profile

Figure 2.3.4. Velocities in Turbulent Flow

The two components of velocity in Figure 2.3.4a can be written as

$$v_x = \bar{v}_x + v_x' \qquad (2.3.6)$$

and

$$v_y = \bar{v}_y + v_y' \qquad (2.3.7)$$

where $\bar{v}_x$ and $\bar{v}_y$ are the time-averaged mean velocities in the x and y direction and v_x' and v_y' are the fluctuating components.

Through theoretical investigation it has been found that turbulence generates shear stress given by

$$\tau = -\rho\overline{v'_x v'_y} \qquad (2.3.8)$$

The term $\overline{v'_x v'_y}$ is the time-average of the product of v_x' and v_y' at a point in the flow. It is called the Reynolds shear stress.

Prandtl (1925) suggested that v'_x and v'_y are related to the velocity gradient dv/dy shown in Figure 2.3.4b. He proposed to characterize the turbulence with a dimension called the "mixing length," ℓ. Accordingly,

$$v'_x \sim \ell\frac{dv}{dy} \qquad (2.3.9)$$

$$v'_y \sim \ell\frac{dv}{dy} \qquad (2.3.10)$$

and

$$\tau \sim \rho\ell^2\left(\frac{dv}{dy}\right)^2 \qquad (2.3.11)$$

If it is assumed that the mixing length can be represented by the product of a constant κ and y (i.e. $\ell = \kappa y$), then for steady uniform turbulent flow,

$$\tau = \rho\kappa^2 y^2\left(\frac{dv}{dy}\right)^2 \qquad (2.3.12)$$

Using different reasoning von Karman (1930) derived the same equation. Equation 2.3.12 can be rearranged to the form

$$\frac{dv}{dy} = \frac{\sqrt{\tau_0 / \rho}}{\kappa y} \tag{2.3.13}$$

where κ is the von Karman universal velocity coefficient. For rigid boundaries κ has the average value of 0.4. The term τ_o is the bed shear stress. The term $(\tau_0/p)^{1/2}$ has the dimensions of velocity and is called the shear velocity, V_*. Integration of Equation 2.3.13 yields

$$\frac{v}{\sqrt{\frac{\tau_0}{\rho}}} = \frac{1}{\kappa} \ln \frac{y}{y'} = \frac{2.31}{\kappa} \log \frac{y}{y'} \tag{2.3.14}$$

Here ln is the logarithm to the base e and log is the logarithm to the base 10. The term y' results from evaluation of the constant of integration assuming v = 0 at some distance y' above the bed.

The term y' depends on the flow and has been experimentally determined. The many experiments have resulted in characterizing turbulent flow into three general types:

1. Hydraulically smooth boundary turbulent flow where the velocity distribution, mean velocity and resistance to flow are independent of the boundary roughness of the bed but depend on fluid viscosity. Then with

$$\delta' = \frac{11.6\upsilon}{\sqrt{\frac{\tau_0}{\rho}}}, \text{ we find } y' = \frac{\delta'}{107}$$

2. Hydraulically rough boundary turbulent flow where velocity distribution, mean velocity and resistance to flow are independent of viscosity and depend entirely on the boundary roughness. For this case, $y' = k_s/30.2$ where k_s is the height of the roughness element.
3. Transition where the velocity distribution, mean velocity and resistance to flow depend on both fluid viscosity and boundary roughness. Then

$$\frac{\delta'}{107} < y' < \frac{k_s}{30.2}$$

As a result, the velocity distribution v, mean velocity V, and resistance to flow equations can be written in the following dimensionless form

$$\frac{v}{V_*} = 5.75 \log\left(30.2\frac{X_y}{k_s}\right) = 2.5 \ln\left(30.2\frac{X_y}{k_s}\right) \qquad (2.3.15)$$

and

$$\frac{V}{V_*} = \frac{C}{\sqrt{g}} = 5.75 \log 12.27\frac{Xy_0}{k_s} = 2.5 \ln 12.27\frac{Xy_0}{k_s} \qquad (2.3.16)$$

Note that any system of units can be used as long as y_o and k_s (and V, v and V_*) have the same dimensions. The symbols of Equation 2.3.15 and Equation 2.3.16 denote

X = a coefficient given in Figure 2.3.5

k_s = the height of the roughness elements. For sand channels, k_s is the D_{65} of the bed material

v = the local mean velocity at depth y

y_o = the depth of flow

V = the depth-averaged velocity

V_* = the shear velocity $\sqrt{\tau_0 / \rho}$ which for steady uniform flow is $\sqrt{gRS_f}$

τ_o = the shear stress at the boundary and for steady uniform flow $\tau_o = \gamma RS_f$

R = the hydraulic radius, equal to the cross-sectional area A divided by the wetted perimeter P

S_f = the slope of the energy gradeline

δ' = the thickness of the viscous sublayer

$$\delta' = 11.6\upsilon / V_* \qquad (2.3.17)$$

$C / \sqrt{g}$ = the Chezy discharge coefficient in the equation

$$V = C\sqrt{RS} \text{ or } C = \left(\frac{8g}{f}\right)^{1/2} \qquad (2.3.18)$$

f = the Darcy-Weisbach resistance coefficient which is given by the expression

$$f = 8\frac{\tau_0}{\rho V^2} \tag{2.3.19}$$

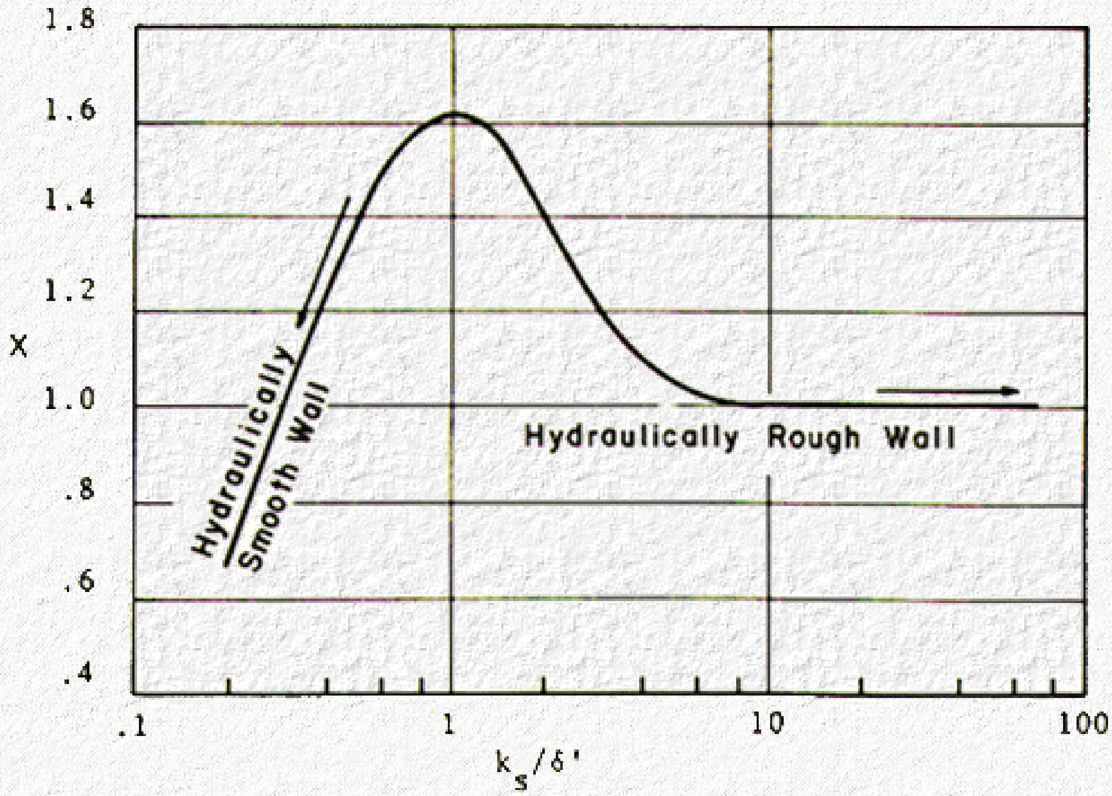


Figure 2.3.5. Einstein's Multiplication Factor X in the Logarithmic Velocity Equations (Einstein, 1950)

2.3.2 Empirical Velocity Equations

Because of the difficulties involved in determining the shear stress and hence the velocity distribution in turbulent flows, the empirical approach to determine mean velocities in rivers has been prevalent. Two such empirical equations are in common use. They are Manning's equation

$$V = \frac{1.486}{n} R^{2/3} S_f^{1/2} \tag{2.3.20a}$$

and Chezy's equation

$$V = CR^{1/2} S_f^{1/2} \quad (2.3.20b)$$

where

V = the average velocity in the waterway cross-section in ft/s

n = Manning's roughness coefficient

R = the hydraulic radius in feet equal to the cross-sectional area A divided by the wetted perimeter P of the waterway

S_f = the friction slope

C = Chezy's discharge coefficient known as Chezy's C.

In these equations, the boundary shear stress is expressed implicitly in the roughness coefficient "n", or in the discharge coefficient C. By equating the velocity determined from Manning's equation with the velocity determined from Chezy's equation, the relation between the coefficients is

$$C = \frac{1.486}{n} R^{1/6} \quad (2.3.21)$$

If the flow is gradually varied, Manning's and Chezy's equations are used with the average friction slope $S_{f_{ave}}$. The term $S_{f_{ave}}$ is determined by averaging over a short time increment at a station or over a short length increment (1000 feet for example) at an instant of time, or both.

Over many decades, a catalog of values of Manning's n and Chezy's C has been assembled so that an engineer can estimate the appropriate value by knowing the general nature of the channel boundaries. An abbreviated list of Manning's roughness coefficients is given in Table 2.3.1. Additional values are given by Barnes (1967) and V. T. Chow (1959) for dredged and lined channels. Manning's n for sandbed and gravel-bed channels is discussed in detail in Chapter 3.

The general approach for estimating n values consists of the selection of a base roughness value for a straight, uniform, smooth channel in the materials involved, then additive values are considered for the channel under consideration

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\, m_5 \quad (2.3.22)$$

in which

n_o = base value for straight uniform channels

n_1 = additive value due to cross-section irregularity

n_2 = additive value due to variations of the channel

n_3 = additive value due to obstructions

n_4 = additive value due to vegetation

m_5 = multiplication factor due to sinuosity

Detailed values of the coefficients are found in Cowan (1956), Chow (1964), Benson and Dalrymple (1967) and Aldridge and Garrett (1973). Typical values are given in Table 2.3.2. Arcement and Schneider (1984) proposed a guide for selecting Manning's roughness coefficients for flood plains. For steeper streams, the reader is also referred to the work of Jarrett (1985).

Table 2.3.1 Manning's Roughness Coefficients for Various Boundaries	
Rigid Boundary Channels	**Manning's n**
Very smooth concrete and planed timber	0.011
Smooth concrete	0.012
Ordinary concrete lining	0.013
Wood	0.014
Vitrified clay	0.015
Shot concrete, untrowelled, and earth channels in best condition	0.017
Straight unlined earth canals in good condition	0.020
Mountain streams with rocky beds	0.040 - 0.050
Minor Streams (top width at flood stage < 100 ft)	
Streams on Plain	
1. Clean, straight, full stage, no rifts or deep pools	0.025 - 0.033
2. Same as above, but more stones and weeds	0.030 - 0.040
3. Clean, winding, some pools and shoals	0.033 - 0.045
4. Same as above, but some weeds and stones	0.035 - 0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040 - 0.055
6. Same as 4, but more stones	0.045 - 0.060
7. Sluggish reaches, weedy, deep pools	0.050 - 0.080
8. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbush	0.075 - 0.150

Mountain Streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stages	
1. Bottom: gravels, cobbles and few boulders	0.030 - 0.050
2. Bottom: cobbles with large boulders	0.040 - 0.070
Flood Plains	
Pasture, No Brush	
1. Short Grass	0.025 - 0.035
2. High Grass	0.030 - 0.050
Cultivated Areas	
1. No Crop	0.020 - 0.040
2. Mature Row Crops	0.025 - 0.045
3. Mature Field Crops	0.030 - 0.050
Brush	
1. Scattered brush, heavy weeds	0.035 - 0.070
2. Light brush and trees in winter	0.035 - 0.060
3. Light brush and trees in summer	0.040 - 0.080
4. Medium to dense brush in winter	0.045 - 0.110
5. Medium to dense brush in summer	0.070 - 0.160
Trees	
1. Dense willows, summer, straight	0.110 - 0.200
2. Cleared land with tree stumps, no sprouts	0.030 - 0.050
3. Same as above, but with heavy growth of sprouts	0.050 - 0.080
4. Heavy stand of timber, a few down trees, little undergrowth, flood stage below branches	0.080 - 0.120
5. Same as above, but with flood stage reaching branches	0.100 - 0.160
Major Streams (Top width at flood stage > 100 ft). The n value is less than that for minor streams of similar description, because banks offer less effective resistance.	

Regular section with no boulders or brush	0.025 - 0.060
Irregular and rough section	0.035 - 0.100
Alluvial Sandbed Channels (no vegetation)[1]	
Tranquil flow, Fr < 1	
plane bed	0.014 - 0.020
ripples	0.018 - 0.030
dunes	0.020 - 0.040
washed out dunes or transition	0.014 - 0.025
plane bed	0.010 - 0.013
Rapid Flow, Fr > 1	
standing waves	0.010 - 0.015
antidunes	0.012 - 0.020

[1] Data is limited to sand channels with D_{50} < 1.0 mm, details to be discussed in Chapter 3.

Table 2.3.2 Adjustment Factors for the Determination of n Values				
Cross-Section Irregularity				
Conditions			**n Value**	**Remarks**
Cross-section Irregularity	n_1	Smooth Minor Moderate Severe	0 0.001 - 0.005 0.006 - 0.010 0.011 - 0.020	Smoothest Channel Slightly Eroded Side Slopes Moderately Rough Bed and Banks Badly Sloughed & Scalloped Banks
Variations In Channel Section				
Conditions			**n Value**	**Remarks**
Variations in Channel Section	n_2	Gradual Alternating Occasionally Alternating Frequently	0 0.001 - 0.005 0.010 - 0.015	Gradual Changes Occasional Shifts From Large to Small Sections Frequent Changes In Cross-Sectional shape
Obstructions				
Conditions			**n Value**	**Remarks**

Obstructions	n_3	Negligible Minor Moderate Severe	0 - 0.004 0.005 - 0.015 0.020 - 0.030 0.040 - 0.060	Obstructions < 5% of Cross-Section Area Obstructions < 15% of Cross-Section Area Obstructions 15 - 50% of Cross-Section Area Obstructions > 50% of Cross-Section Area
Vegetation				
Conditions			**n Value**	**Remarks**
Vegetation	n_4	Small Medium Large Very Large	0.002 - 0.010 0.010 - 0.025 0.025 - 0.050 0.050 - 0.100	Flow Depth > 2 x < Vegetation Height Flow Depth < Vegetation Height Flow Depth > Vegetation Height Flow Depth < 0.5 Vegetation Height
Sinuosity				
Conditions			**n Value**	**Remarks**
Sinuosity	m_5	Minor Moderate Severe	1.00 1.15 1.30	Sinuosity < 1.2 1.2 < Sinuosity < 1.5 Sinuosity > 1.5

The roughness characteristics on the floodplain are complicated by the presence of vegetation, natural and artificial irregularities, buildings, undefined direction of flow, varying slopes and other complexities. Resistance factors reflecting these effects must be selected largely on the basis of past experience with similar conditions. In general, resistance to flow is large on the floodplains. In some instances, conditions are further complicated by deposition of sediment and development of dunes and bars which affect resistance to flow and direction of flow.

The presence of ice affects channel roughness and resistance to flow in various ways. When an ice cover occurs, the open channel is more nearly comparable to a closed conduit. There is an added shear stress developed between the flowing water and the ice cover. This surface shear is much larger than the normal shear stresses developed at the air-water interface. The ice-water interface is not always smooth. In many instances, the underside of the ice is deformed so that it resembles ripples or dunes observed on the bed of sandbed channels. This may cause overall resistance to flow in the channel to be further increased.

With total or partial ice cover, the drag of ice retards flow, decreasing the average velocity and increasing the depth. Another serious effect is its influence on bank stability, in and near water structures such as docks, loading ramps, and ships. For example, the ice layer may freeze into bank stabilization materials, and when the ice breaks up, large quantities of rock and other material embedded in the ice may be floated downstream and subsequently thawed loose and dumped randomly leaving banks raw and unprotected.

2.3.3 Average Boundary Shear Stress

The shear stress at the boundary τ_o for steady uniform flow is determined by applying the conservation of mass and momentum principles to the control volume shown in Figure 2.3.6.

The conservation of mass Equation 2.2.4 is then

$$\rho y_o W V_2 - \rho y_o W V_1 = 0 \tag{2.3.23a}$$

or

$$V_1 = V_2 \tag{2.3.23b}$$

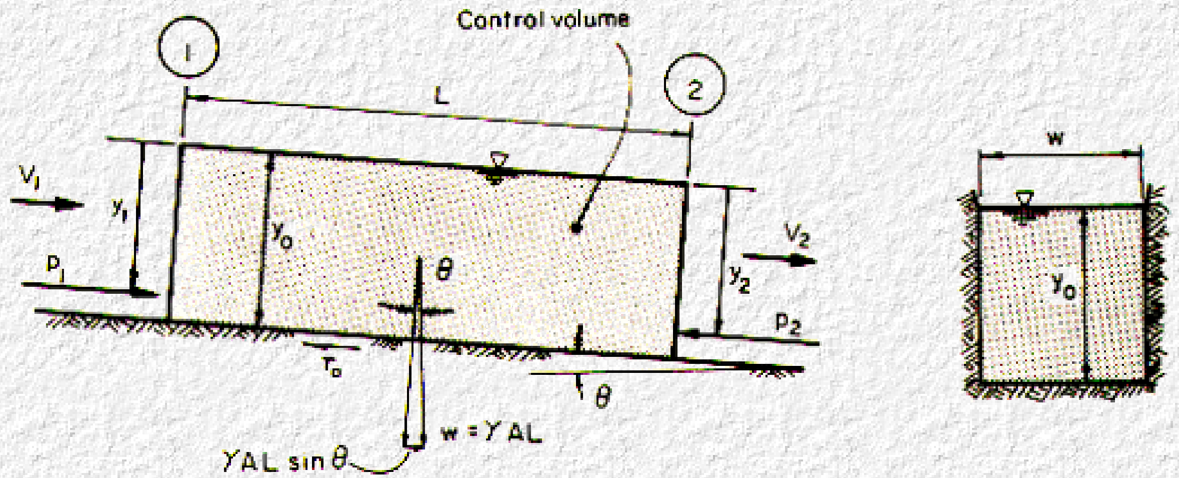


Figure 2.3.6. Control Volume for Steady Uniform Flow

The conservation of momentum in the downstream direction is described from Equation 2.2.9 with $A_1 = A_2 = Wy_o$ and $V_1 = V_2$. The pressure forces acting on the control boundary are approximated by

$$F_1 = F_2 = \frac{\gamma y_o^2 W}{2} \tag{2.3.24}$$

The downstream component of the body force γAL (equal to the weight of fluid in the control volume) in the X direction is

$$F_b = \gamma AL \sin\theta \tag{2.3.25}$$

where θ is the slope angle of the channel bed. The average boundary shear stress is τ_o acting on the wetted perimeter P. The shear force F_s in the x-direction is

$$F_s = \tau_o PL \tag{2.3.26}$$

With the above expressions for the components, the statement of conservation of linear momentum becomes

$$\rho\beta Wy_oV^2 - \rho\beta Wy_oV^2 = \gamma AL\sin\theta + \frac{\gamma y_o^2 W}{2} - \frac{\gamma y_o^2 W}{2} - \tau_o PL \qquad (2.3.27)$$

which reduces to

$$\tau_o = \gamma \frac{A}{P}\sin\theta \qquad (2.3.28)$$

The term A/P is the hydraulic radius R. If the channel slope angle is small,

$$\sin\theta \simeq S_o \qquad (2.3.29)$$

and for steady uniform flow the average shear stress on the boundary is

$$\tau_o = \gamma R S_o \qquad (2.3.30)$$

If the flow is gradually varied nonuniform flow, the average boundary shear stress is

$$\tau_o = \gamma R S_f \qquad (2.3.31)$$

where S_f is the slope of the energy grade line.

2.3.4 Energy and Momentum Coefficients for Rivers

In open channel flow problems it is common to assume that the energy coefficient α and the momentum coefficient β are unity. What are values of α and β for river channels?

From Equation 2.2.26 and Equation 2.2.6

$$\alpha = \frac{1}{V^3 A}\int_A v^3 dA \qquad (2.2.26)$$

and

$$\beta = \frac{1}{V^2 A} \int_A v^2 dA \tag{2.2.6}$$

The velocity distribution in wide channels for turbulent flow over a rough boundary is given by Equation 2.3.15 with X = 1.0

$$\frac{v}{V_*} = 2.5 \ln\left(30.2\, y / k_s\right) \tag{2.3.15}$$

The average velocity in the vertical is

$$V = \frac{1}{y_o} \int_0^{y_o} v\, dy \simeq \frac{2.5 V_*}{y_o - y'} \int_{y'}^{y_o} \ln(y / y')\, dy \tag{2.3.32}$$

Here, the upper limit of integration is y_o, the depth of flow and the lower limit is

$$y' = \frac{k_s}{30.2}$$

the value of y for which Equation 2.3.15 gives a zero velocity. The integration of Equation 2.3.32 yields

$$\frac{V}{V_*} = 2.5 \left\{ \frac{y_o}{y_o - y'} \left(\ln\left(\frac{y_o}{y'} \right) - 1 \right) \right\} \tag{2.3.33}$$

For a vertical section of unit width, the momentum coefficient β ' is

$$\beta' = \frac{1}{V^2 (y_o - y')} \int_{y'}^{y_o} v^2\, dy \tag{2.3.34}$$

If we substitute Equation 2.3.15 and Equation 2.3.33 into Equation 2.3.34 and integrate, the result is the expression

$$\beta' = \frac{1}{\left(\ln 11.11\frac{y_o}{k_s}\right)^2}\left(\frac{y_o}{y_o - y'}\right)\left\{\left(\ln\frac{y_o}{y'}\right)^2 - 2\ln\left(\frac{y_o}{y'}\right) + 2 - \frac{2y'}{y_o}\right\} \quad (2.3.35)$$

Similarly, the energy coefficient for a vertical section unit width is

$$\alpha' = \frac{1}{V^3(y_o - y')}\int_{y'}^{y_o} v^3\, dy \quad (2.3.36)$$

or

$$\alpha' = \frac{1}{\left(\ln 11.11\frac{y_o}{k_s}\right)^2}\left(\frac{y_o}{y_o - y'}\right)\left\{\left(\ln\frac{y_o}{y'}\right)^3 - 3\ln\left(\frac{y_o}{y'}\right)^2 + 6\ln\left(\frac{y_o}{y'}\right) - 6 + \frac{6y'}{y_o}\right\} \quad (2.3.37)$$

These equations (Equation 2.3.35 and Equation 2.3.37) are rather complex, so a graph of α' and β' vs. y_o/k_s has been prepared. The relations are shown in Figure 2.3.7.

For the entire river cross-section (shown in Figure 2.3.8) Equation 2.2.26 can be written

$$\alpha = \frac{1}{\left(\frac{Q}{A}\right)^3 A}\left[\int_0^W \int_0^{y_o} v^3 dy\, dw\right] \quad (2.3.38)$$

where W is the top width of the section, w is the lateral location of any vertical section, y_o is the depth of flow at location w, and v is the local velocity at the position, y, w. The total discharge is Q and the total cross-sectional area is A.

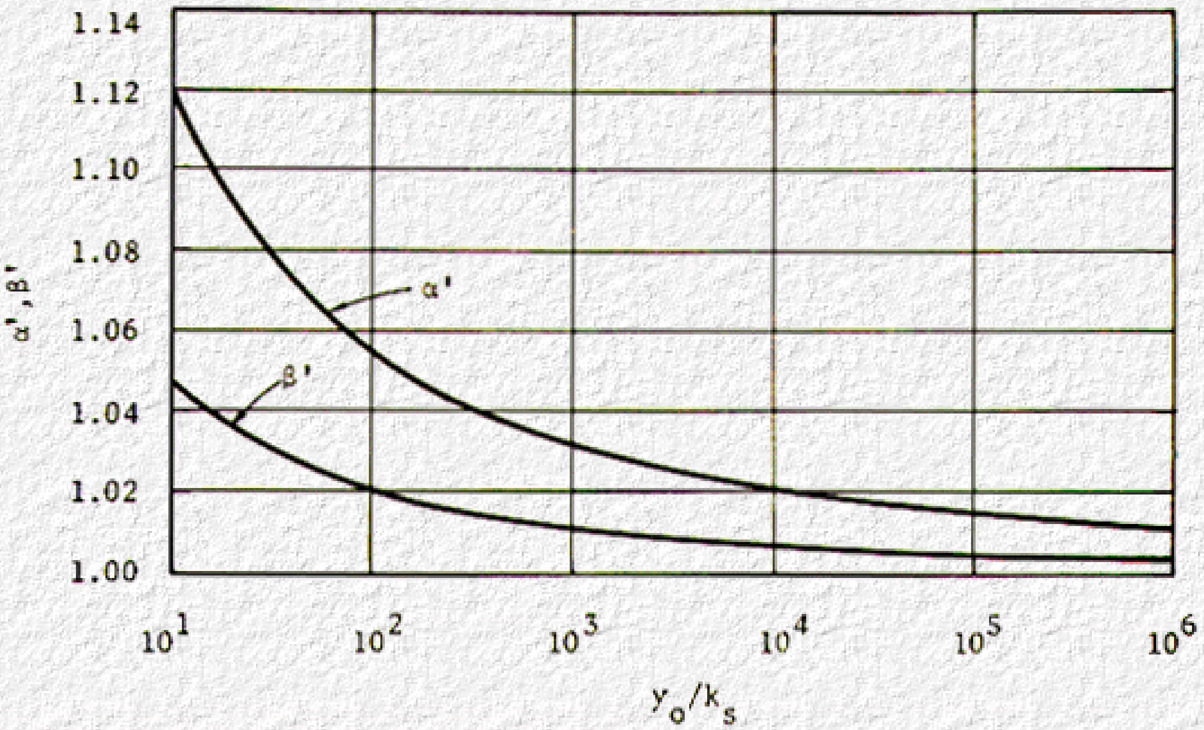


Figure 2.3.7. Energy and Momentum Coefficients for a Unit Width of River

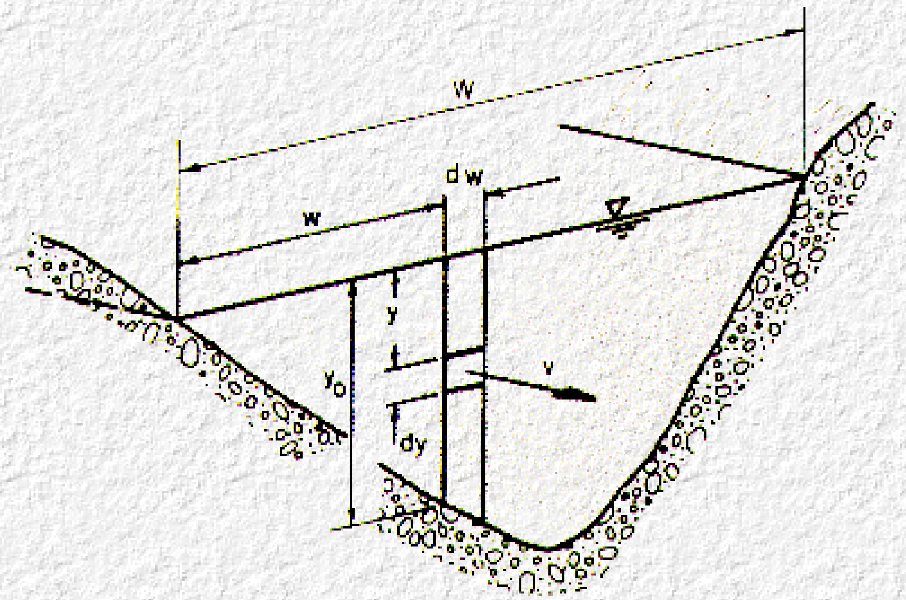


Figure 2.3.8. The River Cross-Section

The term in brackets in Equation 2.3.38 can be written

$$\int_{y'}^{y_o} v^3 dy = \alpha' V^3 (y_o - y') \qquad (2.3.39)$$

Here α' is the energy coefficient for the vertical section dw wide and y_o deep, V is the depth-averaged velocity in this vertical section and $y' = k_s/30.2$.

Now, Equation 2.3.38 can be written

$$\alpha = \frac{A^2}{Q^3} \int_0^W \alpha' V^3 (y_o - y') dw \qquad (2.3.40)$$

Except for cases of low flow in gravel bed rivers, the term y' is very small compared to y_o so

$$\alpha = \frac{A^2}{Q^3} \int_0^W \alpha' V^3 \; y_o \; dw \qquad (2.3.41)$$

The discharge at a river cross-section is determined in the field by measuring the local depth and two local velocities at each of approximately 20 vertical sections. In accordance with this general stream gaging procedure, Equation 2.3.41 should be written

$$\alpha = \frac{A^2}{Q^3} \sum_i \alpha_i' V_i^3 \; y_{oi} \; \Delta w_i$$

or

$$\alpha = \frac{A^2}{Q^3} \sum_i \alpha_i' V_i^2 \, \Delta Q_i \qquad (2.3.42)$$

Here, the subscript i refers to the i-th vertical section, and ΔQ_i is the river discharge associated with the i-th vertical or

$$\Delta Q_i = V_i y_{oi} \Delta w_i$$

In a similar manner, the expression for β is

$$\beta = \frac{A}{Q^2} \sum_i \beta_i' V_i \Delta Q_i \qquad (2.3.43)$$

Now, with Equations 2.3.42 and Equation 2.3.43, and Figure 2.3.7 we are in a position to compute α and β for any river cross-section given the discharge measurement notes. A calculation example is presented in Appendix 2 (Problem 2.1). It is important to recognize that for river flows over floodplains, the correction factors a and β can be significantly larger than α' and β'.

2.4 Unsteady Flow

Unsteady flows of interest to the designer of waterway crossings and encroachments are: (1) waves resulting from disturbances of the water surface by wind and boats; (2) waves resulting from the surface instability that exists for flows with Froude numbers close to 1.0; (3) waves resulting from flow disturbance due to change in direction of flow with Froude numbers greater than about 2.0; (4) surges or bores resulting from sudden increase or decrease in the flow by opening or closing of gates or the movement of tides on coastal streams; (5) standing waves and antidunes that occur in alluvial channel flow; and (6) flood waves resulting from the progressive movement downstream of stream runoff or gradual release from reservoirs.

Waves are an important consideration in bridge hydraulics when designing slope protection of embankments and dikes, and channel improvements. In the following paragraphs, only the basic one-dimensional analysis of waves and surges is presented. Other aspects of waves are presented in other sections.

2.4.1 Gravity Waves

The general equation for the celerity c (velocity of the water relative to the velocity of flow) of a small amplitude gravity wave (a_o << λ) is

$$C = \left\{ \frac{g\lambda}{2\pi} \tanh \frac{2\pi y_o}{\lambda} \right\}^{1/2} \qquad (2.4.1)$$

where the terms are defined in Figure 2.4.1.

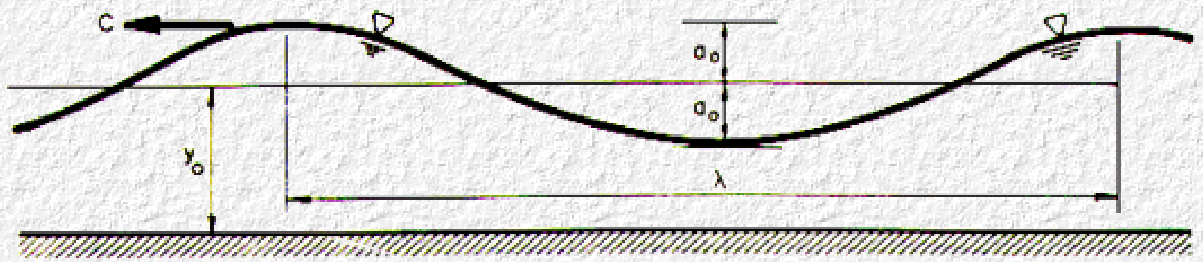


Figure 2.4.1. Definition Sketch for Small Amplitude Waves

For deep water waves (short waves) defined as

$$\frac{y_0}{\lambda} < \frac{1}{2}$$

the celerity relationship (Equation 2.4.1) reduces to

$$C = \left\{\frac{g\lambda}{2\pi}\right\}^{1/2} \qquad (2.4.2)$$

For shallow water waves (long waves)

$$\frac{y_0}{\lambda} < \frac{1}{20} \qquad (2.4.3)$$

Then Equation 2.4.1 gives

$$C = \sqrt{gy_0} \qquad (2.4.4)$$

The time of travel of one water crest to another at a given point is called the period T and can be defined from the celerity and wave length

$$c = \lambda / T \qquad (2.4.5)$$

In Equation 2.4.2, the celerity is independent of depth and depends on gravity g and wave length λ. This is the celerity of ocean waves. In Equation 2.4.4, the celerity is a function of gravity and depth which describes small amplitude waves in open channels.

These two equations apply only to small amplitude waves; that is $a_o/\lambda << 1$.

The celerity of finite amplitude shallow water waves has been determined both analytically using Bernoulli's equation and experimentally, and is given by the expression

$$C = \left\{ \frac{(y_0 + 2a_0)^2}{(y_0 + a_0)y_0} \, g y_0 \right\}^{1/2} \tag{2.4.6a}$$

When $2a_o$ is small in comparison to y_o

$$C = \left\{ 1 + \frac{2a_0}{y_0} \, g y_0 \right\}^{1/2} \tag{2.4.6b}$$

Generally as $2a_o/y_o$ approaches unity the crest develops a sharp peak and breaks.

In the above equations, c is measured relative to the fluid. If the wave is moving opposite to the flow then, when c > V, the waves move upstream; when c = V, the wave is stationary; and when c < V, the wave moves downstream. When V = c for small amplitude flow,

$$V = c = \sqrt{g y_0} \tag{2.4.7}$$

Thus, the ratio of the flow velocity to the celerity of a shallow water wave of small amplitude defines the Froude number

$$Fr = \frac{V}{\sqrt{g y_0}} \tag{2.4.8}$$

When Fr < 1 (subcritical or tranquil), a small amplitude wave moves upstream. When Fr > 1 (supercritical or rapid flow), a small amplitude wave moves downstream and when Fr = 1 (critical flow), a small amplitude wave is stationary. The fact that waves or surges cannot move upstream when the Froude number is equal to or greater than 1.0 is important to remember when determining when the stage-discharge relation at a cross-section can be affected by downstream conditions.

2.4.2 Surges

A surge is a rapid increase in the depth of flow. A surge may result from sudden release of water from a dam, or from an incoming tide. If the ratio of wave height $2a_o$ to the depth y_o is less than unity, the surge has an undulating wave form. If $2a_o/y_o$ is greater than one, the first wave breaks and produces a discontinuous surface. The breaking wave dissipates energy and the previous equations for wave celerity are invalid. However, by applying the momentum and continuity equations for a control volume encompassing the surge shown in Figure 2.4.2a, the equation for the celerity of a surge can be derived as

$$c = \left\{ g y_1 \left[\frac{1}{2} \frac{y_2}{y_1} \left(\frac{y_2}{y_1} + 1 \right) \right] \right\}^{1/2} \tag{2.4.9}$$

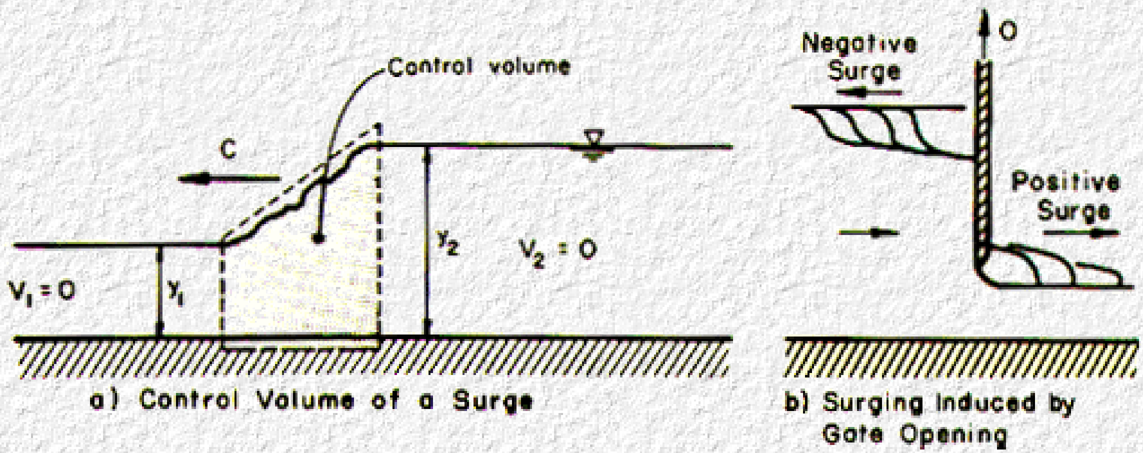


Figure 2.4.2. Sketch of Positive and Negative Surges

Equation 2.4.9 gives the velocity of a surge as it moves upstream as the result of a sudden total or partial closure of gates, or of an incoming tide, or of a surge that moves downstream as the result of a sudden opening of a gate. The lifting of a gate in a channel sketched in Figure 2.4.2b not only causes a positive surge to move downstream, it also causes a negative surge to move upstream. Equation 2.4.9 is approximately correct for the celerity of the negative surge if the height of the surge is small compared to the depth. As it moves upstream a negative surge quickly flattens out.

2.4.3 Hydraulic Jump

When the flow velocity V_1 is rapid or supercritical the surge dissipates energy through a moving hydraulic jump. When V_1 equals the celerity c of the surge the jump is stationary and Equation 2.4.9 is the equation for a hydraulic jump. Equation 2.4.9 can be rearranged to the form

$$\frac{V_1}{\sqrt{g y_1}} = Fr_1 = \left\{ \frac{1}{2} \frac{y_2}{y_1} \left(\frac{y_2}{y_1} + 1 \right) \right\}^{1/2} \tag{2.4.10a}$$

or

$$\frac{y_2}{y_1} = \frac{1}{2}\left\{\left(1 + 8Fr_1^2\right)^{1/2} - 1\right\} \qquad (2.4.10b)$$

The corresponding energy loss in a hydraulic jump is the difference between the two specific energies. It can be shown that this head loss is

$$h_L = \frac{(y_2 - y_1)^3}{4y_1y_2} \qquad (2.4.11)$$

Equation 2.4.11 has been experimentally verified along with the dependence of the jump length L_j and energy dissipation (head loss h_L) on the Froude number of the approaching flow. The results of these experiments are given in Figure 2.4.3.

When the Froude number for rapid flow is less than two, an undulating jump with large surface waves is produced. The waves are propagated for a considerable distance downstream. In addition, when the Froude number of the approaching flow is less than three, the energy dissipation of the jump is not large and jets of high velocity flow can exist for some distance downstream of the jump. These waves and jets can cause erosion a considerable distance downstream of the jump. For larger values of the Froude number, the rate of energy dissipation in the jump is very large and Figure 2.4.3 is recommended.

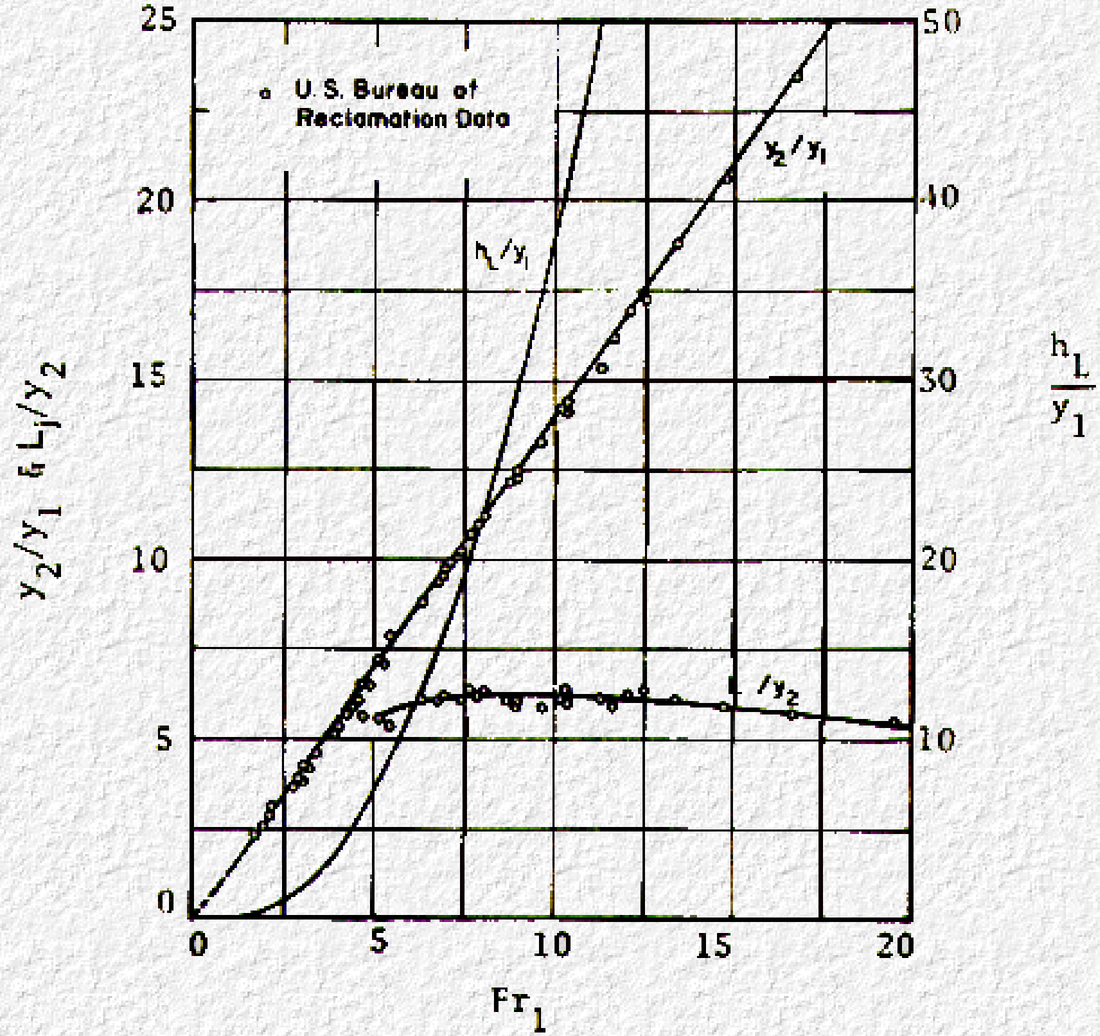


Figure 2.4.3. Hydraulic Jump Characteristics as a Function of the Upstream Froude Number

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2.4.4 Roll Waves

Under certain conditions on steep slopes, surges of an intermittent nature may occur which are called roll waves or slug flow, see Figure 2.4.4. Such flow is not at all uncommon with harmless thin sheets of flow on sloping sidewalks, for example. When these roll waves occur in large open channels, however, they may cause considerable damage, or force the operation of the channel at inefficient discharges in order to prevent damage.

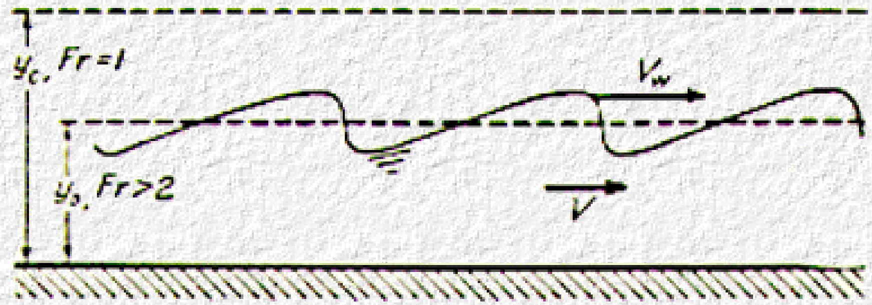

Figure 2.4.4. Roll Waves or Slug Flow

Roll waves consist of waves superposed over the normal flow in an open channel. They travel at velocities greater than the normal flow and grow in size as they progress downstream.

There is no simple criterion for determining the size of roll waves, since their size depends upon the magnitude of the discharge, the type of flow (laminar or turbulent), the roughness and slope of the channel, the length of the channel, and the nature and frequency of the initial disturbances which cause the waves to form. However, a necessary condition required to generate instability of the free surface and induce the formation of roll waves in turbulent flows when Chezy's equation is applicable is

$$Fr = \frac{V}{\sqrt{gy_o}} > 2 \qquad (2.4.12a)$$

which can be expressed in alternate form for a wide channel as

$$S \geq 4\frac{g}{C^2} \qquad (2.4.12b)$$

for turbulent flow with a rough boundary in which y_o is the normal depth, S is the slope of the channel, and C is the Chezy discharge coefficient.

When the flow in a wide channel is turbulent with a smooth boundary, roll waves can form if

$$Fr \geq 1.5 \qquad (2.4.13a)$$

$$S \geq 2.25\frac{g}{C^2} \qquad (2.4.13b)$$

and when the flow in a wide channel is laminar, roll waves can form if

$$Fr \geq 0.5 \qquad (2.4.14)$$

These conditions indicate that, for turbulent flow in a wide channel with a rough boundary, roll waves can occur when the flow velocity is greater than twice the celerity of a wave (that is, the Froude number is greater than 2), or when the slope is four times as great as the slope required for critical depth. They can also form for turbulent flow in a wide channel with a smooth boundary if the velocity of flow is greater than 1.5 times the celerity of a wave, or the slope is 2.25 times the slope required for critical depth. By way of contrast, roll waves can form in laminar flow in a wide channel if the velocity is half the celerity of a gravity wave; in other words, the flow may never pass through critical flow (Fr = 1.0).

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Open Channel Flow

Part II

Go to Chapter 3

2.5 Steady Rapidly Varying Flow

2.5.1 Introduction

Steady flow through relatively short transitions where the flow is uniform before and after the transition can be analyzed using the Bernoulli equation. Energy loss due to friction may be neglected, at least as a first approximation. Refinement of the analysis can be made in a second step by including friction loss. For example, the water surface elevation through a transition is determined using the Bernoulli equation and then modified by determining the friction loss effects on velocity and depth in short reaches through the transition. Energy losses resulting from flow separation cannot be neglected, and transitions where separation may occur need special treatment which may include model studies. Contracting flows (converging streamlines) are less susceptible to separation than for expanding flows. Also, any time a transition changes velocity and depth such that the Froude number approaches unity, problems such as waves, blockage, or choking of the flow may occur. If the approaching flow is supercritical, a hydraulic jump may result. Transitions for supercritical flow are discussed in Section 2.6.5.

Transitions are used to contract or expand a channel width (Figure 2.5.1a); to increase or decrease bottom elevation (Figure 2.5.1b); or to change both the width and bottom elevation. The first step in the analysis is to use the Bernoulli equation (neglecting any head loss resulting from friction or separation) to determine the depth and velocity changes of the flow through the transition. Further refinement depends on importance of freeboard, whether flow is supercritical or approaching critical conditions.

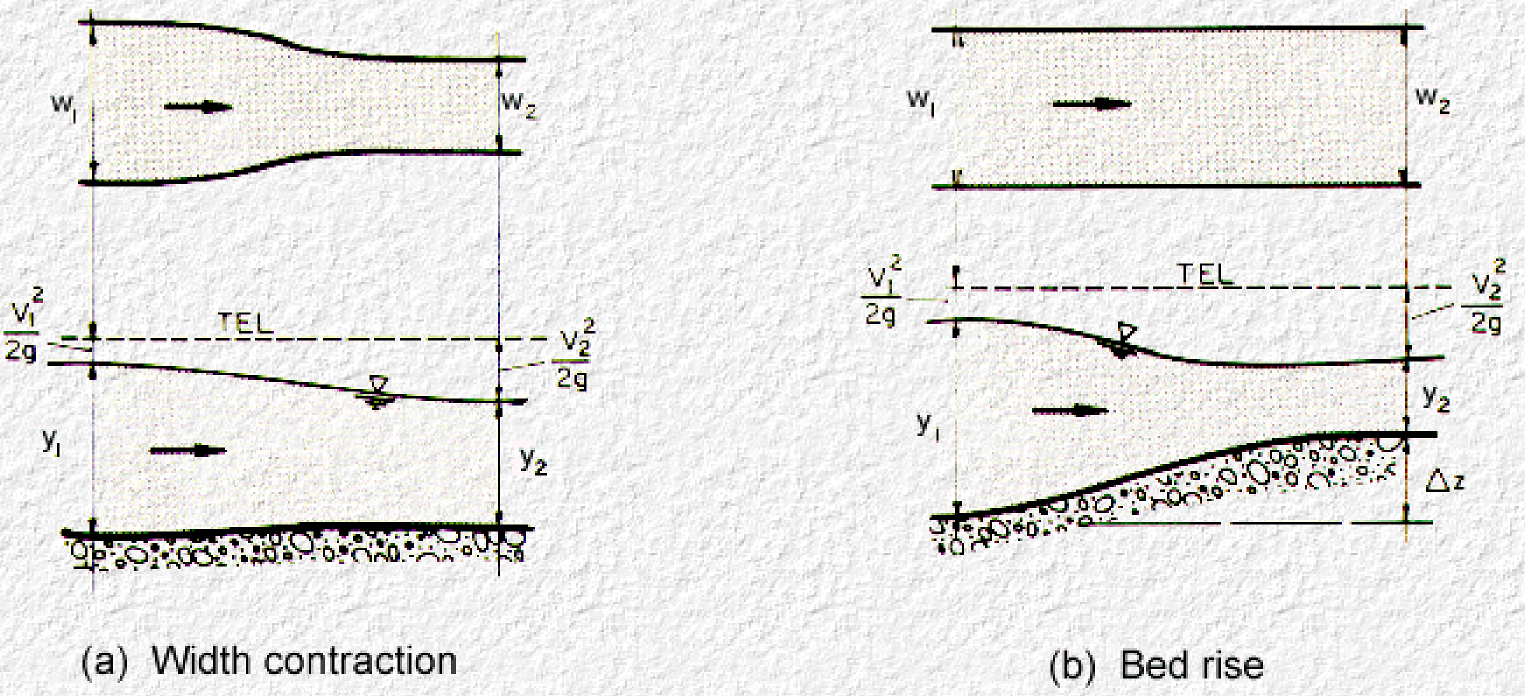

Figure 2.5.1. Transitions in Open Channel Flow

The Bernoulli equation for flow in Figure 2.5.1b is

$$\frac{V_1^2}{2g} + y_1 = \frac{V_2^2}{2g} + y_2 + \Delta z \qquad (2.5.1)$$

or

$$H_1 = H_2 + \Delta z \qquad (2.5.2)$$

where

$$H = \frac{V^2}{2g} + y \qquad (2.5.3)$$

The term H is called the specific head, and is the height of the total head above the channel bed.

2.5.2 Specific Head Diagram

For simplicity, the following specific head analysis is done on a unit width of channel so that Equation 2.5.3 becomes

$$H = \frac{q^2}{2gy^2} + y \qquad (2.5.4)$$

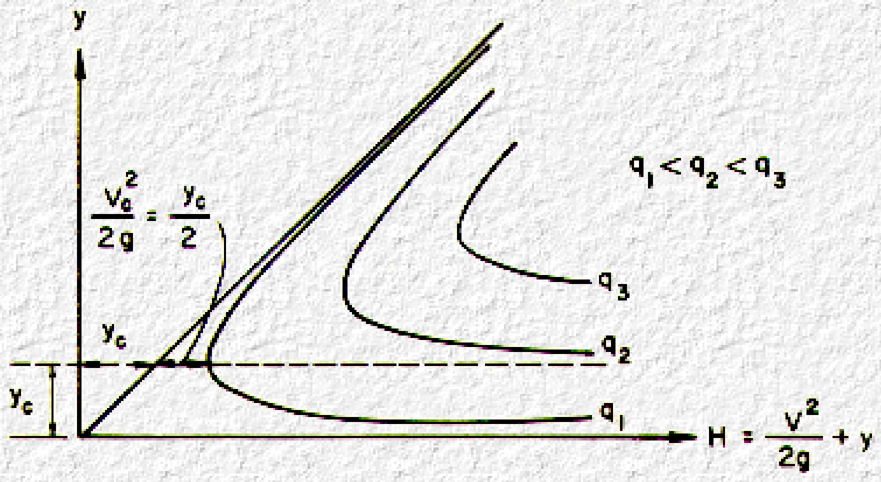

Figure 2.5.2. Specific Head Diagram

For a given q, Equation 2.5.4 can be solved for various values of H and y. When y is plotted as a function of H, Figure 2.5.2 is obtained. There are two possible depths called alternate depths for any H larger than a specific minimum. Thus, for specific head larger than the minimum, the flow may have a large depth with small velocity or small depth with large velocity. Flow cannot occur with specific energy less than the minimum. The single depth of flow at the minimum specific head is called the critical depth y_c and the corresponding velocity, the critical velocity $V_c = q/y_c$. To determine y_c the derivative of H with respect to y is set equal to 0.

$$\frac{dH}{dy} = 1 - \frac{q^2}{gy^3} = 0 \qquad (2.5.5)$$

and

$$q = \left(gy_c^3\right)^{1/2} \qquad (2.5.6)$$

or

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = 2\frac{V_c^2}{2g} \qquad (2.5.7)$$

Note that

$$V_c^2 = y_c g \qquad (2.5.8)$$

or

$$\frac{V_c}{\sqrt{gy_c}} = 1 \qquad (2.5.9)$$

But

$$\frac{V}{\sqrt{gy}} = Fr \qquad (2.5.10)$$

Also

$$H_{min} = \frac{V_c^2}{2g} + y_c = \frac{3}{2} y_c \qquad (2.5.11)$$

Thus, flow at minimum specific energy has a Froude number equal to one. Flows with velocities larger than critical ($Fr > 1$) are called rapid or supercritical and flow with velocities smaller than critical ($Fr < 1$) are called tranquil or subcritical. These flow conditions are illustrated in Figure 2.5.3, where a rise in the bed causes a decrease in depth when the flow is tranquil and an increase in depth when the flow is rapid. Furthermore there is a maximum rise in the bed for a given H_1 where the given rate of flow is physically possible. If the rise in the bed is increased beyond Δz_{max} for H_{min} then the approaching flow depth y_1 would have to increase (increasing H) or the flow would have to be decreased. Thus, for a given flow in a channel, a rise in the bed level can occur up to a Δz_{max} without causing backwater.

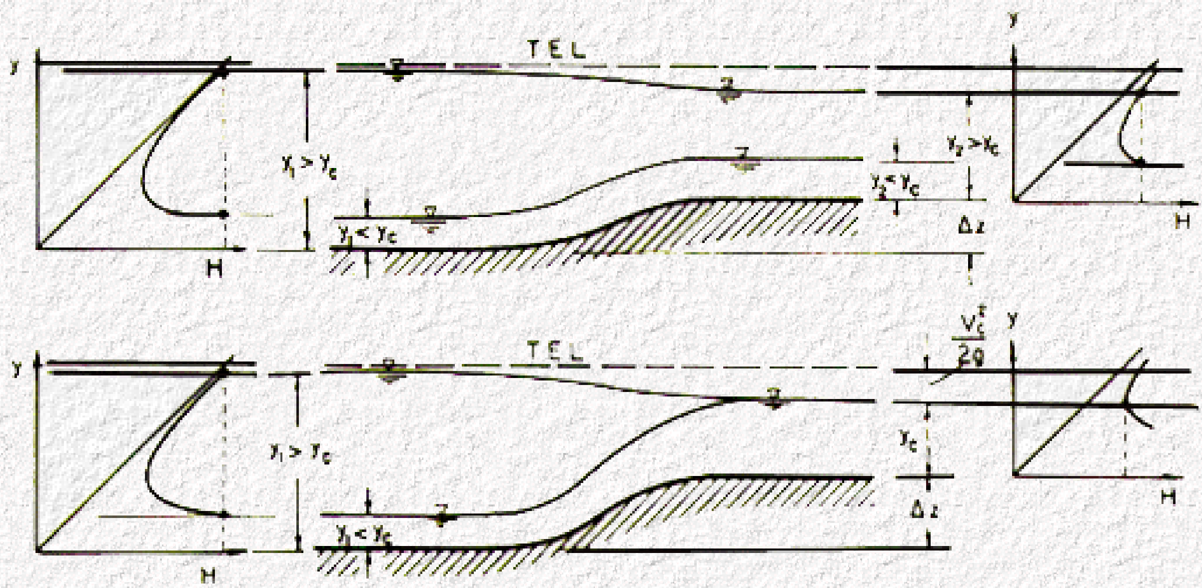

Figure 2.5.3. Changes in Water Surface Resulting from an Increase in Bed Elevation

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2.5.3 Discharge Diagram

For a constant H, Equation 2.5.4 can be solved for y as a function of q. By plotting y as a function of q, Figure 2.5.4 is obtained and for any discharge smaller than a specific maximum, two depths of flow are possible.

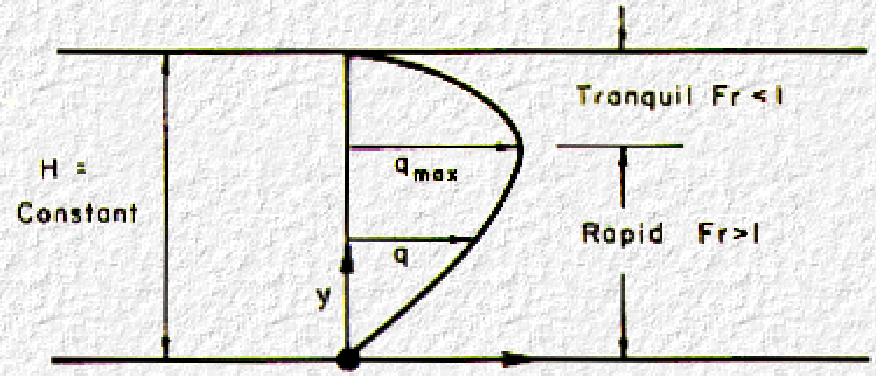


Figure 2.5.4. Specific Discharge Diagram

To determine the value of y for q_{max} Equation 2.5.4 is rearranged to obtain

$$q = y\sqrt{2g(h - y)} \tag{2.5.12}$$

The differential with respect to y is set equal to zero.

$$\frac{dq}{dy} = 0 = \frac{q}{2}\,\frac{(2H - 3y)}{(H - y)^{1/2}} \tag{2.5.13}$$

from which

$$y_c = \frac{2}{3}H = \frac{2V_c^2}{2g} \tag{2.5.14}$$

or

$$V_c = \sqrt{gy_c} \tag{2.5.15}$$

Thus for maximum discharge at constant H, the Froude number is 1.0, and the flow is critical. From this

$$y_c = \frac{2}{3}H = \frac{2V_c^2}{2g} = \left(\frac{q^2\,max}{g}\right)^{1/3} \tag{2.5.16}$$

For critical conditions, the Froude number is 1.0, the discharge is a maximum for a given specific head and the specific head is a minimum for a given discharge.

Flow conditions for constant specific head for a width contraction are illustrated in Figure 2.5.5 assuming no geometrical effects such as eccentricity, skew, piers, scour and expansion. The contraction causes a decrease in flow depth when the flow is tranquil and an increase when the flow is rapid. The maximum possible contraction without causing backwater effects occurs when the Froude number is one, the discharge per foot of width q is a maximum, and y_c is 2H/3. A further decrease in width will cause backwater. That is, an increase in depth upstream will occur to produce a larger specific energy and increase y_c in order to get the flow through the decreased width.

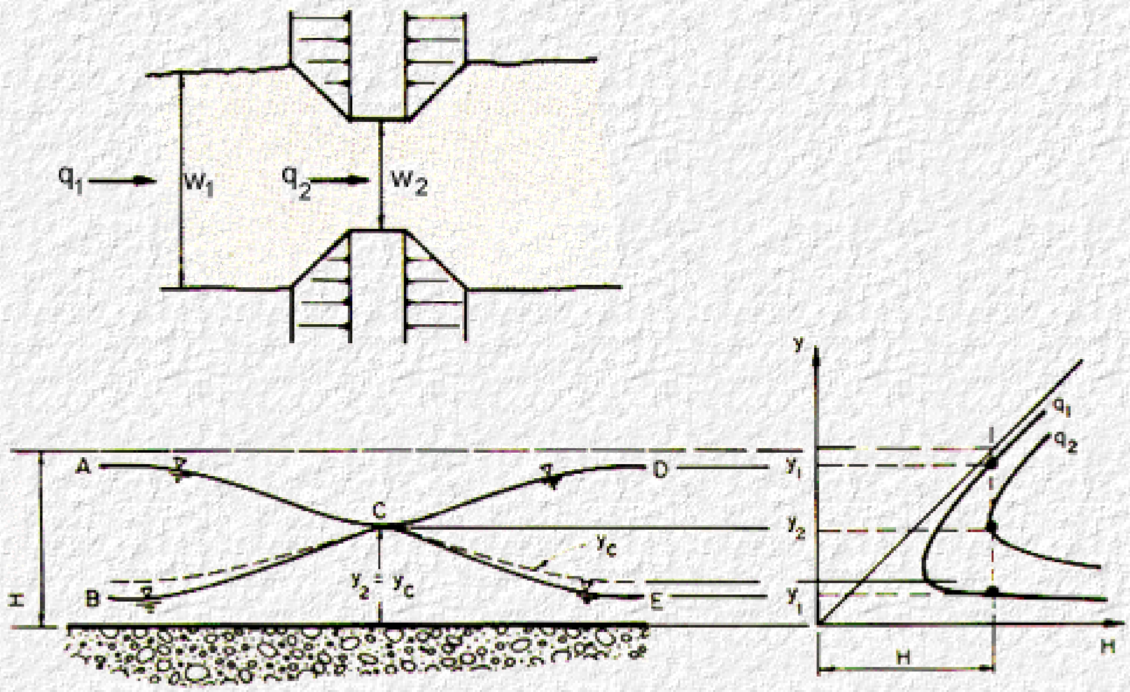


Figure 2.5.5. Change in Water Surface Elevation Resulting from a Change in Width

The flow in Figure 2.5.5 can go from point A to C and then either back to D or down to E depending on the downstream boundary conditions. An increase in slope of the bed downstream from C and no separation would allow the flow to follow the line A to C to E. Similarly the flow can go from B to C and back to E or up to D depending on boundary conditions. Figure 2.5.5 is drawn with the side boundary forming a smooth streamline. If the contraction were due to bridge abutments, the upstream flow would follow a natural streamline to a vena contracta, but then downstream, the flow would probably separate. Tranquil approach flow could follow line A-C but the downstream flow probably would not follow either line C-D or C-E but would have an undulating hydraulic jump. There would be interaction of the flow in the separation zone and considerable energy would be lost. If the slope downstream of the abutments was the same as upstream, then the flow could not be sustained with this amount of energy loss. Backwater would occur, increasing the depth in the constriction and upstream, until the flow could go through the constriction and establish uniform flow downstream.

2.6 Flow in Bends and Transitions

2.6.1 Types of Bends

Two principal types of bends are deepened or entrenched bends and meandering surface bends. The first type includes those in which the river bends follow the curves of the valley so that each river bend includes a promontory of the parent plateau. The second type includes bends which are formed only by the river on a flat, alluvium covered valley floor, and where the slopes of the valley are not involved in the formations of such bends. This division of bends is correct and sufficiently definite with respect to external forms of the relief and the process of formation and development of bends. It is, however, incomplete from the standpoint of the work of the river and of the physical nature of this phenomenon. Both of the morphological types of bends can be put into one category--the category of freely meandering channels, i.e., meandering determined only by the interaction of the stream and the bed material. Such meandering, not disturbed by the influence of external factors, proceeds at an approximately equal rate along the length of the river.

Under natural conditions, a third type of bend is often encountered. This bend occurs when the stream impinging on a practically non-eroding parent bank forms a forced curve which is gradually transformed into a river bend of a more constricted shape. In all cases, the effect of the character (density) of the bank material is important and, to a certain degree, determines the radius of curvature of the channel in a free bend. The radius of curvature increases with the density of the material. Considering both the action of the stream and the interaction between the stream and the channel, as well as the general laws of their formation, one can distinguish the following three types of bends of a natural river channel:

1. Free bends - Both banks are composed of alluvial floodplain material which is usually quite mobile; the free bend corresponds to the common concept of a surface bend;
2. Limited bends - The banks of the stream are composed of consolidated parent material which limits the lateral erosion by the stream. Limited bends are entrenched bends; and
3. Forced bends - The stream impinges onto an almost straight parent bank at a large angle (60° to 90°).

A typical feature of bends is a close relationship between the type of stream bend and the radius of curvature. The forced bend has the smallest radius of curvature. Next in size are the radii of free bends. The limited bends have the greatest radii. The average values of the ratios of the radii of curvature to the width of the stream at bankfull stage for the three types of bends are: (1) Free Bends 4.5 to 5.0; (2) Limited Bends 7.0 to 8.0; and (3) Forced Bends 2.5 to 3.0.

A second characteristic feature of bends is the distribution of depths along the length of the bend. In free bends and limited bends, the depth gradually increases and the maximum depth is found some distance below the apex of the bend. In the forced bend, the depth sharply increases at the beginning of the bend and then gradually diminishes. In forced bends the greatest depth is located in the middle third of the bend, where there appears to be a concentrated deep scour.

2.6.2 Velocity Distribution in Bends

The transverse velocities in bends result from an imbalance of radial pressures on a particle of fluid traveling around the bend. In Figure 2.6.1, a cross section through a typical bend is shown. The radial forces acting on the shaded control volume are the centrifugal force mv^2/r in which r is the radius of curvature and the differential hydrostatic force γdz caused by the superelevation of the water surface dz. As shown in Figure 2.6.1a, the centrifugal force is greater near the surface where the fluid velocity v is greater and less at the bed where v is small. The differential hydrostatic force is uniform throughout the depth of the control volume. As shown in Figure 2.6.1b, the sum of the centrifugal and excess hydrostatic forces varies with depth and can cause a lateral velocity component. The magnitude of the transverse velocity is dependent on the radius of curvature and on the proximity of the banks. In the immediate vicinity of the banks, there can be no lateral velocity if the river is narrow and deep, and this bank constraint to the transverse velocity field is felt throughout the cross section.

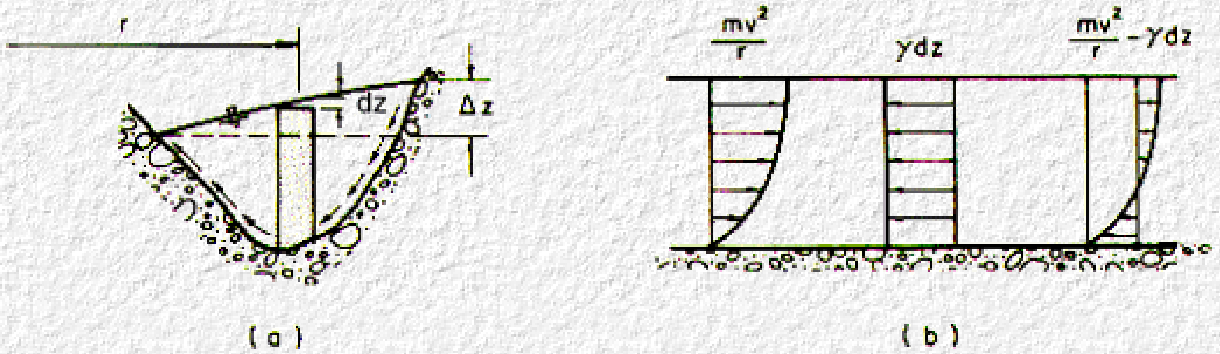


Figure 2.6.1. Schematic Representation of Transverse Currents in a Channel Bed

Several studies have been made of the transverse velocity field in the cross section of an open channel. The equation for transverse velocity developed by Rozovskii (1957) is

$$v_r = \frac{1}{\kappa^2} V \frac{y}{r}\left[F_1(\eta) - \frac{\sqrt{g}}{\kappa C} F_2(\eta)\right] \qquad (2.6.1)$$

in which

v_r = the lateral (radial) velocity corresponding to a flow depth y

V = the average longitudinal velocity

C = the Chezy coefficient

$\eta = y/y_{max}$ = the relative depth

κ = the von Karman velocity coefficient

r = radius of curvature

The functions $F_1(\eta)$ and $F_2(\eta)$ can be determined from Figure 2.6.2

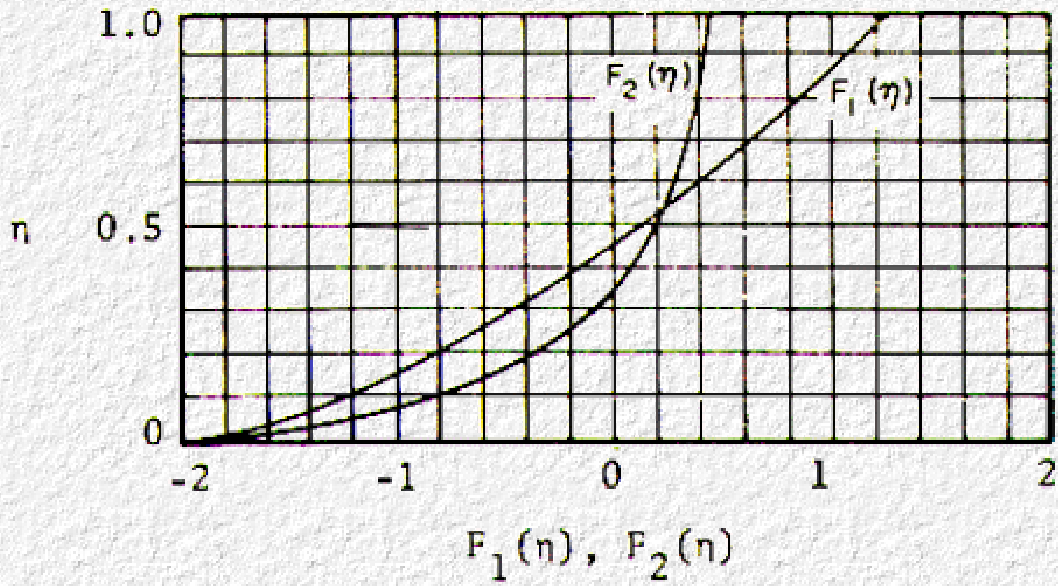


Figure 2.6.2. Graph of Functions $F_1(\eta)$ and $F_2(\eta)$

A comparison of the predicted (Equation 2.6.1) and observed transverse velocity distributions for a river bend yields fairly good results in gradual bends. For the more irregular sections the results are less impressive. Problem 2.8 in Appendix 2 gives an example of these calculations.

The usual way to describe the longitudinal velocity distribution in alluvial channels is by actual measurements. In this way, accurate knowledge of the various velocity components in the cross section can be obtained.

In prismatic channels with rigid beds, it is possible however to compute the velocity field in the bends. At any vertical in the bend, the variation of longitudinal velocity with respect to depth can be described by the von Karman velocity relation (Equation 2.3.15).

$$\frac{v}{V_*} = \frac{2.303}{\kappa} \log \left\{ 30.2 \frac{X_y}{k_s} \right\} \qquad (2.3.15)$$

where

v = the velocity at depth y

V_* = the shear velocity

k_s = the height of the roughness elements taken as D_{65} of the bed material

κ = the von Karman velocity coefficient

X = Einstein's multiplication factor (Figure 2.3.5)

Extending this concept, if one can describe the longitudinal velocity distribution at several verticals in a cross section, the variation of the longitudinal velocity over the width of the stream can be determined in the bend.

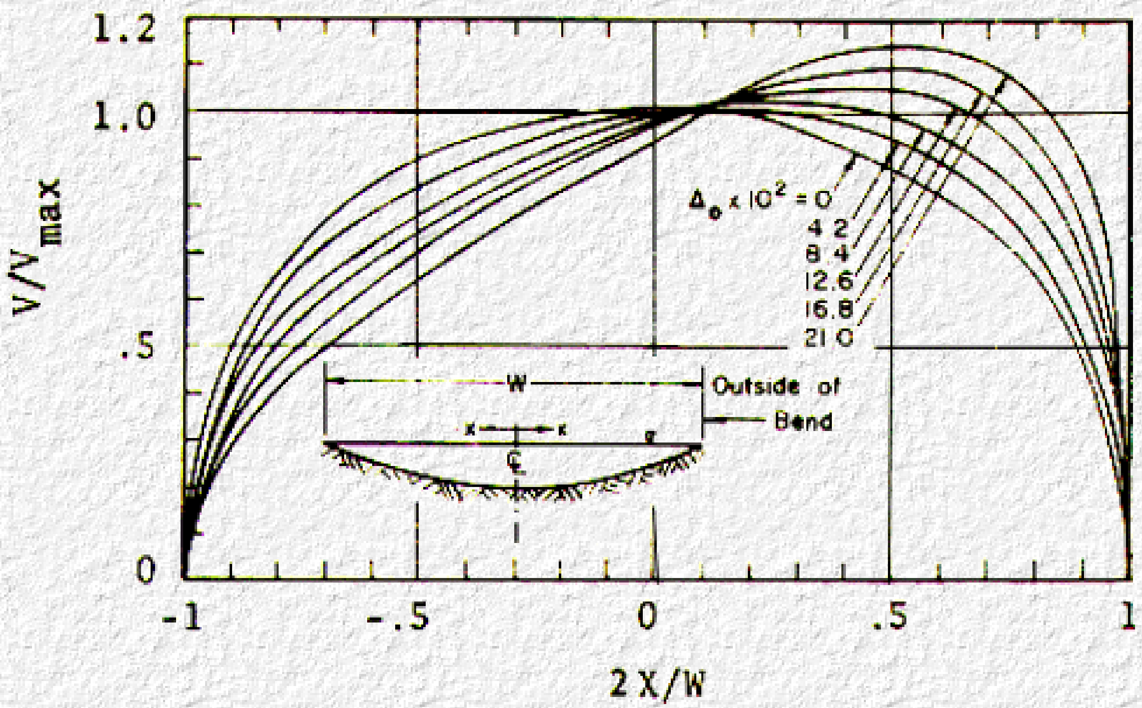


Figure 2.6.3. Lateral Distribution of Longitudinal Velocity

For a gentle bend of a parabolic cross section, Figure 2.6.3 shows the curves for velocities across the width. In Figure 2.6.3, V is the depth-averaged velocity in any vertical, and V_{max} is the maximum velocity in the straight channel. Define the dimensionless bend angle as:

$$\Delta_o = 0.42\phi \frac{y_{max}}{W} \frac{\sqrt{g}}{C} \quad (2.6.2)$$

where ϕ is the angle of the bend in degrees and Δ_o the dimensionless bend angle. The vertical distribution of velocity in the straight reach is assumed to follow the form

$$\frac{V}{V_{mac}} = \left(\frac{y_x}{y_{max}}\right)^{0.4} \quad (2.6.3)$$

The V values for sections within a bend are referenced to V_{max} in the straight reach. The depth y_x across the width of the channel is assumed to vary as

$$\frac{y_x}{y_{max}} = 1 - \left(\frac{2x}{W}\right)^2 \quad (2.6.4)$$

Longitudinal velocities in natural river bends are similar to those shown in Figure 2.6.3. The information in Figure 2.6.3 can be readily used in rivers when their cross-sections in bends are nearly parabolic.

2.6.3 Subcritical Flow In Bends

Because of the change in flow direction which results in centrifugal forces, there is a superelevation of the water surface in river bends. The water surface is higher at the concave bank than at the convex bank. The resulting transverse slope can be evaluated quantitatively. Using cylindrical coordinates (Figure 2.6.4), the differential pressure in the radial direction arises from the radial acceleration or

$$\frac{1}{\rho}\frac{\partial p}{\partial r} = \frac{V_\theta^2}{r} \tag{2.6.5}$$

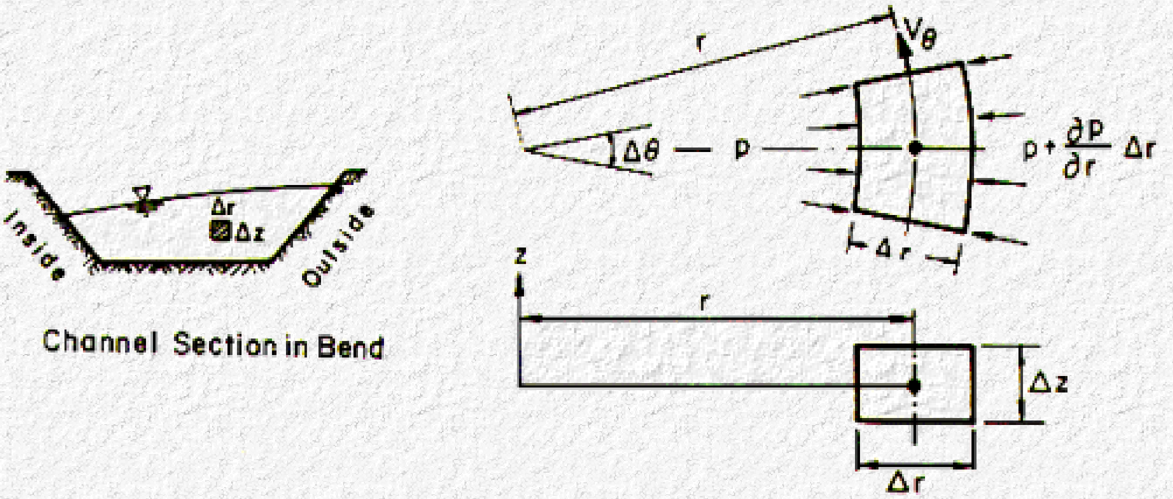


Figure 2.6.4. Definition Sketch of Flow around a Bend

To calculate the total superelevation between the outer and inner bend two assumptions are made: (1) The radial and vertical velocities are small compared to the tangential velocities such that $V_\theta \approx V$; and (2) The pressure distribution in the bend is hydrostatic, i.e., $p = \gamma y$.

Then

$$\Delta Z = \frac{1}{g}\int_{r_i}^{r_o} \frac{V^2}{r}\,dr \tag{2.6.6}$$

To solve Equation 2.6.6, the transverse velocity distribution along the radius of the bend must be known or assumed. The results obtained assuming various velocity distributions follow.

Woodward (1920) assumed V equal to the average velocity Q/A and r equal to the radius to the center of the stream r_c, and obtained

$$\Delta Z = \frac{1}{g}\int_{r_i}^{r_o} \frac{V^2}{r_c}\,dr \tag{2.6.7}$$

or

$$\Delta Z = z_o - z_i = \frac{V^2}{gr_c}(r_o - r_i) \quad , \tag{2.6.8}$$

in which z_i and r_i are the water surface elevation and the radius at the inside of the bend, and z_o and r_o are the water surface elevation and the radius at the outside of the bend.

By assuming the velocity distribution to approximate that of a free vortex ($V_\theta = C_1/r$), Shukry (1950) obtained

$$\Delta Z = \frac{1}{g}\int_{r_i}^{r_o} \frac{C_1^2}{r^3}\,dr = \frac{C_1^2}{2g}\left\{\frac{1}{r_i^2} - \frac{1}{r_o^2}\right\} \tag{2.6.9}$$

in which $C_1 = rV$ is the free vortex constant. By assuming the flow depth of flow upstream of the bend equal to the average depth in the bend, Ippen and Drinker (1962) reduced Equation 2.6.9 to

$$\Delta Z = \frac{V^2}{2g}\frac{2W}{r_c}\left\{\frac{1}{1-\left(\frac{W}{2r_c}\right)^2}\right\} \tag{2.6.10}$$

For situations where high velocities occur near the outer bank of the channel, a forced vortex may approximate the flow pattern. With this assumption and assuming a constant average specific head, Ippen and Drinker (1962) obtained

$$\Delta Z = \frac{V^2}{2g}\frac{2W}{r_c}\left[\frac{1}{1+\frac{W^2}{12r_c^2}}\right] \tag{2.6.11}$$

By assuming that the maximum velocities are close to the centerline of the channel in the bend and that the flow pattern inward and outward from the centerline can be represented as forced and free vortices, respectively, then:

$$\Delta Z = \frac{1}{g}\int_{r_i}^{r_c} \frac{C_2^2 r^2}{r}\,dr + \frac{1}{g}\int_{r_i}^{r_o} \frac{C_1^2}{r^3}\,dr, \tag{2.6.12}$$

and when $r = r_c$, $V = V_{max}$

Therefore, $C_2 = \frac{V_{max}}{r_c}$ and $C_1 = V_{max}\, r_c$

and Equation 2.6.12 becomes:

$$\Delta Z = \frac{V_{max}^2}{2g}\left\{2 - \left(\frac{r_i}{r_c}\right)^2 - \left(\frac{r_c}{r_o}\right)\right\}^2 \qquad (2.6.13)$$

The differences in superelevation that are obtained by using the different equations are small, and in alluvial channels the resulting erosion of the concave bank and deposition on the convex bank leads to further error in computing superelevation. Therefore, it is recommended that Equation 2.6.8 be used to compute superelevation in alluvial channels. For lined canals with strong curvature, superelevation should be computed using Equation 2.6.10 or Equation 2.6.13.

An example showing how to calculate superelevation in bends from velocity measurements is presented in Appendix 2 (Problem 2.2). The example also compares the various approximate equations included in this section.

2.6.4 Supercritical Flow In Bends

Rapid flow or supercritical flow in a curved prismatic channel produces cross wave disturbance patterns which persist for long distances in a downstream direction. These disturbance patterns are the result of non-equilibrium conditions which persist because the disturbances cannot propagate upstream or even propagate directly across the stream. Therefore, the turning effect of the walls is not felt on all filaments of the flow at the same time and the equilibrium of the flow is destroyed. The waves produced form a series of troughs and crests in the water surface along the channel walls.

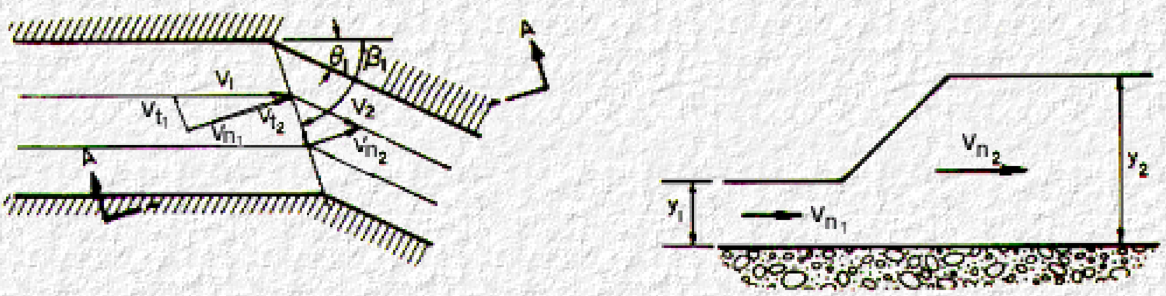


Figure 2.6.5. Definition Sketch for Rapid Flow in a Bend

Figure 2.6.5 is a definition sketch to aid in the analysis of cross wave patterns in a bend with rapid flow. The water surface elevation in a bend can be computed if the following major assumptions are made: (1) the flow is two-dimensional; (2) the velocity is constant throughout the cross-section; (3) the channel is horizontal; (4) there are no boundary shear stresses; and (5) the channel walls are vertical. The outer wall which turns the flow inward produces an oblique hydraulic jump and a corresponding positive disturbance line or positive wave front propagates across the channel. The inner or convex wall causes an oblique expansion or negative wave to propagate across the channel with a corresponding negative disturbance line or wave front. From analysis of Figure 2.6.5 and the hydraulic jump equation the following formulas can be derived.

The initial velocity perpendicular to the wave front is given by

$$V_{n1} = \left\{ \frac{g y_2}{2} \left(1 + \frac{y_2}{y_1} \right) \right\}^{1/2} \tag{2.6.14}$$

The wave front angle is given by

$$\sin \beta_1 \frac{V_{n1}}{V_1} = \frac{\sqrt{g y_1}}{V_1} \left\{ \frac{y_2}{2 y_1} \left(1 + \frac{y_2}{y_1} \right) \right\}^{1/2}$$

or

$$\sin \beta_1 \simeq \frac{1}{Fr_1} \tag{2.6.15}$$

The relationship of the deflection angle θ_1 and the Froude number is given by

$$\theta_1 = \sqrt{3} \tan^{-1} \left\{ \left(\frac{3}{Fr_1^2 - 1} \right)^{1/2} \right\} - \tan^{-1} \left\{ \left(\frac{1}{Fr_1^2 - 1} \right)^{1/2} \right\} + \text{const} \tag{2.6.16}$$

where the constant may be determined from the conditions that for $\theta_1 = 0$, the depth y is the initial depth y_1.

For practical applications, in the case of successive waves as sketched in Figure 2.6.6, Equation 2.6.16 is quite involved and inconvenient to use even with graphical charts. Knapp (1951) developed a much simpler equation which gives adequate results. The depth at the first maximum may be computed from

$$y = \frac{V^2}{g} \sin^2 \left(\beta_1 + \frac{\theta_1}{2} \right) \tag{2.6.17}$$

Equation 2.6.17 results from experimental observations of a constant velocity occurring at a cross-section. The location of the first maximum may be found from:

$$\theta_1' = \tan^{-1}\left\{\frac{2W}{(2r_c + W)\tan\beta_1'}\right\} \quad (2.6.18)$$

where r_c is the radius of curvature and W is the channel width as shown in a plan view of the cross wave pattern given in Figure 2.6.6. The disturbance wave pattern oscillates about a plane located at the normal depth. The distance along the wall to the first maximum subtends a central angle, θ_1', and this distance represents half a wave length.

The amplitude of the disturbance pattern in the downstream tangent is dependent on whether the new disturbance pattern created in the change of flow from curved to straight reinforces or damps out the disturbance pattern already in existence. When the curve has central angles of θ'_1, $3\theta'_1$, $5\theta'_1$, etc., where θ'_1 is given by Equation 2.6.18, the two disturbance patterns reinforce each other and the resulting disturbance pattern in the tangent section oscillates about the normal depth with an amplitude of approximately V^2W/r_cg. By adopting central angles of $2\theta'_1$, $4\theta'_1$, $6\theta'_1$, etc., the disturbance pattern generated by the change from a straight to curved channel will cancel out the disturbance created by the initial curve in the channel.

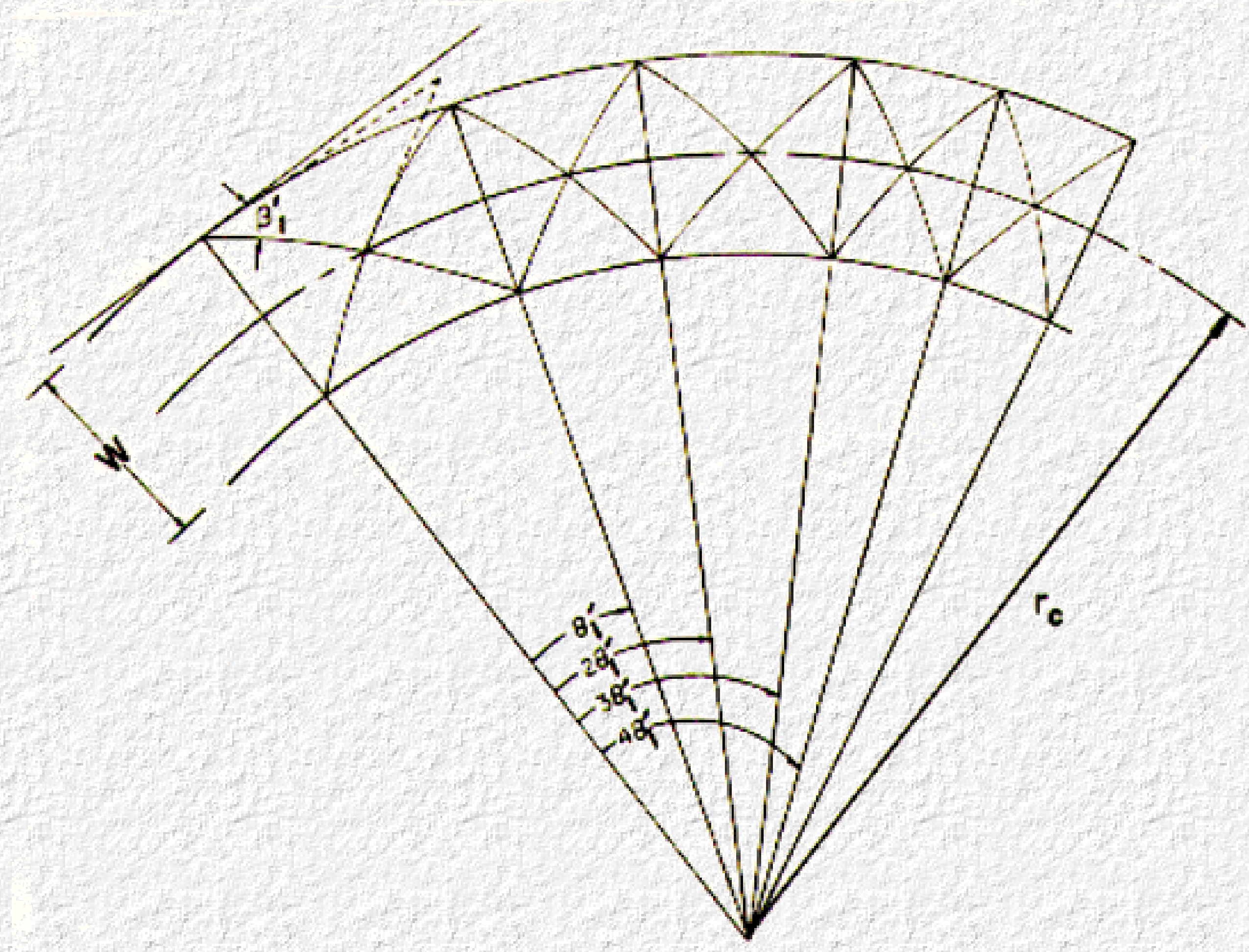


Figure 2.6.6. Plan View of Cross Wave Pattern for Rapid Flow in a Bend

Two methods have been used in the design of curves for rapid flow in channels. One method is to bank the floor of the channel and the other is to provide curved vanes in the flow. Banking on the floor produces lateral forces which act simultaneously on all filaments and causes the flow to turn without destroying the flow equilibrium. Curved vanes break up the flow into a series of small channels and since the superelevation is directly proportional to the channel width, each small channel has a smaller superelevation.

2.6.5 Transitions in Rapid Flows

Contractions and expansions in rapid flows produce cross wave patterns similar to those observed in curved channels. The cross waves are symmetrical with respect to the centerline of the channel. Ippen and Dawson (1951) have shown that in order to minimize the disturbance downstream of a contraction, the length of the contraction should be:

$$L = \frac{W_1 - W_2}{2\tan\theta} \qquad (2.6.19)$$

where W is the channel width and the subscripts 1 and 2 refer to sections upstream and downstream from the contraction. The contraction angle is θ and should not exceed 12°. The relationship between the channel widths and depths, y, can be determined from the continuity of the flow, $W_1y_1V_1 = W_2y_2V_2 = Q$ or

$$\frac{W_1}{W_2} = \left(\frac{y_2}{y_1}\right)^{3/2}\left(\frac{Fr_2}{Fr_1}\right) \qquad (2.6.20)$$

For an expansion, Rouse et al. (1951) found experimentally that the most satisfactory boundary form is given by

$$\frac{w}{W_1} = \frac{1}{2}\left(\frac{x}{W_1 Fr_1}\right)^{3/2} + \frac{1}{2} \qquad (2.6.21)$$

where x is the longitudinal distance measured from the start of the expansion or outlet section and w is the lateral coordinate measured from the channel centerline. A boundary developed from this equation diverges indefinitely. Therefore, for practical purposes, the divergent walls are followed by a transition to parallel lines.

2.7 Gradually Varied Flow

2.7.1 Introduction

Thus far, two types of steady flow have been considered. They are uniform flow and rapidly varied nonuniform flow. In uniform flow, acceleration forces are zero and energy is converted to heat as a result of viscous forces within the flow; there are no changes in cross-section or flow direction, and the depth (called normal depth) is constant. In rapidly varied flow, changes in cross-section, direction, or depth take place in relatively short distances; acceleration forces are not zero; viscous forces can be neglected (at least as a first approximation).

Different conditions prevail for each of these two types of steady flow. In steady uniform flow, the slope of the bed, the slope of the water surface and the slope of the energy gradeline are all parallel and are equal to the head loss divided by the length of the channel in which the loss occurred. In rapidly varied flow through short streamlined transitions, resistance is neglected and changes in depth due to acceleration are dominant. In this section, a third type of steady flow is considered. In this type of flow, changes in depth and velocity take place slowly over large distances, resistance to flow dominates and acceleration forces are neglected. This type of flow is called gradually varied flow, and the study involves: (1) the determination of the general characteristics of the water surface; and (2) the elevation of the water surface or depth of flow.

In gradually varied flow, the actual flow depth y is either larger or smaller than the normal depth y_o and either larger or smaller than the critical depth y_c. The water surface profiles, which are often called backwater curves, depend on the magnitude of the actual depth of flow y in relation to the normal depth y_o and the critical depth y_c. Normal depth y_o is the depth of flow that would exist for steady-uniform flow as determined using the Manning or Chezy velocity equations, and the critical depth is the depth of flow when the Froude number equals 1.0. Reasons for the depth being different than the normal depth are changes in slope of the bed, changes in cross-section, obstruction to flow and imbalances between gravitational forces accelerating the flow and shear forces retarding the flow.

In working with gradually varied flow, the first step is to determine what type of backwater curve would exist. The second step is to perform the numerical computations.

2.7.2 Classification of Flow Profiles

The classification of flow profiles is obtained by analyzing the change of the various terms in the total head equation in the x-direction. The total head is

$$H_T = \frac{V^2}{2g} + y + z \qquad (2.7.1)$$

or

$$H_T = \frac{Q^2}{2gA^2} + y + z \qquad (2.7.2)$$

Then assuming a wide channel for simplicity

$$\frac{dH_T}{dx} + \frac{q^2}{gy^3}\frac{dy}{dx} = \frac{dy}{dx} + \frac{dz}{dx} \qquad (2.7.3)$$

The term dH_T/dx is the slope of the energy gradeline S_f, by assumption. For short distances and small changes in y the energy gradient can be evaluated using the Manning or Chezy velocity equations.

When Chezy's equation (Equation 2.3.20b) is used the expression for dH_T/dx is

$$-\frac{dH_T}{dx} = S_f = \frac{q^2}{C^2 y^3} \qquad (2.7.4)$$

The term dy/dx is the slope of the water surface S_w, and dz/dx is the bed slope $-S_o$. For steady flow, the bed slope is (from Equation 2.3.20b)

$$S_o = \frac{q^2}{C_o^2 y_o^3} \qquad (2.7.5)$$

where the subscript "o" indicates the steady uniform flow values.

When Equation 2.7.4 and Equation 2.7.5 are substituted into Equation 2.7.3, the familiar form of the gradually varied flow equation is obtained

$$\frac{dy}{dx} = S_o \left\{ \frac{1 - \left(\frac{C_o}{C}\right)^2 \left(\frac{y_o}{y}\right)^3}{1 - \left(\frac{y_c}{y}\right)^3} \right\} \qquad (2.7.6)$$

If Manning's equation is used to evaluate S_f and S_o, Equation 2.7.6 becomes

$$\frac{dy}{dx} = S_o \left\{ \frac{1 - \left(\frac{n}{n_o}\right)^2 \left(\frac{y_o}{y}\right)^{10/3}}{1 - \left(\frac{y_c}{y}\right)^3} \right\} \qquad (2.7.7)$$

The slope of the water surface dy/dx depends on the slope of the bed S_o, the ratio of the normal depth y_o to the actual depth y and the ratio of the critical depth y_c to the actual depth y. The difference between flow resistance for steady uniform flow n_o to flow resistance for steady nonuniform flow n is small and the ratio is taken as 1.0. With $n = n_o$, there are twelve types of water surface profiles. These are illustrated in Figure 2.7.1 and summarized in Table 2.7.1.

Table 2.7.1 Characteristics of Water Surface Profiles

Class	Bed Slope	Depth	Type	Classification
Mild	$S_o > 0$	$y > y_o > y_c$	1	M1
Mild	$S_o > 0$	$y_o > y > y_c$	2	M2
Mild	$S_o > 0$	$y_o > y_c > y$	3	M3

Critical Critical	$S_o > 0$ $S_o > 0$	$y > y_o = y_c$ $y < y_o = y_c$	1 3	C1 C3
Steep Steep Steep	$S_o > 0$ $S_o > 0$ $S_o > 0$	$y > y_c > y_o$ $y_c > y > y_o$ $y_c > y_o > y$	1 2 3	S1 S2 S3
Horizontal Horizontal	$S_o = 0$ $S_o = 0$	$y > y_c$ $y_c > y$	2 3	H2 H3
Adverse Adverse	$S_o < 0$ $S_o < 0$	$y > y_c$ $y_c > y$	2 3	A2 A3

Note:

1. With a type 1 curve (M1, S1, C1), the actual depth of flow y is greater than both the normal depth y_o and the critical depth y_c. Because flow is tranquil, control of the flow is downstream.
2. With a type 2 curve (M2, S2, A2, H2), the actual depth y is between the normal depth y_o and the critical depth y_c. The flow is tranquil for M2, A2 and H2 and thus the control is downstream. Flow is rapid for S2 and the control is upstream.
3. With a type 3 curve (M3, S3, C3, A3, H3), the actual depth y is smaller than both the normal depth y_o and the critical depth y_c. Because the flow is rapid control is upstream.
4. For a mild slope, S_o is smaller than S_c and $y_o > y_c$.
5. For a steep slope, S_o is larger than S_c and $y_o < y_c$.
6. For a critical slope, S_o equals S_c and $y_o = y_c$.
7. For an adverse slope, S_o is negative.
8. For a horizontal slope, S_o equals zero.
9. The case where $y \rightarrow y_c$ is of special interest because the denominator in Equation 2.7.7 approaches zero.

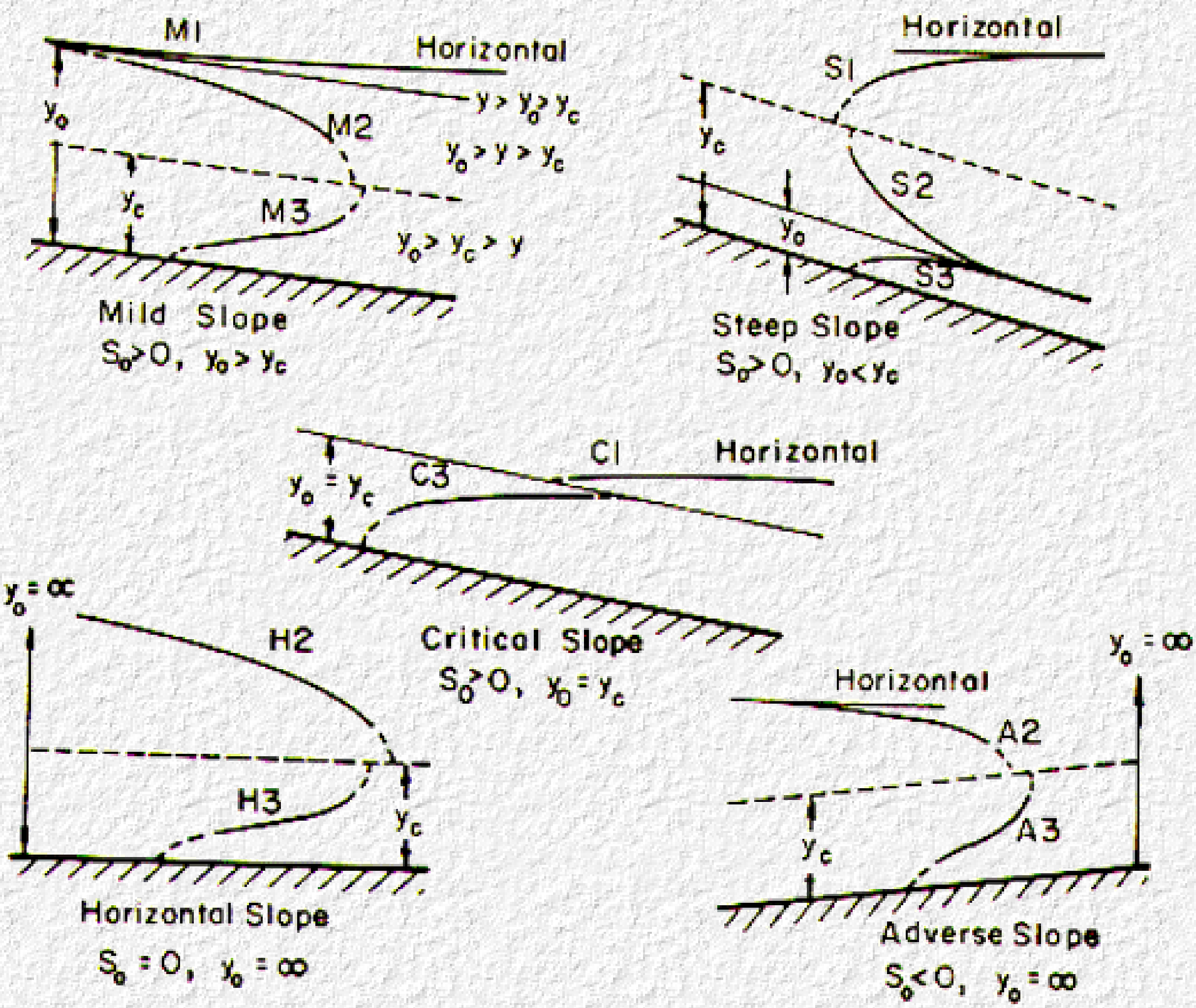


Figure 2.7.1. Classification of Water Surface Profiles

When $y \rightarrow y_c$, the assumption that acceleration forces can be neglected no longer holds. Equation 2.7.6 or Equation 2.7.7 indicate that dy/dx is perpendicular to the bed slope when $y \rightarrow y_c$. For cross-sections close to the cross-section where the flow is critical (a distance from 50 to 10 ft), curvilinear flow analysis and experimentation must be used to determine the actual values of y. When analyzing long distances (100 to 1000 ft or longer) one can assume qualitatively that y reaches y_c. In general, when the flow is rapid ($Fr \geq 1$), the flow cannot become tranquil without a hydraulic jump occurring. In contrast, tranquil flow can become rapid (cross the critical depth line). This is illustrated in Figure 2.7.2.

When there is a change in cross-section or slope or an obstruction to the flow, the qualitative analysis of the flow profile depends on locating the control points, determining the type of curve upstream and downstream of the control points, and then sketching the backwater curves. It must be remembered that <u>when flow is rapid (Fr > 1), the control of the depth is upstream and the backwater proceeds in the downstream direction. When flow is tranquil (Fr < 1), the depth control is downstream and the computations must proceed upstream</u>. The backwater curves that result from a change in slope of the bed are illustrated in Figure 2.7.2

2.7.3 Standard Step Method for the Computation of Water Surface Profiles

The standard step method is a simple computational procedure to determine the water surface profile in gradually varied flows. Prior knowledge of the type of backwater curve as classified in Section 2.7.2 is useful to determine whether the analysis should proceed upstream or downstream.

The standard step method is derived from the energy equation

$$\frac{V_1^2}{2g} + y_1 + \Delta z = \frac{V_2^2}{2g} + y_2 + H_L \qquad (2.7.8)$$

From Figure 2.7.3

$$\frac{V_1^2}{2g} + y_1 + S_o \Delta L = \frac{V_2^2}{2g} + y_2 + S_f \Delta L \qquad (2.7.9)$$

$$H_1 + S_o \Delta L = H_2 + S_f \Delta L \qquad (2.7.10)$$

and

$$\Delta L = \frac{H_2 - H_1}{S_o - S_f} \qquad (2.7.11)$$

of

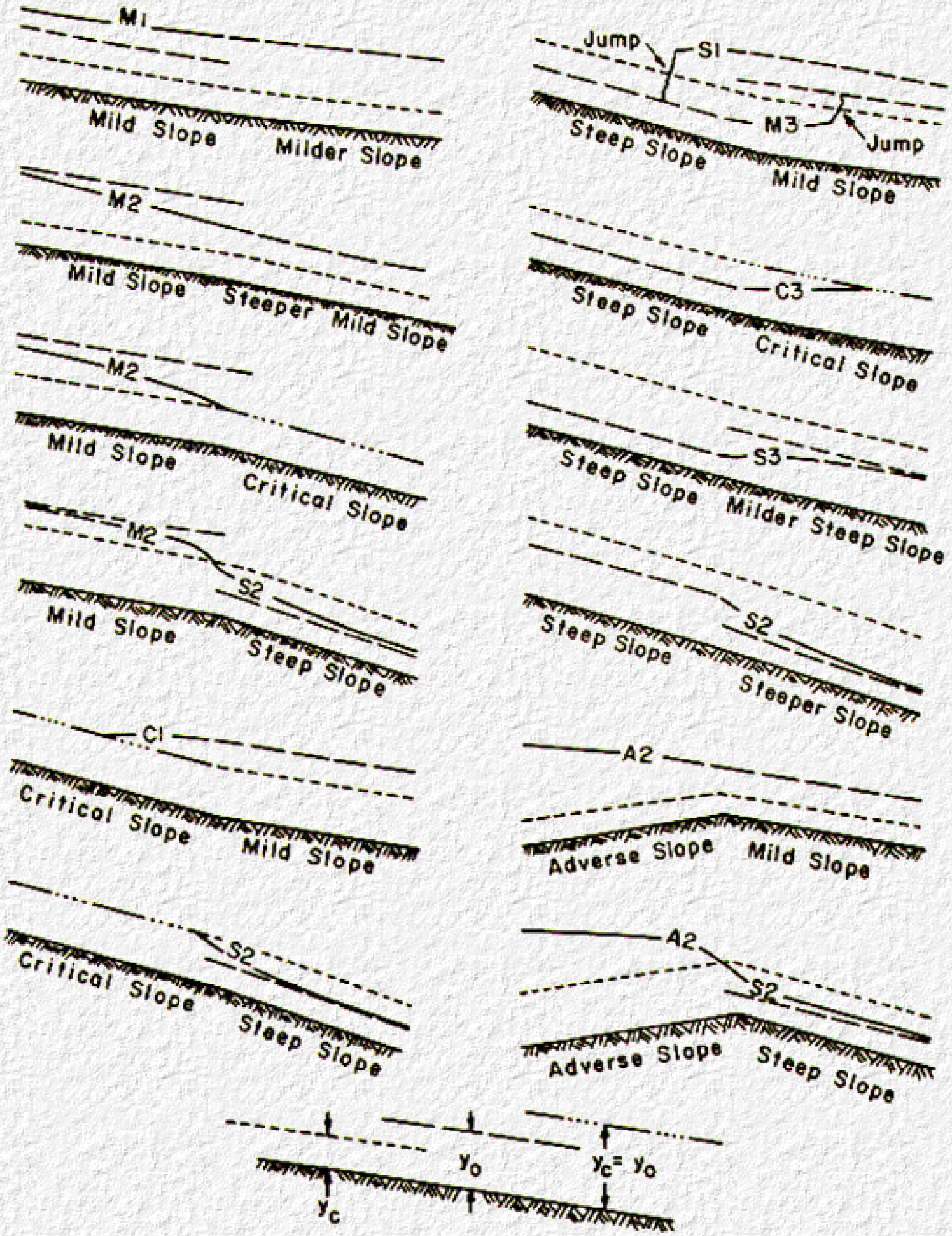


Figure 2.7.2. Examples of Water Surface Profiles

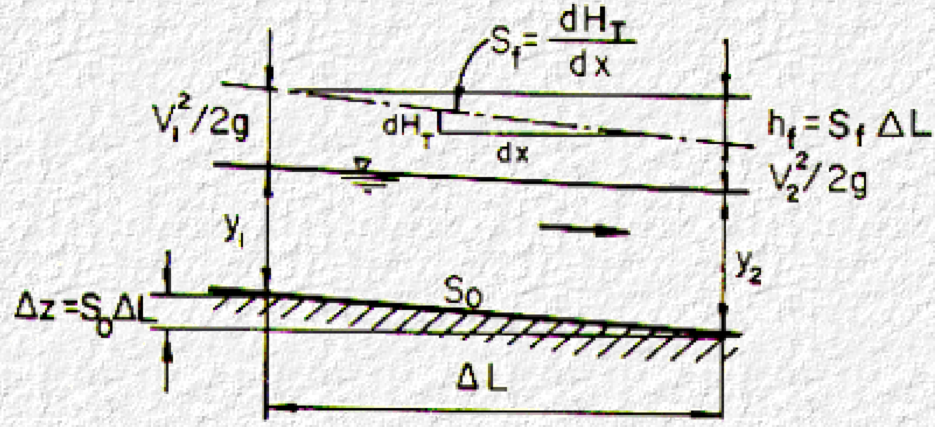


Figure 2.7.3. Definition Sketch for the Standard Step Method for Computation of Backwater Curves

The procedure is to start from some known y, which can be obtained from a stage-discharge relationship, assume another y either upstream or downstream depending on whether the flow is tranquil or rapid, and compute the distance ΔL to the assumed depth using Equation 2.7.11. It is recommended that the assumed depth be kept relatively close to the known value of y_1 in order to keep the interval ΔL as short as possible to obtain better accuracy in the calculation.

An example illustrating how to use the standard step method is given in full detail in Appendix 2 (Problem 2.3).

2.8 Hydraulics of Bridge Waterways

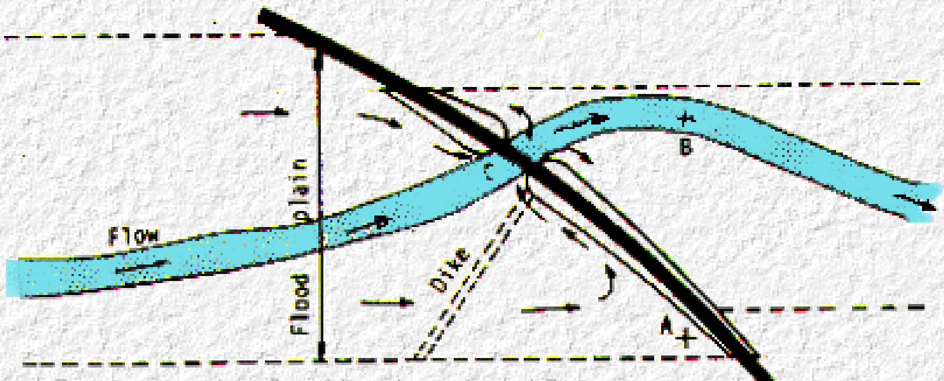
As flow passes through a channel constriction most of the energy losses occur as expansion losses downstream of the contraction. This loss of energy is reflected by a rise in the water surface and the energy line upstream from the bridge. The determination of the rise in water level is referred to as the bridge backwater. Hydraulic engineers are concerned about the computation of backwater with respect to flooding upstream of the bridge. Other concerns discussed in the following chapters include the stability and scour around embankments, general scour depths due to constriction and local scour around piers.

2.8.1 Backwater Effects on Waterway Openings

It is necessary to distinguish between the following types of backwater effects.

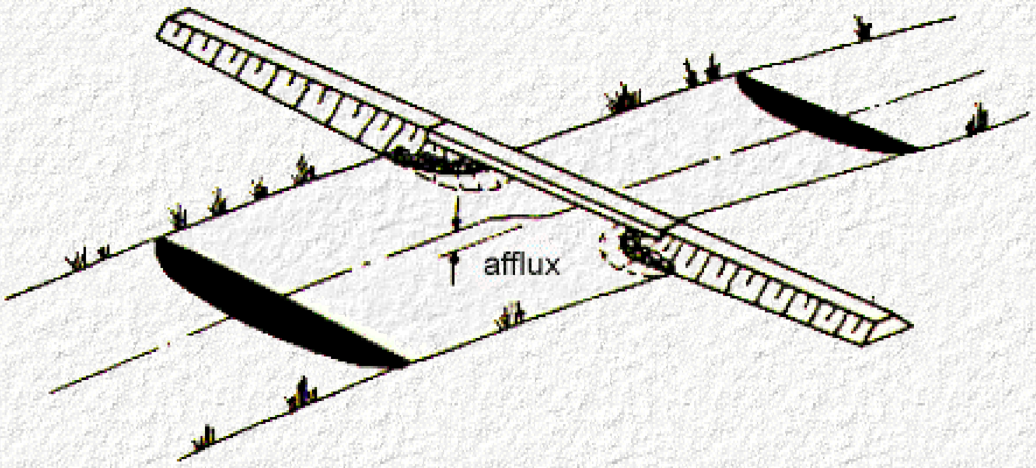
- Backwater on a flood plain resulting from construction of a long, skewed or curved road embankment as sketched in Figure 2.8.1a, where the bridge opening is in effect located up-valley from one end of the embankment. The backwater effect along the embankment arises from ponding of water along a line running obliquely down-valley. In the case of steep rivers with wide flood plains this effect can be very large, since a large pond is created. This type of effect can be prevented by choosing a suitable location and alignment, or by providing dikes shown on the figure to close off the affected part of the flood plain from flood waters, or possibly by providing a relief span.
- Backwater in an incised river channel without substantial overbank flow, resulting in part from constriction of flow through an opening somewhat smaller than the natural cross section, and in part from obstructive effects of piers (Figure 2.8.1b). The backwater effect arising from this type is seldom large, but may be significant in occupied areas.

- Backwater in a river with flood plain where the road crossing is more or less normal to the valley but the road approaches block off overbank flow (Figure 2.8.1c). In these cases the afflux may be significantly greater than in type b. The effect of guide banks shown in Figure 2.8.1c appears to be generally to reduce the backwater effects by improving the hydraulic efficiency of the opening, but there is some doubt as to whether this is necessarily true for steep streams.

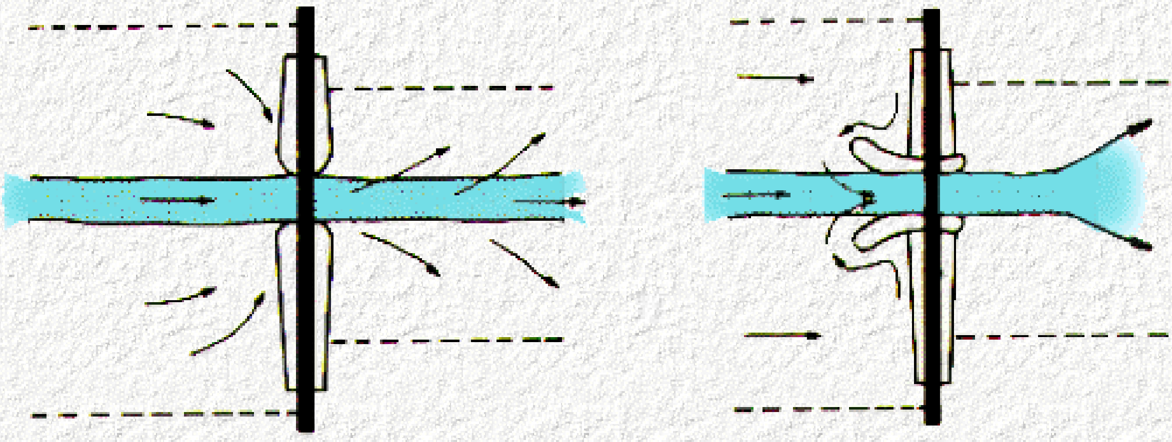


Pre-contruction flood levels at A and B are approximately equal.
Post-construction level at A is higher than at C, which is higher than at B because of channel slope and bridge backwater.
Dike as shown would protect A from backwater.

a



b



c

Figure 2.8.1. Three Types of Backwater Effect Associated with Bridge Crossings; (a) Effect of a Skewed Embankment across a Flood Plain; (b) Effect Due to Constriction of the Channel Flow; (c) Effect Due to Constriction of the Overbank Flow, Both without and with Guide Banks. (after Neill, 1975)

It is advisable to be aware of other unusual backwater effects that might occur in special circumstances, although they might never arise in ordinary bridge design practice.

Effects of a Submerged Superstructure - If the high-water level reaches the bottom of the superstructure, the bridge will act as a short culvert. For bridges which are designed to be submersible under certain conditions, it is advisable to provide a rounded nosing on the leading edge of the girder, in order to improve the hydraulic efficiency and to reduce the tendency to catch driftwood and ice as illustrated in Figure 2.8.2. Also, the superstructure must be anchored to counter buoyancy.

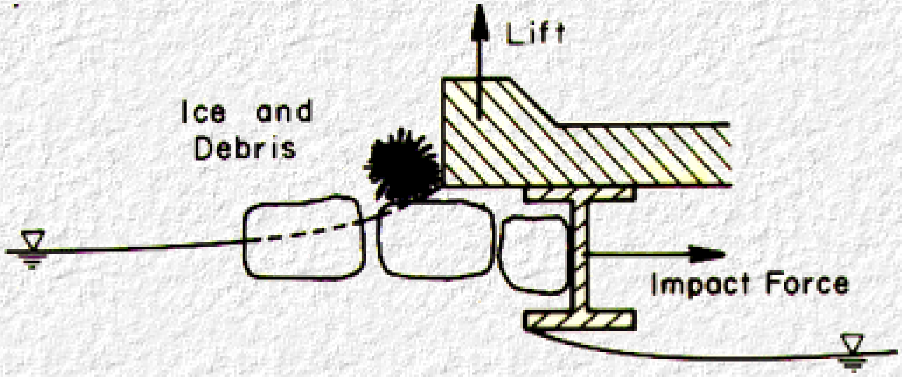


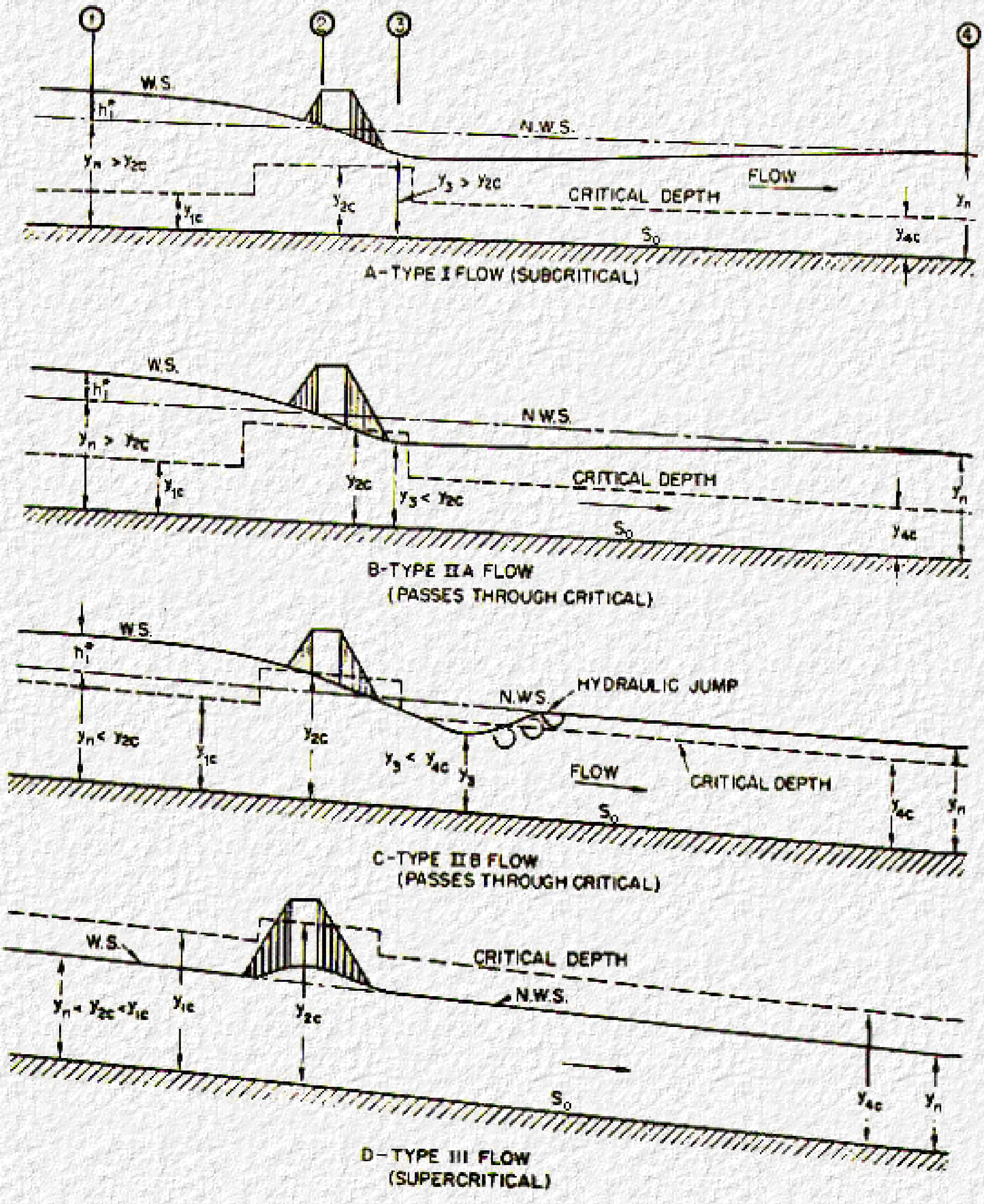
Figure 2.8.2. Submergence of a Superstructure

Effects of Supercritical Flow - In contrast to the usual drop at a constriction in subcritical flow, in supercritical flow water levels may rise suddenly at the contracted section. The phenomenon of "choking" is particularly likely if the Froude number only slightly exceeds 1.0. "Choking" may occur even in subcritical flow if the constriction is severe enough. Wider or additional openings should be designed if choking effects are expected to occur.

2.8.2 Types of Flow In Bridge Openings

Three types of flow, I through III, illustrated in Figure 2.8.3 are often encountered in bridge waterway design. As the scale of the normal depth is the same for all flow profiles, the discharge, boundary roughness and slope of the channel must increase from type I to type II and to type III.

In Type I flow, the normal water surface is everywhere above critical depth and the flow is subcritical. Backwater calculations are obtained by applying the conservation of energy principle between sections 1 and 4.



h_1* = Backwater
N.W.S. = Normal Water Surface
W.S. = Water Surface

Figure 2.8.3. Types of Flow Encountered
(Ref - Hydraulics of Bridge Waterways HDS No. 1)

In Type II flow, subcritical flow upstream of the bridge passes through critical depth in the constriction. The backwater curve giving the water surface elevation upstream from the constriction becomes independent of the water surface elevation downstream. An undulating hydraulic jump (with $Fr < 2$) is formed when the water surface elevation dips below critical depth downstream from the contracted section (Type IIB).

Referring to Type III flow, the flow is supercritical throughout the reach as the normal water surface is everywhere below critical depth. Such conditions require steep channels as experienced in, but not limited to mountainous regions. Backwater should not occur as long as the flow remains supercritical since the flow is controlled from upstream conditions. However, significant rise in the water surface might occur in the vicinity of the constriction due to: (1) changes in the specific energy diagram as indicated in Figure 2.5.2 and Figure 2.5.4; (2) cross waves and transitions; and (3) possible hydraulic jumps near the embankments.

Two solved problems are presented in Appendix 2 to illustrate how to calculate maximum constrictions without causing backwater (Problem 2.4) and to calculate water surface elevation upstream of a grade control structure (Problem 2.5).

2.8.3 Bridge Backwater Analysis

This section presents a brief introduction to the analysis of backwater effects induced by bridge crossings. A practical expression for backwater has been formulated by applying the principle of conservation of energy between sections 1 and 4 illustrated in Figure 2.8.3. The backwater elevation upstream from the bridge is computed as follows:

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \frac{A_{n2}}{A_4}^{2} - \frac{A_{n2}}{A_1}^{2} \frac{V_{n2}^2}{2g} \qquad (2.8.1)$$

where

h_1^* = total backwater (in ft)

K^* = total backwater coefficient for subcritical flow

α_1, α_2 = kinetic energy coefficients

A_{n2} = flow area below normal stage at the bridge opening (ft^2)

$V_{n2} = Q/A_{n2}$ average velocity at the bridge opening (ft/s)

A_1, A_4 = flow areas at section 1 and 4 (ft^2)

Equation 2.8.1 is essentially applicable for Type I flow. For Type II flow, the same equation (Equation 2.8.1) can be used with $\alpha_1 = 0$. For Type III flow, cross waves and hydraulic jumps should be analyzed closely.

The total backwater coefficient K^* is the sum of the coefficients for the base curve K_b; piers ΔK_p; eccentricity ΔK_e; and skew ΔK_s so:

$$K^* = K_b + \Delta K_p + \Delta K_e + \Delta K_s \qquad (2.8.2)$$

The backwater coefficient K_b for subcritical flow depends mainly on the bridge opening ratio M which is defined as the ratio of the flow which can pass unimpeded through the bridge constriction to the total flow of the river. In other words, $M = Q_b/(Q_a + Q_b + Q_c)$ as shown in Figure 2.8.6. Figure 2.8.4 shows the backwater coefficient K_b as a function of the opening ratio M for wing wall and spillthrough abutments.

The value of the incremental backwater coefficient due to piers ΔK_p depends on the ratio J of the projected area of the piers A_p to the gross constricted area A_{n2}, based on the normal surface of the bridge opening. The type of piers, the bridge opening ratio M and the angle of the piers to the flow direction are also considered as shown in Figure 2.8.5. The incremental coefficient for piers is computed from the product of ΔK and σ defined in Figure 2.8.5a and Figure 2.8.5b.

$$\Delta K_p = \sigma \Delta K \qquad (2.8.3)$$

An opening with 10 piles should be given a value of ΔK_p about 20 percent higher than indicated for bents with 5 piles.

Eccentricity describes the location of the opening within the cross-section area. For example centered crossings have no eccentricity. The effect of eccentricity on the backwater coefficient is considered when the discharge components Q_c and Q_a defined in Figure 2.8.6 are not equal. The eccentricity e is computed from Q_a and Q_c and the incremental backwater coefficient ΔK_e is obtained from the diagram in Figure 2.8.6.

Skewness of the crossing is observed when the flow direction is not perpendicular to the embankment. The correction factor for skewed crossings is based on the projected length and cross sectional area parallel to the flow direction. The incremental coefficient varies with the opening ration M and the angle of skew ϕ as shown in Figure 2.8.7.

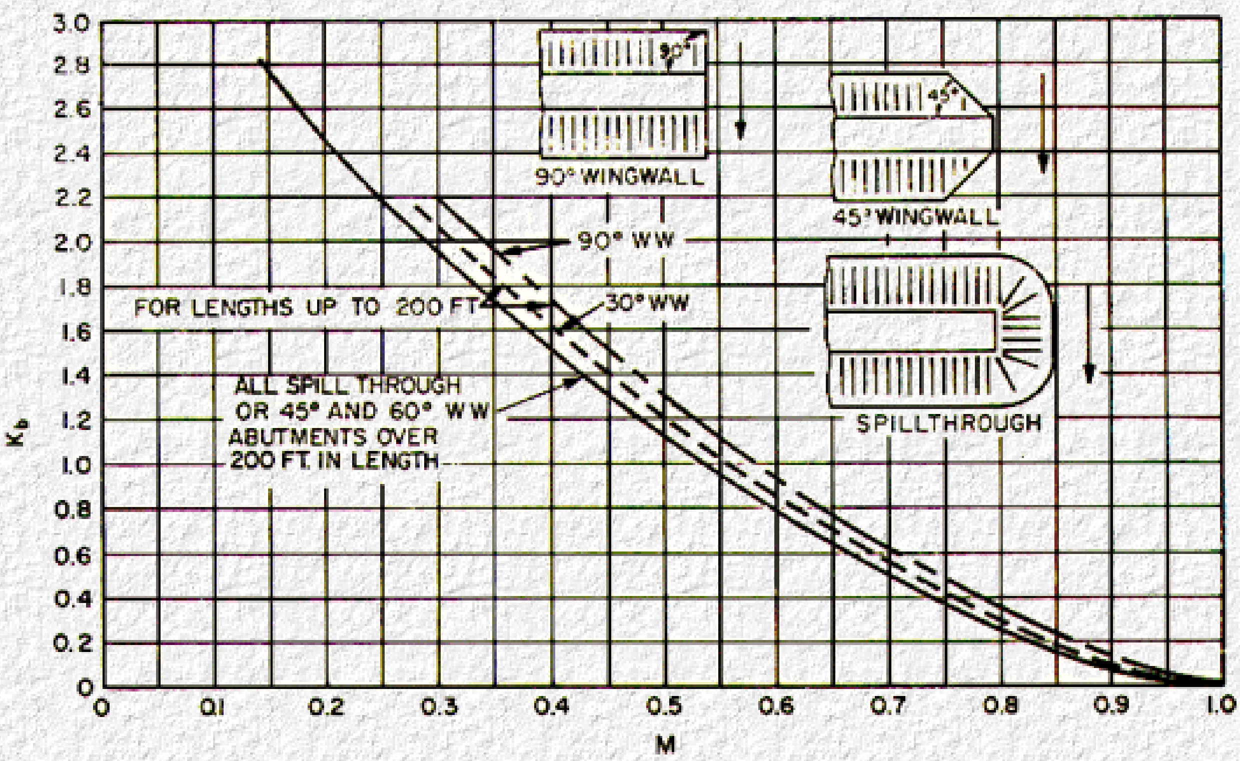


Figure 2.8.4. Backwater Coefficient Base Curves (Subcritical Flow)
(Ref - Hydraulics of Bridge Waterways HDS No. 1)

An example to calculate bridge backwater elevation is detailed in Appendix 2 (Problem 2.6). The problem involves the effects of piers, skewness and eccentricity on the flow.

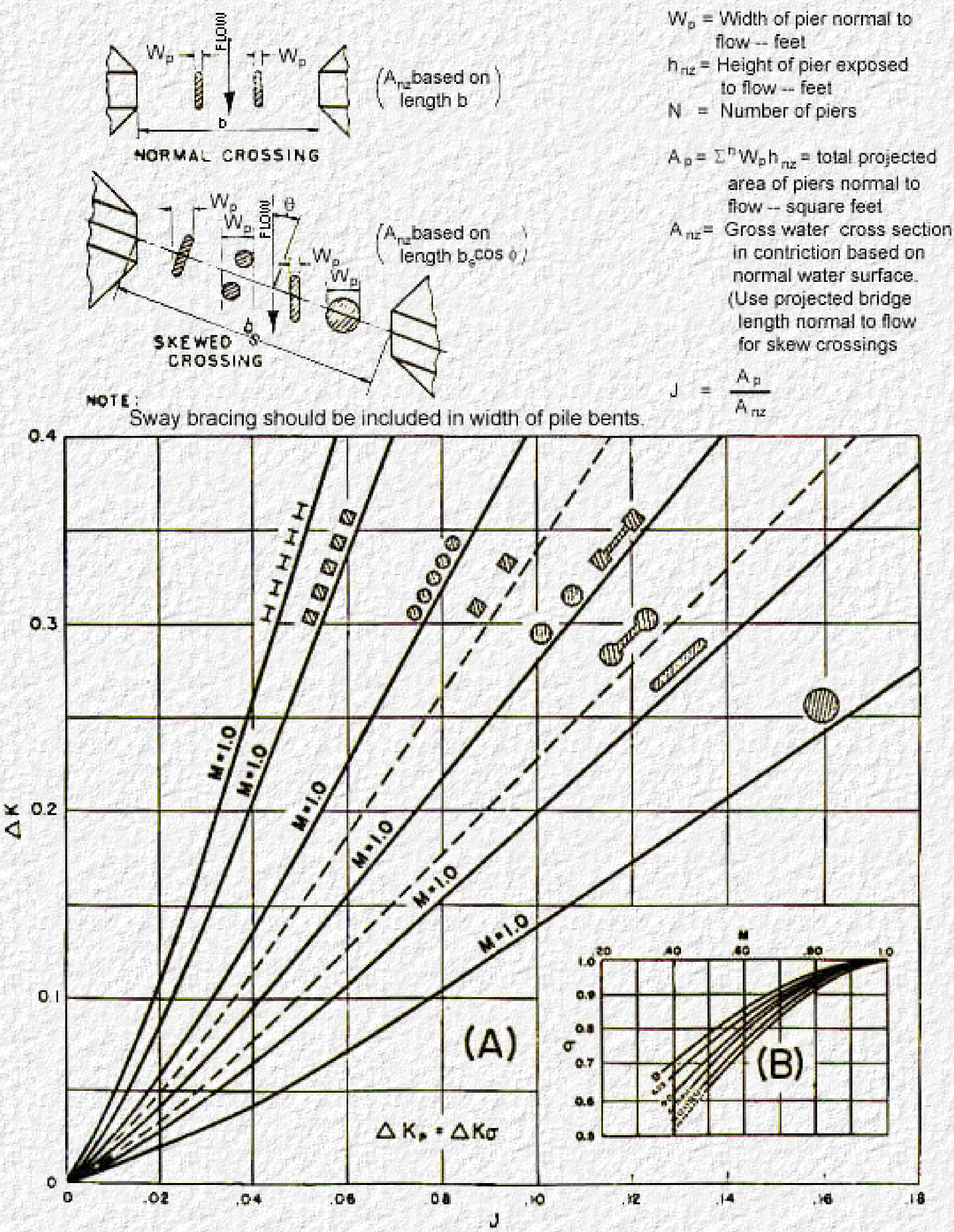


Figure 2.8.5. Incremental Backwater Coefficient for Piers
(Ref - Hydraulics of Bridge Waterways HDS No. 1)

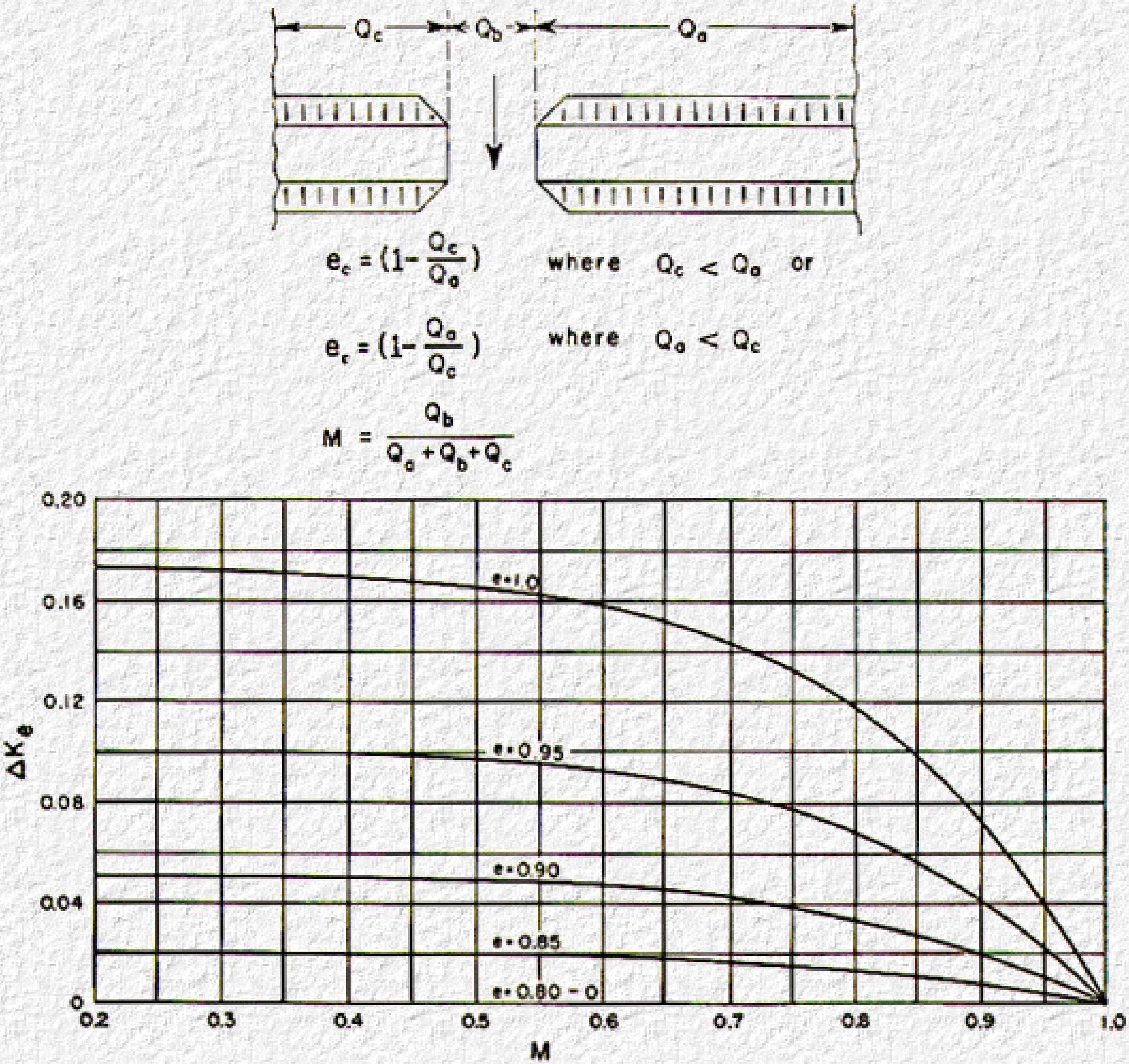


Figure 2.8.6. Incremental Backwater Coefficient for Eccentricity
(Ref - Hydraulics of Bridge Waterways [HDS No. 1])

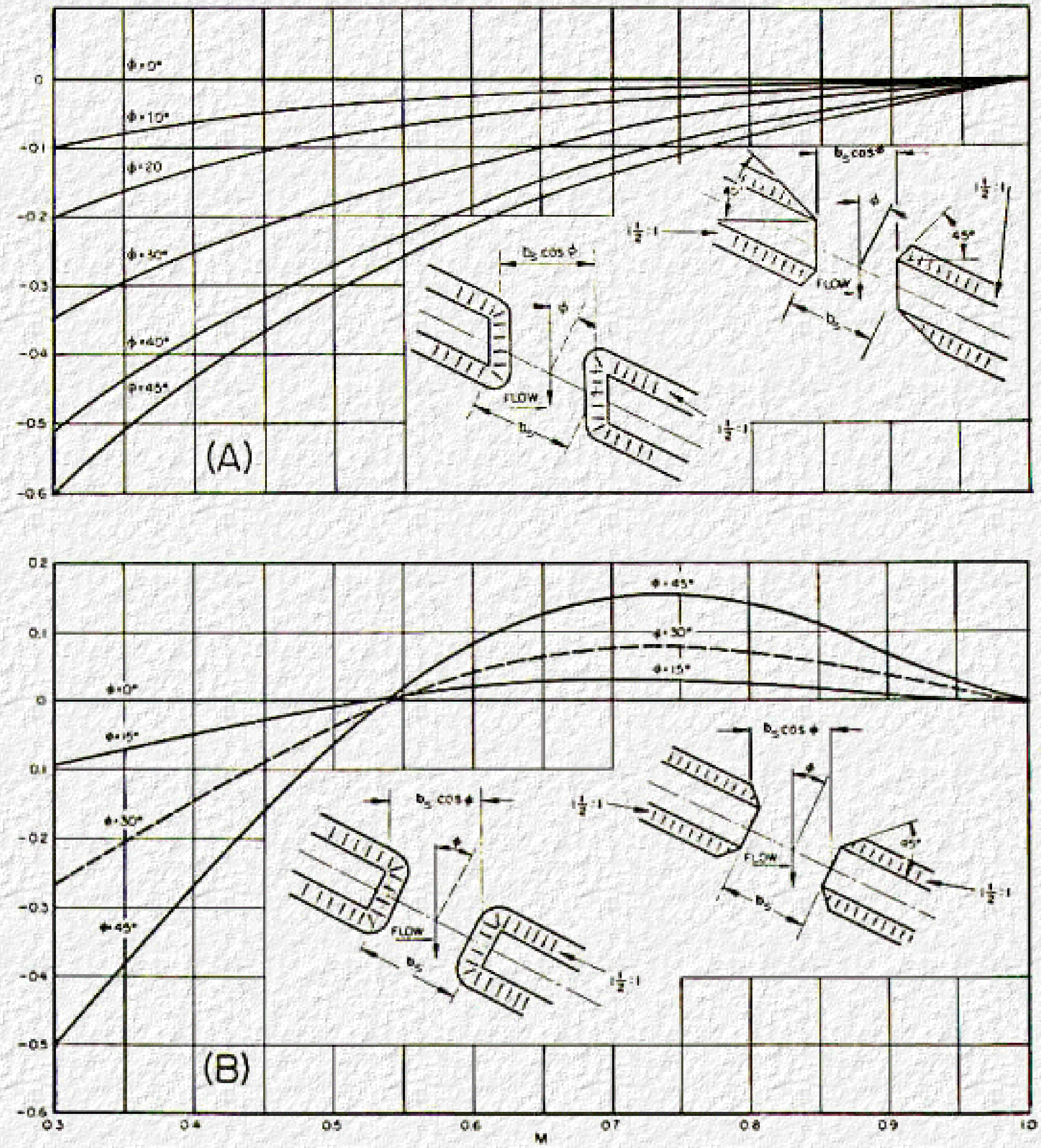


Figure 2.8.7. Incremental Backwater Coefficient for Skew
(Ref - Hydraulics of Bridge Waterways HDS No. 1)

2.8.4 Computer Programs for Highway Bridges

The Department of Transportation had prepared an earlier computer program (Hydraulics of Bridge Waterways - HY-4) to determine the backwater produced by a bridge. The program consists of two parts. Part I analyzes the natural stream cross section, rating curves and stage discharge information. Part II determines velocities and backwater heights for a given discharge and bridge length.

A recent research report (Shearman et al.) in preparation, to be published as a FHWA Research and Development Report, should become the state-of-the-art on computer modeling of water surface elevation in waterway crossings. The report entitled, "Bridge Waterways Analysis Model," describes a Water Surface Profile (WSPRO) computation model. Profile computations for open channel flow are compatible with conventional techniques used in existing step-backwater analysis models. WSPRO incorporates several more desirable features than other existing models. Profile computations for free surface flow through bridges are based on relatively recent developments in bridge backwater analysis and recognize the influence for bridge geometry variations. Pressure flow situations (girders partially or fully inundated) are computed using existing Federal Highway Administration techniques. Embankment overtopping flows, in conjunction with either free surface or pressure flow through the bridge, can be computed. WSPRO is also capable of computing profiles at stream crossings with multiple openings (including culverts). WSPRO also incorporates the effect of wide, wooded, floodplains into the bridge backwater computations.

Although specifically oriented towards hydraulic design of stream highway crossings using economic analysis, WSPRO is equally suitable for water surface profile computations unrelated to highway design. The report provides a detailed discussion of the theory and computational techniques used in the model. Model capabilities and data requirements are described in more general terms. Specific data coding instructions and examples of model applications are to be published in a users manual. Results of model application to five field-verification sites are discussed, along with a comparison of WSPRO results with results obtained from two existing models. Also presented is a discussion of the applicability of WSPRO to design of bridges using economic analysis.

Other one-dimensional models for steady gradually varied flow in channels with fixed boundary include the model HEC-2 developed by the U.S. Army Corps of Engineers to determine water surface elevations by standard step method. The effects of natural obstructions to flow, flood plain, encroachment and hydraulic structures can be simulated by the program.

Erodible-bed models such as SEDIMENT 4H (Ariathurai), FLUVIAL 11 (Chang), HEC-6 (U.S. Army Corps of Engineers), HEC-2SR, KUWASER and UUWSR (Simons, Li and Associates) are also available.

Liou (1983) presented an evaluation of several existing mathematical models in flood zoning studies for FEMA. Fixed-bed models were compared with mobile-bed models to determine whether river-bed degradation during flood passage has an effect on flood stage. It was concluded that the effect of river-bed degradation and aggradation on water surface elevation during flood passage is much smaller than the effects of the uncertainties of channel roughness or flow friction factor, sediment input and initial channel geometry.

Erodible-bed models are appropriate for streams which have experienced extreme local aggradation or degradation. Their use in flood-insurance studies, however, does not seem justified at the present time based upon the current state-of-the-art moveable bed mathematical models.

2.9 Hydraulics of Culvert Flow

A culvert is a hydraulically short conduit which conveys stream flow through a roadway embankment. Most culverts are constructed of concrete, corrugated aluminum, corrugated steel, and sometimes corrogated plastics. Culvert shapes vary from circular to rectangular and elliptical, pipe arch, arch and metal box sections are commonly used.

Two basic types of flow control are recognized depending on the location of the control section: inlet control or outlet control. The characterization of pressure, subcritical and supercritical flow regimes play an important role in determining the location of the control section.

Inlet control occurs when the culvert barrel is capable of carrying more flow than the inlet will accept. Critical flow depth is located at the inlet and the flow is supercritical in the barrel.

Outlet control flow occurs when the culvert barrel is not capable of conveying as much flow as the inlet opening will accept. Under outlet control conditions, either subcritical or pressure flow exists in the culvert barrel.

2.9.1 Tailwater Depth Calculation

Energy is required to force flow to pass through a culvert. This energy takes the form of an increased water surface elevation on the upstream side of the culvert The depth of the upstream water surface measured from the invert at the culvert entrance is referred to as the headwater depth. Tailwater depth is defined as the depth of water downstream of the culvert measured from the outlet invert. High tailwater depth may cause submergence of the barrel outlet and influence the type of control and conveyance characteristics of the culvert.

The headwater depth, the hydraulic grade line and the energy line are shown in Figure 2.9.1. The total energy upstream from the culvert entrance is equated to the energy at the culvert outlet with consideration of all the major energy losses. From the energy equation, the head H is the difference between the elevations of the hydraulic grade line at the outlet and the energy line at the inlet

$$H = (d_1 - d_2) + \frac{V_1^2}{2g} + LS_o = H_v + H_e + H_f \qquad (2.9.1)$$

in which

H = total head in feet

d_1, d_2 = flow depth (in ft) as shown in Figure 2.9.1

$\frac{V_1^2}{2g}$ = velocity head at the inlet in feet

LS_o = length of culvert in feet, times barrel slope

H_v = velocity head in feet

H_e = entrance loss in feet

H_f = friction losses in feet

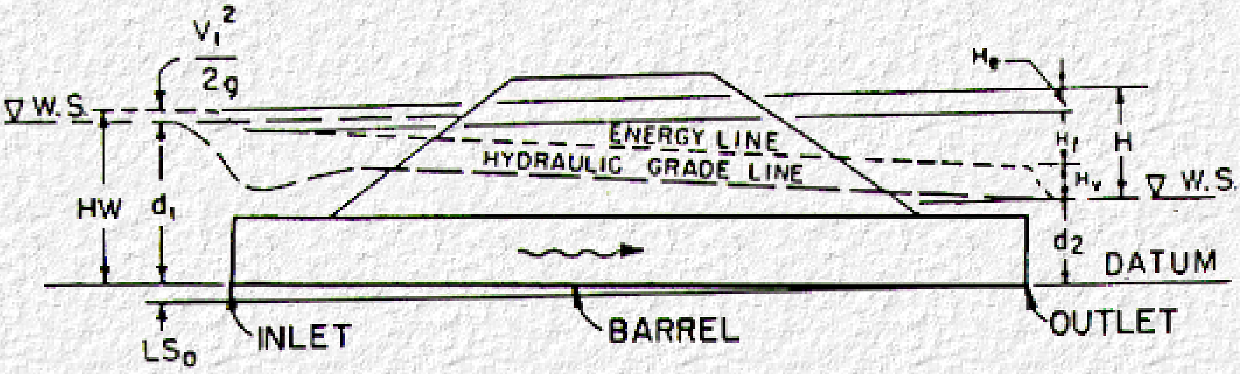


Figure 2.9.1. Hydraulic and Energy Grade Lines in Culvert Flow.

The velocity head H_v equals $\frac{V^2}{2g}$ where V is the mean velocity in the culvert (V = Q/A). The entrance loss depends on the geometry of the inlet and is expressed as a coefficient K_e times the velocity head

$$H_e = K_e \frac{V^2}{2g} \qquad (2.9.2)$$

Values of K_e range from 0.2 to 0.9 for various entrance configuration. Detailed information on the entrance loss coefficient K_e can be found in the report FHWA-IP-85-15. The friction loss H_f can be written as a function of Manning's n as follows:

$$H_f = \frac{29n^2L}{R^{1.33}} \quad \frac{V^2}{2g}$$

in which R is the hydraulic radius.

Therefore,

$$H = 1 + K_e + \frac{29n^2L}{R^{1.33}} \quad \frac{V^2}{2g} \qquad (2.9.3)$$

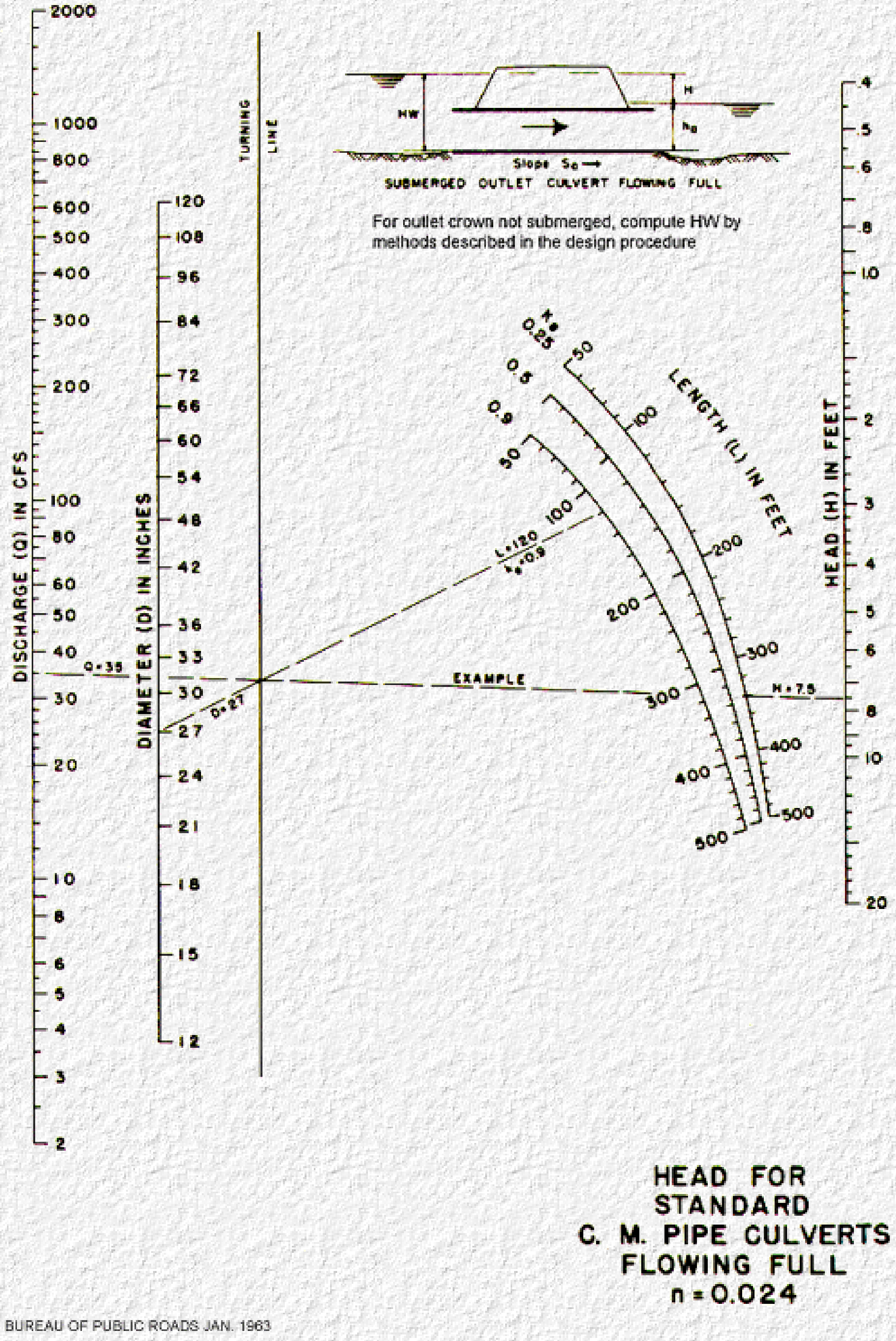


Figure 2.9.2a. Head for Standard Corrugated Metal Culverts Flowing Full (n = 0.024)

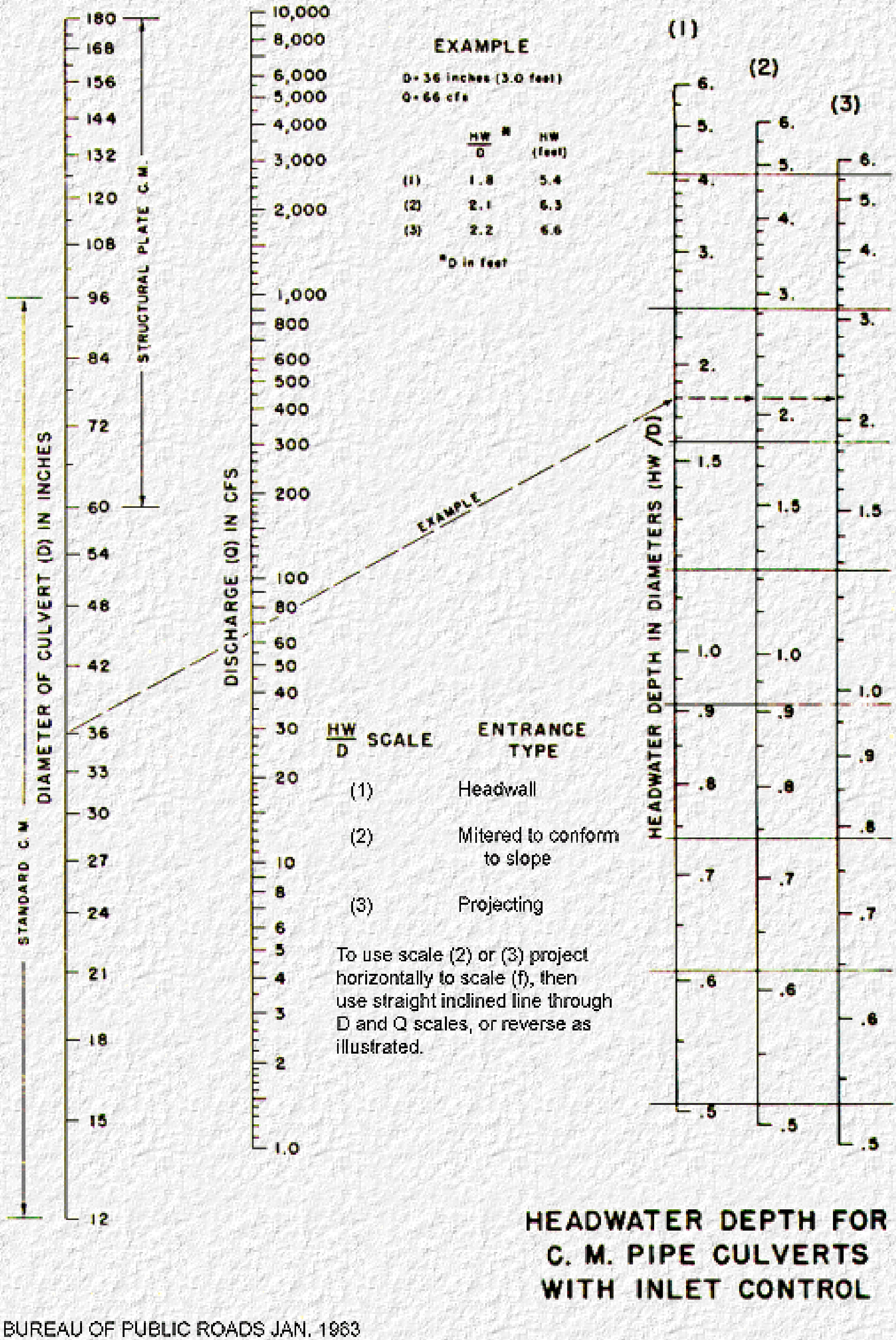


Figure 2.9.2b. Headwater Depth for Corrugated Metal Pipe Culverts with Inlet Control

As an example the nomograph in Figure 2.9.2 gives the solution of the head H for a corrugated metal pipe for the following conditions: L = 120 ft, D = 27 inches, Q = 35 ft^3/s and K_e = 0.9. In this example the head, H, for culverts flowing full is H = 7.5 feet. For a comprehensive treatment of flow in culverts, the reader is referred to the Report FHWA-ID-85-15, HDS-5, entitled, "Hydraulic Design of Highway Culverts."

A comprehensive design example of flow through culverts is detailed in Appendix 2 (Problem 2.7) with both inlet control and full flow conditions.

2.9.2 Performance of Culverts

Performance curves graphically depict the variation of headwater depth as a function of the flow rate. In developing a culvert performance curve both inlet and outlet control curves must be plotted. This is necessary because the dominant control at a given headwater depth is difficult to predict. Control may shift from the inlet to the outlet or vice versa over a range of flow rates. Figure 2.9.3 illustrates a typical culvert performance curve. At the design headwater, the culvert operates under inlet control.

Among its uses, the performance curve displays the consequences of higher flow rates on headwater height and the benefits of inlet improvements. The hydraulic performance of culverts can be improved by changing the inlet geometry. Improvements include bevel-edged, side-tapered and slope-tapered inlets.

A beveled-edge provides a decrease in flow contraction losses at the inlet and K_e is reduced from 0.5 to 0.2. This increases the culvert capacity by as much as 20 percent. Bevels are recommended on all culverts under inlet control conditions.

Side-tapered inlets have an enlarged face area accomplished by tapering sidewalls. It provides an increase in flow capacity of 25 to 40 percent over square-edged inlets. There are two types of control sections for side-tapered inlets: face and throat control. The advantages of side-tapered inlet under throat control are: reduced flow contraction at the throat and increased head at the throat control section.

Slope-tapered inlets provide additional head at the throat section. This type of inlet can have over 100 percent greater capacity than a conventional culvert with square edges. The degree of increased capacity depends upon the drop between the face and the throat section. Both the face and the throat are possible control sections. The inlet face should be designed with a greater capacity than the throat to insure flow control at the throat. More of the potential capacity of the culvert can then be insured.

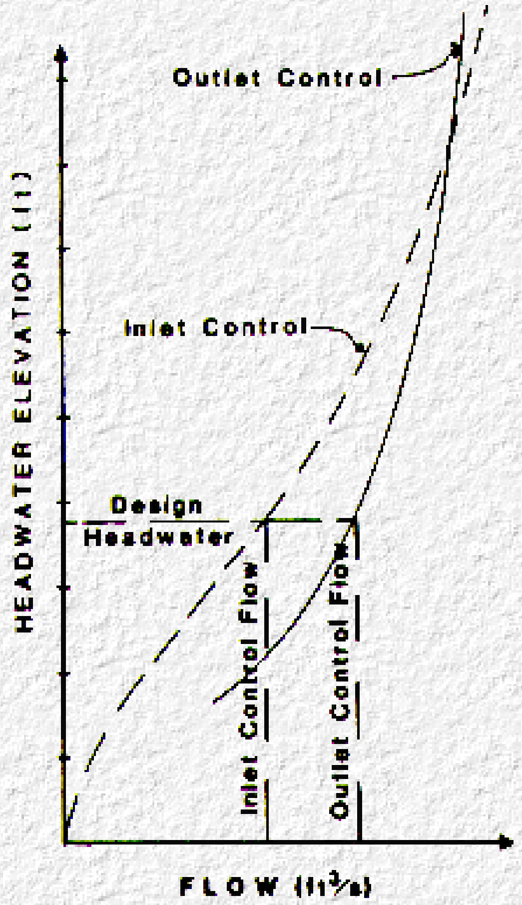


Figure 2.9.3. Culvert Performance Curve

The culvert outlet performs the basic function of releasing water to the channel without excessive erosion. Depending upon the amount of excess energy to be dissipated concrete transitions, baffled outlets, baffled apron drops, energy dissipators or drops and chutes with stilling pools can be designed. Concrete or riprap transitions should be used in preference to other structures to avoid debris and sediment problems. Energy dissipation structures at outlets should be avoided whenever excess energy can be dissipated in the culvert.

Possible aggradation or degradation at culvert crossings must be carefully examined in the design of culverts. An adequate system of culvert design involves passing drainage water and sediment from a natural regime condition upstream of the channel crossing without upsetting the delicate balance between hydraulics and sediment transport flow. Inadequate design can cause either scour or deposition on either side of the crossing. This is more of a local phenomenon that should not be confused with an overall change in stream morphology produced by some outside factor separate from the construction of the culvert system.

The factors affecting the stream morphology are, similar to bridge crossings, hydraulic and geometric parameters such as slope, width, depth, velocity, angle of attack, stream bed and bank material. Factors specifically related to culvert design are headwater elevation, tailwater elevation, and channel invert elevations at the crossing. Changes in these parameters produce changes in the entrance and exit conditions at the crossing giving rise to aggradation and degradation problems. The greater danger produced by aggradation is a partial plugging of the culvert opening resulting in a damming effect and increasing the magnitude and frequency of flooding upstream of the structure. Degrading stream reaches affect culvert systems by reducing their structural stability. General stream bed degradation can undermine the foundations of culvert systems to complete failure.

2.9.3 Computer Programs for the Analysis of Culvert Flows

Computer programs written to assist engineers in making hydraulic analyses of culverts for highway drainage are available. Hydraulic analysis of circular culverts, pipe-arch culverts and box culverts can be performed using respectively the Bureau of Public Roads programs, HY-1, HY-2, HY-3 or HY-6 (FHWA Publications). These existing programs determine the cross sectional details of the culvert. A backwater routine for part-full flow under outlet control conditions defines the free-water surface. The output data include the discharge, the number, width and height of barrels, the headwater and the outlet velocity. These programs will be superseded by the FHWA Culvert Analysis Program (HY-8) in preparation.

The FHWA Culvert Analysis Program (HY-8) is being developed by Pennsylvania State University in cooperation with FHWA. The effort is funded under the Rural Technical Assistance Program (RTAP) which is administered by the FHWA National Highway Institute (NHI). The original contract was to extend the capabilities of a basic program developed by FHWA for the Apple II microcomputer. The contract was later modified to have the program developed for the IBM-PC. The software package has been structured to be self-contained and requires no users manual. This facilitates its use by roadway design squads to design simple culverts. However, the knowledgeable hydraulic engineer will also find the software package very useful because it contains many advanced features.

The program features are listed below:

1. Procedures - FHWA HDS 5, Hydraulic Design of Highway Culverts is the reference for the procedures used in the software package;
2. Approach - Programs are structured to analyze or review user selected variables. This approach was chosen so that the user would not be constrained by the design philosophy of the program and so that design alternatives could be easily compared;
3. Shapes - Circular, box, elliptical, and pipe-arch with constant "n" and user defined shape with different "n" for top and bottom;
4. Inlets - The user selects inlet edge condition from a menu which is consistent with the selected shape; an inlet depression may be provided. If a circular or box shape was selected, the user may select either a circular or rectangular side-tapered inlet or a slope-tapered inlet;
5. Number - The user may choose any number of culverts with the same size, shape, inlet, and "n" or up to six independent culverts; in both cases the culverts share the same headwater pool, tailwater pool, and roadway;
6. Site Data - The station and elevation of the culvert inverts can be entered or will be calculated if embankment slope and toe data is entered;
7. Discharge - User chooses maximum discharge for 11 point performance (rating) curve. The minimum discharge default is 0, but can be changed;
8. Tailwater - User defines a rectangular, trapezoidal, or triangular channel; provides up to 15 cross-section points or 11 rating curve points; or enters a constant tailwater elevation. The rating curve plot can be displayed;
9. Overtopping - If a roadway profile cross-section is provided, flow over the roadway weir will be balanced with the flow through the culverts;
10. Output - A performance curve table is provided which contains, for each discharge: headwater and tailwater elevation, inlet and outlet control headwater depths, tailwater depth, and outlet velocity; at the option of the user, both inlet and outlet control performance curves

can be displayed;

11. Routing - A culvert file and an inflow hydrograph file name may be provided with upstream topographic data and the program will provide both a table and a plot of the outflow hydrograph; if a hydrograph file is not available, it can be generated using the built-in hydrology package;
12. Files - Data is stored on a data disk in relational sub-directories which are keyed to a MS-DOS name provided by the user; and
13. Editing - Existing file data is loaded to a data summary screen and can be edited or run; if a change is desired, the user selects the data to revise: site, culvert, discharge, tailwater, or overtopping; the existing data is then displayed in the data entry screens and can be edited.

2.10 Roadway Overtopping and Low Water Stream Crossings

2.10.1 Roadway Overtopping

Roadway overtopping will begin as the headwater rises to the elevation of the lowest point of the roadway. This type of flow is similar to flow over a broad crested weir. The length of the weir can be taken as the horizontal length across the roadway. The flow across the roadway is calculated from the broad crested weir equation

$$Q_o = k_t C_r L_s (HW_r)^{1.5} \qquad (2.10.1)$$

where

Q_o = is the overtopping discharge in ft^3/s

C_r = is the overtopping discharge coefficient

HW_r = is the flow depth above the roadway in ft

k_t = is the submergence factor

L_s = is the length of the roadway crest along the roadway in ft

The charts in Figure 2.10.1 indicate how to evaluate the correction factors k_t and C_r.

If the elevation of the roadway crest varies, for instance where the crest is defined by a roadway sag vertical curve, the vertical curve can be approximated as a series of horizontal segments. The flow over each is calculated separately and the total flow across the roadway is the sum of the incremental flows for each segment (Figure 2.10.2).

The total flow across the roadway then equals the sum of the roadway overflow plus the culvert flow. A trial and error procedure is necessary to separate the amount of water passing through the culvert from the amount overtopping the roadway. Performance curves must then include both culvert flow and road overflow.

The design example (Problem 2.7, in Appendix 2) illustrates this trial and error procedure.

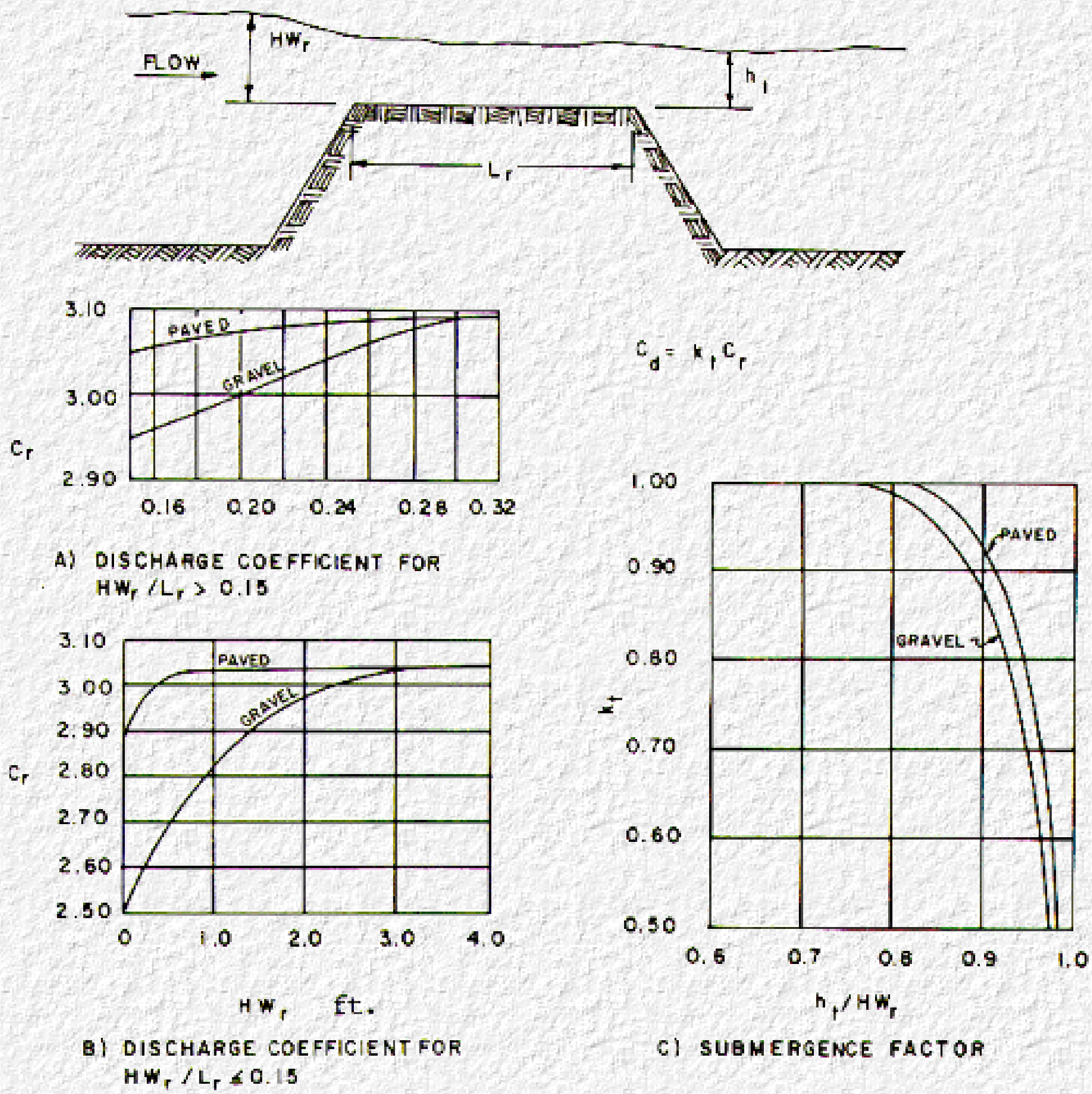
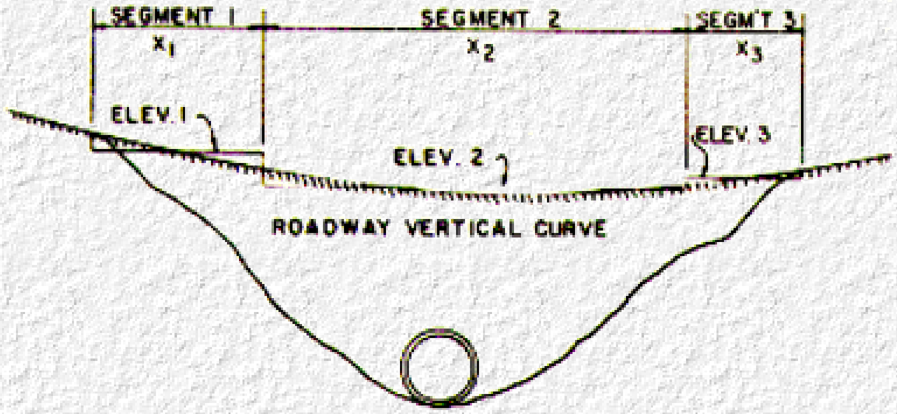
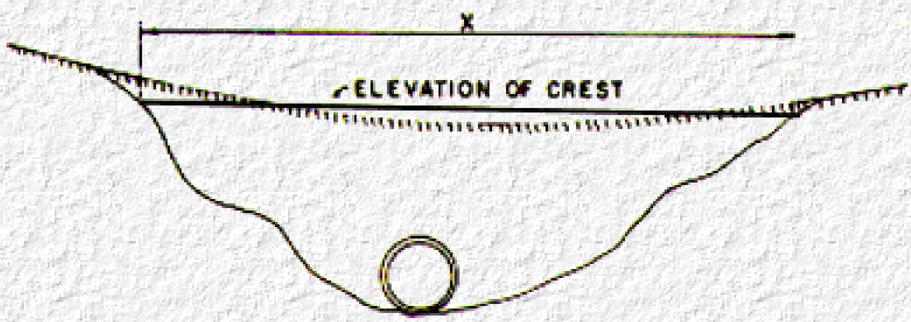


Figure 2.10.1. Discharge Coefficient for Roadway Overtopping (after Petersen, 1986)



A. METHOD I - SUBDIVISION INTO SEGMENTS



B. METHOD 2 - USE OF A SINGLE SEGMENT

Figure 2.10.2. Weir Crest Length Determinations for Roadway Overtopping

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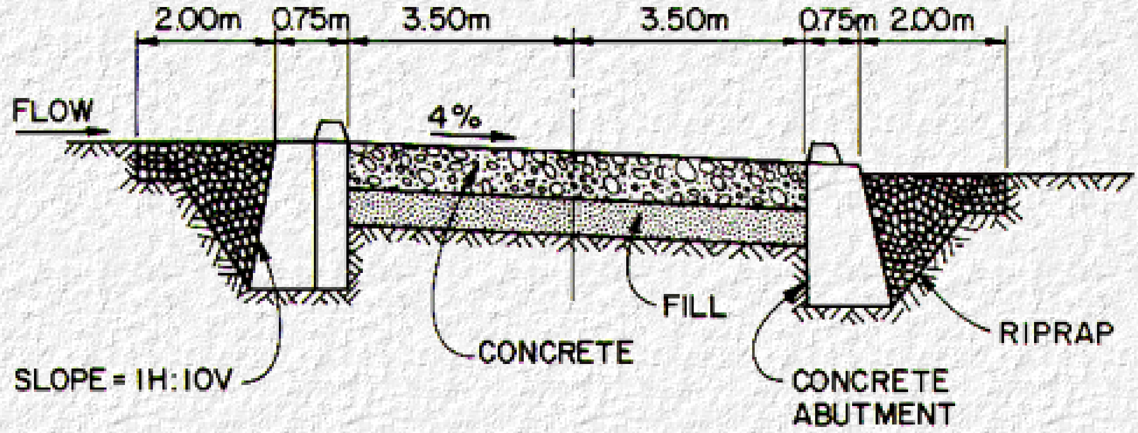
2.10.2 Low Water Stream Crossing

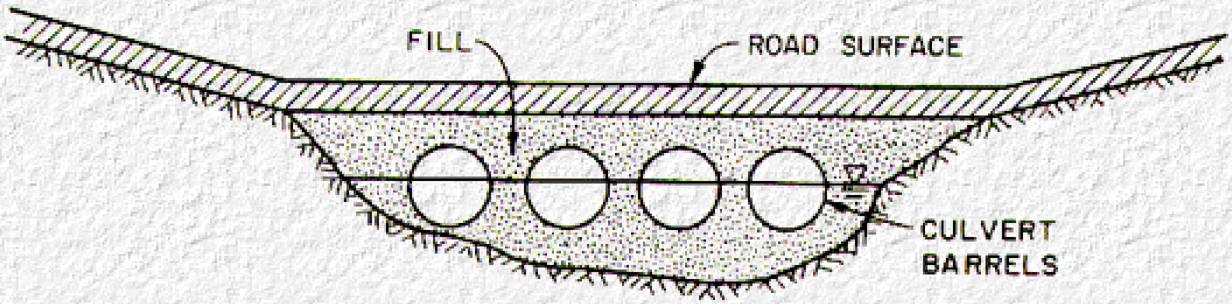
Streams in arid and semi-arid zones are dry most of the year and carry short flashy floods. The frequency and duration of these floods and the relatively high peak discharges may not justify the construction of large expensive bridge structures. Low structures which are overtopped during floods without causing damages are very economical compared to conventional bridges. A low water stream crossing (LWSC) is a cross drainage structure with a low roadway profile to facilitate the passage of floods.

Depending on the design flood, valley characteristics, traffic density, duration of flood flow, flood damages and repair costs, three types of LWSC illustrated in Figure 2.10.3 are considered:

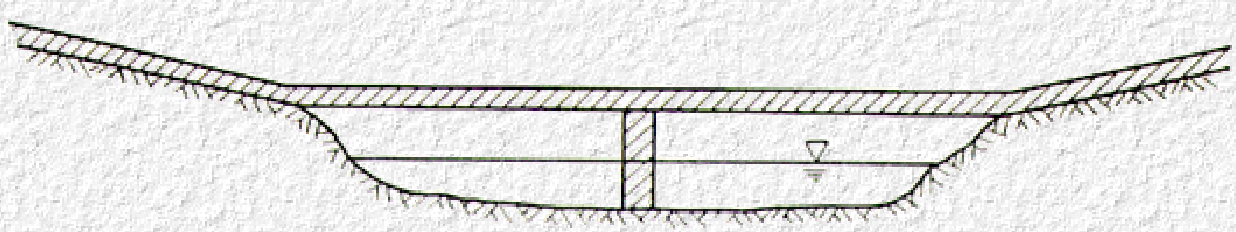
1. Ford or Dip - is a natural stream bed leveled and paved; they are adopted when the drainage crossing is dry most of the year;
2. Vented Ford - with pipes or vents to provide passage for flow depth in excess of 4 - 6 inches; and
3. Low Water Bridges - have a bridge deck structure in place of vent pipes; they have better conveyance capacity than vents and are preferred to vented Ford when the stream carries large amount of debris.



a) FORD ON UNSCOURABLE BED



b) VENTED FORD



c) LOW WATER STREAM CROSSING

Figure 2.10.3. Types of Low Water Stream Crossings

Go to Chapter 3

Chapter 3 : HIRE
Fundamentals of Alluvial Channel Flow

Go to Chapter 4

3.1 Introduction

Most streams that a highway will cross or encroach upon are alluvial. That is, the rivers are formed in cohesive or non-cohesive materials that have been, and can still be, transported by the stream. The non-cohesive material generally consists of silt, sand, gravel, or cobbles, or any combination of these sizes. Silt generally is not present in appreciable quantities in streams having non-cohesive boundaries. Cohesive material consists of clays (sizes less than 0.004 mm) forming a binder with silts and sand. Because of the electro-chemical bonding between clay particles, clays are more resistant to erosion than silts.

In alluvial rivers, bed configuration and resistance to flow are a function of the flow and can change to increase or decrease the water surface level. The river channel can shift its location so that the crossing or encroachment is unfavorably located with respect to the direction of flow. The movable boundary of the alluvial river thus adds another dimension to the design problem and can compound environmental concerns. Therefore, the design of highway crossings and encroachments in the river environment requires knowledge of the mechanics of alluvial channel flow.

This chapter presents the fundamentals of alluvial channel flow. It covers properties of alluvial material, methods of measuring properties of alluvial materials, flow in sandbed channels, prediction of bed forms, Manning's n for sandbed and other natural streams, how bed-form changes affect highways in the river environment, beginning of motion, sediment transport, flow in coarse-material streams and physical and computer modeling of alluvial channel flow. These fundamentals of alluvial channel flow are used in later chapters to develop design considerations for highway crossings and encroachments in river environments.

3.2 Sediment Properties and Measurement Techniques

A knowledge of the properties of the bed material particles is essential, as they indicate the behavior of the particles in their interaction with the flow. Several of the important bed material properties are discussed in the following sections.

3.2.1 Particle Size

Of the various sediment properties, physical size has by far the greatest significance to the hydraulic engineer. The particle size is the most readily measured property, and other properties such as shape, fall velocity and specific gravity tend to vary with size in a roughly predictable manner. In general, size represents a sufficiently complete description of the sediment particle for many practical purposes.

Particle size D_s may be defined by its volume, diameter, weight, fall velocity, or sieve mesh size. Except for volume, these definitions also depend on the shape and density of the particle. The following definitions are commonly used to describe the particle size:

1. Nominal diameter - The diameter of a sphere having the same volume as the particle.
2. Sieve diameter - The diameter of a sphere equal to the length of the side of a square sieve opening through which measured quantities (by weight) of the sample will pass. As an approximation, the sieve diameter is equal to the nominal diameter.
3. Sedimentation diameter - The diameter of a sphere with the same fall velocity and specific

gravity as the particle in the same fluid under the same conditions.

4. Standard fall diameter - The diameter of a sphere that has a specific gravity of 2.65 and also has the same terminal settling velocity as the particle when each is allowed to settle alone in quiescent, distilled water of infinite extent and at a temperature of 24ºC.

In general, sediments have been classified into boulders, cobbles, gravels, sands, silts, and clays on the basis of their nominal or sieve diameters. The size range in each general class is given in Table 3.2.1. The non-cohesive material generally consists of silt (0.004 - 0.062 mm), sand (0.062 - 2.0 mm), gravel (2.0 - 64mm), or cobbles (64 - 250 mm).

Table 3.2.1 Sediment Grade Scale

Size				Approximate Sieve Mesh Openings per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
4000 - 2000	. . . .	. . . . . .	160 - 80	. . .	. . . . . .	Very large boulders
2000 - 1000	. . . .	. . . . . .	80 - 40	. . .	. . . . . .	Large boulders
1000 - 500	. . . .	. . . . . .	40 - 20	. . .	. . . . . .	Medium boulders
500 - 250	. . . .	. . . . . .	20 - 10	. . .	. . . . . .	Small boulders
250 - 130	. . . .	. . . . . .	10 - 5	. . .	. . . . . .	Large cobbles
130 - 64	. . . .	. . . . . .	5 - 2.5	. . .	. . . . . .	Small cobbles
64 - 32	. . . .	. . . . . .	2.5 - 1.3	. . .	. . . . . .	Very coarse gravel
32 - 16	. . . .	. . . . . .	1.3 - 0.6	. . .	. . . . . .	Coarse gravel
16 - 8	. . . .	. . . . . .	0.6 - 0.3	2½	. . . . . .	Medium gravel
8 - 4	. . . .	. . . . . .	0.3 - 0.16	5	5	Fine gravel
4 - 2	. . . .	. . . . . .	0.16 - 0.08	9	10	Very fine gravel
2 - 1	2.00 - 1.00	2000 - 1000	. . . . . .	16	18	Very coarse sand
1 - 1/2	1.00 - 0.50	1000 - 500	. . . . . .	32	35	Coarse sand
1/2 - 1/4	0.50 - 0.25	500 - 250	. . . . . .	60	60	Medium sand
1/4 - 1/8	0.25 - 0.125	250 - 125	. . . . . .	115	120	Fine sand
1/8 - 1/16	0.125 - 0.062	125 - 62	. . . . . .	250	230	Very fine sand
1/16 - 1/32	0.062 - 0.031	62 - 31	. . . . . .			Coarse silt
1/32 - 1/64	0.031 - 0.016	31 - 16	. . . . . .			Medium silt
1/64 - 1/128	0.016 - 0.008	16 - 8	. . . . . .			Fine silt
1/128 - 1/256	0.008 - 0.004	8 - 4	. . . . . .			Very fine silt
1/256 - 1/512	0.004 - 0.0020	4 - 2	. . . . . .			Coarse clay
1/512 - 1/1024	0.0020 - 0.0010	2 - 1	. . . . . .			Medium clay
1/1024 - 1/2048	0.0010 - 0.0005	1 - 0.5	. . . . . .			Fine clay
1/2048 - 1/4096	0.0005 - 0.0002	0.5 - 0.24	. . . . . .			Very fine clay

The boulder class (250 - 4000 mm) is generally of little interest in sediment problems. The cobble and gravel class plays a considerable role in the problems of local scour and resistance to flow and to a lesser extent in bed load transport. The sand class is one of the most important in alluvial channel flow. The silt and clay class is of considerable importance in the evaluation of stream sediment loads, bank stability and problems of seepage and consolidation.

3.2.2 Particle Shape

Generally speaking, shape refers to the overall geometrical form of a particle. Sphericity is defined as the ratio of the surface area of a sphere of the same volume as the particle to the actual surface area of the particle. Roundness is defined as the ratio of the average radius of curvature of the corners and edges of a particle to the radius of a circle inscribed in the maximum projected area of the particle. However, because of simplicity and effectiveness of correlation with the behavior of particles in flow, the most commonly used parameter to describe particle shape is the Corey shape factor, S_p, defined as

$$S_p = \frac{\ell_c}{\sqrt{\ell_a \ell_b}} \qquad (3.2.1)$$

where ℓ_a, ℓ_b, and ℓ_c are the dimensions of the three mutually perpendicular axes of a particle ℓ_a the longest; ℓ_b the intermediate; and ℓ_c the shortest axis.

3.2.3 Fall Velocity

The prime indicator of the interaction of sediments in suspension within the flow is the fall velocity of sediment particles. <u>The fall velocity of a particle is defined as the velocity of that particle falling alone in quiescent, distilled water of infinite extent.</u> In most cases, the particle is not falling alone, and the water is not distilled or quiescent. Measurement techniques are available for determining the fall velocity of groups of particles in a finite field in fluid other than distilled water. However, little is known about the effect of turbulence on fall velocity.

<u>A particle falling at terminal velocity in a fluid is under the action of a driving force due to its buoyant weight and a resisting force due to the fluid drag.</u> Fluid drag is the result of either the tangential shear stress on the surface of the particle, or a pressure difference on the particle or a combination of the two forces. The fluid drag on the falling particle F_D is given by the drag equation

$$F_D = C_D A_s \rho \omega^2/2 \qquad (3.2.2)$$

The buoyant weight of the particle W_s is

$$W_s = (\rho_s - \rho) gV_p \qquad (3.2.3)$$

Where

C_D = coefficient of drag

ω = terminal fall velocity of the particle

A_s = projected area of the particle normal to the direction of flow

ρ = fluid density

ρ_s = particle density

g = acceleration due to gravity

V_p = volume of the particle

The area and volume can be written in terms of the characteristic diameter of the particle D_s or

$$A_s = K_1 D_s^2 \qquad (3.2.4)$$

and

$$V_p = K_2 D_s^3 \qquad (3.2.5)$$

Where the coefficients K_1 and K_2 depend on the shape of sediment particles. For example, $K_1 = \pi/4$ and $K_2 = \pi/6$ for spherical particles. If the particle is falling at its terminal velocity, $F_D = W_s$

$$(\rho_s - \rho) g V_p = C_D A_s \rho\, \omega^2/2 \qquad (3.2.6)$$

By substituting Equation 3.2.4 and Equation 3.2.5 into Equation 3.2.6, the expression

$$\omega^2 = \frac{2}{C_D}\frac{K_2}{K_1}\left(\frac{\rho_s}{\rho} - 1\right) g D_s \qquad (3.2.7)$$

is obtained.

Four dimensionless variables describing fall velocity result from dimensional analysis of Equation 3.2.7

$$f\left(\frac{\omega^2}{g D_s}, C_D, \frac{\rho_s}{\rho}, \frac{K_2}{K_1}\right) = 0 \qquad (3.2.8)$$

The drag coefficient C_D is dependent on the particle Reynolds number ($R_e = \rho\, \omega D_s/\mu$) the shape and the surface texture of the particle. The ratio K_2/K_1 is usually replaced by the Corey shape factor S_p (Equation 3.2.1).

The relation between the fall velocity of particles and the other variables are given in Figure 3.2.1 and Figure 3.2.2.

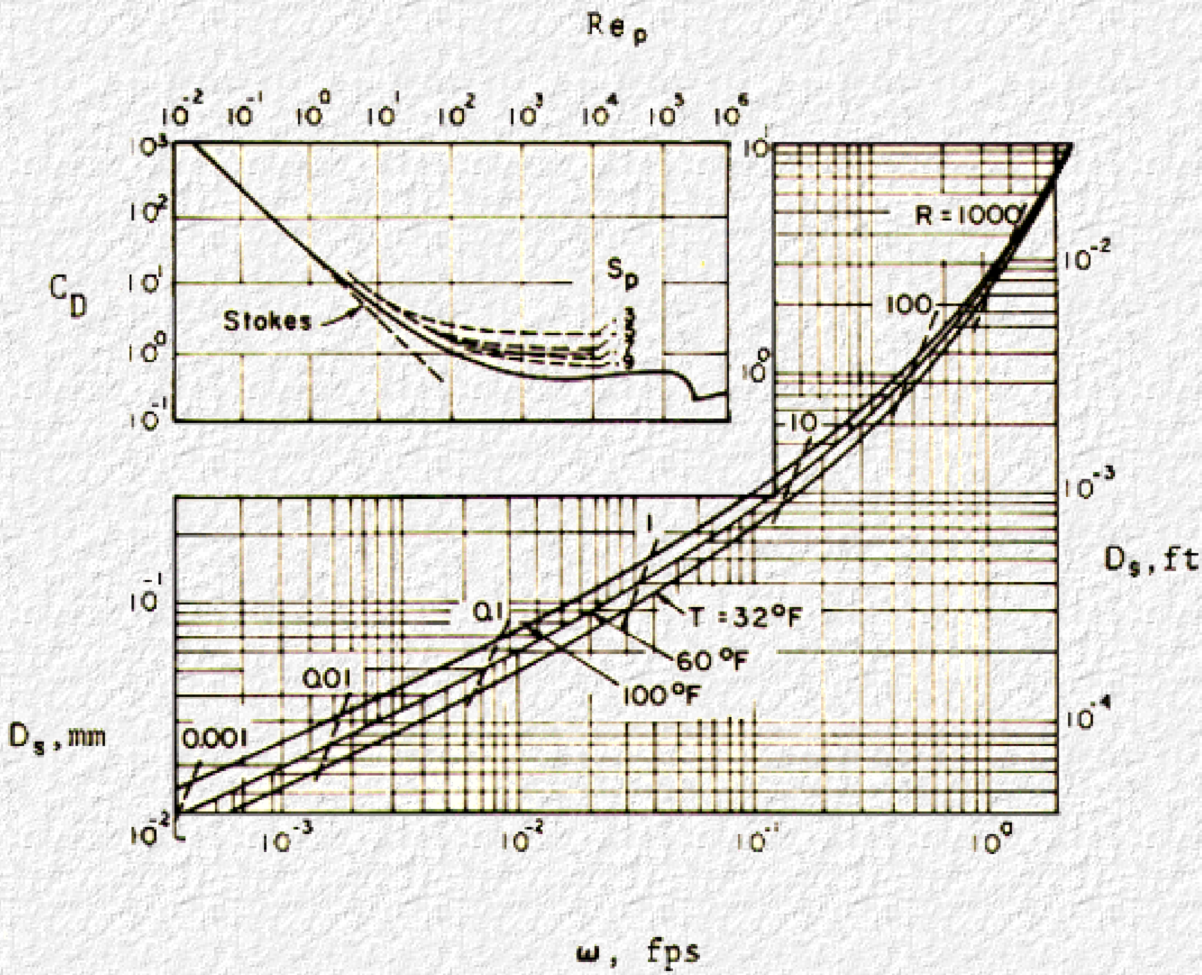


Figure 3.2.1. Drag Coefficient C_D vs. Particle Reynolds Number Re_p for Spheres and Natural Sediments with Shape Factors S_p Equal to 0.3, 0.5, 0.7, and 0.9. Also, Sediment Diameter D_s vs. Fall Velocity ω and Temperature T°

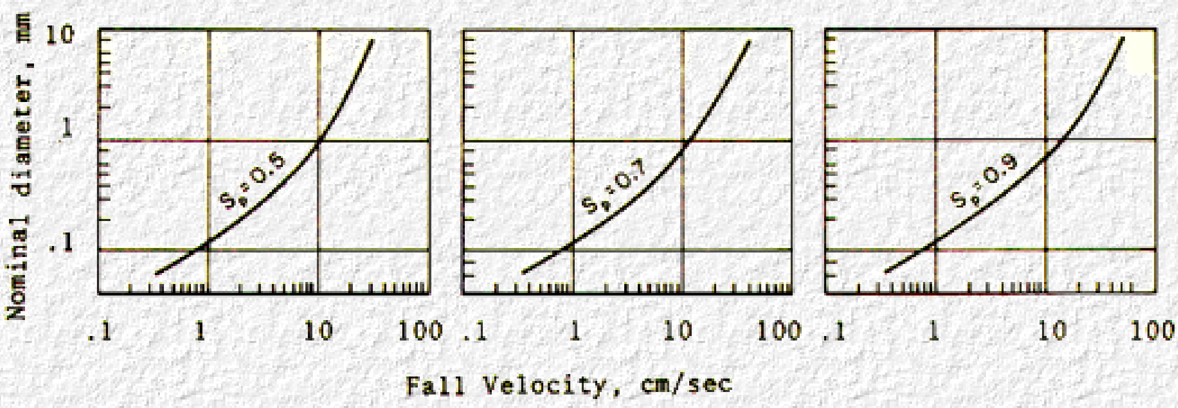


Figure 3.2.2. Nominal Diameter vs. Fall Velocity (Temperature = 24°C)

3.2.4 Sediment Size Distribution

Four methods of obtaining sediment size distribution are described herein: sieve analysis, visual accumulation tube analysis, pebble count method, and pipette analysis. Each method for size distribution analysis is appropriate for only a particular range of particle sizes (see Table 3.2.2). All together the four methods provide a means of obtaining particle size distributions for most bed material samples.

Table 3.2.2 Guide to Size Range for Different Types of Size Analysis

	Size Range	Analysis Concentration (mg/1)	Quantity of Sediment (g) or Pebbles
Sieves	0.062 - 32 mm	------	< 100 - 500
VA tube	0.062 - 2 mm	------	0.05 - 15.0
Pipette	0.002 - 0.062 mm	2,000 - 5,000	1.0 - 5.0
Pebble count	0.5 - 40 in.	------	100 pebbles

If the sediment sample to be analyzed (bed material or suspended sediment) has considerable fine material (D_s < .062 mm) it must be separated prior to analysis. To separate the coarser from the finer sediment, the sediment should be wet-sieved using distilled water and a 250-mesh (0.062 mm) sieve. The material passing through the sieve can be analyzed by pipette analysis if further breakdown of the fine sediment is desired, or dried and included as percent finer than 0.062 mm with the analysis of the coarser material. If it is going to be dry-sieved, the material retained on the sieve is oven dried for one hour after all visible water has been evaporated. If the material is to be analyzed by wet-sieving or with the accumulation tube it is not dried.

- Sieves - Size distribution in the sand and gravel range is generally determined by passing the sample through a series of sieves of mesh size ranging from 4 mm to 0.062 mm. A minimum of about 100 grams of sand is required for an accurate sieve analysis. More is required if the sample contains particles of 1.0 mm or larger. Standard methods employed in soil mechanics are suitable for determining the sieve sizes of sand and gravel sediment samples.

- The visual accumulation (VA) tube is used for determining the size distribution of the sand fraction of sediment samples (0.062 mm $\leq D_s \leq$ 2.0 mm). It is a fast, economical, and accurate means of determining the fall velocity or fall diameter of the sediment. The equipment for the visual accumulation tube analysis consists of: (1) a glass funnel about 25 cm long; (2) a rubber tube connecting the funnel and the main sedimentation tube, with a special clamping mechanism serving as a "quick acting" valve; (3) glass sedimentation tubes having different sized collectors; (4) a tapping mechanism that strikes against the glass tube and helps keep the accumulation of sediment uniformly packed; (5) a special recorder consisting of a cylinder carrying a chart that rotates at a constant rate and a carriage that can be moved vertically by hand on which is mounted a recording pen and an optical instrument for tracking the accumulation; and (6) the recorder chart which has a printed form incorporating the fall diameter calibration.

  In the visual accumulation tube method, the particles start falling from a common source and become stratified according to settling velocities. At a given instant, the particles coming to rest at the bottom of the tube are of one "sedimentation size" and are finer than particles that have previously settled out and are coarser than those remaining in suspension.

  It has been shown that particles of a sample in the visual tube settle with greater velocities than the same particles falling individually because of the effect of mutual interaction of the particles. The visual accumulation tube apparatus is calibrated to account for the effects of this mutual interaction and the final results are given in terms of the standard fall diameter of the particles.

  The visual accumulation tube method may not be suitable for some streams that transport large quantities of organic materials such as root fibers, leaf fragments, and algae. Also, extra care is needed when a stream transports large quantities of heavy or light minerals such as taconite or coal. The method is explained in detail by Guy (1969).

- The pebble count method is used to obtain the size distribution of coarse bed materials (gravel and cobbles) which are too large to be sieved. Very often the coarser material is underlain by sands. Then the underlying sands are analyzed by sieving. The two classes of bed material are either combined into a single distribution or used separately. The large material sizes are measured in situ by laying out a square grid. Within the grid, all the particle sizes are measured and counted by size intervals. For large samples a random selection of particles in the various classes is appropriate to develop frequency histograms of sediment sizes.

- A square-surface sample is obtained by picking up and counting all the surface pebbles in each predetermined size class within a small enclosed area of the bed. The area is taken to be representative of the whole channel bed.

  The pebble count method entails measurement of randomly selected particles in the field, often under difficult conditions. Therefore, use of the Zeiss Particle-Size Analyzer should be considered (Ritter and Helley, 1968). For this method, a photograph of the stream bed is made, preferably at low flow, with a 35 mm camera supported by a tripod about 2 m above the stream bed, the height depending on the size of the bed material. A reference scale, such as a steel tape or a surveyor's rod must appear in the photograph. The photographs are printed on the thinnest paper available. An iris diaphragm, illuminated from one side, is imaged by a lens onto the plane of a Plexiglass plate. By adjusting the iris diaphragm the diameter of the sharply defined circular light spot appearing on the photograph can be changed and its area made equal to that of the individual particles. As the different diameters are registered, a puncher marks the counted particle on the photograph. An efficient operation can count up to 1,000 particles in 30 minutes.

  In the line sampling method of pebble count sampling, a line is laid out or placed either across or along the stream. Particles are picked at random intervals along the line and measured. The measured particles are classified as to size or weight and a percent finer curve or table is prepared. Usually 100 particles are sufficient to give an accurate classification of the size distribution of coarse materials.
- The pipette method of determining gradation of sizes finer than 0.062 mm is one of the most widely accepted techniques utilizing the Oden theory and the dispersed system of sedimentation. The upper size limit of sediment particles which settle in water according to Stokes is about 1/16 mm or 0.062 mm. This corresponds to the lower size limit which can be determined readily by sieves. This size is the division between sand and silt (Table 3.2.1) and is an important division in many phases of sediment phenomena.

  The fundamental principle of the pipette method is to determine the concentration of a suspension in samples withdrawn from a predetermined depth as a function of settling time. Particles having a settling velocity greater than that of the size at which separation is desired will settle below the point of withdrawal after elapse of a certain time. The time and depth of withdrawal are predetermined on the basis of Stokes law.

  Satisfactory use of the pipette method requires careful and precise operation to obtain maximum accuracy in each step of the procedure. Also, for routine analysis, special apparatus can be set up for the analysis of a large number of samples. A complete description of a laboratory setup and procedure for this method is given by Guy (1969).

  The presentation of sediment size analysis is made with:
- Frequency Curves - A histogram is a graphical representation of the number, weight, or volume percentage of items in given class intervals. An example of a histogram is shown in Figure 3.2.3a. The abscissa scale represents the class intervals, usually in geometric progression, and the ordinate scale represents either actual concentration or percent (by number, volume, or weight) of the total sample contained in each class interval. If the class intervals are small, the shape of the histogram will approach a continuous curve. The successive sizes employed in the size analysis of sediment are usually in ratios of 2 or $\sqrt{2}$.

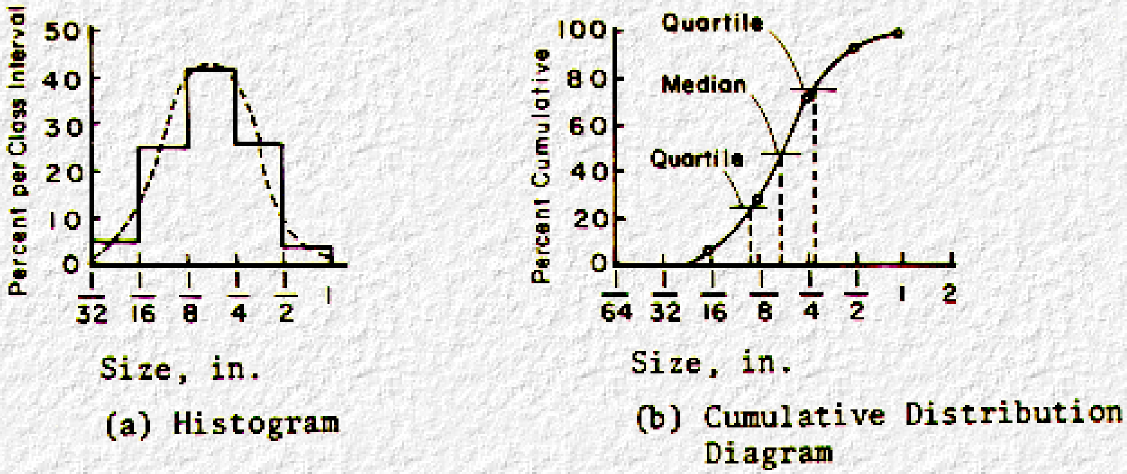


Figure 3.2.3. Frequency Curves

When the ordinates of successive classes are added and plotted against the upper limit of the size class, the cumulative distribution diagram is obtained (see Figure 3.2.3b.) In this diagram, the abscissa scale (usually logarithmic) represents the intervals of the size scale and the ordinate scale is the cumulative percent by weight of the sample up to (or percent finer than) the size in question.

- Quartile and Moment Measures - In a size frequency distribution curve, it is possible to choose certain particle sizes as representing significant values, such as particles just larger than one-fourth of the distribution D_{25} (the first quartile), and particles just larger than three-fourths of the distribution D_{75} (the third quartile). Measures of spread are based on differences or ratios between the two quartiles. Quartile measures are confined to the central half of the frequency distribution and the values obtained are not influenced by larger or smaller sizes. Quartile measures are very readily computed, and most of the data may be obtained directly from the cumulative curve by graphic means.

  In contrast to quartile measures, moment measures are influenced by each individual size class in the distribution. The first moment of a frequency curve is its center of gravity and is called the arithmetic mean and is the average size of the sediment. The second moment is a measure of the average spread of the curve and is expressed as the standard deviation of the distribution.

  Commonly the size distribution of natural sediments plots as a straight line on log probability paper. If this is true, then a natural sediment is completely described by the median diameter (the size of sediment of which 50% is finer) and the slope of the cumulative frequency line on log probability paper. The slope of this line is proportional to the spread of the size distribution in a sediment sample. It is computed with the expression

$$G = \frac{1}{2}\left[\frac{D_{50}}{D_{16}} + \frac{D_{84}}{D_{50}}\right] \qquad (3.2.9)$$

Where

G = gradation coefficient

D_x = the sediment diameter particle of which x percent of sample is finer.

An example problem for determining sediment properties is given in Appendix 3, Problem 3.1.

3.2.5 Specific Weight

Specific weight is weight per unit volume. In the English system of units, specific weight is usually expressed in units of pounds per cubic foot and in the metric system, in grams per cubic centimeter. In connection with granular materials such as soils, sediment deposits, or water sediment mixtures, the specific weight is the weight of solids per unit volume of the material including its voids. The measurement of the specific weight of sediment deposits is determined simply by measuring the dry weight of a known volume of the undisturbed material.

3.2.6 Porosity

The porosity of granular materials is the ratio of the volume of void space to the total volume of an undisturbed sample. To determine porosity, the volume of the sample must be obtained in an undisturbed condition. Next, the volume of solids is determined either by liquid displacement or indirectly from the weight of the sample and the specific gravity of material. The void volume is then obtained by subtracting the volume of solids from the total volume.

3.2.7 Cohesion

Cohesion is the force by which particles of clay are bound together. This force is the result of ionic attraction among individual particles, and is a function of the type of mineral, particle spacing, salt concentration in the fluid, ionic valence, and hydration and swelling properties of the constituent minerals.

Clays are alumino-silicate crystals composed of two basic building sheets, the tetrahedral silicate sheet and the octahedral hydrous aluminum oxide sheet. Various types of clays result from different configurations of these sheets. The two main types of clays are kaolinite and montmorillonite. Kaolinite crystals are large (70 to 100 layers thick), held together by strong hydrogen bonds, and are not readily dispersible in water. Montmorillonite crystals are small (3 layers thick) held together by weak bonds between adjacent oxygen layers and are readily dispersible in water into extremely small particles.

Several laboratory and field measurement techniques are available for determining the magnitude of cohesion, or shear strength, of clays. Among these, the vane shear test, which is performed in the field is one of the simplest. The vane is forced into the ground and then the torque required to rotate the vane is measured. The shear strength is determined from the torque required to shear the soil along the vertical and horizontal edges of the vane.

3.2.8 Angle of Repose

The angle of repose is the maximum slope angle upon which non-cohesive material will reside without moving. It is a measure of the intergranular friction of the material. The angle of repose for dumped granular material is given in Figure 3.2.4.

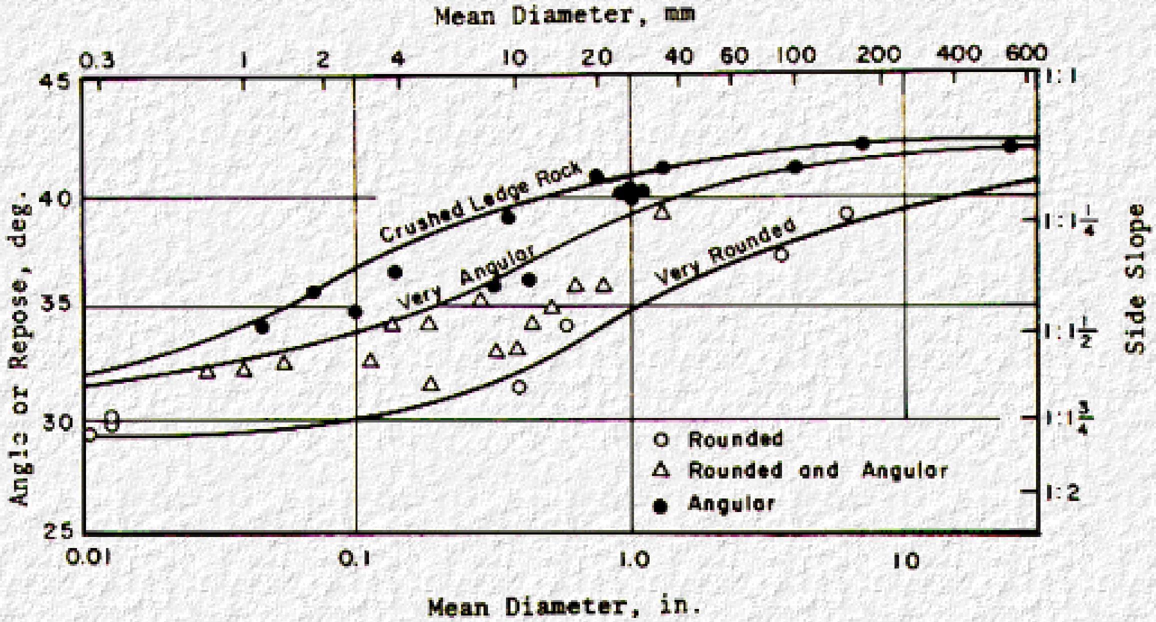


Figure 3.2.4. Angle of Repose of Non-Cohesive Materials

3.3 Flow in Sandbed Channels

Most larger streams flow on sandbeds for the greater part of their length. Thus there are potentially many more opportunities for highway crossings or encroachments on sandbed streams than in cohesive or gravel streams. In sandbed rivers, the sand material is easily eroded and is continually being moved and shaped by the flow. The mobility of the sandbed creates problems for the safety of any structure placed in or over the stream, for the protection of private property along these streams, and in the preservation and enhancement of the stream environment.

The interaction between the flow of the water-sediment mixture and the sandbed creates different bed configurations which change the resistance to flow and rate of sediment transport. The gross measures of channel flow, such as the flow depth, river stage, bed elevation and flow velocity, change with different bed configurations. In the extreme case, the change in bed configuration can cause a three-fold change in resistance to flow and a 10-to-15 fold change in concentration of bed sediment transport. For a given discharge and channel width, a three-fold increase in Manning's n results in a doubling of the flow depth.

The interaction between the flow and bed material and the interdependency among the variables makes the analysis of flow in alluvial sandbed streams extremely complex. However, with an understanding of the different types of bed forms that may occur and a knowledge of the resistance to flow and sediment transport associated with each bed form, the knowledgeable river engineer can better analyze alluvial channel flow.

3.3.1 Regimes of Flow in Alluvial Channels

The flow in alluvial channels is divided into lower and upper flow regimes separated by a transition zone (Simons and Richardson, 1963). These two flow regimes are characterized by similarities in the shape of the bed configuration, mode of sediment transport, process of energy dissipation, and phase relation between the bed and water surfaces. The two regimes and their associated bed configurations shown in Figure 3.3.1 are:

Lower flow regime:	(1) ripples; (2) dunes with ripples superposed; and (3) dunes.

Transitional Flow Regime:	The bed roughness ranges from dunes to plane bed or antidunes.
Upper Flow Regime:	(1) plane bed; (2) antidunes, a) with standing waves, b) with breaking antidunes; and (3) chutes and pools.

- Lower Flow Regime - In the lower flow regime, resistance to flow is large and sediment transport is small. The bed form is either ripples or dunes or some combination of the two. The water-surface undulations are out of phase with the bed surface, and there is a relatively large separation zone downstream from the crest of each ripple or dune. The most common mode of bed material transport is for the individual grains to move up the back of the ripple or dune and avalanche down its face. After coming to rest on the downstream face of the ripple or dune, the particles remain there until exposed by the downstream movement of the dunes; they repeat this cycle of moving up the back of the dune, avalanching, and storage. Thus, most movement of the bed material particles is in steps. The velocity of the downstream movement of the ripples or dunes depends on their height and the velocity of the grains moving up their backs.
- Transition - The bed configuration in the transition zone is erratic. It may range from that typical of the lower flow regime to that typical of the upper flow regime, depending mainly on antecedent conditions. If the antecedent bed configuration is dunes, the depth or slope can be increased to values more consistent with those of the upper flow regime without changing the bed form; or, conversely, if the antecedent bed is plane, depth and slope can be decreased to values more consistent with those of the lower flow regime without changing the bed form. Often in the transition from the lower to the upper flow regime, the dunes decrease in amplitude and increase in length before the bed becomes plane (washed-out dunes). Resistance to flow and sediment transport also have the same variability as the bed configuration in the transition. This phenomenon can be explained by the changes in resistance to flow and, consequently, the changes in depth and slope as the bed form changes. Resistance to flow is small for flow over a plane bed; so the shear stress decreases and the bed form changes to dunes. The dunes cause an increase in resistance to flow which increases the shear stress on the bed and the dunes wash out forming a plane bed, and the cycle continues. It was the transition zone, which covers a wide range of shear values, that Brooks (1958) was investigating when he concluded that a single-valued function does not exist between velocity or sediment transport and the shear stress on the bed.
- Upper Flow Regime - In the upper flow regime, resistance to flow is small and sediment transport is large. The usual bed forms are plane bed or antidunes. The water surface is in phase with bed surface except when an antidune breaks, and normally the fluid does not separate from the boundary. A small separation zone may exist downstream from the crest of an antidune prior to breaking. Resistance to flow is the result of grain roughness with the grains moving, of wave formation and subsidence, and of energy dissipation when the antidunes break. The mode of sediment transport is for the individual grains to roll almost continuously downstream in sheets one or two grain diameters thick; however, when antidunes break, much bed material is briefly suspended, then movement stops temporarily and there is some storage of the particles in the bed.

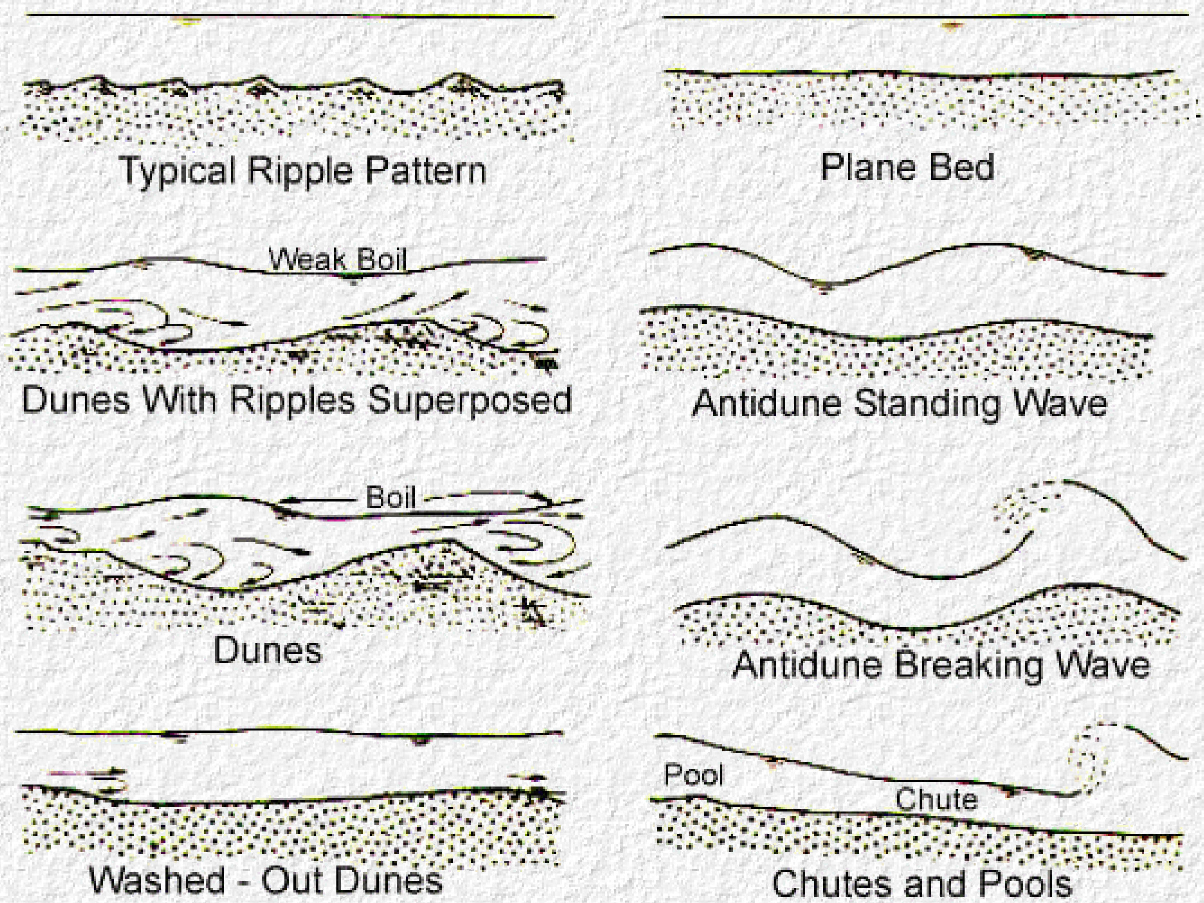


Figure 3.3.1. Forms of Bed Roughness in Sand Channels

3.3.2 Bed Configuration

The bed configurations (roughness elements) that commonly form in sand bed channels are plane bed without sediment movement, ripples, ripples on dunes, dunes, plane bed with sediment movement, antidunes, and chutes and pools. These bed configurations are listed in their order of occurrence with increasing values of stream power ($V\gamma y_o S$) for bed materials having D_{50} less than 0.6 mm. For bed materials coarser than 0.6 mm, dunes form instead of ripples after beginning of motion at small values of stream power. The relation of bed form to water surface is shown in Figure 3.3.2.

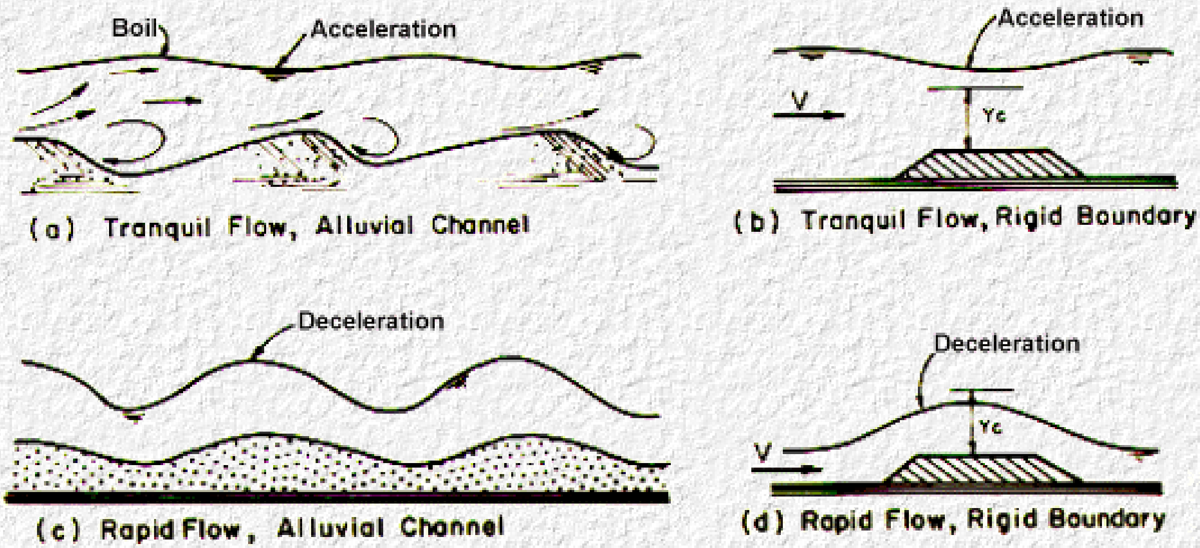


Figure 3.3.2. Relation between Water Surface and Bed Configuration

The different forms of bed-roughness are not mutually exclusive in time and space in a stream. Different bed-roughness elements may form side-by-side in a cross-section or reach of a natural stream, giving a multiple roughness; or they may form in time sequence, producing variable roughness.

Multiple roughness is related to variations in shear stress ($\gamma y_o S$) in a channel cross-section. The greater the width-depth ratio of a stream, the greater is the probability of a spatial variation in shear stress, stream power or bed material. Thus, the occurrence of multiple roughness is closely related to the width-depth ratio of the stream. Variable roughness is related to changes in shear stress, stream power, or reaction of bed material to a given stream power over time. A commonly observed example of the effect of changing shear stress or stream power is the change in bed form that occurs with changes in depth during a runoff event. Another example is the change in bed form that occurs with change in the viscosity of the fluid as the temperature or concentration of fine sediment varies over time. It should be noted that a transition occurs between the dune bed and the plane bed; either bed configuration may occur for the same value of stream power.

In the following paragraphs bed configurations and their associated flow phenomena are described in the order of their occurrence with increasing stream power.

3.3.3 Plane Bed without Sediment Movement

If the bed material of a stream moves at one discharge but not at a smaller discharge, the bed configuration at the smaller discharge will be a remnant of the bed configuration formed when sediment was moving. The bed configurations after the beginning of motion may be those illustrated in Figure 3.3.1, depending on the flow and bed material. Prior to the beginning of motion, the problem of resistance to flow is one of rigid-boundary hydraulics. After the beginning of motion, the problem relates to defining bed configurations and resistance to flow.

Plane bed without movement has been studied to determine the bed configuration that would form after beginning of motion. After the beginning of motion, for flat slopes and low velocity, the plane bed will change to ripples for sand material smaller than 0.6 mm, and to dunes for coarser material. Resistance to flow is small for a plane bed without sediment movement and is due solely to the sand grain roughness. Values of Manning's n range from 0.012 to 0.014 depending on the size of the bed material.

3.3.4 Ripples

Ripples are small triangle-shaped elements having gentle upstream slopes and steep downstream slopes. Length ranges from 0.4 ft to 2 ft and height from 0.02 ft to 0.2 ft (See Figure 3.3.1). Resistance to flow is relatively large (with Manning's n ranging from 0.018 to 0.030). There is a relative roughness effect associated with a ripple bed and the resistance to flow decreases as flow depth increases. The ripple shape is independent of sand size and at large values of Manning's n the magnitude of grain roughness is small relative to the form roughness. The length of the separation zone downstream of the ripple crest is about ten times the height of the ripple. Ripples cause very little, if any, disturbance on the water surface, and the flow contains very little suspended bed material. The bed material discharge concentration is small, ranging from 10 to 200 ppm.

3.3.5 Dunes

When the shear stress or the stream power is increased for a bed having ripples (or a plane bed without movement, if the bed material is coarser than 0.6 mm), sand waves called dunes form on the bed. At smaller shear-stress values, the dunes have ripples superposed on their backs. These ripples disappear at larger shear values, particularly if the bed material is coarse sand with $D_{50} > 0.4$ mm.

Dunes are large triangle-shaped elements similar to ripples (Figure 3.3.1). Their lengths range from two feet to many hundreds of feet, depending on the scale of the flow system. Dunes that formed in the eight-foot wide flume used by Simons and Richardson (1963) ranged from two feet to ten feet in length and from 0.2 to 1 ft in height; whereas, those described by Carey and Keller (1957) in the Mississippi River were several hundred feet long and as much as 40 ft high. The maximum amplitude to which dunes can develop is approximately the average depth. Hence, in contrast with ripples, the amplitude of dunes can increase with increasing depth of flow. With dunes, the relative roughness can remain essentially constant or even increase with increasing depth of flow.

Field observations indicate that dunes can form in any sand channel, irrespective of the size of bed material, if the stream power is sufficiently large to cause general transport of the bed material without exceeding a Froude number of unity.

Resistance to flow caused by dunes is large. Manning's n ranges from 0.020 to 0.040. The form roughness for flow with dunes is equal to or larger than the sand grain roughness.

Dunes cause large separation zones in the flow. These zones, in turn, cause boils to form on the surface of the stream. Measurements of flow velocities within the separation zone show that velocities in the upstream direction exist that are 1/2 to 1/3 the average stream velocity. Boundary shear stress in the dune trough is sometimes sufficient to form ripples oriented in a direction opposite to that of the primary flow in the channel. With dunes, as with any tranquil flow over an obstruction, the water surface is out of phase with the bed surface (See Figure 3.3.2).

3.3.6 Plane Bed with Movement

As the stream power of the flow increases further, the dunes elongate and reduce in amplitude. This bed configuration is called the transition or washed out dunes. The next bed configuration with increased stream power is plane bed with movement. Dunes of fine sand (low fall velocity) are washed out at lower values ofstream power than are dunes of coarser sand. With coarse sands larger slopes are required to affect the change from transition to plane bed and the result is larger velocities and larger Froude numbers. In flume studies with fine sand, the plane-bed condition commonly exists after the transition and persists over a wide range of Froude numbers ($0.3 \le F_r \le 0.8$). If the sand is coarse and the depth is shallow, however, transition may not terminate until the Froude number is so large that the subsequent bed form may be antidunes rather than plane bed. In natural streams, because of their greater depths, the change from transition to plane bed may occur at a much lower Froude number than in flumes. Manning's n for plane bed sand channels range from 0.010 to 0.013.

3.3.7 Antidunes

Antidunes form as a series or train of inphase (coupled) symmetrical sand and water waves (Figure 3.3.1). The height and length of these waves depend on the scale of the flow system and the characteristics of the fluid and the bed material. In a flume where the flow depth was about 0.5 ft deep, the height of the sand waves ranged from 0.03 ft to 0.5 ft. The height of the water waves was 1.5 to 2 times the height of the sand waves and the length of the waves, from crest to crest, ranged from five to ten feet. In natural streams, such as the Rio Grande or the Colorado River, much larger antidunes form. In these streams, surface waves 2 to 5 ft high and 10 to 40 ft long have been observed.

Antidunes form as trains of waves that gradually build up from a plane bed and a plane water surface. The waves may grow in height until they become unstable and break like the sea surf or they may gradually subside. The former have been called breaking antidunes, or antidunes; and the latter, standing waves. As the antidunes form and increase in height, they may move upstream, downstream, or remain stationary. Their upstream movement led Gilbert (1914) to name them antidunes.

Resistance to flow due to antidunes depends on how often the antidunes form, the area of the stream they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as that for flow over a plane bed. If many antidunes break, resistance to flow is larger because the breaking waves dissipate a considerable amount of energy. With breaking waves, Manning's n may range from 0.012 to 0.020.

3.3.8 Chutes and Pools

At very steep slopes, alluvial-channel flow changes to chutes and pools (Figure 3.3.1). In the 8-foot-wide flume at Colorado State University, this type of flow and bed configuration was studied using fine sands. The flow consisted of a long chute (10 to 30 ft) in which the flow was rapid and accelerating followed by a hydraulic jump and a long pool. The chutes and pools moved upstream at velocities of about one to two feet per minute. The elevation of the sandbed varied within wide limits. Resistance to flow was large with Manning's n of 0.018 to 0.035.

The relation between stream power, velocity and bed configuration is shown in Figure 3.3.3. The relation pertains to one sand and was determined in the 8-ft flume at Colorado State University.

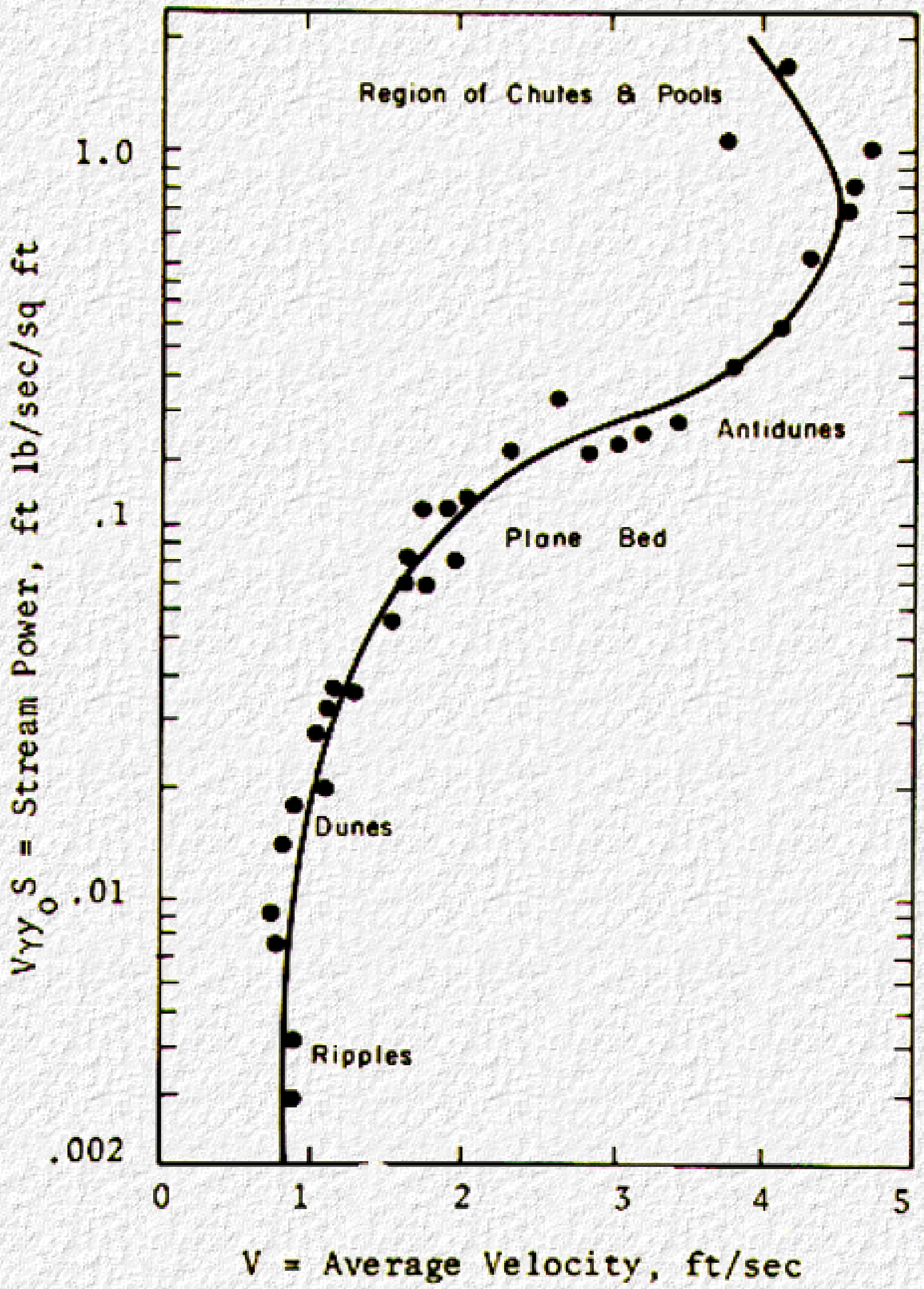


Figure 3.3.3. Change in Velocity with Stream Power for a Sand with D_{50} = 0.19 mm

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3.3.9 Bars

In natural channels, some other bed configurations are also found. These bed configurations are generally called bars and are related to the plan form geometry and the width of the channel.

<u>Bars are bed forms having lengths of the same order as the channel width or greater and heights comparable to the mean depth of the generating flow.</u> Several different types of bars are observed. They are classified as:

1. <u>Point Bars which occur adjacent to the convex banks of channel bends.</u> Their shape may vary

with changing flow conditions and motion of bed particles but they do not move relative to the bends;

2. Alternate Bars which occur in somewhat straighter reaches of channels and tend to be distributed periodically along the reach, with consecutive bars on opposite sides of the channel. Their lateral extent is significantly less than the channel width. Alternate bars move slowly downstream;

3. Transverse Bars which also occur in straight channels. They occupy nearly the full channel width. They occur both as isolated and as periodic forms along a channel, and move slowly downstream; and

4. Tributary Bars which occur immediately downstream from points of lateral inflow into a channel.

In longitudinal section, bars are approximately triangular, with very long gentle upstream slopes and short downstream slopes that are approximately the same as the angle of repose. Bars appear as small barren islands during low flows. Portions of the upstream slopes of bars are often covered with ripples or dunes.

3.4 Resistance to Flow in Alluvial Channels

Resistance to flow in alluvial channels is complicated by the large number of variables and by the interdependency of these variables. It is difficult, especially in field studies, to tell which variables are governing the flow and which variables are the result of this flow.

The slope of the energy grade line for an alluvial stream illustrates the changing role of a variable. If a stream is in equilibrium with its environment, slope is an independent variable. In such a stream, the average slope over a period of years has adjusted so that the flow is capable of transporting only the amount of sediment supplied at the upper end of the stream and by the tributaries. If for some reason a larger or smaller quantity of sediment is supplied to the stream than the stream is capable of transporting, the slope would change and would be dependent on the amount of sediment supplied until new equilibrium is reached.

In the following sections the variables affecting resistance to flow are discussed. The effects produced by different variables change under different conditions. These changing effects are discussed along with approximations to simplify the analysis of alluvial channel flow.

The variables that describe alluvial channel flow are:

V = velocity;

y_o = depth;

S_f = slope of the energy grade line;

ρ = density of water-sediment mixture;

μ = apparent dynamic viscosity of the water-sediment mixture;

g = gravitational acceleration;

D_s = representative fall diameter of the bed material;

G = gradation coefficient of the bed material;

ρ_s = density of sediment;

S_p = shape factor of the particles;

S_R = shape factor of the reach of the stream;

S_c = shape factor of the cross-section of the stream;

f_s = seepage force in the bed of the stream;

C_T = the bed material concentration;

C_f = the fine-material concentration;

ω = the terminal fall velocity of the particles; and

τ_c = critical shear stress.

In general, analysis of river problems is confined to flow of water over beds consisting of quartz particles with constant ρ_s. The gravitational acceleration g is also constant in the present context. The effect of other variables on the flow in alluvial channels is qualitatively discussed in the following sections. Most of this presentation is based on laboratory studies which have been supplemented by data from field experience when available.

3.4.1 Depth

With a constant slope, S_f, and bed material, D_s, an increase in depth, y_o, can change a plane bed (without movement) to ripples, and ripple-bed configuration to dunes, and a dune bed to a plane bed or antidunes. Also, a decrease in depth may cause plane bed or antidunes to change to a dune-bed configuration. A typical break in a depth-discharge relation caused by a change in bed form from dunes to plane bed or from plane bed to dunes is shown in Figure 3.4.1.

Often there is a gradual change in bed form and a gradual reduction in resistance to flow and this type of change prevents the break in the stage-discharge relation. Nevertheless, it is possible to experience a large increase in discharge with little or no change in stage. For this and related reasons the development of dependable stage-discharge relations in alluvial channels is very difficult.

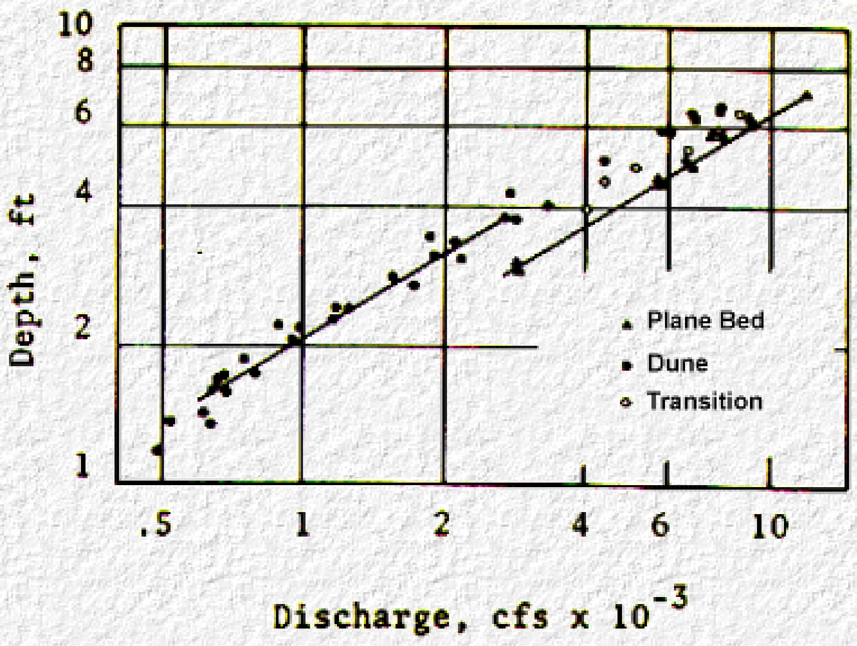

Figure 3.4.1. Relation of Depth to Discharge for Elkhorn River near Waterloo, Nebraska (after Beckman and Furness, 1962)

Resistance to flow varies with depth even when the bed configurations do not change. When the bed configuration is plane bed, either with or without sediment movement or ripples, there is a decrease in resistance to flow with an increase in depth. This is a relative roughness effect.

When the bed configuration is dunes, field and laboratory studies indicate that resistance to flow may increase or decrease with an increase in depth, depending on the size of bed material and magnitude of the depth. Additional studies are needed to define the variation of resistance to flow for flow over dune beds.

When the bed configuration is antidunes, resistance to flow increases with an increase in depth to some maximum value, then decreases as depth is increased further. This increase or decrease in flow resistance is directly related to changes in length, amplitude, and activity of the antidunes as depth is increased.

3.4.2 Slope

The slope, S_f, is an important factor in determining the bed configuration which will exist for a given discharge. The slope provides the downstream component of the fluid weight, which in turn determines the fluid velocity and stream power. The relation between stream power, velocity and bed configuration has been illustrated in Figure 3.3.3.

Even when bed configurations do not change, resistance to flow is affected by a change in slope. For example, with shallow depths and the ripple-bed configuration, resistance to flow increases with an increase in slope. With the dune-bed configuration, an increase in slope increases resistance to flow for bed materials having fall velocities greater than 0.20 ft/s. For those bed materials having fall velocities less than 0.20 ft/s, the effect is uncertain.

3.4.3 Apparent Viscosity and Density

The effect of fine sediment (bentonite) on the apparent kinematic viscosity of the mixture is shown in Figure 3.4.2. The magnitude of the effect of fine sediment on viscosity is large and depends on the chemical make up of the fine sediment.

In addition to increasing the viscosity, fine sediment suspended in water increases the mass density of the mixture (ρ) and, consequently, the specific weight (γ). The specific weight of a sediment-water mixture is computed from the relation,

$$\gamma = \frac{\gamma_w \ \gamma_s}{\gamma_s - C_s(\gamma_s - \gamma_w)} \qquad (3.4.1)$$

where

γ_w = specific weight of the water (about 62.4 lb per cu ft);

γ_s = specific weight of the sediment (about 165.4 lb per cu ft); and

C_s = concentration by weight (in fraction form) of the suspended sediment.

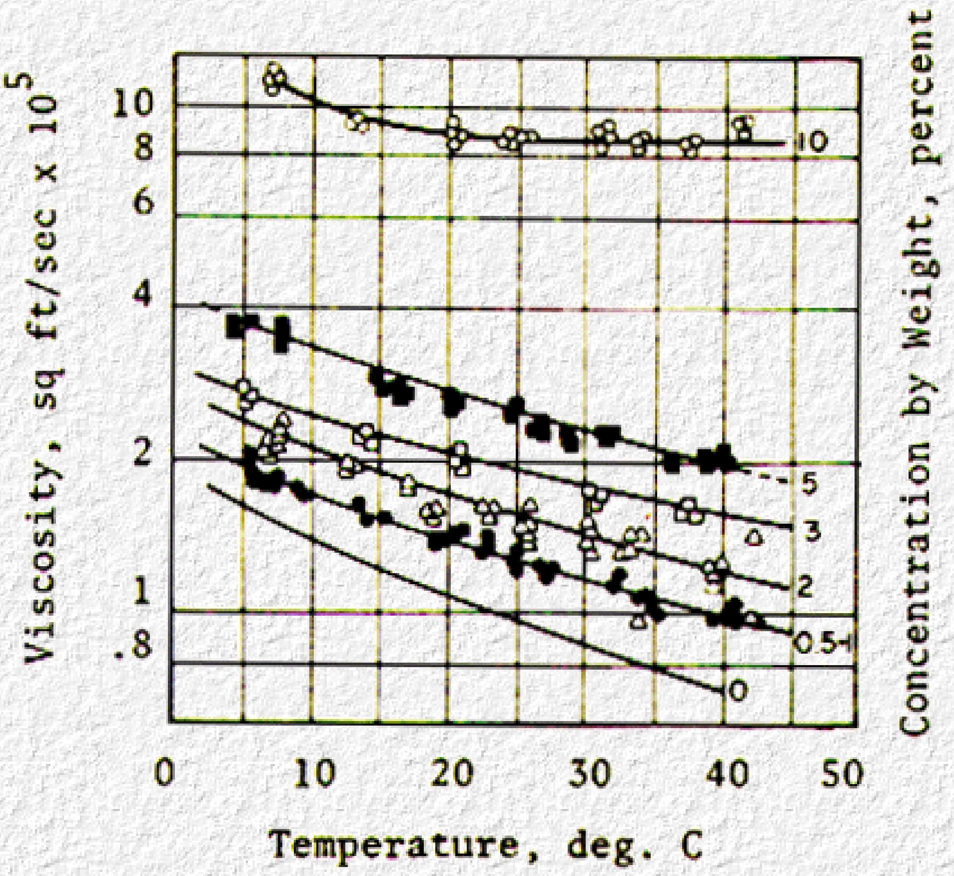

Figure 3.4.2. Apparent Kinematic Viscosity of Water-Bentonite Dispersions

A sediment-water mixture, where C_s = 10 percent, has a specific weight (γ) of about 66.5 lb per cu ft. It is clear any change in γ affects the boundary shear stress and the stream power.

Changes in the fall velocity of a particle caused by changes in the viscosity and the fluid density resulting from the presence of suspended bentonite clay in the water are shown in Figure 3.4.3a. For comparative purposes, the effect of temperature on the fall velocity of two sands in clear water is shown in Figure 3.4.3b.

3.4.4 Size of Bed Material

The effects of the physical size of the bed material, D_s, on resistance to flow are: (1) its influence on the fall velocity ω, which is a measure of the interaction of the fluid and the particle in the formation of the bed configurations; (2) its effect on grain roughness, K_s; and (3) its effect on the turbulent structure and the velocity field of the flow.

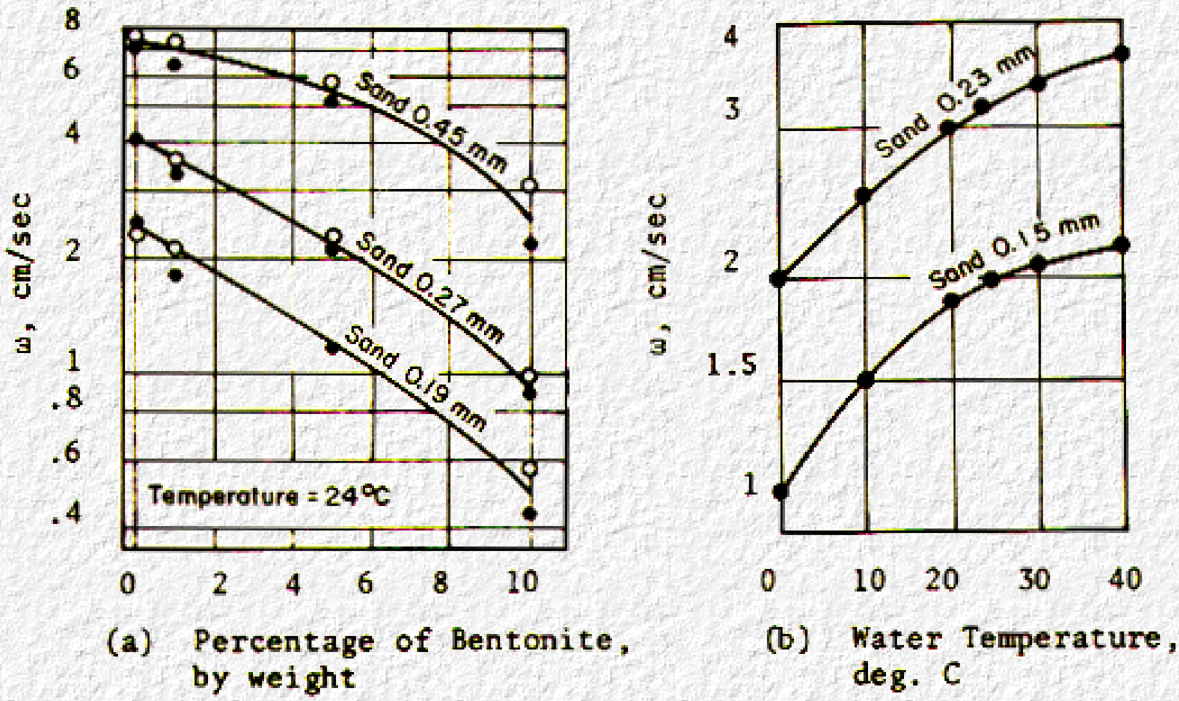

Figure 3.4.3. Variation of Fall Velocity of Several Sand Mixtures with Percent Bentonite and with Temperature

The physical size of the bed material, as measured by the fall diameter or by sieve diameter, is a primary factor in determining fall velocity. Use of the fall diameter instead of the sieve diameter is advantageous because the shape factor and density of the particle can be eliminated as variables. That is, if only the fall diameter is known, the fall velocity of the particle in any fluid at any temperature can be computed; whereas, to do the same computation when the sieve diameter is known, knowledge of the shape factor and density of the particle are also required.

The physical size of the bed material determines the friction factor mainly for the plane-bed condition and for antidunes when they are not actively breaking. The breaking of the waves, which increases with a decrease in the fall velocity of the bed material, causes additional dissipation of energy.

The physical size of the bed material for a dune-bed configuration also has an effect on resistance to flow. The flow of fluid over the back of dunes is affected by grain roughness, although the dissipation of energy by the form roughness is the major factor. The form of the dunes is also related to the fall velocity of the bed material.

3.4.5 Size Gradation

The gradation, G, of sizes of the bed material affects bed form and resistance to flow. Flume experiments indicate that uniform sands (sands of practically the same size) have larger resistance to flow (except plane bed) than graded sands for the various bed forms. Also the transition from upper flow regime to lower flow regime occurs over a narrower range of shear values for the uniform sand. For a plane bed with motion, resistance to flow is about the same for both uniform or graded sand.

3.4.6 Fall Velocity

Fall velocity, ω, is the primary variable that determines the interaction between the bed material and the fluid. For a given depth, and slope, the fall velocity determines the bed form that will occur, the actual dimensions of the bed form and, except for the contribution of the grain roughness, the resistance to flow.

Observations of natural streams have shown that the bed configuration and resistance to flow change with changes in fall velocity when the discharge and bed material are constant. For example, the Loup River near Dunning, Nebraska has bed roughness in the form of dunes in the summer when the water is warm and less viscous but has a nearly plane bed during the cold winter months. Similarly, two sets of data collected by Harms and Fahnestock (1965) on a stable branch of the Rio Grande at similar discharges show that when the water was cold, the bed of the stream was plane, the resistance to flow was small, the depth was relatively shallow, and the velocity was large; but when the water was warm, the bed roughness was dunes, the resistance to flow was large, the depth was large, and the velocity was low.

3.4.7 Shape Factor for the Reach and Cross-Section

The configuration of the reach, S_R, and the shape of the cross-section, S_c, affect the energy losses resulting from the nonuniformity of the flow in a natural stream caused by the bends and the nonuniformity of the banks. Study of these losses in natural channels has long been neglected. Also, flow phenomena, bed configuration, and resistance to flow vary with the width of the stream. In narrow channels dunes and antidunes vary mainly in the downstream direction and resistance to flow is larger than for a wide channel. Also, in wide channels more than one bed form can occur in the cross-section.

3.4.8 Seepage Force

A seepage force, f_s, occurs whenever there is inflow or outflow through the bed and banks of a channel in permeable alluvium. The seepage flow affects the alluvial channel phenomena by altering the velocity field in the vicinity of the bed particles and by changing the effective weight of the bed particles. Seepage may have a significant effect on bed configuration and resistance to flow. If there is inflow, the seepage force acts to reduce the effective weight of the sand and, consequently, the stability of the bed material. If there is outflow, the seepage force acts in the direction of gravity and increases the effective weight of the sand the stability of the bed material. As a direct result of changing the effective weight, the seepage forces can influence the form of bed roughness and the resistance to flow for a given channel flow. For example, under shallow flow a bed material with median diameter of 0.5 mm will be molded into the following forms as shear stress is increased: ripples, dunes, transition, standing sand and water waves, and antidunes. If this same material was subjected to a seepage force that reduced its effective weight to a value consistent with that of medium sand (median diameter, D_s = 0.3 mm), the forms of bed roughness would be ripples, dunes, transition, plane bed, and antidunes for the same range of flow conditions.

A common condition is outflow from the channel during the rising stage; this process increases the stability of the bed and bank material but stores water in the banks. During the falling stage, the situation is reversed; inflow to the channel reduces the effective weight and stability of the bed and bank material and influences the form of bed roughness and the resistance to flow.

3.4.9 Concentration of Bed Material Discharge

The concentration of bed material discharge, C_T, affects the fluid properties by increasing the apparent viscosity and the density of the water-sediment mixture. However, the effect of the sediment on viscosity μ and the density ρ in any resistance to flow relation is accounted for by using their values for the water-sediment mixture instead of their values for pure water. The presence of sediment in the flow causes a small change in the turbulence characteristics, velocity distribution and resistance to flow.

3.4.10 Fine Sediment Concentration

Fine sediment or washload is that part of the total sediment discharge that is not found in appreciable quantities on the bed. If significant amounts of sediment is in suspension, its effect on the viscosity of the water-sediment mixture should be taken into account. The effect of fine sediment on resistance to flow is a result of its effect on the apparent viscosity and the density of the water-sediment mixture. Generally the fine sediment is uniformly distributed in the stream cross-section. The method of defining and treating the fine-material load computations is subsequently discussed in this chapter.

3.4.11 Bedform Predictor and Manning's n Values for Sand-Bed Streams

In Figure 3.4.4, the relation between stream power, median fall diameter of bed material, and form roughness is shown. This relation gives an indication of the form of bed roughness one can anticipate if the stream power and fall diameter of bed material are known. Flume data were utilized to establish the boundaries separating plane bed and ripples, ripples and dunes for all sizes of bed material, and dunes and transition for the 0.93 mm bed material. The lines dividing dunes and transition and dividing transition and upper regime are based on flume data and the following field data: (1) Elkhorn River, near Waterloo, Nebraska (Beckman and Furness, 1962); (2) Rio Grande, 20 miles above El Paso, Texas; (3) Middle Loup River at Dunning, Nebraska (Hubbell and Matejka, 1959); (4) Rio Grande at Cochiti, near Bernalillo and at Angostura heading, N. Mexico (Culbertson and Dawdy, 1964); and (5) Punjab canal data upper regime flows that have been observed in large irrigation canals that have fine sandbeds.

Observations by the authors on natural sandbed streams with bed material having a median diameter ranging from 0.1 mm to 0.4 mm indicate that the bed planes out and resistance to flow decreases whenever high flow occurs. Manning's n changes from values as large as 0.040 at low flow to as small as 0.012 at high flow. An example is given in Figure 3.4.5. These observations are substantiated by Dawdy (1961), Colby (1960), Corps of Engineers (1968) and Beckman and Furness (1962). An example of the use of this information is given in Appendix 3, Problem 3.4.

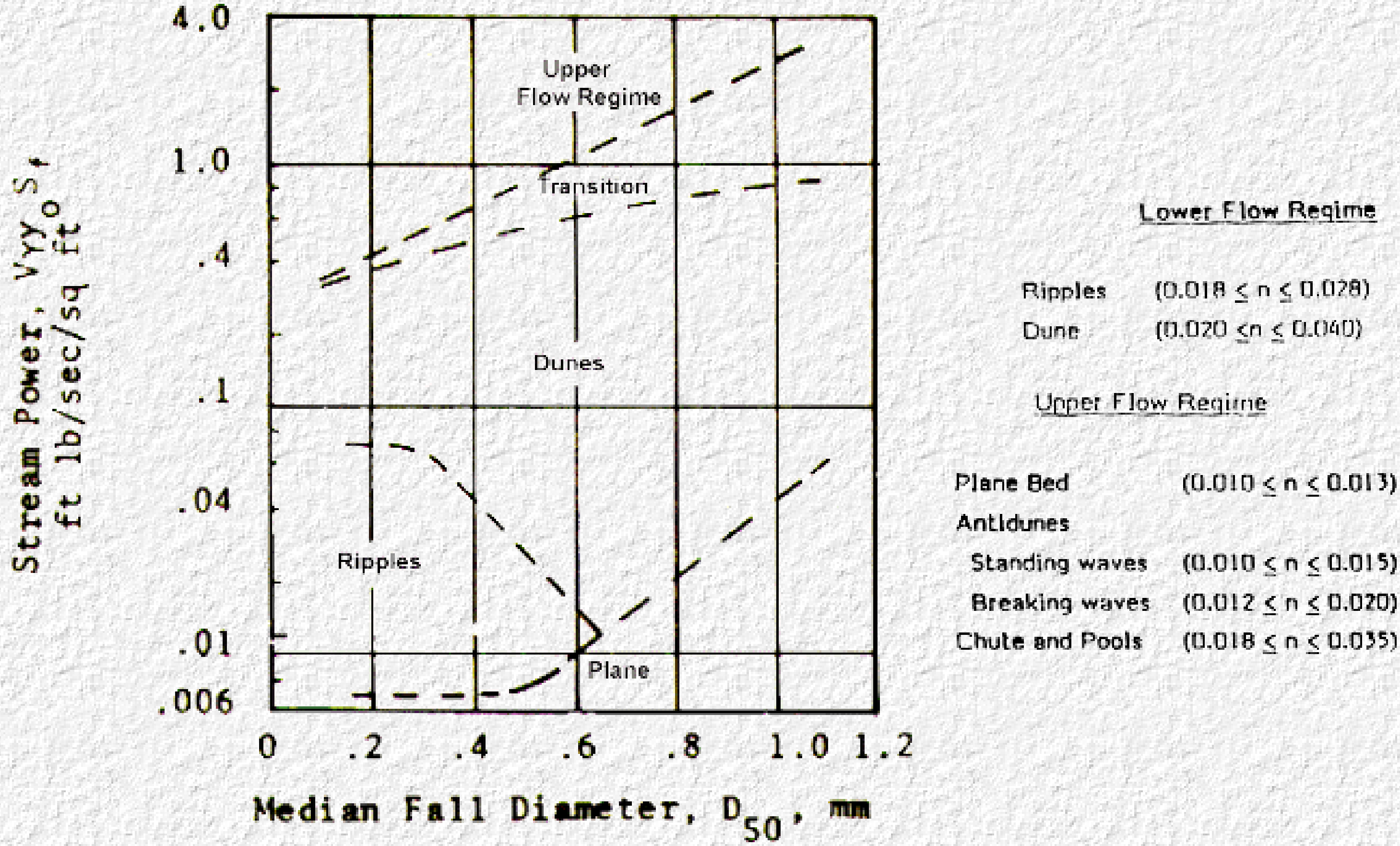

Figure 3.4.4. Relation between Stream Power, Median Fall Diameter, and Bed Configuration and Manning's n Values

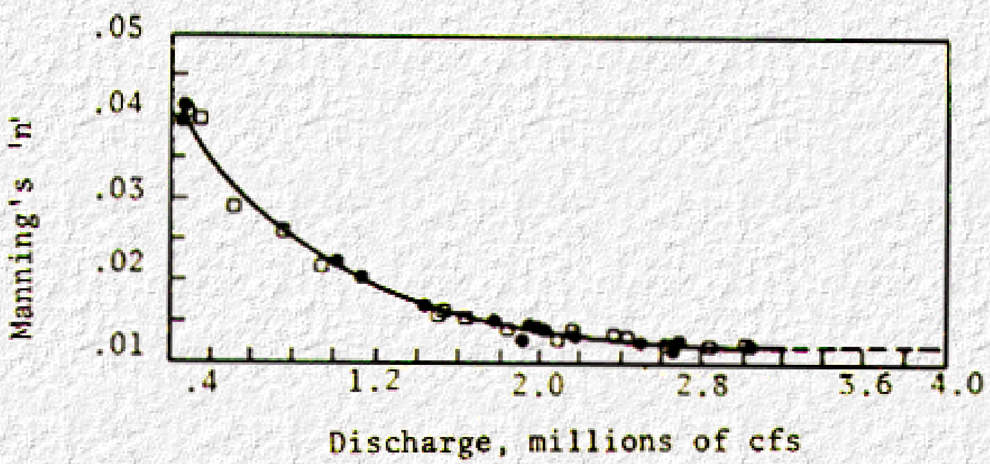

Figure 3.4.5. Change in Manning's n with Discharge for Padma River in Bangladesh

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3.4.12 How Bedform Changes Affect Highways in the River Environment

At high flows, most sandbed channel streams shift from a dune bed to a transition or a plane bed configuration. The resistance to flow is then decreased two to threefold. The corresponding increase in velocity can increase scour around bridge piers, abutments, spur dikes or banks and also increases the required size of riprap. On the other hand, the decrease in stage resulting from the planing out of the bed will decrease the required elevation of the bridge crossing, the height of embankments across the floodplain, the height of any dikes, and the height of any channel control works that may be needed; and the converse is also true.

Another effect of bed form on highway crossings is that with dunes on the bed there is a fluctuating pattern of scour on the bed and around the piers, abutments' guide banks and spur dikes. The average height of dunes is approximately 1/2 to 1/3 the average depth of flow and the maximum height of a dune may approach the average depth of flow. If the depth of flow is 10 feet, the maximum dune height may be of the order of 10 feet, and half of this would be below the mean elevation of the bed. With the passage of this dune through a bridge section, an increase of 5 feet in the local scour would be anticipated when the trough of the dune arrives at the bridge.

A very important effect of bed forms and bars is the change of flow direction in channels. At low flow the bars can be residual and cause high velocity flow along or at a pier or abutment or any of the other structures in the stream bed, causing deeper than anticipated scour. As stated previously large discharges normally experience smaller resistance to flow in a sandbed stream due to the change in bed form. However, if the bridge crossing or encroachment causes appreciable backwater, the dune bed may not plane out at large discharges and a higher resistance to flow results. This increase in resistance to flow can decrease the velocity of flow and also decrease the transport capacity of the channel so that aggradation occurs upstream of the crossing. The aggradation and the roughness increases the river stage and thus the height of any control structure or the levees. Thus, the bridge crossing can adversely affect the floodplain, due to the change in bed form that would occur.

With highways in the sandbed river environment, care must be taken in analyzing the crossing in order to foresee possible changes that may occur in the bed form and what these changes may do to the resistance coefficient, to sediment transport, and to the stability of the reach and its structures.

3.4.13 Alluvial Processes and Resistance to Flow in Coarse Material Streams

The preceding discussion of alluvial channel flow is mainly related to sandbed channels; that is, channels with noncohesive bed materials of size less than 2 mm. The analysis of coarse-material channels is also pertinent to highway engineering. This classification includes all channels with noncohesive bed materials coarser than 2 mm size.

The behavior of coarse material channels is somewhat different from sandbed channels. The main distinction between the two channels lies in the spread of their bed material size distribution. In sandbed channels, for example, the bed may consist of particles from 0.02 to 2 mm; i.e. a 100-fold size range. In coarse-material channels, even if the maximum size is limited to cobbles (250 mm), the size range of particles may be 0.10 to 250 mm, which is a 2,500-fold size range. The armoring of a channel is, therefore, more pronounced in coarse material channels. In general, the coarse-material channels are less active and slower in bank shifting than sandbed channels.

- Armoring - The phenomenon of armoring in mobile bed channels occurs by the rearrangement of bed material during movement. The bed is covered by a one particle thick layer of the coarser material underlain by the finer sizes. An example of calculations to determine armour layer formation is given in Appendix 3, Problem 3.11. The absence of finer sizes from the surface layer is caused by the winnowing away of these sizes by the flow. As the spread of particle sizes available in the bed of coarse-material channels is large, these channels can armor their beds and behave as rigid boundary channels for all except the highest flows. The bed and bank forming activity in these channels is therefore limited to much smaller intervals of the annual hydrographs than the sandbed channels.

  The general lack of mobility in coarse material channels also means the bed forms do not change as much or as rapidly as in sandbed channels. The roughness coefficients in coarse material channels are therefore more consistent during the annual hydrographs than in sandbed channels. Most of the resistance to flow in coarse material channels comes from the grain roughness and from bars. The river bed forms (dunes) are less important in the hydraulic behavior of coarse bed channels.

- Sampling - The purpose of bed material sampling in coarse-material channels is: (1) to determine the conditions of incipient movement; (2) to assess the bed roughness related to the resistance to flow; (3) to determine the bed material load for a given flow; and (4) to determine the long and short time response of the channel to specific activities. For objectives (1) to (3), the properties of the surface layer are needed. If it is anticipated that the bed layer will be disrupted at any given stage, it is necessary to take an adequate sample of both the surface and subsurface material.

  The surface sampling can be easily done on the channel bed by counting particles on a grid, as already explained in this chapter. However, special effort should be made to obtain an objective sample. There is a tendency to select too many large particles. The scoop sample with bed material sizes larger than an inch or so is difficult to obtain and such samples may have to be collected from bars and other exposed areas on channel perimeter.

  In the size distribution analysis of coarse-bed materials, it is sometimes necessary to obtain particle counts by number, rather than by sieving or visual accumulation tube analysis for a part of the sample. Care must be taken in the interpretation of frequency distribution of part of a sample obtained by sieving. Only if the size distribution in a sample follows log-normal probability distribution can we transfer number counts to distributions by size, volume, weight or surface areas directly. For other distributions, special numerical techniques have to be used to transform the number distributions to weight or size distributions.

  If the objectives of bed material sampling include bed roughness and channel response, then the particles coarser than D_{84} or D_{90} need to be analyzed with more care. These sizes also require large samples for their determination.

- Resistance to Flow - In sandbed channels, the form roughness is the primary component of channel roughness. Form roughness can be much greater than the grain roughness when the bed forms are ripples and dunes. In coarse-material channels, the ripples never form and dunes are rare. The main type of bed form roughness in such channels is the pool and riffle configuration. With coarse material channels the grain roughness is the main component of the channel roughness.

A coarse-material channel may have bed material that is only partly submerged during most of the flows. It is difficult to determine the channel roughness for such beds. For other cases, analysis of data from many rivers, canals and flumes (Anderson, et al., 1968) shows that the channel roughness can be predicted by the equations:

V. T. Chow (1959)

$$n = 0.04\, D_{50}^{1/6} \quad (D_{50} \text{ in meters}) \qquad (3.4.2a)$$

Anderson, et. al. (1968)

$$n = \frac{D_{90}^{\;1/6}}{44.4} \quad (D_{90} \text{ in inches}) \qquad (3.4.2b)$$

Lane and Carlson (Simons, Li and Assoc., 1982)

$$n = \frac{D_{75}^{1/6}}{39} \quad (D_{75} \text{ in inches}) \qquad (3.4.2c)$$

In meters, the following equations are also applicable.

$$n = 0.038 D_{90}^{1/6} \quad (D_{90} \text{ in meters}) \qquad (3.4.3)$$

$$n = \frac{0.113 y_o^{1/6}}{1.16 + 2.0 \log\left(\frac{y_o}{D_{84}}\right)} \quad (y_o \text{ and } D_{84} \text{ in meters}) \qquad (3.4.4)$$

Equation 3.4.4 has been proposed by Limerinos (1970) and involves flow depth y_o as a parameter. Recent comparison of several equations has been reported by Bray (1982) and additional information is presented in his report.

An alternative approach, valid when $R/D_{50} > 10$, is to evaluate the Darcy-Weisbach friction factor, f, as a function of the hydraulic radius R in meters with

$$\frac{1}{\sqrt{f}} = 1.9\left(\frac{R}{D_{84}}\right)^{1/4} \qquad (3.4.5)$$

$$\frac{1}{\sqrt{f}} = 2.0 \log\left(\frac{R}{D_{84}}\right) + 1.1 \qquad (3.4.6)$$

Note: R and D_{84} in same units

Manning's n can then be calculated from the relationship between n and f with R in meters.

$$n = 0.113\, R^{1/6} \sqrt{f} \qquad (3.4.7)$$

This approach should not be applied to river reaches with active beds or significant sediment transport.

Charlton et al. (1978) and Bray (1979) have proposed the guidelines in Table 3.4.1 to estimate D_{90}, D_{84}, and D_{65}, for gravel-bed material when size distribution curves cannot be obtained from field data.

Table 3.4.1 Typical Sediment Size Distribution for Gravel-Bed Stream

Ratio	Mean Value	Standard Deviation
D_{90}/D_{50}	2.1	0.46
D_{84}/D_{50}	1.9	0.36
D_{65}/D_{50}	1.3	0.08

An example of the use of these relations is given in Appendix 3, Problem 3.5.

3.5 Beginning of Motion

Beginning and ceasing of sediment motion is of great importance in three areas of application: (1) channel stability and scour; (2) bed load transport equations; and (3) design of riprap.

Beginning of motion can be related to either the shear stress on the grains or the fluid velocity in the vicinity of the grains. When the grains are at incipient motion, these values are called the critical stress and critical velocity. The choice of shear stress or velocity depends on: (1) which is easier to determine in the field; (2) the precision with which the critical value is known for the particle size; and (3) the type of problem. In sediment transport analysis most equations use critical shear. In stable channel design either critical shear or critical velocity is used; whereas, in the design of riprap critical velocity is commonly used.

It is not sufficient to determine the average value of the critical shear or critical velocity because both quantities are fluctuating. For the same mean values they may have larger values that act for a sufficiently long enough time to cause a particle to move. In addition to the forces on the particle resulting from the flowing water, waves and seepage into or out of the bed or banks affect the beginning of motion conditions.

3.5.1 Theory of Beginning of Motion

The forces acting on an individual particle on the bed of an alluvial channel are:

1. the body force F_g due to the gravitational field;
2. the external forces F_n acting at the points of contact between the grain and its neighboring grains; and
3. the fluid force F_f acting on the surface of the grain. The fluid force varies with the velocity field and with the properties of the fluid.

The relative magnitude of these forces determines whether the grain moves or not.

For the individual grain, the body force is

$$F_g = g\rho_s K_2 D^3{}_s \qquad (3.5.1)$$

where ρ_s is the density of the grain, K_2 is a volumetric coefficient and D_s is the grain diameter. The term $K_2D^3{}_s$ is the volume of the grain.

For convenience, the fluid forces acting on the grain are divided into three components:

1. The form drag component F_D

$$F_D = C_D K_1 D_s{}^2 \rho\, (v^2/2); \qquad (3.5.2)$$

2. The viscous drag component F_v

$$F_v = C_s K_1 D_s{}^2 \tau \qquad (3.5.3)$$

3. The buoyant force component F_B

$$F_B = g\rho K_2 D_s{}^3 \qquad (3.5.4)$$

where:

C_D = drag coefficient;

K_1 = a coefficient associated with the area of the grain subjected to drag and shear (the term $K_1D_s^2$ represents the cross-sectional area of the grain);

K_2 = the volumetric coefficient of the grain;

D_s = the diameter of the grain;

v = the velocity in the vicinity of the grain;

C_s = coefficient of shear; and

τ = the average viscous shear stress.

The external forces F_n depend on the values of the fluid and body forces. Under conditions of no flow, the fluid force is $F_f = F_B$. There is no form or viscous drag. Then the external force F_n is $F_n = F_g - F_B$ or:

$$F_n = (\rho_s - \rho)\, gK_2D_s^3 \qquad (3.5.5)$$

That is, the external force is equal to the submerged weight of the grain.

The form drag can be written in terms of the shear velocity. For turbulent flow, the local velocity, v is directly proportional to the shear velocity V_*. Then, Equation 3.5.2 reduces to

$$F_D \sim \rho D_s^2 V_*^2 \qquad (3.5.6)$$

The viscous drag is also related to the shear velocity but it is the shear velocity for laminar flow. For laminar flow

$$\tau = \mu \frac{dv}{dy} \qquad (3.5.7)$$

Again, by replacing v with V_* and y with D_s we can write

$$\tau \sim \mu V_* / D_s \qquad (3.5.8)$$

With this expression for viscous shear, the shear force F_v becomes

$$F_v \sim \mu D_s V_* \qquad (3.5.9)$$

Now, consider the ratio of the form drag force F_D to the viscous shear force F_v. According to Equation 3.5.6 and Equation 3.5.9

$$\frac{F_D}{F_v} \sim \frac{\rho D_s^2 V_*^2}{\mu D_s V_*}$$

or

$$\frac{F_D}{F_v} \sim \frac{D_s V_*}{\upsilon} \qquad (3.5.10)$$

When the flow over the grain is turbulent, the form drag is predominant and the term $D_s V_*/\upsilon$ is large. When the flow over the grain is laminar the viscous shear force is predominant and the term $D_s V_*/\upsilon$ is small. Thus, the Reynolds' number for particle $D_s V_*/\upsilon$ is an indicator of the characteristic of the flow in the vicinity of the grain.

As both the form drag and viscous shear are proportional to the shear velocity, the ratio of the forces tending to move the grain to the forces resisting movement is

$$\frac{F_D}{F_n} \sim \frac{\rho D_s^2 V_*^2}{(\rho_s - \rho) g D_s^3} = \frac{\tau_o}{(\gamma_s - \gamma) D_s} \qquad (3.5.11)$$

Recall that $V_*^2 = \tau_o/p$. The relation between $\tau_o/(\gamma_s - \gamma)D_s$ and $D_s V_*/\upsilon$ for the condition of incipient motion has been determined experimentally by Shields and others. The relation is given in Figure 3.5.1. At conditions of incipient motion, the shear stress τ_o is designated the critical shear stress τ_c. An example of the use of Shields' diagram is given in Appendix 3, Problem 3.3.

Figure 3.5.2 shows relationships between critical tractive force (critical shear stress) and mean diameter as determined and/or recommended by different investigators for different soil types. The difference between investigators could possibly be due to the effects of cohesion, when present, causing the particles to aggregate and therefore not act necessarily as individual particles.

Figure 3.5.3 shows relationships between maximum allowable velocity (velocity against stone) and minimum stone size that can sustain the hydraulic forces without motion.

Criteria based on velocity rather than shear stress have also been proposed. The values of maximum permissible velocity recommended by Fortier and Scobey are given in Table 3.5.1 for clear flows in channels and water transporting colloidal silts.

Non-scour velocities for noncohesive soils and compact cohesive soils suggested by Keown et al. (1977) are shown in Table 3.5.2.

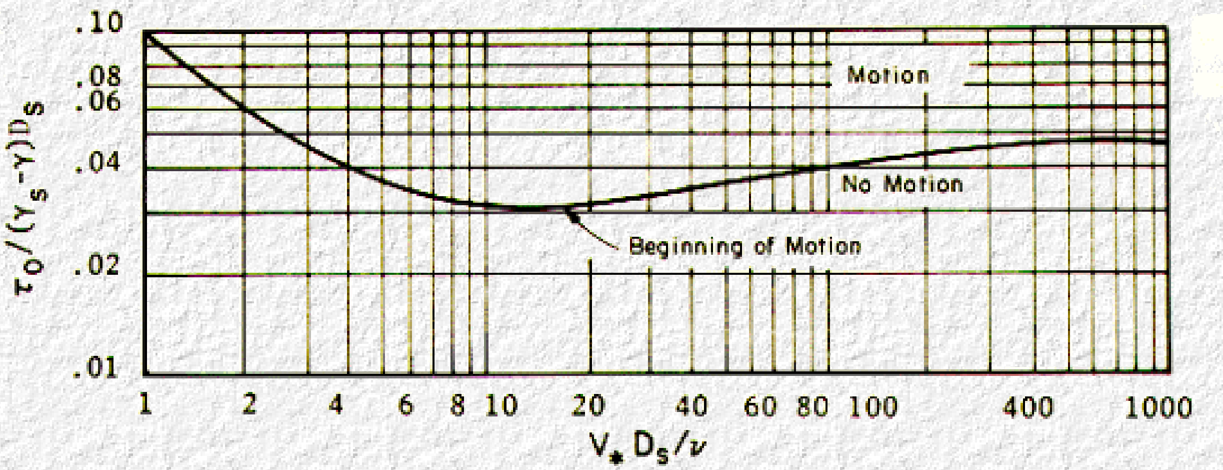

Figure 3.5.1. Shields' Relation for Beginning of Motion (Adapted from Gessler, 1971)

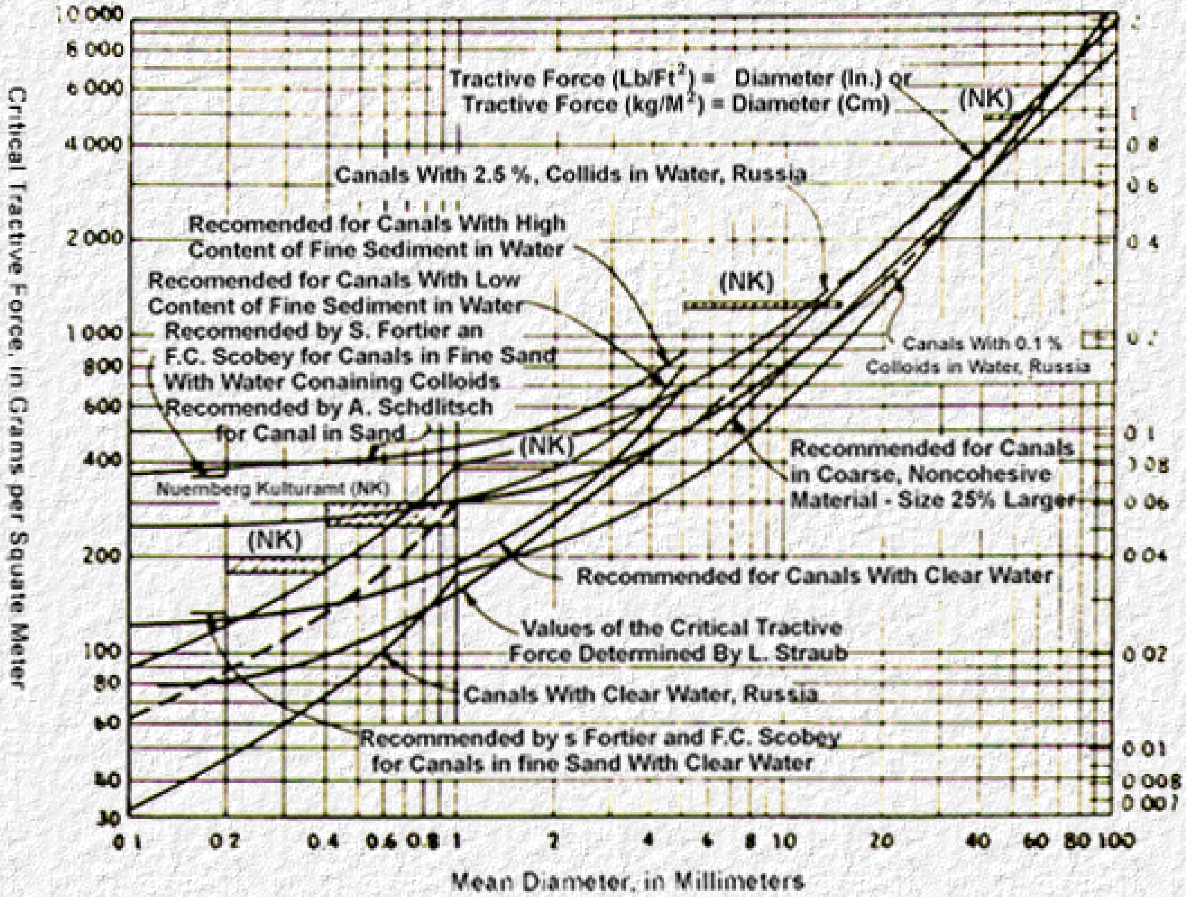

Figure 3.5.2. Recommended Limiting Shear Stress for Canals

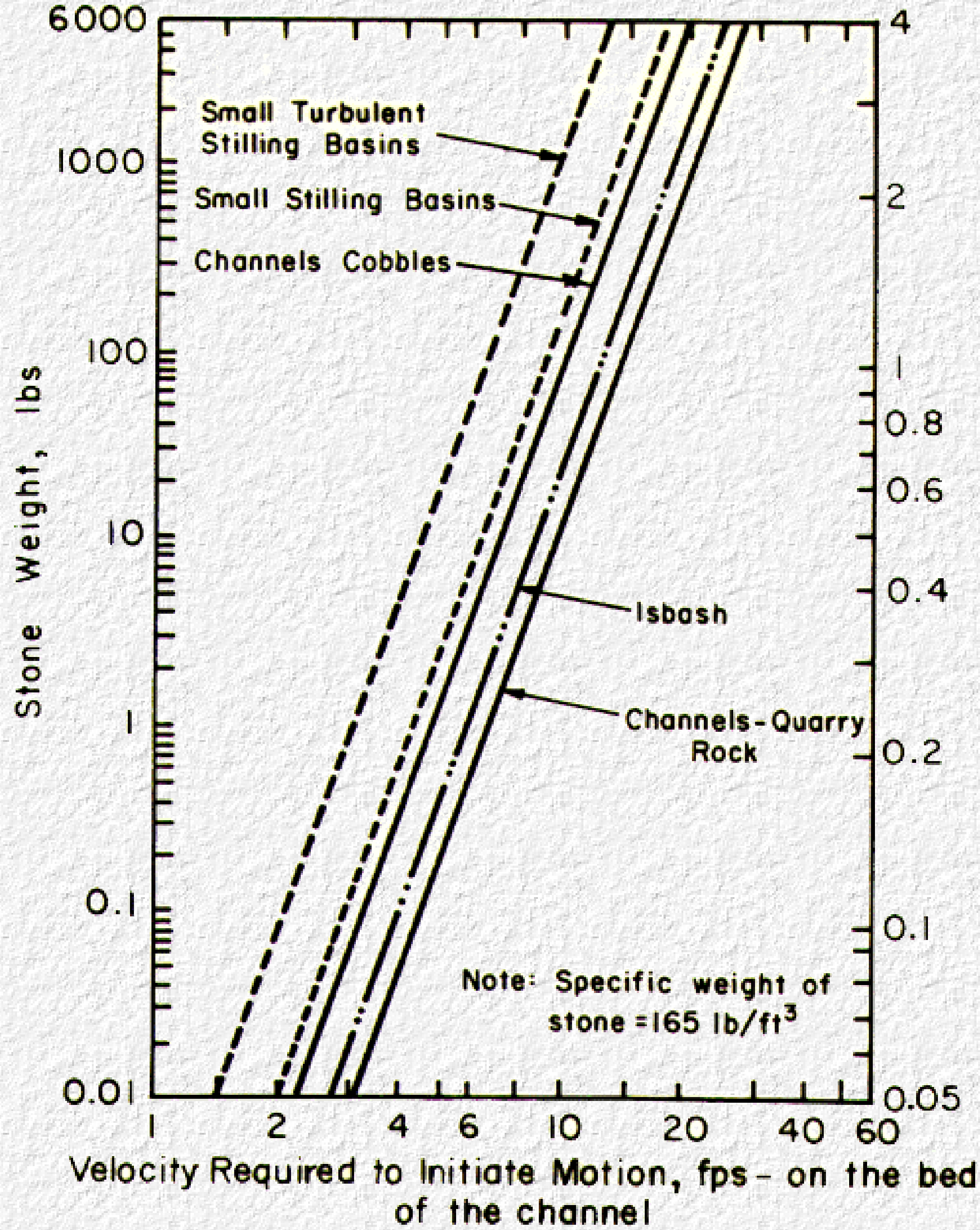

Figure 3.5.3. Critical Velocity as a Function of Stone Size

Table 3.5.1 Maximum Permissible Velocities Recommended by Fortier and Scobey and the Corresponding Unit-Tractive-Force Values Converted by the U.S. Bureau of Reclamation (Adapted from Chow, 1959)

(For straight channels of small slope, after aging)

Material	n	Clear Water		Water Transporting Colloidal Silts	
		V_1 fps	τ_o lb/ft^2	V_1 fps	τ_o lb/ft^2

Fine sand, colloidal	0.020	1.50	0.027	2.50	0.075
Sandy loam, non-colloidal	0.020	1.75	0.037	2.50	0.075
Silt loam, non-colloidal	0.020	2.00	0.048	3.00	0.11
Alluvial silts, non-colloidal	0.020	2.00	0.048	3.50	0.15
Ordinary firm loam	0.020	2.50	0.075	3.50	0.15
Volcanic ash	0.020	2.50	0.075	3.50	0.15
Stiff clay, very colloidal	0.025	3.75	0.26	5.00	0.46
Alluvial silts, colloidal	0.025	3.75	0.26	5.00	0.46
Shales and hardpans	0.025	6.00	0.67	6.00	0.67
Fine gravel	0.020	2.50	0.075	5.00	0.32
Graded loam to cobbles when non-colloidal	0.030	3.75	0.38	5.00	0.66
Graded silts to cobbles when colloidal	0.030	4.00	0.43	5.50	0.80
Coarse gravel, non-colloidal	0.025	4.00	0.30	6.00	0.67
Cobbles and shingles	0.035	5.00	0.91	5.50	1.10

**The Fortier and Scobey values were recommended for use in 1926 by the Special Committee on Irrigation Research of the American Society of Civil Engineers.*

Table 3.5.2 Non-Scour Velocities for Soils
(Modified from a report by Keown and others, 1977)

Kind of Soil	Grain Dimensions		Approximate Non-Scour Velocities (feet per second) Mean Depth			
	mm	feet	1.3 ft	3.3 ft	6.6 ft	9.8 ft
			For Noncohesive Soils			
Boulders	> 256	> 0.840	15.1	16.7	19.0	20.3
Large cobbles	256 -128	0.840 - 0.420	11.8	13.4	15.4	16.4
Small cobbles	128 - 64	0.420 - 0.210	7.5	8.9	10.2	11.2
Very coarse gravel	64 - 32	0.210 - 0.105	5.2	6.2	7.2	8.2
Coarse gravel	32 - 16	0.105 - 0.0525	4.1	4.7	5.4	6.1
Medium gravel	16 - 8.0	0.0525 - 0.0262	3.3	3.7	4.1	4.6
Fine gravel	8.0 - 4.0	0.0262 - 0.0131	2.6	3.0	3.3	3.8
Very fine gravel	4.0 - 2.0	0.0131 - 0.00656	2.2	2.5	2.8	3.1
Very coarse sand	2.0 - 1.0	0.00656 - 0.00328	1.8	2.1	2.4	2.7
Coarse sand	1.0 - 0.50	0.00328 - 0.00164	1.5	1.8	2.1	2.3
Medium sand	0.50 - 0.25	0.00164 - 0.000820	1.2	1.5	1.8	2.0
Fine sand	0.25 - 0.125	0.000820 - 0.000410	.98	1.3	1.6	1.8
			For Compact Cohesive Soils			
Sandy loam (heavy)			3.3	3.9	4.6	4.9
Sandy loam (light)			3.1	3.9	4.6	4.9
Loess soils in the conditions of finished settlement			2.6	3.3	3.9	4.3

3.5.2 Relation between Shear Stress and Velocity

Measuring the average bottom shear stress directly in the field is tenuous. However, the average bottom shear stress can be computed from the expression

$$\tau_o = \gamma R S_f \qquad (3.5.12)$$

For steady uniform flow, the average shear stress on the bed can be estimated by employing the velocity profile equations in Chapter 2. If the local velocity v_1 at depth y_1 is known then, from Equation 2.3.15

$$\tau_o = \frac{\rho v_1^2}{\left[5.75 \log\left(30.2 \frac{y_1}{k_s}\right)\right]^2} \qquad (3.5.13)$$

This equation and the ones given below are valid for fully turbulent uniform flow over a rough boundary in wide channels with a plane bed. Alternatively, if two point velocities in a vertical profile are known (preferably in the lower 15% of the depth)

$$\tau_o = \frac{\rho (v_1 - v_2)^2}{\left[5.75 \log\left(\frac{y_1}{y_2}\right)\right]^2} \qquad (3.5.14)$$

If the depth of flow y_o, the grain size k_s and the average velocity in the vertical V are known, then according to Equation 2.3.16.

$$\tau_o = \frac{\rho v^2}{\left[5.75 \log\left(12.27 \frac{y_o}{k_s}\right)\right]^2} \qquad (3.5.15)$$

The preceding equations deal with average values of the shear stress or velocity.

The instantaneous value of the shear stress or local velocity may be as much as two or three times greater than the average value. The fact that the instantaneous shear stress at the bed would be varying greatly is accounted for in Shields' diagram (Figure 3.5.1) if the channel is prismatic and all the turbulence is generated at the channel boundary. (Shields' curve was determined from experimental tests.) However, if the turbulence is being generated in some other manner (by a hydraulic jump for example), then the best estimate of the average boundary shear stress is (from Equation 2.3.8)

$$\tau_o = \overline{\rho v'_x v'_y} \qquad (3.5.16)$$

As the Reynolds stress $\rho v'_x v'_y$ is extremely difficult to measure in streams, there are few good estimates of the shear stress for flow conditions such as in hydraulic jumps or in regions of flow separation. In special cases such as flow around prismatic bends, the boundary shear stress has been obtained from model tests in laboratories.

In addition to the velocity or shear stress forces, the wave forces and seepage forces must be considered in determining a critical shear stress, critical velocity or size of stone to resist motion.

In view of the bed consisting of particles of various sizes, each one having different shear stresses needed to dislodge, it is customary to consider the critical shear stress corresponding to the D_{50} size of the bed material as that required for beginning of motion. An example of velocity shear stress relations is given in Appendix 3, Problem 3.2.

3.6 Sediment Transport

In this section the basic terms and methods of computing sediment load in alluvial channels are described.

3.6.1 Terminology

Bed layer: The flow layer, several grain diameters thick (usually taken as two grain diameters thick), immediately above the bed.

Bed load: Sediment that moves by rolling or sliding along the bed and is essentially in contact with the stream bed in the bed layer.

Bed material: The sediment mixture of which the stream bed is composed.

Bed sediment discharge: That part of the total sediment discharge which is composed of grain sizes found in the bed. The bed sediment discharge is assumed equal to the transport capacity of the flow.

Contact load: Sediment particles that roll or slide along in almost continuous contact with the stream bed.

Density of water-sediment mixture: The mass per unit volume including both water and sediment.

Discharge-weighed concentration: The dry weight of sediment in a unit volume of stream discharge, or the ratio of the discharge of dry weight of sediment to the discharge by weight of water sediment mixture normally reported in parts per million (ppm) or parts per liter (ppl).

Load (or sediment load): The sediment that is being moved by a stream.

Sediment (or fluvial sediment): Fragmentary material that originates from weathering of rocks and is transported by, suspended in, or deposited from water.

Sediment concentration (by weight or by volume): The quantity of sediment relative to the quantity of transporting fluid, or fluid-sediment mixture. The concentration may be by weight or by volume. When expressed in ppm, the concentration is always in ratio by weight.

Sediment discharge: The quantity of sediment that is carried past any cross-section of a stream in a unit of time.

Sediment yield: The dry weight of sediment per unit volume of water-sediment mixture in place, or the ratio of the dry weight of sediment to the total weight of water-sediment mixture in a sample or a unit volume of the mixture.

Suspended load (or suspended sediment): Sediment that is supported by the upward components of turbulence in a stream and that stays in suspension for an appreciable length of time.

Suspended-sediment discharge: The quantity of suspended sediment passing through a stream cross-section outside the bed layer in a unit of time.

Total sediment discharge: The total sediment discharge of a stream. It is the sum of the suspended-sediment discharge and the contact sediment discharge, or the sum of the bed sediment discharge and the time rate of washload, or the sum of the measured sediment discharge and the unmeasured sediment discharge.

Unmeasured sediment discharge: Sediment discharge close to the bed that is not sampled by a suspended load sampler.

Washload: That part of the total sediment load which is composed of particle sizes finer than those found in appreciable quantities in the bed and is determined by available bank and upslope supply rate.

3.6.2 General Considerations

The amount of material transported or deposited in the stream under a given set of conditions is the result of the interaction of two groups of variables. In the first group are those variables which influence the quantity and quality of the sediment brought down to that section of the stream. In the second group are variables which influence the capacity of the stream to transport that sediment. A list of these variables is given as follows.

Group 1 - Sediment brought down to the stream depends on the geology and topography of watershed; magnitude, intensity, duration, distribution, and season of rainfall; soil condition; vegetal cover; cultivation and grazing; surface erosion and bank cutting.

Group 2 - Capacity of stream to transport sediment depends on hydraulic properties of the stream channel. These are fluid properties, slope, roughness, hydraulic radius, discharge, velocity, velocity distribution, turbulence, tractive force, viscosity and density of the fluid sediment mixture, and size and gradation of the sediment.

These variables are not all independent and, in some cases, their effect is not definitely known. The variables which control the amount of sediment brought down to the stream are subject to so much variation, not only between streams but at a given point of a single stream, that the analysis of any particular case in a quantitative way is extremely difficult. It is practicable, however, to measure the sediment discharge over a long period of time and record the results, and from these records to determine a soil loss from the area.

The variables which deal with the capacity of the stream to transport solids are subject to mathematical analysis. These variables are closely related to the hydraulic variables controlling the capacity of the streams to carry water.

3.6.3 Source of Sediment Transport

Einstein (1964) stated that:

> *"Every sediment particle which passes a particular cross-section of the stream must satisfy the following two conditions: (1) it must have been eroded somewhere in the watershed above the cross-section; (2) it must be transported by the flow from the place of erosion to the cross-section.*
>
> *Each of these two conditions may limit the sediment rate at the cross-section, depending on the relative magnitude of two controls: the availability of the material in the watershed and the transporting ability of the stream. In most streams the finer part of the load, i.e., the part which the flow can easily carry in large quantities, is limited by its availability in the watershed. This part of the load is designated as* ***washload****. The coarser part of the load,*

i.e., the part which is more difficult to move by flowing water, is limited in its rate by the transporting ability of the flow between the source and the section. This part of the load is designated as bed sediment load."

Thus, for engineering purposes there are two sources of sediment transported by a stream: (1) the bed material that makes up the stream bed; and (2) the fine material that comes from the banks and the watershed (washload). Geologically both materials come from the watershed. But for the engineer, the distinction is important because the bed material is transported at the capacity of the stream and is functionally related to measurable hydraulic variables. The washload is not transported at the capacity of the stream. Instead the washload depends on the availability and is not functionally related to measurable hydraulic variables.

There is no sharp demarcation between washload discharge and bed sediment discharge. As a rule of thumb, many engineers assume that the bed sediment load is composed of sizes equal to or greater than 0.062 mm which is also the division point between sand and silt. The sediment discharge consisting of grain sizes smaller than 0.062 mm is considered as washload. A more reasonable criterion is to choose a sediment size finer than the smallest 10 percent of the bed material as the dividing size between washload and bed sediment load. It is important to note that in a fast flowing mountain stream with a bed of cobbles the washload may consist of coarse sand sizes. This invalidates the criterion based on $D_s < 0.062$ mm for coarse bed sediment streams, however, the criterion based on D_{10} might still be applicable.

3.6.4 Mode of Sediment Transport

Sediment particles are transported by rolling or sliding on the bed (bed load or contact load) or by suspension by the turbulence of the stream. Even as there is no sharp demarcation between bed sediment discharge and washload there is no sharp line between contact load and suspended sediment load. A particle may move part of the time in contact with the bed and at other times be suspended by the flow. The distinction is important because the two modes of transport follow different laws. The equations for estimating the total bed material discharge of a stream are based on these laws.

3.6.5 Total Sediment Discharge

The total sediment discharge of a stream is the sum of the bed sediment discharge and the fine sediment (washload) discharge, or the sum of the contact sediment discharge and suspended sediment discharge. In the former sum the total sediment discharge is based on source of the sediments and the latter sum is based on the mode of sediment transport. Whereas suspended sediment load consists of both bed sediments and fine sediments (washload), only the bed sediment discharge can be estimated by the various equations that have been developed. The fine sediment discharge (washload) depends on its availability not on the transporting capacity of the flow and must be measured.

The sediment load that is measured by suspended-sediment samplers consists of both the washload (fine sediment load) and suspended-sediment load. The contact load is not measured, and because samplers cannot travel the total distance in the vertical to the bed, part of the suspended sediment in a vertical is not measured. Generally, the amount of bed material moving in contact with the bed of a large sandbed river is from 5 to 10 percent of the bed material moving in suspension. And in general the measured suspended-sediment discharge is from 90 to 95 percent of the total sediment discharge. However, in shallow sandbed streams with little or no washload the measured suspended-sediment load may be as small as 50 percent of the total load.

The magnitude of the suspended or bed sediment discharge can be very large. Suspended-sediment concentrations as large as 600,000 ppm or 60 percent by weight have been observed. Concentrations of this magnitude are largely fine sediments. By increasing fluid properties (viscosity and density) the fine material in the flow increases the capacity of the flow to transport bed material.

The sediment load of a stream at a cross-section or through a reach of a stream can be determined by measuring the suspended-sediment portion of the load using samplers and estimating the unmeasured discharge or by using one of the many methods that have been developed for computing the bed sediment load and estimating the washload. In many problems, only the bed sediment load, both in suspension and in contact with the bed, is important. In these cases the washload can be eliminated from the measured suspended-sediment load if the size distribution of the material is known.

There have been many equations developed for the estimation of bed sediment transport. The variation between the magnitude of the bed sediment discharge predicted by different equations under the same conditions is tremendous. For the same discharge, the predicted discharge can have a 100 fold difference between the smallest and the largest value. This can be expected given the number of variables, the interrelationship between them, the difficulty of measuring many of the variables and the statistical nature of bed material transport. Nevertheless, with proper use, knowledge of the river and knowledge of the limitations of each method, useful bed material discharge information can be obtained.

In the next sections the basic suspended-sediment equation is developed. Then, three methods of estimating bed-sediment discharge are described. These are Meyer-Peter Muller (1948), Einstein (1950), and Colby (1964). The Meyer-Peter Muller equation is applicable to streams with little or no suspended-sediment discharge and is thus used extensively for gravel and cobble bed streams. The other two methods, based to some degree on Einstein's work, are used for sandbed streams. Their use depends on the amount of information available. Methods for measuring suspended-sediment discharge are not described. For information on these methods, the reader is referred to the publications by Guy (1969).

3.6.6 Suspended Bed Sediment Discharge

The suspended bed sediment discharge in lb per second per unit width of channel, q_s, for steady, uniform two-dimensional flow is

$$q_s = \gamma \int_a^{y_o} v\, c\, dy \qquad (3.6.1)$$

where v and c vary with y and are the time-averaged flow velocity and volumetric concentrations, respectively. The integration is taken over the depth between the distance "a" above the bed and the surface of the flow "y_o". The level "a" is assumed to be 2 grain diameters above the bed layer. Sediment movement below this level is considered as bed load rather than suspended load.

The discharge of suspended sediment for the entire stream cross-section, Q_s, is obtained by integrating Equation 3.6.1 over the cross-section to give

$$Q_s = \gamma_s Q \overline{C} \qquad (3.6.2)$$

where $\overline{C}$ is the average suspended-sediment concentration by volume.

The vertical distribution of both the velocity and the concentration vary with the mean velocity of the flow, bed roughness and size of bed material. The distributions are illustrated in Figure 3.6.1. Also v and c are interrelated. That is, the velocity and turbulence at a point is affected by the sediment at the point, and the sediment concentration at the point is affected by the point velocity. Normally this interrelation is neglected or a coefficient applied to compensate for it.

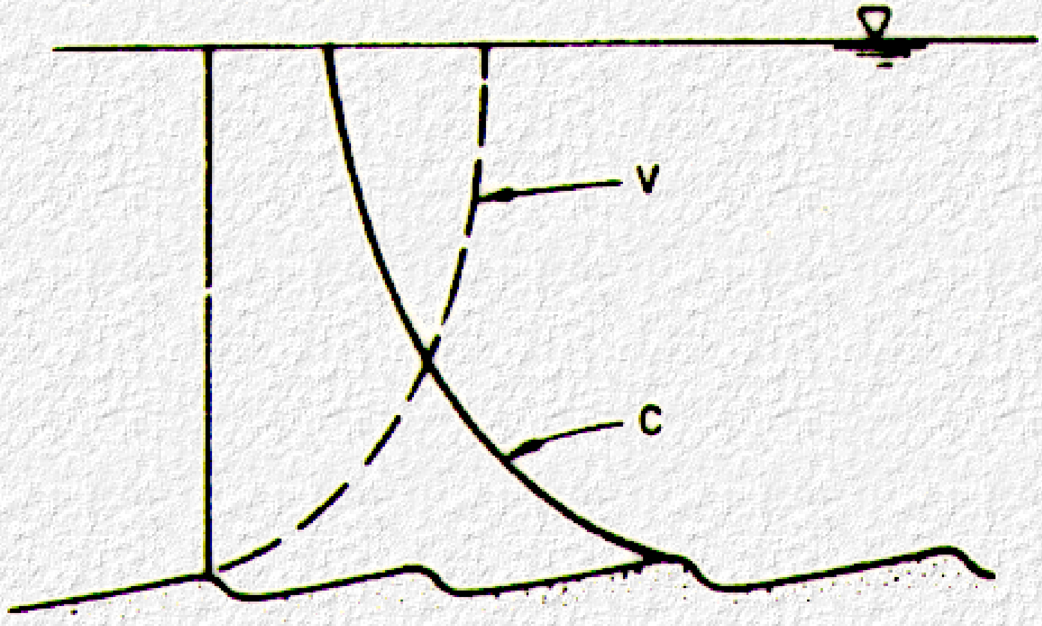

Figure 3.6.1. Schematic Sediment and Velocity Profiles

To integrate Equation 3.6.1, v and c must be expressed as functions of y. The one-dimensional gradient type diffusion equation is employed to obtain the vertical distribution for c and the logarithm velocity distribution is assumed for v in turbulent flows.

The one-dimensional diffusion equation describes the equilibrium condition when the quantity of sediment settling across a unit area due to the force of gravity is equal to the quantity of sediment transported upwards resulting from the vertical component of turbulence and the concentration gradient. The resulting equation for a given particle size is

$$\omega c = -\varepsilon_s dc / dy \qquad (3.6.3)$$

where

ω = the fall velocity of the sediment particle at a point;

c = the concentration of particles at elevation y above the bed;

ε_s = an exchange coefficient, also called the mass transfer coefficient, which characterizes the magnitude of the exchange of particles across any arbitrary boundary by the turbulence;

dc/dy = the concentration gradient;

ωc = the average rate of settling of the sediment particles; and

ε_s dc/dy = the average rate of upward sediment flow by diffusion.

Integrating Equation 3.6.3 yields

$$c = c_a \exp\left\{ -\omega \int_a^{y_0} dy / \varepsilon_s \right\} \qquad (3.6.4)$$

where c_a is a concentration of sediment with settling velocity ω at the level y = a in the flow.

In order to determine the value of c at a given y, the value of c_a and the variation of ε_s with y must be known. To obtain an expression for ε_s the assumption is made that

$$\varepsilon_s = b\,\varepsilon_m \tag{3.6.5}$$

where ε_m is the kinematic eddy viscosity or the momentum exchange coefficient defined by

$$\tau = \rho\varepsilon_m\, dv/dy \tag{3.6.6}$$

where τ and dv/dy are the shear stress and velocity gradient, respectively, at point y.

For two-dimensional steady uniform flow

$$\tau = \gamma\, S\,(y_o - y) = \tau_o\,(1 - y/y_o) \tag{3.6.7}$$

and from Equation 2.3.13

$$\frac{dv}{dy} = \frac{\sqrt{\tau_o / \rho}}{\kappa y} = \frac{V_*}{\kappa y} \tag{3.6.8}$$

Thus

$$\varepsilon_s = \beta\,\varepsilon_m = \beta\,\kappa\,V_*\,y\,(1 - y/y_o) \tag{3.6.9}$$

where

β = a coefficient relating ε_s to ε_m;

κ = the von Karman's velocity coefficient taken as equal to 0.4; and

V_* = the shear velocity equal to $\sqrt{gRS}$ in steady uniform flow.

Equation 3.6.9 indicates that ε_m and ε_s are zero at the bed and at the water surface, and have a maximum value at mid-depth. The substitution of Equation 3.6.9 into Equation 3.6.3 gives

$$\frac{dc}{c} = \frac{-\omega}{\beta V_*} \bullet \frac{dy}{y\left(1 - \dfrac{y}{y_o}\right)} \tag{3.6.10}$$

and after integration

$$\frac{c}{c_a} = \left[\frac{y_o - y}{y} \, \frac{a}{y_o - a} \right]^Z \qquad (3.6.11)$$

where

c = the concentration at a distance y from the bed;

c_a = the concentration at a point a above the bed; and

$Z = \omega / \beta \, \kappa V_*$, the Rouse number, named after the engineer who developed the equation in 1937.

Figure 3.6.2 shows a family of curves obtained by plotting Equation 3.6.11 for different values of the Rouse number Z. It is seen that for small values of Z, the sediment distribution is nearly uniform. For large Z values, little sediment is found at the water surface. The value of Z is small for large shear velocities V_* or small fall velocities ω. Thus, for small particles or for extremely turbulent flows, the concentration profiles are uniform. An example of the calculation of a sediment concentration profile is given in Appendix 3, Problem 3.6.

The values of β and κ have been investigated. For fine particles $\beta \sim 1$. Also, it is well known that in clear water $\kappa = 0.4$ but apparently decreases with increasing sediment concentration.

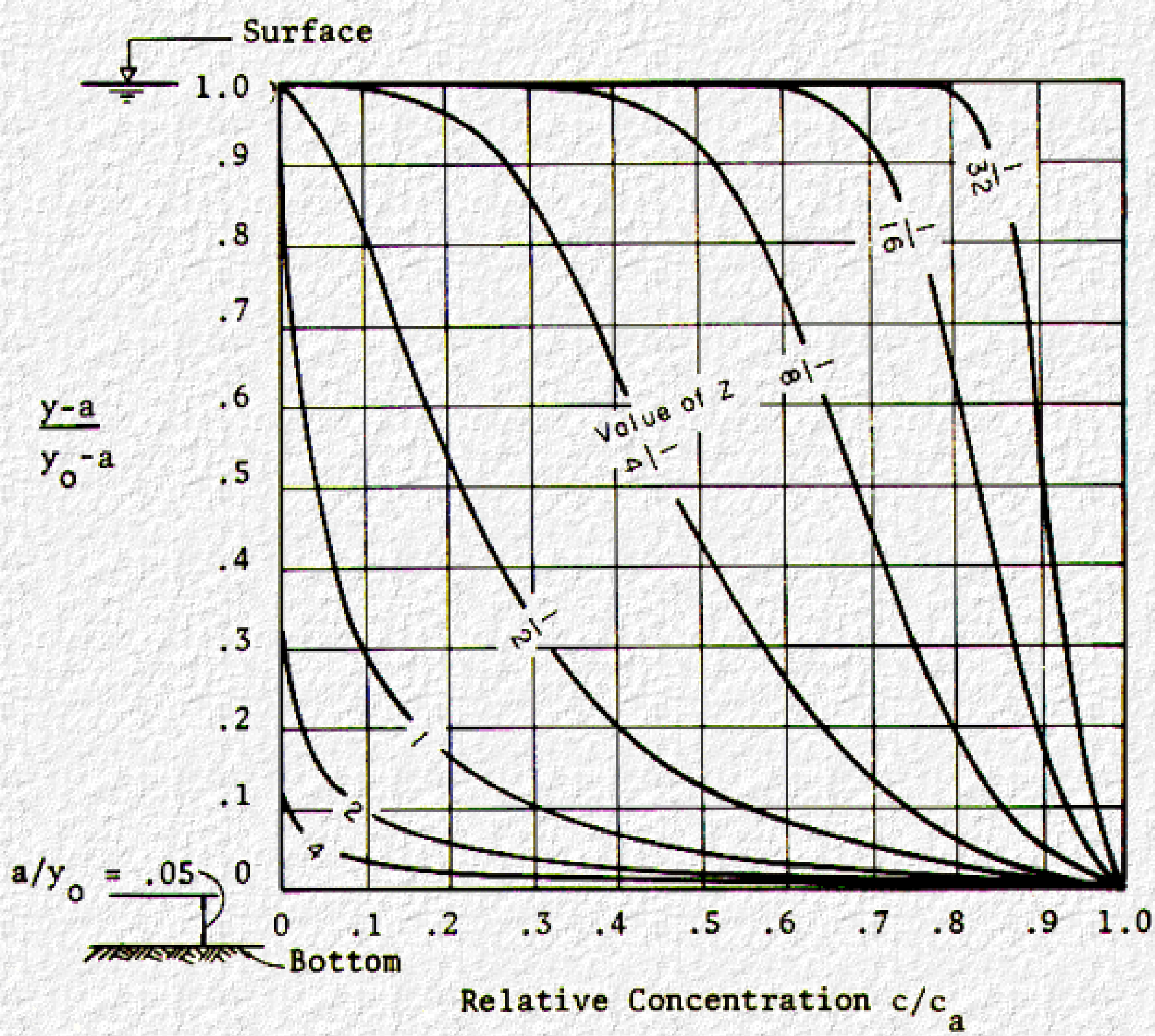

Figure 3.6.2. Graph of Suspended Sediment Distribution

Using the logarithmic velocity distribution for steady uniform flow and Equation 3.6.11 the equation for suspended sediment transport becomes

$$q_s = \gamma V_* c_a \int_a^{y_o} \left[\frac{y_o - y}{y}\,\frac{a}{y_o - a}\right]^Z \left[2.5 \ln\left(30.2\frac{Xy}{k_s}\right)\right] dy \qquad (3.6.12)$$

This equation has been integrated by many investigators and the assumptions and integration made by Einstein are presented in Section 3.6.8.

3.6.7 Meyer-Peter Muller Equation

Meyer-Peter and Muller (1948) developed the following equation based on experiments with sand particles of uniform sizes, sand particles of mixed sizes, natural gravel, lignite, and baryta:

$$\left(\frac{Q_b}{Q}\right)\left(\frac{K_B}{K_r}\right)^{3/2} \gamma y_o S_f = B'(\gamma_s - \gamma)D_m + B\left(\frac{\gamma}{g}\right)^{1/3}\left(\frac{\gamma_s - \gamma}{\gamma_s}\right)^{2/3} q_B^{2/3} \qquad (3.6.13)$$

where q_B is the bed-load rate in weight per unit time and per unit width, Q_b is water discharge quantity determining bed load transport, Q is total water discharge, y_o is the depth of flow, S_f is the energy slope and B' and B are dimensionless constants. B' has the value 0.047 for sediment transport and 0.034 for the case of no sediment transport. B has a value of 0.25 for sediment transport and is meaningless for no transport since q_B is zero and the last term drops out. The quantities K_B and K_r are defined by the expressions

$$V = K_B R^{2/3} S_f^{1/2} \qquad (3.6.14)$$

and

$$V = K_r R^{2/3} S_f'^{\,1/2} \qquad (3.6.15)$$

where S_f' is the part of the total slope, S_f, required to overcome the grain resistance and $S_f - S_f'$ is that part of the total slope required to overcome form resistance. Therefore

$$\frac{K_B}{K_r} = \sqrt{\frac{f_b'}{8}}\,\frac{V}{\sqrt{gRS_f}} \qquad (3.6.16)$$

where f_b' is the Darcy-Weisbach bed friction factor for the grain roughness. f_b' is determined from the Nikuradse, pipe friction data with D = 4R and $K_s = D_{90}$. If the boundary is hydraulically rough ($V_* D_{90}/\upsilon \geq 100$), K_r is given by

$$K_r = \frac{26}{D_{90}^{1/6}} \tag{3.6.17}$$

in which D_{90} is in meters.

Equation 3.6.13 is dimensionally homogeneous so that any consistent set of units may be used. Equation 3.6.13 has been converted to units generally used in the United States in the field of sedimentation for water and quartz particles by the U.S. Bureau of Reclamation (1960). This equation is

$$q_B = 1.606\left[3.306\left(\frac{Q_b}{Q}\right)\left(\frac{D_{90}^{1/6}}{n_b}\right)^{2/3} y_o S_f - 0.627 D_m\right]^{3/2} \tag{3.6.18}$$

where q_B is in tons per day per foot width, Q_b is the water discharge quantity determining the bed-load transport in cfs, Q is the total water discharge quantity in cfs, D_{90} and D_m are in millimeters. The quantity D_m is the effective diameter of the sediment given by

$$D_m = \frac{\sum_i p_i D_{si}}{100} \tag{3.6.19}$$

where p_i is the percentage by weight of that fraction of the bed material with geometric mean size, D_{si}. The quantity n_b for rectangular channels is

$$n_b = n\left[1 + \frac{2y_o}{W}\left(1 - \left(\frac{n_W}{n}\right)^{3/2}\right)\right]^{2/3} \tag{3.6.20}$$

and for trapezoidal channels

$$n_b = n\left\{1 + \frac{2y_o\left(1 + H_s^2\right)^{1/2}}{W}\left[1 - \left(\frac{n_W}{n}\right)^{3/2}\right]\right\}^{2/3} \tag{3.6.21}$$

where, n, n_b, and n_w are roughness coefficients of the total stream, of the bed, and of the banks, respectively; and H_s is the horizontal side slope related to one unit vertically and W is the bottom width.

The ratio Q_b/Q for rectangular channels is given by

$$\frac{Q_b}{Q} = \frac{1}{1+\left(\frac{2y_o}{W}\right)\left(\frac{n_w}{n_b}\right)^{3/2}} \tag{3.6.22}$$

and for trapezoidal channels is

$$\frac{Q_b}{Q} = \frac{1}{1+\frac{2y_o\left(1+H_s^2\right)^{1/2}}{W}\left(\frac{n_w}{n_b}\right)^{3/2}} \tag{3.6.23}$$

The Meyer-Peter and Muller formula (Equation 3.6.13) is often written in the form

$$q_b = K\,(\tau - \tau_c)^{3/2} \tag{3.6.24}$$

where

$$K = \left[\frac{1}{B\left(\frac{\gamma}{g}\right)^{1/3}\left(\frac{\gamma_s-\gamma}{\gamma_s}\right)^{2/3}}\right]^{3/2} \cong \frac{12.9}{\gamma_s\sqrt{\rho}} \tag{3.6.24a}$$

$$\tau = \left(\frac{Q_b}{Q}\right)\left(\frac{K_B}{K_r}\right)^{3/2}\gamma y_o S \tag{3.6.24b}$$

$$\tau_C = B'(\gamma_S - \gamma)D_m \tag{3.6.24c}$$

An example of sediment transport calculations using the Meyer-Peter and Muller equation is given in Appendix 3, Problem 3.7.

3.6.8 Einstein's Method

Einstein's (1950) method for determining total bed sediment discharge is the sum of the contact load and the suspended load. As mentioned earlier, there is no sharp demarcation between the contact bed sediment load and the suspended bed sediment. However, this division is warranted by the fact that there is a difference in behavior of the two different loads which justifies two physical models.

Einstein's bed sediment discharge function gives the rate at which flow of any magnitude in a given channel transports the individual sediment sizes which make up the bed material. This makes his equations extremely valuable in many studies for determining the change in bed material with time that occurs because each size moves at its own rate. For each size D_s of the bed material, the contact load is given as

$$i_B\, q_B \tag{3.6.25}$$

and the suspended sediment load is given by

$$i_s\, q_s \qquad (3.6.26)$$

and the total bed material discharge is

$$i_T q_T = i_s q_s + i_B q_B \qquad (3.6.27)$$

and finally

$$Q_T = \Sigma\, i_T q_T \qquad (3.6.28)$$

where i_T, i_s, and i_B are the fractions of the total, suspended and contact bed sediment discharges q_T, q_s and q_B for a given grain size D_s. The term Q_T is the total bed sediment transport. The suspended sediment total is related to the contact load because there is a continuous exchange of particles between the two modes of transport.

With suspended sediment load related to the contact load, Equation 3.6.27 becomes

$$i_T q_T = i_B q_B\,(1 + P_E\, I_1 + I_2) \qquad (3.6.29)$$

where

$$i_B q_B = \frac{\phi_* i_B \gamma_s}{\left(\dfrac{\rho}{\rho_s - \rho}\,\dfrac{1}{g D_s^3}\right)^{1/2}} \qquad (3.6.30)$$

and

γ_s = the unit weight of sediment;

ρ = the density of the water;

ρ_s = the density of the sediment;

g = gravitational acceleration;

ϕ_* = dimensionless sediment transport function

= f (ψ_*) given in Figure 3.6.3.

$$\psi_* = \xi Y (\log 10.6/B_x)^2\, \psi \qquad (3.6.31)$$

$$\Psi = \left(\frac{\rho_s - \rho}{\rho}\right)\frac{D_s}{R'_b\, S_f}\,; \qquad (3.6.32)$$

ξ = a correction factor given as a function of $D_s/\overline{X}$ in Figure 3.6.4;

$$\overline{X} = 0.77\Delta, \text{ if } \Delta/\delta' > 1.8; \qquad (3.6.33a)$$

$\overline{X}$ = 1.39δ', if Δ/δ' < 1.8; (3.6.33b)

Δ = the apparent roughness of the bed, k_s/X;

X = a correction factor in the logarithmic velocity distribution equation and is given as a function of k_s/δ' in Figure 2.3.5;

$\delta' = 11.6n/V_*'$; (3.6.34)

v/V_*' = Einstein's velocity distribution equation

$= 5.75 \log (30.2\, y/\Delta)$; (3.6.35)

V_*' = the shear velocity due to grain roughness

$= \sqrt{gR_b'S}$; (3.6.35a)

R'_b = the hydraulic radius of the bed due to grain roughness,

$= R_b - R''_b$;

R''_b = the hydraulic radius of the bed due to channel irregularities

S_f = the slope of the energy grade line normally taken as the slope of the water surface;

Y = another correction term given as a function of D_{65}/δ' in Figure 3.6.5; and

$B_x = \log (10.6\, \overline{X}/\Delta)$.

The preceding equations are used to compute the fraction i_B of the load. The other terms in Equation 3.6.29 are

$$P_E = 2.3 \log 30.2\, y_o/\Delta \qquad (3.6.36)$$

I_1 and I_2 are integrals of Einstein's form of the suspended sediment Equation 3.6.11

$$I_1 = 0.216 \frac{E^{Z-1}}{(1-E)^Z} \int_E^1 \left[\frac{1-y}{y}\right]^Z dy \qquad (3.6.37)$$

$$I_2 = 0.216 \frac{E^{Z-1}}{(1-E)^Z} \int_E^1 \left[\frac{1-y}{y}\right]^Z \ln y\, dy \qquad (3.6.38)$$

where

$$Z = \omega/0.4V'_*; \qquad (3.6.39)$$

ω = the fall velocity of the particle of size D_s;

E = the ratio of bed layer thickness to flow depth, a/y_o;

y_o = depth of flow; and

a = the thickness of the bed layer, $2D_{65}$.

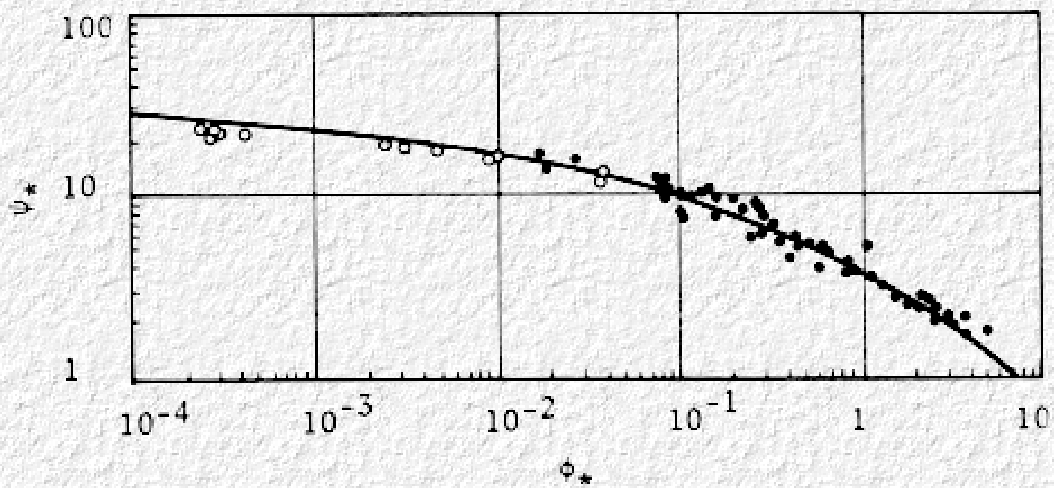

Figure 3.6.3. Einstein's ϕ_* vs ψ_* Bed Load Function, (Einstein, 1950)

The two integrals I_1 and I_2 are given in Figure 3.6.6 and Figure 3.6.7 as a function of Z and E.

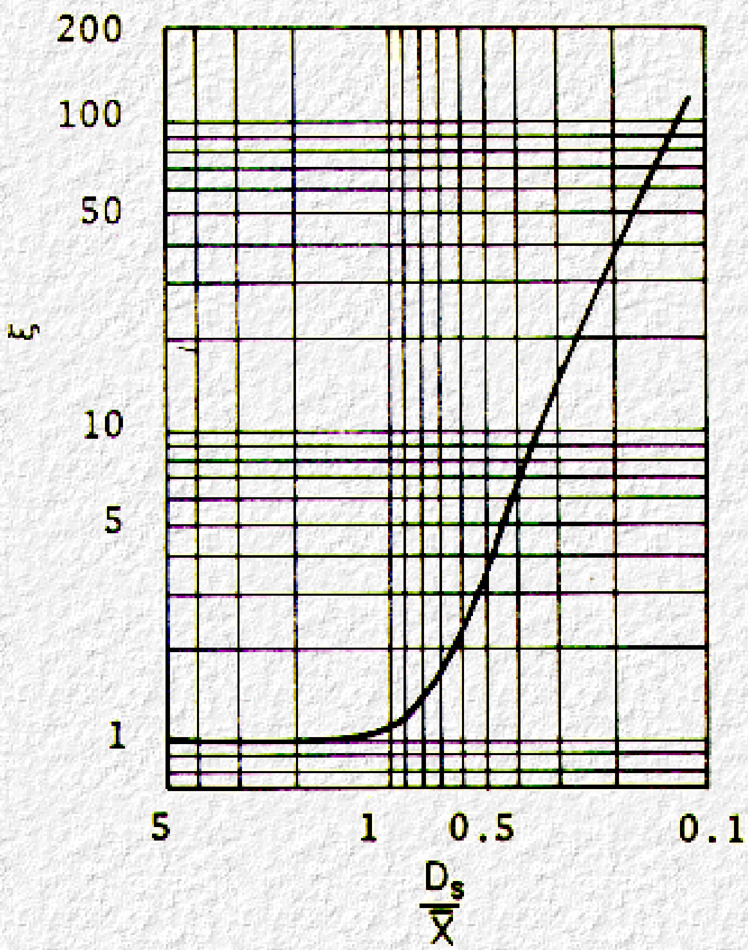

Figure 3.6.4. Hiding Factor, (Einstein, 1950)

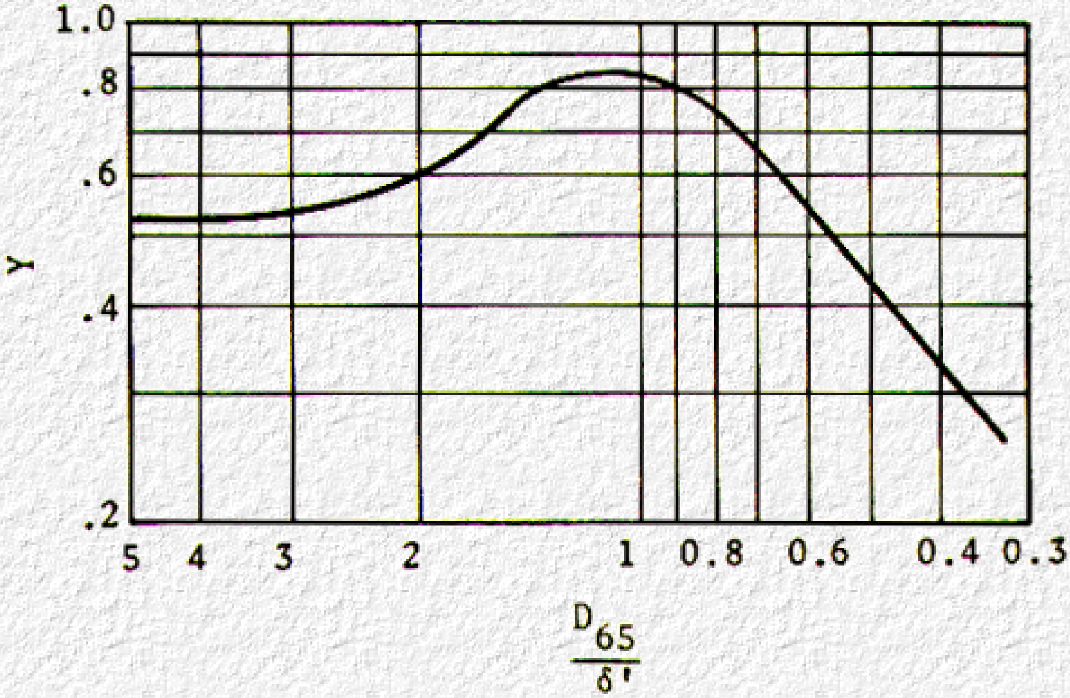

Figure 3.6.5. Pressure Correction, (Einstein, 1950)

In the preceding calculations for the total load, the shear velocity is based on the hydraulic radius of the bed due to grain roughness R'_b. Its computation is explained in the following paragraph.

Total resistance to flow is composed of two parts, surface drag and form drag. The transmission of shear to the boundary is accompanied by a transformation of flow energy into energy turbulence. The part of energy corresponding to grain roughness is transformed into turbulence which stays at least for a short time in the immediate vicinity of the grains and has a great effect on the bed load motion; whereas, the other part of the energy which corresponds to the form resistance is transformed into turbulence at the interface between wake and free stream flow, or at a considerable distance away from the grains. This energy does not contribute to the bed load motion of the particles and may be largely neglected in the sediment transportation.

Einstein's equation for mean flow velocity V in terms of V'_* is

$$V/V'_* = 5.75 \log (12.26\, R'_b/\Delta) \qquad (3.6.40a)$$

or

$$V/V'_* = 5.75 \log (12.26\, R'_b/k_s X) \qquad (3.6.40b)$$

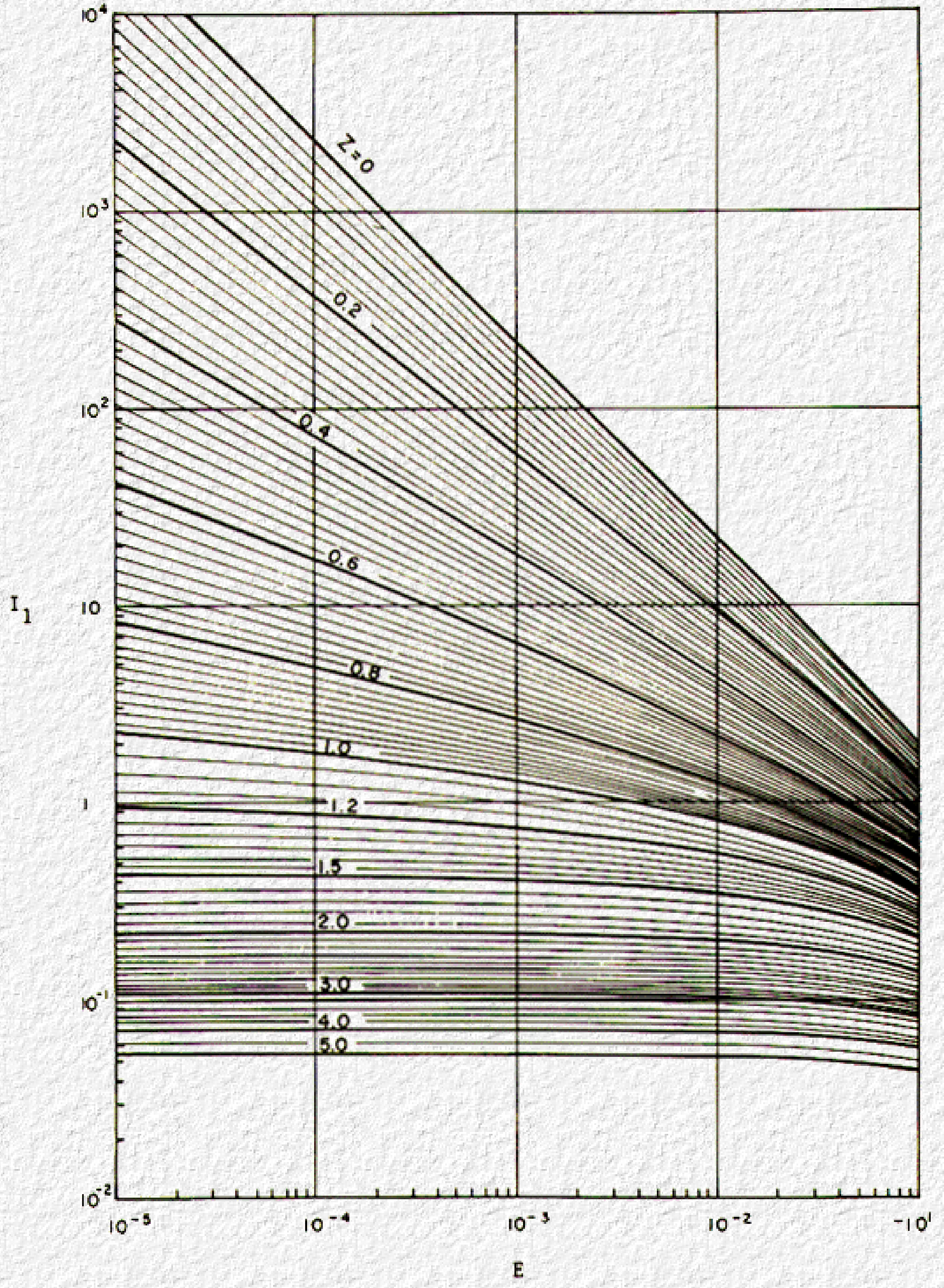

Figure 3.6.6. Integral I_1 in Terms of E and Z, (Einstein, 1950)

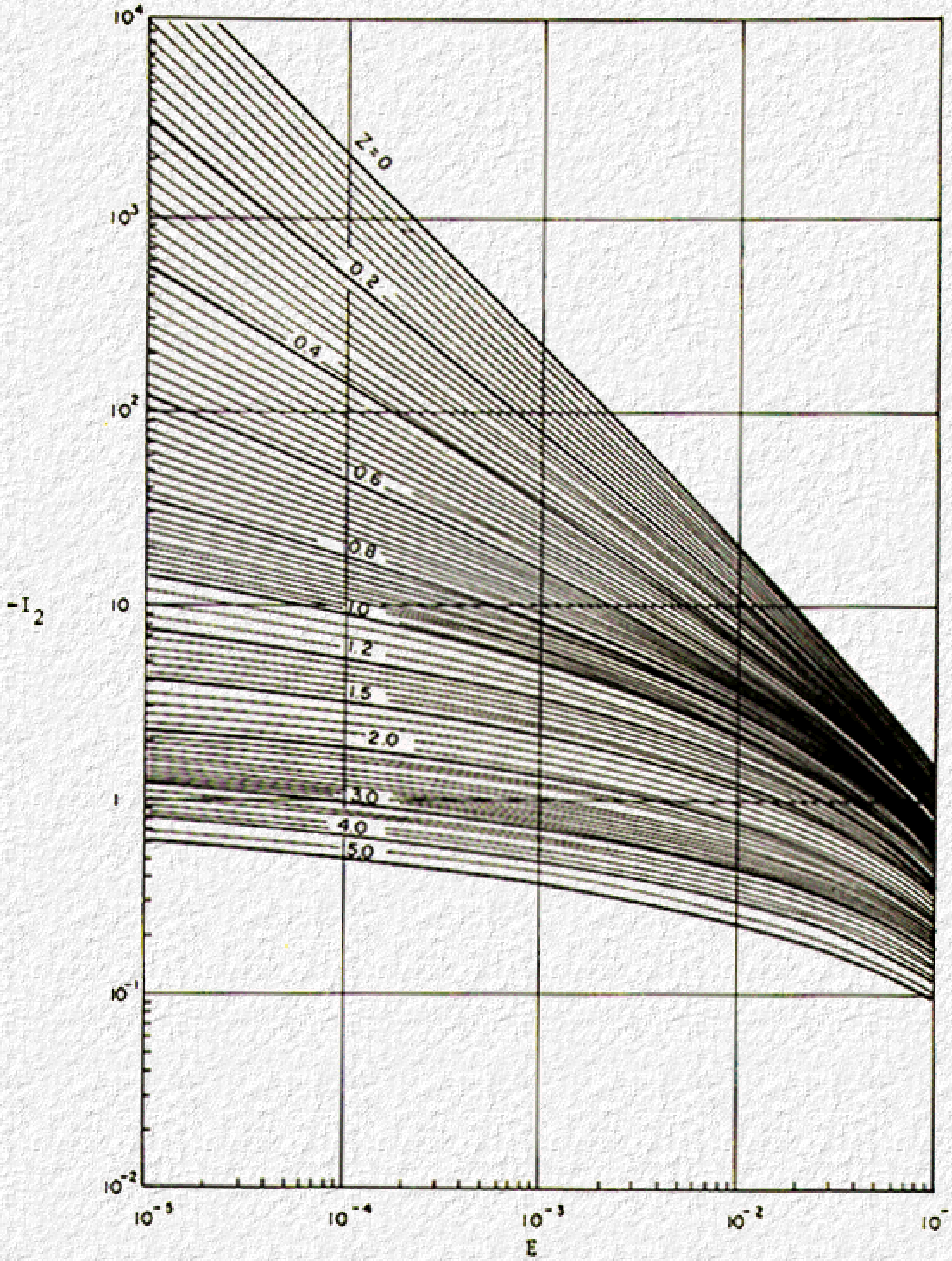

Figure 3.6.7. Integral I_2 in Terms of E and Z, (Einstein, 1950)

Furthermore, Einstein suggested that

$$V/V''_{*} = \theta\ [\psi'] \tag{3.6.41}$$

where

$$\Psi' = \frac{\rho_s - \rho}{\rho} \frac{D_{35}}{R'_b S_f} \tag{3.6.42}$$

The relation for Equation 3.6.42 is given in Figure 3.6.8. The procedure to follow in computing R'_b depends on the information available. If mean velocity V, slope S, hydraulic radius R_b and bed material size are known, then R'_b is computed by trial and error using Equation 3.6.40 and Figure 3.6.8.

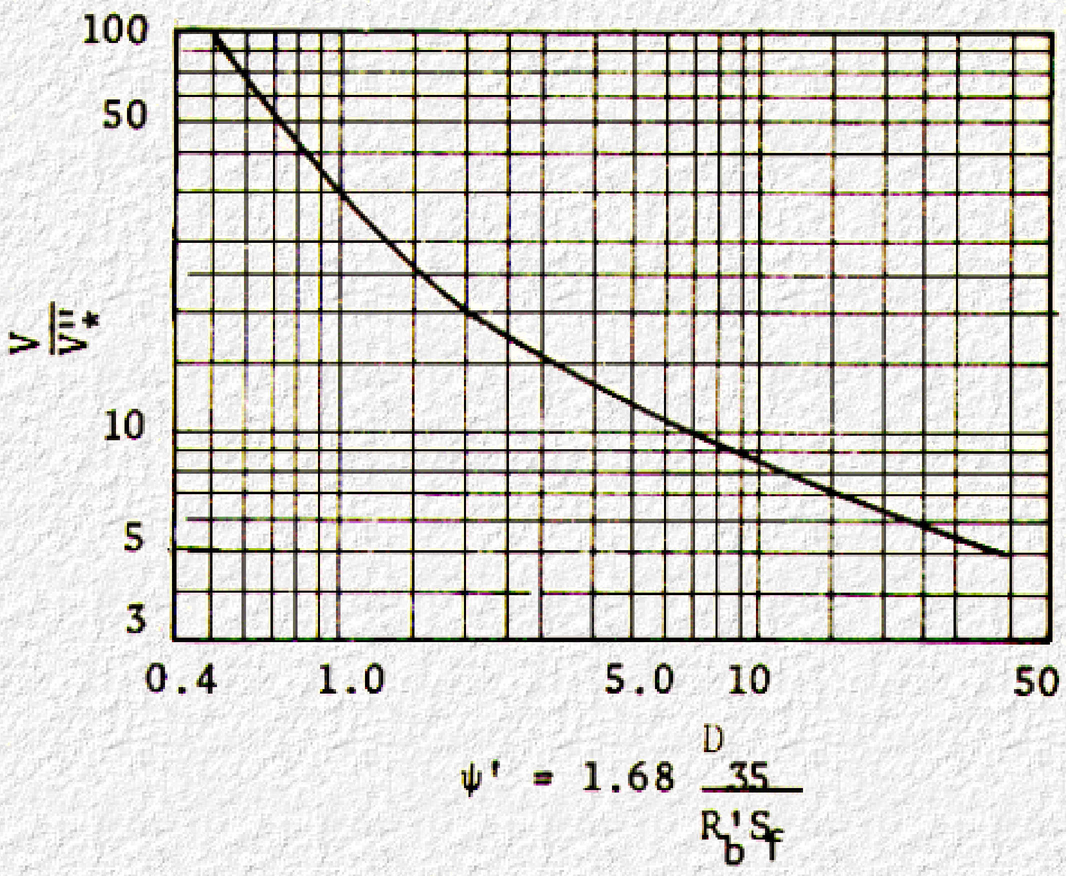

Figure 3.6.8. V/V"$_*$ vs. ψ' (Einstein, 1950)

The procedure for computing total bed sediment discharge in terms of different size fractions of the bed material is:

1. Calculate ψ_* using Equation 3.6.31 for each size fraction;
2. Find ϕ_* from Figure 3.6.3 for each size fraction;
3. Calculate i_B q_B for each size fraction using Equation 3.6.30;
4. Sum up the q_B across the flow to obtain $i_B Q_b$; and
5. Sum up the size fractions to obtain Q_b;

For the suspended sediment discharge:

6. Calculate Z for each size fraction using Equation 3.6.39;
7. Calculate $E = 2D_s/y_o$ for each fraction;
8. Determine I_1 and I_2 for each fraction from Figure 3.6.6 and Figure 3.6.7;
9. Calculate P_E using Equation 3.6.36;

10. Compute the suspended discharge from $i_B q_B$ ($P_E I_1 + I_2$); and
11. Sum up all the q_B and all the i_B to obtain the total suspended discharge Q_{ss}.

Thus, the total bed sediment discharge:

12. Add the results of Step 5 and 11.

A sample problem showing the calculation of the total bed sediment discharge using Einstein's procedure is presented in Appendix 3, Problem 3.8.

3.6.9 Colby's Method of Estimating Total Bed Sediment Discharge

After investigating the effect of all the pertinent variables, Colby (1964) developed four graphical relations shown in Figure 3.6.9 and Figure 3.6.10 for determining the bed sediment discharge. In arriving at his curves, Colby was guided by the Einstein bed-load function (Einstein, 1950) and a large amount of data from streams and flumes. However, it should be understood that all curves for 100 ft depth, most curves of 10 ft depth and part of the curves of 1.0 and 0.1 ft are not based on data but are extrapolated from limited data and theory.

In applying Figure 3.6.9 and Figure 3.6.10 to compute the total bed sediment discharge, the following procedure is proposed: (1) the required data are the mean velocity V, the depth y_o, the median size of bed material D_{50}, the water temperature T° and the fine sediment concentration C_f; (2) the uncorrected sediment discharge Q_n for the given V, y_o and D_{50} can be found from Figure 3.6.9 by first reading q_n knowing V and D_{50} for two depths that bracket the desired depth and then interpolating on a logarithmic graph of depth versus q_n, to get the bed sediment discharge per unit width; (3) the two correction factors k_1 and k_2 shown in Figure 3.6.10 account for the effect of water temperature and fine suspended sediment on the bed sediment discharge. If the bed sediment size falls outside the 0.20 mm to 0.30 mm range, the factor k_3 from Figure 3.6.10 is applied to correct for the effect of sediment size; and (4) the unit bed sediment discharge q_T corrected for the effect of water temperature, presence of fine suspended sediment and sediment size is given by the equation

$$q_T = [1 + (k_1 k_2 - 1)\, k_3]\, q_n \qquad (3.6.43)$$

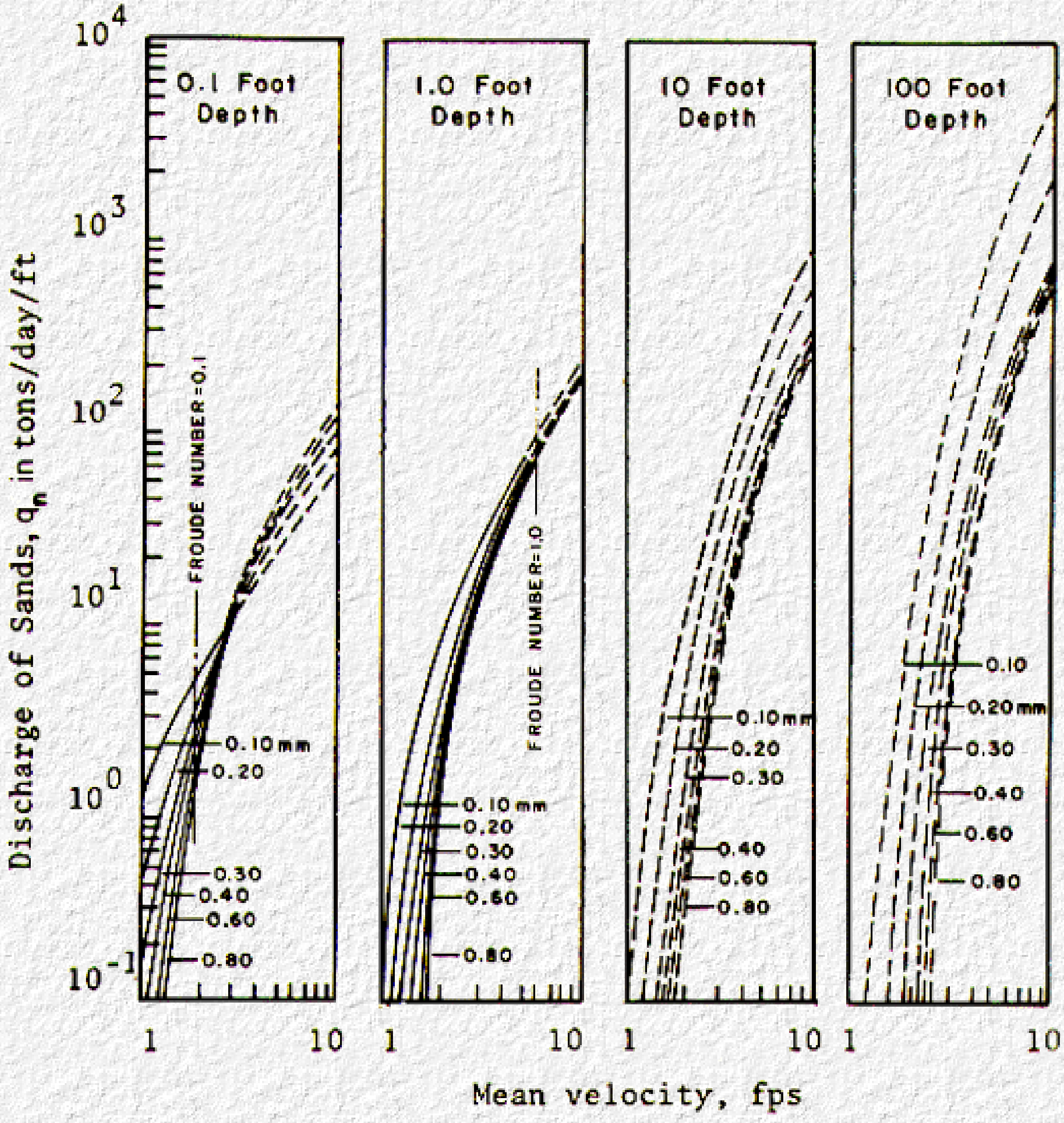

Figure 3.6.9. Relation of Discharge of Sands to Mean Velocity for Six Median Sizes of Bed Sands, Four Depths of Flow, and a Water Temperature of 60ºF (Colby, 1964)

As Figure 3.6.10 shows, k_1 = 1 when the temperature is 60**º** F, k_2 = 1 when the concentration of fine sediment is negligible and k_3 = 1 when D_{50} lies between 0.2 mm and 0.3 mm. The total sand discharge is

$$Q_T = Wq_T \tag{3.6.44}$$

where W is the width of the stream.

In spite of many inaccuracies in the available data and uncertainties in the graphs, Colby (1964) found that ". . . . about 75 percent of the sand discharges that were used to define the relationships were less than twice or more than half of the discharges that were computed from the graphs of average relationship. The agreement of computed and observed discharges of sands for sediment stations whose records were not used to define the graphs seemed to be about as good as that for stations whose records were used." An example showing bed sediment discharge calculations by the Colby method is presented in Appendix 3, Problem 3.9.

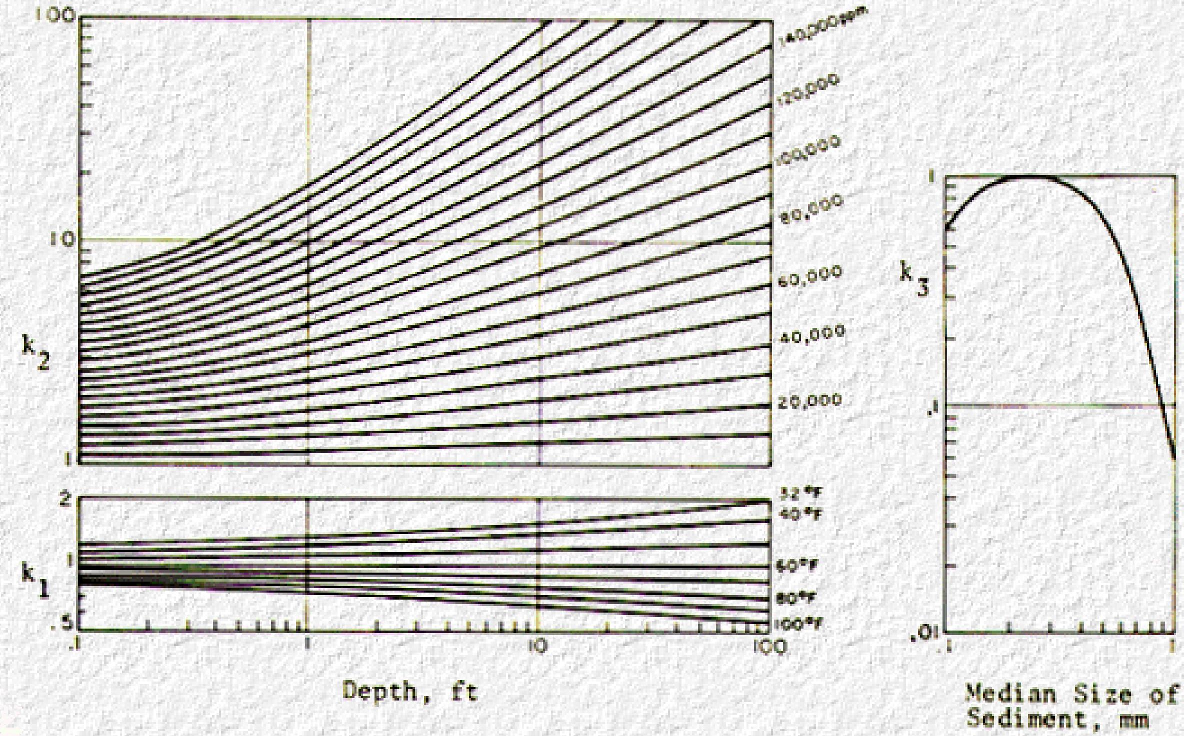

Figure 3.6.10. Colby's Correction Curves for Temperature and Fine Sediment (Colby, 1964)

3.6.10 Comparison of the Meyer-Peter, Muller and Einstein Contact Load Equations

Chien (1954) has shown that the Meyer-Peter and Muller, equation can be modified into the form

$$\phi_* = \left(\frac{4}{\Psi_*} - 0.188 \right)^{3/2} \qquad (3.6.45)$$

Figure 3.6.11 shows the comparison of Equation 3.6.45 with Einstein's ψ_* vs. ϕ_* relation for uniform bed sediment size and for sediment mixtures using D_{35} in the Einstein relation and D_{50} in the Meyer-Peter and Muller relation. They show good agreement.

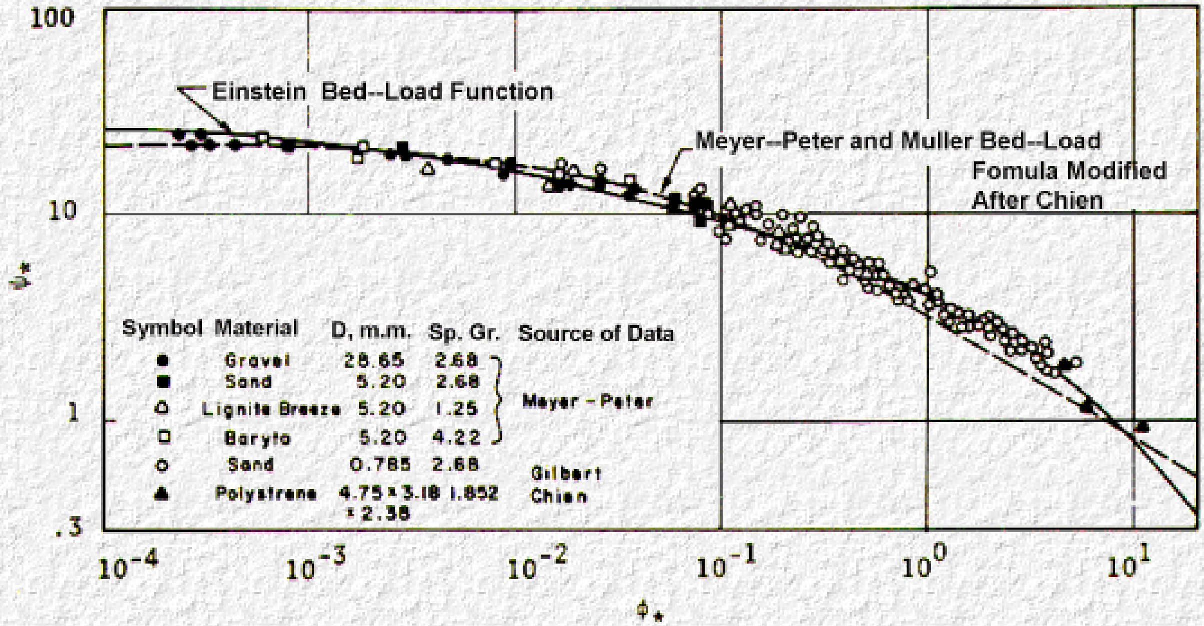

Figure 3.6.11. Comparison of the Meyer-Peter, Muller and Einstein Methods for Computing Contact Load (Chien, 1954)

3.6.11 Power Relationships

The following paragraphs describe an efficient method of evaluating sediment discharge. The method is based on easy-to-apply power relationships that estimate sediment transport based on the velocity and depth of flow. The power relationships were developed from computer-generated data obtained from the solution of the Meyer-Peter, Muller bedload transport equation and Einstein's integration of the suspended bed sediment discharge. This procedure is detailed in Simons, Li and Associates (1982).

The results of the total bed sediment discharge are presented in Table 3.6.1. The high values of a_3 ($3.3 < a_3 < 3.9$) show the high level of dependence that sediment transport rates have with respect to velocity. Comparatively, the influence of depth is less important ($-0.34 < a_2 < 0.7$). For smaller sizes the exponent a_2 is positive since the smaller material is more easily suspended and the resulting sediment concentration profiles are more uniform. Thus, the larger the depth, the more sediment will be suspended at a given velocity. For the larger sediment sizes the sediment is more difficult to suspend and keep in suspension. As the depth increases for a given velocity, the intensity of the turbulent transfer properties decreases for these sizes. The increase in area available for suspended sediment with the increased depth does not totally counterbalance the reduced turbulent transfer characteristics. The result is an inverse dependence of transport rate on depth for these larger sizes. The sizes with no dependence ($a_2 = 0$) on depth in their transport rate fall between these two extremes.

When applying the equations given in Table 3.6.1, care should be taken so that the range of parameters being used is not out of the range used to develop the equations. If conditions are within the ranges outlined in Table 3.6.2 the regression equations should provide results within ten percent of the theoretically computed values.

There are several other checks that should be made in order to ensure the equations are applicable to a given problem. The equations are based on the assumption that all the sediment sizes present can be moved by the flow. If this is not true armoring will take place. The equations are not applicable when armoring occurs. This can be determined using Shields' critical shear stress criteria. The bed shear stress is given by

$$\tau_o = \gamma R S_f$$

in which γ is the unit weight of water, R is the hydraulic radius and S_f is the friction slope. The diameter of the largest particles moving is then

$$D_s = \frac{\tau_o}{0.047\left(\gamma_s - \gamma\right)} \qquad (3.6.46)$$

in which D_s is the diameter of the sediment, γ_s is the unit weight of sediment and 0.047 is the recommended value of the Shields parameter. (All units are in feet, pounds and seconds) If no sediment of the computed size or larger is present in significant quantities, the equations are applicable.

The equations were developed for sand-bed channels. Therefore, they do not apply to conditions when the bed material is cohesive. The equations would overpredict transport rates in a cohesive channel. An example problem showing the calculation of sediment transport using these power relationships is given in Appendix 3, Problem 3.10.

Table 3.6.1 Power Equations for Total Bed Sediment Discharge in Sand and Fine Gravel Bed Streams

$q_s = a_l\, y_o^{a2}\, V^{a3}$							
d_{50} = 0.1mm	**d_{50} = 0.25mm**	**d_{50} = 0.5mm**	**d_{50} = 1.0mm**	**d_{50} = 2.0mm**	**d_{50} = 3.0mm**	**d_{50} = 4.0mm**	**d_{50} = 5.0mm**
G = 1.0							
a_1 3.30x10^{-5} a_2 0.715 a_3 3.30	1.42 x 10^{-5} 0.495 3.61	7.6 x 10^{-6} 0.28 3.82	5.62 x 10^{-6} 0.06 3.93	5.64 x 10^{-6} -0.14 3.95	6.32 x 10^{-6} -0.24 3.92	7.10 x 10^{-6} -0.30 3.89	7.78 x 10^{-6} -0.34 3.87
G = 2.0							
a_1 a_2 a_3	1.59 x 10^{-5} 0.51 3.55	9.8 x 10^{-6} 0.33 3.73	6.94 x 10^{-6} 0.12 3.86	6.32 x 10^{-6} -0.09 3.91	6.62 x 10^{-6} -0.196 3.91	6.94 x 10^{-6} -0.27 3.90	
G = 3.0							
a_1 a_2 a_3		1.21 x 10^{-5} 0.36 3.66	9.14 x 10^{-6} 0.18 3.76	7.44 x 10^{-6} -0.02 3.86			
G = 4.0							
a_1 a_2 a_3			1.05 x 10^{-5} 0.21 3.71				
q_s = sediment transport rate in ft^2/sec (unbulked) y_o = depth in feet)				V = velocity in ft/sec G = gradation coefficient (Equation 3.2.9)			

Table 3.6.2 Range of Parameters Examined

Parameter	Value Range
Froude Number	1 - 4
Velocity	6.5 - 26 (ft/sec)
Manning's n	0.015 - 0.025
Bed Slope	0.005 - 0.040
Unit Discharge	10 - 200 (cfs/ft)
Particle Size	$d_{50} \geq 0.062$mm $d_{90} \leq 15$mm

3.6.12 Relative Influence of Variables on Bed Material and Water Discharge

The study of the relative influence of viscosity, slope, bed sediment size and depth on bed sediment and water discharge is examined in detail using Einstein's bed-load function (1950) and Colby's (1964) relationships. Einstein's bed-load function is chosen because it is the most detailed and comprehensive treatment from the point of fluid mechanics. Colby's relations are chosen because of the large amount and range of data used in their development.

The data required to compute the total bed material discharge using Einstein's relations are: S = channel slope; D_{65} = size of bed material for which 65 percent is finer; D_{35} = size of bed material for which 35 percent is finer; D_i = size of bed sediment in fraction i; υ = kinematic viscosity; n_w = Manning's wall friction coefficient; A = cross-sectional area; P_b = wetted perimeter of the bed; P_w = wetted perimeter of the banks; i_B = percentage of bed sediment in fraction i; γ_s = specific weight; and V = average velocity.

To study the relative influence of variables on bed material and water discharges, the data taken by the U.S. Geological Survey from October 1, 1940 to October 1, 1970 on the Rio Grande near Bernalillo are used. The width of the channel reach was 270 ft. In the analysis the energy slope was varied from 0.7S to 1.5S, in which S is the average bed slope assumed to be equal to the average energy slope. Further, the kinematic viscosity was varied to correspond with variations in temperature from 39.2° to 100° F inclusive. The variation of D_{65}, D_{35}, D_i and i_B was accomplished by using the average bed material distribution given by Nordin (1964) and shifting the curve representing the average bed sediment distribution along a line parallel to the abscissa drawn through D_{50}. The average water temperature was assumed to be equal to 70° F and the average energy gradient of the channel was assumed to be equal to 0.00095 ft/ft = 5.0 ft/mi. The water and sediment discharges were computed independently for each variation of the variables and for three subreaches of the Rio Grande of different width near Bernalillo. The applicability of the results depend on the reliability of the modified Einstein bed-load function and Colby's relationships used in the analysis rather than on the choice of data.

The computed water and sediment discharges are plotted in Figure 3.6.12, Figure 3.6.13 and Figure 3.6.14 and show the variation of sediment discharge due to changes in bed material size, slope and temperature for any given water discharge. Figure 3.6.12 shows that when the bed sediment becomes finer, the sediment discharge increases considerably. The second most important variable affecting sediment discharge is the slope variation (see Figure 3.6.13). Temperature is third in importance (Figure 3.6.14). The effects of variables on sediment discharge were studied over approximately the same range of variation for each variable.

Figure 3.6.15 shows the variation of the sediment discharge due to changes in the depth of flow for any given discharge, computed using Colby's (1964) relations. The values of depth of flow varied from 1.0 to 10.0 ft, the median diameter of the bed sediment is maintained constant equal to 0.030mm, the water temperature is assumed constant and the concentration of fine sediment is assumed less than 10,000 ppm. The channel width is also maintained constant at 270 ft. In Figure 3.6.15, the curves for constant depth of flow show a steep slope. This indicates that the capacity of the stream to transport sands increases very fast for a small increase of discharge at constant depth. Similar figures can be developed for other sizes of bed material, and the relations can be modified to include the effect of washload and viscosity effects.

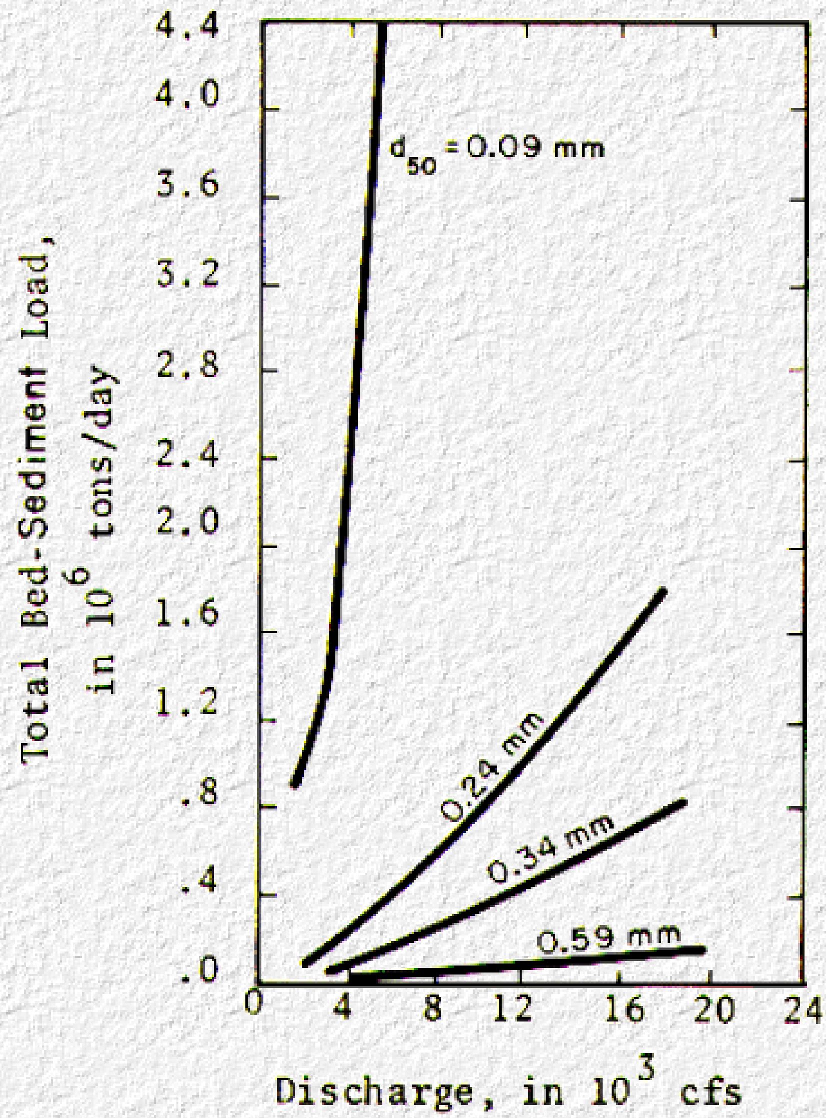

Figure 3.6.12. Bed-Material Size Effects on Bed Material Transport

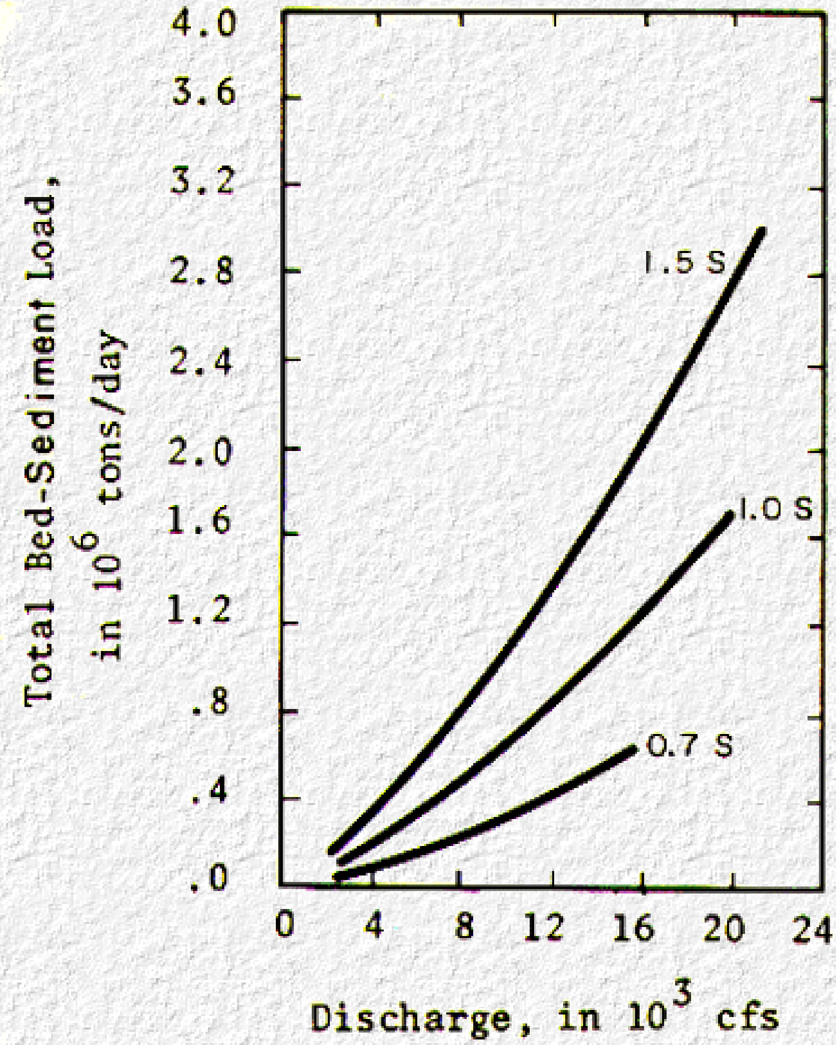

Figure 3.6.13. Effect of Slope on Bed Material Transport

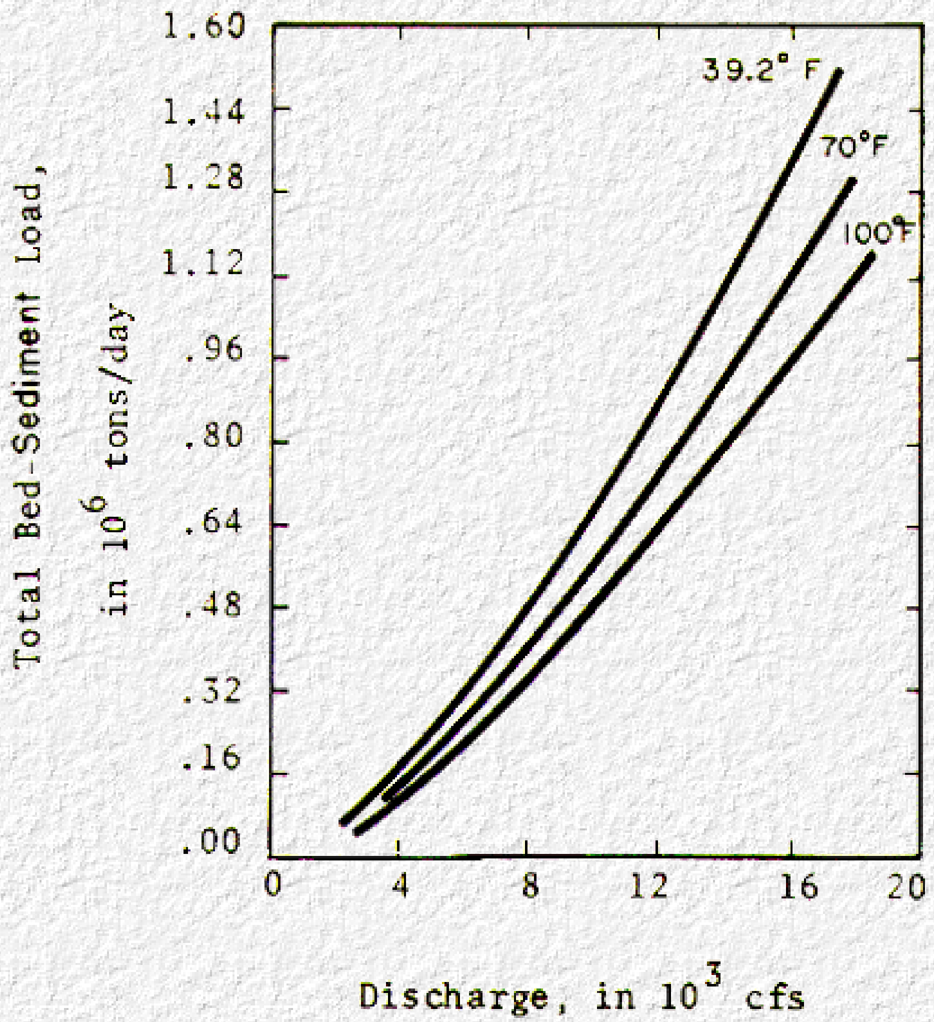

Figure 3.6.14. Effect of Kinematic Viscosity (Temperature) on Bed Material Transport

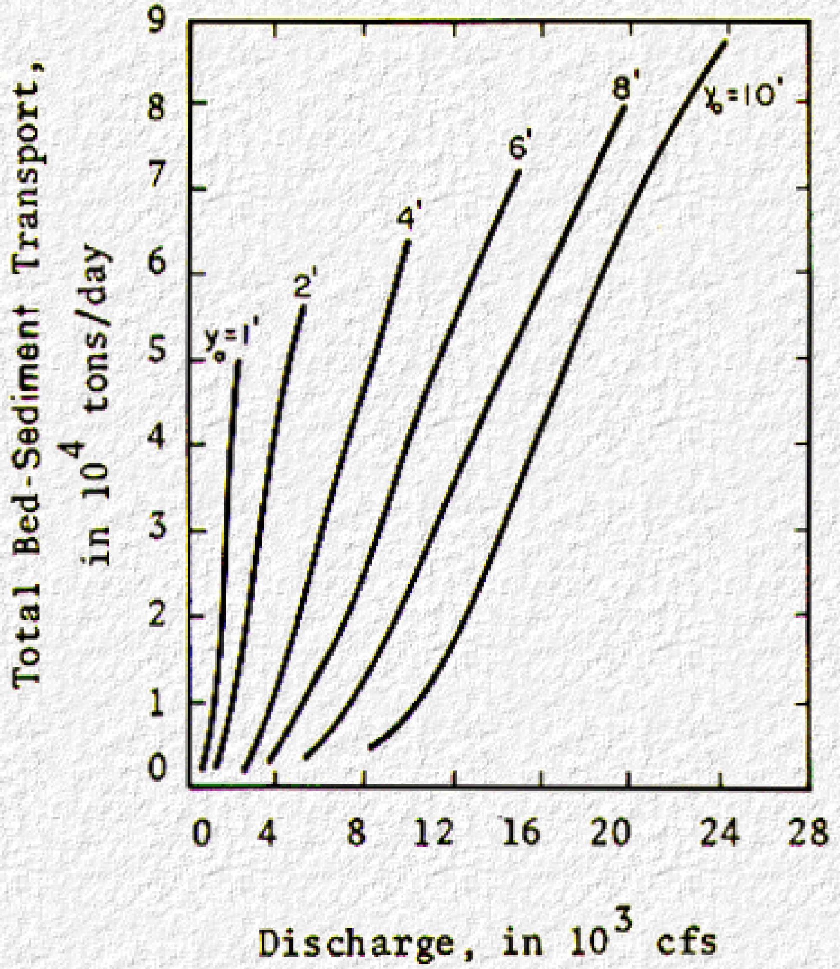

Figure 3.6.15. Variation of Bed Material Load with Depth of Flow

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3.7 Sediment Problems at Bridge Openings and Culverts

3.7.1 Sediment Transport in Coarse Material Channels

In considering the sediment transport in coarse-material channels, the distinction between washload and bed material load is reemphasized. The reason is that in such channels, particles as big as coarse sand may behave as washload. These particles may not be available in sufficient quantities in the bed surface and yet may constitute a large part of the total sediment load. It is repeated here, that the washload in an alluvial channel cannot be related to the local flow or the channel bed and therefore is not predictable from the known channel flow properties.

The bed material load in coarse-bed channels is mostly transported as bed load and not as suspended load. For the bed-load transport, Einstein's bed-load function (without the suspended-load component) and the Meyer-Peter Muller transport function have been found to be fairly useful.

The time response of coarse-material channels also is different from sandbed channels in the time scale of response. This time response is dominated by two factors: (1) the difference in particle size between the surface (armor) layer of the bed and the bed material below it; and (2) the washload may extend to coarse sand sizes. These factors are discussed below.

The formation of an armor layer on the bed may immobilize the bed for a large part of the hydrograph. However, if the conditions for incipient movement of this layer are exceeded, the underlying finer bed material will be readily picked up by the flow. The channel then establishes an armor of larger size particles for which a substantial depth of the bed may be degraded. Thus, extreme flow events in coarse-material channels are capable of inducing rapid and large bed-level changes.

The coarse sand and larger particles may behave as washload in coarse material channels; that is, although the flow may be transporting a large quantity of these particles, the boundary shear may be large, so that these particles are not found in appreciable quantities in the armor layer. If the boundary shear is reduced by afflux at a highway crossing, the flow may not sustain this material as wash load and rapid aggradation may occur. In general, afflux at highway crossings induces rapid and more pronounced channel response in coarse material channels.

3.7.2 Sediment Transport at Bridge Openings

Sediment problems at bridge openings must be considered when significant constriction is imposed by the bridge approach or training works. Scour at bridges can be divided in four main categories: (1) general scour due to constriction; (2) local scour due to piers and abutments; (3) scour related to bed-form and channel shifting; and (4) geological or man-made regime changes. Methods for estimating scour are detailed in Chapter 5 and scour expected at a given bridge site is generally a combination of some or all of these individual effects.

Scour at bridge crossings can be related to the following factors: (1) slope, alignment and channel shifting; (2) bed sediment size distribution; (3) antecedent floods and surging phenomena; (4) accumulation of debris, logs or ice; (5) constriction in flow, alignment, training works and riprap stability; (6) pier geometry and location; (6) type of foundation; (7) natural or man-induced modification of the river; and (8) natural or man-induced failure of a nearby structure.

Rate of scour depend on the erosive forces exerted on the channel boundary and the resistive forces of the material. Resistance to erosion in fine cohesive material results from chemical bonding. Cohesionless materials do not exhibit such properties and resistance to erosion depends primarily on bed sediment size distribution and density.

Under steady flow conditions, scour situations gradually reach equilibrium condition. However, most rivers are active during a very short period of time and equilibrium scour conditions are not necessarily attained during a single event. Unsteady flow conditions are generally observed and a series of events are required to reach equilibrium. During a typical flood hydrograph, experiments indicate that scour tends to lag behind discharge and maximum scour depth occurs after the flood peak. Deposition often occurs during the recession of the hydrograph and the maximum scour depth measured after the flood is generally less than the maximum depth of scour reached during the entire flood event.

When the erodible bed material is stratified, more resistant layers cover more easily erodible material and special protective measures may be taken to prevent scour of the resistant layer. Armoring reduces the rate of degradation and stabilizes the river system. It is essential to monitor and manage sand and gravel mining so as not to induce undesirable instabilities in the river system. In most cases, removal of sand and gravel has caused deepening and widening of the channel. These wider and deeper reaches act as sinks for the sediment loads and may trap the finer clays, altering the environment.

Increased discharge has the tendency to transport bed material and to scour bends, constrictions and obstructions. In streams carrying bed load, scour reaches equilibrium since the sediment transport capacity is supplied from upstream.

3.7.3 Sediment Transport in Culverts

In previous sections, the basic principles and methodologies of sediment transport were presented. This discussion leads to a unique problem encountered in dealing with sediment transport, i.e., the analysis of culvert flow. The following section will briefly illustrate the problems associated with culvert flow and will provide a method of estimating the water and sediment discharge rates in enclosed culverts.

The analysis of culvert flow is necessary to determine if the culvert is large enough to convey both water and sediment without causing a hazardous backwater upstream, or filling the culvert with sediment. The discharge in culverts can be calculated from the method detailed in Section 2.9.

Development of a sediment transport equation for an enclosed culvert is difficult because of complicated flow conditions with inlet and outlet control. The equation developed by Graf and Acaroglu (1968) can be used since it was developed from a data base that included a large range of pipe flow and open channel conditions.

$$Qs_{max} = 3.78\sqrt{g}\, D_s^{-1.02} S_f^{2.52} R^{1.52} A \qquad (3.7.1)$$

where Qs_{max} is the maximum sediment discharge rate, in volume per time, that can be conveyed by the conduit. Qs_{max} is in ft^3/s when D_s and R are in feet, A in ft^2 and g in ft^2/s. The hydraulic radius R is A/P; when flowing full, A is $\pi D^2/4$, and P is πD. If Qs_{max} is less than the incoming sediment discharge rate from the ditch, the culvert begins to fill with sediment, restricting further water flow. An example problem showing the use of this relationship is given in Appendix 3, Problem 3.12.

Past experience has shown significant problems including erosion at the inlet and outlet, sediment buildup in the barrel and clogging of the barrel with debris.

As culvert barrels constrict the natural channel, vortices and areas of high velocity impinge against the upstream slopes and scour away the embankment of the culvert. Upstream slope paving, channel paving, headwalls, wingwalls, and cutoff walls help to protect the slopes and channel bed at the upstream end of the culvert.

Scour at culvert outlets is a common occurrence. The characteristics of the channel bed and bank material, velocity and flow depth in the channel at the culvert outlet, and the amount of sediment and other debris in the flow are all contributing factors to scour potential. Due to the variation in expected flows and the difficulty in evaluating some of the factors, scour prediction remains subjective. Protection against scour at culvert outlets varies from limited riprap placement to complex energy dissipation devices.

The companion problem to erosion is sedimentation. Most streams carry a sediment load and tend to deposit this load when their velocities decrease. A stable channel is expected to balance erosion and sedimentation over time.

The most common culvert installations which encounter sedimentation problems are multi-barrel installations and culverts built with depressions at the entrance. Culverts which are located on and aligned with the natural channel generally do not have sedimentation problems.

Debris is defined as any material moved by a flowing stream. Debris can accumulate at a culvert inlet or become lodged in the inlet or barrel. When this happens, the culvert fails to perform as designed. Upstream flooding and roadway overtopping create hazards to upstream and downstream properties and traffic.

Three debris control methods are available for culvert sites with more serious risks: interception at or above the culvert inlet protecting culvert performance; deflection of debris away from the entrance to a holding area for eventual removal; and passage of the debris through the culverts. Debris interceptors include debris racks, floating drift booms and debris basins, and are located upstream of the culvert entrance. Debris deflectors vary from an inclined steel bar or rail placed in front of the inlet to V-shaped debris deflectors. The passage of debris can be accomplished by oversizing the culvert or utilizing a bridge as a replacement structure. Design information for commonly employed debris control structures are found in HEC No. 9, "Debris Control Structures" (FHWA Publication).

Go to Chapter 4

Chapter 4 : HIRE
River Morphology and River Response

Go to Chapter 5 (Part I)

4.1 Introduction

Rivers and river systems have served man in many ways. Rivers are passage ways for navigation and are essential to agriculture, particularly in the arid and semiarid parts of the world. To a large degree the flooding by rivers and the deposition of sediment therefrom on the river valleys have been a means of revitalizing the river valleys to keep them productive. Rivers have provided a means of traveling inland and developing trade. This has played a significant role in the development of all countries wherever rivers of significant size exist.

Rivers have different alignments and geometry. There are meandering rivers, braided rivers, and rivers that are essentially straight. In general, braided rivers are relatively steep and meandering rivers have more gentle slopes. Meandering channels have characteristics that enable us to utilize them without experiencing extensive improvement and maintenance costs.

Meandering rivers are not subject to rapid movement, are reasonably predictable in behavior and are utilizable to man's benefit. Nevertheless, they are generally unstable with eroding banks which may results in destruction of productive land, bridges, bridge approaches, control works, buildings, and urban properties during floods. Bank protection works are often necessary to stabilize certain reaches of the river and to improve them for other aspects of flood control and navigation.

4.2 Fluvial Cycles and Processes

Fundamental characteristics and processes governing the formation of river systems are discussed in this section. Rivers can be classified according to their age. The morphology of flood plains, delta, and alluvial fans are described as well as the processes of headcutting and nickpoint navigation. The concept of geomorphic threshold completes this section on fundamentals leading to the classification of streams to be presented later in this chapter.

4.2.1 Youthful, Mature and Old Streams

Various methods are used to classify rivers according to their age. One of the methods used by geomorphologists, and widely accepted by the engineering profession, classifies streams as youthful, mature and old. Youthful implies the initial state of streams. As channels are first developed on the earth's surface by the flowing water, they are generally V-shaped, very irregular and consist of fractured erosive and nonerosive materials. Examples of youthful streams are mountain streams with steep slopes and their tributaries developed by overland flow.

There is no distinct line between youthful and mature rivers. In the case of mature channels, the river valleys have widened, the river slopes are flat, and bank cutting has largely replaced downward cutting. The streambed has achieved a graded condition; that is, the slope and the energy of the stream are just sufficient to transport the material delivered to it. With mature channels, narrow floodplains and meanders have formed. The valley bottoms are sufficiently wide to accomodate agricultural and urban developments, and where development has occurred, usually channel stabilization works and other improvements have been made to prevent lateral migration of the river.

River channels classified as old are extensions in age of the mature channel. As erosion continues, the river valleys develop characteristics of greater width and low relief; the stream gradient has flattened further, and meanders and meander belts that have developed are not as wide as the river valley. Natural levees have formed along the stream banks. Landward of the natural levees, there are swamps. The tributaries to the main channel parallel the main channel sometimes for long distances before there is a breach in the natural levee that permits a confluence. In conjunction with an old river and its river valley, wide areas are available for cultivation, improvements of all types are built, and flood levees are generally required to protect those occupying the valley. Because of the more sophisticated development of the river valley, channel stabilization and contraction work such as revetments and dikes are generally constructed.

Rejuvenation refers to an increase in erosional activities in mature or old channels caused by lowering tailwater elevation, tectonic activities or other causes. Rejuvenated mature or old channels then exhibit some properties of youthful channels such as channel incision and erosion processes.

It should be emphasized that the preceding concept of the fluvial cycle is not accepted by all geologists. For example, some consider a channel to be mature only after the trunk stream as well as the side streams have achieved a graded condition. Some define old age as a condition when the entire river system is graded. This concept can only be applied as an average condition extending over a period of years. No stream is continuously graded. A poised stream refers to one that neither aggrades nor degrades its channel over time. Both graded and poised streams are delicately balanced. Any change imposed on the river system will alter the balance and lead to actions by the stream to reestablish balance. For example, a graded or poised stream may be subjected locally to the development of a cutoff. The development of the cutoff increases the channel slope, increases velocity, and increases transport at least locally. Changes in these variables cause changes in the channel and deposition downstream. The locally steepened slope gradually extends itself upstream attempting to reestablish equilibrium.

4.2.2 Floodplain and Delta Formations

Over time, the highlands of an area are worn down. The streams erode their banks. The material that is eroded is utilized further downstream to build banks and bars to further enhance the meandering process. Streams move laterally pushing the highlands back. Low flat valley land and floodplains are formed. As the streams transport sediment to areas of flatter slopes and in particular to bodies of water where the velocity and turbulence are too small to sustain the transport of the material, the material is deposited forming deltas. As deltas build outward the up-river portion of the channel is elevated through deposition and becomes part of the floodplain. Also, the stream channel is lengthened and the slope is further reduced. The upstream river bed is filled in and average flood elevations are increased. As it works across the river valley this type of development causes the total floodplain to raise in elevation. Hence, even old streams are far from static. Old rivers meander, are affected by changes in sea level, are influenced by movements of the earth's crust, are changed by delta formations or glaciation, and are subject to modifications due to climatological changes and as a consequence of man's development.

4.2.3 Alluvial Fans

Alluvial fans are one of the few natural causes of aggradation problems at transverse highway crossings. They occur whenever there is a change from a steep to a flat gradient. As the bed material and water reaches the flatter section of the stream, the coarser bed materials can no longer be transported because of the sudden reduction in both slope and velocity. Consequently, a cone or fan builds out as the material is dropped. The steep side of the fan faces the floodplain. There is considerable similarity between a delta and an alluvial fan. Both result from reductions in slope and velocity. Both have steep slopes at their outer edges. Both tend to reduce upstream slopes. Alluvial fans, like deltas, are characterized by unstable channel geometries and rapid lateral movement. An action very similar to the delta develops where a steep tributary enters a main channel. The steep channel tends to drop part of its sediment load in the main channel building out into the main stream. In some instances, the main stream can be forced to make drastic changes at the time of major floods by the stream's tributaries.

Fans can be of two types, dry or mudflow fans formed by ephemeral streamflow, and wet or fluvial fans formed by perennial stream flow. Two different conditions of fan morphology are observed on modern dry fans. The first situation occurs when deposition is near the mountain front and the fan surface is undissected. The second situation occurs when sediment material is moved through a fan-head trench and deposition occurs at the toe of the fan. Good relationships exist between fan area and drainage basin area (Schumm, 1977). These relationships among fan slope, area, and drainage basin characteristics are not surprising. The presence or absence of fan-head trenches, however, is rather interesting. These differences are sometimes attributed to tectonic activity or climate change. The longitudinal profile of fans is concave. Two types of concavity are recognized. The first is due to intermittent uplift of the mountains which gradually steepens the fan head. The other case is due to trenching and the building out of a low flatter reach of recent alluvium at the toe of the fan.

Fluvial (wet) fans can become very large, which contrast with dry fans. The almost random distribution of erosion and deposition patterns on the arid fan is replaced by a progressively shifting channel. Lateral migration of streams on fluvial fans can be anticipated by the concavity of the contours and avulsion. New orientation of a river channel is also an equally possible shifting process.

Normally the coarsest material is found at the fan apex, although fan-head trenching might result in a slight increase in sediment size with the fan radius. Experimental studies show that growth at the fan-head is intermittent, being interrupted by periods of incision, sediment reworking and downfan distribution of sediment. The greatest variation in sediment yield is related to fan-head trenching and aggradation. Geomorphic thresholds controlling fan growth are sketched in Figure 4.2.1

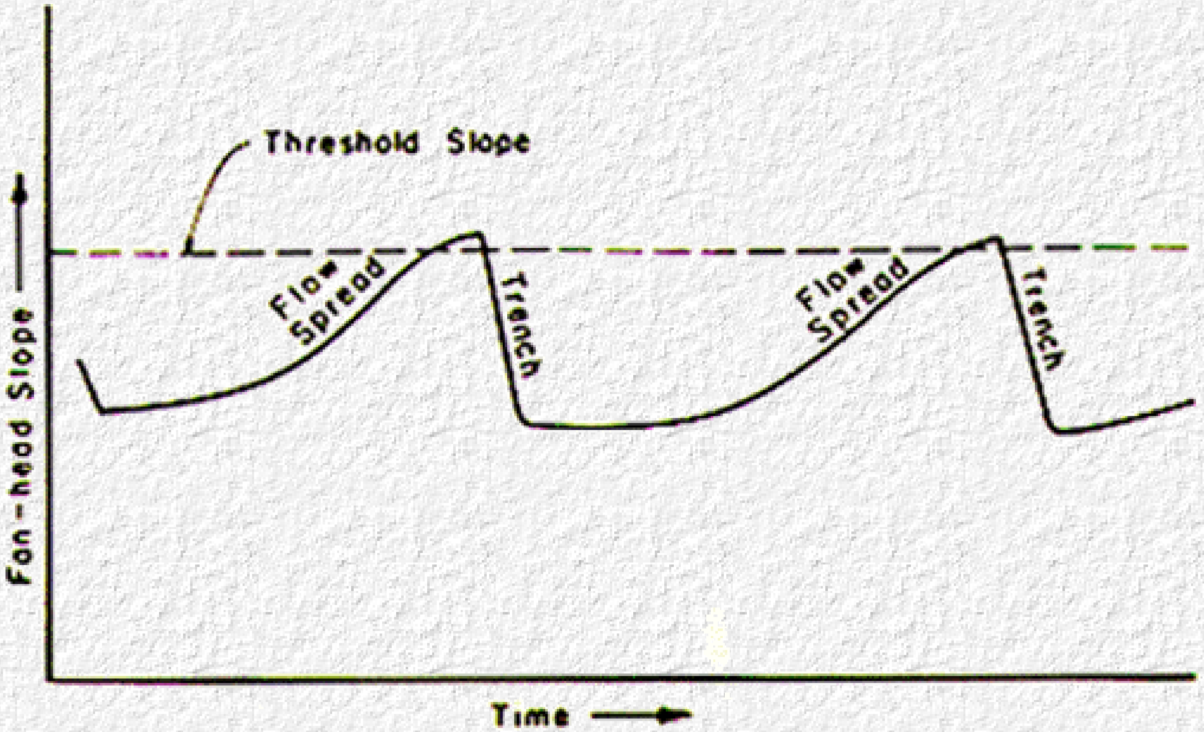
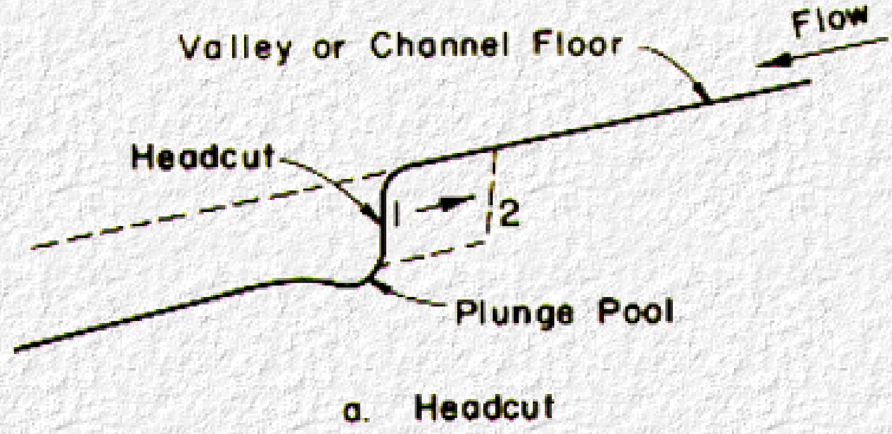


Figure 4.2.1. Changing Slope at Fan-Head Leading to Fan-Head Trenching (Ref. Schumm, 1977)

4.2.4 Nickpoint Migration and Headcutting

Abrupt changes in the longitudinal profile of the stream are shown in Figure 4.2.2. This break in the profile induces a perturbation moving upstream, especially during floods. Above the profile break the river is stable; below the break there is erosion. As the perturbation migrates past a point, a dramatic change in channel morphology and stability occurs. These perturbations are of two types: the first is a sharp break in profile which forms an in-channel scarp called a headcut (Figure 4.2.2a), and the second, called a nickpoint, has a gradual change in elevation over a greater length of channel (Figure 4.2.2b).



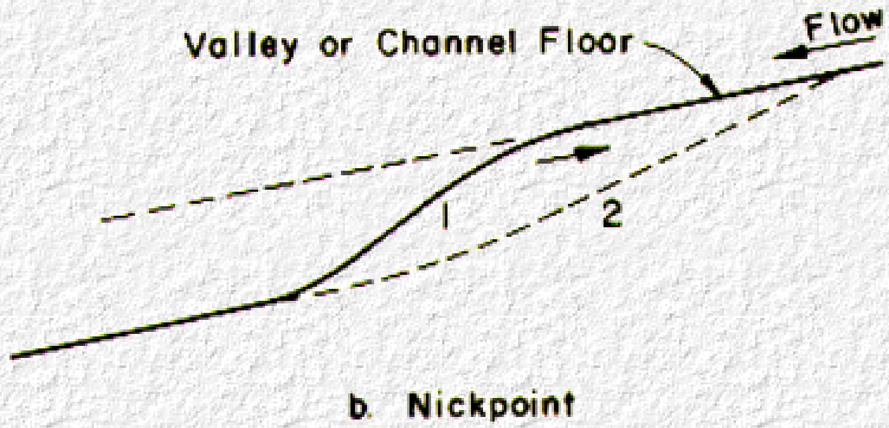


Figure 4.2.2. Headcuts and Nickpoints

The result of nickpoint formation and migration is, of course, lowering of the stream bed. Erosion of the bed material will be dramatic as a headcut or nickpoint migrates under a bridge. During a major flood in Tujunga Wash, California, erosion above the headwall of a gravel pit led to the failure of three highway bridges. In fine alluvium, headcut migration normally lowers the channel abruptly to its new position. The second type of nickpoint (Figure 4.2.2b), however, produces degradation that persists for some distance. The steeper reach type of a nickpoint generally occurs in coarse size material. In both cases scour continues until the gradient has been reduced and bank erosion has widened the channel to the point that deposition can begin. As the nickpoint migrates farther upstream, the quantity of sediment delivered to the reach at which a stream crossing is located increases greatly due to the erosion of the bed upstream and subsequent erosion of the banks of the stream. Therefore, a period of degradation may be followed at a site by a period of aggradation.

The most obvious and simplest way to identify nickpoints is by the use of aerial photographs. Particularly in arid and semi-arid regions, headcuts are very easily recognizable because the upstream valley floor or channel is essentially undisturbed, whereas the channel below the nickpoint shows significant erosion. On topographic maps of large scale the presence of a nickpoint is indicated by closely-spaced contours. Of course, the presence of a nickpoint is verified with field surveys which show the break in the longitudinal profile of the stream. A change in the dimension of the channel and a change in the character of the bank line may indicate the presence of nickpoint migration. A low width-depth ratio below the nickpoint is an indication of scour and deepening of the channel. Bank erosion is also the possible consequence of nickpoint migration and a sharp change in the bankline characteristics representing a change from stability to instability may identify the presence of a nickpoint.

4.2.5 Geomorphic Threshold

Evolution of drainage network and sediment production from drainage basins are very complex features. Geomorphic history and climatic changes introduce a new degree of complexity into the response of watersheds and fluvial systems. Experimental studies demonstrate that within a complex natural system, one event can trigger a complex reaction as the components of the system respond to drainage. The magnitude of this complex response is likely to appear during early stages of an erosion cycle or during rejuvenation of high sediment producing areas. Nevertheless, these areas produce major land management and conservation problems, hence their practical interest.

It is possible to explain variations in sediment yield and channel adjustment by using the concept of geomorphic thresholds described by Schumm (1977). For example, in a given drainage area, it is possible to define a valley slope above which the valley floor is unstable. Stable valley floors above the threshold line are incipiently unstable and any major flood may eventually cause erosion and trenching of the alluvium stored in these valleys (Figure 4.2.1 illustrates the concept of geomorphic threshold). Since permanent changes result only when a geomorphic threshold has been exceeded, events with high magnitude and low frequency may at times have only minor and local effect on the landscape.

4.3 Stream Form

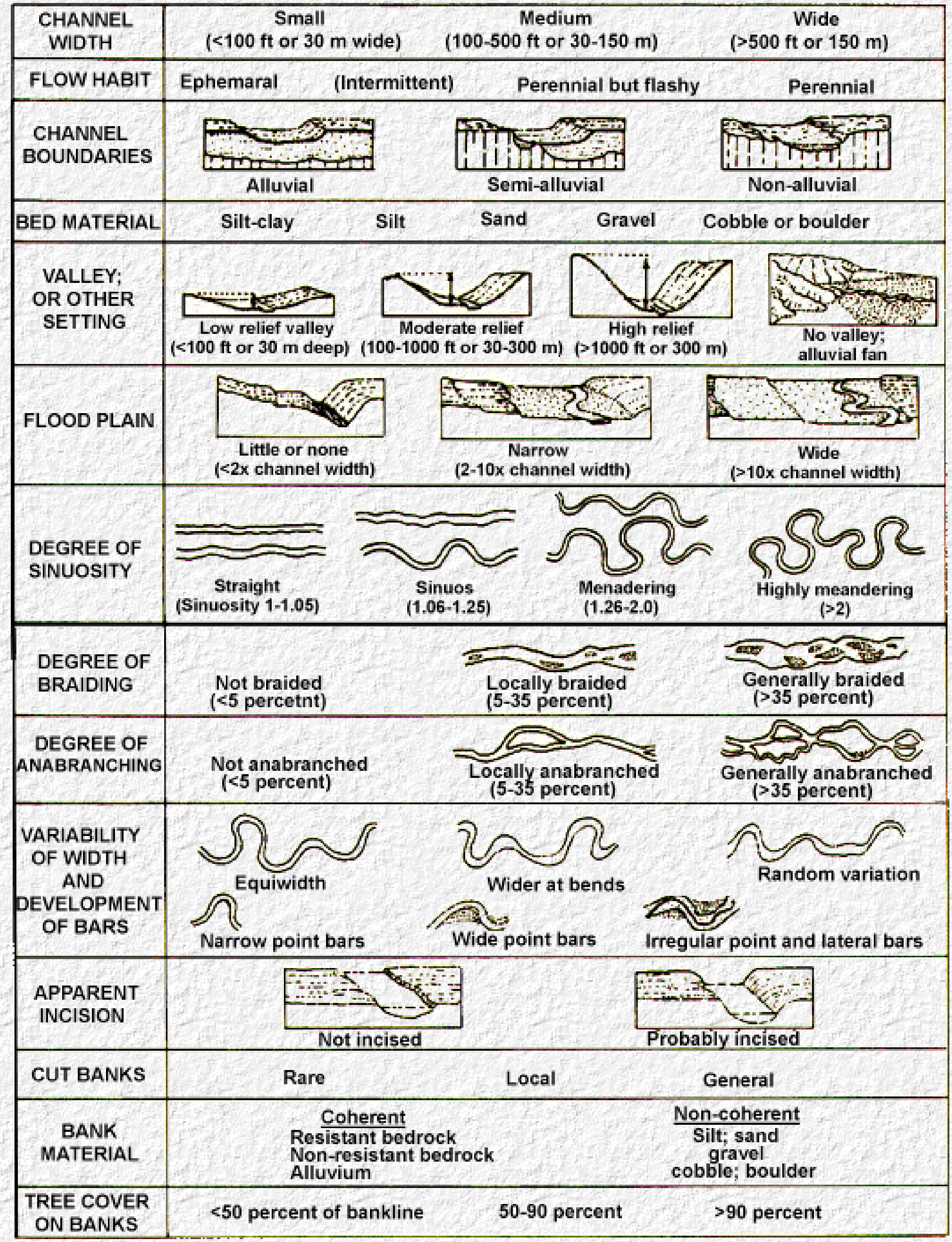
A study of the plan and profile of a stream is very useful in understanding stream morphology. Planview appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a river, adversely affecting a highway crossing or encroachment. Conversely, the highway crossing or encroachment can inadvertently change the plan view or profile, adversely affecting the river environment. In this section the stream form is classified and the channel processes are discussed.

4.3.1 Classification of River Channels

Brice and Blodgett (1978) developed a detailed classification scheme oriented primarily toward lateral stability of rivers. The common geomorphic terms for the various types of streams (meandering, braided, etc.) are shown in Figure 4.3.1. Each term is well defined on the small sketches.

Additional information on the major types of meandering, straight and braided channels is illustrated in Figure 4.3.2. Classification based on oxbow lakes is illustrated in Figure 4.3.2a. In Figure 4.3.2b, types of meander scroll formations are illustrated. By studying scroll formations in terms of age of vegetation the rate and direction of channel migration can be quantified. The sinuosity index is the ratio of the length of the watercourse over the valley length between the same points. Classification based on natural levees is illustrated in Figure 4.3.2c. Well developed levees are associated with older rivers. The floodplain that is broad in relation to the channel width is indicative of an older river. Conversely when the river valley is narrow and confined by terraces or valley walls, the river flowing therein is usually mature. In general, the growth of vegetation is indicative of the presence of silts and clays in the river banks and the floodplain. This is particularly true if the floodplain is well drained. With good drainage the silt and clay are essential to the growth of vegetation because of their water holding capability.

Natural levees are a characteristic of old river systems. Levees form during floods as the river stage exceeds bankfull conditions. Sediment is then deposited on the flood plain due to the reduced velocity and transporting capacity. The natural levees near the river are rather steep because coarse material drops out quickly. Farther from the river the gradients are flatter and the finer materials drop out. Beyond the levees are the swamp areas. On the lower Mississippi River, natural levees on the order of ten feet in height are common. The rate of growth of natural levees is smaller after they reach a height equal to the average annual flood stage.



CHANNEL WIDTH	Small (<100 ft or 30 m wide)	Medium (100-500 ft or 30-150 m)	Wide (>500 ft or 150 m)	
FLOW HABIT	Ephemaral	(Intermittent)	Perennial but flashy	Perennial
CHANNEL BOUNDARIES	Alluvial	Semi-alluvial	Non-alluvial	
BED MATERIAL	Silt-clay	Silt	Sand	Gravel; Cobble or boulder
VALLEY; OR OTHER SETTING	Low relief valley (<100 ft or 30 m deep)	Moderate relief (100-1000 ft or 30-300 m)	High relief (>1000 ft or 300 m)	No valley; alluvial fan
FLOOD PLAIN	Little or none (<2x channel width)	Narrow (2-10x channel width)	Wide (>10x channel width)	
DEGREE OF SINUOSITY	Straight (Sinuosity 1-1.05)	Sinuos (1.06-1.25)	Menadering (1.26-2.0)	Highly meandering (>2)
DEGREE OF BRAIDING	Not braided (<5 percetnt)	Locally braided (5-35 percent)	Generally braided (>35 percent)	
DEGREE OF ANABRANCHING	Not anabranched (<5 percent)	Locally anabranched (5-35 percent)	Generally anabranched (>35 percent)	
VARIABILITY OF WIDTH AND DEVELOPMENT OF BARS	Equiwidth; Narrow point bars	Wider at bends; Wide point bars	Random variation; Irregular point and lateral bars	
APPARENT INCISION	Not incised	Probably incised		
CUT BANKS	Rare	Local	General	
BANK MATERIAL	Coherent: Resistant bedrock, Non-resistant bedrock, Alluvium	Non-coherent: Silt; sand, gravel, cobble; boulder		
TREE COVER ON BANKS	<50 percent of bankline	50-90 percent	>90 percent	

Figure 4.3.1. Stream Properties for Classification (after Brice & Blodgett, 1978)

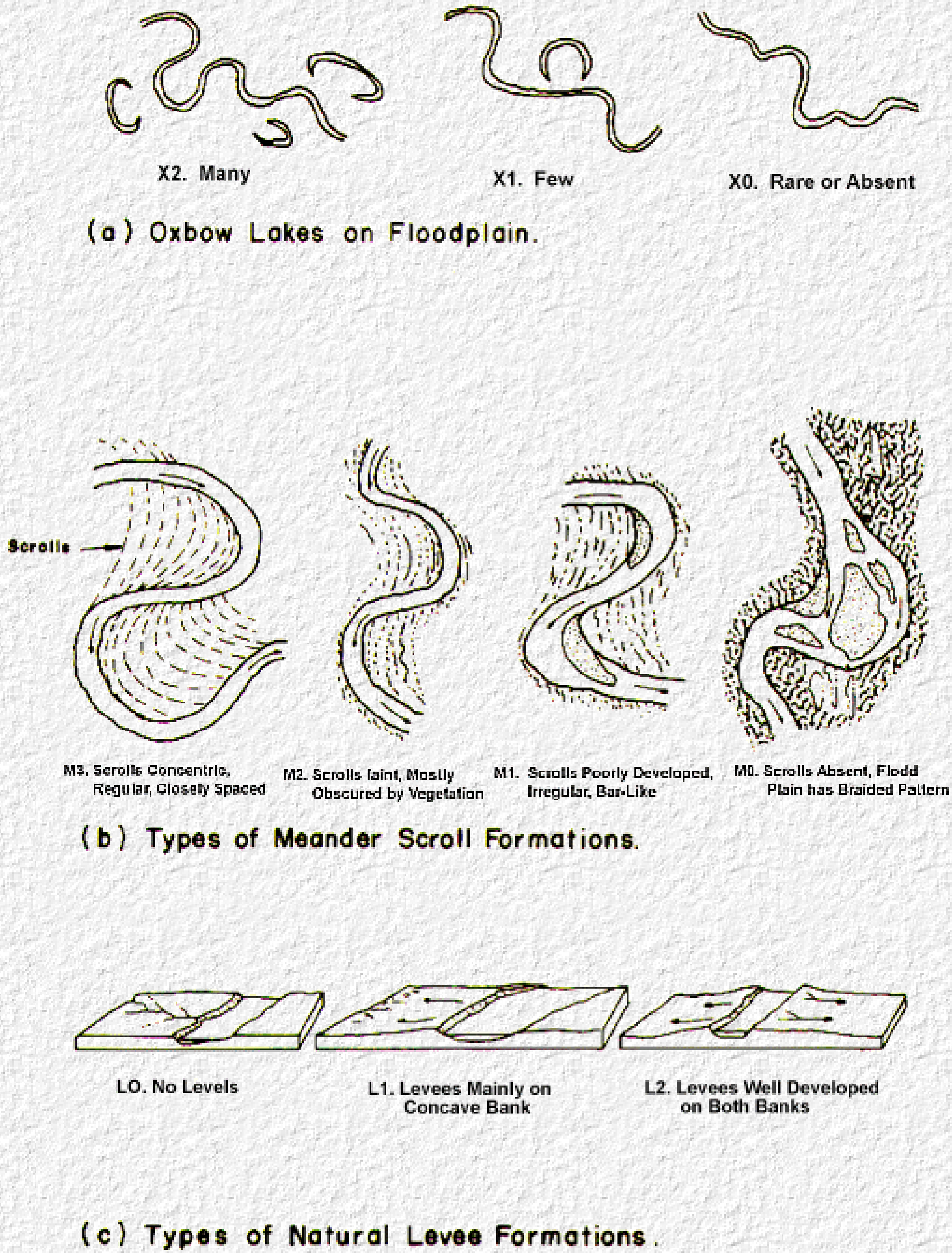


Figure 4.3.2. Classification of River Channels (after Culbertson et al., 1967)

A detailed knowledge of the hydraulic characteristics of different types of streams is of great value when dealing with the location of highway crossings and encroachment, training works, flood control works and other river structures. A channel classification in Figure 4.3.3 shows the relative stability and types of hazards encountered. An example is given in Appendix 4, Problem 4.1 on how to use this classification.

A brief discussion will now be presented concerning the nature and stability of straight, braided, and meandering channels. Each behaves in a slightly different way when subject to man-related or natural impacts. A knowledge of this behavior is important in anticipating and understanding stability problems.

4.3.2 The Straight Channel

The straight channel has small sinuosity at bankfull stage. At low stage the channel develops alternate sandbars and the thalweg meanders around the sandbars in a sinuous fashion. Straight channels are considered as a transitional stage to meandering. If the alluvial channel is unconfined, more than one channel develops, creating middle bars as well as point bars, and the river is braided.

4.3.3 The Meandering Stream

Alluvial channels of all types deviate from a straight alignment. The thalweg oscillates transversely and initiates the formation of bends. In general, the river engineer concerned with channel stabilization should not attempt to develop straight channels fully protected with riprap. In a straight channel the alternate bars and the thalweg (the shallows and deeps) are continually changing; thus, the current is not uniformly distributed through the cross-section but is deflected toward one bank and then the other. Sloughing of the banks, nonuniform deposition of bed load caused by debris such as trees, and the Coriolis force due to the earth's rotation have been cited as causes for meandering of streams. When the current is directed toward a bank, the bank is eroded in the area of impingement and the current is deflected and impinges upon the opposite bank further downstream. The angle of deflection of the thalweg is affected by the curvature formed in the eroding bank and the lateral depth of erosion.

The meandering river consists of pools and crossings. The thalweg, or main current of the channel, flows from the pool through the crossing to the next pool forming the typical S-curve. In the pools, the channel cross-section is somewhat triangular. Point bars form on the inside of the bends. In the crossings, the channel

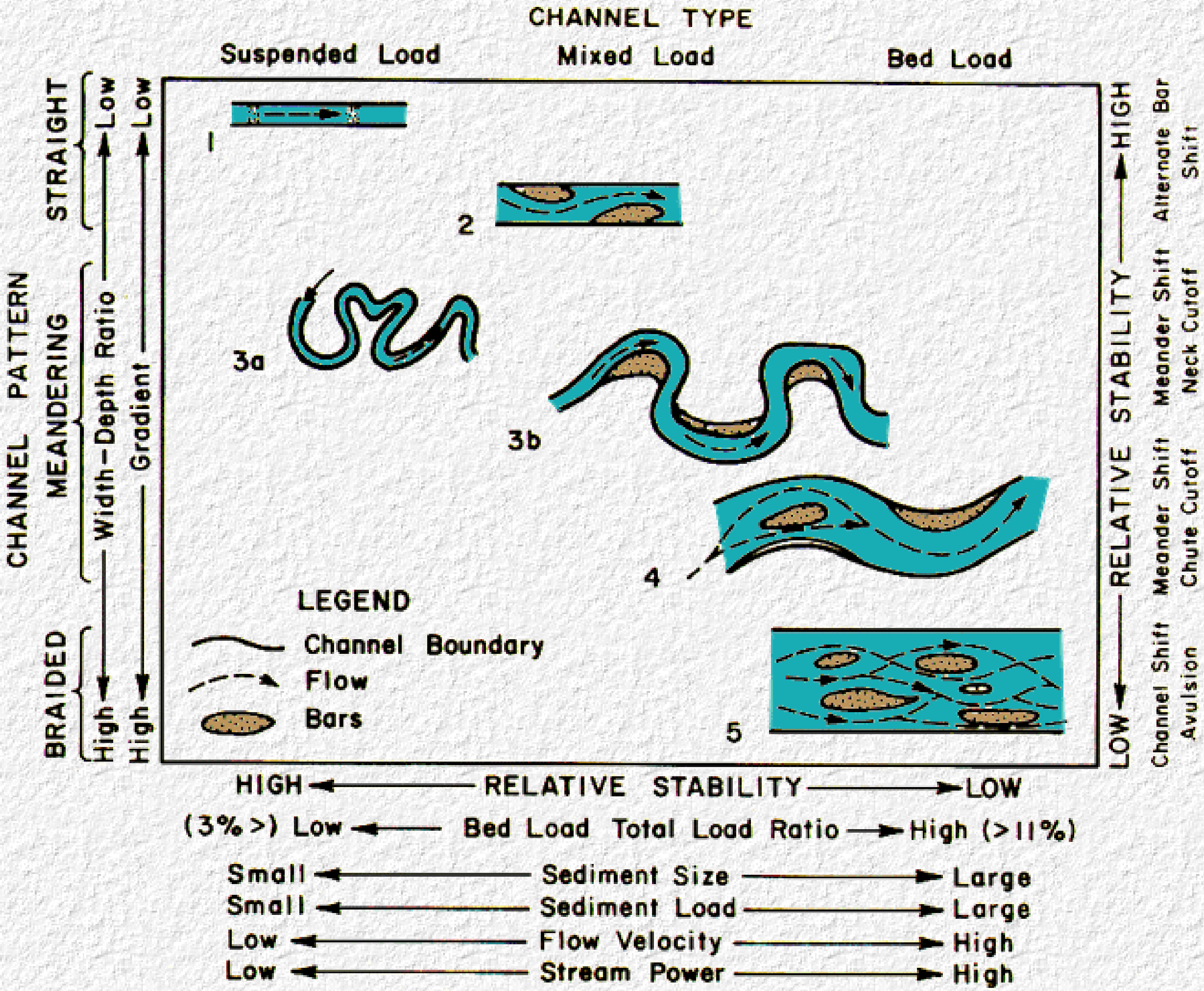


Figure 4.3.3. Channel Classification Showing Relative Stability and Types of Hazards Encountered with Each Pattern (after Shen, et al., 1981)

cross-section is more rectangular and depths are smaller. At low flows the local slope is steeper and velocities are larger in the crossing than in the pool. At low stages the thalweg is located very close to the outside of the bend. At higher stages, the thalweg tends to straighten. More specifically, the thalweg moves away from the outside of the bend encroaching on the point bar to some degree. In the extreme case, the shifting of the current causes chute channels to develop across the point bar at high stages. In Figure 4.3.4, one can observe the position of the thalweg, the location of the point bars, alternate bars and the location of the pools and crossings. Note that in the crossing the channel is shallow compared to pools and the banks may be more subject to erosion.

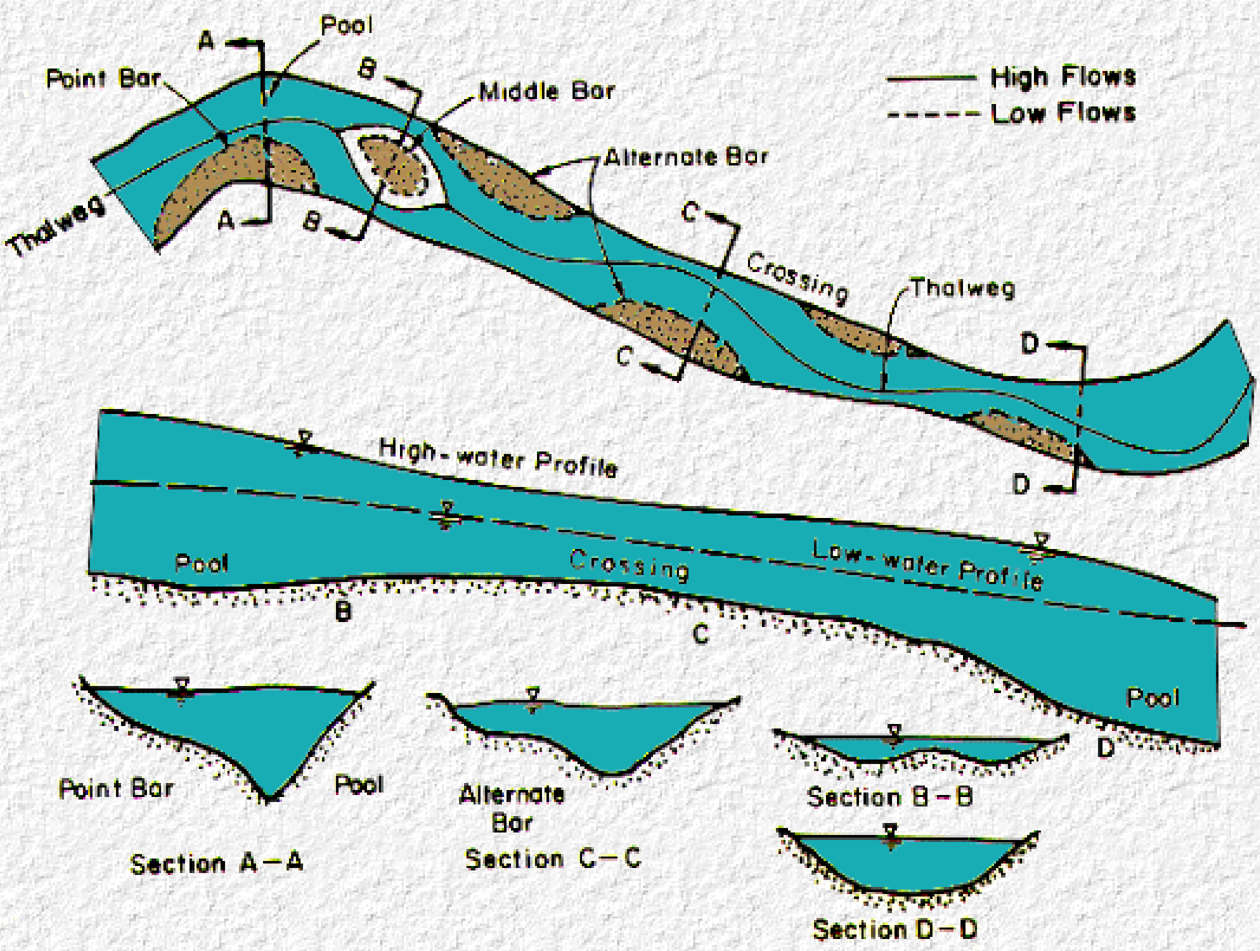


Figure 4.3.4. Plan View and Cross-Section of a Meandering Stream

In general, bends are formed by the process of erosion and deposition. Erosion without deposition to assist in bend formation would result only in escalloped banks. Under these conditions the channel would simply widen until it becomes so large that the erosion would terminate. The material eroded from the bank is normally deposited over a period of time on the point bars that are formed downstream. The point bars constrict the bend and enable erosion in the bend to continue, accounting for the lateral and longitudinal migration of the meandering stream. Erosion is greatest across the channel from the point bar. As the point bars build out from the downstream sides of the points, the bends gradually migrate down the valley. The point bars formed in the bendways clearly define the direction of flow. The bar is generally streamlined and its largest portion is oriented downstream. If there is very rapid caving in the bendways upstream, the sediment load may be sufficiently large to cause middle bars to form in the crossing.

As a meandering river system moves laterally and longitudinally, the meander loops move at an unequal rate because of the unequal erodibility of the banks. This causes the channel to appear as a bulb form slowly developing. The channel geometry depends upon the local slope, the bank material, and the geometry of the adjacent bends. Over time the local steep slope caused by the cutoff is distributed both upstream and downstream. Years may be required before a configuration characteristic of average conditions in the river is attained.

When a cutoff occurs, an oxbow lake is formed (See Figure 4.3.2a). Oxbow lakes may persist for long periods of time before filling. Usually the upstream end of the lake fills quickly to bank height. Overflow during floods carries fine materials into the oxbow lake area. The lower end of the oxbow remains open and the drainage and overland flow entering the system can flow out from the lower end. The oxbow gradually fills with fine silts and clays. Fine material that ultimately fills the bendway is plastic and cohesive. As the river channel meanders it encounters old bendways filled with cohesive materials (referred to as clay plugs). These plugs are sufficiently resistant to erosion to serve as essentially semipermanent geologic controls. Clay plugs can drastically affect river geometry.

The variability of bank materials, and the fact that the river encounters such features as clay plugs, cause a wide variety of river forms even with a meandering river. The meander belt formed by a meandering river is often fifteen to twenty times the channel width.

A meandering river has more or less regular inflections that are sinuous in plan. It consists of a series of bends connected by crossings. In the bends, deep pools are carved adjacent to the concave bank by the relatively high velocities. Because velocities are lower on the inside of the bend, sediments are deposited in this region, forming the point bar. The centrifugal force in the bend causes a transverse water surface slope, and in many cases, helicoidal flow occurs in the bend. Point bar building is enhanced when large transverse velocities occur. In so doing, they sweep the heavier concentrations of bed load toward the convex bank where they are deposited to form the point bar. Some transverse currents have a magnitude of about 15 percent of the average channel velocity. The bends are connected by crossings (short straight reaches) which are quite shallow compared to the pools in the bendways. At low flow, large sandbars form in the crossings if the channel is not well confined. The scour in the bend causes the bend to migrate downstream and sometimes laterally. Lateral movements as large as 2500 feet per year have been observed in alluvial rivers. Much of the sediment eroded from the outside bank is deposited in the crossing and on the point bar in the next bend downstream. Meandering rivers have relatively flat slopes.

The geometry of meandering rivers is quantitatively measured in terms of: (1) meander wavelength λ; (2) meander width W_m; (3) mean radius of curvature r_c; (4) meander amplitude A; and (5) bend deflection angle ϕ. A sketch defining these variables is shown in Figure 4.3.5.

The actual meanders in natural rivers are generally not as regular as indicated in Figure 4.3.5. The precise measurement of meander dimensions is therefore difficult in natural channels and tends to be subjective. An example on how to measure these characteristics is presented in Appendix 4, Problem 4.1. The analysis of the median meander dimension in nature shows that the meander length and meander width are both related to the width of the channels. The empirical relationships for the meander length λ and the bank-full channel width as well as the meander amplitude, A, and the bank-full channel width are shown in Figure 4.3.6 and Table 4.3.1.

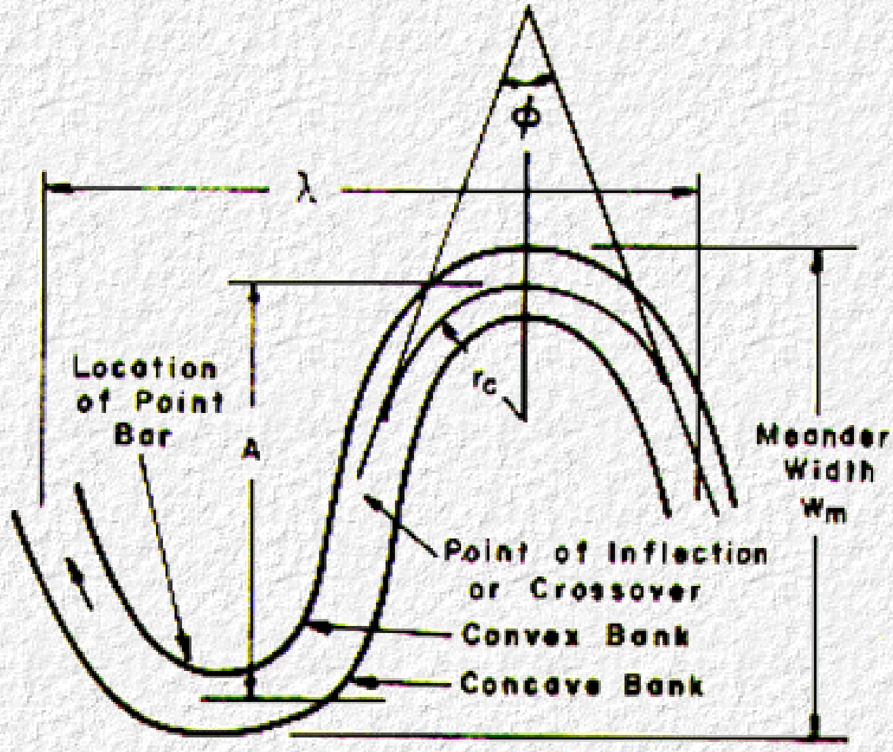


Figure 4.3.5. Definition Sketch for Meanders

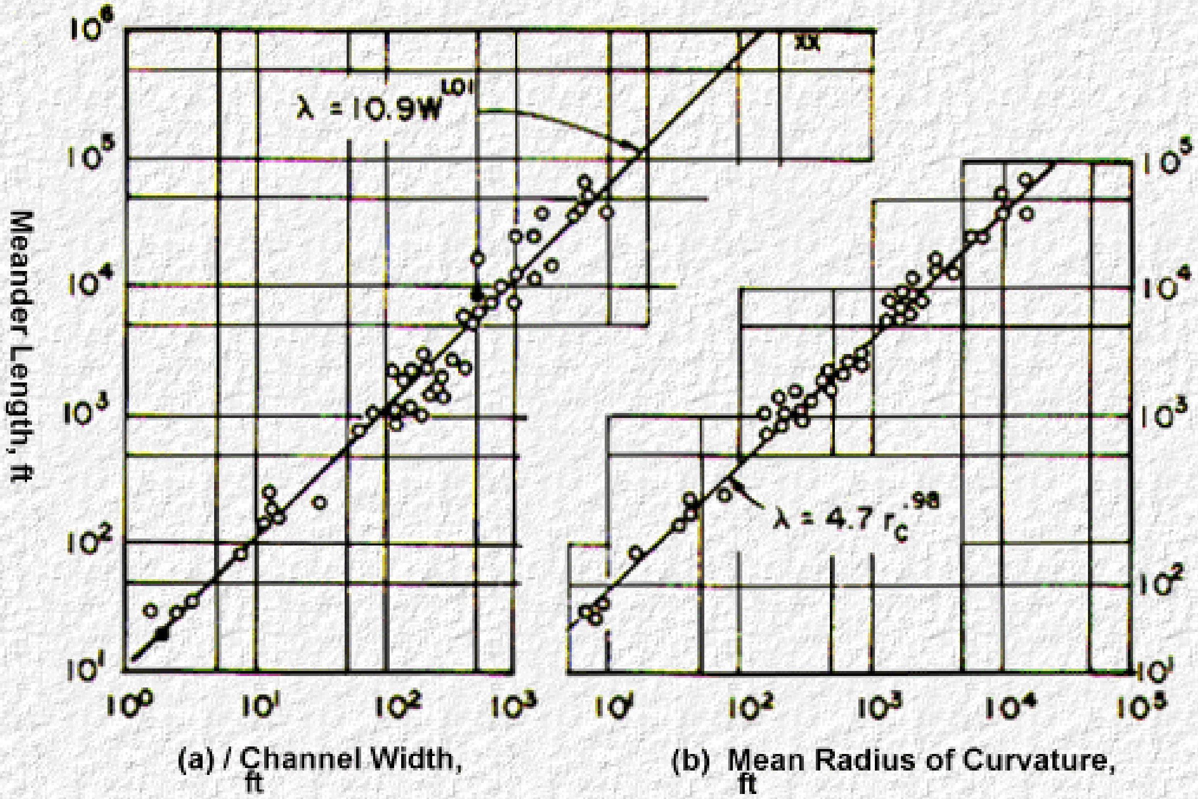


Figure 4.3.6. Empirical Relations for Meander Characteristics (Leopold et al., 1964)

Table 4.3.1 Empirical Relations for Meanders in Alluvial Valleys

Meander Length to Channel Width	Amplitude to Channel Width	Meander Length to Radius of Curvature	Source

$\lambda = 6.6W^{0.99}$ - $\lambda = 10.9W^{1.01}$	$A = 18.6W^{0.99}$ $A = 10.9W^{1.04}$ $A = 2.7W^{1.10}$	- - $\lambda = 4.7r_c^{0.98}$	Inglis (1949) Inglis (1949) Leopold & Wolman (1960)

4.3.4 The Braided Stream

A braided stream is one that consists of multiple and interlacing channels (see Figure 4.3.3). One cause of braiding is the large quantity of bed load that the stream is unable to transport. The magnitude of the bed load is more important than its size. Many geologists claim that braiding is independent of the size of the bed material at least in the sand range. If the channel is overloaded with sediment, deposition occurs, the bed aggrades, and the slope of the channel increases in an effort to obtain a graded state. As the channel steepens, the velocity increases, and multiple channels develop. These interlaced multiple channels cause the overall channel system to widen. Multiple channels are generally formed as bars of sediment are deposited within the main channel.

Another cause of braiding is easily eroded banks. If the banks are easily eroded, the stream widens at high flow and forms bars at low flows which become stabilized, thus forming islands. In general, a braided channel has a large slope, a large bed-material load in comparison with its suspended load, and relatively small amounts of silts and clays in the bed and banks. Figure 4.3.7 will assist in defining the various conditions for multiple channel streams.

The braided stream may present difficulties for highway construction because it is unstable, changes its alignment rapidly, carries large quantities of sediment, is very wide and shallow even at flood flow and is, in general, unpredictable. An example of classification is given in Appendix 4, Problem 4.2.

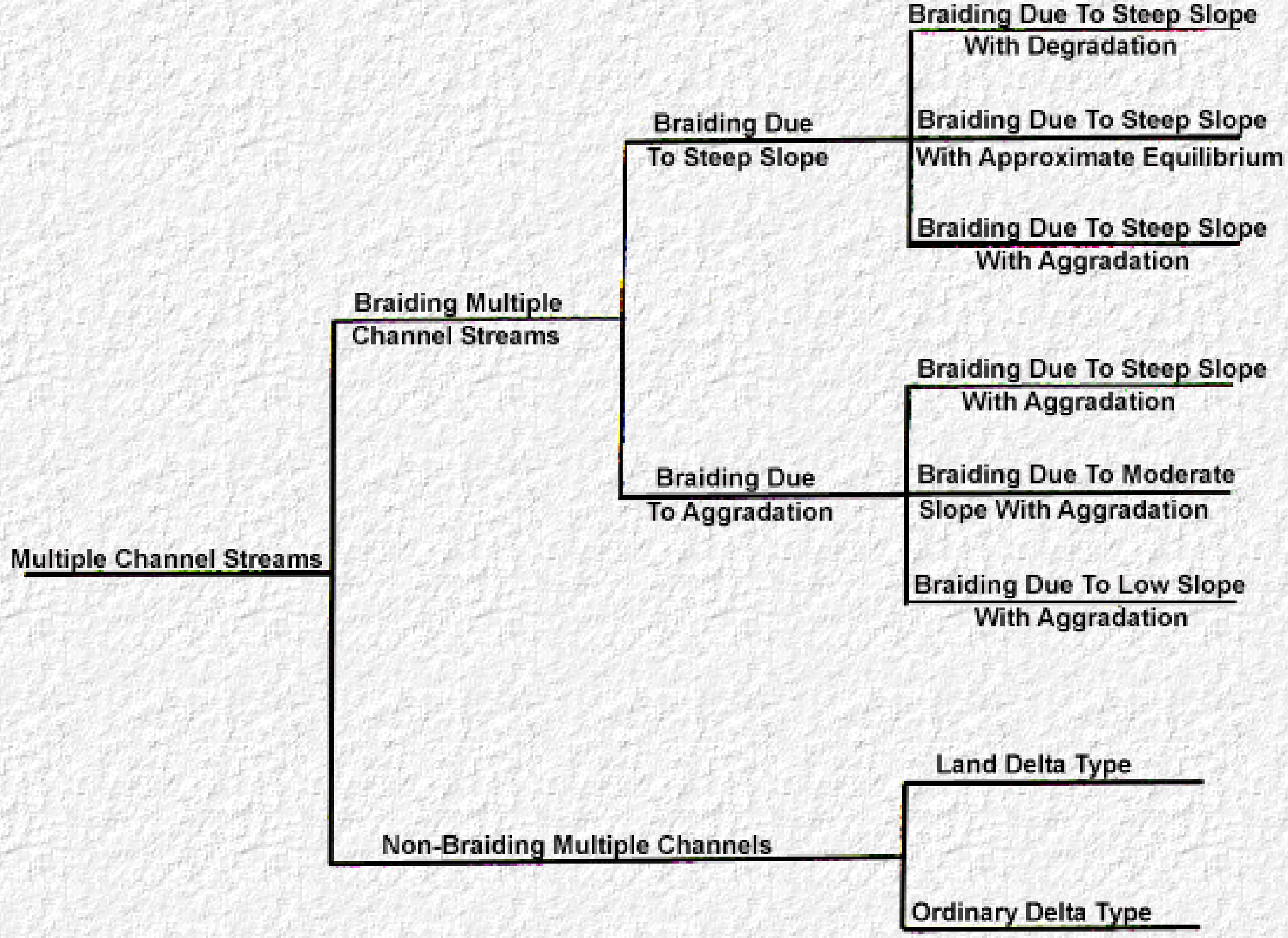


Figure 4.3.7. Types of Multi-Channel Streams

4.4 Geometry of Alluvial Channels

Hydraulic geometry is a general term applied to alluvial channels to denote relationships between discharge Q and the channel morphology, hydraulics and sediment transport. Channels forming in their own sediments are called alluvial channels. In alluvial channels, the morphologic, hydraulic and sedimentation characteristics of the channel are determined by a large variety of factors. The mechanics of such factors are not fully understood. However, alluvial streams do exhibit some quantitative hydraulic geometry relations. In general, these relations apply to channels within a physiographic region and can be easily obtained from data available on gaged rivers. It is understood that hydraulic geometry relations express the integral effect on all the hydrologic, meteorologic, and geologic variables in a drainage basin.

4.4.1 Hydraulic Geometry of Alluvial Channels

The hydraulic geometry relations of alluvial streams are useful in river engineering. The forerunner of these relations are the regime theory equations of stable alluvial canals. Hydraulic geometry relations were developed by Leopold and Maddock (1953) for different regions in the United States and for different types of rivers. In general the hydraulic geometry relations are stated as power functions of the discharge:

$$W = \hat{a}Q^{\hat{b}}$$

$$y_o = \hat{c}Q^{\hat{f}}$$

$$V = \hat{k}Q^{\hat{m}}$$

$$Q_T = \hat{p}Q^{\hat{j}}$$

$$S_f = \hat{t}Q^{\hat{z}}$$

$$n = \hat{r}Q^{\hat{y}}$$

where W is the channel width, y_o is the channel depth, V is the average velocity of flow, Q_T is the total bed sediment load, S_f is the friction slope, n is the Manning's roughness coefficient, and Q is the discharge as defined in the following paragraphs. The coefficients $\hat{a}, \hat{c}, \hat{k}, \hat{p}, \hat{t}, \hat{r}$ and exponents $\hat{b}, \hat{f}, \hat{m}, \hat{j}, \hat{z}, \hat{y}$ in these equations are determined from analysis of available data on one or more streams. From the continuity equation Q = Wy_oV, it is seen that

$$\hat{a} \bullet \hat{c} \bullet \hat{k} = 1$$

and

$$\hat{b} + \hat{f} + \hat{m} = 1$$

Leopold and Maddock (1953) have shown that in a drainage basin, two types of hydraulic geometry relations can be defined: (1) relating W, y_o, V and Q_s to the variation of discharge at-a-station; and (2) relating these variables to the discharges of a given frequency of occurrence at various stations in a drainage basin. Because Q_T is not available, they used Q_s, the suspended sediment transport rate. The former are called at-a-station relationships and the latter downstream relationships. The distinction between at-a-station and downstream hydraulic geometry relations is illustrated in Figure 4.4.1 and Figure 4.4.2.

The mean values of exponents $\hat{b}, \hat{f}, \hat{m}, \hat{j}, \hat{z}$, and $\hat{y}$ as reported by Leopold et al.(1964) are given in Table 4.4.1. These values are based on an extensive analysis of stream data in the United States.

Hydraulic geometry relationships were theoretically derived at Colorado State University. These relations are almost identical to those proposed by Leopold and Maddock. These at-a-station relationships are:

$$W \sim Q^{0.26} \quad (4.4.1)$$

$$y_o \sim Q^{0.40} \quad (4.4.2)$$

$$S_f \sim Q^{0.00} \quad (4.4.3)$$

$$V \sim Q^{0.34} \quad (4.4.4)$$

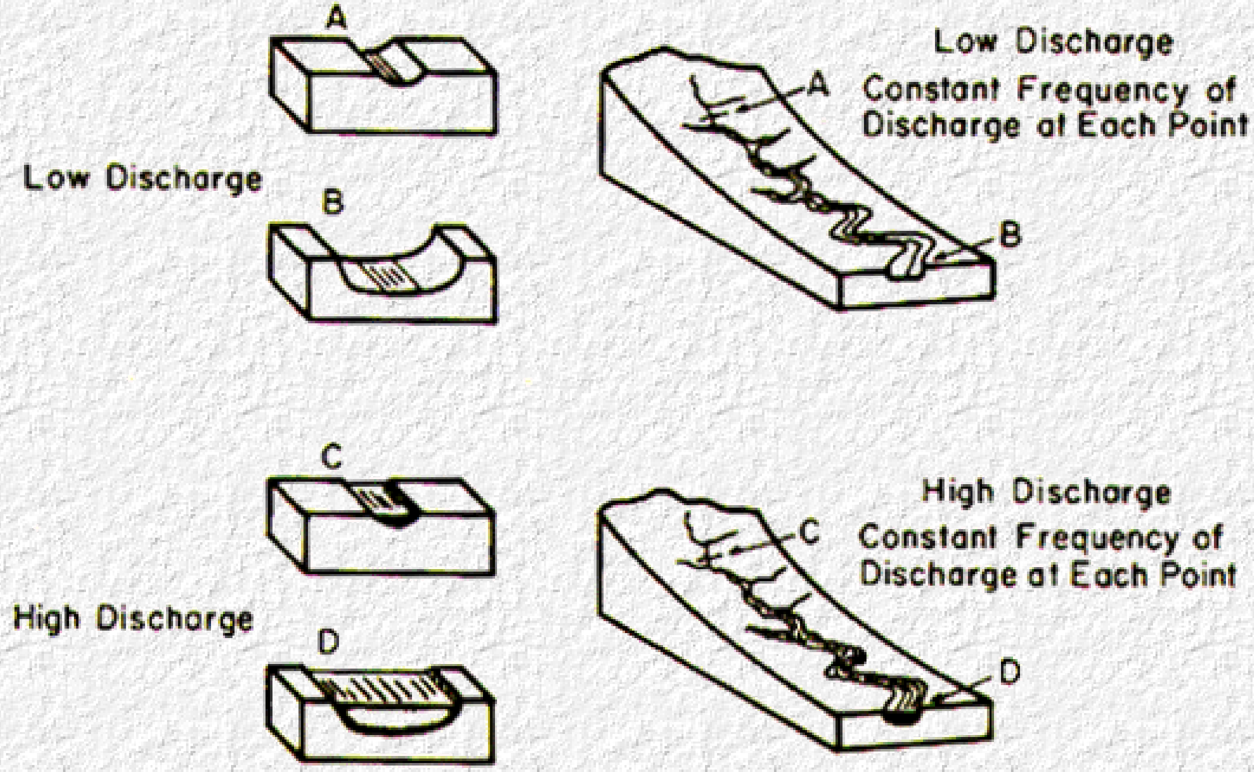


Figure 4.4.1. Variation of Discharge at a Given River Cross Section and at Points Downstream (after Leopold and Maddock, 1953). At-A-Station Relations Pertain to Individual Sites such as A or B. Downstream Relations Pertain to a Channel (Segment A-B) or Drainage Network for Discharge of a Given Frequency of Occurrence.

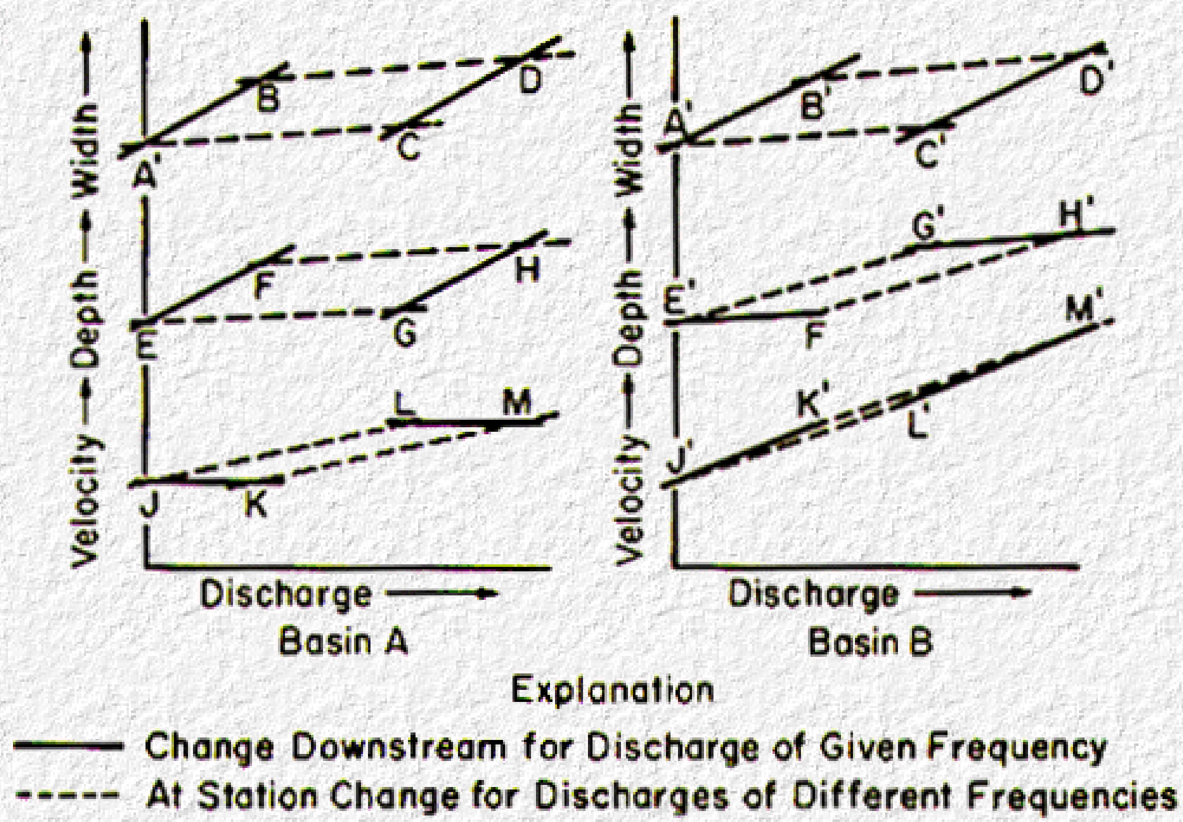


Figure 4.4.2. Schematic Variation of Width, Depth, and Velocity with At-A-Station and Downstream Discharge Variation (after Leopold and Maddock, 1953)

Table 4.4.1 At-A-Station and Downstream Hydraulic Geometry Relationships

Average At-A-Station Relations						
	$\hat{b}$	$\hat{f}$	$\hat{m}$	$\hat{j}$	$\hat{z}$	$\hat{y}$
Average values Midwestern United States	.26	.40	.34	2.5		
Brandywine Creek, Pennsylvania	.04	.41	.55	2.2	.05	-.2
Ephemeral Streams in Semiarid United States	.29	.36	.34			
Average of 158 Gaging Stations in United States	.12	.45	.43			
Ten Gaging Stations on Rhine River	.13	.41	.43			
Average Downstream Relations (Bank-Full or Mean Annual Flow)						
	$\hat{b}$	$\hat{f}$	$\hat{m}$	$\hat{j}$	$\hat{z}$	$\hat{y}$
Average values Midwestern United States	.5	.4	.1	.8	-.49	
Brandywine Creek, Pennsylvania	.42	.45	.05		-1.07	-.28
Ephemeral Streams in Semiarid United States	.5	.3	.2	1.3	-.95	-.3
Appalachian Streams	.55	.36	.09			

Equation 4.4.3 implies that the energy slope is constant at a cross-section. This is not quite true. At low flow the effective channel slope is that of the thalweg that flows from pool through crossing to pool. At higher stages the thalweg straightens somewhat, shortening the path of travel and increasing the local friction slope. In the extreme case, river slope approaches the valley slope at flood stage. It is during high floods that the flow often cuts across the point bars, developing chute channels. This path of travel verifies the shorter path the water takes and that a steeper channel prevails under this condition.

The derived downstream relations for bank-full discharge are:

$$y_o \sim Q_b^{0.46} \qquad (4.4.5)$$

$$W \sim Q_b^{0.46} \qquad (4.4.6)$$

$$S_f \sim Q_b^{-0.46} \qquad (4.4.7)$$

$$V \sim Q_b^{0.08} \qquad (4.4.8)$$

Here the subscript b indicates the bank-full condition.

When the Manning equation is applicable, the theoretical downstream geometry relationships of non-cohesive alluvial streams are a function of discharge Q and sediment size d_s:

$$y_o \sim Q_b^{0.4} \qquad (4.4.9)$$

$$W \sim Q_b^{0.53} D_s^{-0.33} \qquad (4.4.10)$$

$$S_f \sim Q_b^{-0.4} D_s^{1.00} \qquad (4.4.11)$$

$$V \sim Q_b^{0.07} D_S^{0.33} \qquad (4.4.12)$$

These relationships (Bray, 1982; Julien and Simons, 1984) indicate that sediment size is an important variable besides water discharge. Sediment size becomes particularly important when considering the slope relationship. Equation 4.4.9 to Equation 4.4.12 are best used for non-cohesive gravel-bed rivers. An example showing how to use Equation 4.4.1 to Equation 4.4.12 is presented in Appendix 4, Problem 4.5.

4.4.2 Dominant Discharge in Alluvial Rivers

The hydraulic geometry relations discussed in Section 4.4.1 indicate how the channel morphology and other characteristics vary with discharge at-a-station or in the downstream direction in a drainage network. In the hydraulic design of river crossings and encroachments, the relations need to be defined to determine the downstream hydraulic geometry of the channel at a site between two gaged sites. The question then arises about the frequency of discharge to be used in the hydraulic geometry relations. The downstream hydraulic geometry relations expressed in Section 4.4.1 relate to the bank-full stage, which for many U.S. rivers has a frequency of occurrence of one in 1.5 years. In the past, various terms such as dominant or formative discharge have been vaguely used for selecting some arbitrary discharge for the purpose of developing downstream hydraulic geometry relations. This arbitrariness is confusing to a designer.

The characteristics of an alluvial channel, including its hydraulic geometry, vary with the discharge. In natural rivers, the characteristics such as the bed sediment load, energy gradient and meander geometry can be related to channel discharge as simple power functions. The formative discharge corresponding to this average value of such characteristics can then be defined in terms of the power function and the frequency distribution of the flow as follows:

$$F(Q) = \hat{b} Q^{\hat{n}} \qquad (4.4.13)$$

$$\overline{F(Q)} = \frac{1}{T} \int_0^T F(Q)\, dt \qquad (4.4.14)$$

$$Q_f = \left[\frac{1}{\hat{b}} \overline{F(Q)} \right]^{\frac{1}{\hat{n}}} \qquad (4.4.15)$$

where F(Q) is the power function relating the phenomenon of interest, for example the bed sediment load, to discharge, Q; T is the time period over which the occurrence of the phenomenon is averaged and Q_f is the formative discharge for the particular phenomenon. As the functions F(Q) are different for different characteristics, the value of Q_f obtained from the preceding equations will also be different. Also, when $\hat{n}$ > 1, Q_f will be greater than the mean discharge $\overline{Q}$. For a given site, Q_f can be expressed in terms of the frequency of occurrence of a return period.

Analyses of bed sediment load estimations of Q_f on most rivers may show that up to 90 percent of the total transport is caused by flows that are equaled or exceeded about ten percent of the time only. Thus, the average bed sediment load in a river may be described in terms of a formative discharge much larger than the mean annual flow. Also, the average channel width, depth and meander geometry may be defined in terms of different formative discharges rather than an arbitrarily chosen dominant discharge.

The concept of frequency occurrence of flows is important in the hydraulic design of highway crossings and encroachments. Both the at-a-station and downstream hydraulic geometry relations are especially useful when the hydraulic design is based on the frequency of occurrence of flows. The concept of bank-full condition corresponding to a discharge with a period of return of 1.5 years for perennial streams is recommended for practical use when detailed analysis of formative discharge is not possible, or feasible.

4.4.3 The River Profile and Its Bed Material

The slope of a river channel or a river system is steepest in the headwater regions. The river profile is concave upward; the slope of the river profile can be represented by the equation

$$S_x = s_0 e^{-\hat{\alpha} x} \qquad (4.4.16)$$

where

S_x = the slope at any station a distance x downstream of the reference station,

S_o = the slope at the reference station, and

$\hat{\alpha}$ = a coefficient

Similarly, the bed sediment size is coarser in the upper reaches where the channel slopes are steep and the bed sediment size becomes finer with distance downstream. Generally, the size of the bed material reduces with distance according to the relationship

$$D_{50_x} = D_{50_o} e^{-\hat{\beta} x} \qquad (4.4.17)$$

where

D_{50_x} = the size of bed material at distance x downstream of the reference station

D_{50_0} = the size of bed material at the reference station

$\hat{\beta}$ = a coefficient

The hydraulic geometry relations are applicable to continuous channel behavior. The use of Equation 4.4.17 is exemplified with field data in Appendix 4, Problem 4.6. In some cases, this behavior may become discontinuous as the channel pattern changes from meandering to braided by the formation of cutoffs.

4.4.4 River Conditions for Meandering and Braiding

In the preceding examples it was shown that changes in water discharge, sediment discharge or both can cause significant changes in channel slope. The changes in sediment discharge can be in quantity Q_s or sediment size D_{50} or both. Often such changes can alter the plan view in addition to the profile of a river. According to Lane (1957), Figure 4.4.3 illustrates the dependence of sand bed river form on channel slope and discharge. It shows that when

$$S\,Q^{1/4} \leq .0017 \qquad (4.4.18)$$

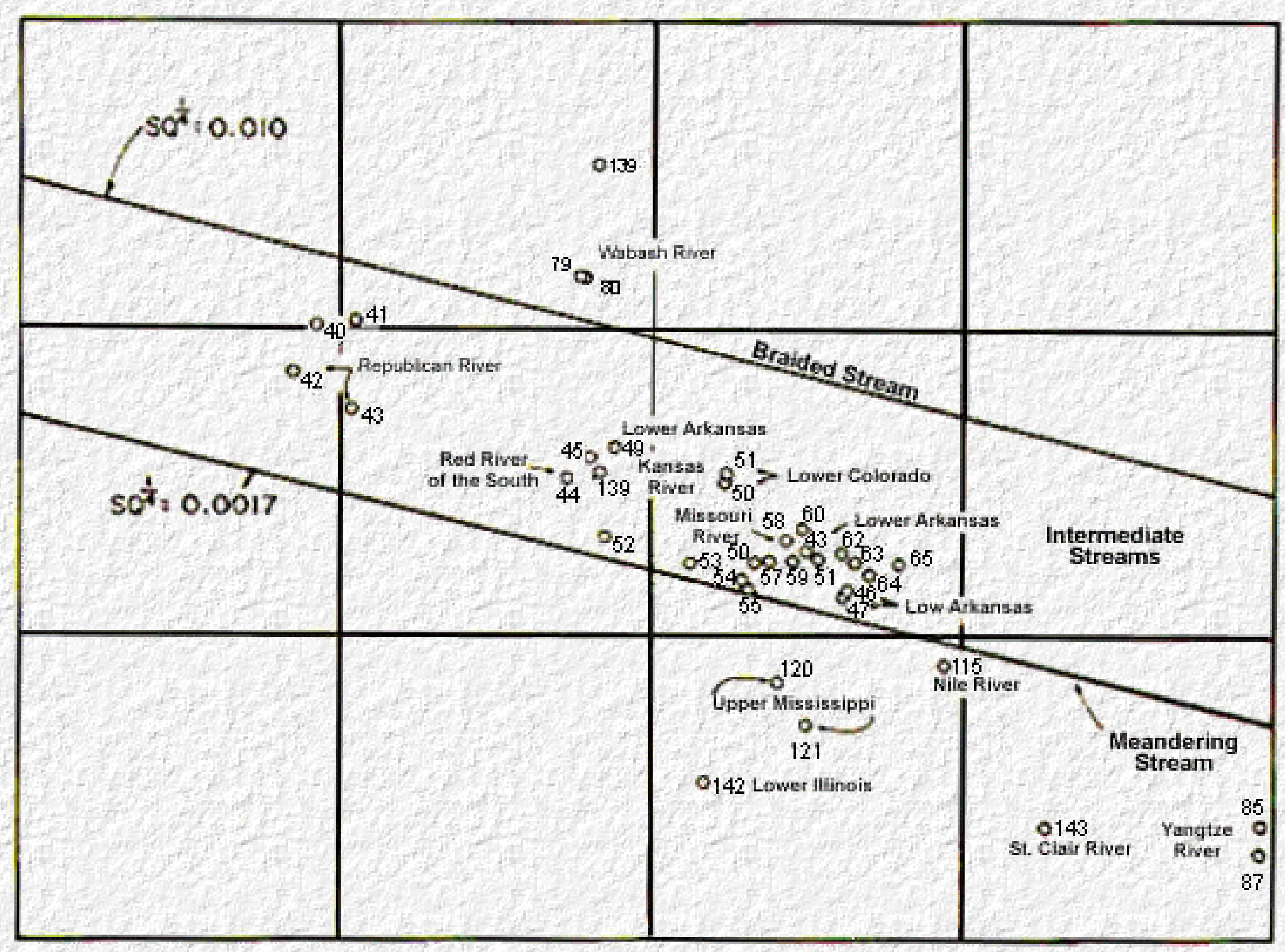


Figure 4.4.3. Slope-Discharge Relation for Braiding or Meandering in Sandbed Streams (after Lane, 1957)

a sandbed channel meanders. Similarly, when

$$S\ Q^{1/4} \geq .010 \qquad (4.4.19)$$

the sandbed river is braided. In these equations, S is the channel slope in feet per foot and Q is the mean discharge in cfs. Between these values of $S\ Q^{1/4}$ is the transitional range and many of the U.S. rivers, classified as intermediate sandbed streams, plot in this zone between the limiting curves defining meandering and braided rivers. If a river is meandering but its discharge and slope borders on the transitional zone a relatively small increase in channel slope may cause it to change, with time, to a transitional or braided river.

Leopold and Wolman (1960) plotted slope and discharge for a variety of natural streams. They observed that a line could separate meandering from braided streams. The equation of this line is

$$S = 0.06\ Q^{-0.44} \qquad (4.4.20)$$

where S is the slope in ft/ft; and Q is the bank-full discharge is cfs.

Streams classified as meandering by Leopold and Wolman are those whose sinuosity is greater than 1.5. Braided streams are those which have relatively stable alluvial islands and, therefore, two or more channels. The authors note that sediment size is related to slope and channel pattern but they do not try to account for the effect of sediment size on the morphology of the streams. They further note that braided and meandering streams can be differentiated based on combinations of slope, discharge, and width/depth ratio, but regard the width as a variable dependent on mainly discharge.

The authors recognize that their analysis treats only two of the many variables affecting morphology, therefore do not expect this method to apply in every condition. However, because the data were all taken from natural streams, and because the analysis obviously does indicate a significant relation between slope and discharge, the analysis should give a reasonably effective prediction of channel pattern if slope and discharge are known. Problem 4.2 in Appendix 4 gives an example of this type of prediction.

4.5 Qualitative Response of River Systems

Many rivers have achieved a state of practical equilibrium throughout long reaches. For practical engineering purposes, these stable reaches can be also called "graded" streams by geologists and "poised" streams by engineers. However, this does not preclude significant changes over a short period of time or over a period of years. Conversely, many streams contain long reaches that are actively aggrading or degrading. These aggrading and degrading channels certainly pose a definite hazard to any highway crossing or encroachment, as compared to poised streams.

Regardless of the degree of channel stability, man's local activities may produce major changes in river characteristics locally and throughout the entire reach. All too frequently the net result of a river improvement is a greater departure from equilibrium than that which originally prevailed. Good engineering design must invariably seek to enhance the natural tendency of the stream toward stable conditions. To do so, an understanding of the direction and magnitude of change in channel characteristics caused by the actions of man and nature is required. This understanding can be obtained by: (1) studying the river in a natural condition; (2) having knowledge of the sediment and water discharge; (3) being able to predict the effects and magnitude of man's future activities; and (4) applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers.

To predict the response to channel modifications is a very complex task. There are large numbers of variables involved in the analysis that are interrelated and can respond to changes in a river system in the continual evolution of river form. The channel geometry, bars, and forms of bed roughness all change with changing water and sediment discharges. Because such a prediction is necessary, useful methods have been developed to predict the response of channel systems to changes both qualitatively and quantitatively.

4.5.1 General River Response to Change

Quantitative prediction of response can be made if all of the required data are known with sufficient accuracy. Usually, however, the data are not sufficient for quantitative estimates, and only qualitative estimates are possible. Examples of studies that have been undertaken by various investigators for qualitative estimates follow. Lane (1955) studied the changes in river morphology caused by modifications of water and sediment discharges. Similar but more comprehensive treatments of channel response to changing conditions in rivers have been presented by Leopold and Maddock (1953), Schumm (1971), and Santos-Cayado (1972). All research results support the following general statements:

1. Depth of flow is directly proportional to water discharge and inversely proportional to sediment discharge;
2. Width of channel is directly proportional to water discharge and to sediment discharge;
3. Shape of channel expressed as width-depth ratio is directly related to sediment discharge;
4. Meander wavelength is directly proportional to water discharge and to sediment discharge;
5. Slope of stream channel is inversely proportional to water discharge and directly proportional to sediment discharge and grain size; and
6. Sinuosity of stream channel is proportional to valley slope and inversely proportional to sediment discharge.

It is important to remember that these statements pertain to natural rivers and not necessarily to artificial channels with bank materials that are not representative of sediment load. In any event, the relations will help to determine the response of water conveying channels to change.

Bed material sediment transport (Q_s) can be directly related to stream power ($\tau_o V$) and inversely related to the fall diameter of bed material (D_{50}).

$$Q_s \sim \frac{\tau_o V\, W\, C_f}{D_{50}} \qquad (4.5.1)$$

Here τ_o is the bed shear, V is the cross-sectional average velocity, W is the width of the stream and C_f is the volumetric concentration of fine sediments. Equation 4.5.1 can be written as

$$Q_s \sim \frac{\gamma\, y_o W\, V\, C_f}{D_{50}} = \frac{\gamma QS\, C_f}{D_{50}} \qquad (4.5.2)$$

If the specific weight, γ, is considered constant and if the concentration of wash load C_f can be incorporated in the fall diameter, D_{50}, then the relationship can be expressed as

$$QS \sim Q_s\, D_{50} \qquad (4.5.3)$$

which is the relationship originally proposed by Lane (1955), except Lane used the median diameter of the bed material as defined by sieving instead of the fall diameter. The fall diameter includes the effect of temperature on the transportability of the bed material and is preferable to the use of physical diameter.

Equation 4.5.3 is very useful to qualitatively predict channel response to climatological changes, river modifications, or both. Two simple example problems are analyzed using Equation 4.5.3.

In a first example, consider a tributary entering the main river at point C that is relatively small but carries a large sediment load (see Figure 4.5.1). This increases the sediment discharge in the main stream from Q_s to Q^+_s. It is seen from Equation 4.5.3 that, for a significant increase in the sediment discharge (Q^+_s) the channel gradient (S) below C must increase if Q remains constant. The line CA (indicating the original channel gradient) therefore changes with time to position C'A. Upstream of the confluence the slope will adjust over a long period of time to the original channel slope. The river bed will aggrade from C to C'. This change may induce a change in the channel morphology downstream of point C as the downstream reach will tend toward braiding. This possible change in planform geometry must be considered if any structure is to be built between points A and C'.

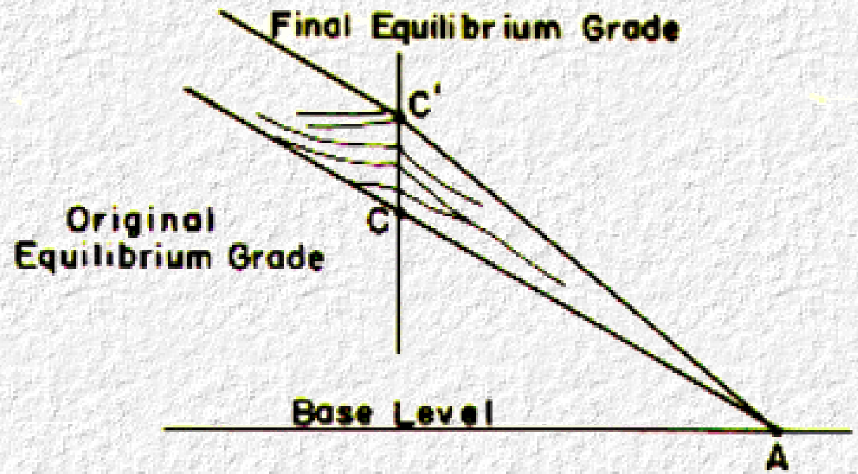


Figure 4.5.1. Changes in Channel Slope in Response to an Increase in Sediment Load at Point C

In a second example, the construction of a dam on a river usually causes a decrease in sediment discharge downstream. Referring to Figure 4.5.2, and using Equation 4.5.3 and the earlier discussion, it can be concluded that for a decrease in bed material discharge from Q_s to Q^+_s, the slope S decreases downstream of the dam. In Figure 4.5.2, the line CA, representing the original channel gradient, changes to C'A, indicating a decrease in bed elevation and slope in the downstream channel with time. Note, however, if the dam fills with sediment so that the incoming sediment discharge passes through, that, except for local scour at the dam, the grade line C'A would return to the line CA. Also upstream of the dam the grade would return to the original equilibrium grade but would be offset vertically by the height of the dam.

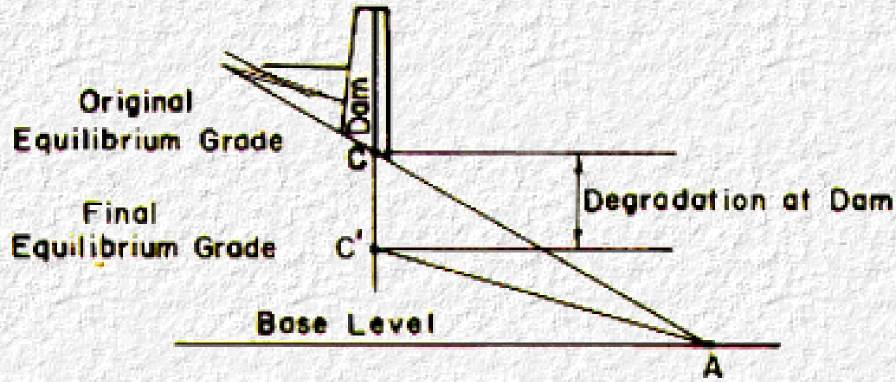


Figure 4.5.2. Changes in Channel Slope in Response to a Dam at Point C

Thus small reservoirs (storage capacity small in relation to annual discharge) may cause degradation below the dam and then aggradation over a relatively short period of time.

The engineer is also interested in quantities in addition to directions of variations. The geomorphic relation $QS \sim Q_s D_{50}$ is only an initial step in analyzing long-term channel response problems. However, this initial step is useful because it warns of possible future difficulties related to channel modifications and flood protection works. The prediction of the magnitude of possible errors in flood protection design, because of changes in stage with time, requires the quantification of changes in stage. To quantify these changes it is necessary to be able to quantify future changes in the variables that affect the stage. In this respect, knowledge of the future flow conditions is necessary.

The information presented in the preceding chapter can be used to determine the direction of change of hydraulic variables when the water and sediment discharges are varied. It is important to notice that Einstein's, Colby's, and Manning's equations apply to a cross section or reach and differ from some of the available geomorphic equations that have been derived by considering a reach or total length of river. Einstein's, Colby's and Manning's equations deal with depth of flow, width of flow and energy slope whereas most geomorphic equations deal with channel depth, channel width and channel slope.

The interdependency of top width, depth of flow, energy slope, bed-material size and kinematic viscosity on the water and sediment discharge allows the establishment of the relative influence of those variables on stage-discharge relationships. Information concerning the interdependency of top width, depth of flow, energy slope, bed material size and kinematic viscosity with water and sediment discharges can be used to establish the direction of variation of hydraulic variables as a consequence of changes imposed on the water and bed-material discharge.

Neither Einstein's bed-load function nor Colby's relationships directly take into account the width of the cross section, except when transforming the sediment discharge per foot of width to the total river width. The influence of the width, nevertheless, indirectly enters any method of estimating transport, since width affects the depth of flow for a given water discharge and energy slope. With the total information provided to date, the response of a river system to changes in variables are given in Table 4.5.1. A plus (+) sign signifies an increase in the value of the variable and a minus (-) sign signifies a decrease in the value of the variable. The letter B indicates an increase in the product $SQ^{1/4}$ and a shift toward a braided condition and the letter M indicates a reduction in $SQ^{1/4}$ and a shift toward the meandering condition. No attempt is made here to determine whether or not the channel braids or meanders.

4.5.2 Prediction of Channel Response to Change

In Section 4.5.1, it was illustrated that Equation 4.5.3 could be used to predict changes in channel profiles caused by changes in water and sediment discharge. It is now possible to talk qualitatively about changes in channel profile, changes in river form and changes in river cross section both at-a-station and along the river channel using the other relations presented above.

This can be best illustrated by application. Referring to Table 4.5.2, consider the effect of an increase in discharge indicated by a plus sign on line (a) opposite discharge. The increase in discharge may affect the river form, energy slope, stability of the channel, cross-sectional area and river regime. Equation 4.4.18 and Equation 4.4.19 or Figure 4.4.3 show that an increase in discharge could change the channel form in the direction of a braided form. Whether or not the channel form changes would depend on the river form prior to the increase in discharge. With the increase in discharge the stability of the channel would be reduced, which indicates an increase in velocity. On the other hand, this prediction could be affected by changes in form of bed roughness that dictate resistance to flow. This effect is discussed later.

Table 4.5.1 Change of Variables Induced by Changes in Sediment Discharge, Size of Bed Sediment and Wash Load

Relationship	Tendency to Braid or Meander
$Q^{+}_{s}\ D_{50} / C_{f} \sim S^{+}\ V^{+}\ y^{-}_{o}\ W^{+}$	B
$Q^{-}_{s}\ D_{50} / C_{f} \sim S^{-}\ V^{-}\ y^{+}_{o}\ W^{-}$	M
$Q_{s}\ D_{50}{}^{+} / C_{f} \sim S^{+}\ V^{+}\ y^{-}_{o}\ W^{+}$	B
$Q_{s}\ D^{-}_{50} / C_{f} \sim S^{-}\ V^{\pm}\ y^{\pm}_{o}\ W^{\pm}$	M
$Q_{s}\ D_{50} / C^{+}_{f} \sim S^{-}\ V^{\pm}\ y^{\pm}_{o}\ W^{\pm}$	M
$Q_{s}\ D_{50} / C^{-}_{f} \sim S^{+}\ V^{+}\ y^{-}_{o}\ W^{+}$	B
$Q^{+}_{s}\ D^{+}_{50} / C_{f} \sim S^{+}\ V^{+}\ y^{-}_{o}\ W^{+}$	B
$Q^{-}_{s}\ D^{-}_{50} / C_{f} \sim S^{-}\ V^{\pm}\ y^{\pm}_{o}\ W^{-}$	M
$Q^{+}_{s}\ D^{+}_{50} / C^{-}_{f} \sim S^{+}\ V^{+}\ y^{-}_{o}\ W^{+}$	B
$Q^{-}_{s}\ D^{-}_{50} / C^{+}_{f} \sim S^{-}\ V^{\pm}\ y^{\pm}_{o}\ W^{\pm}$	M
$Q^{+}_{s}\ D^{+}_{50} / C^{+}_{f} \sim S^{+}\ V^{+}\ y^{-}_{o}\ W^{+}$	B
Note: An increase in the value of the variable is denoted by a +; and a decrease is denoted by a -. As an example, in the first line, if the value of Q_s increases, the slope, velocity and width will increase and the depth of flow will decrease.	

Table 4.5.2 Qualitative Response of Alluvial Channels								
Variable	Change in Magnitude of Variable	Effect On						
		Regime of Flow	River Form	Resistance to Flow	Energy Slope	Stability of Channel	Area	Stage

Discharge	(a) (b)	+ -	+ -	M → B B → M	± ±	- +	- +	+ -	+ -
Bed Material Size	(a) (b)	+ -	- +	M → B B → M	+ -	+ -	± ±	+ -	+ -
Bed Material Load	(a) (b)	+ -	+ -	B → M M → B	- +	- +	+ -	- +	- +
Washload	(a) (b)	+ -	+ -		- +	- +	± ±	- +	- +
Viscosity	(a) (b)	+ -	+ -		- +	- +	± ±	- +	- +
Seepage Force	(a) (b)	Outflow Inflow	- +	B → M M → B	+ -	- +	+ -	+ -	+ -
Vegetation	(a) (b)	+ -	- +	B → M M → B	+ -	- +	+ -	+ -	+ -
Wind	(a) (b)	Downstream Upstream	+ -	M → B B → M	- +	+ -	- -	- +	- +

Recall that the wash load increases the apparent viscosity of the water and sediment mixture. This makes the bed material behave as if it were smaller. In fact, the fall diameter of the bed material is made smaller by significant concentrations of wash load. With more wash load, the bed material is more susceptible to transport and any river carrying significant wash load will change from lower to upper regime at a smaller Froude number than otherwise. Also, the viscosity is affected by changes in temperature.

Seepage forces resulting from seepage losses help stabilize the channel bed and banks. With seepage inflow, the reverse is true. Vegetation adds to bank stability and increases resistance to flow, reducing the velocity. Wind can retard flow, increasing roughness and depth, when blowing upstream. The reverse is true with the wind blowing downstream. Wind generated waves and their adverse influence on channel stability are the most significant effects of wind.

In many instances it is important to assess the effects of changes in water and sediment discharge on specific variables such as depth of flow, channel width, characteristics of bed materials, velocity and so forth. For this type of analysis we can use Equation 4.5.3 and the at-a-station hydraulic geometry relation. Equation 4.5.3 is written in terms of width, depth, velocity, concentration of bed sediment discharge C_s and water discharge Q as follows

$$QS \sim Q_s D_{50} = QC_f D_{50} \qquad (4.5.4)$$

and

$$C_f D_{50} \sim S \qquad (4.5.5)$$

These equations are helpful for detailed analysis.

Our discussion thus far indicates three geomorphic responses or processes can result from changes in dominant channel flow and sediment conditions. These are channel widening, channel deepening, and changing plan view form (a change in sinuosity or meander pattern). All of these responses will cause some level of streambank erosion.

Channel widening is evidenced through an increase in channel width, with or without an increase in channel depth. An increase in flow or sediment discharge results in a tendency toward channel widening. When both sediment discharge and flow increase, however, the channel section can be expected to increase its depth as well as its width. When only sediment load increases, width increases but the depth may decrease. In this case the channel is said to be aggrading, implying that the channel has filled in because of an excess of sediments.

Channel deepening is a process of channel degradation that increases the depth of the channel. Channel degradation can cause bank instability by producing a steeper bank angle. Whether or not instability actually occurs is a function of the properties of the bank materials and the original bank geometry. Channel deepening results from increased flow without an appreciable increase in sediment discharge. Increased flow rates can result from an overall increase in the volume of water moving through the channel or an increase in channel slope.

Changing plan view form includes changes in channel shape and position as viewed from above. Changes in plan view form are most often exhibited through the downstream migration of meandering bends and changes in the sinuosity of meander bends. Other examples include the shifting of channels and the cutting off of meander bends. Generally, these changes are manifested by an adjustment of channel slope to conform with changes in flow or sediment discharge.

A reduction in sediment discharge or an increase in water discharge will result in a reduction of the channel slope. These slope reductions result in increased channel sinuosity and/or channel-bed degradation; both of which lead to a tendency toward increased bank erosion. Also, a reduction in sediment discharge will result in an increase in channel sinuosity, again, leading to increased bank erosion.

It is important to recognize that the three geomorphic processes just discussed (channel widening, channel deepening, and changing plan view form) are often interrelated and can occur simultaneously or in sequence. For example, adjustments in channel slope through degradation often are accompanied by increases in channel sinuosity and bank caving or channel widening. Also, the initiation of a given process at a particular site may initiate another process either upstream or downstream. For example, an aggrading channel reach can cause an increase in sinuosity in a downstream reach.

4.6 Modeling of River Systems

The necessity for quantitative prediction of river channel response is increasing. The accuracy of such a prediction depends on the quality of the data. There are generally two ways of predicting response. One is the mathematical model and the other is the physical model. Mathematical models utilize a number of mathematical equations governing the motion of water and sediments in a channel. Regardless of the potential of mathematical models, to date they have been best used to study channel response using one-dimensional approximations. For complex three-dimensional channel processes, it is very difficult to accurately formulate mathematically what happens in a river. Studies of channel response to development for complex situations are usually made using a physical model. The physical model is designed to achieve similar behavior as that of the prototype. The relationship of the governing physical processes and parameters must then be the same for the model as for the prototype.

4.6.1 Physical Modeling

In the present context, physical models are a replication of phenomena associated with the behavior of highway crossings. Physical models are used to test the performance of a design or to study the details of a phenomenon. The performance tests of proposed structures can be made at moderate costs and small risks on small-scale (physical) models. Similarly, the interaction of a structure and the river environment can be studied in detail.

The natural phenomena are governed by appropriate sets of governing equations. If these equations can be integrated, the prediction of a given phenomena in time and space domains can be made mathematically. In many cases related to river engineering, all the governing equations are not known. Also, the known equations cannot be directly treated mathematically for the geometries involved. In such cases, physical models are used to physically represent solutions to the governing equations.

The similitude required between a prototype and a model implies two conditions:

1. To each point, time and process in the prototype, a uniquely coordinated point, time and process exists in the model; and
2. The ratios of corresponding physical magnitudes between prototype and model are constant for each type of physical quantity.

- Rigid Boundary Models - To satisfy the preceding conditions in clear water, geometric, kinematic, and dynamic similarities must exist between the prototype and the model. Geometric similarity refers to the similarity of form between the prototype and its model. Kinematic similarity refers to similarity of motion, while dynamic similarity is a scaling of masses and forces. For kinematic similarity, patterns or paths of motion between the model and the prototype should be geometrically similar. If similarity of flow is maintained between the model and prototype, mathematical equations of motion will be identical for the two. Considering the equations of motion, the dimensionless ratios of $V / \sqrt{gy}$ (Froude number) and Vy/υ (Reynolds number) are both significant parameters in models of rigid boundary clear water open channel flow.

It is seldom possible to achieve kinematic, dynamic and geometric similarity all at the same time in a model. For instance, in open channel flow, gravitational forces predominate, and hence, the effects of the Froude number are more important than those of the Reynolds number. Therefore, the Froude criterion is used to determine the geometric scales, but only with the knowledge that some scale effects, that is, departure from strict similarity, exists in the model.

Ratios (or scales) of velocity, time, force and other characteristics of flow for two systems are determined by equating the appropriate dimensionless number which applies to a dominant force. If the two systems are denoted by the subscript m for model and p for prototype, then the ratio of corresponding quantities in the two systems can be defined. The subscript r is used to designate the ratio of the model quantity to the prototype quantity. For example, the length ratio is given by

$$L_r = x_m/x_p = y_m/y_p = z_m/z_p \qquad (4.6.1)$$

for the coordinate directions, x, y, and z. Equation 4.6.1 assumes a condition of exact geometric similarity in all coordinate directions.

Frequently, open channel models are distorted. A model is said to be distorted if there are variables that have the same dimension but are modeled by different scale ratios. Thus, geometrically distorted models can have different scales in horizontal (x, y) and vertical (z) directions and two equations are necessary to define the length ratios in this case.

$$L_r = x_m/x_p = y_m/y_p \qquad (4.6.2)$$

and

$$z_r = z_m/z_p \qquad (4.6.3)$$

If perfect similitude is to be obtained the relationships that must exist between the properties of the fluids used in the model and in the prototype are given in Table 4.6.1 for the Froude, Reynolds and Weber criteria. An example on how to use this table is presented in Appendix 4, Problem 4.7.

In free surface flow, the length ratio is often selected arbitrarily, but with certain limitations kept in mind. The Froude number is used as a scaling criterion because gravity has a predominant effect. However, if a small length ratio is used (very shallow water depths) then surface tension forces, which are included in the Weber number $(V/\sqrt{\sigma / \rho L})$, may become important and complicate the interpretations of results of the model. It is desirable if the length scale is made as large as possible so that the Reynolds number is sufficiently large and friction becomes a function of the boundary roughness and essentially independent of the Reynolds number. A large length scale also ensures that the flow is as turbulent in the model as it is in the prototype.

The boundary roughness is characterized by Manning's roughness coefficient, n, in free surface flow. Analysis of Manning's equation and substitution of the appropriate length ratios, based upon the Froude criterion, results in an expression for the ratio of the roughness which is given by

$$n_r = L_r^{\ 1/6} \qquad (4.6.4)$$

It is not always possible to achieve boundary roughness in a model and prototype that corresponds to that required by Equation 4.6.4 and additional measures such as adjustment of the slope, may be necessary to offset disproportionately high resistance in the model.

- Mobile Bed Models - In modeling highway crossings and encroachments in the river environment, three-dimensional mobile bed models are often used. These models have the bed and sides molded of materials that can be moved by the model flow. Similitude in mobile bed models implies that the model reproduces the fluvial processes such as bed scour, bed deposition, lateral channel migration, and varying boundary roughness. It has not been considered possible to faithfully simulate all of these processes simultaneously on scale models. Distortions of various parameters are often made in such models.

Table 4.6.1 Scale Ratios for Similtude

Characteristic	Dimension	Re	Fr	We
Length Area Volume	L L^2 L^3	L L^2 L^3	L L^2 L^3	L L^2 L^3
Time Velocity Acceleration	T L/T L/T^2	$\rho L^2/\mu$ $\mu/L\rho$ $\mu^2/\rho^2 L^3$	$(L\rho/\gamma)^{1/2}$ $(L\gamma/\rho)^{1/2}$ γ/ρ	$(L^3\rho/\sigma)^{1/2}$ $(\sigma/L\rho)^{1/2}$ $\sigma/L^2\rho$
Discharge	L^3/T	$L\mu/\rho$	$L^{5/2}(\gamma/\rho)^{1/2}$	$L^{3/2}\ (\sigma/\rho)^{1/2}$

Mass	M	$L^3\rho$	$L^3\rho$	$L^3\rho$
Force	ML/T^2	μ^2/ρ	$L^3\gamma$	$L\sigma$
Density	M/L^3	ρ	ρ	ρ
Specific Weight	M/L^2T^2	$\mu^2/L^2\rho$	γ	σ/L^2
Pressure	M/LT^2	$\mu^2/L^2\rho$	$L\gamma$	σ/L
Impulse and Momentum	ML/T	$L^2\mu$	$L^{7/2}(\rho\gamma)^{1/2}$	$L^{5/2}(\rho\sigma)^{1/2}$
Energy and Work	ML^2/T^2	$L\mu^2/\rho$	$L^4\gamma$	$L^2\sigma$
Power	ML^2/T^3	$\mu^3/L\rho^2$	$L^{7/2}\gamma^{3/2}\rho^{-1/2}$	$\sigma^{3/2}(L\rho)^{1/2}$

Two approaches are available to design mobile bed models. One approach is the analytical derivation of distortions explained by Einstein and Chien (1956) and the other is based on hydraulic geometry relationships given by Lacey, Blench, and others (see Mahmood and Shen, 1971). In both of these approaches, a first approximation of the model scales and distortions can be obtained by numerical computations. The model is built to these scales and then verified for past information obtained from the prototype. In general, the model scales need adjusting during the verification stage.

The model verification consists of the reproduction of observed prototype behavior under given conditions on the model. This is specifically directed to one or more alluvial processes of interest. For example, a model may be verified for bed-level changes over a certain reach of the river. The predictive use of the model should be restricted to the aspects for which the model has been verified. The use is based on the premise that if the model has successfully reproduced the phenomenon of interest over a given hydrograph as observed on the prototype, it will also reproduce the future response of the river over a similar range of conditions.

The mobile bed models are more difficult to design and their theory is vastly more complicated as compared to clear water rigid bed models. However, many successful examples of their use are available the world over. In general, all important river training and control works are invariably studied on physical models. The interpretation of results from a mobile bed model requires a basic understanding of the fluvial processes and some experience with such models. Even in the many cases where it is only possible to obtain qualitative information from mobile bed models, this information is of great help in comparing the performance of different designs.

Furthermore, the construction, operation, and modification of physical models are expensive, time consuming and laborious, especially when long-term response is investigated.

Adoption of a particular method for estimating river response depends on quality and availability of data as well as the engineer's experience. More detail on physical modeling can be found from works by Gessler (1971), Yalin (1971), Bogardi (1974), and Novak and Cabelka (1981).

4.6.2 Computer Modeling

The design engineer's interest in alluvial river response is generally focused on anticipating how the river bed and water-surface elevations will change if an existing stable or equilibrium situation is perturbed. This perturbation may be the occurrence of an unusually large annual flood that temporarily scours the bed and banks to accomodate the higher flow before returning to normal conditions. Or the perturbation may be a permanent change in river discharge patterns and geometry caused by upstream regulation of flows or bank stabilization and channelization. The first type of perturbation is often susceptible to simulation using a physical scale model. Although problems arise with interpretation of the results, physical models, in the hands of experience modelers, can yield valuable information on local scour and deposition around structures. However, the sheer expense and space requirements of physical scale models generally disquality them for simulation of long-term, large-distance river bed response to the second type of perturbation. This is where numerical, computer-based models, which can simulate both short- and long-term response, find their natural area of application.

Numerical models of alluvial river response are the natural outgrowth of rigid-boundary, unsteady flood propagation models that have proven so useful in engineering design. These unsteady flow models have succeeded because they are based on mathematical descriptions that incorporate all the important physical processes involved and use reliable, carefully implemented numerical methods to obtain approximate solutions to the appropriate partial-differential equations. However, alluvial river-response models have enjoyed nowhere near the success of their rigid-boundary cousins, precisely because of the weaknesses in our understanding and mathematical formulation of the relevant physical processes. Notwithstanding this fundamental difficulty, design engineers have an immediate need for reliable numerical simulations, and hydraulic research engineers have targeted alluvial river hydraulics as a prime area for continuing fundamental and applied research. Out of this fortunate confluence of interest have arisen a variety of simulation techniques and industrialized software systems, as well as many apparently successful simulations of prototype situations.

The most basic one-dimensional description of water and sediment flow in an alluvial river consists of four relations: conservation of water; conservation of water momentum; conservation of sediment; and sediment-transport law. These equations form a nonlinear partial-differential system that in general cannot be solved analytically.

When the water wave propagation effects are of secondary importance for sediment-transport phenomena, the system of equations can be simplified by assuming that the water flow remains quasi-steady during a certain interval of time.

Virtually all published software systems for the solution of the water- and sediment-flow equations use one form or another of the finite-difference method, in which time and space derivatives are approximated by differences of nodal values of grid functions that replace the continuous functions, leading to a system of algebraic equations. Some authors have used the finite-element method, but in one dimension there does not appear to be any strong reason for doing so. In any case, the quality and reliability of numerical models for bed evolution are determined primarily by the sediment-transport formulation and mechanisms adopted for sorting, armoring, and so forth. The particular numerical method used, as long as it is consistent with the partial-differential equations and is stable, has only a secondary effect on simulation quality.

Whether the full unsteady set of equations or the quasi-steady set of equations is solved numerically, two basic approaches are possible: coupled or uncoupled. In the coupled case, a simultaneous solution of both water and sediment equations is sought. This is evidently the physically proper way to proceed, because the water-flow and sediment-transport processes occur simultaneously. However, the simultaneous solution may involve certain computational complications, especially when the sediment-transport flow resistance equation involves not just an analytic mathematical expression but a whole series of procedures and computations to simulate armoring, sorting, bed forms, and so forth.

The uncoupled procedure has arisen essentially to circumvent the computational difficulties of the coupled approach. The uncoupling of the liquid and solid transport occurs during a short computational time step, Δt. First the water-flow equations are solved to yield new values of depth and velocity throughout the reach of interest, assuming that neither the bed elevation nor the bed-sediment characteristics change during the time step. Then the depths and velocities are taken as constant, known inputs to the sediment continuity and transport equations; these equations then become relatively easy to solve numerically, yielding the new bed elevations. When the overall model includes bed-sediment sorting or armoring, these processes are simulated in a third uncoupled computational phase using new depths, velocities, and bed elevations as known inputs. Although it is difficult to quantify the error associated with this artificial uncoupling of simultaneous, mutually dependent processes, it is intuitively obvious that the uncoupling is justified only if bed elevations and bed-material characteristics change very little during one time step. Experience in the use of uncoupled models, with both the unsteady and quasi-steady water-flow equations, has shown that the uncoupling is not a serious obstacle to successful simulation.

Another distinguishing feature of numerical bed-evolution models is the representation of sediment sorting and bed-surface armoring. Alluvial sediments are rarely of uniform grain size. A broad range of sizes are represented, from gravels and coarse sands down to fine silt and clay in varying proportions. Finer particles are preferentially entrained into the flow as erosion occurs, so that the material remaining on the bed contains a progressively higher proportion of coarser material. This so-called sorting process tends to increase the mean bed-sediment size as degradation occurs, thus affecting the sediment-transport rate, river regime (existence of ripples and dunes), and flow resistance through both particle roughness and bed-form effects. If the original bed material contains a high enough proportion of large, non-moveable materials (coarse gravel, cobbles, and small boulders), an interlocking armor layer may form on the surface, arresting further degradation. These processes are qualitatively reversed during deposition, but become even more difficult to quantify.

No computer-based models presently available incorporate a general, adequate treatment of sorting and armoring processes. Nevertheless, some models attempt to simulate their effects on bed evolution; others ignore them completely. Thus another important distinguishing feature of computer-based models is the degree to which they incorporate sorting and armoring effects.

Numerical modeling of alluvial river flows has become very popular in recent years because of the advancement of digital-computer technology. However, the number of computer-based, alluvial riverbed prediction models that are readily available for application to prototype cases seems to be quite small. Most of the available models have been developed for specific rivers under particular flow and alluvial riverbed conditions, and many of them are, to some extent, well tuned or calibrated only for those particular rivers. The summarized presentation of selected models follows the recent review of Holly et al. (1984).

The assessment of the selected models is made for two different groups: short-term models and long-term models. The short-term models are best suited to compute changes in alluvial riverbed level during a relatively short time period. They are suited for a single-flood event because of the relatively high cost of backwater computation using either unsteady flow equations or a rather complex fixed-bed water-routing model such as WSPRO or HEC-2 or the alluvial bed model BRI-STARS. On the other hand, the long-term models employ simpler implementations of steady-state flow equations, and thus are suited for long-term prediction of riverbed level for multiple-flood events over multiple years. However, it should be recognized that the short-term models can also be applied for long-term prediction if variable time steps are employed. In that case a shorter time step is used for highly unsteady flows and a longer time step is used otherwise.

- **Short-Term Models**

HEC2SR (HEC-2 with Sediment Routing)

This known-discharge, uncoupled, water- and sediment-routing model was developed by Simons, Li and Associates (SLA, 1980) for simulating watershed sediment yield and the attendant riverbed aggradation and degradation in a river system. The model uses the HEC-2 fixed-bed, backwater-computation program developed by the U.S. Army Corps of Engineers (COE), Hydrologic Engineering Center (HEC) for water routing. HEC-2 solves one-dimensional, steady-state, gradually varied flow using the flow-continuity and flow-energy equations. HEC-2 accounts for various kinds of flow encroachments, such as bridge constrictions and multiple channels, and allows for nonuniform distribution of the bed-roughness coefficient across the channel.

Once various hydraulic parameters are determined by the HEC-2 computation, the bed material and washload discharges are estimated for each computational reach. The model uses the Meyer-Peter and Muller formula for the bed load discharge computation and the Einstein formula for the suspended load discharge. The combined bed sediment transport rates are further corrected for wash load effects using Colby's empirical relationships (1964). The sediment volume change determined from the balance between the sediment inflow and outflow of each subreach is distributed uniformly along the reach. Therefore, the sediment routing model that solves the sediment continuity equation cannot predict local scour or deposition patterns. However, dredging effects can be incorporated during the computation of the sediment volume change. The change in cross-sectional profile is determined by a weighting factor based on flow conveyances in adjacent lateral subsections. Armoring effects and changes of bed material composition are considered during each sediment routing phase. After the sediment routing phase, hydraulic and bed profile data in the HEC-2 data file are updated, and the waterand sediment-routing computation for the next time step begins.

Because of the high cost of backwater computation, the model is not suitable for the long-term prediction of riverbed changes. The model is purely one dimensional and accounts for neither lateral channel migration nor secondary flows.

UUWSR (Uncoupled, Unsteady Water and Sediment Routing)

This model was developed at Colorado State University by Chen (1973) and Simons and Chen (1979) for simulating one dimensional, gradually varied, unsteady, water and sediment flows in complicated river networks. The model first solves the unsteady flow-continuity and flow-momentum equations by an unconditionally stable, four-point, implicit, finite-difference scheme assuming a fixed bed during one time step. It is assumed that the bed-roughness coefficient for the unsteady flow is the same as that for a steady flow. Three types of boundary conditions may be used: upstream discharge hydrograph, upstream stage hydrograph, and downstream stage-discharge rating curve. The water-routing model also considers the effects of tributary confluences and dams on water-surface profiles in the study reach.

The computed flow information is used to compute the sediment-transport capacity. Computed sediment discharges are then applied to the sediment-continuity equation to compute the change in the cross-sectional area by means of an explicit finite-difference scheme. Changes in bed material composition are not taken into account. It should be noted that steady-state conditions are assumed at confluences and dams of the study reach. The model is able to simulate, with minimal computer cost, a complex river-network system in which islands, meander loops, and tributaries are connected to the main channel. The model can also account for effects of hydraulic structures such as dikes, locks, and dams. The flood-wave movement in a long reach can be simulated by this unsteady flow-routing model.

FLUVIAL-11

This uncoupled model was developed at San Diego State University in 1976 by Chang and Hill to simulate one dimensional, unsteady, gradually varied, water and sediment flows for channels with erodible banks. FLUVIAL-11 first solves the unsteady, flow-continuity and flow-momentum equations in one time step by neglecting storage effects due to unsteady flow. The model uses an implicit, central-difference, numerical scheme in solving for the two unknown variables of water discharge and cross-sectional area. The flow information is then used to compute the bed sediment discharge at each section using either the Graf formula or the Engelund-Hansen formula.

The net change in cross-sectional area is next obtained by solving the sediment-continuity equation using a backward-difference scheme for space and a forward-difference scheme for time. The computed cross-sectional area change is then adjusted for the effects of channel migration. Width adjustments are made in such a manner that the spatial variation in power expenditure per unit channel length is reduced along the reach by a trial and error technique. Further adjustment of cross-sectional area is made to reduce the spatial variation in power expenditure along the channel. The effect of lateral channel migration is determined by solving the sediment-continuity equation in the transverse direction, which incorporates the effect of radius of curvature of the river bend into the transverse component of the sediment transport rate. FLUVIAL-11 is unique because of its capability to predict changes in erodible channel width, changes in channel-bed profile, and lateral migration of a channel in bends. A comparison of the performance of these short-term methods is presented in Appendix 4, Problem 4.8.

- **Long-Term Models**

KUWASER (Known-Discharge, Uncoupled, Water and Sediment Routing)

The KUWASER model was developed at Colorado State University by Brown (1982), Simons, Li and Brown (1979). The water discharge is taken as steady during a specified time interval, so that water-flow routing consists of simply solving the backwater equation with an additional term for explicit representation of energy losses other than those caused by bedshear stress. The equation of continuity of sediment is solved by first computing the sediment volume to be removed or added to each reach, then allocating 25 percent of this volume to the upstream half of the reach and 75 percent to the downstream half. Cross-sectional changes are computed in a quasi-two-dimensional manner by allocating the volume change across the channel in direct proportion to the local longitudinal hydraulic conveyance factor. Lateral channel boundaries are assumed to be fixed (nonerodible banks); neither hydraulic-sorting nor bed-armoring processes are taken into account explicitly, though their effects may appear indirectly in the regression coefficient of the sediment transport function.

KUWASER uses an empirical sediment-transport function. Flow resistance is uncoupled from bed evolution through use of simple Manning-Strickler equations for energy loss.

The use of KUWASER is limited to subcritical flows and channels without extremely irregular grade and geometry. However, it has the capability to model the mainstem and tributaries of a river system and can simulate divided flow associated with bars, islands, or channel breaches.

BRI-STARS MODEL (The Bridge Stream Tube Model for Alluvial River Simulation)

The BRI-STAR model developed under the National Cooperative Highway Research Program (NCHRP) is a semi-two-dimensional model capable of computing alluvial scour/deposition through subcritical, supercritical, and a combination of both flow conditions involving hydraulic jumps. This model, unlike the conventional water and sediment routing computer models, is capable of simulating channel widening/narrowing phenomenon as well as local scour due to highway encroachments.

The channel widening/narrowing is accomplished by coupling a stream tube computer model with a decision-making algorithm using rate of energy dissipation or total stream power minimization. The first component, the fixed-width streamtube computer model, simulates the scouring/deposition process taking place in the vertical direction across the channel. The second component, the total stream power minimization algorithm, determines what takes place in the lateral or vertical direction. It is this component that allows the lateral changes in channel geometries. Finally, the bridge component allows the computation of the hydraulic flow variables and the resulting scour due to highway encroachments.

HEC-6 (Hydrologic Engineering Center)

The HEC-6 program was developed at the Hydrologic Engineering Center of the U.S. Army Corps of Engineers in 1977. The quasi-steady backwater equation is used to compute water-flow conditions uncoupled from the sediment-continuity equation, with expansion and contraction losses explicitly taken into account. The Manning-Strickler equation is used to compute energy loss caused by bed and bank roughness; roughness coefficients must be specified as input data, though they can be allowed to vary with discharge or stage.

The sediment-continuity equation is solved using an explicit finite-difference scheme, with sediment-transport capacities determined from water-flow conditions previously determined in the uncoupled backwater computation. The entire movable bed portion of the channel is assumed to aggrade or degrade uniformly. Sediments are routed by individual size fraction, which makes possible a detailed accounting of hydraulic sorting and development of an armored layer. Bank lines are assumed to be stable and fixed in the HEC-6 computation.

HEC-6 is strictly a one-dimensional model with no provision for simulating the development of meanders or specifying a lateral distribution of sediment transport rate across the section. The model is not suitable for rapidly changing flow conditions but can be applied to predict reservoir sedimentation, degradation of the streambed downstream from a dam, and long-term trends of scour or deposition in a stream channel, including the effects of dredging.

CHAR II (Charriage dams les Rivieres)

The CHAR II modeling system was developed by the French consulting engineering firm SOGREAH in the early 1970s (Cunge et al., 1981). It is a coupled, quasi-steady model using an implicit finite-difference scheme. Energy losses caused by bed roughness are based on the Manning-Strickler equation, with overall section conveyances computed as the sums of individual rectangular sections following Chow's method. Localized energy losses and hydraulic works are modeled with the appropriate equations discretized between two adjacent computational points.

CHAR II considers banks to be nonerodible. Degradation and aggradation volumes are assumed to be uniformly distributed across the wetted channel section. No procedures for hydraulic sorting or armoring are included in the methodology, which considers only a single representative size fraction.

Sediment transport in the present version of CHAR II is limited to bed load, computed with either the Meyer-Peter and Muller, Engelund-Hansen, DuBoys, or Einstein-Brown formulae. Hydraulic roughness and sediment transport are uncoupled in CHAR II; SOGREAH'S CHAR IV program, although less industrialized than CHAR II, does take this coupling into account through use of the Einstein method.

IALLUVIAL (Iowa Alluvial River Model)

The IALLUVIAL program was developed between 1979 and 1982 by Karim and Kennedy at IIHR. It is formally classified as an iteratively coupled, quasi-steady model.

The sediment continuity equation includes sediment contributions from bank erosion and tributaries, and the effects of bankline geometry changes can be simulated by explicit introduction of known width changes with time. The effects of dredging, cutoffs, and vertical variations in bed-sediment composition are taken into account in the computation.

IALLUVIAL is based on the total load transport model (TLTM) of Karim and Kennedy (1981). This system of nonlinear equations, developed through dimensional reasoning and regression analysis of extensive laboratory and field data, specifically incorporates the coupling between sediment transport capacity and hydraulic energy losses.

IALLUVIAL is best suited for the prediction of long-term bed changes following a perturbation to the mainstem river. It has recently been used for extensive study of Missouri River degradation following upstream regulation and channelization.

An example illustrating the comparison of the performance of these long term models is given in Appendix 4, Problem 4.9.

4.6.3 Data Needs and State-of-the-Art Assessment

Common to all alluvial river-flow models are requirements for the following input information: (1) accurate initial conditions, including a cross-sectional profile and bed-material size distribution at each computational cross section; (2) accurate boundary conditions such as water and sediment inflows along the boundaries, quantitative expressions of bed-load and suspended-load discharges, size distributions of boundary-sediment input, and stage hydrographs at the upstream and downstream boundaries; and (3) bed-roughness characteristics at each computational point. It is clear that a computer simulation would be meaningless without the first and second requirements, and the lack of the third requirement would yield an erroneous estimation of flow characteristics, resulting in erroneous feedback of flow information to the riverbed.

The exclusion of even one of these three requirements may lead to serious errors in computer simulations. However, one can hardly be provided with a complete set of input data in any prototype numerical application. Therefore, a great number of assumptions often have to be made to fill the gap in the input data. Even if adequate data are provided for a study river, there still remains a need to calibrate and verify the model by means of field data. In most natural rivers, only extremely limited field data are available for high flood stages at which major riverbed changes occur, and, consequently, adequate calibration or verification of the models normally cannot be obtained. In this sense, the capability of the alluvial river-flow models can best be assessed according to how accurately they can predict riverbed changes with limited sources of input data. A numerical modeler should be aware of which input information is most important to the final result of predicting riverbed changes.

A National Research Council study pointed out that a principal deficiency of most of the available numerical models described is their inability to accurately predict channel roughness when calibration data are insufficient. It was in the calculation of sediment-discharge capacities that the various models examined differed most widely. A reliable sediment-transport formula is a prerequisite to reliable estimates of channel-geometry changes because riverbed degradation and aggradation are computed from streamwise gradients in the sediment-transport capacity of streams as the sediment-continuity equation states. The bed-armoring process during channel degradation is also not well understood and has not been adequately formulated. Armoring and the resulting coarsening of the bed-material size have a direct effect on the sediment transport capacity and the channel-bed roughness or friction factor and thereby impact on the mean velocity, depth, and friction slope of the flow. Bed-degradation processes are generally slowed by bed armoring.

The surprisingly large discrepancies among the computed results may be taken as symptomatic of inadequate input and calibration data. However, it also may be true that any modeler would be able to simulate observed changes in thalweg elevation exactly by adjusting the model's "tuning knobs" (calibration parameters) if there were fully adequate river data available. At present no alluvial riverbed model seems mature enough to answer the question: What are the input and calibration data required for the model to yield convincing, reliable results? Simple artificial adjustments of the tuning knobs in the numerical simulation, based on the availability of plentiful data, does not appear to be a satisfactory way of predicting riverbed changes.

The most important overall need is for better interpretation of physical processes and their incorporation in the numerical models. Numerical techniques for solution of the governing equations are now adequately developed for accurate prediction of alluvial riverbed profiles if an accurate sediment transport function and a bed roughness predictor were available. Improvement in model reliability requires further research in the areas described hereafter: (1) There is a strong need for a very reliable sediment transport relation because alluvial river-bed changes are the result of a streamwise gradient in the stream's sediment transport capacity; (2) The bed-armoring process during channel degradation is not well understood and has not been adequately formulated in a conceptual model. Armoring and coarsening of the bed-material size have a direct effect on the sediment-transport capacity and the bed-friction factor, and consequently affect the velocity, depth, and energy slope of the flow; (3) There is a need to develop a better friction-factor predictor that depends on flow depth and velocity and sediment discharge; (4) There is a need to incorporate into models the bank-erosion and channel-migration effects of channel widening; and (5) It is unlikely that an alluvial riverbed model that is applicable to all types of rivers will be forthcoming in the near future. Instead, each model will be most dependable for rivers of the type for which it was developed. Therefore, there is a need for an effort to classify natural rivers in terms of their hydraulic and geomorphologic characteristics, to guide engineers in the selection and application of a model that uses formulations of sediment discharge, channel roughness, channel widening, and so on that are most appropriate for their study cases.

If there is one important message to be drawn from this catalog of deficiencies, it is the following: Model developers and users must not let their preoccupations with improvements in numerical methods, user friendliness, program generalization, and other pleasant but peripheral concerns cause them to lose sight of the central and often unpleasant need to obtain a better understanding and conceptual formulation of the basic physical processes of alluvial riverbed evolution.

4.7 Highway Problems Related to Gradation Changes

Gradation problems at highway crossings include aggradation and degradation; lateral erosion problems often occur as a consequence of these changes. The highway problem most associated with aggradation is reduction of flow area, which increases backwater effects upstream of bridges and culverts. Problems associated with degradation are undermining of footings, pile bents, abutments, cutoff walls, and other flow-control or crossing structures. Degradation has also been found to undermine bank protection resulting in the instability of channel banks and increasing debris problems. A common problem associated with lateral erosion is bank slumping, which undermines abutments and piers located near the bank line. Another very common problem arises when beds of meandering streams encroach upon roadways.

Causes of gradation changes that have an impact on highway crossings can be classified into two basic categories: (1) the result of man's activities; and (2) natural causes or factors. An analysis of the case histories indicates that very few gradation changes were due to natural factors. Some gradation changes should perhaps be classified as being caused by a combination of both natural and man-induced factors. However, their number is so small that a separate category is not warranted. Because man's activities dominate the causes for gradation problems, they will be discussed first.

4.7.1 Changes Due to Man's Activities

The activities of man are literally changing the face of the Earth and generate accelerated erosion from watersheds. Some activities have had far-reaching consequences on streams and have caused, or contributed to aggradation and degradation problems at bridges. Construction of a bridge and approach embankments may also have consequences, but they are unlikely to be far-reaching. Man's activities were found to be the major cause of streambed elevation changes. Because accelerated erosion is associated with man's activities, it is often possible to anticipate many impacts on bank stability and provide adequate bank protection in advance.

From an analysis of the case histories, man's activities resulting in gradation problems can be classified into the following categories: (1) channel alterations; (2) land use changes; (3) streambed mining/excavation; and (4) dams and reservoirs.

- **Channel Alterations**

  Straightening, dredging, clearing and snagging, artificial constrictions, and other alterations of natural channels are the major causes of streambed elevation changes. Channel straightening is the dominant activity. Examples of straightening and several others are presented in Keefer et al. (1980).

  Many of the straightened channels have degraded, and degradation is usually accompanied by widening of the channel, unstable banks and serious debris problems. The degradation is attributed to an increase in channel slope that results from shortening of channel length. The increase in channel slope increases the velocity and the shear stress on the bed. As a result, the channel bed degrades until the bed becomes armored or the channel widens and begins to meander to reduce the channel slope back to an equilibrium, or stable condition. There is some evidence that degradation, if it is to occur as a consequence of channel alteration, will be most rapid during a period shortly following the alteration and will thereafter occur at a decreasing rate.

- **Land Use Changes**

  Urbanization, agriculture, strip mining, and unregulated logging are other activities of man that cause gradation problems. Natural vegetation is extremely important in maintaining channel stability. The lateral stability of most streams in the United States, particularly in regions where agriculture or lumbering is practiced, has very probably been affected by the clearing of natural vegetation. Because this clearing has occurred more or less gradually over the past hundred years, the magnitude of the effect at a particular crossing site is sometimes difficult to assess.

  The result of deforestation and agricultural activities is generally toward increased peak flows and increased sediment yield. Channel widening and reduced sinuosity are common. Grazing along the streambanks may have significant effects on bank stability.

  Urbanization normally causes significant increase in the magnitude of runoff events while reducing their duration. Urban areas are also low sediment producers because of the large percentage of land covered by impervious surfaces. The combination of increased peak runoff rates and reduced sediment loads result in channel degradation, channel widening, and a reduction in channel sinuosity.

  Improper construction activities on the other hand are known to increase both discharge and sediment load. The removal of the vegetative cover accelerates the erosion process. The response of the system to the increased discharge is to increase channel width and reduce the radius of curvature. In response to increased sediment load, the stream will increase its tendency for bank erosion.

  Mining in an upland area may cause aggradation of channels, which are then subject to degradation after the mining ceases.

- **Streambed Mining/Excavation**

  If sand or gravel is removed from an alluvial channel in quantities that represent a substantial percentage of the annual bedload in transport, the channel will probably degrade. In addition, removal of gravel from pits or trenches in or along the stream may result in a change in flow alignment at the bridge.

Downstream mining can also produce headcutting through a bridge waterway, undermining the structure. Mining operations upstream of the bridge waterway can also produce degradation at the bridge site and endanger the structure. Equation 4.5.3 provides a subjective tool for analysis of gradation changes. A case study illustrating problems at a bridge crossing due to gravel mining is presented in Chapter 7.

Highway engineers should, as a minimum, conduct bi-annual inspections of bridges upstream and downstream for gradation problems.

- **Dams and Reservoirs**

The effects of dams and reservoirs on a stream are complex and have not been thoroughly investigated. The consequences of dams and reservoirs include clear water releases; high, sustained, regulated flows; backwater; low, sustained, regulated flows; dam breach or removal; and high, controlled, irrigation canal releases.

Downstream from a reservoir, channel degradation is to be expected because of removal of sediment. This effect has been documented for many streams. The total amount of degradation is difficult to predict; if a sand-bed channel becomes armored with gravel, the amount may be small. On gravel-bed streams aggradation may occur downstream from the dam because the flow releases are insufficient to transport gravel brought in by tributary streams. Channel avulsions, which can present a serious threat to many engineering structures, are associated with most aggrading situations. Rapid lowering of river stage may result in severe bank slumping from pore-water pressures in the banks. However, the more general effect of reservoirs is probably to lessen hydraulic problems at highway crossings bridges, both by reduction of flood peaks and a reduction of lateral erosion rates.

Interbasin transfers of flow and diversions result in periods of channel instability and bank erosion until the new channel regime is established.

4.7.2 Natural Causes

Although problems resulting from natural causes are not as frequent as those resulting from man's activities, it is important to recognize natural causes in both design and maintenance of highway crossings.

Natural causes and complications from gradation problems include: alluvial fans, natural armoring, braiding, meandering/migration (natural cutoffs), recurrent flooding, high stream velocity, channel bed and bank material erodibility, fire, floating debris, mud and debris flows, earthquakes, tectonic activity, volcanic activity, and landslides.

- **Floating Debris**

Floating debris causes hydraulic problems at highway crossings nationwide. The problems are the greatest in the Pacific Northwest and the upper and lower Mississippi River Valley. Debris hazards are local and infrequent phenomena often associated with large floods. Most bridge destruction from debris is due to accumulation of debris against bridge components. Debris may partially or totally block waterways, create adverse hydraulic conditions that erode pier foundations and bridge abutments, may overtop roadways and cause structural damage. Many debris problems exist in forested areas with active logging operations. Highway crossings on streams where stream slopes are mild or moderate, in contrast to headwater streams, are more vulnerable to debris related hazards. Debris hazards occur more frequently in unstable streams where bank erosion is active. Countermeasures presently used by highway agencies include: (1) sufficient freeboard, (2) proper pier spacing, (3) solid piers, (4) debris deflectors, (5) special superstructure designs, (6) flood relief structures, and (7) routine and emergency removal of debris at bridge crossings. Most debris transported in floods does not travel a great distance and often is observable locally along the streambanks upstream from the bridge prior to the flood. Rather than in congregations, debris usually moves as individual logs in a non-random path concentrating in the thalweg of the stream. Therefore, methods for evaluating its abundance and for mitigating its hazard are deemed feasible. Examples of debris control structures for culverts are given in Reihsen (1964).

- **Mud Flows and Debris Flows**

Fast melting snowpack and overabundance of soil moisture on steep slopes throughout the Western United States causes mudflows, debris flows and landslides, threatening bridges and highway structures. There is considerable evidence of damages to highway structures in the literature. For example Hungr et al., (1984) documented a bridge for which a concrete bridge beam has been demolished by point impact during a debris flow event.

In another example, the volcanic eruption from the magmatic blast of Mount St. Helens triggered a major slope failure on the north flank of the mountain. Mudflows and debris flows were generated and swept down the Toutle and Cowlitz Rivers destroying bridges, inundating buildings, and eventually blocking the navigation channel of the Columbia River. Bradley (1984) reported the Cowlitz has aggraded markedly as a result of the post eruption hyperconcentrated flows. The upper Cowlitz and the lower Toutle have shifted from meandering to braided streams, thereby causing some difficulties in preventing the failure of some remaining bridges.

Examination of typical watershed behavior and response provides information on the impact of changes on the fluvial system. Channel stability assessments and possible gradation changes are indicated in Table 4.7.1. Those findings reflect the observations of Keefer et al. (1980). An example relating to the use of this table is presented in Appendix 4, Problem 4.3.

4.7.3 Resulting Problems at Highway Crossings

Brown et al. (1980) reviewed current design practices to evaluate crossing design procedures and the effect of grade changes on these procedures. The parameters most influenced by grade changes are those used as input to the hydraulic design procedures currently in use. These input parameters include: design discharge; channel roughness; energy slope; bed slope; velocities; shear stresses; cross-sectional geometry; base level; flow depth; and flow alignment. Other components of crossing design affected by grade changes include foundation depth, bridge deck clearance, and flow opening size.

Problems encountered at bridge crossings include bridge capacity, backwater, pier and abutment alignment, footing depth at piers and abutments, and construction depth for flow-control and debris-control structures. With respect to bridge capacity and backwater, aggradation produces the most severe problems. However, debris problems associated with degradation can also have a significant impact on flow capacity and scour. Foundation depths for piers, abutments, and flow-control structures can be influenced in two ways by grade changes: the normal streambed base level will be altered; and the "normal" hydraulic conditions at a site used as input to local scour computations will be changed. The important components of bank protection design adversely affected by grade changes are key depths and the vertical extent of bank protection above and below the streambed.

Table 4.7.1 Channel Response to Changes in Watershed and River Condition (after Keefer et al., 1980)

Observed Condition	Channel Response			
	Stable	Unstable	Degrading	Aggrading
Alluvial Fan				
Upstream		X		X
Downstream		X	X	
Dam and Reservoir				
Upstream		X		X
Downstream		X	X	
River Form				
Meandering		X	Unknown	Unknown
Straight		X	Unknown	Unknown
Braided	X	X	Unknown	Unknown
Bank Erosion		X	Unknown	Unknown
Vegetated Banks	X		Unknown	Unknown
Headcuts		X	X	
Diversion				
Clear Water Diversion		X		X
Overloaded with Sediment				X
Channel Straightened		X	X	
Deforested Watershed		X		X
Drought Period	X			X
Wet Period		X	X	
Bed Material Size				
Increase		X		X
Decrease		X	Unknown	X

Problems encountered at culvert crossings can be the result of general grade changes produced by long-term changes in stream morphology or inadequate design and/or construction of culvert systems. The design components most often influenced are culvert capacity and structural stability. The greatest danger produced by aggradation is partial plugging of the culvert opening resulting in a damming effect and increasing the magnitude and frequency of flooding upstream of the structure. Degrading stream reaches affect culvert systems by reducing their structural stability. General streambed degradation has undermined the foundations of culverts resulting in their complete failure.

4.8 Stream Stability Problems at Highway Crossings

In the United States, the annual damages related to hydraulic problems at bridges and highways has been estimated at 100 million dollars during years of extreme floods. Damages by streams can be reduced by considering channel stability in site selection, bridge design, and countermeasure placement. Ideally, a stable channel is one that does not change in size, form, or position through time. However, all alluvial channels change to some degree and therefore have some degree of instability. For engineering purposes, an unstable channel is one whose rate or magnitude of change is great enough to be a significant factor in the planning or maintenance of a highway crossing during the service life of the structure. The kinds of changes considered here are: (1) lateral bank erosion; (2) degradation or aggradation of the streambed that continues progressively over a period of years; and (3) natural short-term fluctuations of streambed elevation that are usually associated with the passage of a flood (scour and fill).

Stability is inferred mainly from the nature of point bars, the presence or absence of cut banks, and the variability of stream width.

4.8.1 Bank Stability

On a laterally unstable channel, or at actively migrating bends on an otherwise stable channel, the point bars are usually wide and unvegetated and the bank opposite to a point bar is cut and often scalloped by erosion. The crescentic scars of slumping may be visible from place to place along the bankline. The presence of a cut bank opposite to a point bar is evidence of instability, even if the point bar is vegetated. Sand or gravel on the bar appears as a light tone on airphotos. The unvegetated condition of the point bar is attributed to a rate of outbuilding that is too rapid for vegetation to become established. However, the establishment of vegetation on a point bar is dependent on other factors besides rate of growth, such as climate and the timing of floods. If the width of an unvegetated point bar is considered as part of the channel width, the channel tends to be wider at bends. Streams whose width at bends is about twice or more the width at straight reaches are here called wide-bend streams.

Oxbow lakes are formed by the cutoff of meander loops, which occurs either by gradual closure of the neck (neck cutoffs) or by a chute that cuts across the neck (chute cutoffs). Neck cutoffs are associated with relatively stable channels, and chute cutoffs with relatively unstable channels. Recently formed oxbow lakes along a channel are evidence of recent lateral migration. A recently formed lake is usually immediately adjacent to the channel and it transmits flow at high river stages. Commonly, a new meander loop soon forms at the point of cutoff and grows in the same direction as the previous meander. Cutoffs tend to induce rapid bank erosion at adjacent meander loops. The presence of abundant oxbow lakes on a flood plain does not necessarily indicate a rapid channel migration rate, because an oxbow lake may persist for hundreds of years.

Along an unstable channel, bank erosion tends to be localized at bends, and straight reaches tend to be relatively stable. However, meandering of the thalweg in a straight reach is likely to be a precursor of instability. Bars that occur alternately from one side to the other of a straight reach are somewhat analogous to point bars and are indicative of a meandering thalweg.

- **For Unstable Banks with Moderate to High Erosion Rate**

  The slope angle of unstable banks usually exceeds 30 percent, and a cover of woody vegetation is rarely present. At a bend, the point bar opposite to an unstable cut bank is likely to be bare at normal stage, but it may be covered with annual vegetation and low woody vegetation, especially willows. Where very rapid erosion is occurring, the bankline may have irregular indentations. Fissures, which represent the boundaries of actual or potential slump blocks along the bankline indicate the potential for very rapid bank erosion.

- **For Unstable Banks with Slow to Moderate Erosion Rate**

  If a bank is partly graded, the degree of instability is difficult to assess and reliance is placed mainly on vegetation. The grading of a bank typically begins with the accumulation of slumped material at the base such that a slope is formed, and progresses by smoothing of the slope and the establishment of vegetation.

- **For Stable Banks with Very Slow Erosion Rate**

Stable banks tend to be graded to a smooth slope and the slope angle is usually less than about 30 percent. In most regions of the United States, the upper parts of stable banks are vegetated, but the lower part may be bare at normal stage, depending on bank height and flow regime of the stream. Where banks are low, dense vegetation may extend to the water's edge at normal stage. Mature trees on a graded bank slope are particularly convincing evidence for bank stability. Where banks are high, occasional slumps may occur on even the most stable graded banks. Shallow mountain streams that transport coarse bed sediment tend to have stable banks.

Field information on lateral migration rates for channels of different sizes has been compiled by Brice (1982). Bank erosion rates tend to increase with increasing stream size. In Figure 4.8.1 channel width is taken as a measure of stream size. The dashed line is drawn arbitrarily to have a slope of 1 and a position (intercept) to separate most equiwidth streams from most wide-bend and braided point-bar streams. For a given channel width, equiwidth streams tend to have the lowest erosion rates, and braided point-bar streams the highest. Braided streams without point bars (diamond symbol, Figure 4.8.1) plot well below the arbitrary curve because their channels are very wide relative to their discharges. Channel width is an imperfect measure of stream size, as are drainage area and discharge, particularly for the comparison of streams in arid and semiarid regions with streams in humid regions. If braided streams and braided point-bar streams (which are uncommon in most parts of the United States) are excluded, the dashed curve in Figure 4.8.1 provides a preliminary estimate of erosion rates that may be encountered at a particular site. An example on how to use these results is presented in Appendix 4, Problem 4.4.

4.8.2 Stability Problems Associated with Channel Relocation

In some circumstances it could be advantageous to change the river channel alignment because of highway encroachments. When a river crossing site is so constrained by non-hydraulic factors that consideration to alternative sites is not possible, the engineer must attempt to improve the local situation to meet specific needs. Also, the engineer may be forced to make channel improvements in order to maintain and protect existing highway structures in or adjacent to the river.

Suppose a meandering river is to be crossed with a highway, as shown in Figure 4.8.2a. Assume that the alignment is fixed by constraints in the acquisition of the right-of-way.

To create better flow alignment with the bridge, consideration is given to channel improvement as shown in Figure 4.8.2b. Similarly, consideration for improvement to the channel would also be advisable for a hypothetical lateral encroachment of a highway as depicted in Figure 4.8.2c. In either case, the designer's questions are how to realign the channel, and what criteria to use to establish the cross-sectional dimensions.

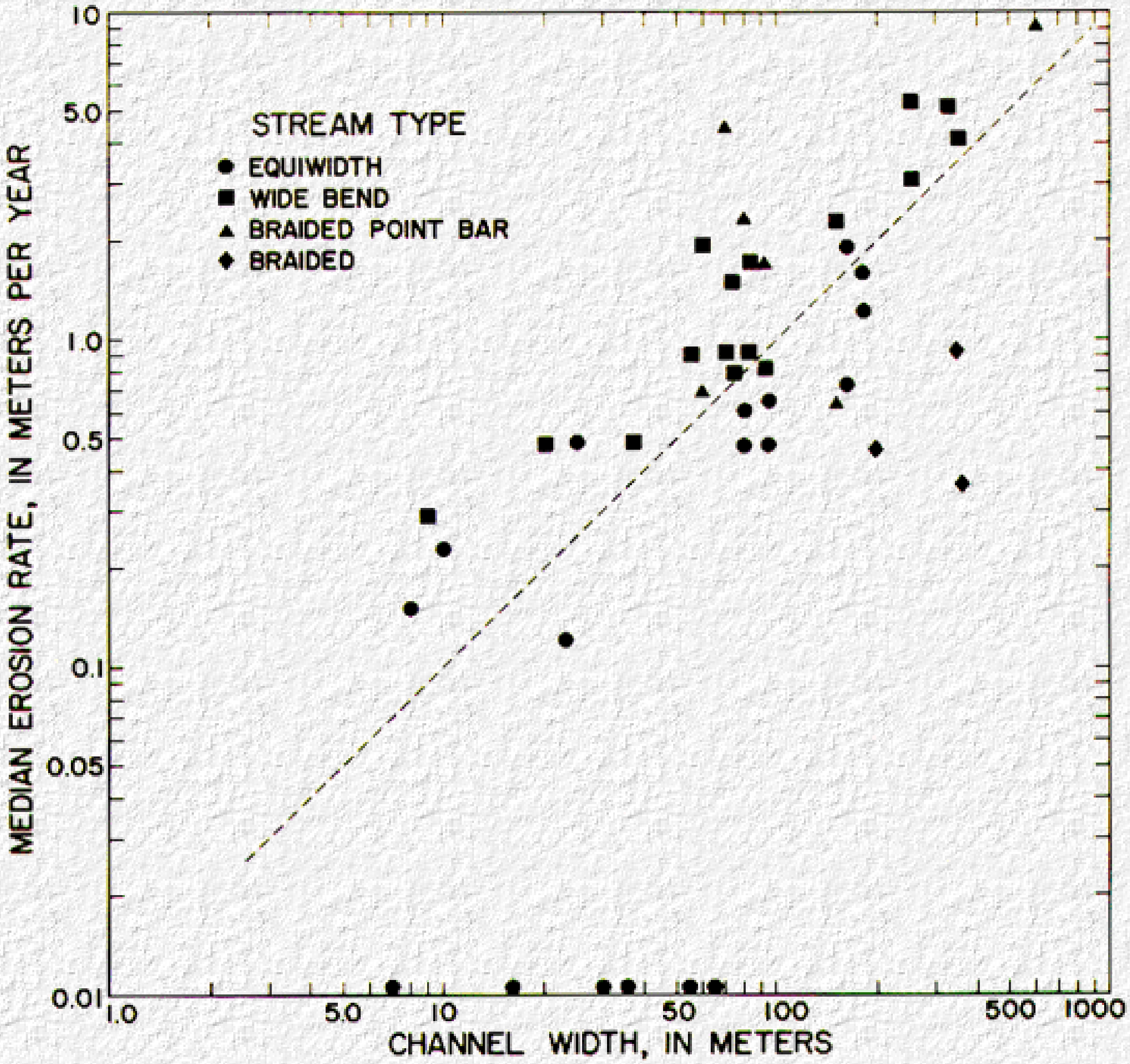


Figure 4.8.1. Median Bank Erosion Rate in Relation to Channel Width for Different Types of Streams (after Brice, 1982)

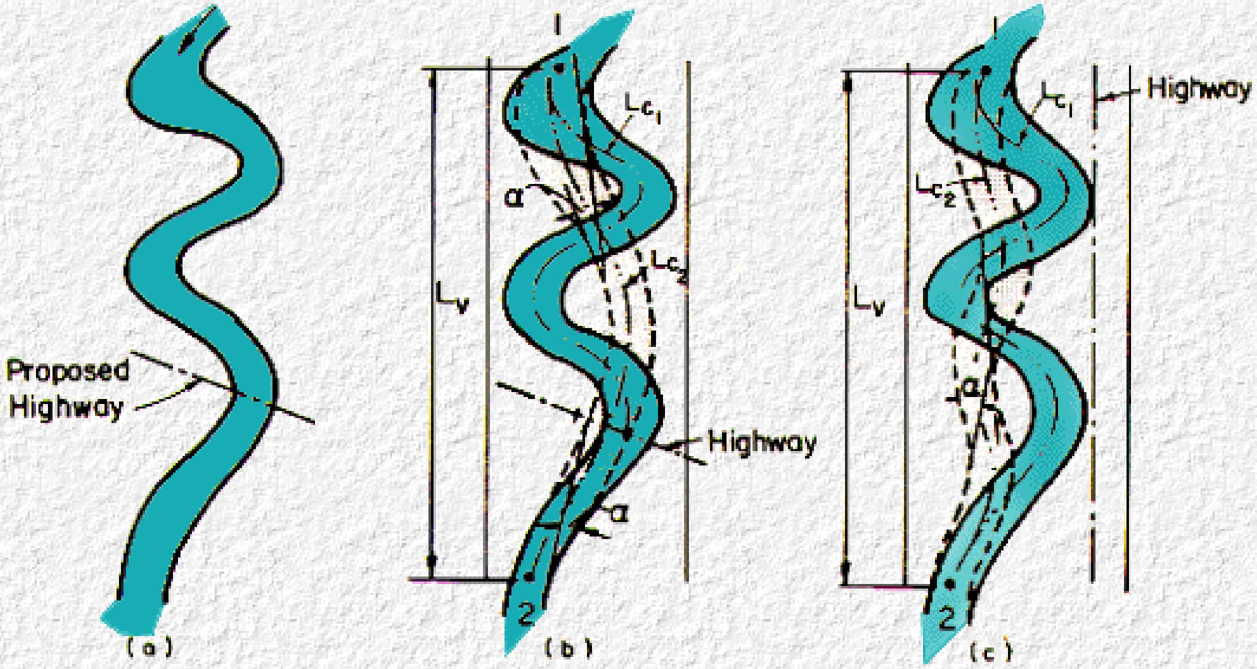


Figure 4.8.2. Encroachment on a Meandering River

Prior to realigning a river channel the stability of the existing channel must be examined using the methods outlined earlier. The stream classification, recent and past aerial photographs and field surveys are deemed necessary. The realigned channel may be made straight without curves, or may include one or more curves. If curves are included, the radii of curvature, the number of bends, the limits of rechannelization (hence the length or slope of the channel) and the cross-sectional area are decisions which have to be made by the designer. Different rivers have different characteristics and historical background with regard to channel migration, discharge, stage, geometry and sediment transport, and as indicated in the previous chapters, it is important for the designer to understand and appreciate river hydraulics and geomorphology when making decisions concerning channel relocation. It is difficult to state generalized criteria for channel relocation applicable to any river. Knowledge about river systems has not yet advanced to such a state as to make this possible. Nevertheless, it is important to provide some principles and guidelines for the design engineer.

As the general rule, the radii of bends should be made about equal to the mean radii of bends r_c, in extended reaches of the river. When the angle ϕ defined in Figure 4.3.5 exceeds about 40 degrees, this enables a sufficient crossing length for the thalweg to shift from one side of the channel to the other. Generally, it is necessary to stabilize the outside banks of the curves in order to hold the new alignment and depending upon crossing length, some amount of maintenance may be necessary to remove sandbars after large floods so that the channel does not develop new meander patterns in the crossings during normal flows.

The sinuosity and channel bed slope are related in the following way. The bed elevations at the ends of the reach being rechannelized, (designated 1 and 2, in Figure 4.8.2) are established by existing boundary conditions. Hence, the total drop in water surface bed elevation for the new channels (subscript 2) and the old channels (subscript 1) are the same.

$$\Delta z_1 = \Delta z_2 = \Delta z \qquad (4.8.1)$$

The length of channel measured along the thalweg is labeled L_c. Thus, the mean slope of the channel bed before relocation is

$$S_1 = \frac{\Delta z}{L_{c1}} \tag{4.8.2}$$

and after relocation is

$$S_2 = \frac{\Delta z}{L_{c1}} \tag{4.8.3}$$

Sinuosity is defined as the ratio of the length of channel, L_c to the length of the valley, or

$$S_n = \frac{L_c}{L_v} \geq 1 \tag{4.8.4}$$

Clearly,

$$S_{n1} = \frac{L_{c_1}}{L_{v1}} \tag{4.8.5}$$

$$S_{n2} = \frac{L_{c_2}}{L_{v2}} \tag{4.8.6}$$

but

$$L_{v_1} = L_{v_2} = L_v \tag{4.8.7}$$

and

$$\frac{\Delta z_1}{L_{v_1}} = \frac{\Delta z_2}{L_{v_2}} = \frac{\Delta z}{L_v} \tag{4.8.8}$$

Thus,

$$S_{n1} S_1 = \frac{L_{c_1}}{L_v} \bullet \frac{\Delta z}{L_{c1}} = \frac{L_{c_2}}{L_v} \bullet \frac{\Delta z}{L_{c2}} = S_{n2} S_2 \tag{4.8.9}$$

The new channel slope and channel sinuosity are inversely related. If $S_{n2} < S_{n1}$ then $S_2 > S_1$. The new channel alignment, hence S_{n2}, can be chosen by the designer with due consideration given to the radii of curvature, deflection angles and tangent lengths between reversing curves. As indicated before, consideration should also be given to prevailing average conditions in the extended reach. The new slope S_2 can be calculated from Equation 4.8.9, and the relationship for meandering should be satisfied.

$$S_2\, Q^{1/4} \le 0.0017 \qquad (4.4.18)$$

If S_1 is of such magnitude that Equation 4.4.18 cannot be satisfied with still larger S_2, the possibility of the river changing to a braided channel because of steeper slope should be carefully evaluated. With steeper slope, there could be an increase in sediment transport which could cause degradation and the effect would be extended both upstream and downstream of the relocated reach. The meander patterns could change. Considerable bank protection might be necessary to contain lateral migration which is characteristic of a braided channel, and if the slope is too steep, head cuts could develop which migrate upstream with attendant effects on the plain geometry of the channel. Even when changes in slope are not very large, a short-term adjustment of the average river slope occurs, consistent with the sediment transport rate, flow velocities and roughnesses, beyond the upstream and downstream limits of channel improvement. For small changes in slope, the proportionality (Equation 4.5.3), $QS \sim Q_s\, D_{50}$ tends toward equilibrium with slight increases in bed sediment size D_{50} and adjustment in the sediment transport rate Q_s.

A small increase in the new channel width could be considered which tends to maintain the same stream power, $\tau_o\, V$, in the old and new channels. That is,

$$(\tau_o V)_1 = (\tau_o V)_2 \qquad (4.8.10)$$

With substitution of $\tau_o = \gamma RS$, $V = Q/A$ and $R = A/P \cong A/W$, Equation 4.8.10 leads to

$$W_2 = S_2\, W_1/S_1 \qquad (4.8.11)$$

Any designed increase in width should be limited to about 10 to 15 percent. Wider channels would be ineffective. Deposition would occur along one bank and the effort of extra excavation would be wasted. Furthermore, bar formation would be encouraged, with resultant tendencies for changes in the meander pattern leading to greater maintenance costs of bank stabilization and removal of the bars to hold the desired river alignment.

The depth of flow in the channel is dependent on discharge, effective channel width, sediment transport rate (because it affects bed form and channel roughness) and channel slope. Methods for determining flow depth were discussed earlier in this chapter.

The foregoing discussion pertains to alluvial channels with silt and sand sized bed materials. For streams with gravel and cobble beds, the usual concern is to provide adequate channel cross-sectional dimensions to convey flood flows. If the realigned channels are made too steep, there is an increased stream power with a consequent increase in transport rate of the bed material. The deposition of material in the downstream reaches tends to form gravel bars and encourages changes in the plan form of the channel. Short-term changes in channel slope can be expected until equilibrium is reestablished over extended reaches both upstream and downstream of the rechannelized reach. Bank stabilization may be necessary to prevent lateral migration and periodic removal of gravel bars may also be necessary.

4.8.3 Assessment of Stability for Relocated Streams

Brice (1980) reported case histories for channel stability of relocated streams in different regions of the United States. Based on his study, the recommendations and conclusions presented here apply to specific aspects of the planning and construction of channel relocation. They are intended for assessment of the risk of instability and for reduction of the degree of instability connected with relocation. Serious instability resulting from relocation can be observed either when the prior natural channel is unstable or when floods of high recurrence interval occur during or soon after construction. Although there is an element of chance in channel stability, the experience represented by the study sites provides useful guidelines for improvement in the performance of channels relocated by highway agencies. Consideration of the following aspects of the channel relocation is required.

- **Channel Stability Prior to Relocation.** Assessment of the stability of a channel prior to relocation is needed to assess erosion-control measures and risk of instability. An unstable channel is likely to respond unfavorably to relocation. Bank stability is assessed by field study and the stereoscopic examination of aerial photographs. The most useful indicators of bank instability are cut or slumped banks, fallen trees along the bankline, and wide, unvegetated point bars exposed. Bank recession rates are measured by comparison of time-sequential aerial photographs. Vertical instability is equally important but more difficult to determine. It is indicated by changes in channel elevation at bridges and gaging stations. Serious degradation is usually accompanied by generally cut or slumped banks along a channel.
- **Erosional Resistance of Channel Boundary Materials.** The stability of a channel, whether natural or relocated, is partly determined by the erosional resistance of materials that form the wetted perimeter of the channel. Resistant bedrock outcrops, which extend out into the channel bottom, or that lie at shallow depths, will provide protection against degradation. Not all bedrock is resistant. Erosion of shale, or of other sedimentary rock types interbedded with shale, has been observed. Degradation was slight or undetected at most sites where bed sediment was of cobble and boulder size. However, serious degradation may result from relocation. <u>Degradation may result from the relocation of any alluvial channel, whatever the size of bed material,</u> but the incidence of serious degradation of channels relocated by highway agencies is slight.

  The cohesion and erosional resistance of banks tend to increase with clay content. Banks of weakly coherent sand or silt are clearly subject to rapid erosion, unless protected with vegetation. No consistent relation was found between channel stability and the cohesion of bank materials, probably because of the effects of vegetation.
- **Length of Relocation**. The length of relocation contributes significantly to channel instability at sites where its value exceeded 250 channel widths. When the value is below 100 channel widths, the effects of length of relocation is dominated by other factors. The probability of local bank erosion at some point along a channel increases with the length of the channel. The importance of vegetation, both in appearance and in erosion control, would seem to justify a serious and possibly sustained effort to establish it as soon as possible on the graded banks.
- **Bank Revetment**. Revetment makes a critical contribution to stability at many sites where it is placed at bends and along roadway embankments. Rock riprap is by far the most commonly used and effective revetment. Concrete slope paving is prone to failure. Articulated concrete block is effective when vegetation can establish in the interstices between blocks. Bank revetment is discussed in detail in the next chapter.
- **Check Dams (Drop Structures)**. In general, check dams are effective in preventing channel degradation. The potential for erosion at a check dam depends on its design and construction, its height and the use of revetment on adjoining banks. A series of low check dams, less than about 0.5 m in height, is probably preferable to a single higher structure, because the potential for erosion and failure is reduced. By simulating rapids, low check dams may add visual interest to the flow in a channel.

One critical problem arising with check dams relates to improper design for large flows. Higher flows have worked around the ends of many installations to produce failure.

- **Maintenance**. The following problems, subject to improvement by maintenance, were observed along relocated channels: (1) growth of annual vegetation in channel; (2) reduction of channel conveyance by overhanging trees; (3) local bank cutting; and (4) bank slumping. The expense of routine maintenance or inspection of relocated channels beyond the highway right-of-way is probably prohibitive.However, most of the serious problems could be detected by periodic inspection, perhaps by aerial photography, during the first 5 or 10 years after construction.
- **The Relationship between Sinuosity and Stability.** These are summarized as follows: (1) Meandering does not necessarily indicate instability; an unstable stream will not remain highly sinuous for very long, because the sinuosity will be reduced by frequent meander cutoffs; (2) Where instability is present along a reach, it occurs mainly at bends; straight segments may remain stable for decades; and (3) The highest instability is for reaches whose sinuosity is in the range of 1.2 to 2 and whose type is either wide bend or braided point bar.

4.8.4 Estimation of Future Channel Stability and Behavior

One objective of stability assessment is to anticipate the migration of bends and the development of new bends. Lateral erosion is probably more frequently involved in hydraulic problems at bridges than any other stream process. Problems caused by general scour, local scour, channel degradation and accumulation of debris are comparatively less common.

The lateral stability is measured from records of its position at two or more different times and the available records are usually maps or aerial photographs. Old surveyed cross-sections are extremely useful although unfortunately rarely available. It is recognized that some progress is being made on the numerical prediction of loop deformation and bend migration. At present, however, the best available estimates are based on past rates of lateral migration at a particular reach. However, erosion rates may fluctuate substantially from one period of years to the next.

Measurements of bank erosion on two time-sequential aerial photographs (or maps) require the identification of reference points which are common to both. Useful reference points include roads, buildings, irrigation canals, bridges and fence corners. This analysis of lateral stability is greatly facilitated by a drawing of time changes in bankline position. To prepare such a drawing, aerial photographs are matched in scale and the photographs are superimposed holding the reference points fixed.

Bank erosion rates increase with the stream size as shown in Figure 4.8.1. Sinuous canaliform streams can then be expected to have the lowest erosion rates and the sinuous braided streams, the highest.

The lateral stability of different stream reaches can be compared by means of a dimensionless erosion index. The erosion index is the product of its median bank erosion rate expressed in channel widths per year, multiplied by the percent of reach along which erosion occurred, multiplied by 1000. Erosion indexes for 41 streams in the United States are plotted against sinuosity in Figure 4.8.3. The length of most of these reaches is 25 to 100 times the channel width. The highest erosion index values are for reaches with sinuosity ranging between 1.2 and 2. Erosion indexes are large for sinuous braided and sinuous point bar streams as compared to sinuous canaliform streams. The erosion index value of 5, in Figure 4.8.3, is suggested as a boundary between stable and unstable reaches. Brice (1984) considers that reaches having erosion indexes values less than 5 are unlikely to cause lateral erosion problems at bridges.

A general assessment of bank stability is made considering the following aspects.

- **Bank Erosion Rates.** Although it is theoretically possible to determine bank erosion rates from factors such as water velocity and resistance of the banks to erosion, practical and accurate means of making such a determination are still deficient. The results in Figure 4.8.1 provide a first approximation of migration rate of a bend regardless of the hydraulic conditions and sediment characteristics. Past rates of erosion at a particular site provide the best estimate of future rates. In projecting past rates into the future, consideration must be given to the following factors: (1) the past flow history of the site during the period of measurement, in comparison with the probable future flow history during the life span of the highway crossing. The duration of floods, or of flows near bankfull stages, is probably more important than the magnitude of floods; and (2) man-induced factors that are likely to affect bank erosion rates. Among the most important of these are urbanization and the clearing of flood plain forests.
- **Behavior of Meander Loops**. If the proposed bridge or roadway is located near a meander loop, it is useful to have some insight into the probable way in which the loop will migrate or develop, as well as its rate of growth. No two meanders will behave in exactly the same way, but the meanders on a particular stream reach tend to conform to one of the several modes of behavior illustrated in Figure 4.8.4.

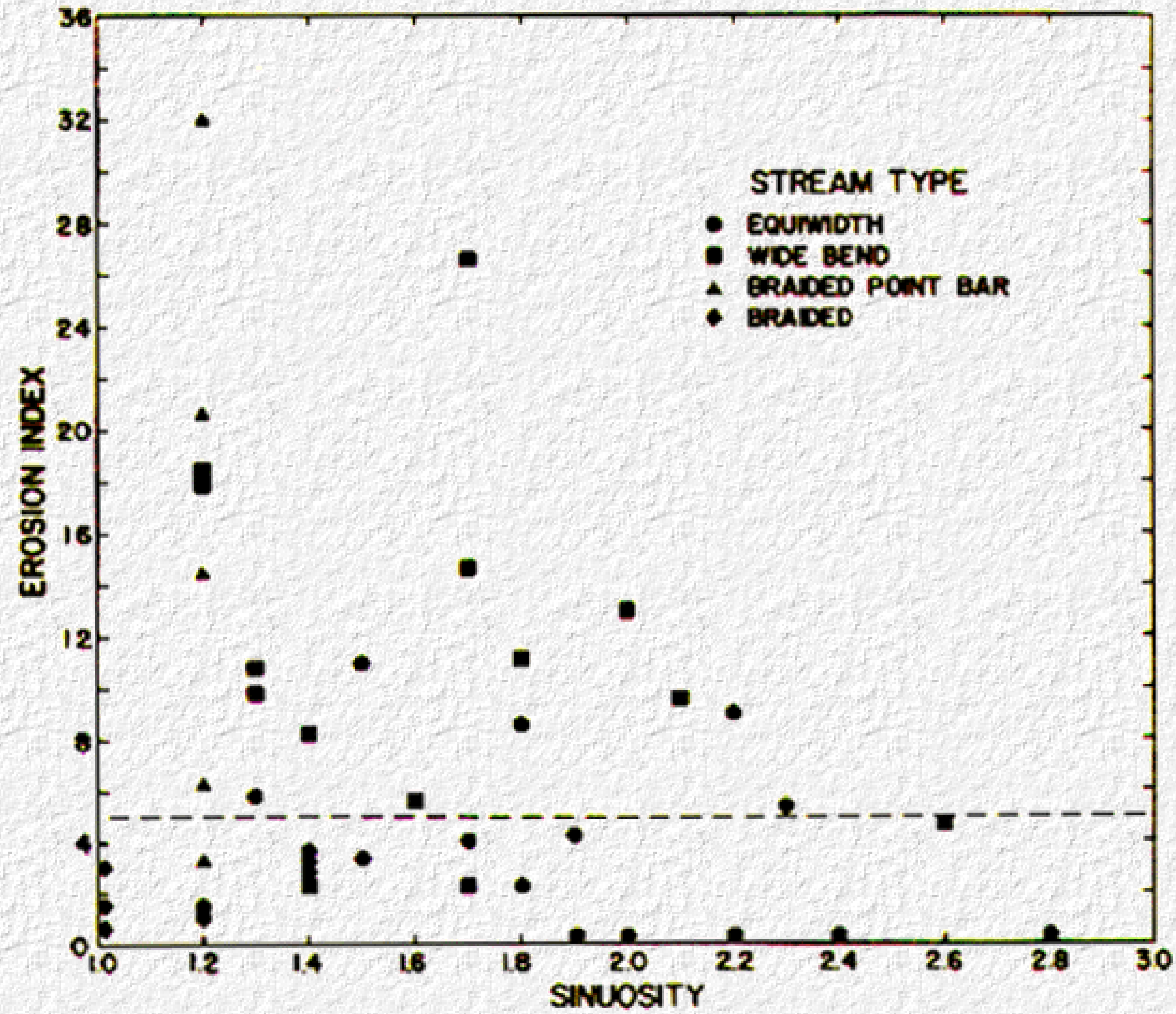


Figure 4.8.3. Erosion Index in Relation to Sinuosity (Ref. Brice, 1984)

Mode A (Figure 4.8.4) represents the typical development of a loop of low amplitude, which decreases in radius as it extends slightly in a downstream direction. Mode B rarely occurs unless meanders are confined by valley sides on a narrow flood plain, or are confined by artificial levees. Well developed meanders on streams that have moderately unstable banks are likely to follow Mode C. Mode D applies mainly to large loops on meandering or highly meandering streams. The meander has become too large in relation to stream size and flow, and secondary meanders develop along it, converting it to a compound loop. Mode E also applies to meandering or highly meandering streams, usually of the equiwidth point-bar type. The banks have been sufficiently stable for an elongated loop to form (without being cut off), but the neck of the loop is gradually being closed and cutoff will eventually occur at the neck. Modes F and G apply mainly to locally braided sinuous or meandering streams having unstable banks. Loops are cut off by chutes that break diagonally or directly across the neck.

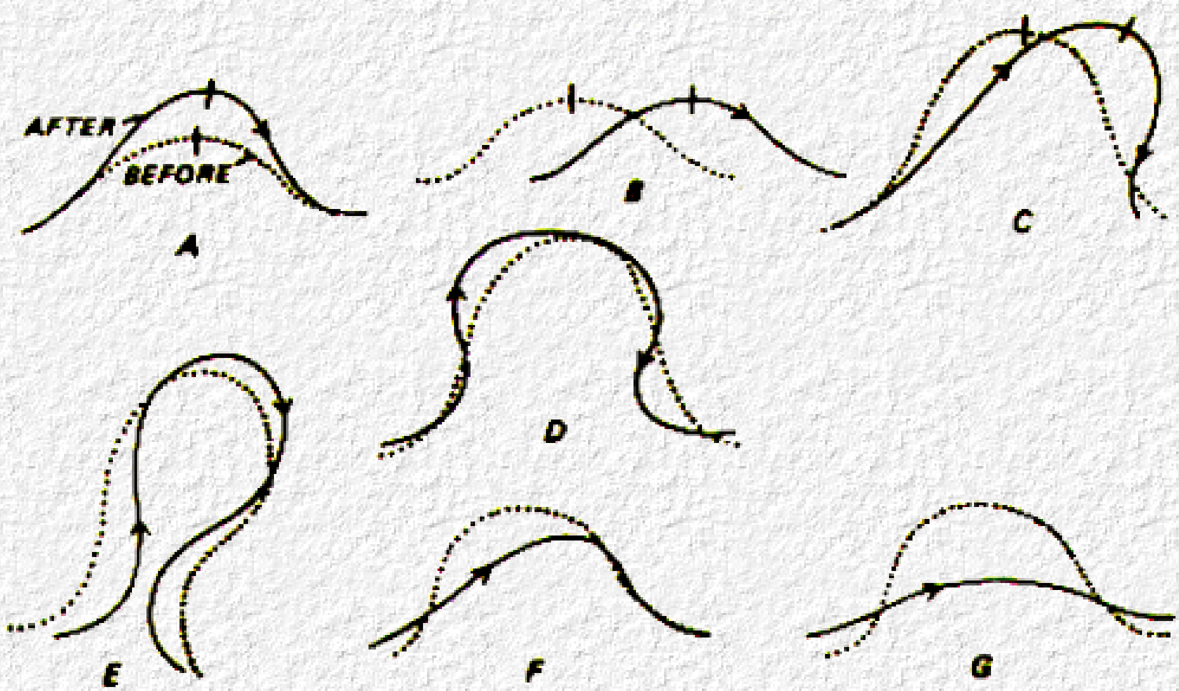


Figure 4.8.4. Modes of Meander Loop Development. A, Extension. B, Translation. C, Rotation. D, Conversion to a Compound Loop. E, Neck cutoff by Closure. F, Diagonal Cutoff by Chute. G, Neck Cutoff by Chute

- **Effects of Meander Cutoff** - If cutoffs seem imminent at any meanders in the vicinity of a proposed bridge crossing the probable effects of cutoff need to be considered. The local increase in channel slope due to cutoff usually results in an increase in the growth rate of adjoining meanders, and an increase in channel width at the point of cutoff. On a typical wide-bend point-bar stream the effects of cutoff do not extend very far upstream or downstream.
- **Assessment of Degradation** - Field sites having degradation problems are more numerous than sites having aggradation problems. Annual rates of degradation averaged from past records such as the closure of a dam give poor estimates of future rates of degradation. Typical situations exhibit an exponential decay function of the rate of channel degradation.

Recent evidence of degradation can be detected from field surveys or by stereo viewing of aerial photographs. Indicators of degradation are listed below in approximate order of reliability: (1) channel scarps, headcuts and nickpoints; (2) gullying of minor side tributaries; (3) high and steep unvegetated banks; (4) measurements of streambed elevation from a bridge deck; (5) changes in stream discharge relationships; and (6) measurements of longitudinal profiles.

- **Assessments of Scour and Fill** - Natural scour and fill refer to fluctuations of streambed elevation about an equilibrium condition. These fluctuations are associated mainly with floods and occur by three different mechanisms operating jointly or independently: (1) bed form migration; (2) convergence and divergence of flow; and (3) lateral shift of thalweg or braids.

  The maximum scour induced by the migration of a dune is almost one-half dune height, and dune heights are roughly estimated as one-third of the mean flow depth. In gravel bed streams, most migrating bed forms can be regarded as bars, the height of which is related to flow depth. The migration of a bar through a bridge waterway is mainly of concern because of the deflection and concentration of flow. Bar migration tends to be a random process and its motion can best be tracked from time-sequential aerial photographs.

  Gravel bars tend to migrate on braided streams and to remain fixed at riffles on unbraided pool and riffle streams.

  Flow convergence in natural streams is associated with scour whereas divergent currents are associated with deposition. Persistent pools have the strongest convergence of flow and the greatest potential for scour. Such pools are best identified by a continuous bed profile along the thalweg. In braided streams, scour holes are found at the confluence of braids. Field measurement of cross-sectional area and flow velocity at an incised reach near bankfull stage provides a good basis for calculation of scour by extrapolation to the design flood.

  Instability of the streambed that results from shift of thalweg is related to stream type and can be assessed from study of aerial photographs. On sinuous canal form streams, shift of the thalweg during flood is minimal. A greater shift of the thalweg can be expected on sinuous point-bar streams. In straight reaches, alternate bars visible on aerial photographs taken at low stage are commonly present. These alternate bars indicate the potential for thalweg shifting and also for bank erosion when the current is deflected against the bankline. Shift of the thalweg with increase in stage indicates the location of the point of maximum bed scour and bank erosion and also the alignment of piers with flood flow.

- **Site Selection for Highway Crossings** - For most streams the magnitude of scour is substantially greater at some place along the channel than at others. Bends and narrow sections may scour at high stages regardless of the effect of bridge structures. Straight or gently curved reaches with stable banks are preferred.

  Considerations for the selection of a crossing site on a non-sinuous reach include: (1) is the site at a pool, riffle or transition section; (2) are alternate bars visible at low stage; and (3) what is the effect of migration of mid-channel bars, if any? With respect to meandering reaches, questions seeking solution include: (1) what has been the rate and mode of migration of the meander; (2) what is the probable future behavior, as based on the past; (3) is the site at a pool, riffle or transition section; and (4) is meander cutoff probable?

Go to Chapter 5 (Part I)

Chapter 5 : HIRE
River Stabilization, Bank Protection and Scour Part I

Go to Chapter 5, Part II

From a study of the chapter on river morphology and river response, it should be clear that both short-term and long-term changes can be expected on river systems as a result of natural and man-made influences. Recommended structures and design methods for river control are presented in this chapter. The integrated and interactive effects of these structures with the river are discussed in Chapter 7.

Numerous types of river control and bank stabilization devices have evolved through past experience. Concrete, brick, willow and asphalt mattresses, sacked concrete and sand, riprap grouted slope protection, sheet piles, timber piles, steel jack and brush jetties, angled and sloped rock-filled, earth-filled, and timber dikes, automobile bodies, and concrete tetrahedrons have all been used in the practice of training rivers and stabilizing river banks. An early treatise on the subject of bank and shore protection was prepared by the California Division of Highways (1959). A large number of publications on river training and stabilization have been prepared by the Corps of Engineers and the U.S. Bureau of Reclamation. Many more publications on the subject exist in the open literature. It is not intended that an exhaustive coverage of the various types of river control structures and methods of design be made in this manual; rather, the purpose of this manual is to recommend methods and devices which provide useful alternatives to the highway engineer for the majority of circumstances which are likely to be encountered in highway practice. A treatise of great interest in relation to highway crossings is the Report FHWA-RD-78-162 and 163 on countermeasures for hydraulic problems at bridges by Brice, Blodgett and others. The interested reader is referred to these two volumes for an analysis and assessment (Vol. 1) and 283 case histories (Vol. 2).

Generally, changes to river alignment, river cross section, training, and bank stabilization of rivers associated with highway projects are confined to short reaches of the river. While the methods for river training and bank stabilization discussed herein are applicable to short and long reaches of the river, they are not a panacea to all problems associated with highway encroachments on rivers. Handbook analyses and designs usually lead to poor solutions of specific problems. Also, the solution to a particular problem may generate problems elsewhere in the river system.

5.1 Stream Bank Erosion

Processes of bank erosion are of primary importance in the context of process/response systems involving flowing water as it interacts with bed and bank material.

Changes in channel geometry with time are particularly significant during periods when alluvial channels are subjected to high flows. The converse situation exists during relatively dry periods. Erosive forces during high flow periods may have a capacity approximately 100 times greater than those forces acting during periods of intermediate and low flow. In most instances when considering the instability of alluvial rivers, it can be shown that approximately 90 percent of all river changes occur during the small percentage of the time when the discharge exceeds the dominant discharge.

Regardless of the fact that the majority of bank changes occur during comparatively short time periods, there may also be regions within a river in which some degree of instability is exhibited for all flow conditions. Raw banks may develop on the outside of bends as a consequence of direct impingement of the flowing water. Sloughing banks may occur as a result of seepage and other secondary forces created by water draining back through the banks into the river. Continuous wave action, generated either naturally or by man's activities, may also precipitate erosion problems.

5.1.1 Causes of Streambank Failure

A summary of the variables and factors affecting erosion of river banks is outlined below.

I. Hydraulic Parameters
- A. Fluid Properties
    - 1. Specific Weight
    - 2. Temperature/Viscosity
- B. Flow Characteristics
    - 1. Discharge
    - 2. Duration
    - 3. Frequency
    - 4. Velocity
    - 5. Velocity Distribution
    - 6. Turbulence
    - 7. Shear Stress
    - 8. Drag Force
    - 9. Lift Force
    - 10. Momentum

II. Characteristics of Bed and Bank Material
- A. Size
- B. Gradation
- C. Shape
- D. Specific Weight

III. Characteristics of the Banks
- A. Noncohesive
- B. Cohesive
- C. Stratified
- D. Rock
- E. Height

IV. Subsurface Flows
- A. Wave Forces
- B. Seepage Forces
- C. Piping

V. Wind Waves and Boat Waves
- A. Wave Forces
- B. Surface Erosion
- C. Piping

VI. Climatic Factors
- A. Freezing

        1. Ice Thickness
        2. Duration
        3. Frequency
    B. Thawing
    C. Permafrost
VII. Biological Factors
    A. Vegetation
        1. Trees
        2. Shrubs
        3. Grass
    B. Animal Life
VIII. Man-induced Factors
    A. Pool Fluctuations Caused by Power Generation
    B. Agricultural Activities
    C. Mining
    D. Transportation
    E. Urbanization
    F. Drainage
    G. Floodplain Development
    H. Recreational Boating
    I. Commerical Ship-Traffic

Due to the complex nature of erosional processes and interaction of the variables and forces that cause erosion, the mechanics of erosional patterns of channels and banks are inadequately understood at present. A better understanding of these erosional processes can only be made through a detailed evaluation of adequate data.

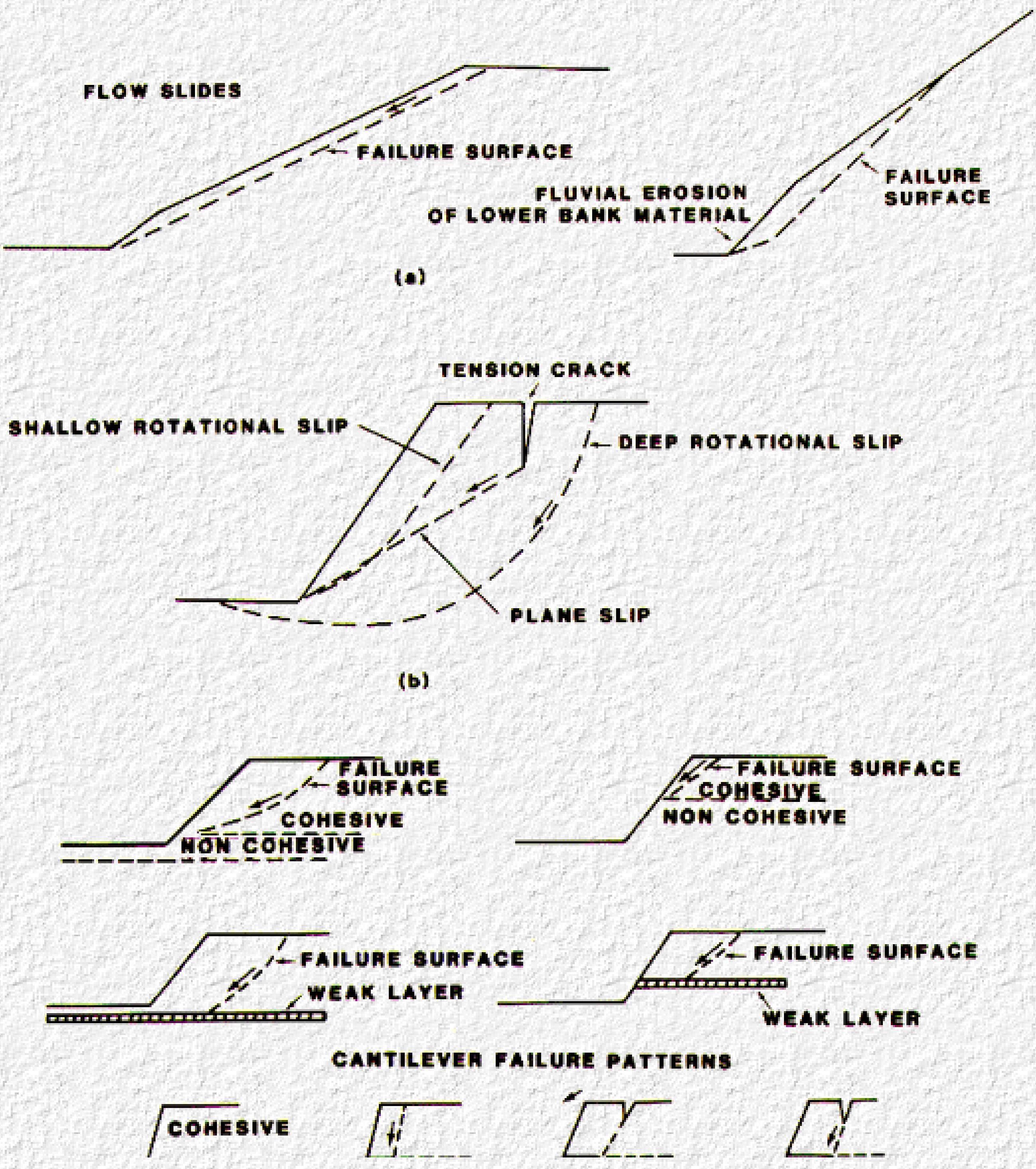
A comprehensive literature review by Simons et al. (1979a) indicates that little progress has been made in quantifying specific causes of bank erosion, either man-made or naturally induced. A recent analysis by Ghaboosi (1985) revealed that the erosive factors contribute to streambank erosion if these factors increase the main flow shear stress or if they decrease the bank stability or both. According to this assumption, the magnitude of the erosive factors is expressible partly in terms of shear stress and partly in terms of bank stability. By developing a method to convert the reduced rate of the bank stability into equivalent shear stress, the erosional action of the erosive factors could be related directly and indirectly to the shear stress.

5.1.2 Bed and Bank Material

Resistance of a river bank to erosion is closely related to several characteristics of the bank material. Bank material deposited in the river can be broadly classified as cohesive, noncohesive, and composite. Failure of banks for various situations is shown in Figure 5.1.1.

- **Cohesive material** is more resistant to surface erosion and has low permeability which reduces the effects of seepage, piping, frost heaving, and subsurface flow on the stability of the banks. However, such banks when undercut and/or saturated are more likely to fail due to mass wasting processes such as sliding.

- **Noncohesive bank material** tends to be removed grain by grain from the bank line. The rate of particle removal, and hence the rate of bank erosion, is affected by factors such as the direction and magnitude of the velocity adjacent to the bank, the turbulent fluctuations, the magnitude and fluctuations in the shear stress exerted on the banks, seepage force, piping and wave forces, many of which may act concurrently.
- **Composite or stratified banks** are very common on alluvial rivers and generally are the product of past transport and deposition of sediment by the river. More specifically, these types of banks consist of layers of materials of various sizes, permeability, and cohesion. The layers of noncohesive material are subject to surface erosion, but may be partly protected by adjacent layers of cohesive material. This type of bank is also vulnerable to erosion and sliding as a consequence of subsurface flows and piping.



(c)

Figure 5.1.1. Typical Bank Failure Surfaces (A) Noncohesive, (B) Cohesive, (C) Composite (after Brown, 1985)

5.1.3 Subsurface Flow

With flow of water from the river into the adjacent banks, a stabilizing seepage force is generated. Rivers that continuously seep water into the banks tend to have smaller widths and larger depths for a particular discharge. The reverse is true of the rivers that continuously gain water by an inflow through their banks. The inflowing water creates a seepage force that makes the banks less stable. Forces that cause the movement of water through the bank material are caused by various factors.

If the water table is higher than river stage, flow will be from the banks into the river. The high water table may result from: (1) a wet period during which water draining from tributary watersheds saturates the floodplain to a higher level; (2) poor drainage conditions resulting from deterioration or failure of surface drainage systems; (3) increased infiltration resulting from changes in land use causing an increase in water level; (4) irrigated flood plains; and (5) development of the adjacent floodplain for homes and businesses that utilize septic tanks and leach fields to dispose of waste water and sewage.

With a rise in river stage an outward gradient is developed that induces flow into the banks. This can be caused by: (1) the storage and release of water for pumped storage hydropower generation which causes numerous fluctuations in river stage; (2) boat and wind waves which cause local variations in stage that introduce inflow and outflow of water from the banks; (3) predominately dry and semi-arid channels subject to intermittant floods. However, because the duration of the change in stage is small, the inflow and outflow phenomena are usually concentrated locally in the surface of the banks; and (3) the formation and loss of backwater caused by ice flows and ice jams which lead to both seepage into and out of the banks. Frequent stage fluxuations, such as may occur with hydropower operations, may exacerbate the bank erosion provess.

The presence of water in the banks of rivers and its movement toward or away from the river affect bank stability and bank erosion in various ways. The related erosion of banks is a consequence of seepage forces, piping, and mass wasting.

5.1.4 Piping of River Banks

Piping is another phenomenon common to the alluvial banks of rivers. With stratified banks, i.e., lenses of sand and coarser material sandwiched between a layer of finer cohesive materials, flow is induced in more permeable layers by changes in river stage and by wind-and boat-generated waves. If the flow through the permeable lenses is capable of dislodging and transporting particles from the permeable lenses, the material is slowly removed, undermining portions of the bank. Without this foundation material to support the overlying layers, a block of bank material drops down and results in the development of tension cracks sketched in Figure 5.1.1.c. These cracks allow surface flows to enter, further reducing the stability of the affected block of bank material. Bank erosion may continue on a grain-by-grain basis or the block of bank material may ultimately slide downward and outward into the channel, causing bank failure as a result of a combination of seepage forces, piping, and mass wasting.

5.1.5 Mass Wasting

An alternative form of bank erosion is caused by local mass wasting. If the bank becomes saturated and possibly undercut by flowing water, blocks of the bank may slump or slide into the channel. Mass wasting may be further aggravated by construction of homes on river banks, operation of equipment on the floodplain adjacent to the banks, added gravitational force resulting from tree growth, location of roads that cause unfavorable drainage conditions, saturation of banks by leach fields from septic tanks, and increased infiltration of water into the floodplain as a result of changing land-use practices.

Landslides, the downslope movement of earth and organic materials, result from an imbalance of forces. Various forces are involved in mass wasting. These forces are associated with the downslope gravity component of the slope mass. Resisting these downslope forces are the shear strength of the earth's materials and any additional contributions from vegetation via root strength or man's slope reinforcement activities. When a slope is acted upon by a stream or river, an additional set of forces is added. These forces are associated with removal of material from the toe of the slope, fluctuations in groundwater levels, and vibration of the slope. A slope may fail if stable material is removed from the toe. When the toe of a slope is removed, the slope loses more resistance by buttressing than it does by downslope gravitational forces. The slope materials may then tend to move downward into the void in order to establish a new balance of forces or equilibrium. Oftentimes, this equilibrium is a slope configuration with less than original surface gradient. The toe of the failed mass can provide a new buttress against further movements. However, if this buttress is removed by stream erosion, the force equilibrium may again be upset. For slope toes acted upon by erosive stream water, the continual removal of toe material can upset the force balance.

5.1.6 River Training and Stabilization

Various devices and structures have been developed to control river flow along a preselected path and to stabilize the banks. Most have been developed through trial and error applications, aided in some instances by hydraulic model studies. Specific functions of bank protection and training works in relation to bridges and their approaches include: (1) stabilize eroding river banks and channel location in the case of shifting streams; (2) economize on bridge lengths by constricting the natural waterway; (3) direct flow parallel to piers and thereby minimize local scour; (4) improve the hydraulic efficiency of a waterway opening, thereby reducing afflux and scour and facilitating passage of ice and debris; (5) protect road approaches from stream attack and prevent meanders from folding onto the approaches; (6) permit construction of a square bridge crossing by diverting the channel from a skewed alignment; (7) reduce the overall cost of a road project by diverting the channel away from the base of a valley slope, thereby allowing a reduction in bridge length and height; (8) secure existing works, or to repair damage and improve initial designs; and (9) protect longitudinal encroachments.

A comprehensive bank stabilization and channel rectification program to control a river reach completely normally requires extensive work on concave banks in bends, minor work on convex bars, and control work on both banks through crossings.

To minimize attack by the stream on stabilization and rectification structures, the river is shaped to an alignment consisting of a series of easy bends, with the flow directed from one bend into the next bend downstream in such a way as to maintain a direction essentially parallel to the channel control line. Straight reaches and reaches of very small curvature should be avoided, insofar as practicable, because there is a tendency for flows to shift from side to side in such reaches. The optimum bend radius approximates that of relatively stable bends in the general river reach.

- **Fixed Points** - One of the essential requirements in designing a system of stabilization works is that construction starts at a stable, fixed point on the bank and continues downstream to another stable location or to some point below which the river can safely be left uncontrolled. Construction of relatively short isolated stabilization work has often proved unsuccessful because eventual changes in the direction of flow inherent in bank caving in the upstream uncontrolled reach either will set up a direct attack against the isolated protective work and severely damage or destroy it or will shift the attack to some other nearby reach of bank, requiring additional work and possible abandonment of the original work.

  Revetments should be constructed on a smooth alignment, with no irregularities, in order to avoid eddies set up by such disturbances to the flow that can lead to local scour and subsequent undermining of the revetment.
- **Radius of Curvature** - The most appropriate radius of curvature for rectification and stabilization varies from river to river and from reach to reach for a given river. It must be determined on the basis of relatively stable natural bends for each stream.

  The shorter the radius of curvature of a bend, the deeper the channel will be adjacent to the concave bank. The deeper the channel is, the greater the possibility of undermining bank protection work in the bend and the greater the cost of maintaining the structure. Therefore, sharp curvature of bends should be avoided to obtain the most economical control of the river.

  Bank stabilization and channel rectification works are of two broad types generally defined as follows:

Revetments are structures parallel to the current and are used for such purposes as stabilizing concave banks of bends. (Section 5.2 deals with riprap design for revetments and Section 5.3 describes design for types other than riprap).

Flow Control Structures which are used for:

- Directing flow from one bend into the next bend downstream
- Flair out sharp bends to a larger radius of curvature to provide a more desirable channel alignment
- Close off secondary channels and old bendways
- Concentrate flow on a limited width within a wider channel.

Rock riprap is probably the most widely used material to stabilize river banks and protect the side slopes of embankments. In the final report to Congress, the Corps of Engineers (1981) concluded that rock will likely continue to be the first choice of bank protection materials where material of sufficient size is available and affordable, because of durability, and other advantages. Because of its wide use and importance in highway practice, a separate section (Section 5.2) is devoted to design and analysis of rock riprap.

5.2 Riprap Size and Stability Analysis

When available in sufficient size, rock riprap is usually the most economical material for bank protection. Rock riprap has many other advantages over other types of protection. A riprap blanket is flexible and is neither impaired nor weakened by slight movement of the bank resulting from settlement or other minor adjustments. Local damage or loss is easily repaired by the placement of more rock. Construction is not complicated so special equipment or construction practice is not necessary. Riprap is usually durable and recoverable and may be stockpiled for future use. The cost-effectiveness of locally available riprap provides a viable alternative to many other types of bank protection. Riprap stability increases with increasing thickness as more material is available to move to damaged areas and more energy is dissipated before it reaches the filter and streambank. Although the riprap must be placed to the proper level below the bed, there are no special foundation requirements. The appearance of rock riprap is natural and after a period of time vegetation will grow between the rocks. Wave runnup on rock slopes is usually less than on other types. Finally, when the usefulness of the protection is finished, the rock is salvagable.

The important factors to be considered in designing rock riprap blanket protection are:

- The durability of the rock.
- The density of the rock.
- The velocity (both magnitude and direction) of the flow in the vicinity of the rock.
- The slope of the bed and bankline being protected.
- The angle of repose for the rock.
- The shape and angularity of the rock.
- What shape and weight of stones will be stable in the streamflow?
- What blanket thickness is required?
- Is a filter (see Glossary) needed between the bank and the blanket to allow seepage but to prevent erosion of bank soil through the blanket?
- How will the blanket be stabilized at the toe of the bank?
- How will the blanket be tied into the bank at its upstream and downstream ends?

5.2.1 Stability Factors for Riprap

In the absence of waves and seepage, the stability of rock riprap particles on a side slope is a function of: (1) the magnitude and direction of the stream velocity in the vicinity of the particles; (2) the angle of the side slope; and (3) the characteristics of the rock including the geometry, angularity and density. The functional relations between the variables is developed below. This development closely follows that given by Stevens and Simons

(1971)

Consider flow along an embankment as shown in Figure 5.2.1. The fluid forces on a rock particle identified as P in Figure 5.2.1a result primarily from fluid pressure around the surface of the particles. The lift force F_l is defined herein as the fluid force normal to the plane of the embankment. The lift force is zero when the fluid velocity is zero. The drag force F_d is defined as the fluid force acting on the particle in the direction of the velocity field in the vicinity of the particle. The drag force is normal to the lift force and is zero when the fluid velocity is zero. The remaining force is the submerged weight of the rock particle W_s.

Rock particles on side slopes tend to roll rather than slide, so it is appropriate to consider the stability of rock particles in terms of moments about the point of rotation. In Figure 5.2.1b the direction of movement is defined by the vector $\bar{R}$. The point of contact about which rotation in the $\bar{R}$ direction occurs is identified as point "0" in Figure 5.2.1c.

The forces acting in the plane of the side slope are F_d and $W_s\sin\theta$ as shown in Figure 5.2.1b. The angle θ is the side slope angle. The lift force acts normal to the side slope and the component of submerged weight $W_s\sin\theta$ acts normal to the side slope as shown in Figure 5.2.1.c.

At incipient motion, there is a balance of moments about the point of rotation such that

$$e_2 W_s \cos\theta = e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_l \quad (5.2.1)$$

The moment arms e_1, e_2, e_3 and e_4 are defined in Figure 5.2.1c and the angles δ and β are defined in Figure 5.2.1b.

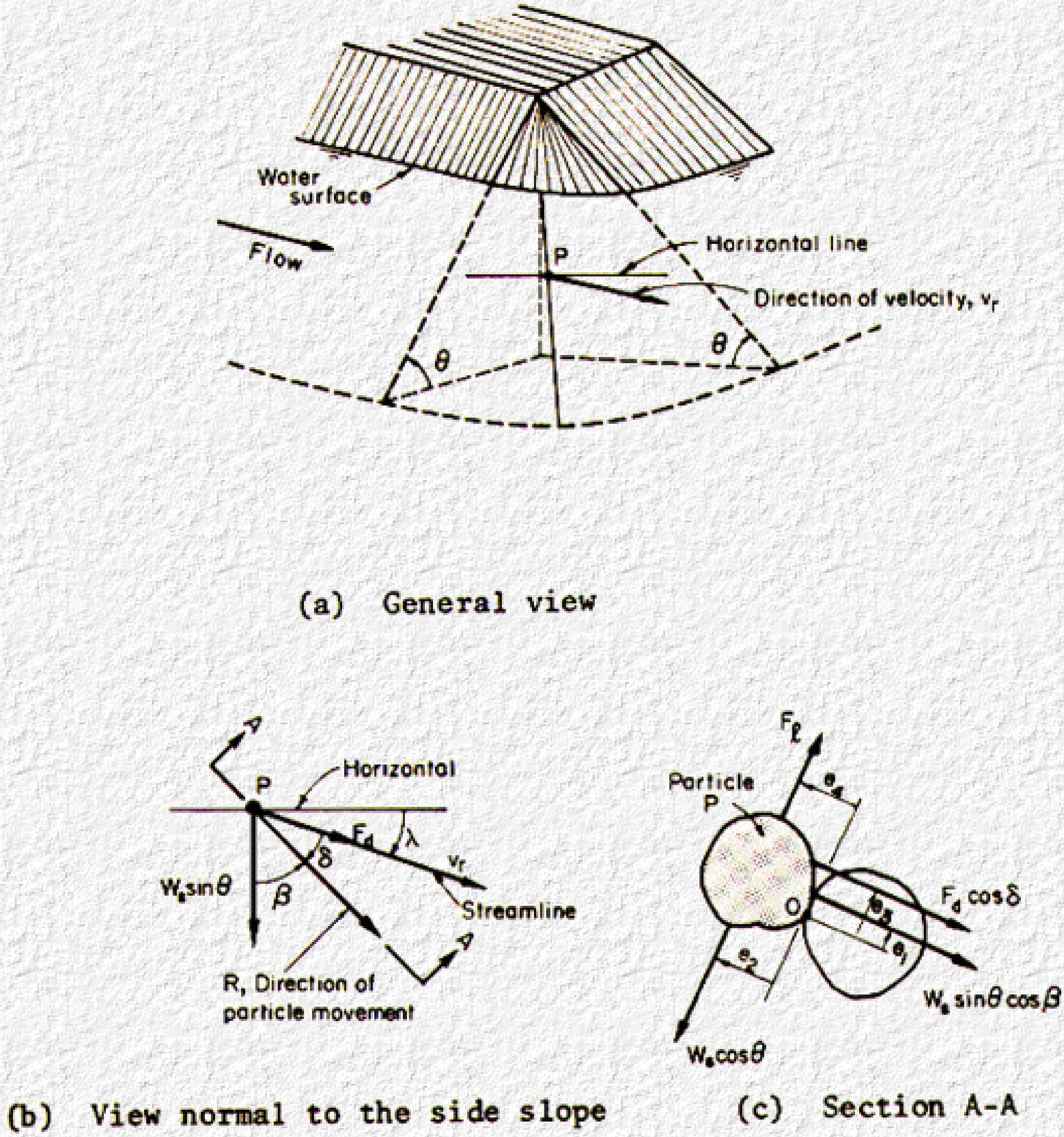


Figure 5.2.1. Diagram for Riprap Stability Conditions

The stability factor S.F. against rotation of the particle is defined as the ratio of the moments resisting particle rotation out of the bank to the submerged weight and fluid force moments tending to rotate the particle out of its resting position. Accordingly,

$$S.F. = \frac{e_2 W_s \cos\theta}{e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell} \qquad (5.2.2)$$

The following particle stability analysis was first derived by Stevens (1968). The analysis is also presented in Simons and Senturk (1977). This analysis shows that the stability factor for rock riprap on side slopes where the flow has a non-horizontal velocity vector is related to properties of the rock, side slope and flow by the following equations:

$$S.F. = \frac{\cos\theta \tan\phi}{\eta' \tan\phi + \sin\theta \cos\beta} \qquad (5.2.3)$$

in which

$$\beta = \tan^{-1}\left\{\frac{\cos\lambda}{\frac{2\sin\theta}{\eta\tan\phi} + \sin\lambda}\right\} \qquad (5.2.4)$$

$$\eta = \frac{21\,\tau_o}{(S_s - 1)\gamma D_s} \qquad (5.2.5)$$

and

$$\eta' = \eta\left\{\frac{1 + \sin(\lambda + \beta)}{2}\right\} \qquad (5.2.6)$$

Given a rock size D_s, of specific weight S_s and angle of repose ϕ and given a velocity field at an angle λ to the horizontal producing a tractive force τ_o on the side slope of angle θ, the set of 4 equations (Equation 5.2.3, Equation 5.2.4, Equation 5.2.5, and Equation 5.2.6) can be solved to obtain the stability factor S.F. If S.F. is greater than unity, the riprap is stable; if S.F. is unity, the rock is at the condition of incipient motion; if S.F. is less than unity, the riprap is unstable. Problem 5.1 in Appendix 5 illustrates how to determine the stability of riprap.

5.2.2 Simplified Design Aid for Side Slope Riprap

When the velocity along a side slope has no downslope component (i.e., the velocity factor is along the horizontal), some simple design aids can be developed.

For horizontal flow along a side slope, the equations relating the stability factor, the stability number, the side slope angle, and the angle of repose for the rock are obtained from Equation 5.2.4 and Equation 5.2.6 with $\lambda = 0$.

$$\beta = \tan^{-1}\left(\frac{\eta \tan\phi}{2\sin\theta}\right) \qquad (5.2.7)$$

and

$$\eta' = \eta\left(\frac{1+\sin\beta}{2}\right) \qquad (5.2.8)$$

When Equation 5.2.7 and Equation 5.2.8 are substituted into Equation 5.2.3, the expression for the stability factor for horizontal flow on a side slope is

$$S.F. = \frac{S_m}{2}\left\{\sqrt{\zeta^2 + 4} - \zeta\right\} \qquad (5.2.9)$$

in which

$$\zeta = S_m\, \eta\, \sec\theta \qquad (5.2.10)$$

and

$$S_m = \frac{\tan\phi}{\tan\theta} \qquad (5.2.11)$$

If we solve Equation 5.2.9 and Equation 5.2.10 for h, then

$$\eta = \frac{S_m^2 - S.F.^2}{S.F.\, S_m^2}\cos\theta \qquad (5.2.12)$$

The interrelation of the variables in these two equations is represented in Figure 5.2.2. Here, the specific weight of the rock is taken as 2.65 and a stability factor of 1.5 is employed. This recommended stability factor for the design of riprap (S.F. = 1.5) is the result of studies of the riprap embankment model data obtained by Lewis. These studies were reported by Simons and Lewis (1971).

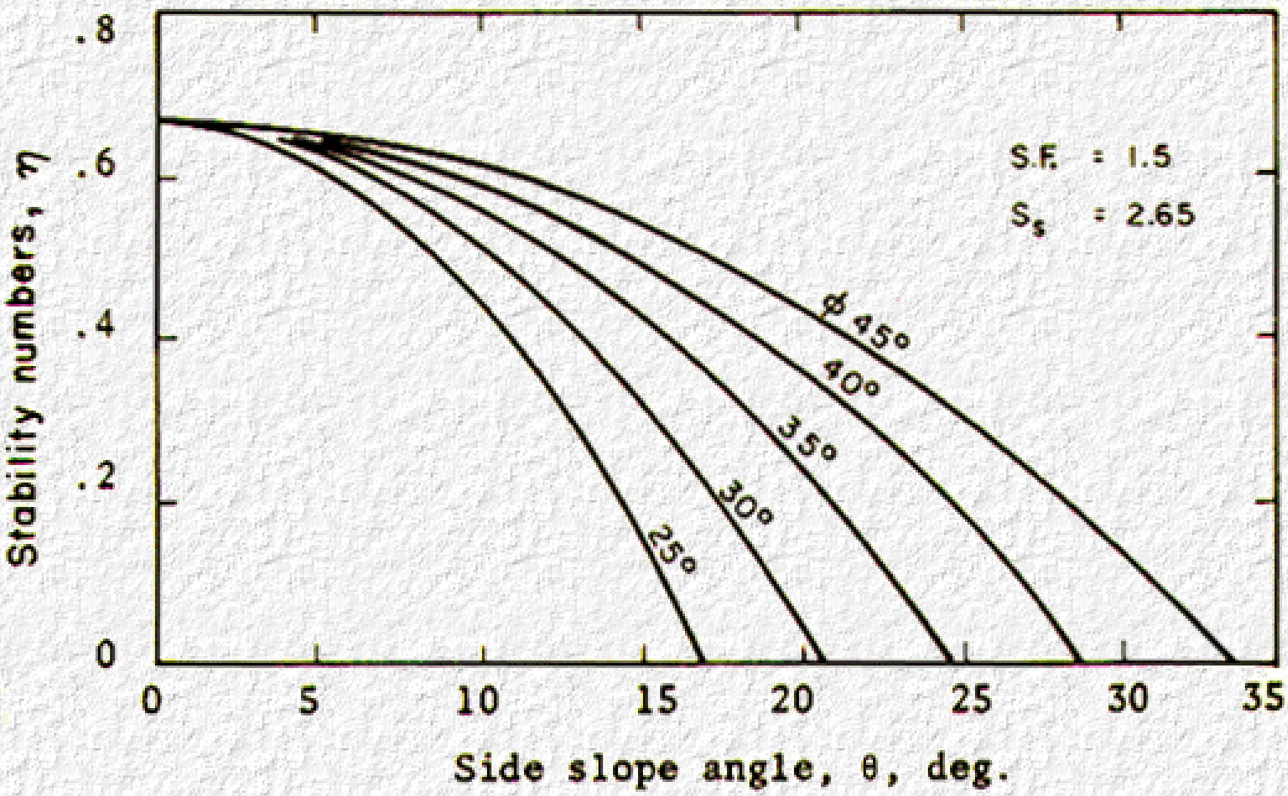


Figure 5.2.2. Stability Numbers for a 1.5 Stability Factor for Horizontal Flow along a Side Slope

The curves in Figure 5.2.2 are computed in the following manner: (1) Select an angle of repose ϕ. For example ϕ = 45°; (2) Select a side slope angle. For example r = 25°; (3) Compute S_m from Equation 5.2.11;

$$S_m = \frac{\tan 45^\circ}{\tan 25^\circ} = 2.14;$$

(4) Compute h from Equation 5.2.12 with S. F. = 1.5

$$\eta = \left(\frac{(2.14)^2 - (1.5)^2}{(1.5)(2.14)^2} \right) \cos 25^\circ = 0.31; \text{and}$$

(5) Repeat the above steps for the full range of interest for Φ and Θ.

If the shear stress τ_o on the side slope is known, then, from Equation 5.2.5, the riprap size required is obtained from

$$D_m = \frac{21\,\tau_o}{(S_s - 1)\gamma\,\eta} \qquad (5.2.13)$$

the stability number η being obtained from Figure 5.2.2. The Problem 5.2 in Appendix 5 illustrates how to use this method for riprap design or embankment slopes.

5.2.3 Velocity Method for Riprap Design

The size of stone needed to protect a streambank or highway embankment from erosion by a current moving parallel to the embankment can be determined by the use of Figure 5.2.3 and Figure 5.2.4. The diameter, D_{50} in feet, is that of a spherical stone that would have the same weight as the 50 percent size of stone. The size of stone is found by a trial-and-error procedure which consists of first estimating a stone size.

The mean velocity, V_m, of the stream during the design flood must then be converted to velocity against the stone, V_s, by that of Figure 5.2.3. The ratio (D_{50}/y_o) of the equivalent spherical diameter of the 50 percent stone size to the depth of flow during the design flood is computed by using 0.4 of the total depth when the depth of flow exceeds about 10 feet. The reason for this is that use of the total depth would result in a stone size which would be adequate at the total depth but which might be too light to provide protection near the water surface.

With the velocity against the stone enter Figure 5.2.4 and read the stone size for the embankment slope. This stone size is the 50 percent (median) size, by weight, of a well-graded mass of stone with a unit weight of 165 pounds per cubic foot. If the stone size agrees with the assumed stone size, this is the correct size. If not, the procedure is repeated until the assumed size is in reasonable agreement with the size from Figure 5.2.4.

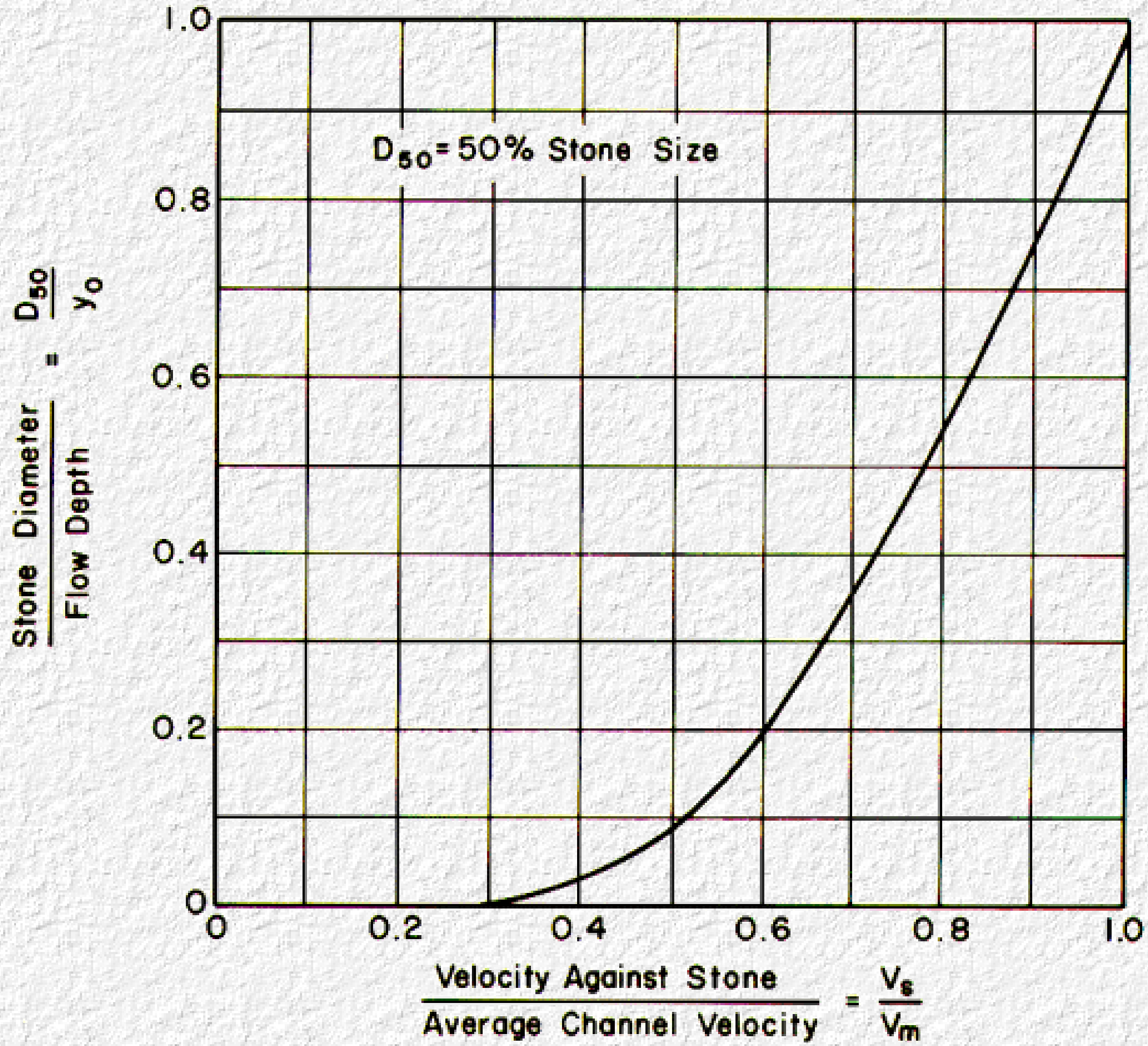
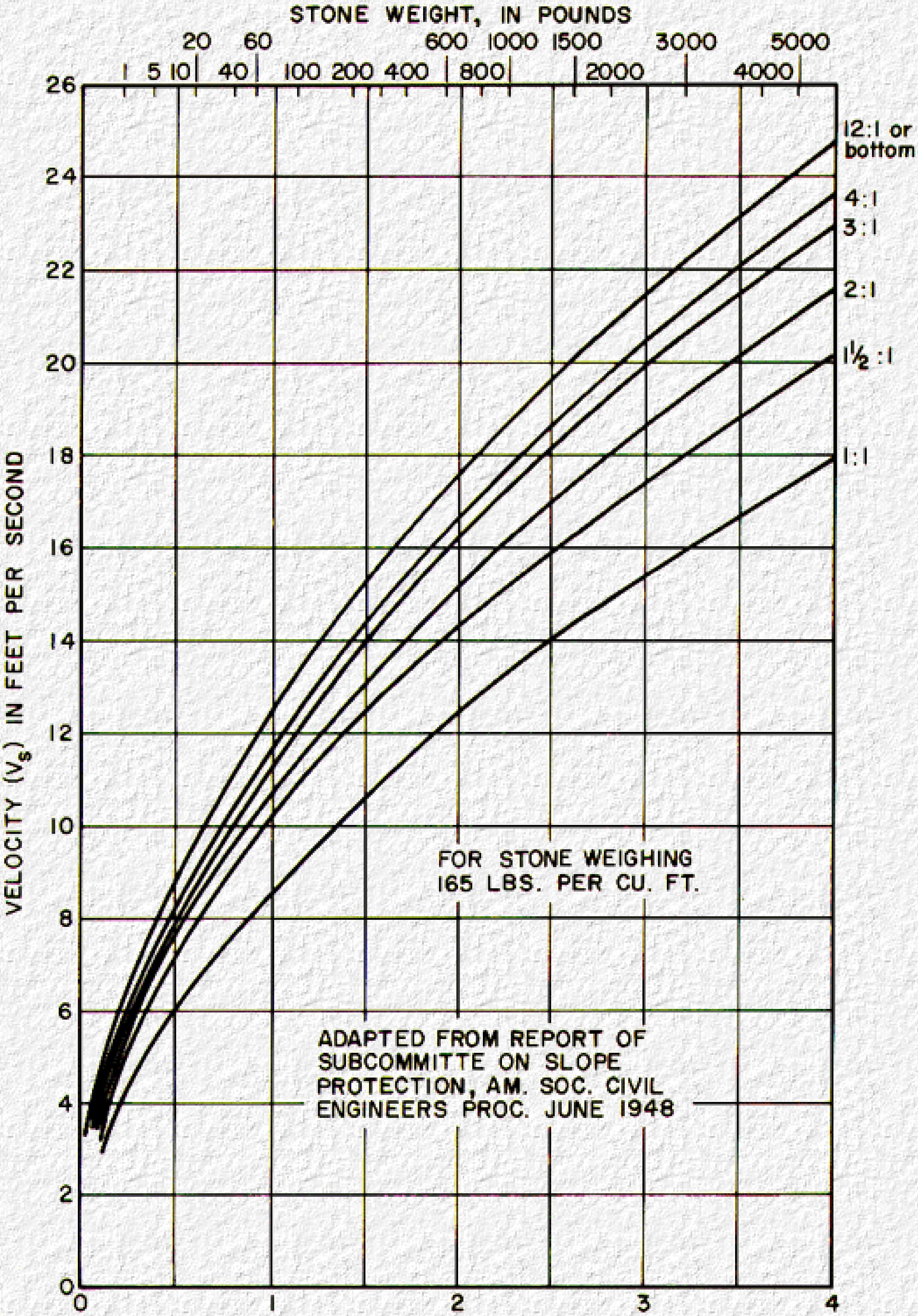


Figure 5.2.3. Velocity against Stone on Channel Bottom (from HEC 11, U.S. DOT, 1967)



STONE WEIGHT, IN POUNDS
1 5 10 20 40 60 100 200 400 600 800 1000 1500 2000 3000 4000 5000
VELOCITY (V_s) IN FEET PER SECOND
0 2 4 6 8 10 12 14 16 18 20 22 24 26
EQUIVALENT SPHERICAL DIAMETER OF STONE, IN FEET
0 1 2 3 4
12:1 or bottom
4:1
3:1
2:1
1½:1
1:1
FOR STONE WEIGHING 165 LBS. PER CU. FT.
ADAPTED FROM REPORT OF SUBCOMMITTE ON SLOPE PROTECTION, AM. SOC. CIVIL ENGINEERS PROC. JUNE 1948

Figure 5.2.4. Size of Stone That Will Resist Displacement for Various Velocities and Side Slopes (from HEC 11, 1967)

When the unit weight of the stone is other than 165 pounds per cubic foot, the size of the stone should be corrected as follows:

$$D'_{50} = \frac{102.5\, D_{50}}{\gamma_s - 62.5} \tag{5.2.14}$$

where

D_{50} = stone size for Figure 5.3.5
D'_{50} = stone size for stones of γ_s pounds per cubic feet

The size of stone required to resist displacement from direct impingement of the current as might occur with a sharp change in stream alignment is greater than the value obtained from Figure 5.2.4, although research data is lacking on just how much larger the stone should be. The California Division of Highways recommends doubling the velocity against the stone as determined for straight alignment before entering Figure 5.2.4 for stone size. Lane recommends reducing the allowable velocity by 22 percent for very sinuous channels; for determining stone size by Figure 5.2.4, the velocity against the stone (V_s) would be increased by 22 percent. Until data are available for determining the stone size at the point of impingement, a factor which would vary from 1 to 2 depending upon the severity of the attack by the current, should be applied to the velocity V_s before entering Figure 5.2.4.

Problem 5.3 in Appendix 5 illustrates how to determine stability factors for riprap from velocity.

In riprap design, it is often desirable to relate the tractive force acting on the riprapped bed or bank to the fluid velocity in the vicinity of the riprap. For fully turbulent flow, the reference velocity V_r at a distance D_{50} above the bed is determined from Equation 2.3.15

$$V_r = 2.5\, V_* \ell n\left(30.2 \frac{D_{50}}{D_{50}}\right) = 8.52\ V_* = 8.52\sqrt{\frac{\tau_o}{p}} \tag{5.2.15}$$

thus the relation between V_r and τ_o is

$$\tau_o = \frac{\rho V_r^2}{72} \tag{5.2.16}$$

The relation is strictly valid for uniform flow in wide prismatic channels in which the flow is fully turbulent. For the purpose of riprap design, Equation 5.2.16 can be used when the flow is accelerating such as on the tip of spur dikes or abutments. The equation should not be used in areas where the flow is decelerating or below energy dissipating structures. In these areas, the shear stress is larger than calculated for Equation 5.2.16.

One can also demonstrate (Richardson et al., 1975) that the reference velocity V_r is related to the velocity against the stone V_s ($V_r \simeq 1.4\ V_s$).

In summary the following expressions for h are equivalent:

$$\eta = \frac{21\,\tau_o}{(S_s - 1)\gamma D_s} \qquad (5.2.5)$$

$$\eta = \frac{0.30\,V_r^2}{(S_s - 1)gD_s} \qquad (5.2.17)$$

$$\eta = \frac{0.60\,V_s^2}{(S_s - 1)gD_s} \qquad (5.2.18a)$$

$$\eta = 0.30\left\{\frac{3.4}{\ln\left(12.3\,\frac{y_o}{D_s}\right)}\right\}^2 \frac{V^2}{(S_s - 1)gD_s} \qquad (5.2.18b)$$

5.2.4 Riprap Design on Abutments

When the drawdown through a bridge opening is large there is an appreciable downslope component of the velocity vector on the nose of an abutment or spur dike. This downslope component of velocity is illustrated in Figure 5.2.5. A model study of this phenomena was reported by Simons and Lewis (1971).

To apply the equations presented in Section 5.2.1 for the design of riprap bank protection, the flow parameters τ_o, V_r, V_s or V, and λ must be evaluated. Lewis (1972) developed an analytical model for determination of the complete velocity and depth distribution in the vicinity of abutments but the application of his method is complex. Lewis pointed out, based on the model and analytical studies, that the initial losses of the riprap protection occur at one or both of two zones on the embankment. One zone is near the flow separation point located approximately at midway around the upstream spill-slope. The other zone is along the abutment toe through the constriction. Riprap losses initially occurred on the upstream spill-slope for small constrictions ($\Delta h/L_a$ is small) and along the abutment toe for severe constrictions ($\Delta h/L_a$ is large) as shown in Figure 5.2.6. Here Δh is the drop in water surface elevation through the bridge opening and L_a is the horizontal distance shown in Figure 5.2.7 with the abutment angle ω_a, shown in Figure 5.2.7, equal to 45 degrees.

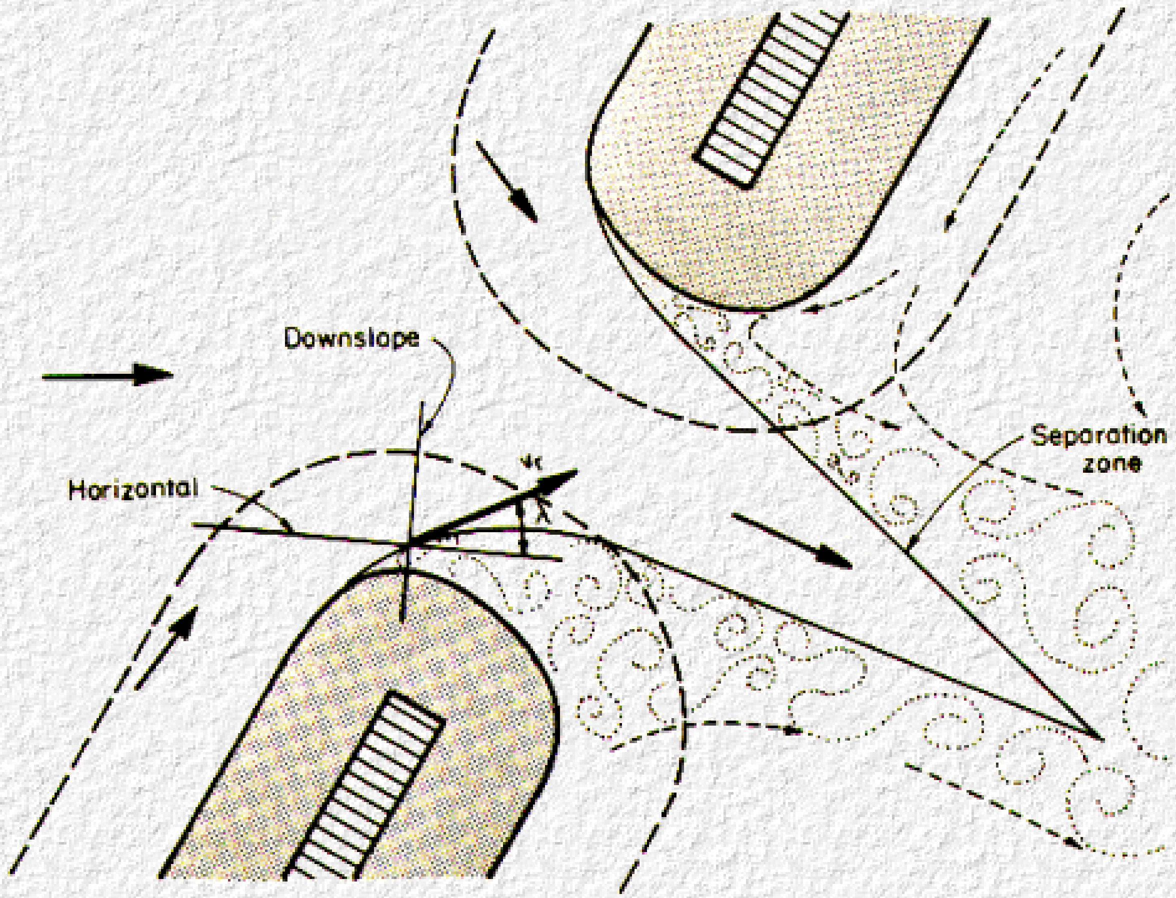


Figure 5.2.5. Flow around an Embankment End

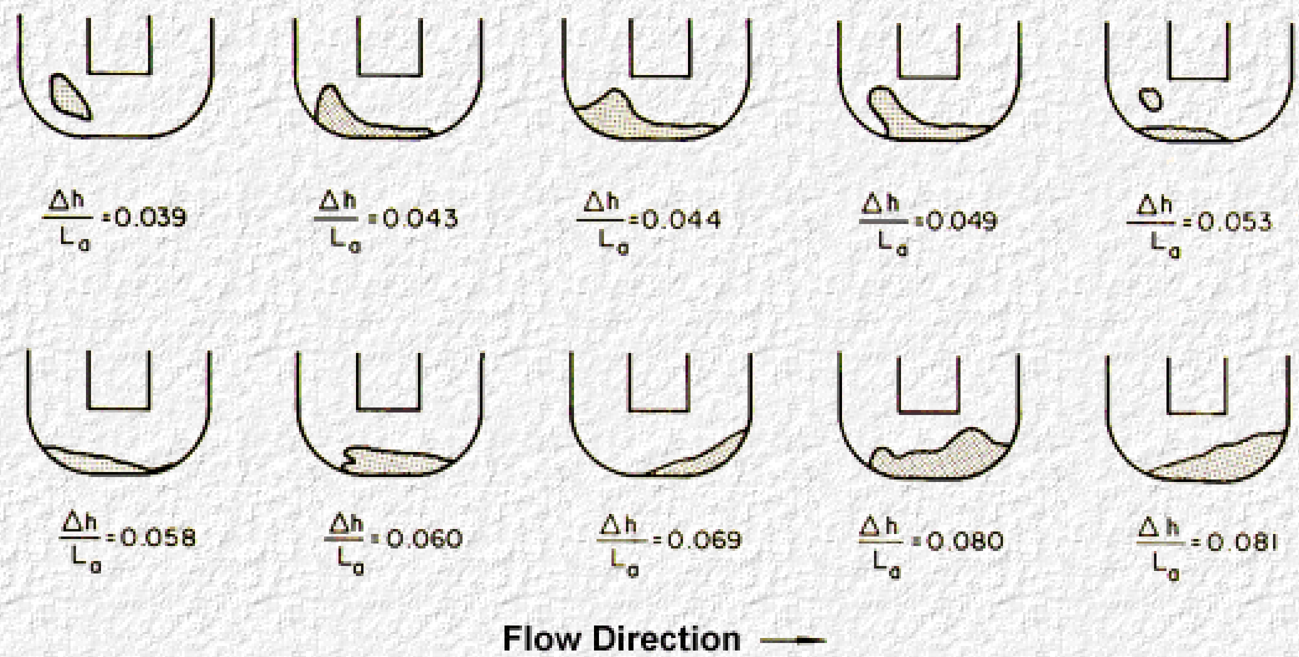


Figure 5.2.6. Zones of Failure on Riprapped Abutments

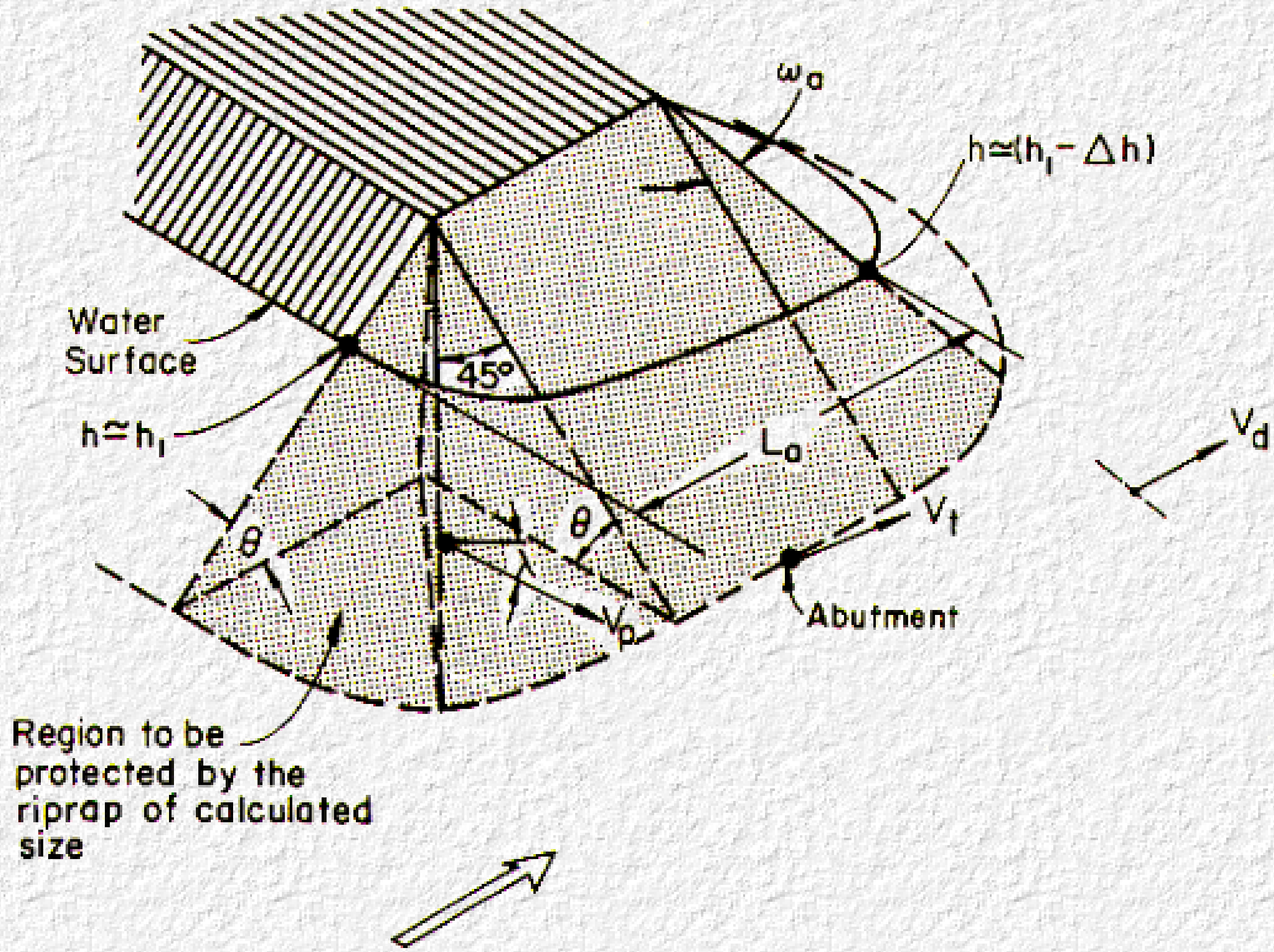


Figure 5.2.7. Flow around a Spill-Through Abutment

For design purposes, the estimation of the velocities and depths in the entire bridge crossing is not necessary. It is adequate to determine the riprap required for protecting the most hazardous zones subject to failure. Then this riprap is used to protect the abutment. The flow parameters for determining the required riprap at midway around the upstream spill-slope and along the abutment toe through the construction are determined below. The sizes of riprap required to produce a stability factor of 1.5 at those two positions are computed and the larger size rock is chosen for abutment protection.

The flow at the toe of the abutment is approximately horizontal and parallel to the side slope (see Figure 5.2.7). For design purposes it is assumed that $\lambda = 0$ degrees. The depth-averaged flow velocity at the toe V_t is related to the depth-averaged velocity at the vena contracta V_d by the curves in Figure 5.2.8. The quantity V_d is found by using the expression

$$V_d = \left(\alpha_1 V_1^2 + 2g\Delta h\right)^{1/2} \qquad (5.2.19)$$

where α_1 and V_1 are the velocity head correction coefficient and the mean velocity in the channel at the upstream section of maximum backwater, respectively, and Δh is the total water surface drop through the bridge opening. The total water surface drop is computed by the method recommended in Chapter 2. The quantity L_a in Figure 5.2.7 is estimated from the geometry of embankment, the stage at the maximum backwater section h_1 and the water surface drop Δh. The velocity at the toe is then found from Figure 5.2.8.

The depth-averaged velocity midway around the upstream spill-slope V_p (see Figure 5.2.7) is related to the vena contracta velocity V_d and can be determined for given values of V_p and $\Delta h/L_a$ by using the relation in Figure 5.2.8. The angle between the velocity vector and the horizontal line, λ, is given in Figure 5.2.9.

After the values of V_t, V_p and λ are estimated from Figure 5.2.8 and Figure 5.2.9, the riprap sizes for a given stability factor are determined at the toe and midway around the upstream spill-slope by employing Equation 5.2.17. The larger of the two calculated riprap sizes should be used for the abutment protection.

A stability factor S.F. = 1.5 is recommended for abutments to allow for error in the hydraulic predictions and for unknown scale effects which may exist in the models.

The region around the abutment nose requiring the calculated size of riprap protection is shown in Figure 5.2.7. The angle ω_a varies according to the relation given in Table 5.2.1. The outer region of the embankment may be protected by a smaller size of riprap than the calculated one.

An example on how to evaluate the stability of riprap on an abutment is presented in Appendix 5, Problem 5.4.

5.2.5 Riprap Gradation and Placement

The concept of a representative grain size for riprap is simple. A uniformly graded riprap with a median size D_{50} scours to a greater depth than a well-graded mixture with the same median size. The uniformly distributed riprap scours to a depth at which the velocity is less than that required for the transportation of D_{50} size rock. The well-graded riprap, on the other hand, develops an armor plate. That is, some of the finer materials, including sizes up to D_{50} and larger, are transported by the high velocities, leaving a layer of large rock sizes which cannot be transported under the given flow conditions. Thus, the size of rock representative of the stability of the riprap is determined by the larger sizes of rock. The representative grain size D_m for riprap is larger than the median rock size D_{50}.

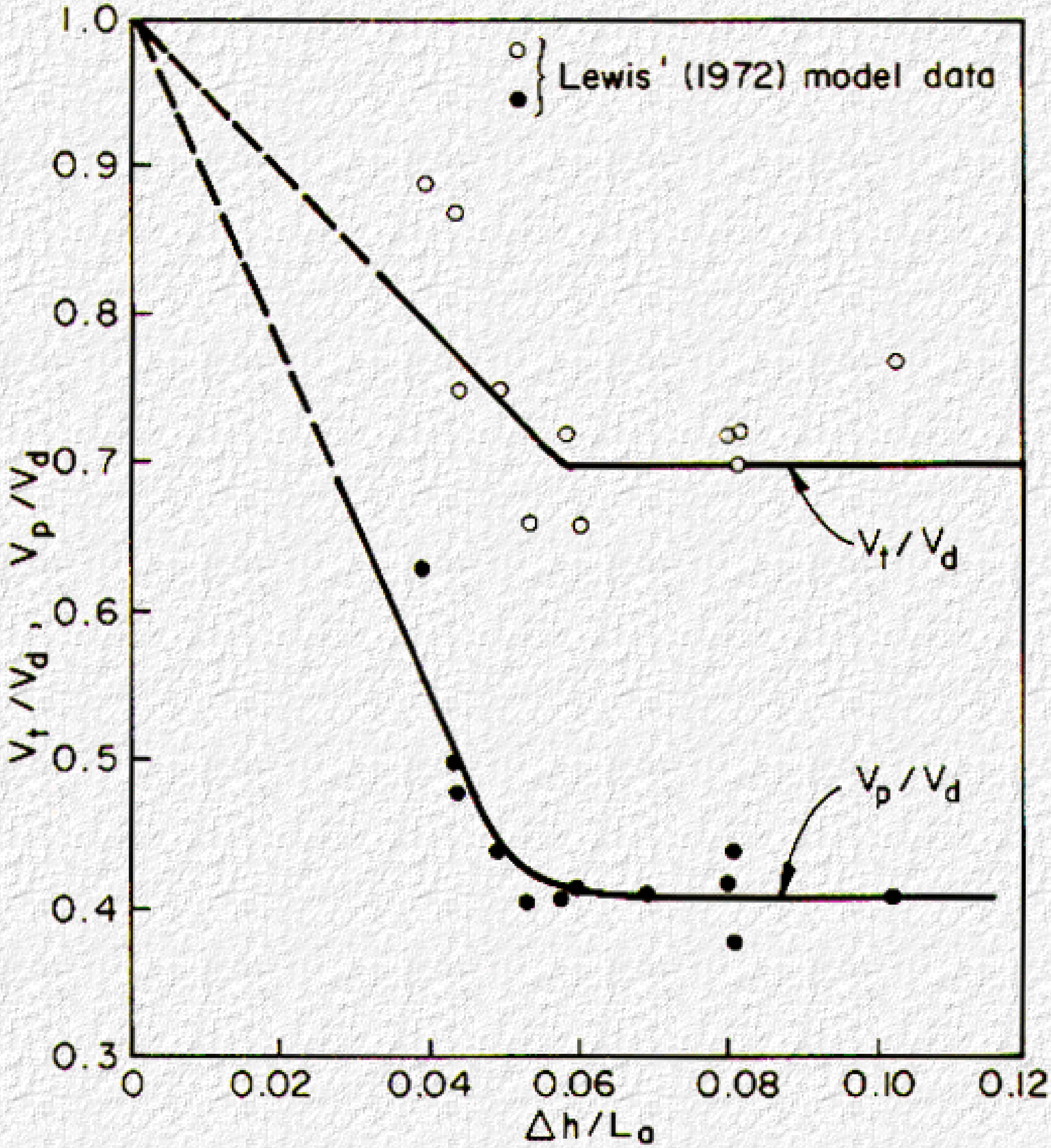


Figure 5.2.8. Relation between Relative Velocities and the Drop Ratio

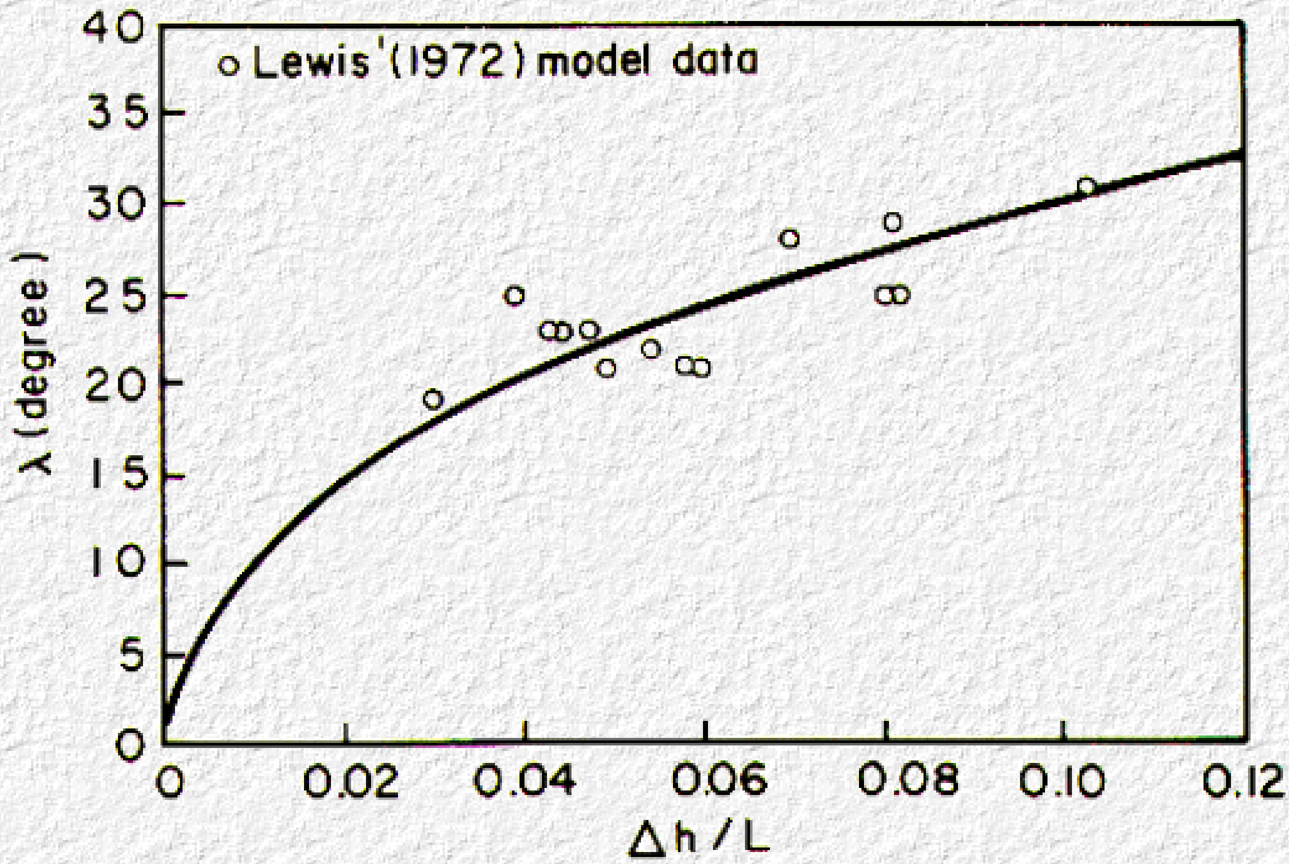


Figure 5.2.9. Relations between γ and the Drop Ratio Midway around the Upstream Spill-Slope

Table 5.2.1 Termination Angle for Riprap Protection

$\Delta h/L_a$	ω_a degree
0 to .05	45
.06	55
.07	65
.08	75
.09	85
.10	90

The recommended gradation for riprap is illustrated in Figure 5.2.10 in terms of D_{50}. The computations of the representative grain size D_m for the recommended gradation are given in Table 5.2.2.

Table 5.2.2 Data for Suggested Gradation

Percent Finer	Sieve Diameter	D_i
0	0.25 D_{50}	--
10	0.35 D_{50}	0.28 D_{50}
20	0.5 D_{50}	0.43 D_{50}
30	0.65 D_{50}	0.57 D_{50}
40	0.8 D_{50}	0.72 D_{50}

50	1.0 D_{50}	0.90 D_{50}
60	1.2 D_{50}	1.10 D_{50}
70	1.6 D_{50}	1.50 D_{50}
90	1.8 D_{50}	1.70 D_{50}
100	2.0 D_{50}	1.90 D_{50}

The rock sizes in the last column in Table 5.2.2 are used in the following equation (Stevens, 1968) to find the representative grain size D_m. This effective grain size, D_m, of the mixture corresponds to the size D_{65} of the riprap.

$$D_m = \left[\frac{\sum_{i=1}^{10} D_i^3}{10} \right]^{1/3} \simeq 1.25\, D_{50} \qquad (5.2.20)$$

When the bed material has a log-normal distribution, the representative size of the bed material based on the weight of the particles is given by Mahmood (1973) as a function of the gradation coefficient G:

$$D_m = D_{50} \exp\left\{ \frac{3}{2} (\ell n\, G)^2 \right\} \qquad (5.2.21)$$

For gradation coefficients of 2 and 3, D_m = 0.72 D_{50} and 1.81 D_{50} respectively. G is determined by Equation 3.2.9.

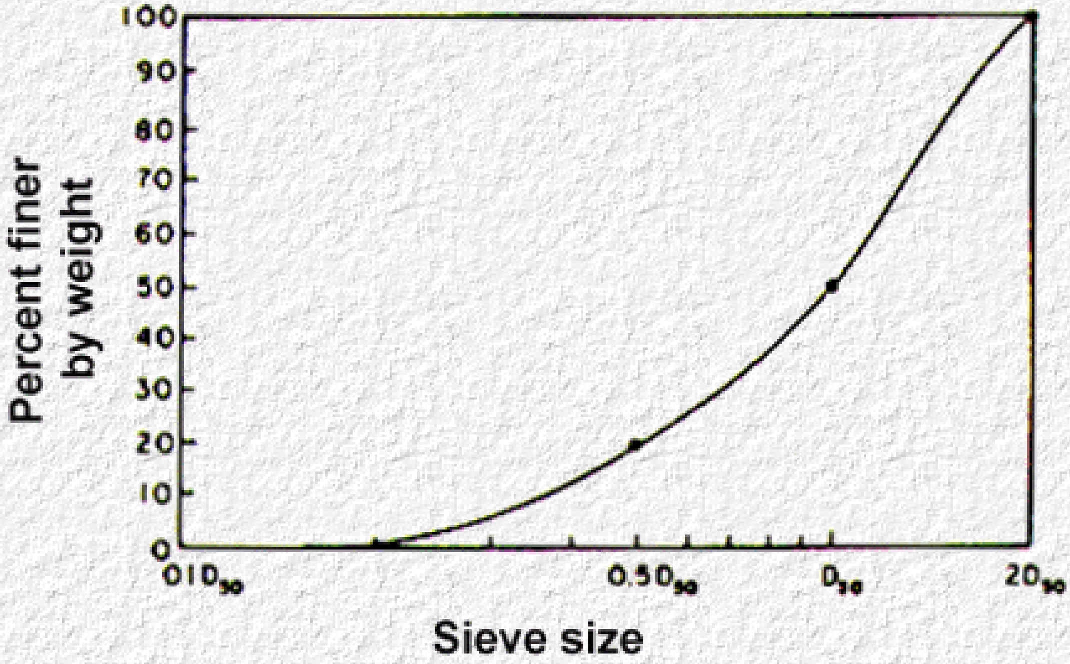


Figure 5.2.10. Suggested Gradation for Riprap

With a distributed size range, the interstices formed by the larger stones are filled with the smaller sizes in an interlocking fashion, preventing formation of open pockets. Riprap consisting of angular stones is more suitable than that consisting of rounded stones. Control of the gradation of the riprap is almost always made by visual inspection. If it is necessary, poor gradations of rock can be employed as riprap provided the proper filter is placed between the riprap and the bank or bed material. Where available rock size is inadequate, wire enclosed (gabion) riprap can be used.

Considering the practical problems of quarry production, a gradation band is usually specified by the U.S. Army, Corps of Engineers (1981) rather than a single gradation curve, and any stone gradation within the limits is acceptable. The Corps criteria for establishing gradation limits in terms of stone weight (W) for riprap are as follows:

- The lower limit of W_{50} stone should not be less than the weight of stone required to withstand the design shear forces.
- The upper limit of W_{50} stone should not exceed five times the lower limit of W_{50} stone, the size which can be obtained economically from the quarry, or the size that satisfies layer thickness requirements.
- The lower limit of W_{100} stone should not be less than two times the lower limit of W_{50} stone.
- The upper limit of W_{100} stone should not exceed five times the lower limit of W_{50} stone, the size which can be obtained economically from the quarry, or the size that satisfies layer thickness requirements.
- The lower limit of W_{15} stone should not be less than one-sixteenth the upper limit of W_{100} stone.
- The upper limit of W_{15} stone should be less than the upper limit of the filter material.
- The bulk volume of stone lighter than the W_{15} stone should not exceed the volume of

voids in the structure without this lighter stone.

The riprap thickness should not be less than 12 in. for practical placement, less than the diameter of the upper limit of W_{100} stone, or less than 1.5 times the diameter of the upper limit W_{50} stone, whichever is greater. If riprap is placed under water, the thickness should be increased by 50 percent, and if it is subject to attack by large floating debris or wave action it should be increased 6 to 12 in.

Riprap placement is usually accomplished by dumping directly from trucks. If riprap is placed during construction of the embankment, rocks can be dumped directly from trucks from the top of the embankment. Rock should <u>never</u> be placed by dropping down the slope in a chute or pushed downhill with a bulldozer. These methods result in segregation of sizes. With dumped riprap there is a minimum of expensive hand work. Poorly graded riprap with slab-like rocks requires more work to form a compact protective blanket without large holes or pockets. Draglines with orange peel buckets, backhoes and other power equipment can also be used to place the riprap.

Hand placed rock riprap is another method of riprap placement. Stones are laid out in more or less definite patterns, usually resulting in a relatively smooth top surface. This form of placement is used rarely in modern practice because it is usually more expensive than placement with power machinery, and it is more likely to fail than dumped riprap.

Dumped riprap keyed (or plated) by tamping has proved to be effective. Guidelines for placement of keyed riprap have been developed by the Oregon Department of Transportation and distributed by the Federal Highway Administration. In the keying of a riprap, a 4,000-lb (1,818 kg) or larger piece of steel plate is used to compact the rock into a tight mass and to smooth the revetment surface. <u>Keyed riprap is more stable than loose riprap revetment because of reduced drag on individual stones, its angle of repose is higher, and its cost is less because a lesser volume of rock per unit area is required.</u>

5.2.6 Filters for Riprap

Filters are used under riprap to allow water to drain easily from the bank without carrying out soil particles. <u>Filters must meet two basic requirements: stability and permeability</u>. The filter material must be fine enough to prevent the base material from escaping through the filter, but it must be more permeable than the base material. There is no standard filter that can be used in all cases. Two types of filters are commonly used: gravel filters and synthetic filter cloths.

- **Gravel filters**
  A layer or blanket of well-graded gravel should be placed over the embankment or riverbank prior to riprap placement. Sizes of gravel in the filter blanket should be from 3/l6 in. to an upper limit depending on the gradation of the riprap with maximum sizes of about 3 to 3-l/2 in. Thickness of the filter may vary depending upon the riprap thickness but should not be less than 6 to 9 inches. Filters that are one-half the thickness of the riprap are quite satisfactory. Suggested specifications for gradation are as follows:

(1) $$\frac{D_{50}\ (\text{Filter})}{D_{50}\ (\text{Base})} < 40 \qquad (5.2.22)$$

(2) $$5 < \frac{D_{15}\ (\text{Filter})}{D_{15}\ (\text{Base})} < 40 \qquad (5.2.23)$$

(3) $$\frac{D_{15}\ (\text{Filter})}{D_{85}\ (\text{Base})} < 5 \qquad (5.2.24)$$

If the base material is a fine-grained cohesive soil, such as fat or lean clay, these requirements are not applicable, and the stability criterion is that the D_{15} size of the filter cannot exceed 0.4 mm.

When the base material is very fine, the required filter material may also be quite fine, and more than one layer of filter (a graded filter) may be needed. In such a case, each layer must satisfy the stability and permeability requirements relative to the underlying layer.

If the filter is designed for protection against the upward flow of water, the graded filter is constructed so that each layer is coarser than the one beneath (a "reverse" or "inverted" filter).

A Design Example on the use of gravel filters is presented in the Appendix 5, Problem 5.5.

- **Synthetic Filter Cloths**
  Synthetic fabrics are also used as filters, replacing a component of a graded filter. Numerous synthetic filter fabrics are on the market, with a wide variation in size and number of openings and in strength and durability of material. Fabrics which provide opening areas of 25 to 30 percent are desirable to minimize the possibility of clogging and to reduce head loss.

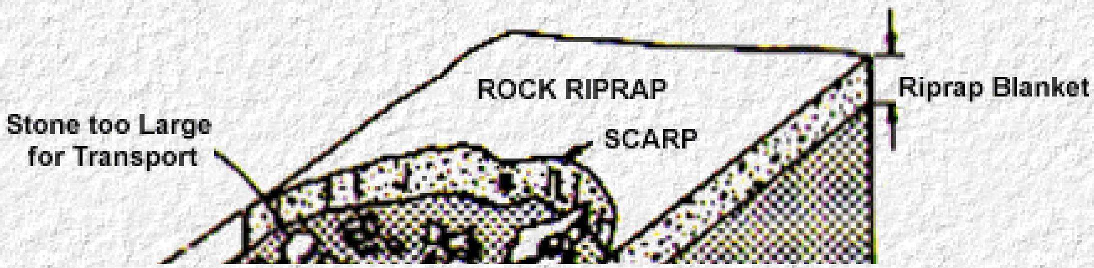
  When filter fabric is used, care must be taken not to puncture the fabric during construction. If the filter fabric is placed on top of the base material, gravel can sometimes be placed directly on the fabric, eliminating the need for filter sand. If the paving materials is dumped or cast stone, however, it is desirable to place a protective blanket of sand or gravel on the filter-or to take care in placing the rock-so that the filter fabric is not punctured. Stones weighing as much as 3,000 lbs have been placed on synthetic filters with no apparent damage. If a protective covering is not used, the size and drop of the rock should be limited. The sides and toe of the filter fabric must be sealed or trenched so that base material does not leach out around the filter fabric. Care is also required in joining adjacent sections of filter fabric together; sewn, overlapped, and welted seams are used. Fabrics are generally in 100 ft long rolls, 12 to 18 ft wide. Overlap of 8 to 12 inches is provided with pins at 2 to 3 ft intervals along the seam to prevent separation in case of settlement of the base material.

5.2.7 Riprap Failure and Protection

In the recent preliminary evaluation of various riprap design techniques, Blodgett and McConaughy (1985) concluded that the procedures based on velocity as a means of estimating stresses on the boundary provide the most reliable and consistent results. The following procedures were investigated: The 1967 version of FHWA HEC-11, FHWA HEC-15, Corps of Engineers (EM-1601), Caltrans Bank and Shore Manual, Simons and Senturk, and Oregon Department of Transportation. A major shortcoming of all present design techniques is their assumption that failures of riprap revetment are due only to particle erosion. Procedures for the design of riprap protection will need to be prepared that consider all the various causes of failures.

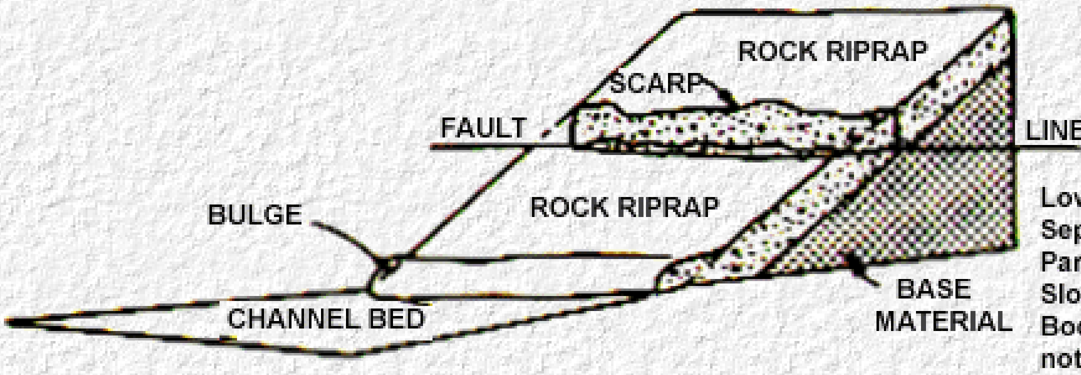
Classic riprap failure modes are identified as follows: (1) particle erosion; (2) translational slide: (3) modified slump; and (4) slump. These modes of failure are illustrated in Figure 5.2.11.

- **Particle erosion** is the most commonly considered erosion mechanism (Figure 5.2.11a). Particle erosion occurs when individual particles are dislodged by the hydraulic forces generated by the flowing water. Particle erosion can be initiated by abrasion, impingement of flowing water, eddy action/reverse flow, local flow acceleration, freeze/thaw action, ice, or toe erosion. Probable causes of particle erosion include: (1) stone size not large enough; (2) individual stones removed by impact or abrasion; (3) side slope of the bank so steep that the angle of repose of the riprap material is easily exceeded; and (4) gradation of riprap too uniform.
- **A translational slide** is a failure of riprap caused by the downslope movement of a mass of stones, with the fault line on a horizontal plane (Figure 5.2.11b). The initial phases of a translational slide are indicated by cracks in the upper part of the riprap bank that extend parallel to the channel. This type of riprap failure is usually initiated when the channel bed scours and undermines the toe of the riprap blanket; this could be caused by particle erosion of the toe material, or some other mechanism which causes displacement of toe material. Any other mechanism which would cause the shear resistance along the interface between the riprap blanket and base material to be reduced to less than the gravitational force could also cause a translational slide. It has been suggested that the presence of a filter blanket may provide a potential failure plane for translational slides. Probable causes of translational slides are as follows: (1) bank side slope too steep; (2) presence of excess hydrostatic (pore) pressure; and (3) loss of foundation support at the toe of the riprap blanket caused by erosion of the lower part of the riprap blanket.



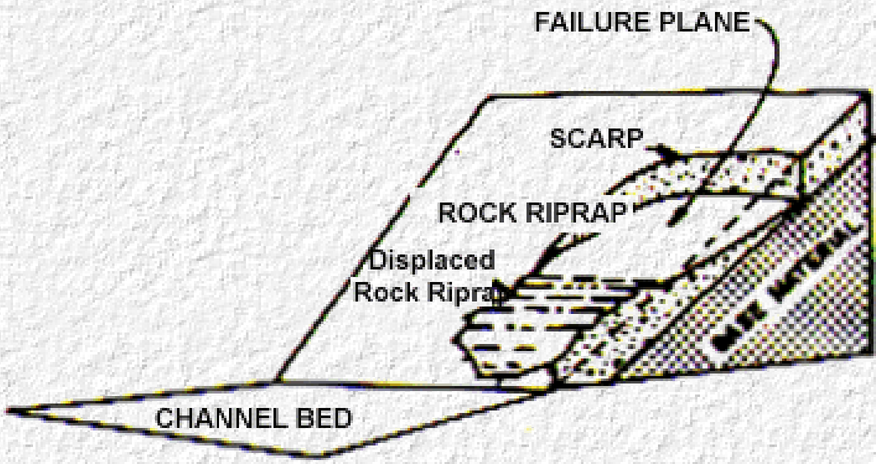
A. PARTICLE EROSION

Mound of desplaced Rock Riprap. Particle Erosion Results if Flow Shear Stress or Velocities are Excessive. If Displaced Stones are not Transported from the Eroded Area, the Channel Bed Will Show a Mound.



B. TRANSLATIONAL SLIDE

Lower Part of Riprap Separates from Upper Part, and Moves Down-Slope as Homogeneous Body. The Toe may not show a Bulge if Channel Bed is Secured. Translational Slide Usually Occurs if Side Slope is too Steep or Toe of Riprap is Underminded.



Filter Blanket at Surface of Base Material (Not Shown).

C. MODIFIED SLUMP

Rock Riprap Moves Downslope Along a Failure Plane that Lies at or above Base Material. Failure Plane is at a Flatter Slope than Original Rip-rap Blanket. This Type of Failure is Usually Caused by Excess Hydrostatic Pressure in Riprap Blanket or Shear Along Filter Blanket.

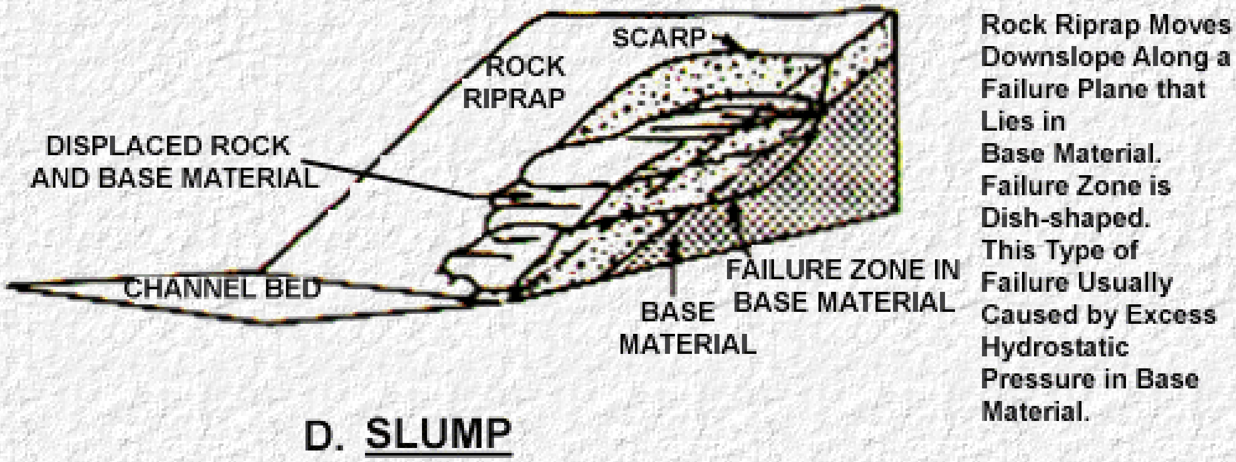


Figure 5.2.11. Riprap Failure Models (after Blodgett and McConaughy, 1985)

- **Modified slump failure** of riprap (Figure 5.2.11c) is the mass movement of material along an internal slip surface within the riprap blanket; the underlying material supporting the riprap does not fail. This type of failure is similar in many respects to the translational slide, but the geometry of the damaged riprap is similar in shape to initial stages of failure caused by particle erosion. Probable causes of modified slump are: (1) bank side slope is so steep that the riprap is resting very near the angle of repose, and any imbalance or movement of individual stones creates a situation of instability for other stones in the blanket; and (2) material critical to the support of upslope riprap is dislodged by settlement of the submerged riprap, impact, abrasion, particle erosion, or some other cause.
- **Slump failure** is a rotational-gravitational movement of material along a surface of rupture that has a concave upward curve (Figure 5.2.11d). The cause of slump failures is related to shear failure of the underlying base material that supports the riprap. The primary feature of a slump failure is the localized displacement of base material along a slip surface, which is usually caused by excess pore pressure that reduces friction along a fault line in the base material. Probable causes of slump failures are: (1) nonhomogeneous base material with layers of impermeable material that act as a fault line when subject to excess pore pressure; and (2) side slopes too steep and gravitational forces exceeding the inertia forces of the riprap and base material along a friction plane.

Because of the general effectiveness of dumped riprap, a more detailed analysis of the relatively small number of cases in which it failed has been presented by Brice, Blodgett et al (1978). The principal causes of failure and methods of mitigation are given as follows.

Cause	Solution
Inadequate size of riprap.	Larger riprap.
Impingement of current directly upon riprap rather than having flow parallel to riprap.	Heavier stones, flatten riprap slopes, redirect flow.

Channel degradation.	Provide a volume of reserve riprap at the revetment toe.
Internal slope failure (slump).	Reduce the riprap slope angle.
Riprap with high percentage of fines causes washing out of the fines.	Follow gradation specifications.

Riprap should not be used at slopes steeper than 1.5:1. This criterion is widely followed for abutment fill-slopes, but it is sometimes disregarded for stream banks. Use of riprap on steep banks, even in emergency situations, is probably not worth the cost.

Broken concrete is used for riprap in many states where rock riprap is unavailable or unusually expensive. The use of broken concrete riprap has proven to be more or less unsatisfactory.

Rounded stones less than 6 in. in diameter have a significantly lower angle of repose than angular stones (Figure 3.2.4). Although they are less desirable than angular stones, rounded stones are nevertheless effective for larger diameters.

The blanket should be stabilized at its base with a key trench or apron to prevent the stone from sliding down the bank. The upstream and downstream ends of the blanket should be tied back into the bank to prevent stream currents from unravelling the blanket. The most common method to tie into the bank is to dig a trench at the ends of the blanket. (Figure 5.2.12) The depth of a trench should be twice the blanket thickness and the bottom width of the trench three times the thickness.

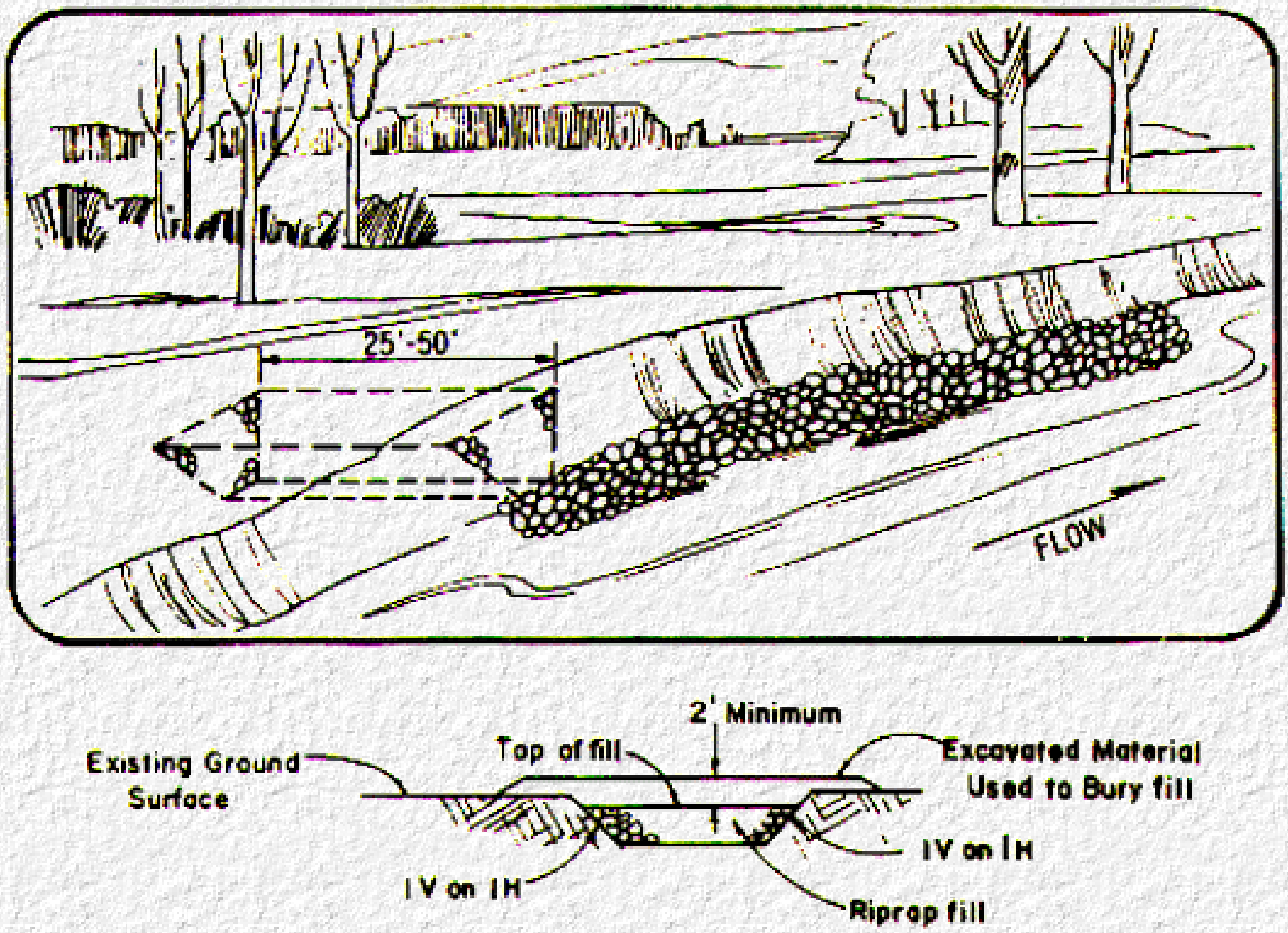


Figure 5.2.12. Tie-in Trench to Prevent Riprap Blanket from Unravelling (after Kesum, 1983)

[Go to Chapter 5 (Part II)]

Chapter 5 : HIRE
River Stabilization, Bank Protection and Scour Part II

Go to Chapter 5, Part III

5.3 Bank Protection Other Than Riprap

This section discusses bank protection measure other than riprap: vegetation (Section 5.3.1); rock and wire mattresses (Section 5.3.2); gabions (Section 5.3.3); sacks (Section 5.3.4); blocks (Section 5.3.5); articulated mattresses (Section 5.3.6); used tires (Section 5.3.7); rock-fill trenches (Section 5.3.8); windrow revetment (Section 5.3.9); soil cement (Section 5.3.10); and bulkheads (Section 5.3.11).

5.3.1 Vegetation

Vegetation is probably the most natural method for protecting streambanks because it is relatively easy to establish and maintain, is visually attractive and environmentally more desirable.

Below a stream's waterline, vegetation can effectively protect a bank in two ways. First, the root system helps to hold the soil together and increases overall bank stability by forming a binding network. Second, the exposed stalks, stems, branches and foliage provide resistance to the streamflow, causing the flow to lose energy by deforming the plants rather than by removing soil particles. Above the waterline, vegetation prevents surface erosion by absorbing the impact of falling raindrops and reducing the velocity of overbank drainage flow and rainfall runoff. Further, vegetation takes water from the soil providing additional capacity for infiltration and may improve bank stability by water withdrawal.

Vegetation is generally divided into two broad categories: grasses and woody plants (trees and shrubs). The grasses are less costly to plant on an eroding bank above the toe and require a shorter period of time to become established. Woody plants offer greater protection against erosion because of their more extensive root systems; however, under some conditions the weight of the plant will offset the advantage of the root system. On very high banks, tree root systems do not always penetrate to the toe of the bank. If the toe becomes eroded, the weight of the tree and its root mass may cause a bank failure.

The major factor affecting species selection is the length of time required for the plant to become established on the slope.

Water-tolerant grasses such as canarygrass (Phalaris), reedgrass (Calamagrostis), cordgrass (Spartina), and fescue (Festuca) are effective in prevent erosion on upper banks which are inundated from time to time and are primarily subject to erosion due to rainfall, overland flow, and minor wave action. Along the lower bank, where erosive forces are high, vegetation is generally not effective as a protective measure; however, cattails (Typha), bulrushes (Scripus), reeds (Phragmites), knotweed and smartweed (Plygonum), rushes (Juncus), and mannagrass (Glyceria) are helpful in inducing deposition and reducing velocities in shallow water or wet areas at the bank toe and in protecting the bank in some locations. Willows (Salix) are among the most effective woody plants in protecting low banks because they are resilient, are sufficiently dense to promote deposition of sediment, can withstand inundation, and become established easily.

Grass can be planted by hand seeding, sodding, sprigging, or by mechanical broadcasting of mulches consisting of seed, fertilizer, and other organic mixtures. Several commercial manufacturers now market economical erosion control matting that will hold the seed and soil in place until new vegetation can become established. The matting is generally installed by hand and secured to the bank where plantings have been made to prevent erosion, then a fence should be placed along the top of bank. If livestock require access to the stream for watering or crossing, gates should be placed in the fence at locations where the cattle will do the least amount of damage to the planted bank; additionally, crossings should be fenced.

5.3.2 Rock-and-Wire Mattresses

When adequate riprap sizes are not available, rocks of cobble sizes may be placed in wire mesh mats made of galvanized fencing and placed along the bank forming a mattress. The individual wire units are called baskets if the thickness is greater than 12 inches. The term mattress implies a thickness no greater than 12 inches. Toe protection is offered by either extending the mattresses on to the channel bed as shown in Figure 5.3.1. or embedding the mattress to some predetermined scour depth. As the bed along the toe is scoured, the extended mattress drops into the scour hole. Special wire baskets and mattresses are manufactured and sold throughout the United States. It should be noted that when rock-and-wire mattresses are used in streams transporting cobble and rocks, the wires of the basket can be cut by abrasion rather rapidly, which will destroy the intended protection along the base of the bank. Corrosion of the wire mesh and vandalism may also be a problem.

Mattresses and baskets can be made up in large sizes in the field. These are flexible and can conform to scour holes which threaten the stability of the banks. They should be linked together to prevent separation as subsidence takes place.

The most economical combination of rock and wire for streambank protection is simply laying wire mesh over stone. The major problem with this approach is keeping the mesh in place. One successful solution has been to bend pipe or rebar into the shape of a staple and then drive it through the mesh into the bank. The major drawback is that a rock and wire mattress generally costs more to place than a comparable riprap blanket.

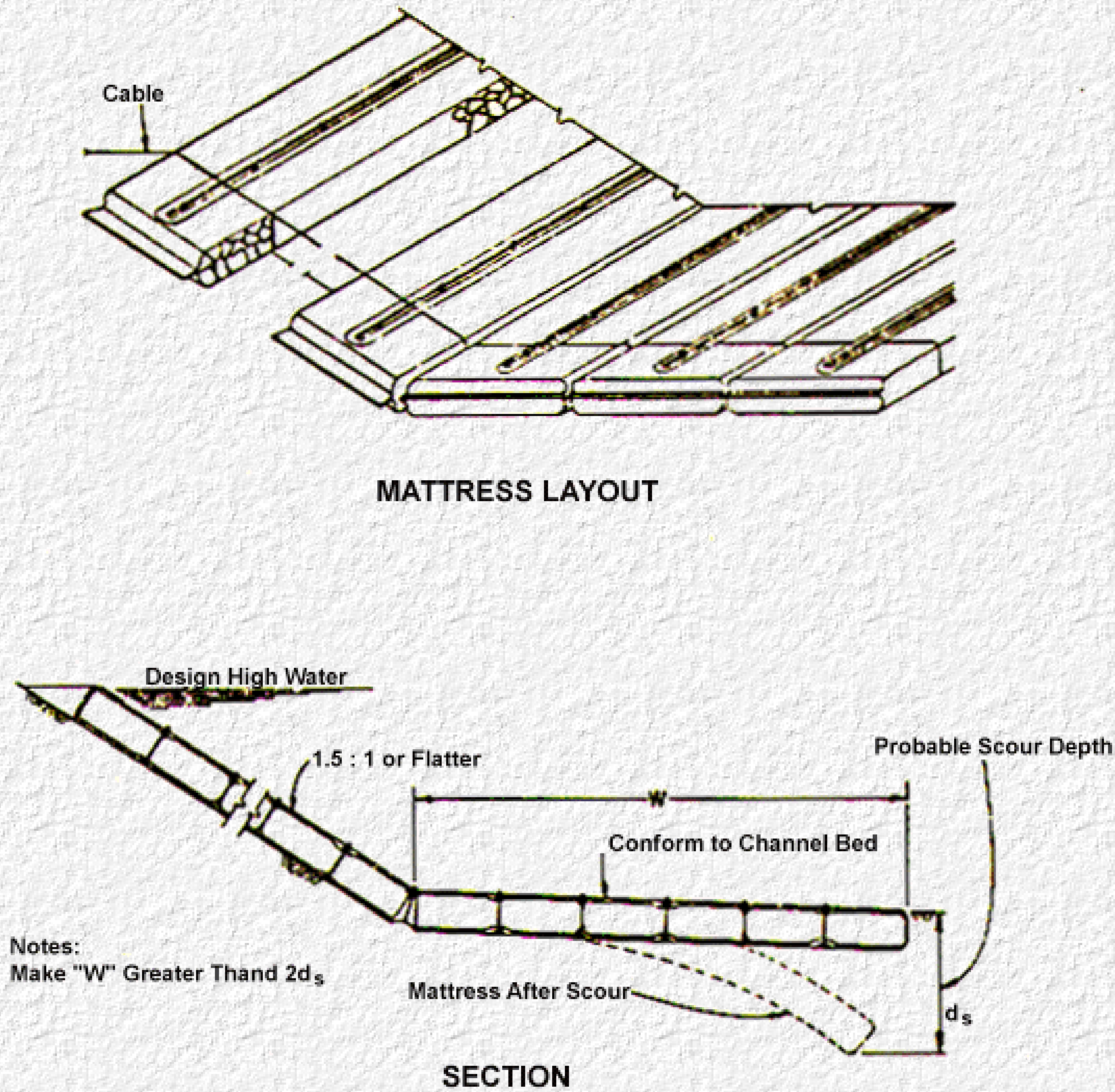


Figure 5.3.1. Rock and Wire Mattress

5.3.3 Gabions

Gabions are patented rectangular wire boxes (or baskets) filled with relatively small-size stone, usually less than 8 in. in diameter. Where flow velocities are such that small stone would not be stable if used in a riprap blanket, the wire boxes provide an effective restraint. Limiting recommended maximum velocity for use of gabions ranges from 8 to 15 ft/sec, depending on the manufacturer. Gabions are used primarily for revetment-type structures, but have also been used for dikes and sills.

Gabions act as a large heavy porous mass having some flexibility. The baskets are commercially available in a range of standard sizes and are made of heavy galvanized wire (coated when used in a corrosive environment). They are supplied at a job site folded flat and are assembled manually, using noncorrosive wire. The baskets are normally 0.5 m deep by 1 m by 2 m and are set on a graded bank for revetments. A filter blanket or syntheitc filter fabric is used where required to prevent leaching of base material and undermining of the baskets.

5.3.4 Sacks

Burlap sacks filled with soil or sand-cement mixtures have long been used for emergency work along levees and streambanks during floods (Figure 5.3.2). In recent years commercially manufactured sacks (burlap, paper, plastics, etc.) have been used to protect streambanks in areas where riprap of suitable size and quality is not available at a reasonable cost. Although most types of sacks are easily damaged and will eventually deteriorate, those sacks filled with sand-cement mixtures can provide long-term protection if the mixture has set up properly. Sand-cement sack revetment construction is not economically competitive in areas where good stone is available. However, if quality riprap must be transported over long distances, this type of sack revetment can often be placed on an eroding streambank at a lesser cost than riprap.

If a permanent revetment is to be constructed, the sacks should be filled with a mixture of 15 percent cement (minimum) and 85 percent dry sand (by weight). The filled sacks should be placed in horizontal rows like common house brick beginning at an elevation below any toe scour (alternatively, riprap can be placed at the toe to prevent undermining of the bank slope). The successive rows should be stepped back approximately 1/2-bag width to a height on the bank above which no protection is needed. The slope steepness of the completed revetment should be no more than 1 foot vertical for 1 foot horizontal. After the sacks have been placed on the bank, they can be hosed down for a quick set or the sand-cement mixture can be allowed to set up naturally through rainfall, seepage or condensation. If cement leaches through the sack material, a bond will form between the sacks and prevent free drainage. For this reason weepholes should be included in the revetment design. The installation of weepholes will allow drainage of groundwater from behind the revetment thus helping to prevent pressure buildup that could cause revetment failure.

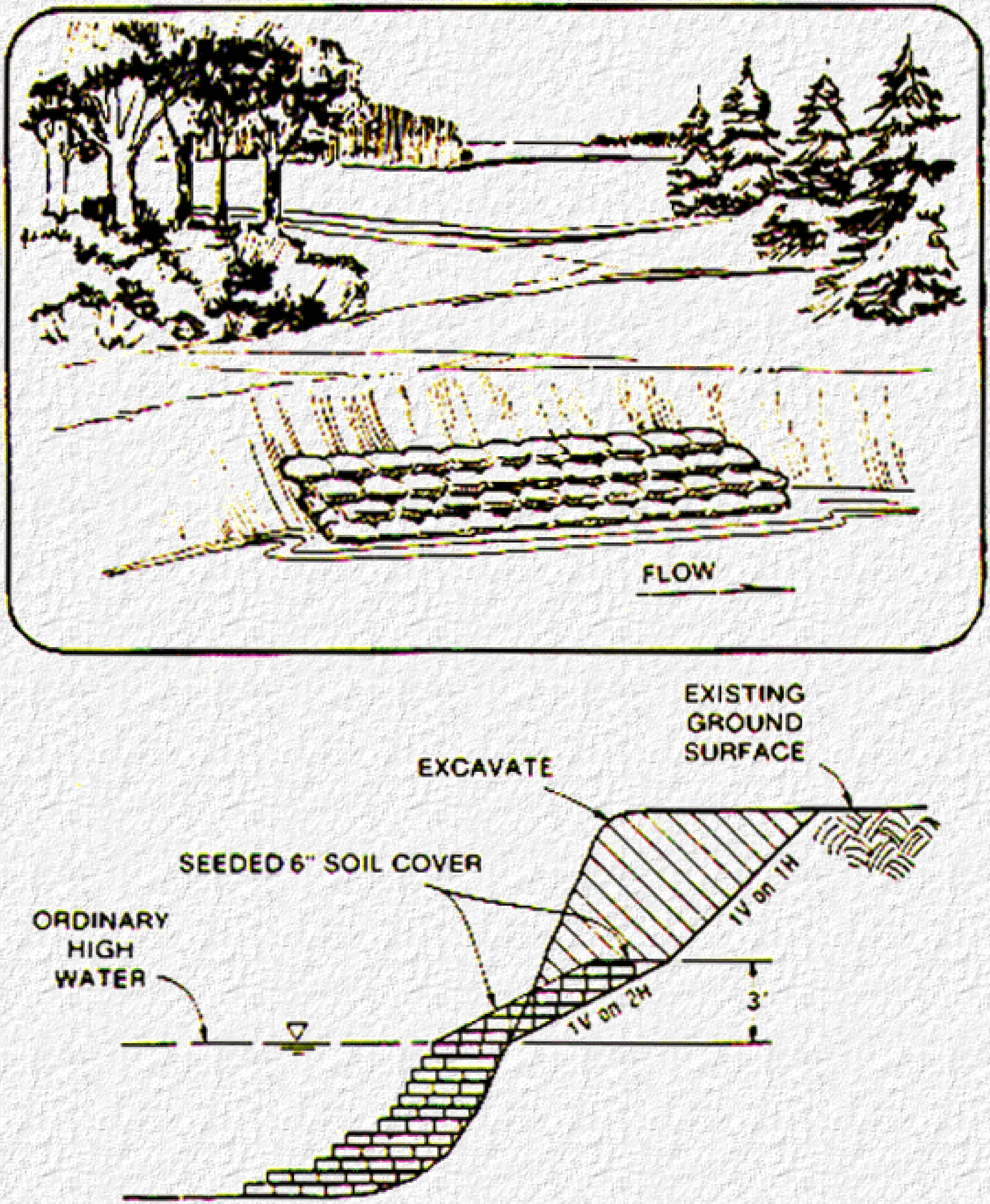


Figure 5.3.2. Typical Sand-Cement Bag Revetment (after Keown, 1983)

5.3.5 Blocks

Precast cellular blocks can be manufactured using locally available sand, cement, and aggregate or can be obtained from commercial sources. Cellular blocks are cast with openings to provide for drainage and to allow vegetation to grow through the blocks thus permitting the root structure to strengthen the bank. Fabric or a gravel blanket can be used as a filter under the blocks if there is any danger that the bank soil will be eroded through the block openings by streamflow or seepage. Although specialized equipment can be used to install large sections of blocks, hand placement is frequently used when mechanized apparatus is not available, access to the bank is limited, or costs need to be minimized. After the blocks have been placed, the revetment has sufficient flexibility to conform to minor changes in bank shape. Solid blocks should not be used because the bank may not be able to drain freely and failure could occur.

5.3.6 Articulated Concrete Mattress

Small precast concrete blocks held together by steel rods or cables can be used to form a flexible mat as shown in Figure 5.3.3.

The sizes of blocks may vary to suit the contour of the bank. It is particularly difficult to make a continuous mattress of uniform sized blocks to fit sharp curves. The open spacing between blocks permits removal of bank material unless a filter blanket of gravel or plastic filter cloth is placed underneath. For embankments that are subjected only to occasional flood flows, the spaces between blocks may be filled with earth and vegetation can be established.

The use of articulated concrete mattresses has been limited primarily to the Mississippi River. This is due to the large cost of the plant required for the placement of the mattress beneath the water surface. Thus, it is economically feasible to use articulated concrete mattresses only on rivers which require extensive bank protection. The expense of the installation plant is not required, however, for placement of articulated concrete mattresses above the water surface. Thus, paving the upper bank with articulated concrete mattresses has been used occasionally in the United States and Europe.

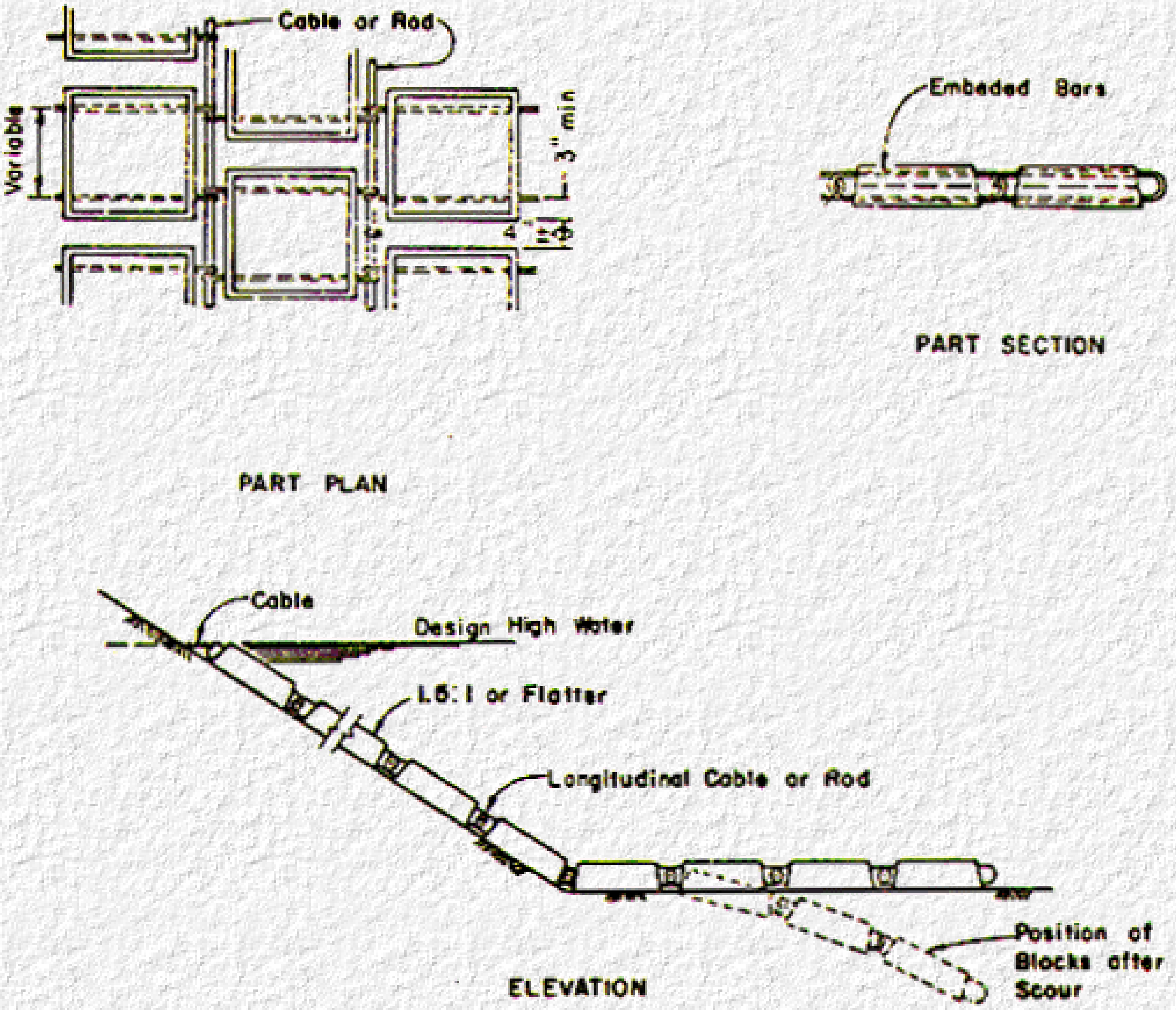


Figure 5.3.3. Articulated Concrete Mattress

5.3.7 Used Tires

Tires have been placed both as a mattress and stacked back against the bank. Both methods appear to have good potential as an economical approach to protect a streambank.

During construction of a tire mattress on an eroding bank, two precautions should be considered to ensure that the mattress will stay in place.

- The tires must be banded together; alternatively, cables running the length and width of the mattress can be woven through the tires.
- The top, toe and the upstream and downstream ends of the mattress must be tied to the bank (Figure 5.3.4). If scour is anticipated, riprap should be placed at the toe of the mattress for additional protection.

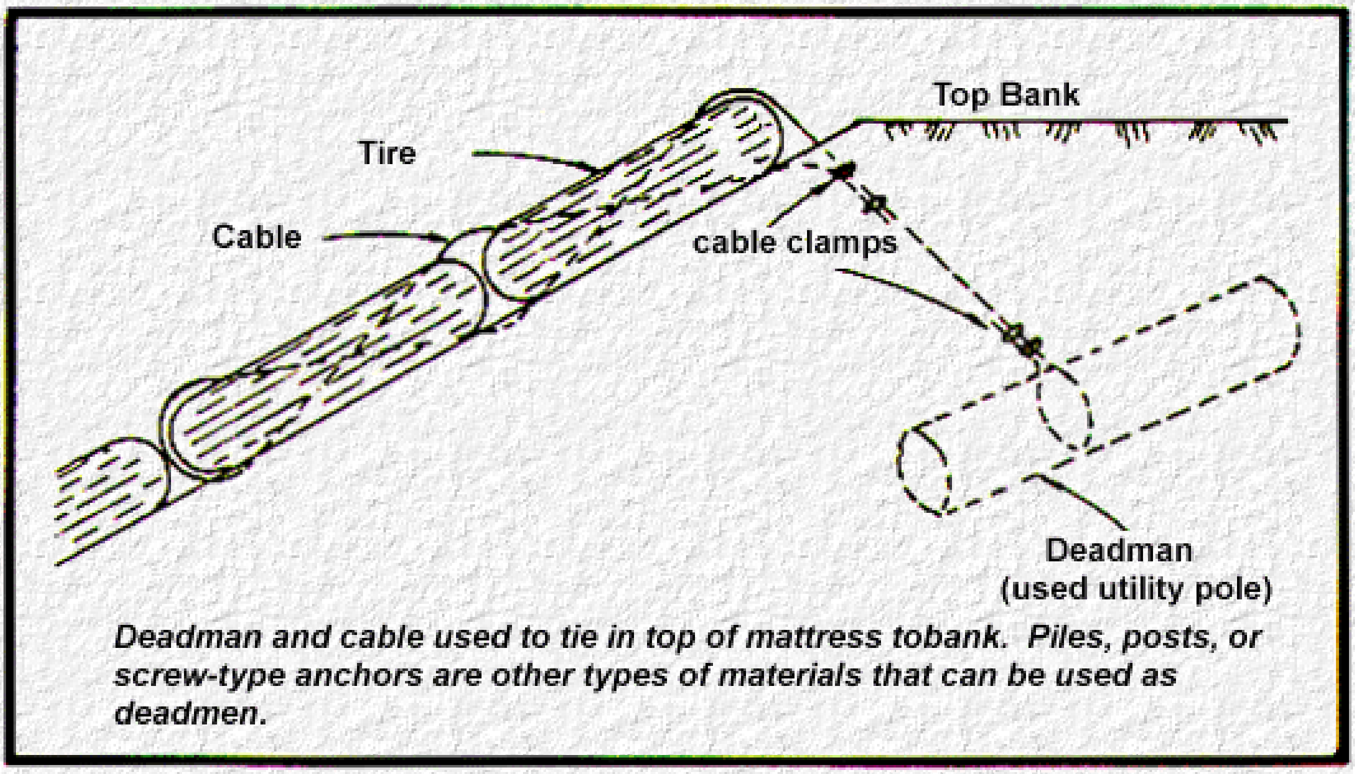


Figure 5.3.4. Used Tire Mattress (after Keown, 1983)

While the precautions listed above are essential for successful construction of a stable mattress, other considerations can further improve the chances that the revetment will provide long-term bank protection.

- Holes can be cut, drilled, or burned in the tire sidewalls to prevent flotation.
- Presorting the tires by size may help to fit them together.
- Earth screw anchors (or some other type of anchor) fastened to the mattress can be placed in the bank at various points on the face of the revetment.
- The tires can be packed with stone or rubble.
- Willows can be planted inside the tires preferably at the beginning of the growing season. Once established the root system will further strengthen the bank and obscure the unsightly mattress. If willows are not readily available, other species should be planted. Possible species for use are discussed later in this text.

If the mattress effectively controls the streambank erosion and remains intact, sediment may gradually cover the revetment. If willows have not been planted, volunteer vegetation will probably become established in areas with a temperate climate.

Prior to constructing a stacked-tire revetment, the bank face should be shaped so that the tires can be laid in horizontal rows. The revetment should be started at the toe of the bank and stepped back 6 to 12 inches per row. Each tire should overlap the two tires under it. The stacked tires should be packed tightly with stone or rubble. Any space behind the tires should be filled with free-draining soil so that the soil mass will not become saturated and cause the revetment to fail. In addition, the upstream and downstream ends of the revetment should be tied into the bank so that there is no flow behind the revetment.

5.3.8 Rock-Fill Trenches

Rock-fill trenches are structures used to protect banks from caving caused by erosion at the toe. A trench is excavated along the toe of the bank and filled with rocks as shown in Figure 5.3.5.

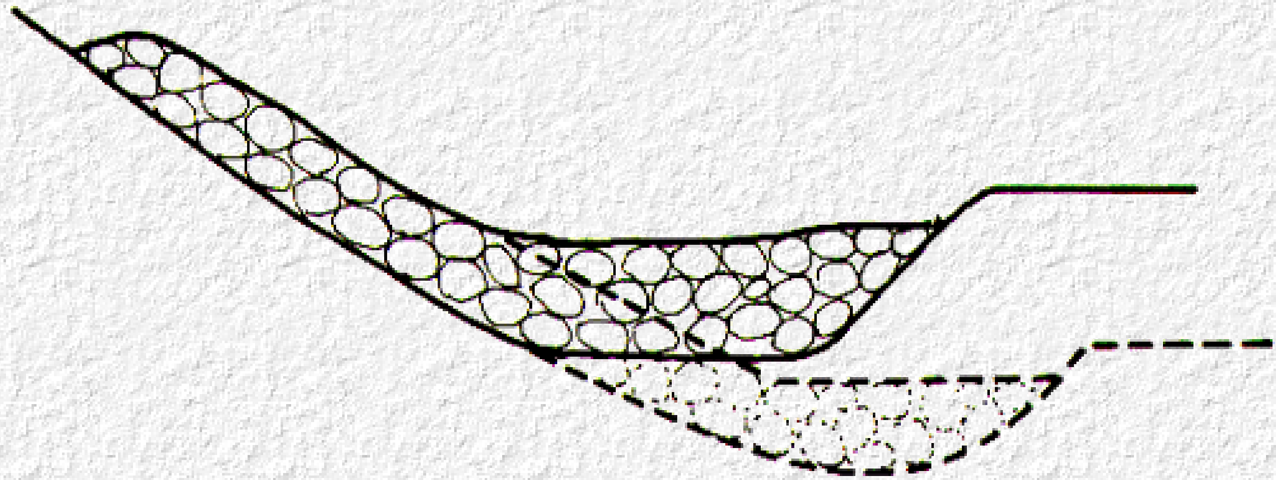


Figure 5.3.5. Rock-Fill Trench

As the stream bed adjacent to the toe is eroded, the toe trench is undermined and the rock fill slides downward to pave the bank. The size of trench to hold the rock fill depends on expected depths of scour. It is advantageous to grade the banks before paving the slope with riprap and placing rock in the toe trench. The slope should be at such an angle that the saturated bank is stable while the river stage is falling.

The rock-fill trench need not be at the toe of the bank. An alternative method is to excavate a trench above the water line along the top of the river bank and backfill with rocks. Then as the bank erodes toward the trench, the rocks in the trench slide down and pave the bank. This method is applicable in areas of rapidly eroding banks of medium to large size rivers.

A variation of this method of toe protection is to pile the rocks in a "windrow" along the bank line instead of excavating a trench. Then as the bank is scoured, the rocks in the windrow drop down to pave the bank.

5.3.9 Windrow Revetment

Windrow revetment is an erosion control technique (Figure 5.3.6) consisting of the depositing of a fixed amount of erosion-resistant material landward from the existing bank line at a predetermined location, beyond which additional erosion is to be prevented. The technique consists of burying or piling a sufficient supply of erosion-resistant material in a windrow below or on the existing land surface along the bank, then permitting the area between the natural riverbank and the windrow to erode though natural processes until the erosion reaches and undercuts the supply of rock. As the rock supply is undercut, it falls onto the eroding area, thus giving protection against further undercutting, and eventually halting further landward movement. The resulting bank line remains in a near natural state, with an irregular appearance due to intermittent lateral erosion in the windrow location. The treatment particularly lends itself to the protection of adjacent wooded areas, or placement along stretches of presently eroding, irregular bank line. The following observations and conclusions were obtained from model investigations on windrow revetments.

1. The "application rate" is the weight of stone applied per foot of bank line. The amount of stone in the windrow indicates the degree to which lateral erosion will be permitted to occur;

2. Various windrow shapes were investigated in the model investigations, and a rectangular cross section was the best windrow configuration. This type of windrow is most easily placed in an excavated trench of the desired width. The second best windrow shape was found to be a trapezoidal shape. This shape provides a steady supply of stone to produce a uniform blanket of stone on the eroding bank line. A triangular shape was found to be the least desirable;

3. Studies indicated that varying the bank height did not significantly affect the final revetment; however, high banks tended to produce a nonuniform revetment alignment. Studies showed that the high banks had a tendency for large segments of the bank to break loose and rotate slightly, whereas the low banks simply "melted" or sloughed into the stream. The slight rotation of the high bank segment probably induced a tendency for ragged alignment; and

4. The velocity and characteristics of the stream dictate the size of stone that must be used to form a windrow revetment. The size of stone used in the windrow was not significant as long as it was large enough to resist being transported by the stream. An important design parameter is the ratio of the relative thickness of the final revetment to the stone diameter. It was found that large stone sizes will require more material than smaller stone sizes to produce the same relative thickness. A well-graded stone is important to ensure that the revetment does not fail from leaching of the underlying bank material. The stream velocity was found to have strong influence on the ultimate side slope of the revetment. It was determined that the initial bank slope was on the average approximately 15 percent steeper than the final revetment slope. In general, the greater the velocity, the steeper the side slope of the final revetment.

5.3.10 Soil Cement

In areas where riprap is scarce, use of in-place soil can sometimes be combined with cement to provide a practical alternative. Figure 5.3.7 shows a detail of a typical soil-cement construction for bank protection. For use in soil-cement, soils should be easily pulverized and contain at least five percent, but not more than 35 percent, silt and clay (material passing the No. 200 sieve). Finer textured soils usually are difficult to pulverize and require more cement as do 100 percent granular soils which have no material passing the No. 200 sieve. Soil cement can be placed and compacted on slopes as steep as one horizontal to one vertical.

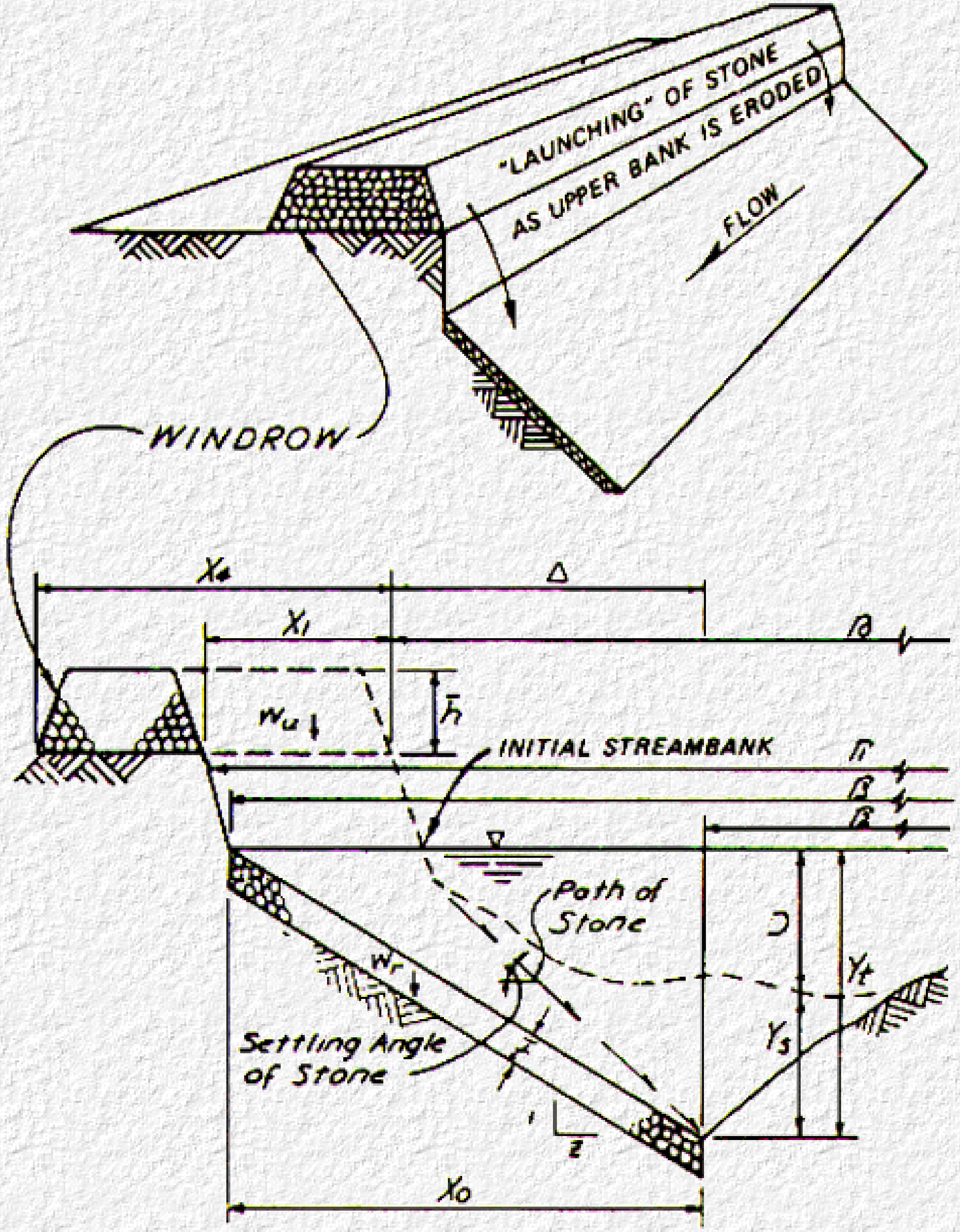


Figure 5.3.6. Windrow Revetment, Definition Sketch (after U.S.A. COE, 1981)

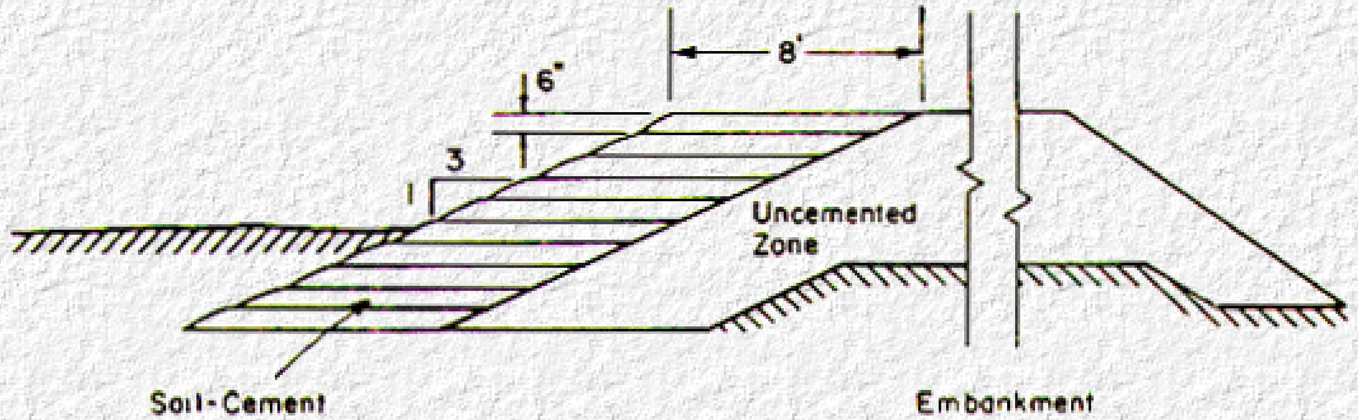


Figure 5.3.7. Typical Soil-Cement Bank Protection

A stairstep construction is recommended on channel embankments with relatively steep slopes. Placement of small quantities of soil-cement for each layer (six inch layers) can progress more rapidly than a large quantity of fill material. Special care should be exercised to prevent raw soil seams between successive layers of soil-cement. If uncompleted embankments are left at the end of the day, a sheepsfoot roller should be used on the last layer to provide an interlock for the next layer. The completed soil-cement installation must be protected from drying out for a seven day hydration period. After completion, the material has sufficient strength to serve as a roadway along the embankment. Procedures for constructing soil-cement slope protection by the stairstep method can be found in "Suggested Specifications for Soil-Cement Slope Protection for Embankments (Central-Plant Mixing Method)," Portland Cement Association Publication IS052W.

When velocities exceed six to eight feet per second and the flow carries sufficient bed load to be abrasive, special precautions are advisable. The aggregates in this case should contain at least 30 percent gravel particles retained on a No. 4 (4.75 mm) sieve. It should be emphasized that soil-cement provides a rigid bank protection. The depth of the bank protection should be sufficient to protect the installation from the anticipated total scour.

A soil cement blanket with 8 to 15 percent cement may be an economical and effective streambank protection method for use in areas where vegetation is difficult to establish and the bank material is predominately sand. The sand can be mixed with cement by hand or mechanically to a depth of at least 4 inches. The mixture should then be wet down and allowed to set up. This method has the advantage of low cost. However, there are three major disadvantages: impermeability, low strength, and susceptibility to temperature variations. If the bank behind the blanket becomes saturated and cannot drain, failure may occur. Also, because a sand-cement blanket is relatively brittle, very little if any traffic (vehicular, pedestrian, or livestock) can be sustained without cracking the thin protective veneer. In northern climates the blanket can break up during freeze-thaw cycles.

5.3.11 Bulkheads

Bulkheads can be used to prevent streambank erosion or failure. As an additional benefit, a bulkhead may provide a substantial increase in waterfront area and an improvement in water/land access. Concrete, steel, timber and more recently, aluminum, corrugated asbestos, and used tires have been used to construct bulkheads. Concrete and steel bulkheads generally cost at least four time as much as a comparable bulkhead of another material; however, the service life is longer and less maintenance is required. Timber is the most commonly available material for economical bulkhead construction.

Timber bulkhead construction is similar to common fence construction except that a few precautions should be observed:

- All wood should be treated with preservative to minimize deterioration due to repetitive wetting and drying or insect activity.
- The toe of the bulkhead should always be protected with riprap. The most common cause of bulkhead failure is scour around the pilings, followed by the structure tipping over due to the pressure of the bank behind the bulkhead.
- Piles should be anchored to deadmen buried in the bank.
- Fill material placed between the bulkhead and natural bank should be free draining so that the soil behind the bulkhead will not become saturated and push the structure over.
- If there are no cracks between the planks, weepholes should be drilled in the fence at regular intervals to allow the bank to drain. Filter fabric or gravel can be placed as a filter behind openings in the fence to prevent fine soils from leaching through. A filter must be properly designed to match the filter with the soil.
- The bulkhead should be tied into the bank at the upstream and downstream end of the structure to prevent flow behind the bulkhead.

5.3.12. Protection of Banks and Training Works against Undermining

Lack of protection against undermining is a frequent cause of revetment failure. Basically four methods may be used to prevent undermining:

1. Excavate and continue the slope revetment down to a nonerodible material or to below the expected scour level. This method is the most permanent, but it may be impractical or uneconomical if deep scour is expected;

2. Drive a 'cut-off wall' of sheet piling from the toe of the revetment down to a nonerodible material or to below the expected scour level. Such walls are subject to risk of failure from earth pressure on the bank side after scour occurs on the channel side, and tend to cause deeper scour than paved slopes. The risk of failure resulting from unforeseen scour can be reduced by tying back the piling to deadmen or similar anchors;

3. Lay a flexible 'launching apron' horizontally on the bed at the foot of the revetment,

so that when scour occurs the materials will settle and cover the side of the scour hole on a natural slope. This method is recommended for cohesionless channel beds where deep scour is expected, as being generally the most economical.

Materials used for launching aprons include stone riprap, articulated concrete matting, concrete blocks, gabions, and wire mesh mattresses filled with stone. Stone riprap is most commonly used.

In cohesionless channel beds the design of stone aprons should be based on the stone launching to a slope of 1 upon 2. Model tests have indicated that such a slope is realistic for sand beds, but little field confirmation seems to have been reported.

Stone sizes should be determined as for slope revetment. The volume of stone should be sufficient to cover the final scoured slope to a thickness of 1.25 times the size of the largest stones in the specified grading. At the nose of a guide bank or spur, there should be sufficient stone to cover the final conical surface of the scoured slope. Piers should not be located within the launching apron slope unless it is unavoidable.

Launching aprons do not perform well on cohesive channel beds where scour occurs in the form of slumps with steep slip faces. In such cases bank revetment should be continued down to the expected worst scour level, and the excavation then refilled; and

4. Pave the entire bed across the bridge waterway opening. This method is economical only for relatively small streams. Scour tends to occur at the downstream edge of the paving unless this is tied into a natural nonerodible formation or unless an artificial stilling basin in formed. Stone sizes for riprap paving may be estimated with the aid of Section 5.2.

5.4 Flow Control Structures

A flow control structure is defined here as a structure, either within or outside a channel that acts as a countermeasure by controlling the direction, velocity, or depth of flowing water. Structures within this category are sometimes called "river training works". Among the most important properties of a flow control structure is its degree of permeability. An impermeable structure may deflect a current entirely, whereas a permeable structure may serve mainly to reduce water velocity. As used here, the term "permeable" means that a structure has definite openings through which water is intended to pass, such as openings between adjacent boards or pilings, or the meshes of wire. Structures made of riprap, or filled with riprap, have some degree of permeability, but these are classed as impermeable because they act essentially as impermeable barriers to a rapidly moving current of water.

Types of flow control structures are distinguished on Figure 5.4.1.

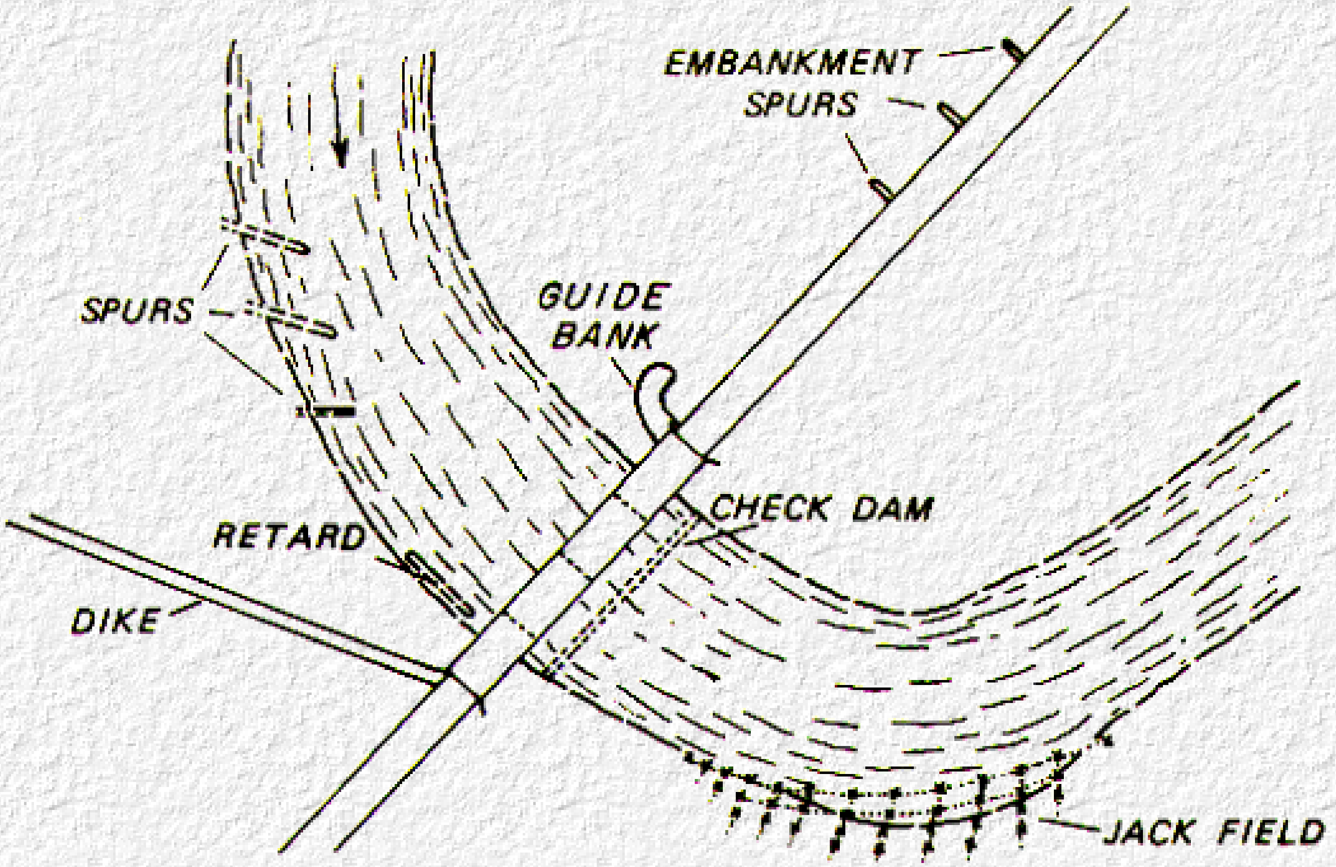


Figure 5.4.1. Placement of Flow Control Structures Relative to Channel Banks, Crossing, and Flood Plain. Spurs, Retards, Dikes, and Jack Fields May Be Either Upstream or Downstream from the Bridge (from Brice et al., 1978)

5.4.1 Spurs

A spur is a structure or embankment projected into a stream from the bank at some angle and for a short distance to deflect flowing water away from critical zones, to prevent erosion of the bank, and to establish a more desirable channel alignment or width. By deflecting the current from the bank and causing sediment deposits behind them, a spur or a series of spurs may protect the stream bank more effectively and at less cost than riprapping the bank. Also, by moving the location of any scour away from the bank, failure of the riprap on the spur can often be repaired before damage is done to structures along and across the rivers. Conversely, failure of riprap on the bank may immediately endanger structures.

Spurs are used to protect highway embankments that form the approaches to a bridge crossing. Often these highway embankments cut off the overbank flood flows causing these flows to run parallel to the embankment enroute to the bridge opening. Spurs constructed perpendicular to the highway embankment keep the potentially erosive current away from the embankment, thus protecting it. Spurs as used in this report encompass the terms dikes, jetties, groins, and spur dikes which are also used to describe these structures.

Spurs are also used to channelize a wide, poorly defined stream into a well-defined channel that neither aggrades nor degrades, thus maintaining its location from year to year. Spurs on streams with suspended sediment discharge can cause deposition to establish and maintain the new alignment. The use of spurs in this instance may decrease the length necessary for the bridge opening and may make a more suitable, stable channel approach to the bridge. This decreases the cost of the bridge structure.

The following major recommendations from Brown (1985) are organized by design component for easy reference.

- **Extent of Channelbank Protection**
  - A common mistake in streambank protection is to provide protection too far upstream and not far enough downstream.
  - The extent of bank protection should be evaluated using a variety of techniques, including: empirical methods, field reconnaissance, evaluation of flow traces for various flow stage conditions, and review of flow and erosion forces for various flow stage conditions. Information from these approaches should then be combined with personal judgment and a knowledge of the flow processes occurring at the local site to establish the appropriate limits of protection.
- **Spur Length**
  - As the spur length is increased:

    the scour depth at the spur tip increases,
    the magnitude of flow concentration at the spur tip increases,
    the severity of flow deflection increases, and
    the length of channel bank protection increases.
  - The projected length of impermeable spurs should be held to less than 15 percent of the channel width at bank-full stage.
  - The projected length of permeable spurs should be held to less than 25 percent of the channel width. However, this criterion depends on the magnitude of the spur's permeability. Spurs having permeabilities less than 35 percent should be limited to projected lengths not to exceed 15 percent of the channel's flow width. Spurs having permeabilities of 80 percent can have projected lengths up to 25 percent of the channel's bank-full flow width. Between these two limits, a linear relationship between the spur permeability and spur length should be used.
- **Spur Spacing**
  - The spacing of spurs in a bank-protection scheme is a function of the spur's length, angle, and permeability, as well as the channel bend's degree of curvature.
  - The direction and orientation of the channel's flow thalweg plays a major role in determining an acceptable spacing between individual spurs in a bank-stabilization scheme.

- Reducing the spacing between individual spurs below the minimum required to prevent bank erosion between the spurs results in a reduction of the magnitude of flow concentration and local scour at the spur tip.
- Reducing the spacing between spurs in a bank-stabilization scheme causes the flow thalweg to stabilize further away from the concave bank towards the center of the channel.
- A spacing criteria based on the projection of a tangent to the flow thalweg, projected off the spur tip, as presented in the above discussions, should be used.

- **Spur Angle/Orientation**
  - The primary criterion for establishing an appropriate spur orientation for the spurs within a given spur scheme is to provide a scheme that efficiently and economically guides the flow through the channel bend, while protecting the channel bank and minimizing the adverse impacts to the channel system.
  - Spurs angled downstream produce a less severe constriction of flows than those angled upstream or normal to flow.
  - The greater an individual spur's angle in the downstream direction, the smaller the magnitude of flow concentration and local scour at the spur tip. Also, the greater the angle, the less severe the magnitude of flow deflection towards the opposite channel bank.
  - Impermeable spurs create a greater change in local scour depth and flow concentration over a given range of spur angles than do permeable spurs. This indicates that impermeable spurs are much more sensitive to these parameters than are permeable spurs.
  - Spur orientation does not in itself result in a change in the length of channel bank protected for a spur of given projected length. It is the greater spur length parallel to the channel bank associated with spurs oriented at steeper angles that results in the greater length of channel bank protected.
  - Retardance spurs should be designed perpendicular to the primary flow direction.
  - Retardance/diverter and diverter spurs should be designed to provide a gradual flow training around the bend. This is accomplished by maximizing the flow efficiency within the bend while minimizing any negative impacts on the channel geometry.
  - The smaller the spur angle, the greater the magnitude of flow control as represented by a greater shift of the flow thalweg away from the concave (outside) channel bank.
  - It is recommended that spurs within a retardance/diverter or diverter spur scheme be set with the upstream-most spur at approximately 150 degrees to the main flow current at the spur tip, and with subsequent spurs having incrementally smaller angles approaching a minimum angle of 90 degrees at the downstream end of the scheme.

- **Spur Height**
    - The spur height should be sufficient to protect the regions of the channel bank impacted by the erosion processes active at the particular site.
    - If the design flow stage is lower than the channel bank height, spurs should be designed to a height no more than three feet lower than the design flow stage.
    - If the design flow stage is higher than the channel bank height, spurs should be designed to bank height.
    - Permeable spurs should be designed to a height that will permit the passage of heavy debris over the spur crest and not cause structural damage.
    - When possible, impermeable spurs should be designed to be submerged by approximately three feet under their worst design flow condition, thus minimizing the impacts of local scour and flow concentration at the spur tip and the magnitude of flow deflection.
- **Spur Crest Profile**
    - Permeable spurs should be designed with level crests unless bank height or other special conditions dictate the use of a sloping crest design.
    - Impermeable spurs should be designed with a slight fall towards the spur head, thus allowing different amounts of flow constriction with stage (particularly important in narrow-width channels), and the accomodation of changes in meander trace with stage.
- **Channel Bed and Channel Bank Contact**
    - Careful consideration must be given to designing a spur that will maintain contact with the channel bed and channel bank so that it will not be undermined or outflanked.
- **Spur Head Form**
    - A simple straight spur head form is recommended.
    - The spur head or tip should be as smooth and rounded as possible. Smooth, well-rounded spur tips help minimize local scour, flow concentration, and flow deflection.

5.4.2 Hardpoints

Hardpoints are an erosion control technique consisting of stone fills spaced along an eroding bank line (Figure 5.4.2). The structures protrude only short distances into the river channel and are supplemented with a root section extending landward into the bank to preclude flanking, should excessive erosion persist. The majority of the structure cannot be seen as the lower part consists of rock placed underwater, and the upper part is covered with topsoil and seeded with native vegetation. The structures are especially adaptable in long, straight reaches not subject to direct attack.

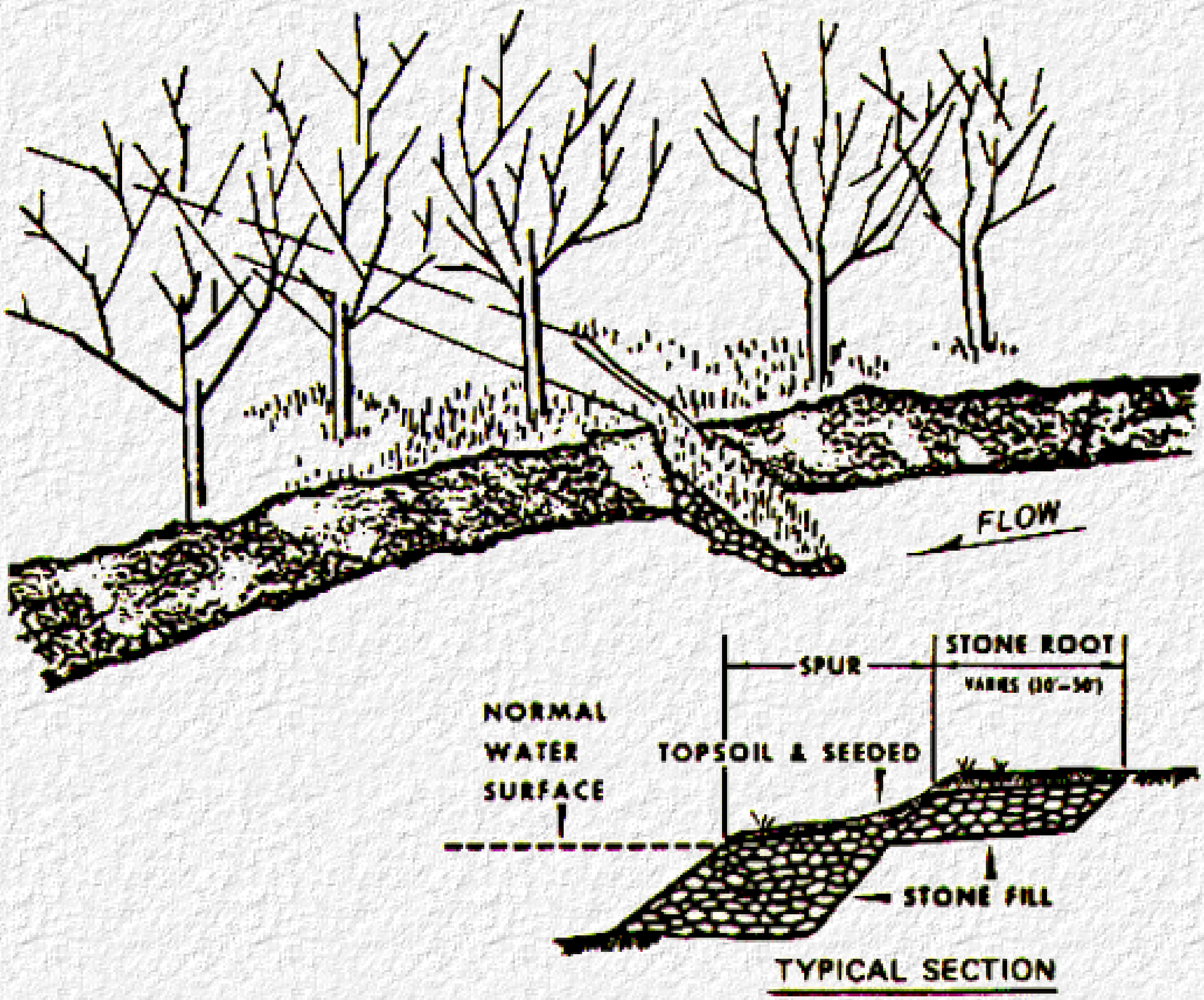


Figure 5.4.2. Perspective of Hard Point with Section Detail (after Brown, 1985)

5.4.3 Retards

Retards are devices placed parallel to embankments and river banks to decrease the stream velocities and prevent erosion.

- **Pile retards** can be made of concrete, steel or timber. The design of timber pile retards is essentially the same as timber pile dikes shown in Figure 5.4.5. They may be used in combination with bank protection works such as riprap. The retard then serves to reduce the velocities sufficiently so that either smaller riprap can be used, or riprap can be eliminated.

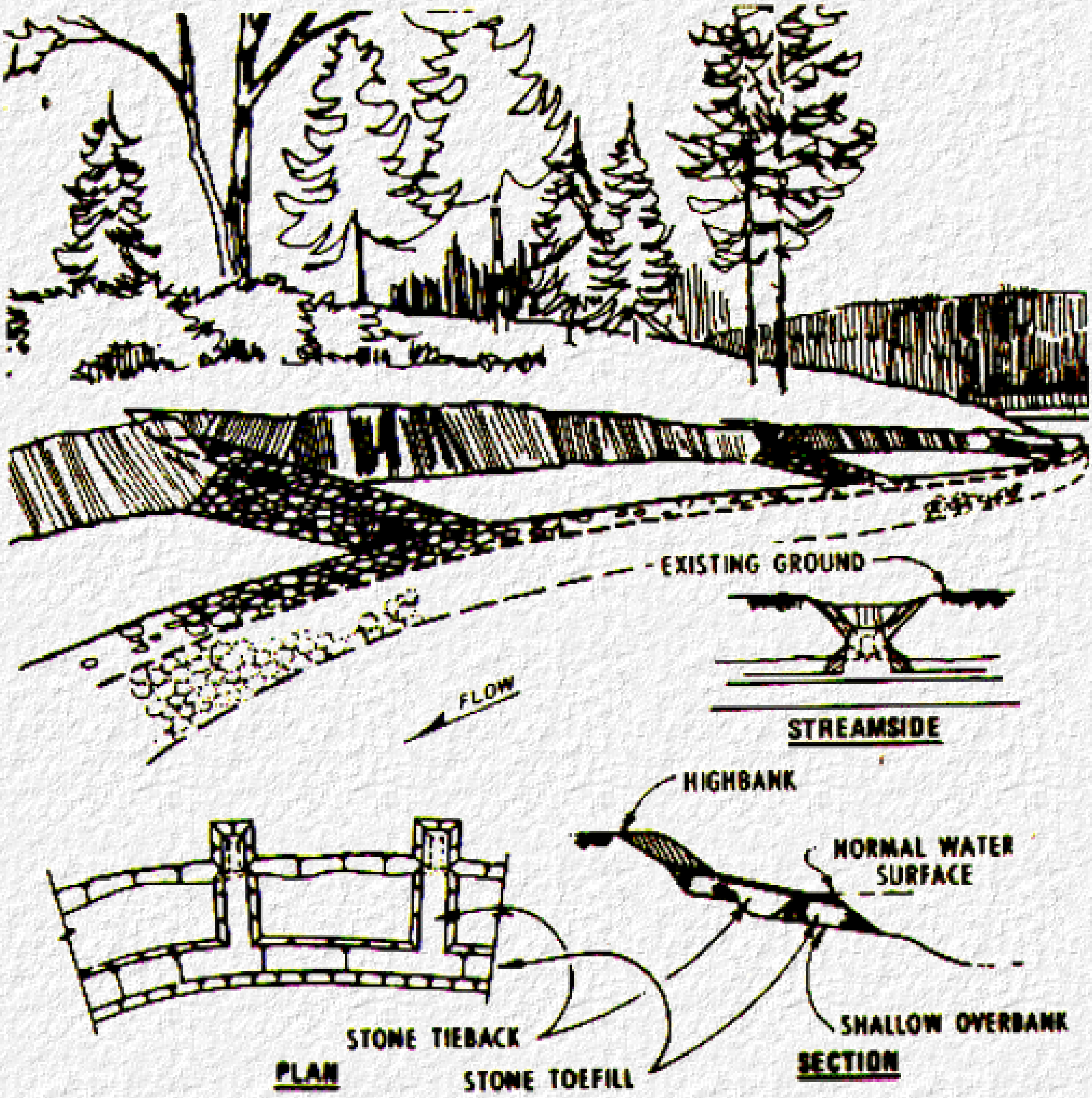


Figure 5.4.3. Retard

- **Timber or concrete cribs** - Timber and concrete cribs are sometimes used for bulkheads and retaining walls to hold highway embankments, particularly where lateral encroachment into the river must be limited. Cribs are made up by interlocking pieces together in the manner shown in Figure 5.4.4. The crib may be slanted or vertical depending on height and the crib is filled with rock or earth. Reinforced concrete retaining walls are alternatives to timber cribs which can be considered. However, concrete retaining walls are expensive and are generally only used in special confined locations where space precludes other methods of bank protection. In constructing concrete retaining walls drainage holes (weep holes) must be provided. The foundation of these walls should be placed below expected scour depths.

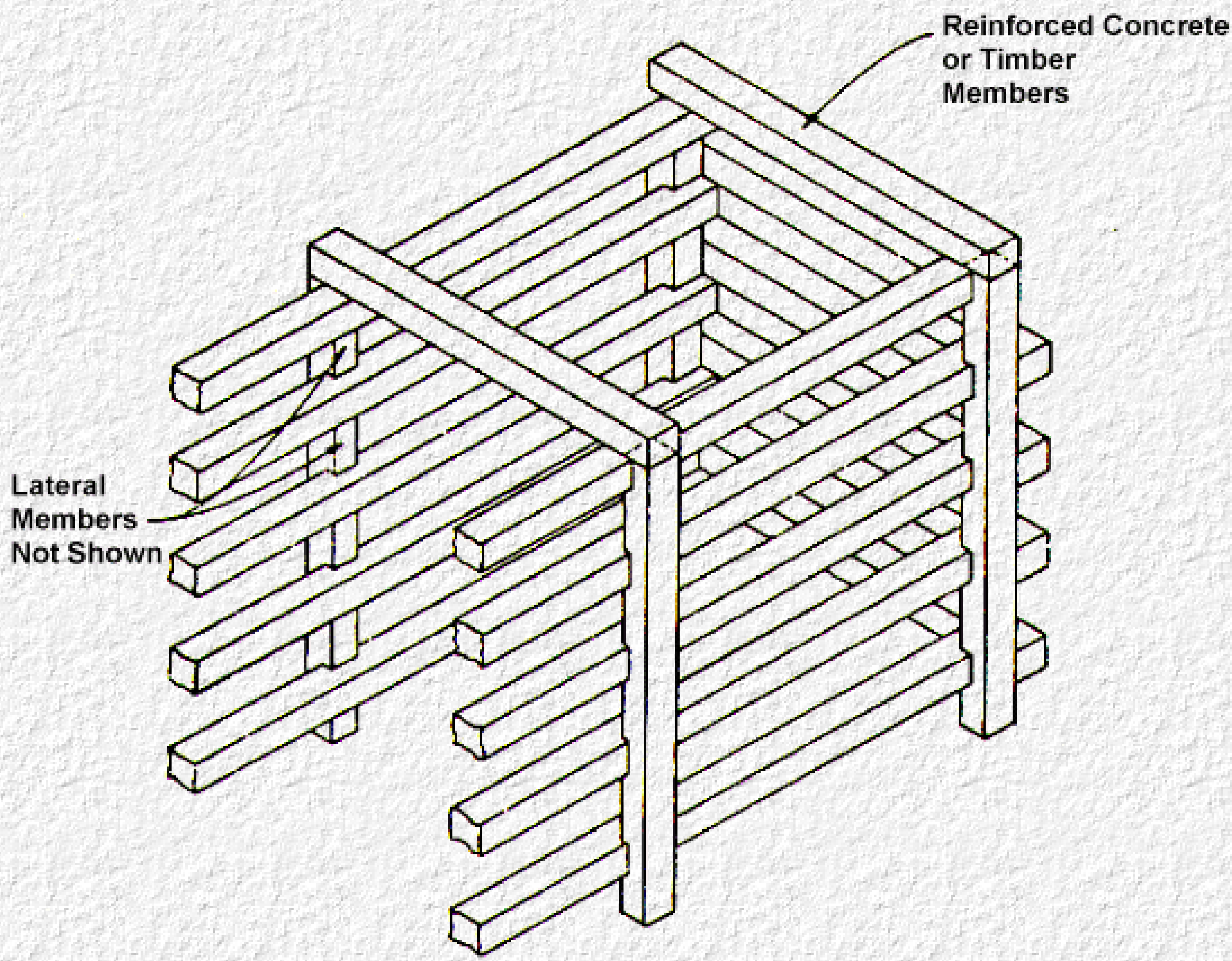
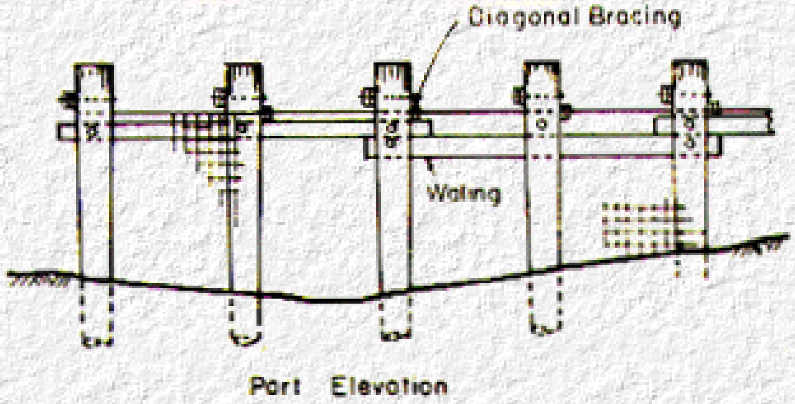


Figure 5.4.4. Concrete or Timber Cribs

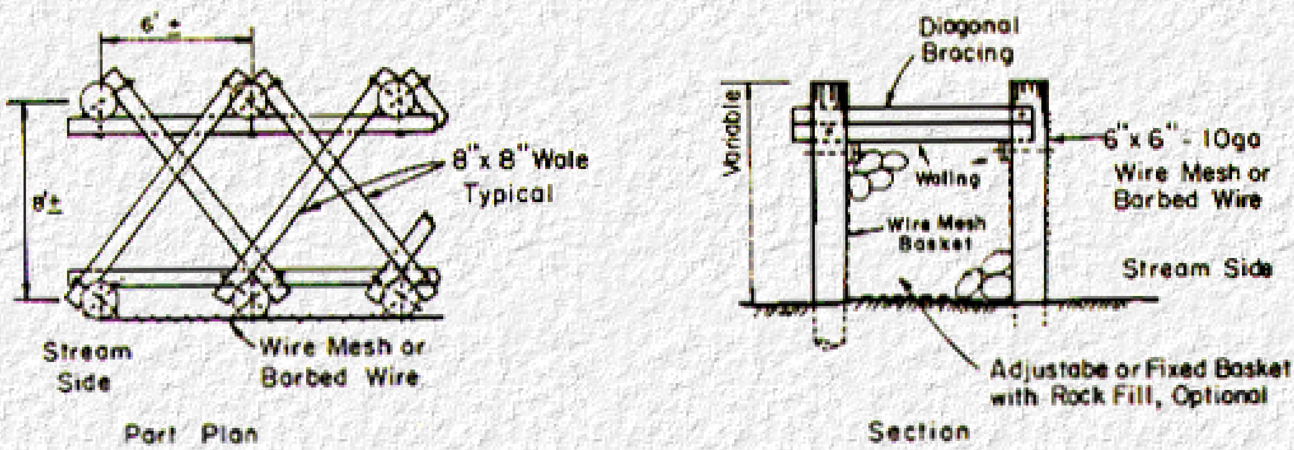
5.4.4 Dikes

There are two principal types of dikes, permeable and impermeable. Permeable dikes are those which permit flow through the dike but at reduced velocities, thereby preventing further erosion of the banks and causing deposition of suspended sediment from the flow.

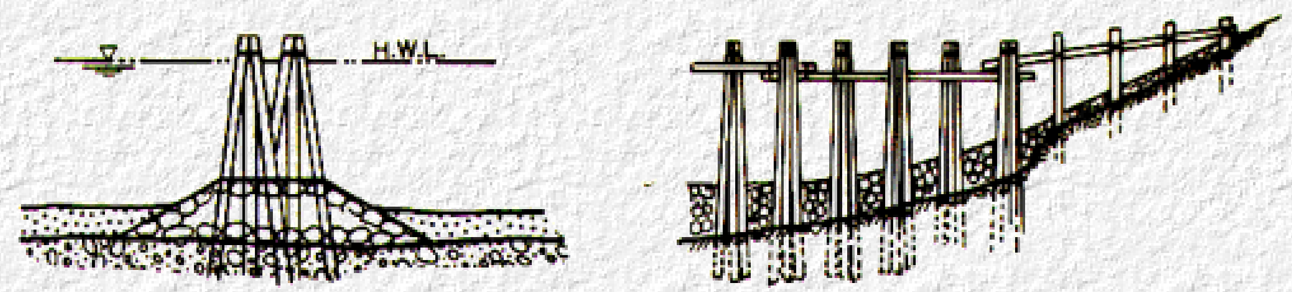
- **Timber or steel pile dikes** - Pile dikes (also retards) may consist of closely-spaced single, double, or multiple rows. There are a number of variations to this scheme. For example, wire fence may be used in conjunction with pile dikes to collect debris and thereby cause effective reduction of velocity. Double rows of piles can be placed together to form cribs, and rocks may be used to fill the space between the piles. Pile dikes are vulnerable to failure through scour. This can be overcome if the piles can be driven to a large depth to achieve safety from scour, or the base of the piles can be protected from scour with dumped rock in sufficient quantities. The various forms of pile dikes are illustrated in Figure 5.4.5.



(a) Single row timber pile with wire fence



(b) Double row timber piles with rocks and wire fence



(c) Pile clusters

Figure 5.4.5. Pile Dikes (Retards Would Be Similar)

The arrangement of piles depends upon the velocity of flow, quantity of suspended sediment transport, and depth and width of the river. If the velocity of flow is large, pile dikes are not likely to be very effective. Stabilization of the bank by other methods should be considered. On the other hand, in moderate flow velocities with high concentrations of suspended sediments, these dikes can be quite effective. Deposition of suspended sediments in the pile dike field is a necessary consequence of reduced velocities. If there is not sufficient concentration of suspended sediment in the flow, or the velocities in the dike fields are too large for deposition, the permeable pile dikes will only partially be effective in training the river and protecting the bends.

The length of each dike depends on channel width, position relative to other dikes, flow depth and available pile lengths. Generally, pile dikes are not used in large rivers where depths are great, although timber pile dikes have been used in the Columbia River. On the other hand, banks of wide shallow rivers can be successfully protected with dikes. The spacing between dikes varies from 3 to 20 times the length of the upstream dike, with closer spacing favored for best results.

- **Stone-fill dikes** - Stone-fill dikes are classed as impermeable dikes and do not depend on deposition of sediment between dikes nearly as much as permeable dikes. The principal function is to deflect the flow away from the bank and the dikes must be long enough to accomplish this purpose. The dikes may be angled downstream, angled upstream, or constructed normal to the bank. Variations such as a sloping dike, with declining top elevation away from the bank, L or T head dikes, and curved dikes have been used. Stone-fill dikes are illustrated in Figure 5.4.6.

  The spacing between dikes may vary from three or four dike lengths to 10 or 12 dike lengths depending upon velocity and depth. Short dikes with long spacing are generally not useful for bank protection unless jacks or riprap are used to protect the bank between them.

  The ends of the dikes are subjected to local scour and appropriate allowance should be made for loss of dike material into the scour hole. The size of rock to be used for the dike depends on availability of material. Large rocks are generally used to cover the surface, while the internal section may be constructed with smaller rocks or earthfill. Side slopes of 1.5:1 and 2:1 are common.

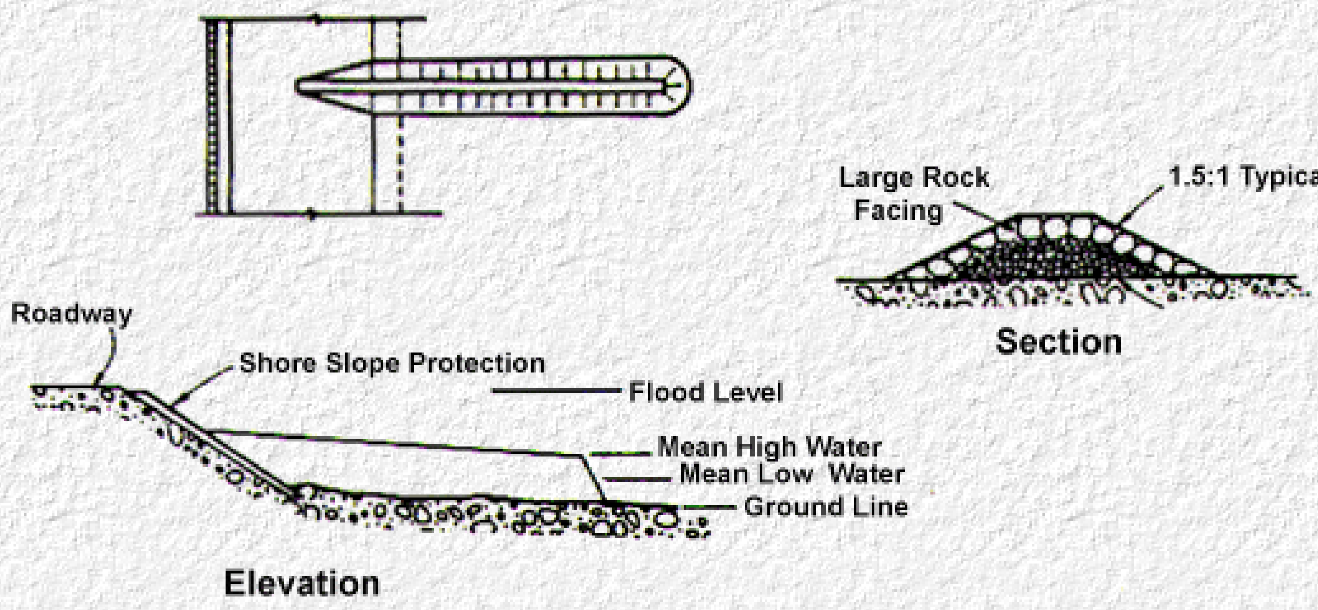


Figure 5.4.6. Typical Stone Fill Dike

- **Vane Dikes** are low-elevation structures designed to guide the flow away from an eroding bank line (Figure 5.4.7). The structures can be constructed of rock or other erosion-resistant material, the tops of which are constructed below the design water surface elevation and would not connect to the high bank. Water would be free to pass over or around the structure with the main thread of flow directed away from the eroding bank. The structures will discourage high erosive velocities next to an unprotected bank line, encourage diversity of various channel depths, and protect existing natural bottomland characteristics. The findings from a model investigation of these structures include the effects of various vane dike orientation, vane dike length, and gap length (U.S. Army Corps of Engineers, 1981).

Figure 5.4.7. Vane Dike Model, Ground Walnut Shell Bed, during Low Sage Portion of Test Run, Elevation 1.482 ft (U.S. Army Corps of Engineers, 1981)

5.4.5 Jetties

The purpose of a jetty field is to add roughness to a channel or overbank area to train the main stream along a selected path. The added roughness along the bank reduces the velocity and protects the bank from erosion. Jetty fields are usually made up of steel jacks tied together with cables. Both lateral and longitudinal rows of jacks are used to make up the jetty field as shown in Figure 5.4.8.

The lateral rows are usually angled about 45 to 70 degrees downstream from the bank. The spacing varies, depending upon the debris and sediment content in the stream, and may be 50 to 250 feet apart. Jetty fields are effective only if there is a significant amount of debris carried by the stream and the suspended sediment concentration is high.

When jetty fields are used to stabilize meandering rivers, it may be necessary to use jetty fields on both sides of the river channel because in flood stage the river may otherwise develop a chute channel across the point bar. A typical layout is shown in Figure 5.4.8.

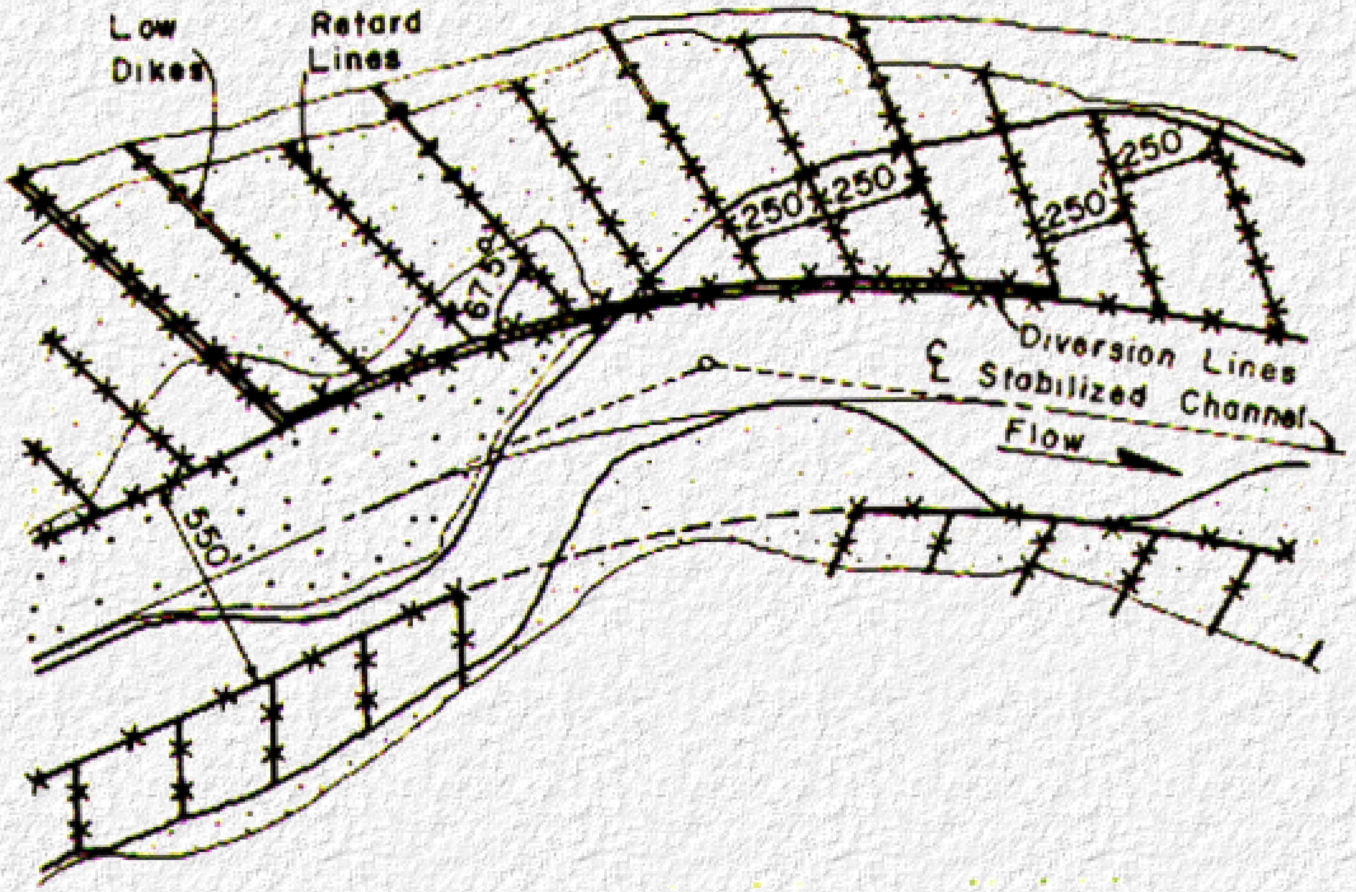
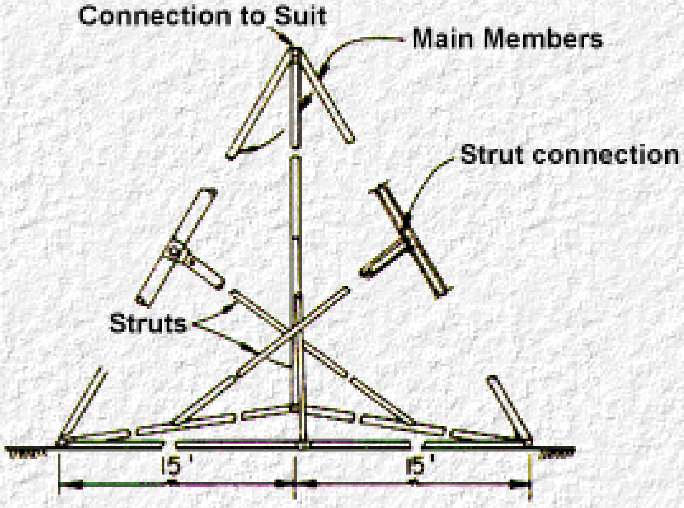


Figure 5.4.8. Typical Jetty-Field Layout

Steel jacks are devices with basic triangular frames tied together to form a stable unit. The resulting framework is called a tetrahedron. The tetrahedrons are placed parallel to the embankment and cabled together with the ends of the cables anchored to the bank. Wire fencing may be placed along the row of tetrahedrons. In order to function well, there must be considerable debris in the stream to collect on the fence and the suspended sediment concentration must be large so that there will be deposition behind the retard. Various forms of steel jacks may be assembled. Two types are shown in Figure 5.4.9. Tiebacks should be spaced every l00 feet and space between jacks should not be greater than their width.



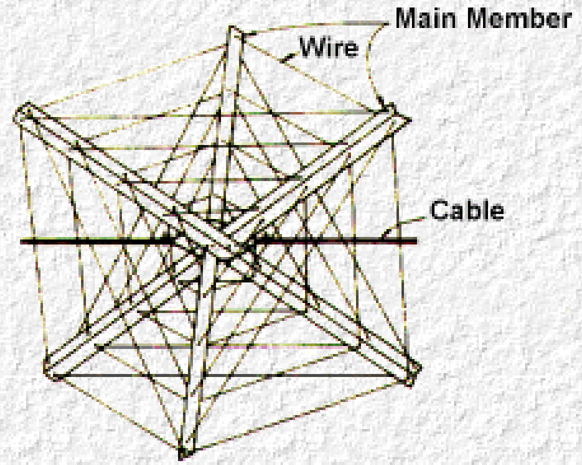


Figure 5.4.9. Steel Jacks

5.4.6 Fencing

Fencing can be used as a low-cost bank protection technique on small to medium size streams. Special structural design considerations are required in areas subject to ice and floating debris. Both longitudinal (parallel to stream) fence retards and transverse (perpendicular to stream) fences have been used in the prototype with varying degrees of success. A model investigation and literature review of longitudinal fence retards with tiebacks were conducted to identify the following important design considerations:

1. Channel gradient must be stable and not be steep (tranquil flow);

2. Toe scour protection can be provided by extending the support posts well below the maximum scour expected or by placing loose rock at the base of the fence to launch downward if scour occurs at the toe;

3. Tiebacks to the bank are important to prevent flanking of the fence and to promote deposition behind the fence;

4. Fence retards generally reduce attack on the bank so that vegetation can establish; and

5. Metal or concrete fences are preferred due to ice damage and fire loss of wooden fences.

5.4.7 Guidebanks

Guide banks are placed at or near the ends of approach embankments to guide the stream through the bridge opening. Constructed properly, flow disturbances, such as eddies and cross-flow, will be minimized to make a more efficient waterway under the bridge. They are also used to protect the highway embankment and reduce or eliminate local scour at the embankment and adjacent piers. The effectiveness of guidebanks is a function of river geometry, quantity of flow on the floodplain, and size of bridge opening. A typical guidebank at the end of an embankment is shown in Figure 5.4.10.

The recommended shape of a guidebank is a quarter ellipse with a major to minor axis ratio of 2.5. The major axis should be approximately parallel to the main flow direction. For bridge crossings normal to the river, the major axis would be normal to the highway embankment. However, for skewed crossings, the guidebank should be placed at an angle with respect to the embankment with the view of streamlining the flow through the bridge opening. An illustration of guidebanks for a skewed crossing is shown in Figure 5.4.11 and design dimensions recommended by Karaki are shown in Figure 5.4.12.

The length of the spur dike, L_s, required depends upon quantity of flow on the floodplain, width of bridge opening and skewness of the highway crossing. Shorter spur dikes may be used where floodplain flow is small or scour potential at piers and embankment ends are small.

The upstream and downstream lengths for straight guidebanks are as follows: the upstream length = 0.75 to 1.5 times the width of the opening; and the downstream length = 0.1 to 0.25 times the width of the opening. It is not necessary that both guidebanks on the upstream side be the same length. For some flow conditions a short curved guidebank on one side and a long straight bank on the other may be the best solution.

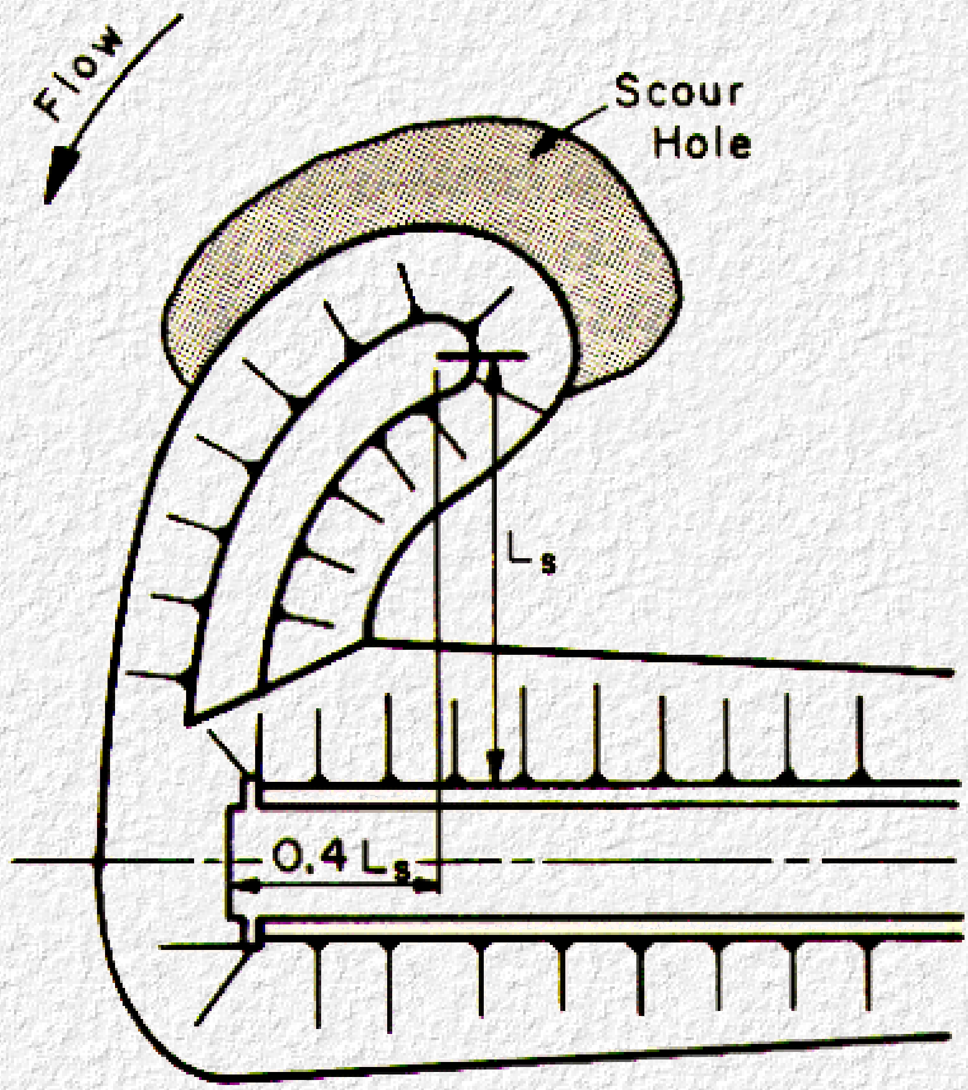


Figure 5.4.10. Guidebank

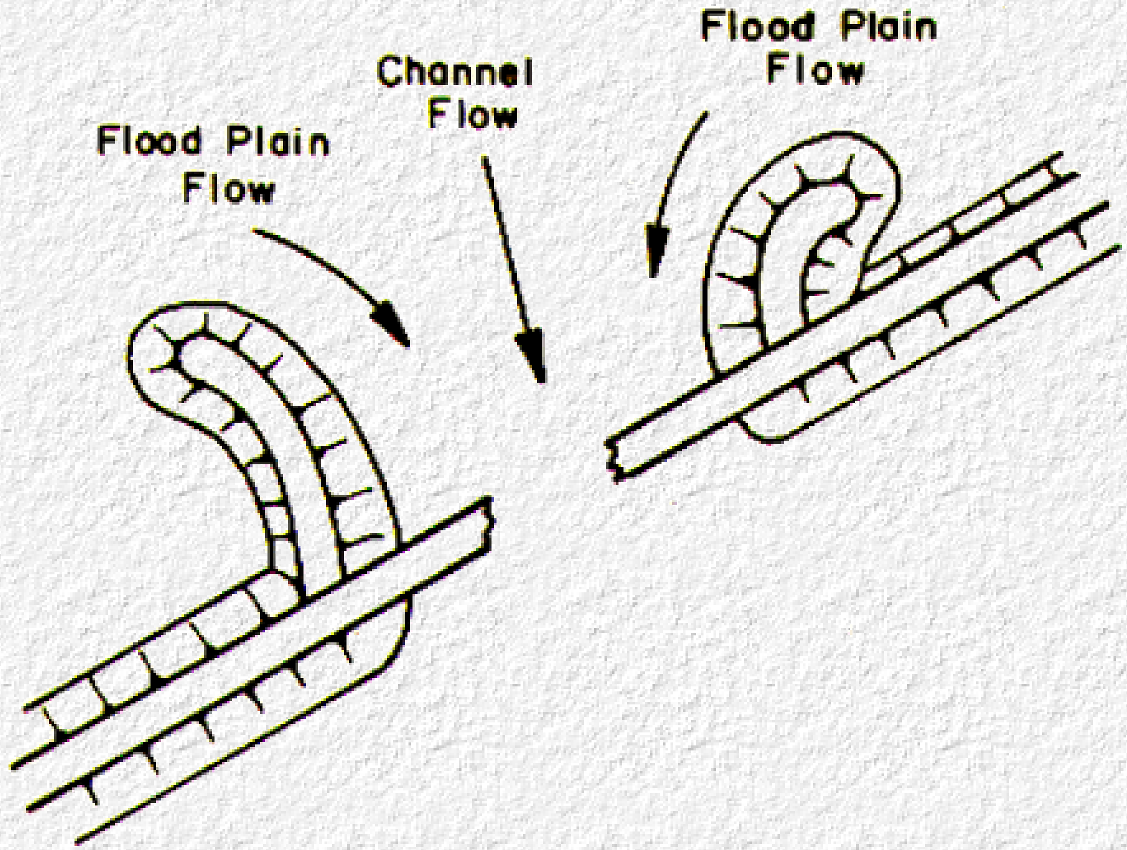


Figure 5.4.11. Guidebank at Skewed Highway Crossing

The crest elevation should be 1 ft higher than the elevation of the design flood taking into consideration the effect of the contraction of the flow; this is because the design flow should not overtop the guidebank.

Beside erosion protection, guidebanks provide a more efficient (less head loss) flow of water through a bridge opening. They also decrease scour depth and move the scour energy away from the abutments.

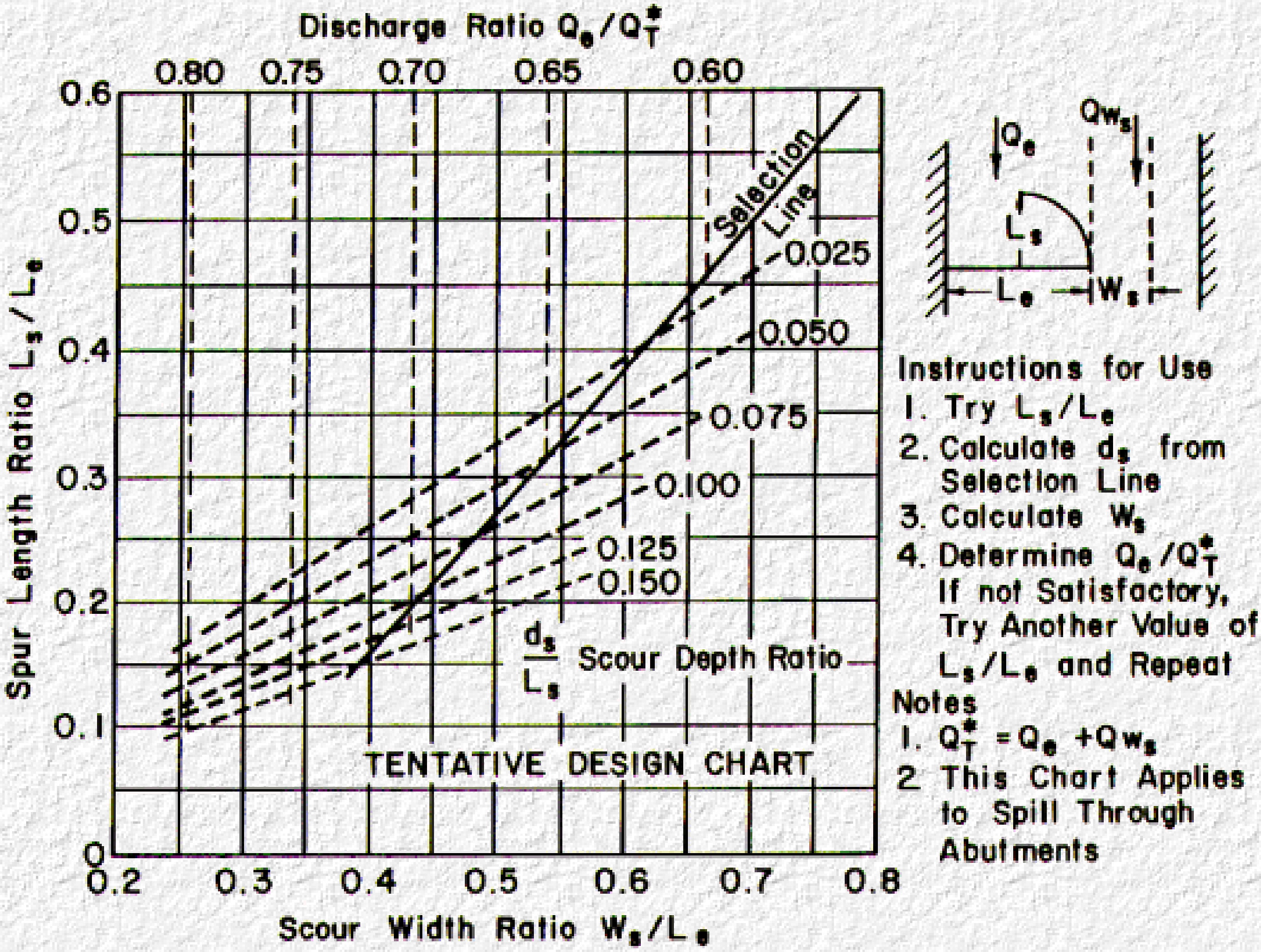


Figure 5.4.12. Guidebank Design Procedure (from Karaki, 1959)

5.4.8 Drop Structures

Drop structures are useful to reduce the slope of a channel. Concrete, soil-cement, gabion sheet pile or timber crib drop structures can be designed considering the stability of the structure and the depth of the scour hole at the toe of the structure. A riprap blanket upstream of the structure is also quite effective as shown in Figure 5.4.13 for a soil cement drop structure. Definition sketches for a vertical wall and sloping silldrop structure are shown in Figure 5.4.14 and Figure 5.4.15. The design of vertical wall or sloping sill structures are given in texts by Peterson (1986), Simons, Li and Associates (1982).

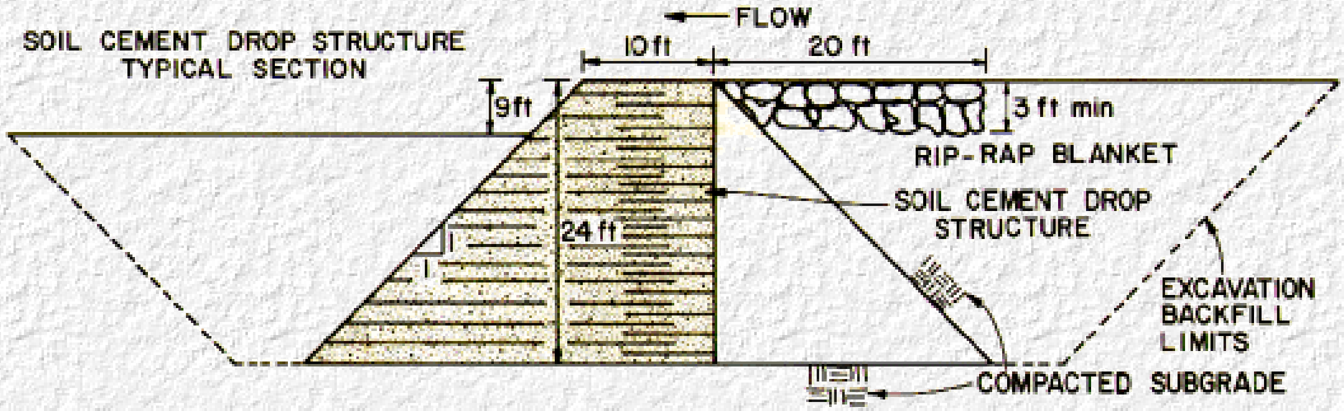


Figure 5.4.13. Design Example of a Soil Cement Drop Structure

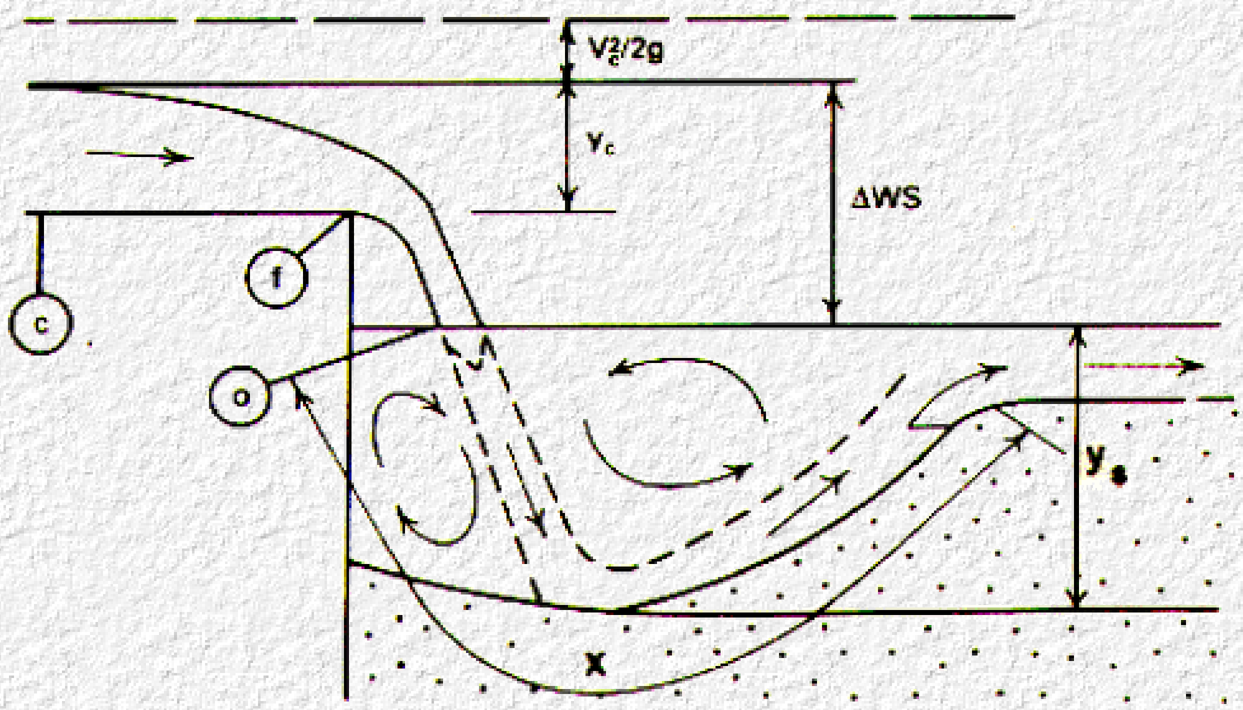


Figure 5.4.14. Flow and Scour Patterns at a Vertical Wall (after Laursen and Flick, 1983)

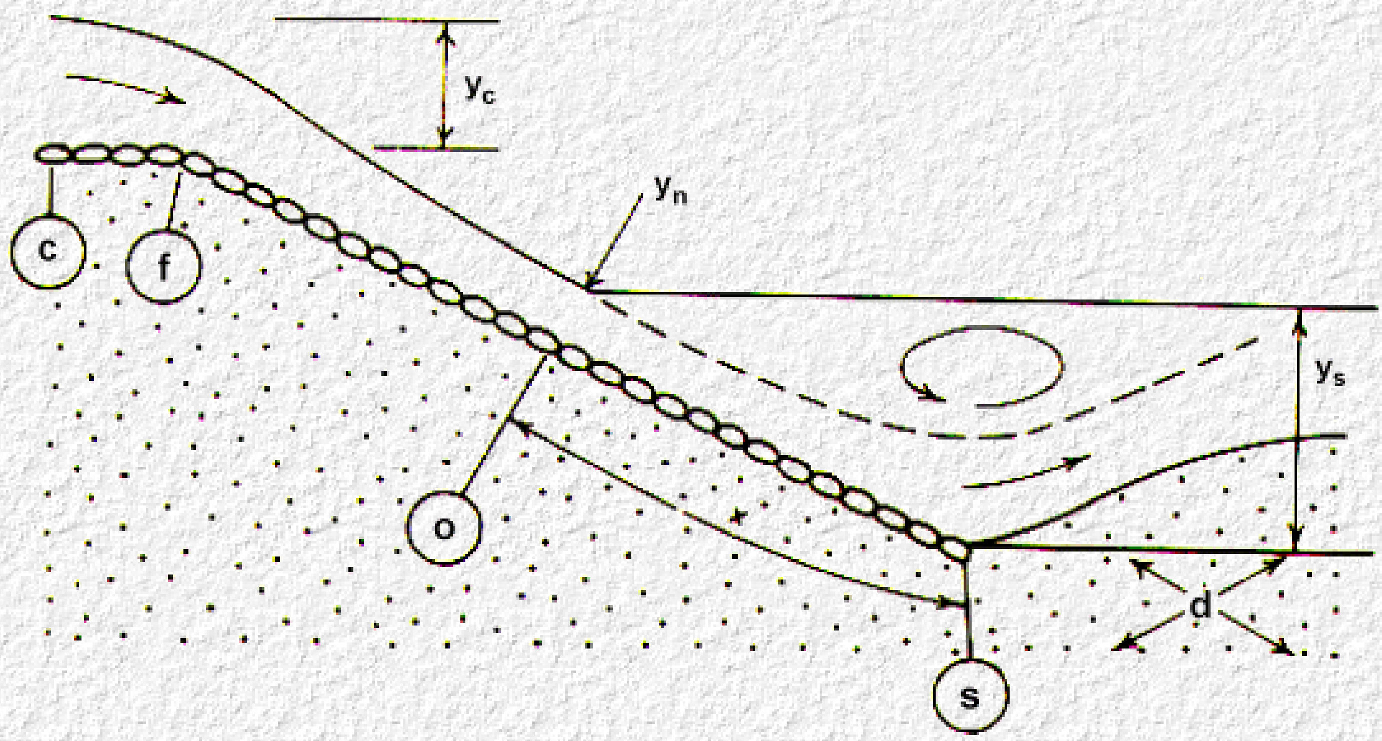


Figure 5.4.15. Flow and Scour Patterns at a Sloping Sill (after Laursen and Flick, 1983)

Go to Chapter 5 (Part III)

Chapter 5 : HIRE
River Stabilization, Bank Protection and Scour Part III

[Go to Chapter 6]

5.5 Bridge Scour

5.5.1 General

All material in a stream bed will erode. It is just a matter of time. However, some material such as granite may take hundred's of years to erode. Whereas, sandbed streams will erode to the maximum depth of scour in hours. Sandstone, shales, and other sedimentary bedrock materials, although they will not erode in hours or even days will, over time, if subjected to the erosive forces of water, erode to the extent that a bridge will be in danger unless the substructures are founded deep enough. Cohesive bed and bank material such as clays, silty clays, silts and silty sands or even coarser bed material such as glacial tills, which are cemented by chemical action or compression, will erode if subjected to the forces of flowing water. The erosion of cohesive and other cemented material is slower than sand bed material but their ultimate scour will be as deep if not deeper than the scour depth in a non-cohesive sandbed stream. It might take the erosive action of several major floods but ultimately the scour hole will be equal to or greater in depth than with a sand bed material.

This does not mean that every bridge foundation must be buried below the calculated scour depth determined for non-bedrock streams. But it does mean that so called bed rock streams must be carefully evaluated.

5.5.2 Total Scour

Total scour at a highway crossing is composed of three components. In general the components are additive. These components are:

(1) Long Term Aggradation or Degradation

The change in river bed elevation (aggradation or degradation) over long lengths and time due to changes in controls, such as dams, changes in sediment discharge, head cuts and changes in river geomorphology, such as changing from a meandering to a braided stream. May be natural or man induced.

(2) General Scour and Contraction Scour

The scour that results from the acceleration of the flow due to either a natural or bridge contraction or both (contraction scour). General scour may also result from the location of the bridge on the stream. For example, its location with respect to a stream bend or its location upstream from the confluence with another stream. In this latter case, the elevation of the downstream water surface will affect the backwater on the bridge, hence, the velocity and scour. General scour may happen during the passage of a flood and the stream may fill in on the falling stage. This type of scour involves the removal of material from the bed and banks across all or most of the width of a channel.

(3) Local Scour

The scour that occurs at a pier or abutment as the result of the pier or abutment obstructing the flow. These obstructions to the flow accelerate it and create vortexes that remove the material around them.

Generally, scour depths from local scour are much larger than the other two. Often by a factor of ten. But, if there are major changes in the stream conditions, such as a large dam built upstream or downstream of the bridge or severe straightening of the stream, long-term bed elevation changes can be the larger element in the total scour.

(4) Laterial Shifting of the Stream

In addition to the above, lateral shifting of the stream may also erode the approach roadway to the bridge and, by changing the angle of the flow in the waterway at the bridge crossing, change the total scour.

5.5.3 Long-Term Bed Elevation Changes

Long-term bed elevation changes (aggradation or degradation) may be the natural trend of the stream or may be the result of some modification to the stream or watershed condition.

The stream bed may be aggrading, degrading or not changing (equilibrium) in the bridge crossing reach. When the bed of the stream is neither aggrading or degrading it is in equilibrium with the sediment discharge supplied to the bridge reach and the elevation of the bed does not change. In this section we are considering long-term trends not the cutting and filling of the bed of the stream that might occur during a runoff event. A stream may cut and fill during a runoff event and also have a long-term trend of an increase or decrease in bed elevation. The problem for the engineer is to determine what the long-term bed elevation changes will be during the life time of the structure. What is the current rate of change in the stream bed elevation? Is the stream bed elevation in equilibrium? Is the stream bed degrading? Is it aggrading? Is there a head cut or nickpoint moving upstream? What is the future trend in the stream bed elevation?

During the life of the bridge the present trend may change. These long-term changes are the result of modifications of the state of the stream or watershed. Such changes may be the result of natural processes of the result of man's activities. The engineer must assess the present state of the stream and watershed and determine future changes in the river system and from this assessment determine the long-term stream bed elevation.

Factors that affect long-term bed elevation changes are: dams and reservoirs (up or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoff of a meander bend (natural or man made), changes in the downstream base level (control) of the bridge reach, gravel mining from the stream bed, diversion of water into or out of the stream, natural lowering of the total system, movement of a bend, bridge location in reference to stream planform and stream movement in relation to the crossing. In Chapter 7 examples of long-term bed elevation changes are given.

Analysis of long-term stream bed elevation changes must be made using the principals of river mechanics in the context of a fluvial system analysis. Such analysis of a fluvial system requires the consideration of all influences upon the bridge crossing, i.e., runoff from the watershed to the channel (hydrology), the sediment delivery to the channel (erosion), the sediment transport capacity of the channel (hydraulics) and the response of the channel to these factors (geomorphology and river mechanics). Many of the largest impacts are from man's activities, either in the past, the present or the future. This analysis requires a study of the past history of the river and man's activities on it; a study of present water and land use and stream control activities and finally contacting all agencies involved with the river to determine future changes in the river.

A method to organize such an analysis is to use a three level fluvial system approach. This method provides three levels of detail in an analysis, they are (1) a qualitative determination based on general geomorphic and river mechanics relationships; (2) engineering geomorphic analysis using established qualitative and quantitative relationships to establish the probable behavior of the stream system to various scenarios of future conditions; and (3) quantify the changes in bed elevation using available physical process mathematical models such as HEC-6, straight line extrapolation of present trends and engineering judgment to be expected as the result of the changes in the stream and watershed. Methods to be used in stage 1 and 2 are given in this manual and recent FHWA reports such as Stream Channel Degradation and Aggradation: Analysis of Impacts to Highway Crossings (Brown, et al., 1981).

5.5.4 General Scour

General scour at a bridge can be caused by a decrease in channel width, either naturally or by the bridge, which decreases flow area and increases velocity. This is contraction scour. General scour can also be caused by short-term (daily, weekly, yearly or seasonally) changes in the downstream water surface elevation that controls the backwater and hence the velocity through the bridge opening. Because this scour is reversible it is included in general scour rather than in long-term scour. General scour can result from the location of the bridge with regard to a bend. If the bridge is located on or close to a bend the concentration of the flow on the outer part of the channel can erode the bed.

General scour can be cyclic. That is, during a runoff event the bed scours during the rise in stage (increasing discharge) and fills on the falling stage (deposition).

General scour from a contraction occurs when the flow area of a stream is decreased from the normal either by a natural constriction or by a bridge. With the decrease in flow area there is an increase in average velocity and bed shear stress. Hence, there is an increase in stream power at the contraction and more bed material is transported through the contracted reach than is transported into the reach. The increase in transport of bed material lowers the bed elevation. As the bed elevation is lowered, the flow area increases and the velocity and shear stress decreases until equilibrium between the bed material that is transported into the reach is equal to that which is transported out of the reach.

The contraction of the flow by the bridge can be caused by a decrease in flow area of the stream channel by the abutments projecting into the channel and/or the piers taking up a large portion of the flow area. Also, the contraction can be caused by the approaches to the bridge cutting off the overland flow that normally goes across the flood plain during high flow. This latter case causes clear-water scour at the bridge section because the overland flow normally does not transport any bed material sediments. This clear-water picks up additional sediment from the bed when it returns to the bridge crossing. In addition, if it returns to the stream channel at an abutment it increases the local scour there. A guide bank at that abutment decreases the risk from scour of that abutment from this returning overbank flow. Also, relief bridges in the approaches, by decreasing the amount of flow returning to the natural channel, decrease the scour problem at the bridge cross section.

Other Factors that can cause contraction scour are:

(1) a natural stream constriction;

(2) long approaches over the flood plain to the bridge;

(3) ice formation or jams;

(4) berm forming along the banks by sediment deposits;

(5) island or bar formations upstream or downstream of the bridge opening;

(6) debris; and

(7) the growth of vegetation in the channel or flood plain.

To determine the magnitude of general scour from a variable backwater requires a study of the stream system to (1) determine if there will be variable backwater and if this condition exists to (2) determine the magnitude of general scour for this condition. Of particular value in determining if backwater affects exist and the magnitude of the affects on the velocity and depth is the WSPRO computer model. The difference in depth between the highest expected bed elevation and the lowest expected bed elevation for the design discharge is the value of the general scour.

General scour of the bridge opening may be concentrated in one area. If the bridge is located on or close to a bend the scour will be concentrated on the outer part of the bend. In fact there may be deposition on the inner portion of the bend, further concentrating the flow, which increases the scour at the outer part of the bend. Also, at bends the thalweg (the part of the stream where the flow or velocity is largest) will shift toward the center of the stream as the flow increases. This can increase scour and the non-uniform distribution of the scour in the bridge opening.

Often the magnitude of general scour cannot be predicted and inspection is the solution for general scour problems. Also, a physical model study can be used to determine general scour.

(1) Predicting Contraction Scour

There are several methods for estimating the magnitude of contraction scour. These will be given in this section. Unfortunately, the equations are based on laboratory studies with very little field data.

Contraction scour can be caused by different bridge site conditions. There are 4 main conditions (cases), as shown in Figure 5.5.1 which are as follows:

Case 1. Overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge. The bridge and/or channel width is narrower than the normal stream width.

Case 2. The normal river channel width becoming narrower either because of the bridge itself or by the bridge site being on a narrower reach of the river.

Case 3. A relief bridge in the overbank area. With little or no bed material transport in the overbank area.

Case 4. A relief bridge over a secondary stream in the overbank area.

Case 1

Overbank flow on a flood plain being forced back to the main channel by the approaches to the bridge. The bridge and/or the channel width is narrower than the normal stream width. Laursen's (1960) equation given below is used to predict the depth of scour in the contracted section y_2.

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{A} \left(\frac{n_2}{n_1}\right)^{B} \tag{5.5.1}$$

$y_s = y_2 - y_1$ Average scour depth

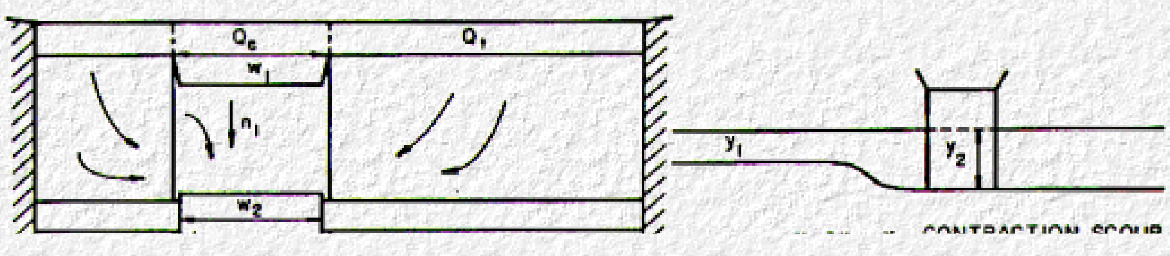
Where

y_1 = average depth in the main channel
y_2 = average depth in the contracted section
W_1 = width of the main channel
W_2 = width of the contracted section
Q_t = flow in the contracted section
Q_c = flow in the main channel
n_2 = Manning n for contracted section
n_1 = Manning n for main channel

A and B are transport coefficients from the following:

V_{*c}/ω	e	A	B	Mode of Bed Material Transport
< 0.5	0.25	0.59	0.066	mostly contact load
1.0	1.0	.64	.21	some suspended bed material
>2.0	2.25	.69	.37	mostly suspended bed material

$V_{*c} = (gy_1S_1)^{0.5}$, shear velocity
w = fall velocity of D_{50} of bed material
g = gravity constant, 32.2
S_1 = slope, energy grade line main channel
A = 6 (2+e)/7 (3+e)
B = 6e/7 (3+e)
e = transport factor



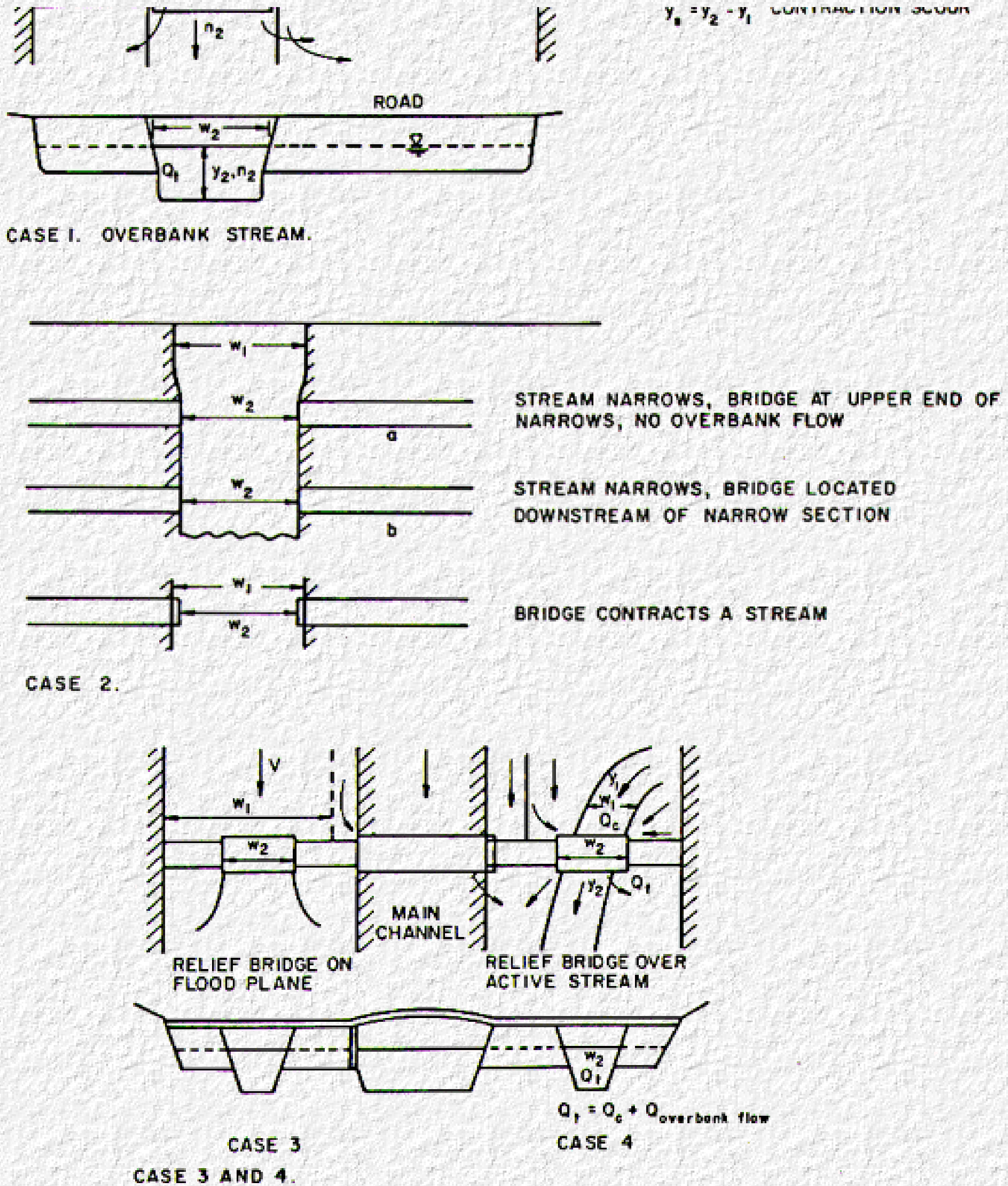


Figure 5.5.1. The Four Main Cases of Contraction Scour

Note 1. The Manning n ratio can be significant if you have a dune bed in the main channel and plain bed, washed out dunes or antidunes in the contracted channel (Chapter 3).

Note 2. The average width of the bridge opening W_2 is normally taken as the top width with the width of the piers subtracted.

Note 3. Laursen's equation for a long contraction will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of the contraction or if the contraction is the result of the bridge abutments and piers. But at this time it is the best equation available.

Case 2

No overbank flow but the stream channel narrows either naturally or by the bridge abutments encroaching on the channel. That is, flow is confined to the channel.

If the contraction of the channel is less than 10 percent the contraction scour should be negligible.

Three methods will be given for estimating contraction scour for this case. These will be:

(1) the use of Equation 5.5.1 given above;

(2) a method developed by Nordin (1971), and;

(3) an equation developed by Straub (1940).

1. To estimate contraction scour using Equation 5.5.1 set Q_t equal to Q_c.
2. Nordin's method is as follows:

The approach flow depth, y_1, and average approach flow velocity, V_1, results in the sediment transport rate q_{s1}. The total transport rate to the contraction is $W_1 q_{s1}$ in which W_1 is the width of the approach. If the water flow rate, $Q_1 = W_1 q_1$, in the upstream channel is equal to the flow rate at the contracted section, then by continuity

$$q_2 = \frac{W_1}{W_2} q_1 \qquad (5.5.2)$$

Here $q_1 = y_1 V_1$ and $q_2 = y_2 V_2$ and subscript 2 refers to conditions in the contracted section. The sediment transport rate at the contracted section after equilibrium is established must be

$$q_{S2} = \frac{W_1}{W_2} q_{S1} \qquad (5.5.3)$$

The relationships of y and V at sections 1 and 2 are shown in Figure 5.5.2 for constant q_1 and q_2.

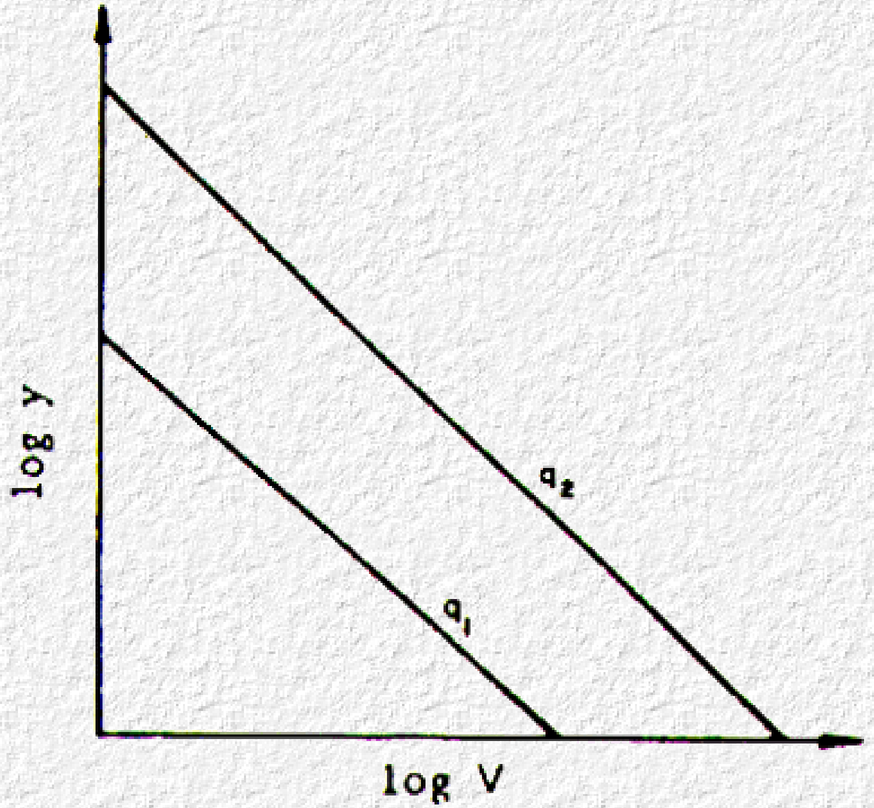


Figure 5.5.2. Unit Discharge as a Function of Depth and Velocity

From one of the sediment transport equations given in Chapter 3 it is possible to construct curves for transport rates of sediment of given median size as functions of flow depth and velocities. An illustration of such dependence is shown in Figure 5.5.3 using the method of Colby. Now overlap Figure 5.5.2. with Figure 5.5.3. The result is shown in Figure 5.5.4.

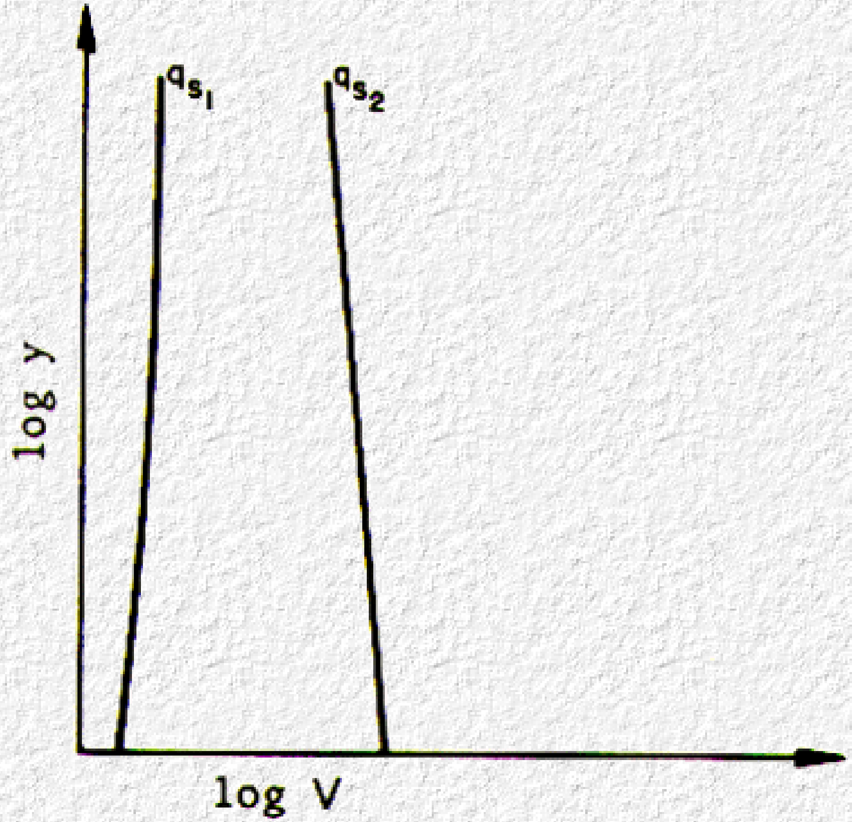


Figure 5.5.3. Sediment Transport Rate as a Function of Depth and Velocity

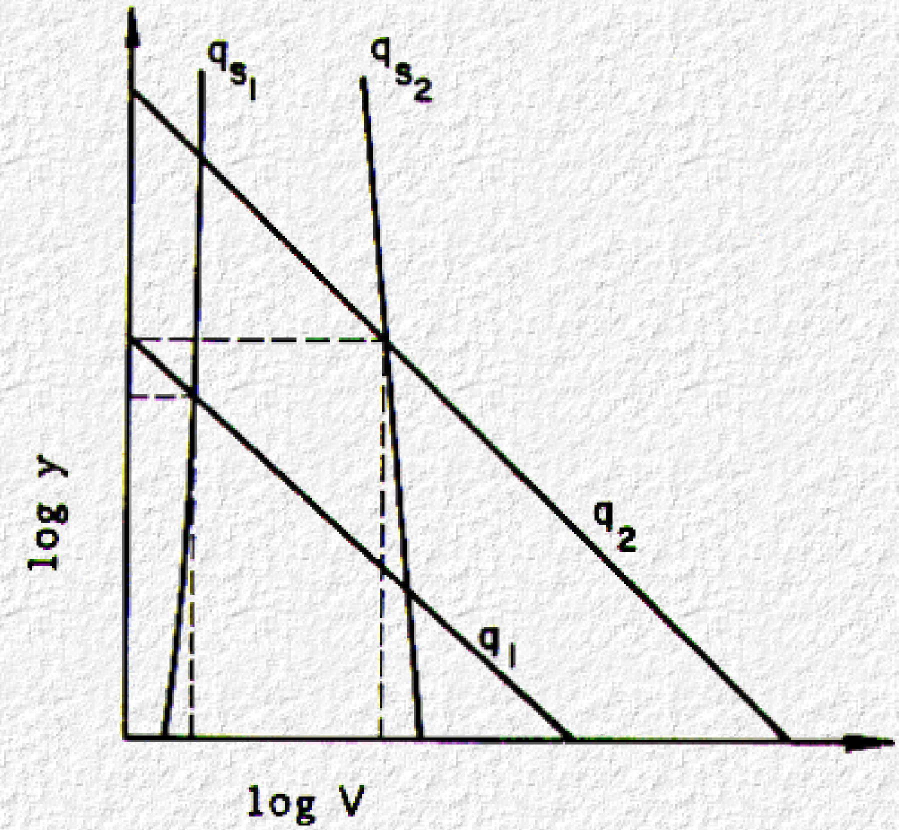


Figure 5.5.4. Determination of Contraction Scour Depth

The depth of scour due to the contraction is then

$$y_s = y_2 - y_1 \tag{5.5.4}$$

Here y_1, V_1 and W_1 are the depth, velocity and width of the approach flow, and y_2 is the general scour flow depth at the bridge. The term D_{50} is the median diameter of the bed materials at the bridge.

3. Straub's Equation - Consider the long contraction shown in Figure 5.5.5. In the wide approach reach, at the cross section designated section 1, the average velocity is V_1, the average depth of flow is y_1 and the width is W_1. The flowrate across section 1 is

$$Q = V_1\, y_1\, W_1 \tag{5.5.5}$$

In the contracted reach at the cross section designated section 2, the average velocity is V_2, the flow depth is y_2 and the width is W_2. The flowrate across section 2 is

$$Q = V_2\, y_2\, W_2 \qquad (5.5.6)$$

For a given flowrate Q and a given contraction ratio W_2/W_1 we would like to know the depth ratio y_2/y_1 for the clear-water scour case.

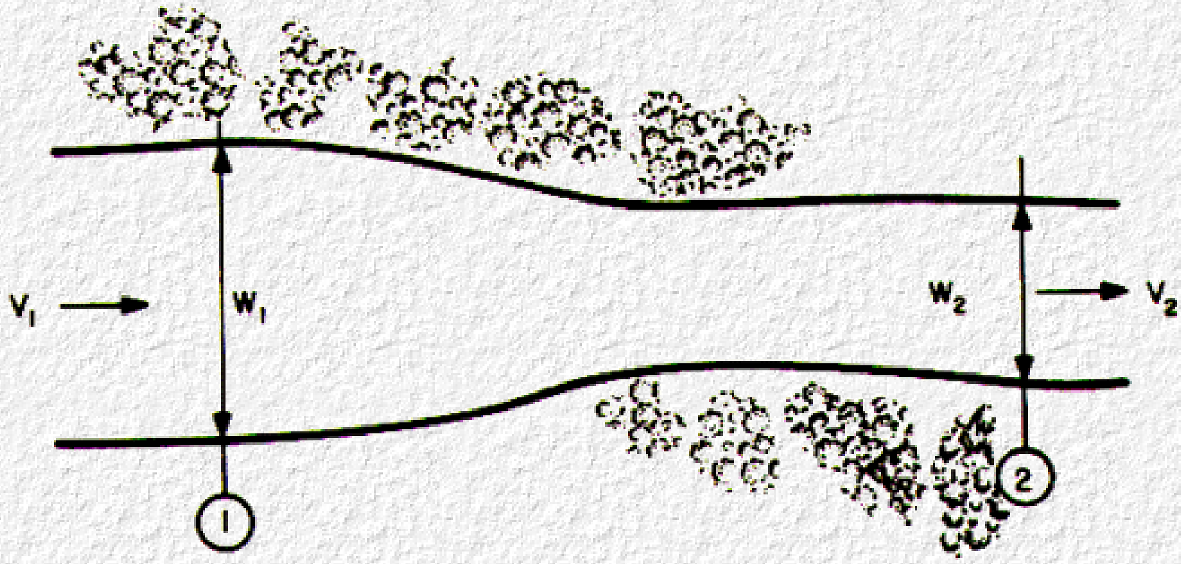


Figure 5.5.5. Plan View of the Long Contraction

In the clear-water scour case, there is no transport in the wide upstream section. The shear stress here is less than the critical shear stress (the shear stress causing initial movement of the bed particles). That is,

$$\tau_1 = \gamma y_1\, S_{f1} < \tau_c \qquad (5.5.7)$$

Here S_{f1} is the slope of the energy grade line at section 1.

Assume for the time being that scour occurs in the long contraction. Scour will continue until the bed shear stress in the long contraction has been reduced to the critical shear stress. Then, at section 2

$$\tau_2 = \tau_c = \gamma y_2\, S_{f2} \qquad (5.5.8)$$

When this condition is reached there is no longer a sediment transport at section 2. As well, there is no sediment transport at section 1 ($\tau_1 < \tau_c$). Hence the term "clear-water scour" is employed.

By employing Equation 5.5.7 and Equation 5.5.8, the depth ratio is

$$\frac{y_2}{y_1} = \frac{S_{f1}}{S_{f2}} = \frac{\tau_c}{\tau_1} \qquad (5.5.9)$$

Manning's equation can be employed to determine the friction slope ratio. Accordingly

$$\frac{S_{f1}}{S_{f2}} = \left(\frac{n_1}{n_2}\right)^2 \left(\frac{V_1}{V_2}\right)^2 \left(\frac{y_2}{y_1}\right)^{4/3} \qquad (5.5.10)$$

so

$$\frac{y_2}{y_1} = \left(\frac{n_1}{n_2}\right)^2 \left(\frac{V_1}{V_2}\right)^2 \left(\frac{y_2}{y_1}\right)^{4/3} \left(\frac{\tau_c}{\tau_1}\right) \qquad (5.5.11)$$

The velocity ratio V_1/V_2 is obtained by equating Equation 5.5.5 and Equation 5.5.6 (constant discharge) or

$$\frac{V_1}{V_2} = \frac{y_2}{y_1}\frac{W_2}{W_1} \qquad (5.5.12)$$

By putting this ratio into Equation 5.5.11, the expression

$$\frac{y_2}{y_1} = \left(\frac{n_2}{n_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{\tau_1}{\tau_c}\right)^{3/7} \qquad (5.5.13)$$

is obtained for clear-water scour. If it is assumed that

$$n_1 = n_2$$

then Equation 5.5.13 reduces to

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{\tau_1}{\tau_c}\right)^{3/7} \qquad (5.5.14)$$

which is the form of clear-water scour equation first developed by Straub (1940).

Case 3

Relief bridge where there is no bed material transport on the upstream flood plain use Laursen (1980) equation given below:

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{6/7} \left[\frac{V_1^2}{120\, y_1^{1/3} D_{50}^{2/3}}\right]^{3/7} \qquad (5.5.15)$$

Where

1 subscript = refers to the upstream conditions;
2 subscript = refers to the width and depth in the relief bridge;
W_1 = width upstream of the relief bridge, it is estimated by assuming a point of stagnation between the main bridge and the relief bridge;
V_1 = average velocity one bridge length upstream;
D_{50} = median diameter of bed material at relief bridge.

Case 4

Relief bridge with bed material transport. For this case use the equation given for Case 1 with appropriate adjustments of the variables. This case can occur when a relief bridge is over a secondary channel on the flood plain.

5.5.5 Local Scour

The basic mechanism causing local scour at a pier or abutment is the formation of a vortex at their base. The formation of these vortexes results from the pileup of water on the upstream face and subsequent acceleration of the flow around the nose of the pier or embankment. The action of the vortex is to remove bed materials away from the base region. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scour hole develops. As the depth of scour is increased, the strength of the vortex or vortexes is reduced. The transport rate is reduced and an equilibrium is reestablished and scouring ceases.

The vortex is illustrated in Figure 5.5.6 with their formation around a pier. With a pier, in addition to the vortex around the base, the horseshoe vortex, there is a vertical vortex downstream of the pier, the wake vortex. Both vortexes remove material from around the pier. However, immediately downstream of a long pier there is often deposition of material.

(1) Factors Effecting Local Scour

The factors that effect local scour are as follows:

(1) width of the pier (a);

(2) projection length (a) of the abutment into the flow;

(3) length of the pier (L);

(4) depth of flow (y_1);

(5) velocity of the approach flow (V_1);

(6) size of the bed material (D);

(7) angle of the approach flow to the pier or abutment (angle of attack);

(8) shape of the pier or abutment;

(9) bed configuration;

(10) ice formation or jams; and

(11) debris.

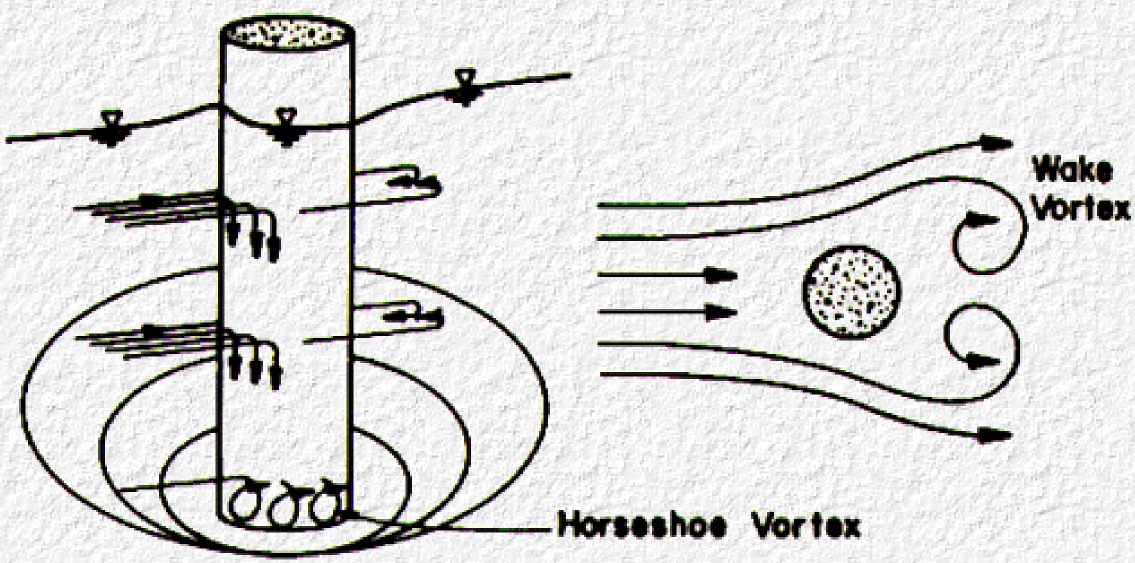


Figure 5.5.6. Schematic Representation of Scour at a Cylindrical Pier

1. Width of pier has a direct affect on the depth of scour. With an increase in pier width there is an increase in scour depth.

2. Projected length of an abutment into the stream affects the depth of scour. With an increase in the projected length of an abutment into the flow there is an increase in scour. However, there is a limit on the increase in scour depth with an increase in length. This limit is reached when the ratio of projected length into the stream (a) to the depth of the approaching flow (y_1) is 25.

3. Length of a pier has no appreciable affect on scour depth as long as the pier is lined up with the flow. If the pier is at an angle to the flow the length has a very large affect. At the same angle of attack doubling the length of the pier increases scour depth 33 percent. Some equations take the length factor into account by using the ratio of pier length to depth of flow or pier width and the angle of attack of the flow to the pier. Others use the projected area of the pier to the flow in their equations.

4. Flow depth has a direct affect on scour depth. An increase in flow depth can increase scour depth by a factor of 2 or larger for piers. With abutments the increase is from 1.1 to 2.15 depending on the shape of the abutment.

5. Velocity of the approach flow increases scour depth. The larger the velocity the deeper the scour depth. There is also a high probability that whether the flow is tranquil or rapid (subcritical or supercritical) will affect the scour depth. Most research and data is for flows with Froude numbers much less than one (Fr. < 1).

6. Size of the bed material in the sand size range has no affect on scour depth. Larger size bed material, if it will be moved by the approaching flow or by the vortexes and turbulence created by the pier or abutment, will not affect the ultimate or maximum scour but only the time it takes to reach it. Very large particles in the bed material, cobbles or boulders, may armor plate the scour hole. But in the case of the Schoharie Creek bridge collapse large riprap was in time removed from around the piers by a series of large flows (Richardson et al., 1987). The size of the bed material also determines whether the scour at a pier or abutment is clear-water of live-bed scour. This topic is discussed later in this section.

Fine bed material (silts and clays) will have scour depths as deep or deeper than sandbed streams. This is true even if bonded together by cohesion. The affect of cohesion is to determine the time it takes to reach the maximum scour. With sand bed material the maximum depth of scour is measured in hours. With cohesive bed materials it may take days, months or even years to reach the maximum scour depth.

7. Angle of attack of the flow to the pier or abutment has a large affect on local scour as was pointed out in the discussion of the affect of pier length above. The affect on piers will not be repeated here. With abutments the depth of scour is reduced for embankments angled downstream and is increased if the embankments are angled upstream. According to the work of Ahmad (Richardson et al., 1975 and 1987) the maximum depth of scour at an embankment inclined 45 degrees downstream is reduced by 20 percent. Whereas, the scour at an embankment inclined 45 degrees upstream is increased about 10 percent.

8. Shape of pier or abutment has a significant affect on scour. With a pier, streamlining the front end reduces the strength of the horseshoe vortex reducing scour depth. Streamlining the downstream end of piers reduces the strength of the wake vortices. A square-nose pier will have maximum scour depths about 20 percent larger than a sharp-nose pier and 10 percent larger than a cylinder or round-nose pier. Abutments with vertical walls on the streamside and upstream side will have scour depths about double that of spill slope abutments.

9. Bed configuration effects the magnitude of local scour. In streams with sand bed material the shape of the bed (bed configuration) as determined by Simons and Richardson (1963) and discussed in Chapter III, may be ripples, dunes, plane bed and antidunes. The bed configuration depends on the size distribution of the sand bed material, flow conditions and fluid viscosity. The bed configuration may change from dunes to plain bed or antidunes during an increase in flow. It may change back with a decrease in flow. The bed configuration may also change with a change in water temperature or change in concentration of silts and clays.

10. Ice and debris by increasing the width of the piers, changing the shape of piers and abutments, increasing the projected length of an abutment or causing the flow to plunge downward against the bed can increase both the local and general (contraction) scour. The magnitude of the increase is still largely undetermined. But debris can be taken into account in the scour equations by estimating how much the debris will increase the width of the pier or length of the abutment. Debris and ice affects on general (contraction) scour can also be accounted for by estimating the amount of flow blockage (decrease in width of the bridge opening) in the equations for contraction scour. Field measurements of scour at ice jams indicate the scour can be in the 10's of feet.

(2) Clear-Water and Live-Bed Scour

There are two conditions of local scour. These are (1) clear-water scour and (2) live-bed scour.

Clear-Water Scour

Clear-water scour occurs when there is no movement of the bed material of the stream upstream of the crossing but the acceleration of the flow and vortices created by the piers or abutments causes the material at their base to move.

Live-Bed Scour

Live-bed scour occurs when the bed material upstream of the crossing is also moving.

Bridges over coarse bed material streams often have clear-water scour at the lower part of a hydrograph, live-bed scour at the higher discharges and then clear-water scour on the falling stages.

Clear-water scour reaches its maximum over a longer period of time than live-bed scour, Figure 5.5.7. This is because clear-water scour occurs mainly on coarse bed material streams. In fact clear-water scour may not reach its maximum until after several floods. Also, maximum clear-water scour is about 10 percent greater than the maximum live-bed scour.

Live-bed scour in sand bed streams with a dune bed configuration fluctuates about an equilibrium scour depth, Figure 5.5.7. The reason for this is the fluctuating nature of the sediment transport of the bed material in the approaching flow when the bed configuration of the stream is dunes. In this case (dune bed configuration in the channel upstream of the bridge) maximum depth of scour is about 30 percent larger than equilibrium depth of scour.

The maximum depth of scour is the same as the equilibrium depth of scour for live-bed scour with a plain bed configuration. With antidunes ocurring upstream and in the bridge crossing the maximum depth of scour from the limited research of Jain and Fisher (1979) is about 20 percent greater than the equilibrium depth of scour. In general, with sand bed streams a dune bed changes to plain bed or antidune flow during flood flow.

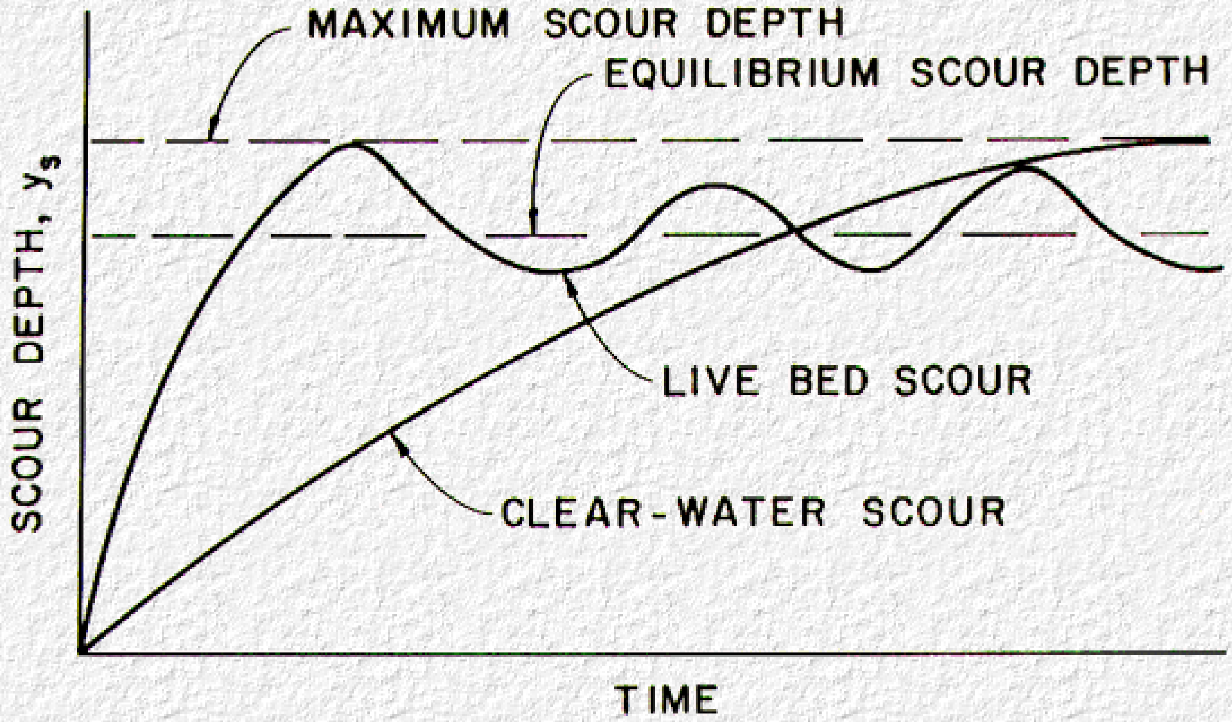


Figure 5.5.7. Scour depth as a Function of Time

(3) Armoring

Armoring occurs on a stream or in a scour hole when the forces of the water during a particular flood are unable to move the larger sizes of the bed material. This protects the underlying material from movement. Scour around an abutment or pier may initially occur but as the scour hole deepens the courser bed material moves down in the hole and protects the bed so that the full scour potential is not reached.

When armoring occurs, the coarser bed material will tend to remain in place or quickly redeposit so as to form a layer of riprap-like armor in the scour holes and thus limit further scour for a particular discharge. This armoring affect can decrease scour hole depths which were predicted to occur based on formulae developed for sand or other fine material channels for particular flow conditions. When larger flow conditions occur the armor layer can be broken and the scour hole deepened either until a new armor layer is developed or the maximum scour as given by the sand bed equations is reached. Unfortunately, reliable knowledge of how to predict the decrease in scour hole depth when there are large particles in the bed material is lacking. Research in New Zealand by Raudkivi (1986) and in Washington State (Copp and Johnson, 1987) gives a basis for calculating the decrease in scour depth by armoring but their equations need field verification. The results of this research for pier scour will be given later.

(4) Estimating Local Scour Depths

Equations for estimating local scour are based on three methods of analysis. These methods are:

a. Dimensional analysis of the basic variables causing local scour.

b. The use of transport relations in the approaching flow and in the scour hole.

c. Regression analysis of the available data.

Equations for estimating local scour at abutments or piers developed by the three methods are given in the next sections. In HEC 18 (FHWA 1989) and in the June 1988 "Interim Procedures for Evaluating Scour at Bridges," (FHWA 1988) only one method or equation is recommended. The additional equations are given in this manual for basis of comparison, to be used in additional study of a particular bridge site and for use in research. It should be noted that these equations were developed from laboratory experiments with limited field data.

In analysis for scour the engineer should evaluate his problem and select the equation or method that in his judgment best suits the case at hand. Sometimes it may be necessary to use more than one equation or method and then use engineering judgment in selecting the local scour depth. For example, if the stream contains large quantities of course bed material. Based on the knowledge of the stream, the bed material, the flows and type of highway select the value of the scour.

5.5.6 Local Scour at Abutments

(1) General

Equations for predicting scour depths are based almost entirely on either laboratory data or inductive reasoning from sediment continuity equations. There are little field data to compare abutment scour equations. The equations give the worse case scour depths. Equations for estimating abutment scour are derived by three methods . . . . .

Types of Equations

. From dimensional analysis of the variables and developing relationships among the major dimensionaless parameters such as, the ratio of scour depth y_s to flow depth y_1; ratio of abutment length a to flow depth y_1; the Froude Number Fr, etc. Liu, et al.'s 1961 equation given here is an example.

b. From the use of transport relations and the change in transport because of the acceleration of the flow caused by the abutment. Laursen's (1980) equations given here are an example.

c. From regression analysis of available data. Froehlich's 1988 equation given here is an example.

Position of Abutments

Abutments can be set back from the natural stream bank or can project out into the flow; they can have various shapes (vertical walls, spill through slopes) and they can be set at an angle to the flow. Scour at abutments can be live-bed or clear-water scour. Finally, there can be varying amounts of overbank flow that is intercepted by the approaches to the bridge and returned to the stream at the abutment.

Scour at abutments can be caused by the abutment projecting into the flow, it can be caused by the approaches to the bridge intercepting overland flow and forcing it back into the channel at the abutment, or it can be a combination of conditions. The various conditions (cases) are given in Table 5.5.1 and illustrated in Figure 5.5.8. In Table 5.5.1 equations are given for each case. No single equation is recommended for a given situation when more than one equation is applicable, because with the lack of field data for verification, it is not known which equation is best.

It is recommended that the designer determine what case fits the design situation and then use all equations that apply to the case. IT IS MOST IMPORTANT THAT THE COMMENTARY ON EACH OF THE EQUATIONS BE READ AND UNDERSTOOD

PRIOR TO ATTEMPTING TO USE THE EQUATIONS FOR DESIGN PURPOSES. Engineering judgement must be used to select the depth of foundations. The designer should take into consideration the potential cost of repairs to an abutment and danger to the traveling public in selecting scour depths. Finally protection measures such as spur dikes and riprap can be used in order to decrease the depth piles have to be driven or the foundations set.

Comments on Table 5.5.1 and Figure 5.5.8.

. Equations for these cases (except for Case 6) are based on laboratory studies with little or no field data.

b. The factor a/y_1 = 25 as a limit for Cases 1-5 is rather arbitrary but it is not practical to assume that scour depth, y_s, would continue to increase with an increase in abutment length "a."

c. There are two general shapes for abutments. These are vertical wall abutments with wing or box walls and spill-through abutments, Figure 5.5.9. Depth of scour is about double for vertical wall abutments as compared with spill-through abutments.

d. Froelich's equations (5.5.21 and 5.5.22) can be used to calculate scour for all six cases.

Maximum Depth of Scour

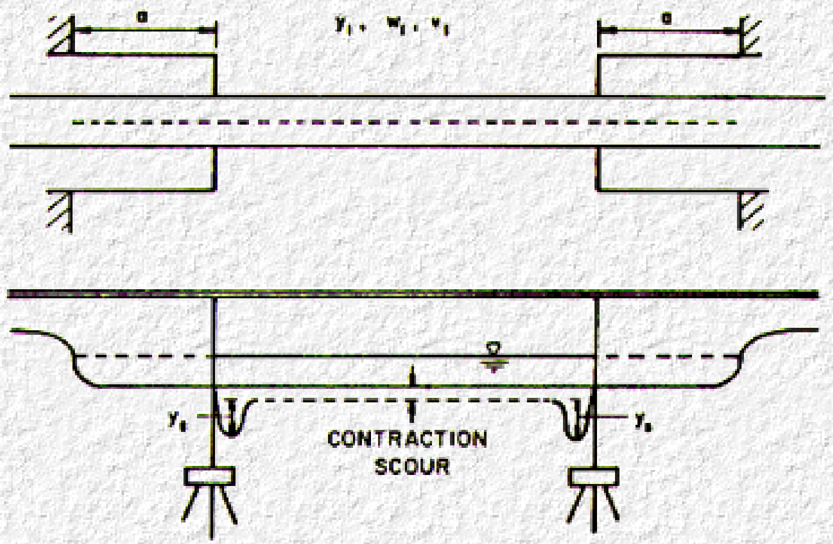
. For live-bed scour with a dune bed configuration, the maximum depth of scour is about 30 percent greater than equilibrium scour depth given by Liu, et al's (1961) equations (Equation 5.5.16 and Equation 5.5.17). Therefore, the values of scour that are calculated for these equations should be increased by 30 percent when the bed form is dunes upstream of the bridge. The reason for this is that the research that was used for determining scour depth for the live-bed scour case was run with a dune bed and equilibrium scour was measured.

b. For clear-water scour (Equation 5.5.20), the maximum depth of scour is about 10 percent greater than live-bed scour; however, there is no need to increase the scour depths because the equations predict the maximum scour.

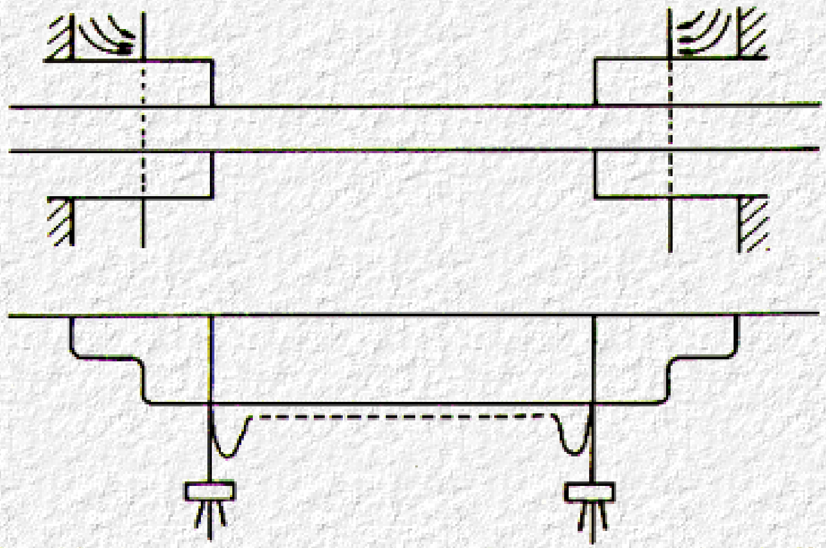
Table 5.5.1 Summary of Abutment Scour Equations

Case	Abutment Location	Overbank flow	Value of a/y_1	Bed Load Condition	Abutment Type	Euation Number
1	Projects into channel	No	$a/y_1 < 25$	Live bed	Vertical Wall	17,18,22
					Spill through	16,18,22
				Clear water	Vertical Wall	20,21
2	Projects into channel	Yes	$a/y_1 < 25$	Live bed	Not designated	18,22,24
				Clear water	Not designated	20,21,24
3	Set back from main channel	Yes	$a/y_1 < 25$	Clear water	Not designated	20,21
4	Relief bridge on floodplain	Yes	$a/y_1 < 25$	Clear water	Not designated	20,21
5	Set at edge of main channel	Yes	$a/y_1 < 25$	Live bed	Not designated	22,24

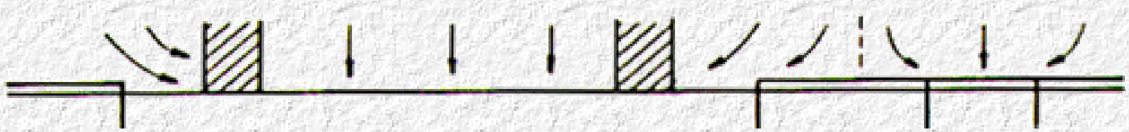
6	Not designated	Yes	$a/y_1 < 25$	Not designated	Not designated	25
7	Skewed to stream	-	-	-	-	*
** Adjust scour estimate for Equations 3-11 using [Figure 5.5.17]()*						

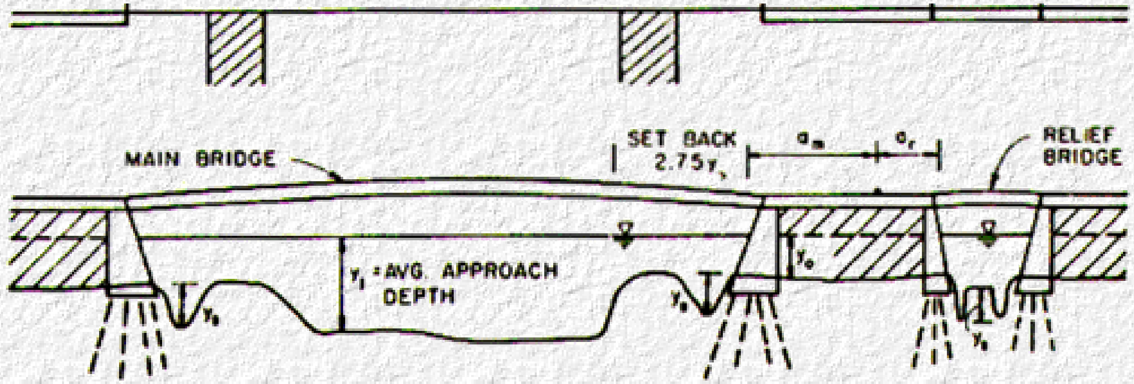


Case 1. Abutments project into channel, no overbank flow.



Case 2. Abutments project into channel, overbank flow.

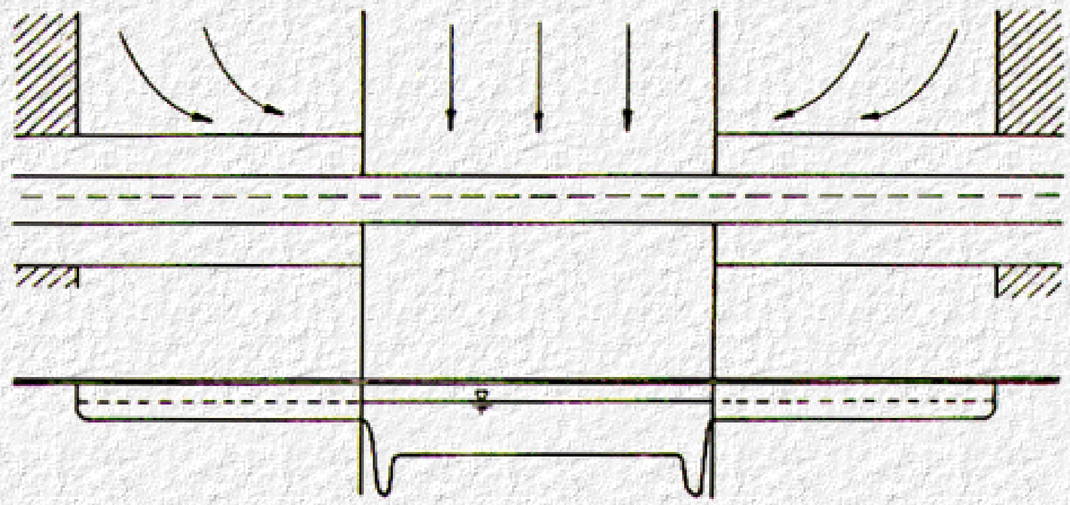




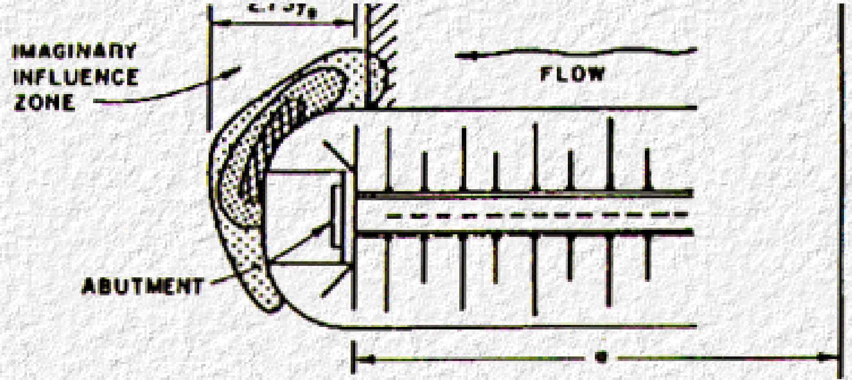
Case 3. Abutment setback from the channel more than 2.75 y_s.

Case 4. Relief bridge.

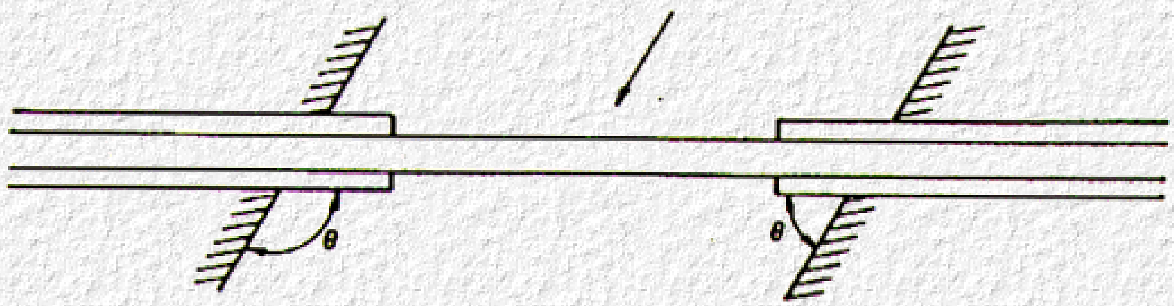
Figure 5.5.8. Abutment Scour, Cases 1, 2, 3 and 4



Case 5. Abutment set at edge of channel, overbank flow.



Case 6. Ratio of abutment length, a, to flow depth, y_1, larger than 25.



Case 7. Abutment set at an angle, Θ, to the flow.

Figure 5.5.8. Abutment Scour, Cases 5, 6 and 7 (Continued)

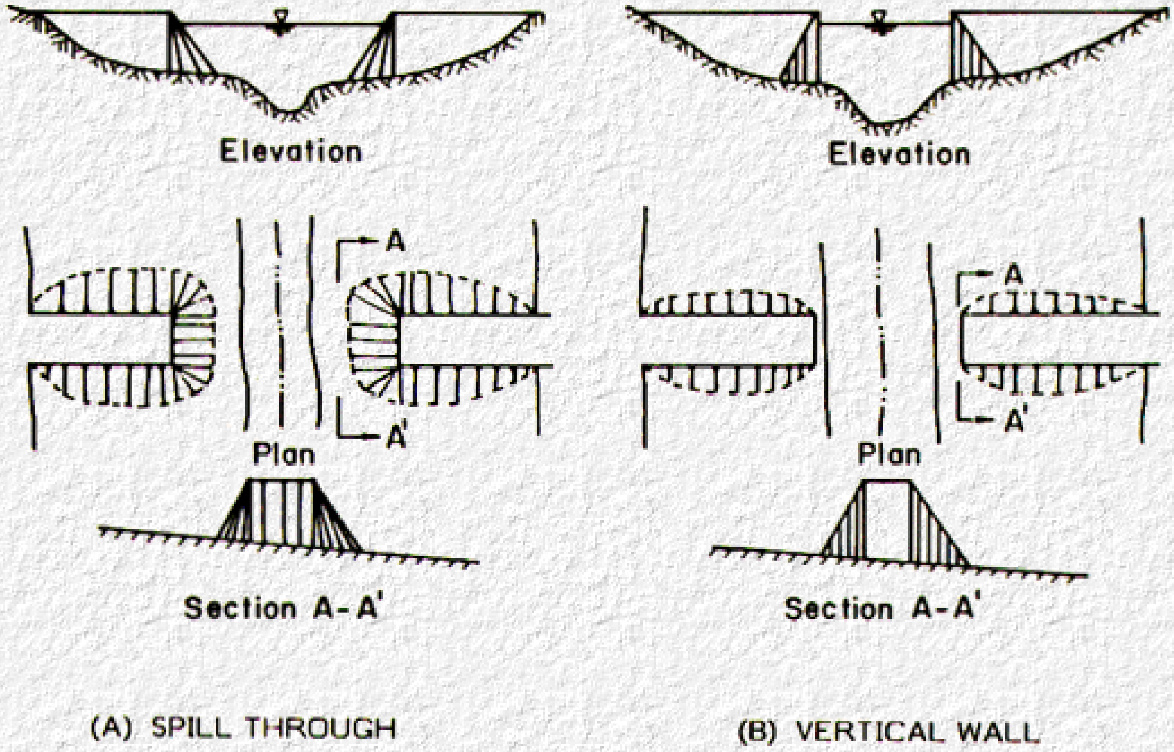


Figure 5.5.9. Abutment Shape

(2) Calculating Abutment Scour for the Different Cases

Case 1. Abutments Project Into Channel, No Overbank Flow - Figure 5.5.10.

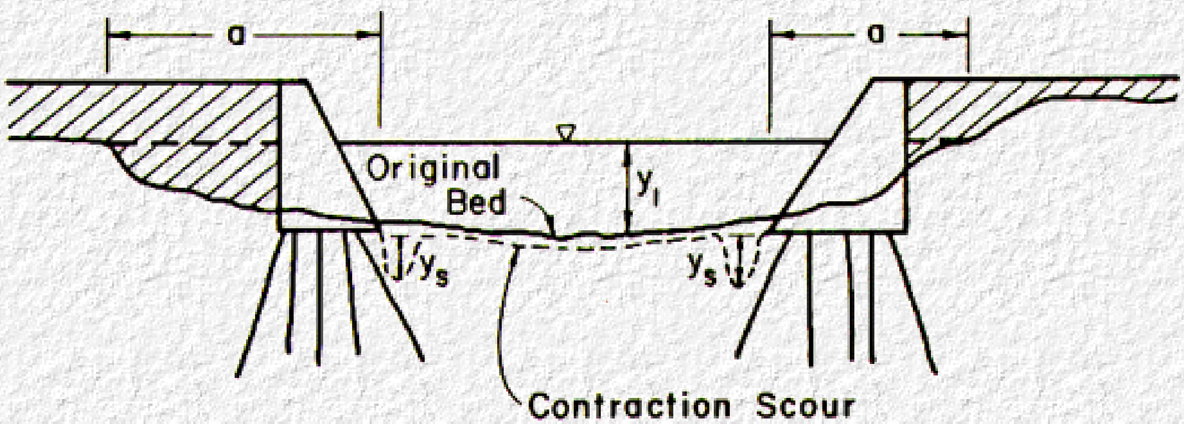


Figure 5.5.10. Definition Sketch for Case 1 Abutment Scour

Seven equations are given for case 1 (Equation 5.5.16, 5.5.17, 5.5.18, 5.5.19, 5.5.20, 5.5.21 and 5.5.22). These are equations based on dimensional analysis, transport relations and regression analysis. These equations are limited to cases where $a/y_1 < 25$. For $a/y_1 > 25$, go to Case 6.

Liu, et al's Equations

Live-bed scour at the spill through abutment, Equation 5.5.16.

According to the studies of Liu, et al., (1961) the equilibrium scour depth for local live-bed scour in sand at a stable spill slope when the flow is subcritical is determined by Equation 5.5.16.

$$\frac{y_s}{y_1} = 1.1\left(\frac{a}{y_1}\right)^{0.40} Fr_1^{\,0.33} \qquad (5.5.16)$$

y_s = Equilibrium depth of scour (measured from the mean bed level to the bottom of the scour hole).

y_1 = Average upstream flow depth in the main channel.

a = Abutment and embankment length (measured at the top of the water surface and normal to the side of the channel from where the top of the design flood hits the bank to the outer edge of the abutment).

Fr_1 = Upstream Froude number

$$Fr = \frac{V_1}{(gy_1)^{0.5}}$$

Live bed scour at a vertical wall abutment, Equation 5.5.17

If the abutment terminates at a vertical wall and the wall on the upstream side is also vertical, then the scour hole in sand calculated by Equation 5.5.16 nearly doubles (Liu, et al., 1961 and Grill, 1972).

$$\frac{y_s}{y_1} = 2.15\left(\frac{a}{y_1}\right)^{0.40} Fr_1^{\,0.33} \qquad (5.5.17)$$

Laursen's Equations

Live-bed scour at vertical wall abutment, Equation 5.5.18.

Laursen (1980) suggested two relationships for scour at vertical wall abutments for Case 1. One for live-bed scour and another for clear-water scour depending on the relative magnitude of the bed shear stresses to the critical shear stress for the bed material of the stream. For live-bed scour ($\tau_1 > \tau_c$), use Equation 5.5.18 or 5.5.19.

$$\frac{a}{y_1} = 2.75\,\frac{y_s}{y_1}\left[\left(\frac{y_s}{11.5\,y_1}+1\right)^{1.7}-1\right] \qquad (5.5.18)$$

Simplified form:

$$\frac{y_s}{y_1} = 1.5\left(\frac{a}{y_1}\right)^{0.48} \qquad (5.5.19)$$

Clear-water scour ($\tau_1 > \tau_c$) at vertical wall abutment, Equation 5.5.20.

$$\frac{a}{y_1} = 2.75\left(\frac{y_s}{y_1}\right)\left[\frac{\left(\frac{y_s}{11.5\,y_1}+1\right)^{7/6}}{\left(\frac{\tau_1}{\tau_c}\right)^{0.5}}-1\right] \qquad (5.5.20)$$

τ_1 = shear stress on the bed upstream

τ_c = critical shear stress of the D_{50} of the upstream bed material. The value of τ_c can be obtained from Figure 5.5.11.

Scour at other abutment shapes.

Scour values given by Laursen's equations are for vertical wall abutments. He suggests the following multiplying factors for other abutment types for small encroachment lengths:

Abutment Type	**Multiplying Factor**
45 degree Wing Wall	**0.90**
Spill-Through	**0.80**

Laursen's equations are based on sediment transport relations. They give maximum scour and include contraction scour. FOR THESE EQUATIONS, DO NOT ADD CONTRACTION SCOUR TO OBTAIN TOTAL SCOUR AT THE ABUTMENT.

Laursen's equations require trial and error solution. Curves developed by Chang (1987) are given in Figure 5.5.12. Note that the equations have been truncated at a value of y_s/y equal to 4. The reason the equations were truncated at 4.0 is that field observations of scour around spur dykes on the Mississippi River never exceeded 4y, see Equation 5.5.25. That is, the maximum scour, y_s, that should be accepted from Laursen's equations is 4 y_1.

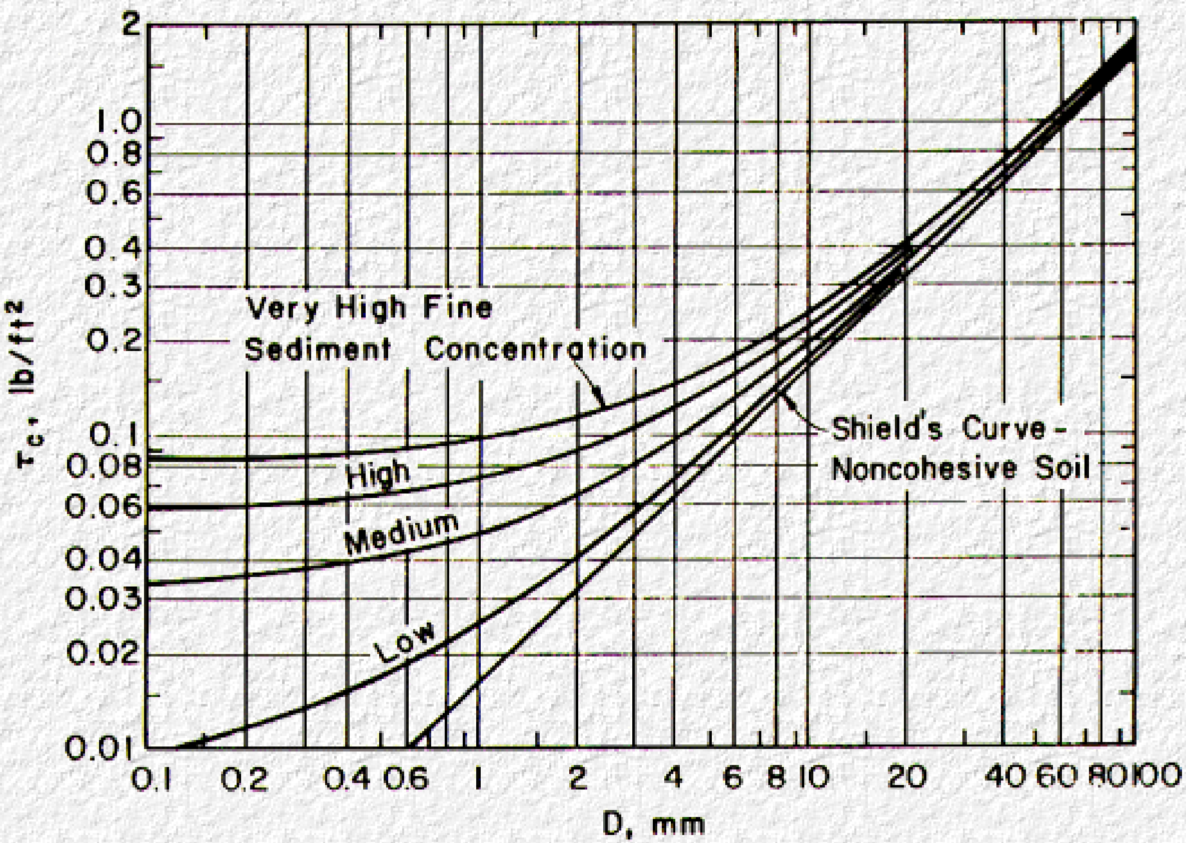


Figure 5.5.11. Critical Shear Stress as a Function of Bed Material Size and Suspended Fine Sediment

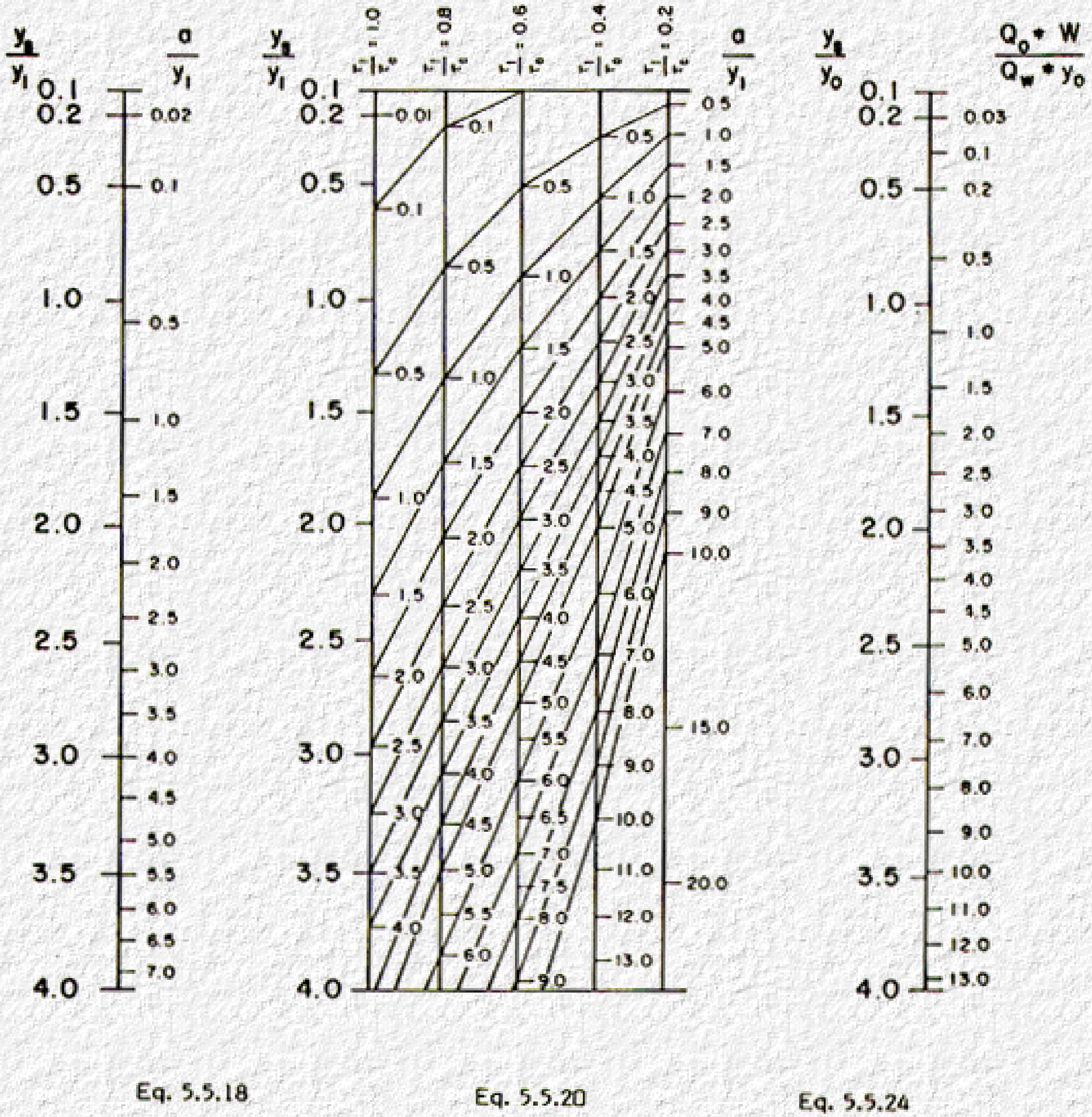


Figure 5.5.12. Nomographs for Abutment Scour

<u>Froehlich's Equations</u>

Clear-water scour at an abutment, Equation 5.5.21.

Froehlich (1987) using dimensional analysis and multiple regression analysis of 164 clear-water scour measurements in laboratory flumes developed the following equation:

$$\frac{y_s}{y_1} = 0.78\,K_1K_2\left(\frac{a'}{y_1}\right)^{.63} Fr^{1.16}\left(\frac{y_1}{D_{50}}\right)^{.43} G^{-1.87} + 1.0 \qquad (5.5.21)$$

Live-bed scour at an abutment, Equation 5.5.22.

He analyzed 170 live-bed scour measurements to obtain the following equation:

$$\frac{y_s}{y_1} = 2.27\,K_1K_2\left(\frac{a'}{y_1}\right)^{.43} Fr^{.61} + 1.0 \qquad (5.5.22)$$

where

K_1 = Coefficient for abutment shape.

K_2 = Coefficient for angle of embankment to flow.

a' = Length of abutment projected normal to flow.

Fr = Froude number of flow upstream of abutment.

G = Geometric standard deviation of bed material.

y_1 = Depth of flow at abutment

y_s = Scour depth

Values of K_1	K_1
Vertical abutment	1.0
Vertical abutment With Wingwalls .	82
Spill through abutment	.55

Values of K_2

$$K_2 = \left(\frac{\theta}{90}\right)^{.13}$$

Θ angle of embankment to flow

Values of a' $a' = A_e / y_1$

A_e = Flow area of approach cross section obstructed by embankment.

$G = (D_{84} / D_{16})^{.5}$

D_{50}, D_{84}, D_{16} are size of bed material. The subscript indicates the percent finer than,

$Fr = V_e / (gy_1)^{.5}$

V_e = Velocity of flow approaching abutment

The + 1 in Froehlich's equation is a safety factor that makes the equation predict a scour depth larger than any of the measured scour depths in the data. NOTE, FROEHLICH'S EQUATIONS, UNLIKE LAURSEN'S, DOES NOT INCLUDE CONTRACTION SCOUR. Also, Froehlich's equations does not make allowances for bed forms. Where dunes are expected, scour depths should be increased by a factor of $y_1/6$.

Case 2. Abutment Projects into the Channel, Overbank Flow, Figure 5.5.13

No bed material is transported in the overbank area and $a/y_1 < 25$. This case is illustrated in Figure 5.5.13.

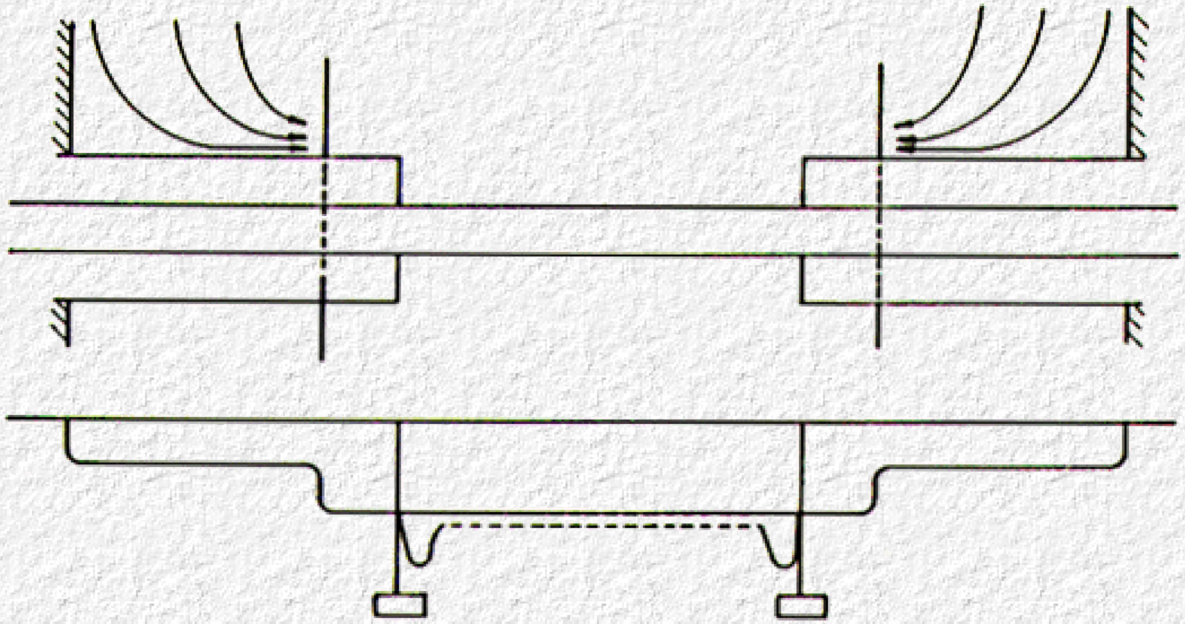


Figure 5.5.13. Bridge Abutment in Main Channel and Overbank Flow

Equation 5.5.18 or Equation 5.5.20 should be used to calculate the scour depth with abutment length "a" determined by **Equation 5.5.23**. Engineering judgement must be used in selecting the scour depth for design purposes. Equation 5.5.24 and Froehlich's equations can also be used for this case with the appropriate selection of variables. Live bed scour ($\tau_1 > \tau_c$) use **Equation 5.5.18**, **Equation 5.5.22 and Equation 5.5.24**. Clear water scour ($\tau_1 < \tau_c$) use **Equation 5.5.20**, **Equation 5.5.21, and Equation 5.5.24**.

$$a = \frac{Q_o}{V_1 y_1} \tag{5.5.23}$$

τ_1 = The shear stress in the main channel.

τ_c = The critical shear stress for the bed material in the main channel. The value can be determined from Figure 5.5.11.

Q_o = Flow obstructed by abutment and bridge approach.

y_1 = Average upstream flow depth in the main channel.

V_1 = Average upstream velocity in the main channel.

Case 3. Abutment Is Set Back from Main Channel More Than 2.75 y_s

There is overbank flow with no bed material transport (clear-water scour). Figure 5.5.14. illustrates this case.

With no bed material transport in overbank flow, scour at a bridge abutment, set back more than 2.75 times the scour depth from the main channel back line, can be calculated using Equation 5.5.20 from Laursen (1980) with:

τ_o = Shear stress on the overbank area upstream of the abutment.

τ_c = Critical shear stress of material in overbank area. Can be determined from Figure 5.5.11. using the D_{50} of the bed material of the cross-section under consideration. Alternately, it can be calculated using the Shield's relation for beginning of motion given in Chapter 3.

a = a_m if there is a relief bridge.

The lateral extent of the scour hole from the edge of the abutment is nearly always determinable from the depth of scour and the natural angle of repose of the bed material. Laursen suggested that the width of the scour hole is 2.75 y_s.

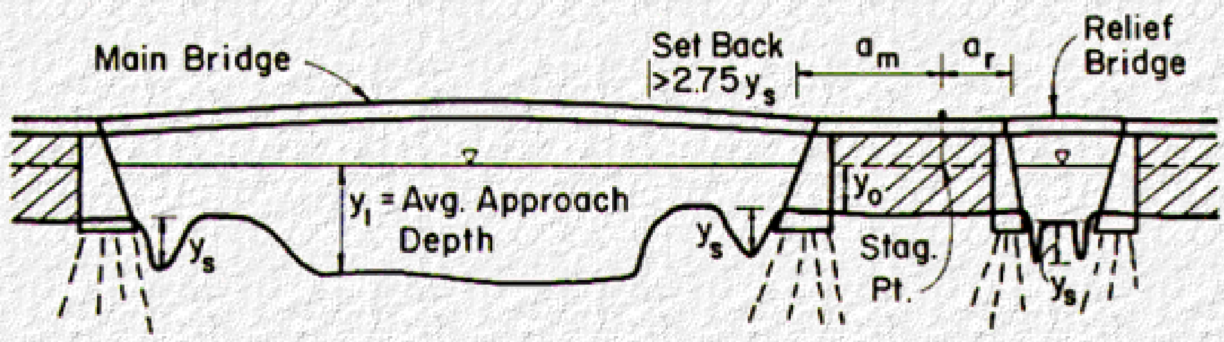


Figure 5.5.14. Bridge Abutment Set Back from Main Channel Bank and Relief Bridge

Case 4. Abutment Scour at Relief Bridge

Scour depth for a relief bridge on the overbank flow area having no bed material transport is calculated using **Equation 5.5.20** where y_1 is average flow depth on the flood plain. Use a_r for a in the equation. Draw stream lines or field observations to delineate where the separation point is for the flow going to the main channel and to the relief bridge. (See Figure 5.5.14). Also, Froehlich's Equation 5.5.2 can be used with fine bed material G is 1.0.

Case 5. Abutment Set at Edge of Channel

The case of scour around an abutment set right at the edge of the main channel as sketched in Figure 5.5.15 can be calculated with Equation 5.5.24 proposed by Laursen (1980) when $\tau_o < \tau_c$ on the flood plain. Also, Froehlich's equation can be used.

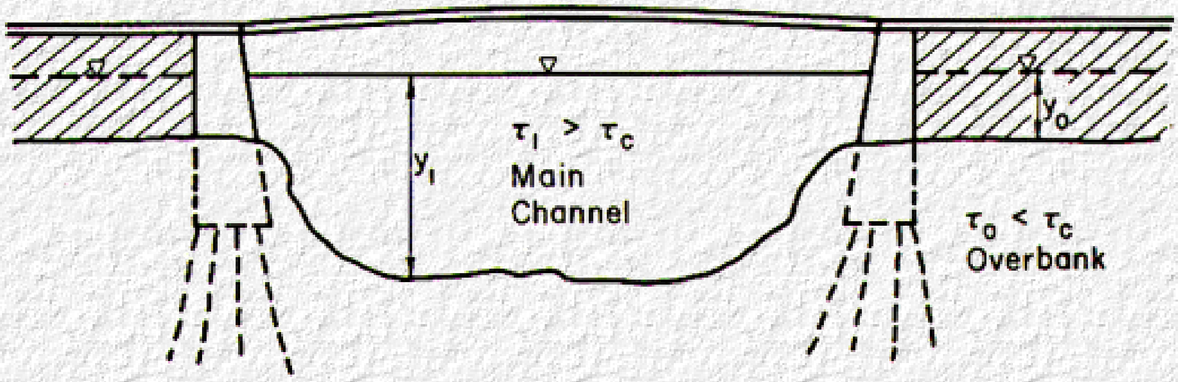


Figure 5.5.15. Abutment Set at Edge of Main Channel

$$\frac{Q_o}{q_{mc} y_o} = 2.75 \frac{y_s}{y_o} \left[\left(\frac{y_s}{4.1\, y_o} + 1 \right)^{7/6} - 1 \right] \quad (5.5.24)$$

q_{mc} = The unit discharge in the main channel, Q/W.
Q = Discharge in main channel.
W = Width of the main channel.
Q_o = Overbank flow discharge.
y_o = Overbank flow depth.

Note that if there is no overbank flow for this case, then there is no appreciable scour.

Values of calculated relative scour depth from Equations 5.5.18, 5.5.20 and 5.5.24 are given in Figure 5.5.16.

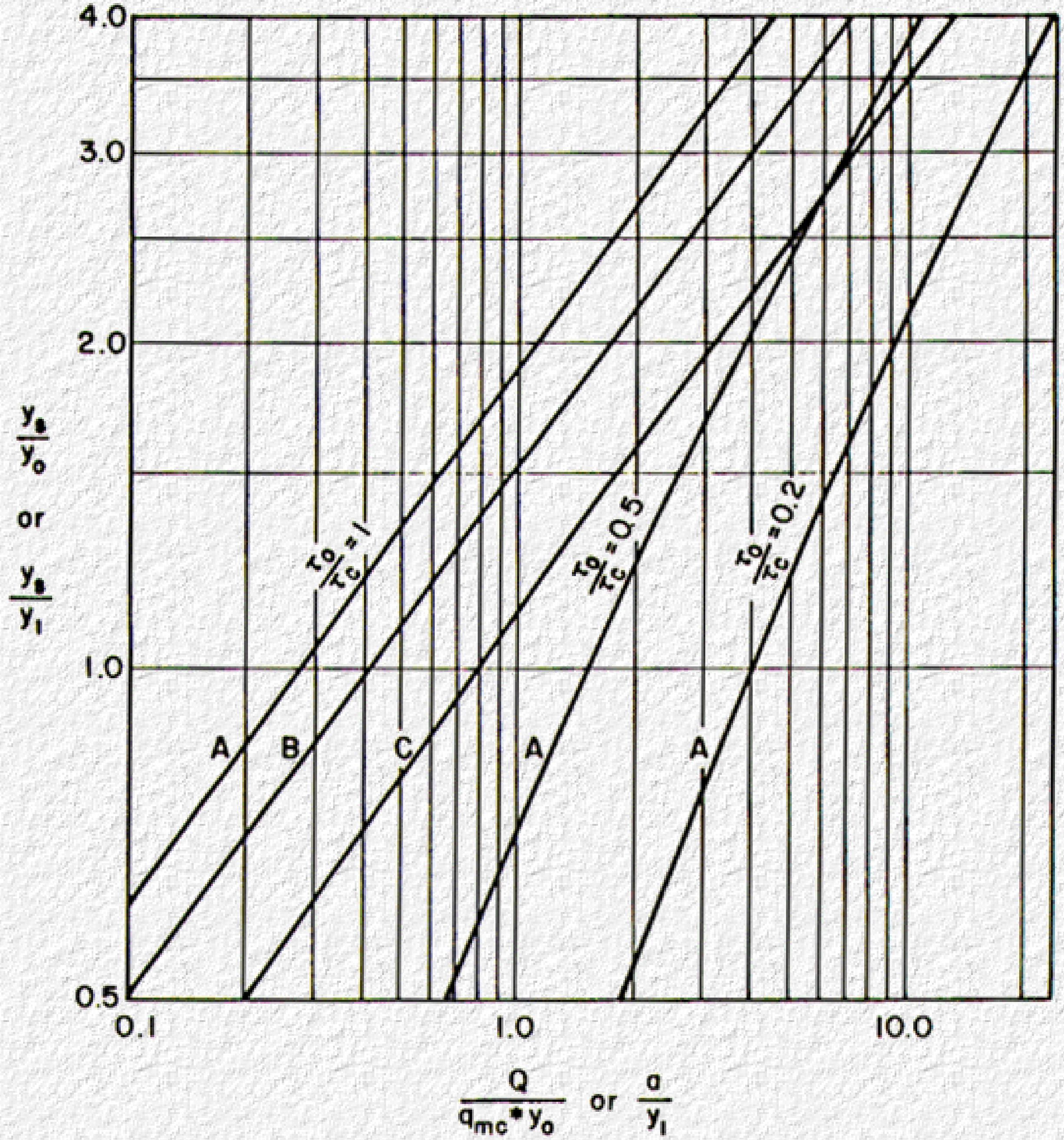


Figure 5.5.16. Values of Calculated Scour Depths
(A is Equation 5.5.20, B is Equation 5.5.18, C is Equation 5.5.24)

Case 6. Scour At Abutments When a/y_1 > 25

Field data for scour at abutments for various size streams are scarce, but data collected at rock dikes on the Mississippi indicate the equilibrium scour depth for large a/y_1 values can be estimated by Equation 5.5.25:

$$\frac{y_s}{y_1} = 4\ Fr_1^{0.33} \qquad (5.5.25)$$

The data are scattered, primarily because equilibrium depths were not measured. Dunes as large as 20 to 30 feet high move down the Mississippi and associated time for dune movement is very large in comparison to time required to form live-bed local scour holes. Nevertheless, it is believed that these data represent the limit in scale for scour depths as compared to laboratory data and enables useful extrapolation of laboratory studies to field installations.

According, it is recommended that Equations 5.5.18 through 5.5.24 be applied for abutments with $0 < a/y_1 < 25$ and Equation 5.5.25 be used for $a/y_1 > 25$.

Case 7. Abutments Skewed to the Stream

With skewed crossings, the approach embankment that is angled downstream has the depth of scour reduced because of the streamlining effect. Conversely, the approach embankment which is angled upstream will have a deeper scour hole. The calculated scour depth should be adjusted in accordance with the curve of Figure 5.5.17 which is patterned after Ahmad (1953).

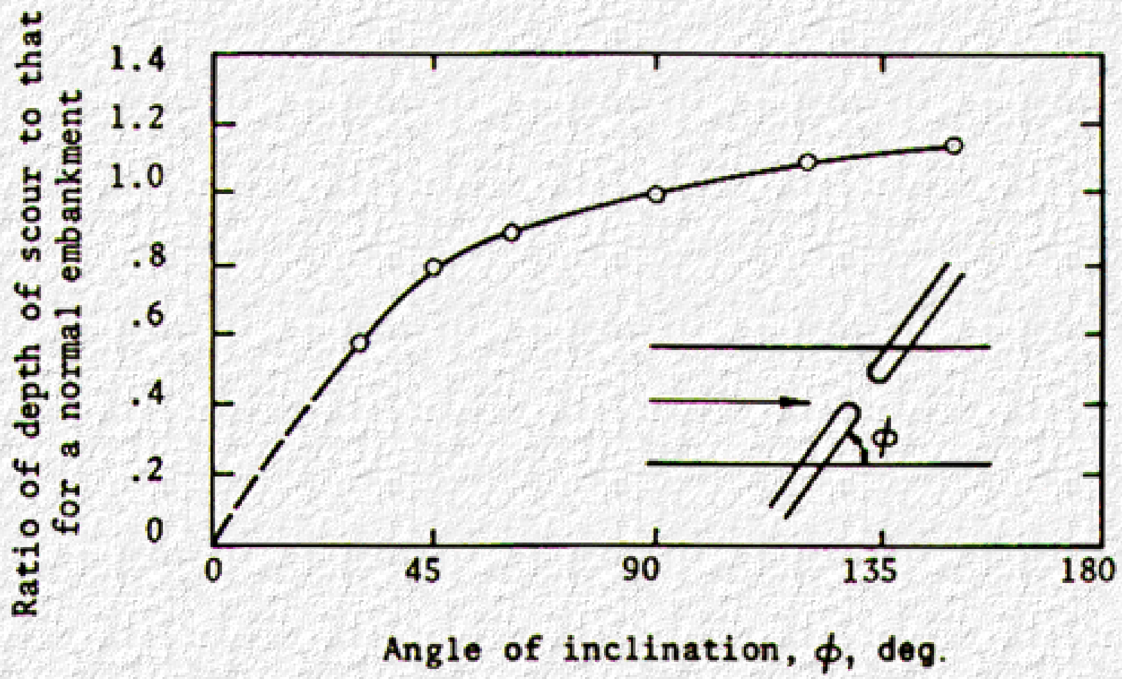


Figure 5.5.17. Scour Reduction Due to Embankment Inclination

5.5.7 Local Scour at Piers

(1) Introduction

Local scour at piers is a function of bed material size, flow characteristics, fluid properties and the geometry of the pier. The subject has been studied extensively in the laboratory but there is very little field data. As a result of the many studies there are many equations. In general, the equations are for live-bed scour in cohesionless sand bed streams, and they give similar results . In this section, we will give several equations. These equations will be as follows:

1. Colorado State University's (CSU) equation.
2. Jain and Fisher's equation.
3. Graded and/or armored streambeds equations.
4. Froehlich's equations.

As will be explained in the following paragraphs, the CSU equation is recommended but the other equations are given for comparison, research and for special cases such as streams with a large quantity of large-size particles. It is believed that the CSU equation will give the ultimate scour. Engineering judgement will be needed in the case of the other equations.

Sterling Jones (1983) compared many of the more common equations. His comparison of these equations is given in Figure 5.5.18. Some of the equations have velocity as a variable (normally in the form of a Froude number). However, some equations, such as Laursen's do not include velocity. A Froude number of 0.3 was used (Fr = 0.3) in Figure 5.5.18 for purposes of comparing commonly used scour equations. In Figure 5.5.19, the equations are compared with some field data measurements. As can be seen from Figure 5.5.18, the CSU equation encloses all the points but gives lower values of scour than Jain, Laursen and Niel's equations. The CSU equation includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. Chang (1988) points out that Laursen's (1980) equation is essentially a special case of the CSU equation with the Fr = 0.5.

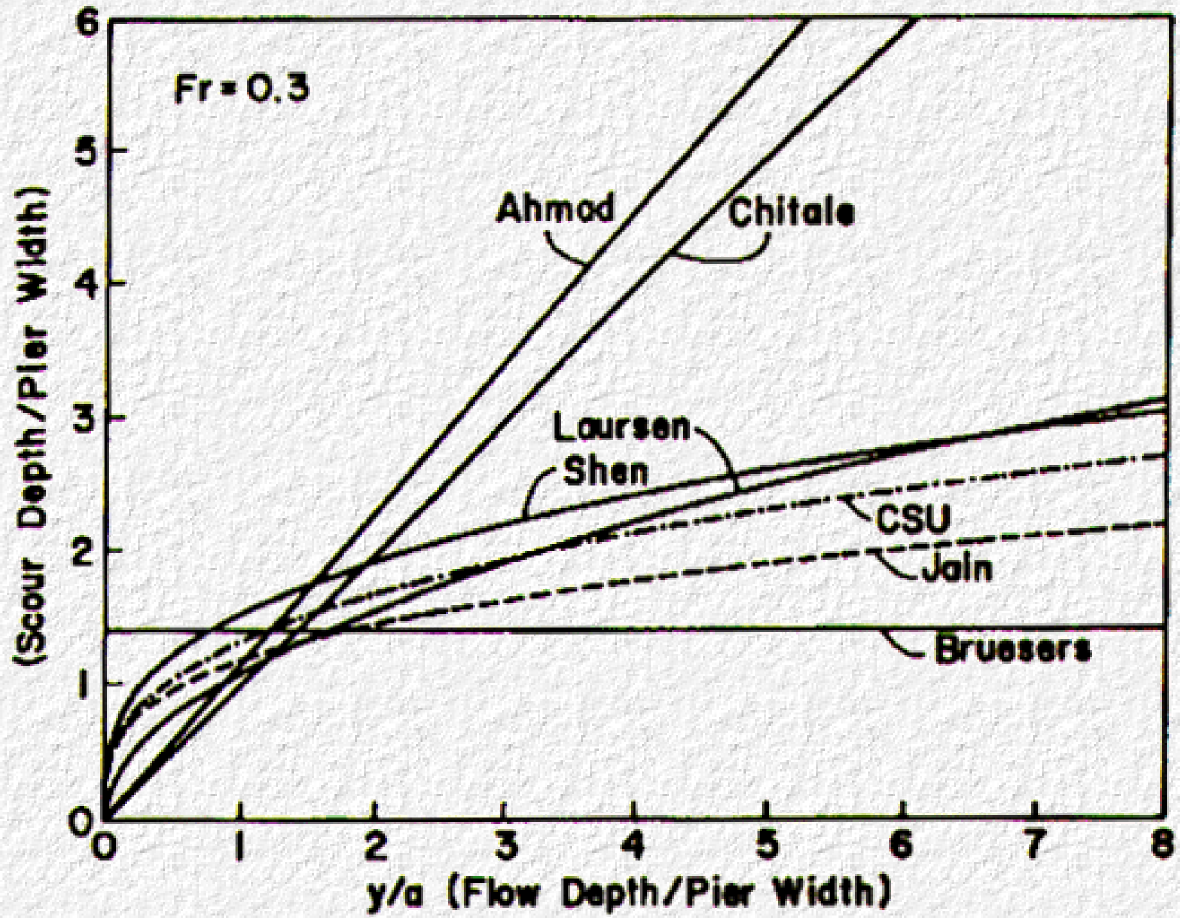


Figure 5.5.18. Comparison of Scour Formulas for Variable Depth Ratios (y/a) (Jones 1983)

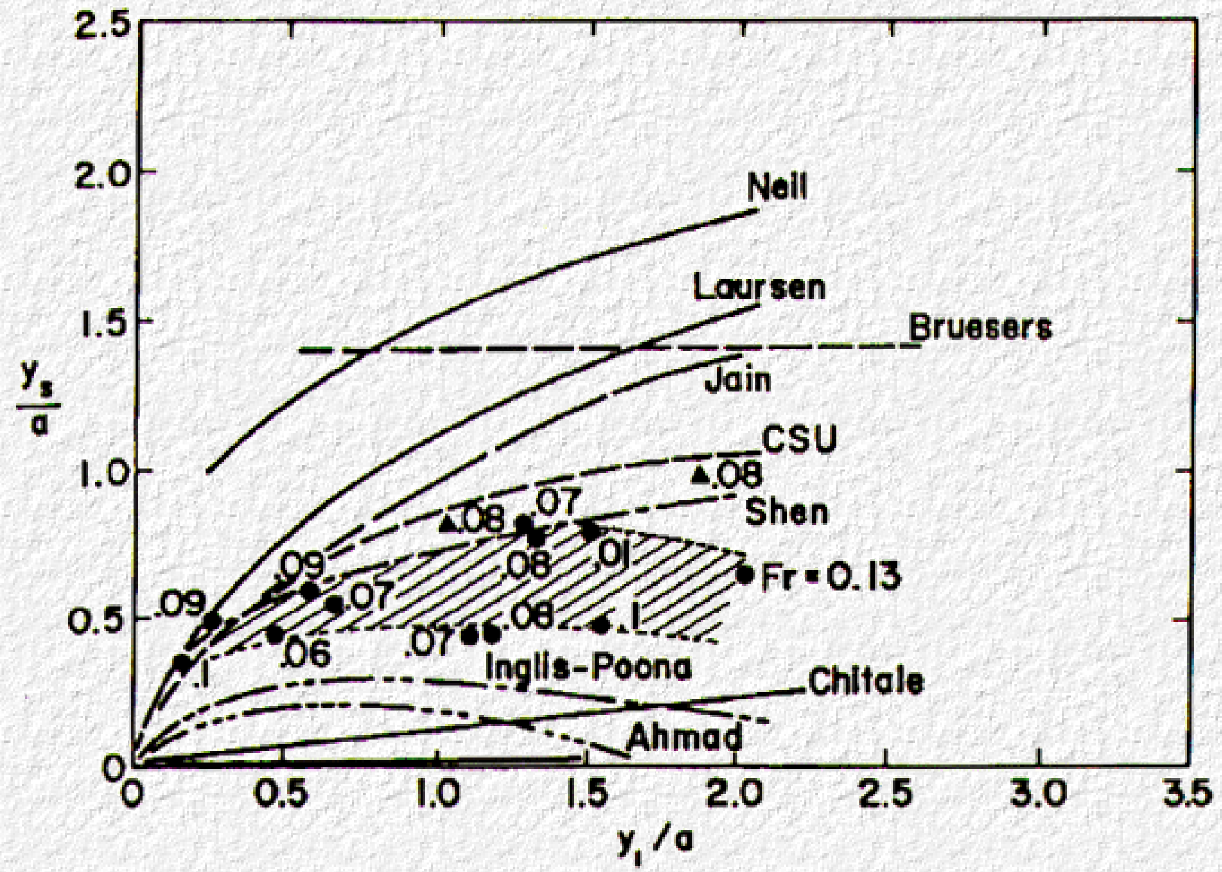


Figure 5.5.19. Comparison of Scour Formulas with Field Scour Measurements (Jones 1983)

The equations illustrated in Figure 5.5.18 and Figure 5.5.19 do not take into account the possibility that larger sizes in the bed material could armor the scour hole. That is, the large sizes in the bed material will at some depth of scour limit the scour depth. Raudkivi (Raudkivi and Sutherland, 1981, and Raudkivi and Ettema, 1983, Raudkivi, 1986) studied pier scour in streams with large particles in the bed. Washington State Department of Transportation (Copp and Johnson, 1987, and Copp, Johnson and McIntosh, 1988) developed an equation based on Raudkivi's research for streams with a large range of particle sizes which would tend to armor the scour hole. The significance of this factor of armoring of the scour hole over a long time frame and over many floods is not known. Therefore, their equation is not recommended for use at this time. However, it is given in this manual.

For the determination of pier scour, the Colorado State University's equation is recommended for both live-bed and clear water scour. With a dune bed configuration, the equation predicts equilibrium scour depths and maximum scour will be 30 percent greater. For flow with plane bed configuration given by Colorado State University's equation gives the maximum scour. And for antidunes the computed scour depths should be increased by 20 per-cent.

The extent to which a pier footing or pile cap affects local scour at a pier is not clearly determined. Under some circumstances the footing may serve as a scour arrester, impeding the horseshoe vortex and reducing the depth of scour hole. In other cases where the footing extends above the stream bed into the flow, it may serve to increase the effective width of the pier, thereby increasing the local pier scour. As an interim guide, if the top of the pier footing is slightly above (1 - 3 feet) or below the stream bed elevation (taking into account the effect of contraction scour) and the depth of flow significantly submerges the footing (y_1 2 to 3 times exposed footing), use the width of the pier shaft for the value of "a" in the pier scour equation. If the pier footing projects well above the stream bed (4 - 5 feet) to the extent that it significantly obstructs the flow, use the width of the pier footing for the value of "a." Interpolate between these two values depending upon the extent to which the footing may be expected to affect the local scour patterns.

(2) Pier Scour Equations

Colorado State University's Equation

The Colorado State University's Equation (Richardson et al., 1975) is as follows:

$$\frac{y_s}{y_1} = 2.0\, K_1 K_2 \left(\frac{a}{y_1}\right)^{0.65} Fr^{0.43} \qquad (5.5.26)$$

Where

y_s = Scour depth.

y_1 = Flow depth just upstream of the pier.

K_1 = Correction for pier shape from Table 5.5.2. and Figure 5.5.20.

K_2 = Correction for angle of attack of flow from Table 5.5.3.

a = Pier width.

Fr_1 = Froude number = $V_1 / (gy_1)^{0.5}$

V_1 = Velocity upstream of pier

Y_1 = Depth upstream of pier

Table 5.5.2 K_1 for Pier Type		Table 5.5.3 Correction Factor, K_2 for Angle of Attack of the Flow			
Type of Pier	K_1	Angle	L/a=4	L/a=8	L/a=12

(a)	Square nose	1.1	0	1.0	1.0	1.0
(b)	Round nose	1.0	15	1.5	2.0	2.5
(c)	Circular cylinder	1.0	30	2.0	2.5	3.5
(d)	Sharp nose	0.9	45	2.3	3.3	4.3
(e)	Group of cylinders	1.0	90	2.5	3.9	5.0
			Angle = skew angle of flow L = length of pier			

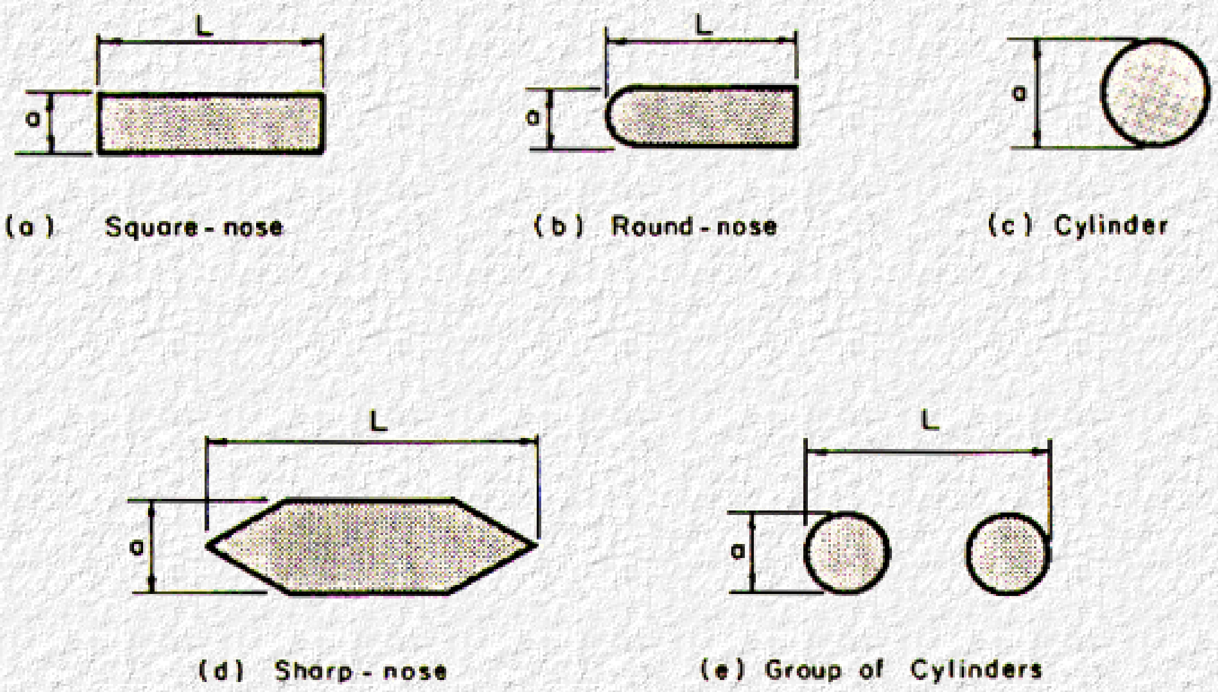


Figure 5.5.20. Common Shapes

The form of Equation 5.5.26 for piers is similar to Liu, et al's (1961) Equation 5.5.17 for vertical wall abutments, although the variable "a" takes on different meanings. This similarity exists because the local scour mechanisms for the two cases are similar.

Cylindrical piers have been widely investigated in the laboratory. The exponents in Equation 5.5.26 were determined from laboratory data shown in Figure 5.5.21. In this figure, the abscissa is labeled $(a/y_1)\ ^{3}Fr_1^{2}$ to spead the data.

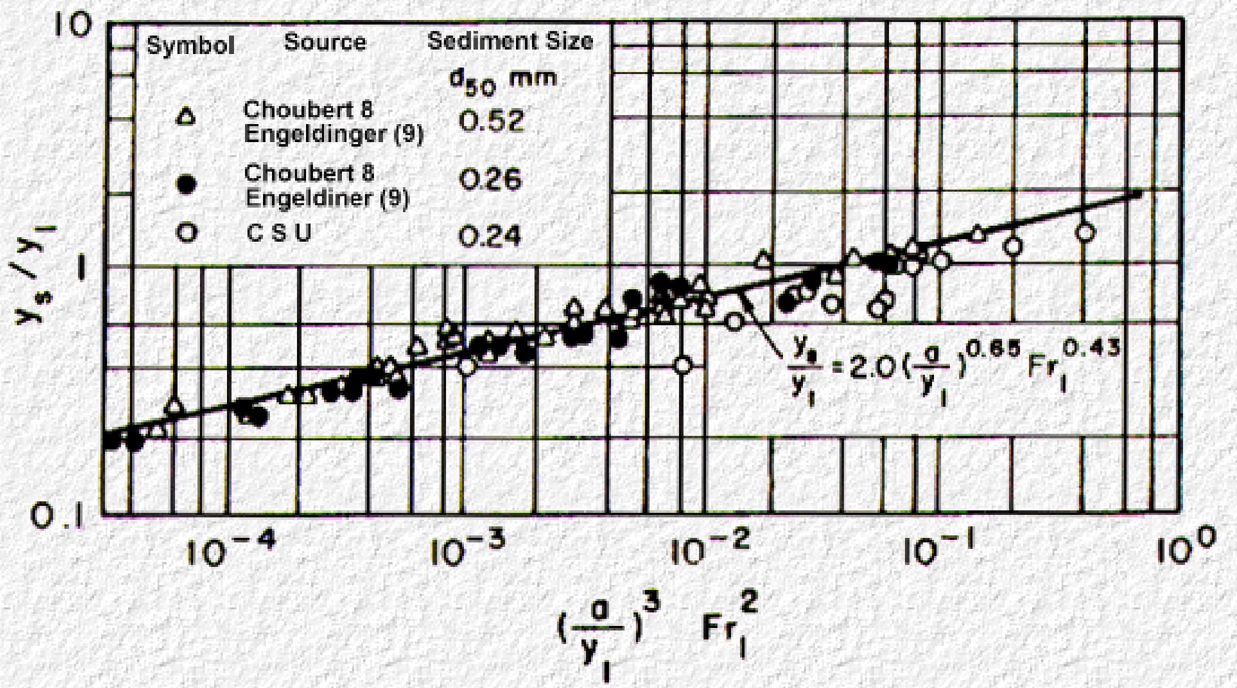


Figure 5.5.21. Results of Laboratory Experiments for Scour at Circular Piers

Jain and Fisher's Equations

Jain and Fisher (1979) studied in the laboratory local pier scour at large Froude numbers. They found that scour at a circular pier in sediment transport regime ($Fr > Fr_c$) first slightly decreased and then increased with the increase in the Froude number. Scour depth at high Froude numbers is larger than the maximum clear-water scour. The contribution of bed-form scour to the total scour depth in the upper flow regime becomes significant with higher flow velocities. They developed the following two equations:

For live-bed scour $(Fr - Fr_c) > 0.2$.

$$y_s / a = 2.0\,(Fr - Fr_c)^{0.25}(y_1 / a)^{0.5} \qquad (5.5.27)$$

For maximum clear water scour,

$$y_s / a = 1.84\,(Fr_c)^{0.25}(y_1 / a)^{0.3} \qquad (5.5.28)$$

These equations are a function of the critical Froude number Fr_c corresponding to pending sediment transport, the procedure for computing Fr_c is as follows:

1. Estimate the median diameter, D_{50}, for the bed material;

2. Determine τ_c from Lane's diagram (Figure 3.5.2);

3. Compute $U_{*}c = (\tau_c/\rho)^{0.5}$

4. Compute $\delta = 11.6\ \upsilon / U_{*}c$ (assume $\upsilon = 1.08 \times 10^{-5}$ ft^2/s);

5. Compute D_{50}/δ

6. Select X from Figure 2.3.5;

7. Compute $V_c = U_{*}c\ [(2.5\ \ell n\ (11\ y\ X/D_{50})]$ and

8. Compute $Fr_c = V_c/(gy_1)^{0.5}$

It is also recommended that the scour depth for $0 < (Fr - Fr_c) < 0.2$ can be assumed equal to the larger of the two values of scour obtained from Equation 5.5.27 and Equation 5.5.28.

For shapes different than circular piers and pier alignment other than parallel with the flow direction multiply the results given by Jain and Fisher's equations by the coefficients given in Table 5.5.2 and Table 5.5.3.

Graded and/or Armored Stream Bed Equations

There are very little field data for determining the decrease in scour depth as the result of coarse particles in the bed material of a stream. However, there are good indications (laboratory studies and some field data) that larger size particles in the bed material armor the scour hole and decrease scour depths.

Although field data is limited, equations are given here for this case. Until additional field data is available, they should be used with care and use of good engineering judgement.

The equations for circular piers, adapted from equations developed by Washington State Department of Transportation (Copp and Johnson, 1987, and Copp, Johnson, and McIntosh, 1988) from the work of Ravdkivi at the University of Aukland for streams with a large range of particle sizes which would tend to armor the scour hole, are as follows:

University of Aukland (UAK) Equations

For $(a/D_{50} > 18)$

$$y_s/a = 2.1\ K_1\ K_2\ K_3 \qquad (5.5.29)$$

For $(a/D_{50} < 18)$

$$y_s/a = 0.45\ K_1\ K_2\ K_3\ (a/D_{50})^{0.53} \qquad (5.5.30)$$

Where

y_s = Depth of local scour

a = Pier width

K_1 = Coefficient for pier type, Table 5.5.2.

K_2 = Coefficient for angle of attack of the flow, Table 5.5.3.

K_3 = Coefficient for effect of sediment grading, Figure 5.5.28.

K_g = Gradation coefficient = $(D_{84}/D_{16})^{0.5}$.

Copp and Johnson (1987) recommend that values obtained from the above equations be multiplyed by a factor of safety K_{fs} because there is little actual field data on scour depths in graded streambed material. They state the following:

"A purely heuristic approach is to select K_{fs} equal to $1/K_3$ whenever K_g is less than about 2.0. If K_3 is greater than 2.0, select K_{fs} = 1.5. This nullifies scour depth reductions for material gradations when K_3 < 2.0 but allows for the full depth of scour when K_3 > 2.0."

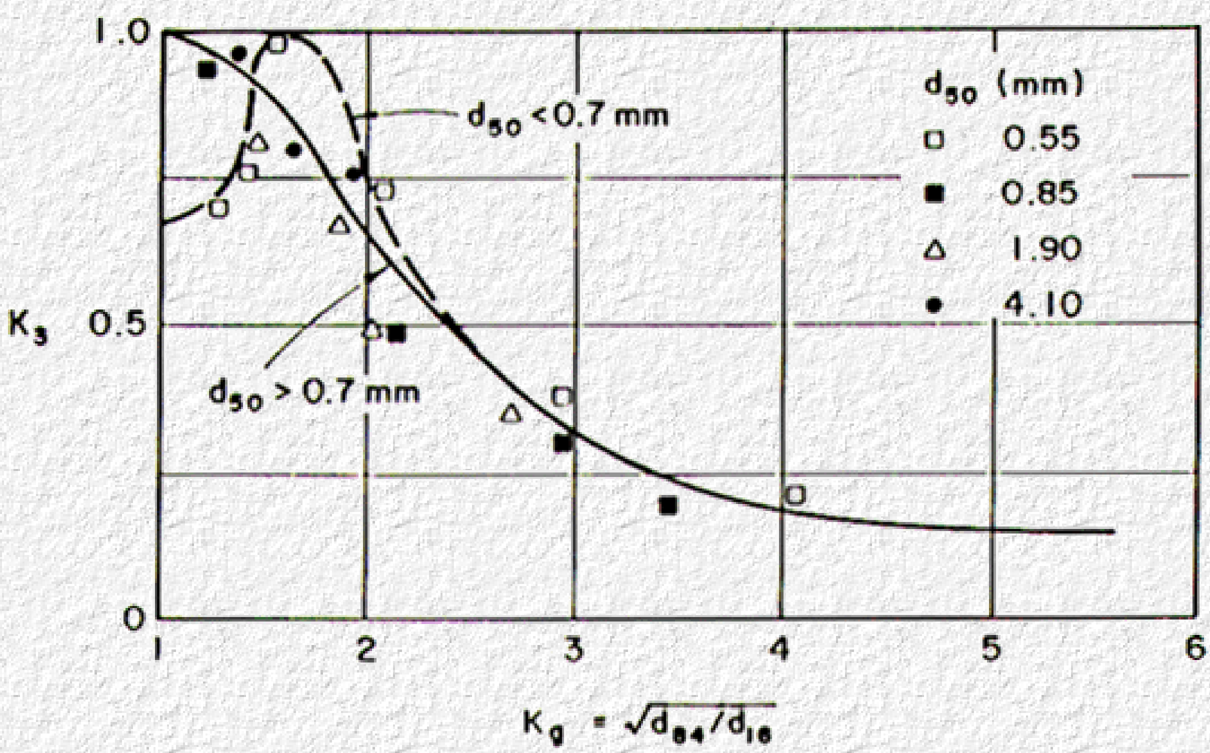


Figure 5.5.22. Particle Size Coefficient, K_3, vs. Geometric Deviation, K_g (Ettema, 1980)

Froehlich's Equation

Live-bed Scour.

Using linear regression analysis of 83 field measurements of pier scour, Froehlich (1988) developed the following equation:

$$y_s = 0.32\, K_1\, (a'/a)^{0.62}\, (y_1/a)^{0.46}\, Fr^{0.20}\, (a/D_{50})^{.08} + 1.0 \qquad (5.5.31)$$

Where

K_1 = Coefficient for pier type. Froehlich obtained K_1 = 1.3 for a square-nosed pier, 1.0 for round and round-nosed piers, and 0.7 for a sharp-nosed pier from his regression analysis. This is close agreement with values given in Table 5.5.2.

a' = Pier width projected normal to the approach flow.

a' = a cosΘ + L sinΘ. Where Θ is the angle of attach and L is pier length.

The other symbols are as defined before.

The = +1.0 in the equation is to give a factor of safety for design purposes. The regression analysis gives the expected value of the scour depths. Fifty percent of the scour holes could be deeper and fifty percent shallower. All the measured values of scour were less than the expected value + 1.0 as given in Equation 5.5.31. Froehlich's equation does not make allowances for bed forms. Where dune bed forms are expected, the estimated scour depth should be increased by a factor equal to $y_1/6$.

Clear-water Scour.

Froehlich (1988) only used live-bed scour data to develop Equation 5.5.31. He classified the data as to either clear-water or live-bed scour data on the basis of Neill's (1968) equation given below:

$$V_c = 1.58\, ((S_s - 1)\, gD_{50})^{0.5}\, (y_i/D_{50})^{0.167} \qquad (5.5.32)$$

Where

S_s = 2.65 was assumed for all measurements.
V_c = Critical mean velocity.

If V_c was larger than the mean velocity, then the scour was clear-water scour. However, all the clear-water scour depths in his data were less than those given by Equation 5.5.31. So Equation 5.5.31 can be used for clear-water scour.

(3) Example Problem

In Appendix 4, an example problem of scour around a circular pier is given.

5.5.8. Protection of Structures from Local Scour

Three basic methods may be used to protect structures from damage due to local scour. The first is to place the foundations of structures at such depth that the deepest scour hole will not threaten the stability of the structure. The second is to prevent erosive vortices from developing. The third is to provide protection at some level at or below the stream bed to arrest development of the scour hole. The first method is the recommended procedure for design unless risk analysis, cost and engineering judgment indicate a lesser scour depth could be used.

- **Vortex reduction** - Streamlining the piers can reduce scour depth by l0 to 20 percent. Another method of reducing the vortex strength at the pier is to construct barriers upstream of bridge piers, for instance, with a cluster of piles. While the piles are subjected to scour, failure of these piles is not damaging to the bridge. Debris can collect on the upstream piles keeping the nose of the bridge pier relatively free of debris. The pileup of water at the upstream piles reduces the dynamic pileup of water at the pier and reduces the vortex strength at the pier. Guidebanks can be placed at the ends of approach embankments to reduce local scour at the bridge.

  A structural concrete or steel shelf placed at about 0.5 y_s and extending laterally from the pier and completely surrounding the pier may be effective in limiting the scour depth. The lateral extent of the shelf should be about 0.3 y_s $\cot\Phi$ where Φ is the angle of repose of the bed material. While this method may be effective for $y_s < 20$ feet, it may become impractical for larger y_s.

- **Bed protection** - Riprap piled up around the base of the pier is a common method of local scour protection. The principle is the same as toe trenches for bank protection. The region of the bed beyond the riprap pile scours and as the scour hole is formed, the riprap slides down into the scour hole, eventually armoring the side of the scour hole adjacent to the pier. However, the riprap can be moved downstream during high flows and not protect the pier or abutment. Therefore, excess riprap should be placed and frequent inspections made. Preshaping the scour hole before placing the riprap may decrease the amount of riprap moved downstream. An estimate of the depth of scour is needed to determine the quantity of riprap required for effective protection. Because of armoring, the effective depth of scour is less than that calculated by the equations. There are few studies to establish dependable guidelines, but 50 to 60 percent reduction in y_s may be used for an estimate of final scour depth. By frequent inspection it can be determined whether the size and quantity of riprap used initially is adequate. If additional amounts of riprap are necessary, placement from the water surface may be possible in times of low flow with consideration given to the falling path of rocks in a flowing stream.

  Protective mattresses such as rock and wire have been suggested in the past, and have been used in a few circumstances. While they may have merit where adequate size riprap may be scarce, anchoring and stabilization of the mattresses to conform with scour holes is usually difficult. Use of mattresses in conjunction with riprap may be quite effective, if the mattress performs essentially as a flexible filter blanket which deforms as scour holes develop.

The size of rirrap can be determined from Figure 3.5.3. Using the curve for small turbulent stilling basins and the mean vertical velocity upstream of the pier.

Where rock mattresses or loose riprap is used consideration must be given to piping of fine, from underneath the armor layer. Piping would allow the armor layer to move downward. Should this occur, scour depths as deep as those without protection may develop. A gravel filter designed to prevent piping (Section 5.2.6), or a filter cloth is recommended where piping may occur.

5.6 Environmental Considerations

Streambank protection projects should be planned, designed and constructed with consideration being given to environmental quality factors and project objectives. Environmental quality should address preservation or restoration of environmental resources within the project boundaries and avoidance of adverse impacts associated with the project. Some environmental factors are project specific and are necessarily defined during the planning phase while others are mandated by existing regulations. Streambank protection projects should strive to preserve or restore existing environmental quality to the extent possible.

5.6.1 Environmental Impacts

Impacts of streambank protection projects are dependent on project location and regional characteristics. For example, in arid regions of the western United States, forested habitat may be restricted to riparian areas and be directly and extensively impacted by project construction; whereas in the southeastern United States, forested habitat may be more extensive, but changes in stream hydraulics caused by the project can result in adverse impacts throughout the floodplain ecosystem. While general categories of impacts may be stated, site-specific and regional considerations and individual project features are critical in determining the magnitude and type of environmental impact. In some cases streambank protection is performed in conjunction with other projects having different purposes, and it is difficult to isolate impacts due to streambank protection alone. Categories of environmental impacts associated with streambank protection projects include aesthetic, physical, water quality, and biological.

- Aesthetic impacts most often occur because the natural appearance of the project area is disturbed or changed and replaced by an artificial appearance due to structures or channel alignment.
- The physical impacts of streambank protection can affect channel morphology, sediment-carrying capacity of the stream resulting in localized accretion or degradation, and stream hydraulics. These physical effects tend to manifest themselves as changes in landscape diversity and associated aquatic habitat diversity or quality; for example, loss of side channels or shallow areas or replacement of natural bank with revetment. Losses or changes in habitat will affect wildlife and aquatic life either by a reduction or change in community structure; however, changes in habitat composition for a specific project can be either detrimental or beneficial depending on circumstances.
- Water quality impacts from changes in turbidity together with alteration of riparian habitat (e.g., shading) affect stream temperature and photosynthetic activities that in turn may affect algae or aquatic plant populations, dissolved oxygen, and other water quality parameters. Temporary changes in water quality may occur as a result of construction activities.
- Biological impacts can be broadly categorized into either terrestrial or aquatic. The major terrestrial impact involves alteration or elimination of riparian zone vegetation due to construction or project features. The riparian zone provides and supports a wide variety of plant and animal life and often provides critical habitat for certain species. Riparian vegetation also supports aquatic species by providing habitat for these species and input to the food chain. Channel stabilization can affect succession of riparian vegetation and decrease diversity. Aquatic organisms, including benthos and fish, may also be affected due to changes or reductions in required habitats as a result of project features.
- Other impacts that may occur due to streambank protection projects include loss of wetlands and historic sites, changes in land use, increased recreational pressure, and economic or social impacts.

5.6.2 Effects of Channelization on the Aquatic Life of Streams

Patrick (1973) assessed that the stream and its floodplain constitutes an integrated system that is well designed for moderating the effects of flooding waters and for maintaining high productivity in the stream. Disturbing the system inevitably results in a reduction in diversity of species and productivity. Because the functioning of the aquatic ecosystems is impaired, the ability of the stream to cleanse itself and to assimilate wastes is lessened, and the improvement of water quality is slower. The stream, instead of being one that is aesthetically pleasing and highly productive, becomes degraded and its recreational use is minimized. The chief effects of channelization are as follows:

1. Removes the natural diverse substrate materials that allow the development of many types of habitats for aquatic organisms;
2. Increases sediment load that decreases light penetration and primary production;
3. Creates a shifting bed load that is inimical to bottom-dwelling organisms;
4. Simplifies the current pattern and eliminates habitats of diverse currents;
5. Lowers the stream channel and often drains adjacent swamp areas and aquifers that help to maintain stream flow during time of low precipitation;
6. Destroys floodplain ponds that are the breeding ground for aquatic life and that act as a reservoir for species of the river proper; and
7. Reduces the stability of the banks and causes cave-in of trees and other overhanging vegetation that are an important food source for stream life and whose shade reduces high stream temperatures during the summer months.

If man is going to interfere and modify natural waterways, he should design his alterations to maintain the functioning of the aquatic ecosystem that makes possible the continuance or improvement of a stream's water quality.

More recently, Kellerhals et al. (1985) concluded that most existing engineering design criteria for river training works are in direct conflict with general concepts of fish habitat maintenance since their aims are well aligned, uniform and stable channels, with minimum local scour and no opportunity for debris jamming. The objectives of fisheries mitigation works are often quite the opposite; a degree of instability, rough and irregular banks, deep local scour holes, debris jams and overhanging vegetation. Well founded guidelines for designing such diverse and irregular channels (particularly in larger rivers) are needed and can only be developed on the basis of studies with a far broader scope than the normal project-oriented work funded by developers.

5.7 Guidlines for Channel Improvement, River Training and Bank Stabilization

The type of channel improvement and devices used for training and bank stabilization depends upon the size of river with regard to width, depth and discharge; type of river, that is, meandering, braided or straight; sediment transport in terms of concentration and size distribution; length of river to be protected; availability of materials; environmental considerations; aesthetics; legal aspects; river use with regard to navigation, recreation, agriculture, municipal and industrial purposes; and perhaps other factors. Decisions concerning highway locations near rivers or across rivers, and designs for specific devices to integrate the highways with the river systems are therefore very complex.

Table 5.7.1 is offered as a guide to assist the highway engineer with regard to decisions for channel improvement and selection of type of bank protection and river training works. The rivers are first categorized as to size and type. The descriptors large, medium and small are relative terms but should give no interpretive problem. Straight rivers are those which have sinuosity less than 1.5, but in terms of highway concerns, long reaches between meander bends which are essentially straight may be included in the straight river classification. Because they are part of the meander systems, stabilization and improvements may be required. This is the interpretation to be used in the table. An X indicates that consideration could be given to use of the particular devices, but consideration could be given to them all in special circumstances. Additional remarks are noted on the table.

The selection of an appropriate countermeasure type for a specific bank erosion/channel instability problem is dependent on many factors or selection criteria, including:

I. Structure function or purpose

- protection of an existing bankline, and
- flow control and/or constriction.

Table 5.8.1 Guide for Selection of Methods and Devices for River Channel Improvement and Bank Protection Works

Size of River	Type of River	Channel Improvement	DIKES			RETARDS		JETTIES		BANK PROTECTION					
												Mattresses			
			Timber	Stone-Fill	Earth	Timber	Steel Jacks	Timber	Steel Jacks	Riprap	Rock Trench	Rock & Wire	Concrete	Other	Cribs
Large	Meandering	X	X	X	*	*		X		X	X	*	X	X	
	Braided	X	X	X	*	*		X		X	X	*	X	X	
	Straight	X	X	X		*		X		X	X	*		X	
Medium	Meandering	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Braided	X	X	X	X	X	X	X	X	X	X	X	X	X	X
	Straight	X	X			X		X		X	X	#		X	X
Small	Meandering	X				X	X	X	X	X	X	#		X	X
	Braided	X				X	X	X	X	X	X	#		X	X
	Straight	X								X	X	#		X	X

* Floodplain Embankment Protection
# Where Large Rocks For Riprap Are Not Available

II. Erosion mechanisms

- streamflow - toe attack,
- streamflow - bank surface attack,
- surface weathering,
- abrasion,
- subsurface flow,
- wave erosion, and
- chemical action.

III. River characteristics
- channel size (width)
- channel bank characteristics,
- channel bed environment,
- radius of curvature,
- channel hydraulics, and
- ice and debris loadings.

IV. System impacts

V. Vandalism

VI. Maintenance

VII. Construction-related factors

VIII. Legal considerations

IX. Environmental considerations

X. Costs

Of these, the primary criteria are structure function, erosion mechanism encountered, and river environment.

These factors define the set of specific countermeasures that are best suited to specific site conditions. A typical design procedure is sketched in Figure 5.7.1. From this point, consideration of potential environmental impacts, maintenance, construction-related activities, and legal aspects can be used to refine the selection. The final section criteria, and perhaps the most important, is structure cost, the structure that provides the desired level of protection at the lowest cost will be the "best" for a particular application.

The following principles should be followed in designing and constructing bank protection and training works:

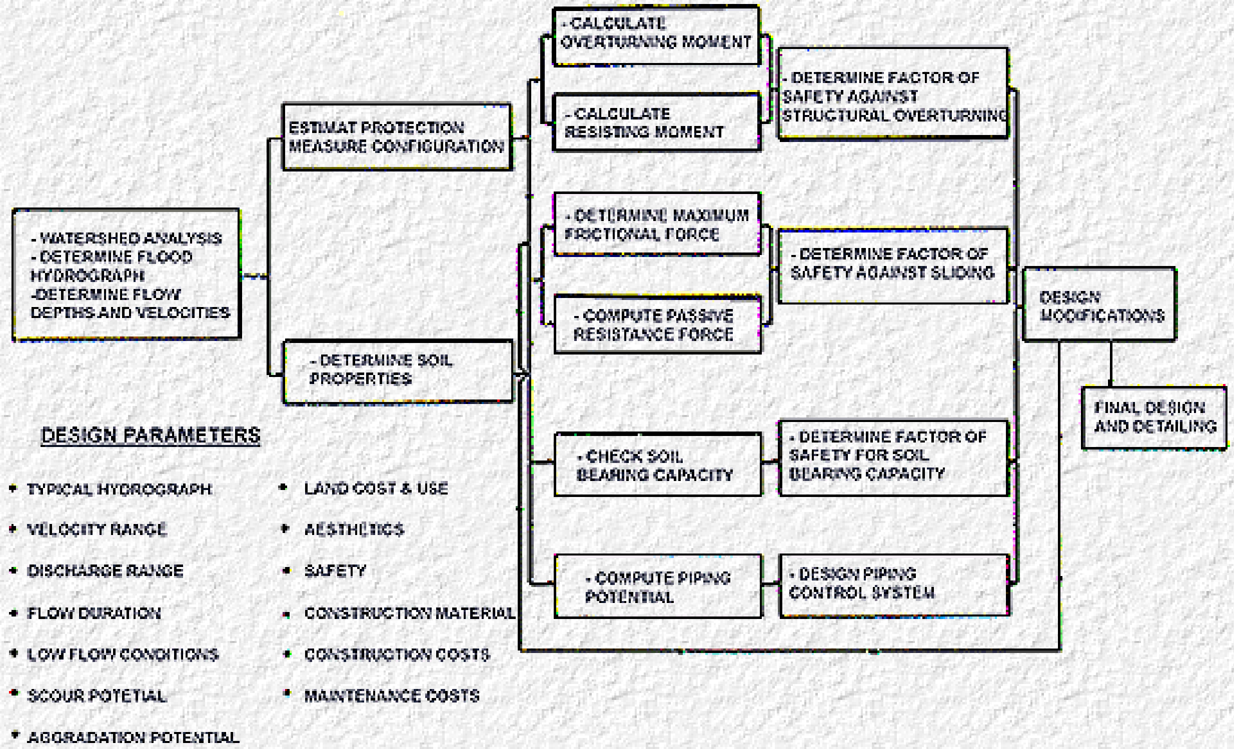


Figure 5.7.1. Typical Design Procedure - Streambank Stabilization and Drop Structure

1. The cost should not exceed the benefits to be derived. Permanent works should be used for important bridges on main roads and where the results of failure would be intolerable. Expendable works may be used where traffic volumes are light, alternative routes are available, and the risk of failure is acceptable;
2. Designs should be based on studies of channel trends and processes and on experience with comparable situations. The ultimate effects of the works on the natural channel both downstream and upstream should be considered;
3. Site reconnaissance by the designer is highly desirable. If circumstances prevent on-site inspection, aerial reconnaissance or air-photo study are possible substitutes;
4. The possibility of using model studies as a design aid should receive consideration at an early stage;
5. The works should be inspected periodically after construction with the aid of surveys, to check results and modify the design if necessary. The first design may require modification. Continuity in treatment, as opposed to sporadic attention, is advisable; and
6. In lieu of maintaining an existing road or bridge, consideration should be given to relocating it away from the river hazard.

Figure 5.7.2 indicates the results of a cost analysis conducted by Brown (1985). A total of 515 sites was used in the analysis; of these, 48 were spurs, 201 were revetments, 149 were retardance structures, 105 were longitudinal dikes, and 12 were bulkheads. The number of individual countermeasure types is included in the figure next to the name. The bar following each countermeasure type represents the cost range found. The darkened portion of the bar represents the dominant data range. The dominant data range was computed by first computing the average cost and then two standard deviations; the standard deviation of the data falling above the mean and the standard deviation of the data falling below the mean. Adding and subtracting these values (respectively) from the mean yields the dominant data range. When a countermeasure type did not have more than five sites for analysis of the dominant data range, no dominant range was computed, and only the total range is shown.

A quick scan of [Figure 5.7.2](#) reveals those countermeasures that are least expensive. Arbitrarily setting $100.00 per foot of bank protected as a cutoff point, the analysis indicates that rock riprap, spurs, horizontal wood-slat spurs, rock windrow revetments, vegetation, jack retards, wood-fence retards, and rock toe dikes will usually be the least expensive. Figure 5.7.2 also indicates that Henson-type spurs, large permeable diverter spurs, cellular block revetments, and concrete-filled mats typically will be the most expensive schemes. Also, tire mattresses show the largest variance in cost.

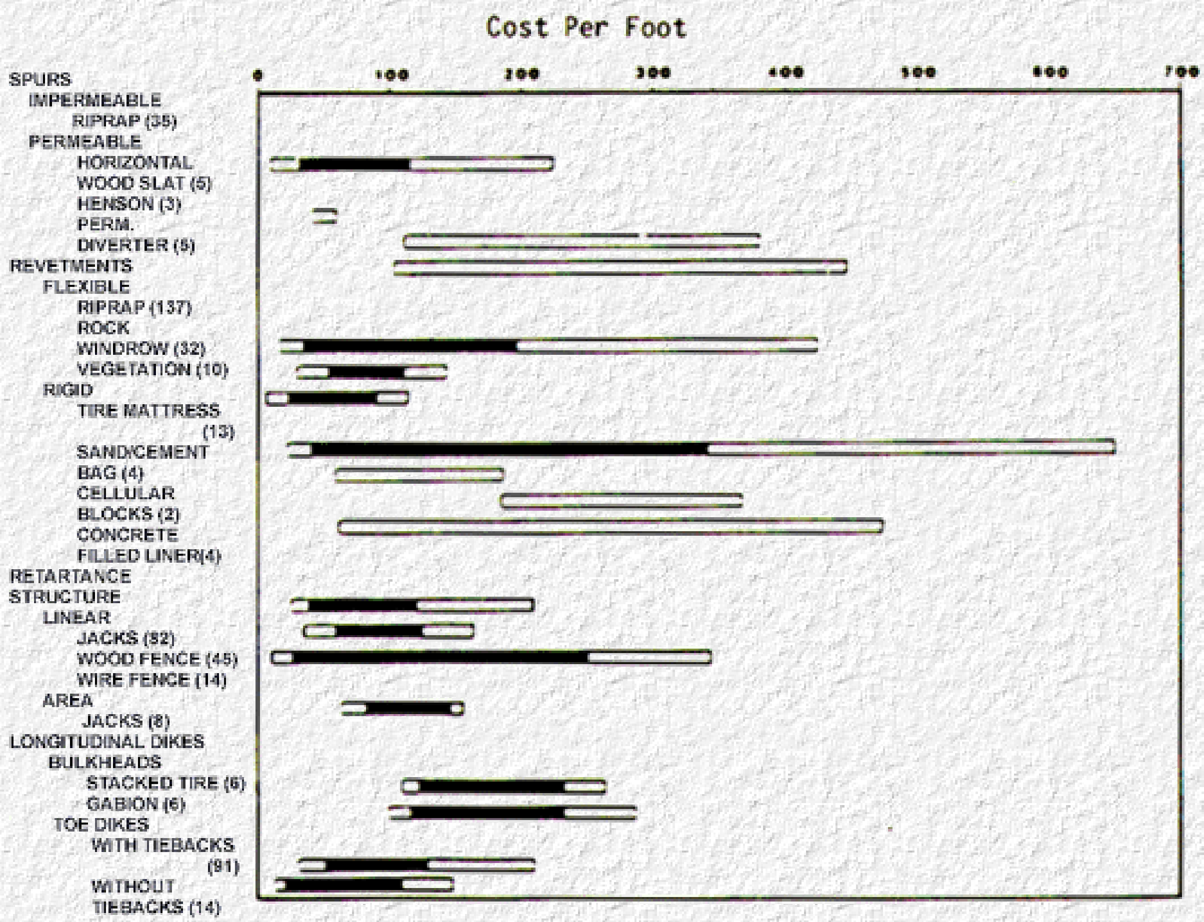


Figure 5.7.2. Costs Per Foot of Bank Protected (Brown, 1985)

Go to Chapter 6

Chapter 6 : HIRE
Data Needs and Data Sources

Go to Chapter 7, Part I

The objective and purpose of this chapter is to identify data needed for calculations and analyses which will lead to recommendations for highway crossings and encroachments of rivers. The types and amounts of data needed for planning and designing river crossings and lateral encroachments can vary from project to project depending upon the class of the proposed highway, the type of river and geographic area.

6.1 Basic Data Needs

The data, preliminary calculations, alternative route selections and analyses of these routes should be documented in a report. Such a report serves to guide the detailed designs, and provides reference background for environmental impact analysis and other needs such as application for permits and historical documentation for any litigation which may arise.

6.1.1 Area Maps

An area map is needed to identify the location of the entire highway project and all streams and river crossings and encroachments involved.

The purpose of the map is to orient the highway project geographically with other area features. The map may be very small scale showing towns, cities, mountain ranges, railroads and other highways and roads. The area map should be large enough to identify river systems and tributaries.

6.1.2 Vicinity Maps

Vicinity maps for each river crossing or lateral encroachment are needed to layout the proposed highway alignment and alternate routes. There should be sufficient length of river included on the vicinity map to enable identification of stream type and to locate river meanders, sand bars, and braided channels. Other highways and railroads should be identified. The maps should show contours and relief. Intakes for municipal and industrial water, diversions for irrigation and power, and navigation channels should be clearly identified.

Recreational areas such as camping, picnic grounds, beaches, and recreational boat docks should be identified. Cultivated areas and urban and industrial areas, in the vicinity of towns and cities should be noted on the map. The direction of river flow should of course be clearly identified.

6.1.3 Site Maps

Site maps are needed to determine details for hydraulic, roadway, and structural designs. The site map should show detailed contours of one or two foot intervals, vegetation distribution and type, and other structures. The site map is used to locate highway approach embankments, piers and alignments of piers, channel changes, and protection works. High water lines should be indicated on the site maps for the purpose of estimating flood flows and distributions across the river cross section.

6.1.4 Aerial and Other Photographs

It is highly desirable in preparing vicinity and site maps that aerial photographs be obtained. Modern multi-image cameras use different ranges of the light spectrum to assist in identifying various features such as sewer outfalls, groundwater inflows, types of vegetations, sizes and heights of sandbars, river thalwegs, river controls and geologic formations, existing bank protection works, old meander channels, and other features. Contour lines can also be developed from aerial photographs for vicinity and site maps where such information is not readily available. Land photographs (as opposed to aerial photos) of existing structures near the crossing are always helpful in documentation and evaluation of potential effects of highway construction. Photographs of water intake works likely to be affected by the highway project should be obtained, and specific data should be noted and briefly discussed. High water marks recorded photographically along with dates of occurrence are useful. Photographs aid the designer, who may not have the opportunity to visit the site, to visualize crossings and encroachments, and they aid documentation.

Conditions of the river channel in the river reach of concern are easy to record photographically, and such pictures can be very helpful in analysis of the river reach. Vegetation on floodplains and seasonal variations of vegetation should be recorded photographically. Notable geologic formations should be photographed as well and supplemented with adequate notes. All photographs should be referenced on the site or on vicinity maps.

6.1.5 Field Inspection

A field inspection of potential highway encroachment sites of rivers should be made prior to the analysis. This has been implied in the foregoing paragraphs but is emphasized again because of the underlying importance of making first hand appraisals of specific sites before conclusions and recommendations are advanced for possible highway routes. Of course, they are important in making detailed designs as well but it is not always feasible to provide opportunities for site inspection by the entire design staff.

6.1.6 Geologic Map

A geologic vicinity map, on which geophysical features are indicated, is a basic need. The rock formations, outcroppings, and glacial and river deposits which form control points on rivers are valuable in analysis of rivers. Soil type determines the size of sediment in transport, infiltration rates, and groundwater flows. Channel geometry and roughness are important factors in river mechanics.

Soil survey maps with engineering interpretations are available for a significant proportion of the United States. They may be helpful in selecting layouts and assessing the suitability of fill materials.

6.1.7 Climatologic Data

Stream gaging stations have been established on many streams throughout the United States. However, there are some streams where either a gaging station does not exist near the project site or a gaging station does not exist at all. In such cases it is necessary to estimate flood flows. These estimates may be based on regionalized estimating procedures or other prediction models using meteorological and watershed data inputs. These meteorological data are available from the National Weather Service (NWS) Data Center of the National Oceanic and Atmospheric Administration (NOAA), and estimates of average conditions can be made from rainfall data published by the NWS. Temperature records are helpful in making snowmelt estimates, and wind data are helpful in making wave height estimates on rivers, lakes and reservoirs as well as for coastal areas.

6.1.8 Hydraulic Data

Whenever possible, sediment load data should be provided as auxiliary data for river analyses. Bed-material load, suspended load and wash load data may be obtained for some rivers in the water supply papers published by the U.S. Geological Survey, state engineers' reports, flood control and other water resources investigation reports. Information may also be obtained by direct sampling of the river.

Riverbed cross sections and profiles may be obtained with an ultrasonic depth sounder and are helpful in sediment transport and backwater studies. It is also helpful to know water temperatures. Direct measurement of flood flows should be made when historical records may be deficient. Depth and velocity measurements need to be made at a sufficient number of subsections in a cross section to determine total flow rate. Discharge measurements made at various stages at a gaging site can provide data for developing a stage-discharge rating curve.

Observations of high water marks along the river reach should be made. Each high water mark and relevant profile should be established. These are helpful in calculating historical flood discharges. Also, stages achieved by ice jams at specific locations should be noted.

Channel changes which have occurred after floods are of particular interest in evaluating future effects on channel planform. Whenever possible, historic aerial photographs or equivalent maps which show river channels should be obtained.

Records of the performance of existing bridges and other drainage structures should be obtained. Data on scour at piers of existing bridges (or at bridges which have failed) in the vicinity should be obtained. For bridges which have failed, as much information as possible should be obtained relative to direction of flow (angle of attack) at the piers or embankment ends. Flood duration, debris in the river, distribution of flows, and magnitudes of scour are useful information. Historical records of damage to adjacent property and results of legal actions brought about because of damage are useful information also.

6.1.9 Hydrologic Data

The purpose of hydrologic data is to determine the stream discharge, flood magnitudes, and duration and frequencies of flood prior to analysis of river behavior and design of the river encroachments and crossings. Hydrologic data and hydraulic analyses should be documented in report form for project development. After construction the documentation would be helpful in evaluating any damage from floods and failures, in the event they occur, and providing background for any litigation which may arise as a consequence.

Sometimes a highway crossing and/or encroachment may have a significant effect on flood hydrographs and a hydrologic analysis should be made to determine the level of significance. This analysis would involve hydrograph development and flow routings within the zone of influence of such highway structures.

The basic data needed are stream discharge data at the nearest gaging station, historical floods and highwater marks. It is also desirable to prepare a drainage map for the region upstream of the proposed highway project, with delineation of size, shape, slope, land use, and water resource facilities such as storage reservoirs for irrigation and power and flood control projects. It is desirable whenever possible to obtain flood histories of the river from residents and accounts by the news media, particularly for events prior to stream gaging records. The accounts of high water and period of years estimates of flood discharge can be made which are valuable in flood-frequency analysis.

A flood-frequency curve is prepared from recorded stream flow data and augmented by estimated discharges (using Manning's equation or equivalent) from high water marks. Several methods ranging from sophisticated stochastic analysis to simple methods have been developed. The greatest difficulty in constructing a flood-frequency curve is lack of sufficient and reliable data. Approximate methods for extrapolating the range of flood-frequency curves are available but are not discussed in detail here. (See HEC-19).

A simple graphical method based on extreme value theory is reasonably satisfactory. The method consists of ordering the annual peak flood discharges of record from the largest to smallest, irrespective of chronological order. The annual (flood) discharge is plotted against its recurrence interval on special probability (Gumbel, or other) paper. The recurrence interval, RI is calculated from

$$RI = \frac{n+1}{m} \qquad (6.1.1)$$

in which n is the number of years of records, and m is the order (largest flood is ranked 1) of the flood magnitude. Thus, the highest flood discharge would have a recurrence interval of n + 1 years and lowest would have a recurrence interval of (1 + 1/n) years. The U.S. Water Resources Council (1972) has adopted the log-Pearson III distribution for use as a base method for determining flood flow frequencies. Details of the method and plotting paper may be obtained from the U.S. Geological Survey or the Federal Highway Administration (Washington or regional offices).

When adjusting discharge records from a nearby gaging station to the project site, the flood peaks are often prorated on the basis of drainage area ratios. Depending on drainage basin characteristics, the exponent of the ratio varies from 0.5 to 0.8. Slope-area calculations for peak discharges can also be used. In using this method, the conveyance of the channel is calculated using the Manning equation in which the roughness coefficient, n, needs to be estimated from the discussion presented in Chapter 2 and Chapter 3. By referring to a catalog of (color) photographs, similar channel situations to the specific site can be identified and a relatively inexperienced engineer may make a reliable estimate for n.

Whatever approach is used, the reader is cautioned not to blindly accept computer printout as the final answer in estimating a flood frequency relationship. The data should be plotted on probability paper as analyzed by several commonly used methods. Sometimes paleo (ancient) hydrology techniques need to be employed to resolve historic outliers at very sensitive sites.

6.1.10 Environmental Data

In making environmental impact analyses of highway projects on streams and rivers, it is necessary to obtain water quality and biological data for the streams. Such data is not readily available for many rivers. Municipal water and sewage treatment facilities and industrial plants utilizing river water should have recent records regarding river water quality which will be helpful in making comprehensive environmental analyses. Water quality data for certain rivers can be obtained from the U.S. Geological Survey. Wildlife information such as migration patterns of deer and elk should be determined and local game refuges should be located. Information regarding fishes and their river habitat should be obtainable from the state fish and game agencies. Species of trees and other vegetation should be determined, and some information regarding sensitivity of the flora to auto emissions should be obtained. Data should also be obtained in order to enable assessment of stream turbidity during and after highway construction. Information on soil type to be used in construction of embankments would be helpful in this regard.

6.2 Checklist of Data Needs

As an aid in collecting data preparatory to analysis of rivers and highway encroachment of rivers, the relevant types of data have been listed in Table 6.2.1. There may be more data items included in this table than are needed for a given project site, and some judgment is required. For data which are not available, the checklist should be helpful for planning a field investigation or other data acquisition program.

Table 6.2.1 Checklist of Data Needs
Maps and Charts:

(1) Geographic
(2) Topographic
(3) Geologic
(4) Navigation Charts
(5) Potamology Surveys
(6) County and City Plats

Aerial and Other Photos:

(1) Large Scale Photos for Working Plans
(2) Small Scale Stereo Pairs of River and Surrounding Terrain
(3) Color Infrared Photos for Flow Patterns, Scour Zones, and Vegetation
(4) Ground Photos
(5) Underwater Photos

Information on Existing Structures, Bridges, Dams, Diversion or Outfalls:

(1) Plans and Details
(2) Construction Details
(3) Alterations and Repairs
(4) Foundations
(5) Piers and Abutments
(6) Scour
(7) Dikes
(8) Field Investigations:

- Investigating bridge structure & repairs to bridge & approach
- Damage due to ice or debris

Hydraulic, Hydrology and Soils:

(1) Discharge Records
(2) Stage-Discharge Records
(3) Flood Frequency Curves for Stations Near Site
(4) Flow Duration Curves (hydrographs)
(5) Newspaper, Radio, Television, Accounts of Large Floods
(6) Channel Geometry:

- Main channel
- Side channel
- Navigation channel
- Floodplain
- Slopes
- Backwater calculation
- Bars
- Sinuosity
- Type (braided, meandering, straight)
- Controls (falls, rapids, restriction, rock outcropping dams, diversions)

(7) Sediment Discharge:

- Size distribution
- Bed and Bank Material Sizes
- Roughness Coefficient n

(8) Ice:

- Recorded thickness
- Dates of freeze up and break up
- Flow patterns and jams

(9) Regulating Structures:

- Dams, diversions
- Intake, outfalls
- Scour survey around existing piers, abutments, spur dikes
- Inspect and photograph stabilization works, riprap sizes, filter blankets
- Check wells for groundwater levels in areas
- Install gaging stations

(10) Soils Information:

- Excavation data
- Borrow pits
- Gravel pits
- Cuts
- Tunnels
- Core boring logs
- Well drilling logs
- Soil tests
- Permeability
- Rock for riprap

(11) Planned and Anticipated Water Resources Projects
(12) Lakes, Tributaries, Reservoirs or Side Channel Impoundments
(13) Field Surveys:

- Onsite inspections and photographs
- Samples of sediments
- Measure water and sediment discharge
- Observe channel changes or realignment since last maps or photos
- Identify high water lines or debris deposits due to recent floods
- Check magnitude of velocities and direction of flow in vicinity of proposed structure

- Outcroppings
- Subsurface Exploration

Climatological Data:
(1) National Weather Service Records for Precipitation (2) Wind (3) Temperatures
Land Use:
(1) Zoning Maps (2) Recent Aerial Photographs (3) Planning Committee Records (4) Urban Areas (5) Industrial Areas (6) Recreational Areas (7) Primitive Areas (8) Forests (9) Vegetation

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6.3 Data Sources

The best data sources are national data centers where the principal function is to disseminate data. But it might be necessary to collect data from a variety of other sources such as from a field investigation, interviews with local residents, and a search through library material. Detailed information on the location of these federal agencies across the U.S. is available in Appendix A of the manual HEC-19, Report FHWA-IP-89-15. The following list of sources is provided to serve as a guide to the data collection task:

- Topographic Maps:

  (1) Quadrangle maps -- U.S. Department of the Interior, Geological Survey, Topographic Division; and U.S. Department of The Army, Army Map Service.
  (2) River plans and profiles -- U.S. Department of the Interior, Geological Survey, Conservation Division.
  (3) National parks and monuments -- U.S. Department of the Interior, National Park Service.
  (4) Federal reclamation project maps -- U.S. Department of the Interior, Bureau of Reclamation.
  (5) Local areas -- commercial aerial mapping firms.
  (6) American Society of Photogrammetry.

- Planimetric Maps:

  (1) Plans of public land surveys -- U.S. Department of the Interior, Bureau of Land Management.

(2) National forest maps -- U.S. Department of Agriculture, Forest Service.
(3) County maps -- State Highway Agency.
(4) City plans -- city or county recorder.
(5) Federal reclamation project maps -- U.S. Department of the Interior, Bureau of Reclamation.
(6) American Society of Photogrammetry.
(7) ASCE Journal -- Surveying and Mapping Division.

- Aerial Photographs:

(1) The following agencies have aerial photographs of portions of the United States: U.S. Department of the Interior, Geological Survey, Topographic Division; U.S. Department of Agriculture, Commodity Stabilization Service, Soil Conservation Service and Forest Service; U.S. Air Force; various State agencies; commercial aerial survey and mapping firms; National Oceanic and Atmospheric Administration.
(2) American Society of Photogrammetry.
(3) Photogrammetric Engineering.
(4) Earth Resources Observation System (EROS); Photographs from Gemini, Apollo, Earth Resources, Technology Satellite (ERTS) and Skylab.
(5) City or Country Records
(6) State Highway Agency

- Transportation Maps:

(1) State Highway Agency.
(2) Large Cities

- Triangulation and Benchmarks:

(1) State Engineer.
(2) State Highway Agency.
(3) Cities

- Geologic Maps:

(1) U.S. Department of the Interior, Geologic Survey, Geologic Division; and State Geological Surveys Departments. (Note - some regular quadrangle maps show geological data also.)

- Soil Data:

(1) County soil survey reports -- U.S. Department of Agriculture, Soil Conservation Service.
(2) Land use capability surveys -- U.S. Department of Agriculture, Soil Conservation Service.
(3) Land classification reports -- U.S. Department of the Interior, Bureau of Reclamation.
(4) Hydraulic laboratory reports -- U.S. Department of the Interior,

Bureau of Reclamation.
(5) State Universities and State Agricultural and Conservation Agencies.

- Climatological Data:

(1) National Weather Service Data Center.
(2) Hydrologic bulletin -- U.S. Department of Commerce, National Oceanic and Atmospheric Administration.
(3) Technical papers -- U.S. Department of Commerce, National Oceanic and Atmospheric Administration.
(4) Hydrometeorological reports -- U.S. Department of Commerce, National Oceanic and Atmospheric Administration, and U.S. Department of the Army, Corps of Engineers.
(5) Cooperative study reports -- U.S. Department of Commerce, National Oceanic and Atmospheric Administration and U.S. Department of the Interior, Bureau of Reclamation.

- Stream Flow Data:

(1) Water supply papers -- U.S. Department of the Interior, Geological Survey, Water Resources Division.
(2) Reports of State Engineers.
(3) Annual reports -- International Boundary and Water Commission, United States and Mexico.
(4) Annual reports -- various interstate compact commissions.
(5) Hydraulic laboratory reports -- U.S. Department of the Interior, Bureau of Reclamation.
(6) Corps of Engineers, U.S. Army, Flood control studies.
(7) Tennessee Valley Authority.
(8) State Highway Agency.
(9) USGS, FEMA Flood Studies.
(10) University Studies

- Sedimentation Data:

(1) Water supply papers -- U.S. Department of the Interior, Geological Survey, Quality of Water Branch.
(2) Reports -- U.S. Department of the Interior, Bureau of Reclamation; and U.S. Department of the Agriculture, Soil Conservation Service.
(3) Geological Survey Circulars -- U.S. Department of the Interior, Geological Survey.

- Quality of Water Reports:

(1) Water supply papers -- U.S. Department of the Interior, Geological Survey, Quality of Water Branch.
(2) Reports -- U.S. Department of Health, Education, and Welfare, Public Health Service.
(3) Reports -- State Public Health Departments.
(4) Water Resources Publications -- U.S. Department of the Interior,

Bureau of Reclamation.
(5) Environmental Protection Agency, regional offices.
(6) State Water Quality Agency.

- Irrigation and Drainage Data:

(1) Agricultural census reports -- U.S. Department of Commerce, Bureau of the Census.
(2) Agricultural Statistics -- U.S. Department of Agriculture, Agricultural Marketing Service.
(3) Federal Reclamation Projects -- U.S. Department of the Interior, Bureau of Reclamation.
(4) Reports and Progress Reports -- U.S. Department of the Interior, Bureau of Reclamation.

- Power Data:

(1) Directory of Electric Utilities -- McGraw Hill Publishing Co.
(2) Directory of Electric and Gas Utilities in the United States -- Federal Power Commission.
(3) Reports -- various power companies, public utilities, State power commissions, etc.

- Basin and Project Reports and Special Reports:

(1) U.S. Department of the Army, Corps of Engineers.
(2) U.S. Department of the Interior, Bureau of Land Management, Bureau of Mines, Bureau of Reclamation, Fish and Wildlife Service, and National Park Service.
(3) U.S. Department of Agriculture, Soil Conservation Service.
(4) U.S. Department of Health, Education, and Welfare, Public Health Service.
(5) State Departments of Water Resources, Departments of Public Works, power authorities, and planning commissions.

- Environmental Data:

(1) Sanitation and public health -- U.S. Department of Health, Education, and Welfare, Public Health Service; State Departments of Public Health.
(2) Fish and Wildlife -- U.S. Department of the Interior, Fish and Wildlife Service; State Game and Fish Departments.
(3) Municipal and Industrial Water Supplies -- City Water Departments,; State Universities; Bureau of Business Research; State Water Conservation Boards or State Public Works Departments; State Health Agencies; Environmental Protection Agency, Public Health Service.
(4) Watershed Management -- U.S. Department of Agriculture, Soil Conservation Service, Forest Service; U.S. Department of the Interior, Bureau of Land Management, Bureau of Indian Affairs.
(5) State Highway Administration.

6.4 Computerized Literature and Data Search

Recent literature information can be retrieved from computerized databases of technical information. The principal databases related to highway and river environment are: COMPENDEX, ENVIRONMENTAL BIBLIOGRAPHY, FLUIDEX, WATER RESOURCES ABSTRACTS, NAWDEX/WATSTORE, STORET, and TRIS. Some details regarding these databases are given below:

6.4.1 COMPENDEX

1970 - Present, 1,415,000 records, monthly updates (Engineering information, Inc., New York, New York). The COMPENDEX database is the machine-readable version of the Engineering Index (Monthly/Annual), which provides abstracted information from the world's significant engineering and technological literature. The COMPENDEX database provides worldwide coverage of approximately 3500 journals and selected government reports and books. SDI: $5.95/update - $99.00 per online connect hour, 47¢ per full record printed offline, 35¢ per full record typed or displayed online.

6.4.2 ENVIRONMENTAL BIBLIOGRAPHY

1973 - Present, 275,000 records, bimonthly updates (Environmental Studies Institute, Santa Barbara, California). The ENVIRONMENTAL BIBLIOGRAPHY covers the fields of general human ecology, atmospheric studies, energy, land resources, water resources, and nutrition and health. More than 300 periodicals are indexed in ENVIRONMENTAL BIBLIOGRAPHY, thereby providing quick-and-easy access to article references for every environment research need. Librarians, chemists, land-use planners, government officials, and corporate executives, among others will find this database a functional asset to their work. $60.00 per connect hour, 15¢ per full record printed offline.

6.4.3 FLUIDEX

1973 - Present, 158,000 records, monthly updates (BHRA, The Fluid Engineering Centre, Cranfield, Bedford, England). FLUIDEX provides indexing and abstracting of every aspect of fluid engineering, including theoretical research as well as the latest technology and applications. BHRA produces twelve secondary abstract publications and the content of these represents about 70 percent of the database contents. Database coverage includes aerodynamics; wind energy and the fluid dynamics and aspects of noise; coastal and inland fluid engineering works; offshore technology; river and flood control; engineering of surface water management structures; multi-phase flow; mixing/separation; flow measurement and instrumentation; fluid power; fluidics; high pressure technology - jet cutting; computational fluid mechanics; mathematical modeling; fluid sealing; materials properties; process engineering ; dredging/mining; pumps and pump technology; tribology; rheology and energy extraction; storage and conversion. Nearly 1000 technical journals are covered as well as books, conference proceedings, standards, some British patents, and research reports from relevant institutions worldwide. $69.00 per online connect hour, 28¢ per full record printed offline, 18¢ per full record typed or displayed online.

6.4.4 WATER RESOURCES ABSTRACTS

1968 - Present, 176,000 records, monthly updates (U.S. Dept. of the Interior, Washington, D.C.) WATER RESOURCES ABSTRACTS is prepared from materials collected by over 50 water research centers and institutes in the United States. The file covers a wide range of water resource topics including water resource economics, ground and surface water hydrology, metropolitan water resources planning and management and water-related aspects of nuclear radiation and safety. The collection is particularly strong in the literature on water planning (demand, economics, cost allocations), water cycle (precipitation, snow, groundwater, lakes, erosion, etc.), and water quality (pollution, waste treatment). WRA covers predominantly English-language materials and includes monographs, journal articles, reports patents, and conference proceedings. $45.00 per online connect hour, 15¢ per full record printed offline.

6.4.5 TRIS

TRIS is a composite file whose records are either abstracts of documents and data holdings, or resumes of research projects that are relevant to the planning, development, operation, and performance of transportation systems and their components. Source files and updates for the database are supplied through the cooperation of information services and centers that specialize in transportation information and whose combined holdings cover much of the transportation field. Users may search the entire TRIS database or may restrict their searches to any combination of source files and record types (abstracts or resumes).

6.5 Expert Systems

An expert system is a computer program that draws on a base of knowledge in some specific area to make conclusions by means that imitate human reasoning. Expert systems have at least two basic components: a knowledge base (or set of rules) and an inference engine - logic that arranges and works with the rules.

Information assembled under the Environmental and Water Qualities Studies (EWQOS) Program offers some assistance to environmental resources personnel who wish to screen available bank protection techniques and select a small number of candidate alternatives for detailed analysis by design personnel. Most of the information is contained in Henderson and Shields (1984), and short-term technical assistance is available under the Water Operations Technical Support (WOTS) Program. In order to provide this information in an easy-to-use format that allows the planner to focus on the most promising techniques, the Waterways Experiment Station (WES) is developing an expert system that will be known as ENDOW. ENDOW, an acronym for "Environmental Design of Waterways," is being developed to assist designers in evaluating environmental aspects of various bank protection alternatives. Eventually ENDOW will contain modules to assist with channelization and levee projects as well as streambank erosion control.

Information services operated by the transportation research board include: Air Transportation Research Information Service (ATRIS), sponsored by the Federal Aviation Administration, U.S. Department of Transportation (DOT); Highway Research Information Service (HRIS), sponsored by the State Departments of Highways and Transportation and the Federal Highway Administration, U.S. DOT; Maritime Research Information Service (MRIS), sponsored by the Maritime Administration, U.S. Department of Commerce; Railroad Research Information Service (RRIS), sponsored by the Federal Railroad Administration, U.S. DOT; Urban Mass Transportation Research Information Service (UMTRIS), sponsored by the Urban Mass Transportation Administration, U.S. DOT.

The TRIS database is supplied by the Transportation Research Board under contract to the Research and Special Programs Administration, U.S. DOT.

Go to Chapter 7 (Part I)

Chapter 7 : HIRE

Design Considerations for Highway Encroachment and River Crossings Part I

Go to Chapter 7, Part II

7.1 Intoduction

The objective of this chapter is to present applications of the fundamentals in hydraulics, hydrology, fluvial geomorphology and river mechanics to the hydraulic and environmental design of river crossings and highway encroachments. The principal factors to be considered in design are presented, followed by a discussion of the procedures to approach the evaluation, analysis and design of river crossings and encroachments. The design of most complex problems in river engineering can be facilitated by a qualitative evaluation combined with a quantitative analysis. It is believed that the systematic approach of qualitative assessment of channel response, followed by a quantitative estimate, will enable a meaningful analysis of complex river response problems.

This chapter contains several hypothetical cases of river environments and their response to crossings and encroachments based upon geomorphic principles given in Chapter IV. These cases indicate the trend of change in river morphology for given initial conditions. The hypothetical cases are followed by actual case histories for river crossings in the United States. These histories document river response to highway crossings and encroachments and illustrate qualitatively, river response.

This chapter uses three types of examples (conceptual, practical and overview examples) related to river crossings and highway encroachments. The application of basic principles developed in Chapters 1 through 6 is illustrated by specific numerical examples related to the subject matter of this manual.

7.2 Principal Factors to be Considered in Design

Identification of the principal factors to be considered in design of river crossings and encroachments is useful. These factors are generally interrelated and fundamental mechanisms must be clearly understood prior to design.

7.2.1 Types of Rivers

In selecting the site for a crossing or an encroachment on a river it is necessary to give detailed consideration and study to the type of river or rivers involved. A sandbed river may be meandering, it may be essentially straight, or it may be braided. In addition, a meandering river may be small, medium, or large. The same channel can be classified as youthful, mature, or old. Each of these different river types requires different design procedures. For example, in designing training works for large sandbed channels, braided or meandering, it is unlikely that Kellner jetties alone will be useful to stabilize the bank alignment. It may be necessary to stabilize the banks with rock riprap and to control the overbank flows using jetties to achieve a set of specific purposes. Gravel and cobble bed channels are normally considerably steeper than sandbed channels and in general have narrower river valleys. In the extreme are torrential rivers, the beds of which are comprised of large rocks. These type of rivers usually exist in a youthful or canyon type environment near the upper end of large river systems where the slopes are relatively steep.

7.2.2 Location of the Crossing or the Longitudinal Encroachment

In selecting the site of a crossing or a longitudinal encroachment several considerations are necessary. First of all, the crossing or encroachment must mesh with the transportation system in the area. Secondly, environmental considerations should be considered. In fact, unless appropriate weight is given to the environmental impacts it may not be possible to obtain permission to proceed with the project at all. Economic considerations are equally important. Depending upon the characteristics of the rivers and the environmental considerations, the cost of a particular crossing or encroachment can be significantly affected by its location. The length of the approaches versus the length of the crossing, the cost of real estate that must be acquired to accomplish the crossing, the maintenance cost required to keep the crossing functional over its estimated life and the method of the construction are some of the more specific aspects that should be considered in locating the crossing. The cost of protective measures should also be considered in locating an encroachment.

7.2.3 River Characteristics

The subclassifications of river form can be utilized to identify the range of conditions within which the particular river operates. It is necessary to determine if a river is relatively stable in form or is likely to be unstable. In Figure 1.2.2 of Chapter 1, it was pointed out that rivers can be essentially poised so that a small change in discharge characteristics can change a river from meandering to braided or vice versa. It is important to know the sensitivity of any river system to change. Criteria given in Chapter 4, for example Figure 4.4.3, or Chapter 3, Figure 3.4.4, can be used to predict this sensitivity. A meandering stream whose slope and discharge plot close to the braided river line in Figure 4.4.3 may change to a braided stream with a small increase in discharge or slope.

In addition to river form, it is important to determine other characteristics of the channel: that is, the channel may have a sand bed and cohesive banks; it may be formed in cobbles or it may be formed in other combinations of these materials. Each of these river systems behave differently depending upon the characteristics of the floodplain material, the bank material, and the bed material of the river both over short time and long time. Hence, a rather detailed survey of the characteristics of the bed and bank material coupled with river form plus other pertinent information is essential to design.

7.2.4 River Geometry

For planning a river crossing or an encroachment it is important to know the river geometry and its variation with discharge and time. It is essential to know the slope of the channel and preferably the energy gradient through the reach. In Chapter 4, relations were presented that illustrate how width and depth vary with stage at-a-section as well as along the length of a channel. For most rivers, if the appropriate hydraulic and hydrologic data are available, it is possible to develop simple relations showing how width and depth vary with discharge.

7.2.5 Hydrologic Data

It is necessary to gather all of the hydrologic data pertinent to the behavior of the river and to the design of the river crossing or encroachment. As pointed out in Chapter 6, records of the flood flows are essential. From such information, flow duration curves can be developed, seasonal variations in the river system can be considered and design discharge values can be established depending upon the discharge frequency criteria used in the design. Highway projects constructed with

Federal-aid funds and projects under the direct supervision of the Federal Highway Administration should be designed for a "basic flood". Design standards for the basic flood are specified by the Federal Highway Administration (1974) and the Water Resources Council (1972)

Also it is important to consider the low flows that the river channel will be subjected to and the possible changes in flow conditions that may be imposed on the river system as a consequence of water resources development in the area. Sometimes low flows may lead to a more severe local scour situation at bridge piers and footings. Finally, in terms of hydrologic data it is usually necessary to synthesize some of the required data. Conventional techniques may be used to fill in missing records or it may be essential to synthesize records where few hydrological data exist. In synthesizing data it is very important to compare the particular watershed with other watersheds having similar characteristics. With this information, reasonably good estimates of what can be anticipated at the site can be established.

7.2.6 Hydraulic Data

At the site of a crossing or a longitudinal encroachment it is essential to know the discharge and its variation over time. Coupled with this, it is necessary to know the velocity distribution across the river cross-section and its variation in the river system. This involves determining the type of velocity distribution across the channel as well as in the vertical. Knowledge of the distribution of velocities should be coupled with a study of changes in position of the thalweg to estimate the severity of attack that may occur along the river banks and in the vicinity of the crossing. Furthermore, it is essential to develop stage-discharge relations since these relations fix key elevations of the structure in design and serve as bench-mark data when considering the channel protection measures that may alter the stage of the river. Large changes in velocity can occur in a river system with changing discharge and stage. In a sandbed river, as flow conditions bring about a switch from lower regime to upper regime, the average velocity in the cross section may actually double. From another viewpoint, changes induced in the river system, such as those due to artificial cutoffs or channelization, may sufficiently steepen the gradient so the river operates in upper regime over its whole range of discharge. These possibilities must be considered in the detailed design.

7.2.7 Characteristics of the Watershed Feeding the River System

The water flowing in the river system and the sediment transported therein are usually intimately related to the watershed feeding the river system. Consequently, one needs to study the watershed considering its geology, geometry and land use. In the case of development, land uses include recreation, industrial development, urbanization, flood control, agriculture, and grazing. Similarly, we need to consider the vegetation cover on the watershed and the watershed response to changes in vegetation cover by man or by climatic changes. Significant changes in vegetation cover affect the amount of sediment delivered from the watershed to the river system. It is possible to study the sources of sediment in a watershed. One of the most common techniques is to employ aerial photography and remote sensing techniques coupled with ground investigations. The utilization of remote sensing techniques enables the skilled observer to determine which areas of the watershed are stable and which are unstable. Viewing the total watershed from this viewpoint and using water and sediment routing techniques, it is possible to evaluate the sediment yield as a function of time. Such information can be used to help control the water and sediment yield from the watershed to the river system through engineering analysis of watershed systems.

7.2.8 Flow Alignment

In order to design a safe crossing or longitudinal encroachment, it is necessary to consider the flow alignment in detail. The direction of flow must be considered as a function of time. The position of the thalweg will vary with low, intermediate and high stages. The changing characteristics of the river with stage, such as the change in velocity distribution, the position of the thalweg and the river form can have a significant effect on the intensity of attack on the approaches, the abutments, the piers and embankments. This detailed study of the behavior of the river over time and with varying discharge is necessary for proper design of training works. Only with this type of information can one adequately consider the intensity of attack, the duration of attack and the necessity for training works to make the river system operate within a range of conditions acceptable at the crossing or encroachment. Certainly changes over time at a particular crossing affect the channel geometry, the geometry of the crossing itself, general scour and local scour. If we know the characteristics of the flow and how they vary with time, then one can utilize the information in Chapter 5 to design against excessive contraction and local scour in order to make the highway functional with minimum maintenance over the life of the project.

7.2.9 Flow on the Floodplain

Up to this point we have principally concerned ourselves with flow in the main channel. However, design floods usually flow in both the main channel and on the floodplain. Only by studying the characteristics and geometry of the river and the floodplain can we determine the type of flows that are apt to occur on the floodplain. This particular topic should be studied in adequate detail so that the magnitude and intensity of the flows on the floodplain can be approximated. The characteristics of flow on the floodplain are especially relevant to the design study of longitudinal encroachments. As an example, consider a sinuous channel. At flood stage there is a tendency for the water to flow in the main channel in such a way as to develop chute channels across the point bars. Often, the water spills over the outsides of the bends onto the floodplain. Flow conditions on the floodplain and in the main channel can be greatly different at flood stage than at low flow, which must be taken into consideration. A case in point is a new bridge being constructed across the Mississippi River. In this instance, the flow on the floodplain was sufficiently intense and the alignment of the approaches to the bridge in relation to the flow on the floodplain was such that a large channel was scoured along the upstream side of the approach embankment. Ultimately, a large segment of the approach embankment was lost into this channel and washed downstream as a huge sand wave on the floodplain. In the extreme case, it is entirely possible for cutoffs to form naturally in river systems and only by considering the intensity of flow in the channel and on the floodplain can we determine the possibility of such an occurrence.

7.2.10 Site Selection

Most of the factors cited in the preceding sections have a bearing on the final site selection. In summary, such factors as the form of the river, the alignment of the river, variations of the river form over time, the type of bed and bank material, the hydrologic and hydraulic characteristics of the river, and past, present and future watershed conditions are all important inputs to the site selection. In addition, it is necessary to consider the requirements of the area to be served and the economic and environmental factors that relate to the crossing. Having made a detailed study of possible alternate sites, and having determined the best site considering these important factors, one can then proceed with the determination of the geometry and length of the approaches to the crossing, the type and location of the abutments, the number and location of the piers, the depth to the footing supporting the piers to insure against danger from local scour, the location of the longitudinal encroachment in the floodplain, the amount of allowable longitudinal encroachment into the main channel, and the required river training works to insure that river flows approach the crossing or the encroachment in a complementary way.

7.2.11 Channel Stability Investigations

In conjunction with the background information discussed in the preceding paragraphs it is essential to determine the necessity for bank stabilization. The location, design, and various types of river training works must be considered. The selection of training works is significantly affected by the characteristics of the river and the river system itself. The magnitude of local scour at the training structure must be considered. The possible necessity of holding the river in a selected alignment must also be adequately explored. With regard to these particular issues, one can apply the principles of Chapter 5 to develop suitable designs for stabilizing the approaches, the spur dikes at the end of the approaches, the banks of the main channel and the design of training works that assist in controlling the alignment of the river relative to the crossing or longitudinal encroachment.

7.2.12 Short-Term Response

Having completed the tentative design of the crossing or the encroachment based on river form, channel geometry, hydrologic and hydraulic data it is essential to take a look at the short term response of the river system to the construction. Similarly, the river developments upstream and downstream of the site and at the site itself should also be considered. The techniques that may be utilized to investigate the short-term response at the site or in the vicinity of the crossing or encroachment involves the utilization of qualitative geomorphic relationships followed by the application of more sophisticated analyses using the principles presented in the chapters on open channel flow, sediment transport and river mechanics. In fact, it is possible to establish a mathematical model designed to route both water and sediment through the system. If this model is appropriately designed and utilized, it is possible to evaluate the response of the river system to both the construction of the crossing or encroachment and to other river development projects in the immediate area. For example, it may be important to establish the pattern of clear water releases from a dam upstream of a crossing. Knowing the type of flow the channel would be subjected to and that the water being released is clear, one can make an estimate of the extent of degradation in the channel, the amount of sediment derived from the bed and bank, the instability of the banks and even the types of lateral shifting that may be induced in the river system as it affects the crossing or encroachment.

7.2.13 Long-Term Response

The long-term river response at a crossing or a longitudinal encroachment and in the river system itself should be considered based on all river development projects including the highway. This type of treatment is, in general, beyond the scope of this particular manual. Nevertheless, sufficient advances have been made pertaining to the mathematical modeling of river systems, considering both their short and long-term response. Mathematical modeling can be time consuming and expensive requiring a substantial amount of additional data for calibration of the model. This approach is worth considering on important projects.

7.3 Procedure for Evaluation and Design of River Crossings and Encroachments

This section presents a summary of general procedures to evaluate and design river crossings and encroachments. Due to the multi-disciplined complexity of these problems, it is difficult to develop a procedure which is applicable to all situations that may be encountered. A generalized approach can be described, however modification to this procedure must be made to tailor the procedure to the individual projects.

7.3.1 Approach to River Engineering Projects

The evaluation and design of river crossings should proceed from a broad evaluation of the characteristics of the river and the principles to be considered in design (described in Section 7.2 to detailed computations and discussion. The evaluation should begin with a qualitative assessment of the river. As the analysis progresses, the analysis becomes more and more detailed and subsequently more quantitative. At all stages of the investigation and design it is important to evaluate qualitatively and, if possible, determine the interrelationship of all the aspects of the project.

The three level approach of Simons and Li (1982) and the schematic procedure of Neill (1975) shown in Figure 7.3.1 are two useful methods to approach and evaluate projects in the river environment. Both methods are similar in that they begin broad and narrow down to the finer points of the project. Additionally both approaches provide for back checking or "feedback" loops to insure that the interdependence of all the variables are continually adjusted. The evaluation and analysis of the project is broken into a three level approach.

In level 1, the analysis consists of: 1) identifying the goals of the project; 2) developing several options to achieve those goals; 3) determining the problems and possible solutions to problems associated with each option; and 4) performing a qualitative assessment of all the aspects associated with the project.

The level 2 analysis involves a more detailed qualitative analysis combined with a quantitative evaluation. Computation of water surface profiles can be included in this level of analysis. In many cases the evaluation and analysis can be considered adequate at this level if the goals are met, the interrelationship between different aspects are adequately explained and all of the problems resolved.

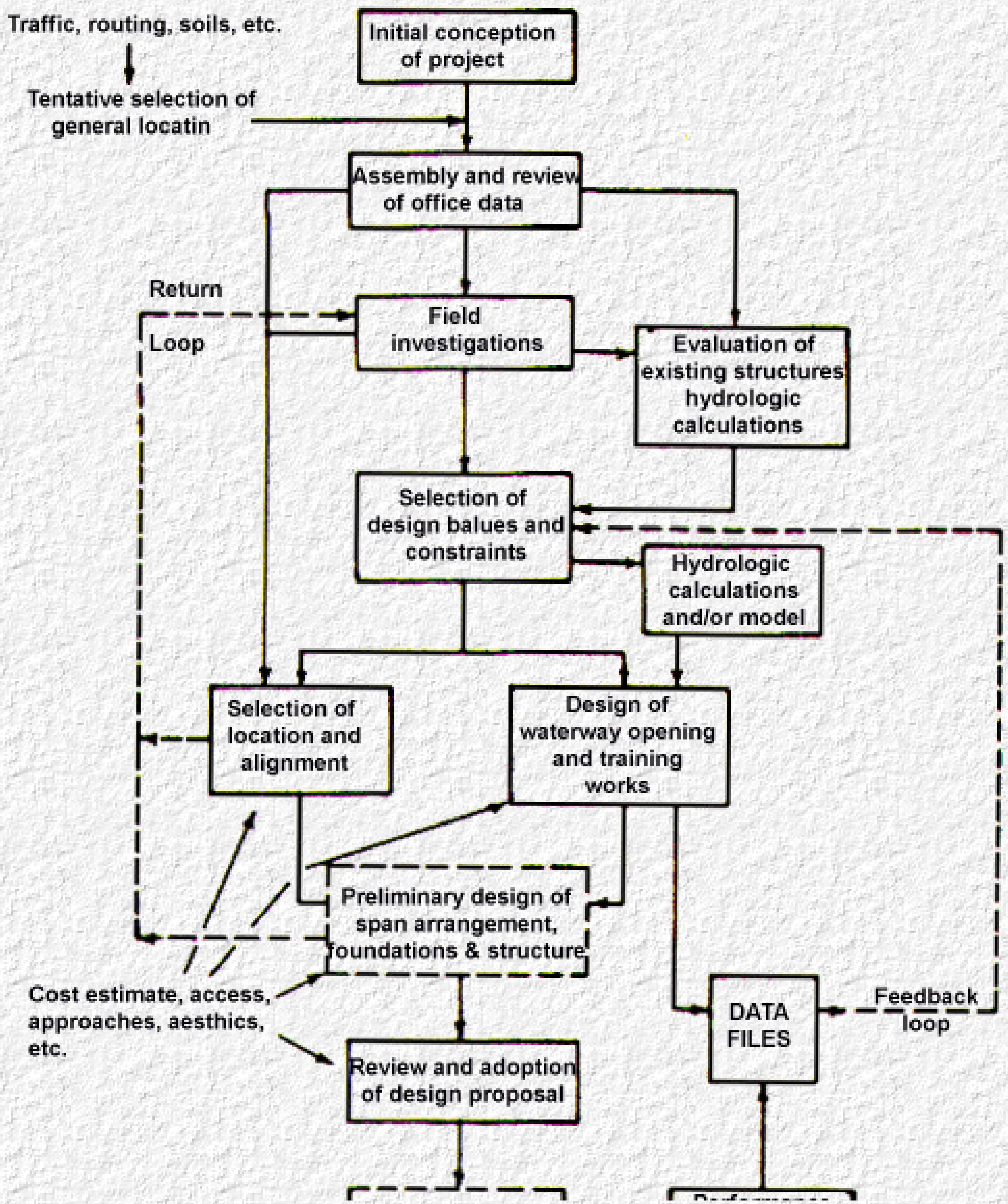


Figure 7.3.1. Flow Chart Indicating a Systematic Procedure for Carrying Out Hydraulic Studies and Showing Their Relation to Other Factors in Bridge Design (after Neill, 1973)

The level 3 analysis involves mathematical modeling of water and sediment. These procedures are not always necessary. Water can be modeled using a steady or unsteady rigid boundary flow model. In some cases a movable boundary flow model can be employed, routing both water and sediment through the study reach. Sediment routing models will require a substantial historic data set and analysis time to calibrate and verify the model. Experienced modelers should be employed if sediment routing is to be performed.

The schematic approach presented by Neill (1975) illustrates the steps to follow from project conception to final design, construction and monitoring. This approach is similar to the three level approach. The schematic can be used as a guide, however it will probably require modification to meet the needs and goals of specific projects.

7.3.2 Initialization of the Project

The success of a project can be dramatically influenced by careful planning in the initial stages of the project. The following are guidelines which will help insure the success of the project.

- **Conception of Project** - When the project is conceived the goals of the project should be carefully designed and several options to meet these goals identified. The factors discussed in Section 7.2. should be considered when identifying design options. These options will be refined as the project progresses, eventually focusing on one or two options.
- **Assemble Available Data** - All available data should be compiled and checked. The data checklist presented in Chapter VI should be used as a guide. Data which is unavailable or periods of missing data should also be listed on the checklist. Missing data can be ranked according to need (i.e.: essential, nonessential and optional). Field programs designed to collect the essential data could be implemented at this time, however it is recommended that a field reconnaissance and evaluation be completed prior to implementing field programs. The field reconnaissance will provide a clearer definition of the project and will influence the types and quantity of additional data requirements.
- **Field Reconnaissance** - An initial field reconnaissance should be performed

by a small group of technical personnel. It is advisable that the group be multi-disciplined so that geologic, geomorphic, hydrologic, hydraulic, alignment and highway constraints can be identified. They should define the problems for each option and identify possible solutions to each problem. Options which are least feasible should be eliminated.

The field reconnaissance team should identify the most favorable options, recommend the types of analysis which will be needed, and design field programs to collect specific data which will be required by the analysis.

- **Collection of Additional Field Data** - Field programs should be designed to collect only data that will be required to analyze and design the project. The field reconnaissance discussed previously is an important tool to the design and implementation of efficient field programs. By designing and implementing field programs after the field reconnaissance, the collection of unnecessary data can be avoided, providing more time and funding for collection of essential data.

It is also advisable that the field crews be supervised in the field by personnel will be directly involved in the analysis and design. These personnel should be completely familiar with the types of data and the methods used to collect the data, providing an interface between the field and the analysis in the office. In this way the field work, analysis and design can be closely coordinated.

7.4 Conceptual Examples

This section discusses conceptual cases of river response to highway encroachments. Several hypothetical cases of river response to highway encroachments are tabulated in Table 7.4.1. Each individual case is identified in the first column to show the physical situation that exists prior to the construction of the highway crossing. In the following three columns some of the major local effects, both upstream and downstream, resulting from construction of a particular crossing are given. It is necessary to emphasize that only the gross local upstream and downstream effects are identified in this table. In an actual design situation, it is worthwhile first of all to consider the gross effects as listed in Table 7.4.1.

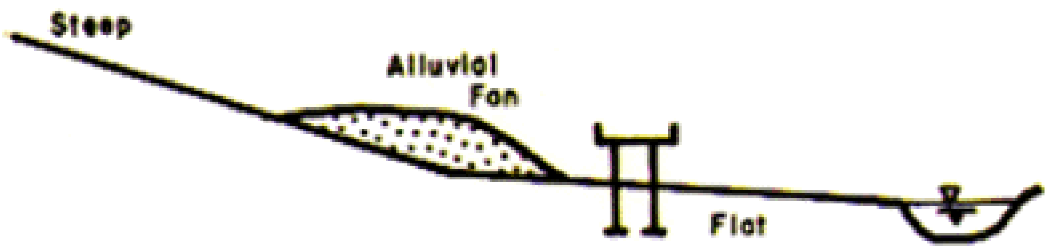
The relation QS ~ Q_s D_{50} is one valuable method for determining qualitative river response. Having identified the qualitative response that can be anticipated, river mechanics techniques given in Chapter 4 and Chapter 5 can be used to predict the possibility of change in river form and to estimate the magnitude of local upstream and downstream river response. In some cases water and sediment routing can be coupled with the preceeding relations to predict possible river response.

The initial river conditions in Table 7.4.1 include storage dams or water diversions. These examples are used as illustrations relating to common experience. In general, the effect of a storage reservoir is to cause a sudden increase of base level for the upstream section of the river. The result is aggradation of the channel upstream, degradation downstream and a modification of the flow hydrograph. Similar changes in the channel result if the base level is raised by some other mechanism, say a tectonic uplift. The effect of diversions is to decrease the river discharge downstream of the diversion with or without an overall reduction of the sediment concentration. Similarly, changes in water and sediment input to a river often occur due to river development projects upstream from the proposed crossings or due to natural causes.

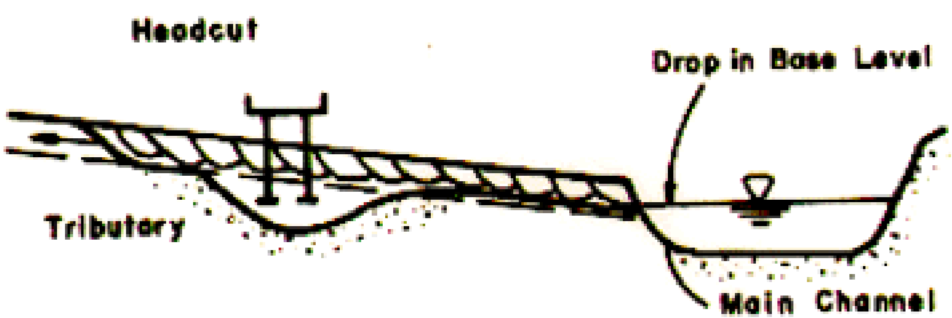
Case (1) of Figure 7.4.1 involves the construction of a bridge across a tributary stream downstream of where the steeper tributary stream has reached the floodplain of the parent stream. In most cases, the change in gradient of the tributary stream causes significant deposition. In the case illustrated for Case (1), an alluvial fan develops which in time can divert the river around the bridge, or, if the water continues to flow under the bridge, the waterway is significantly reduced, endangering the usefulness and stability of the structure. Usually, streams on alluvial fans shift laterally so that the future direction of the approach flow to the bridge is uncertain.

Table 7.4.1 River Response to Highway Encroachments and to River Development

Bridge Location



(1) Crossing downstream of an alluvial fan



(2) Lowering of base level for the channel

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
1	1-Fan reduces waterway 2-Direction of flow at bridge site is uncertain 3-channel location is uncertain	1-Erosion of banks 2-Unstable channel 3-Large transport rate	1-Aggradation 2-Flooding 3-Development of tributary bar in the main channel
2	1-Headcutting 2-General scour 3-Local scour 4-Bank instability 5-High velocities	1-Encreased velocity 2-Increased bed material transport 3-Unstable channel 4-Possible change of form of river	1-Increased transport to main channel 2-Aggradation 3-Increased flood stage

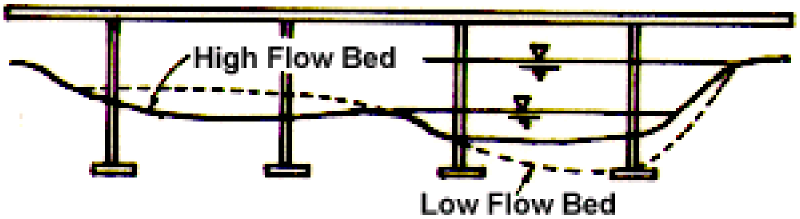
Case (2) sketched in Table 7.4.1 illustrates a situation where a bridge is constructed across a tributary stream. The average water surface elevation in the main channel acts as the base level for the tributary. It is assumed that at some point in time after the construction of the bridge the base level has been lowered. Under the new condition, the local gradient of the tributary stream is significantly increased. This increased energy gradient induces head cutting and causes a significant increase in water velocities in the tributary stream. The result is bank instability, possible major changes in the geomorphic characteristics of the tributary stream and increased local scour. When the base level is raised the gradient in the tributary is decreased, resulting in deposition, lateral channel instability, increased flood levels, and a decrease in flow area under the bridge. This case is similar to Case (1).

Case (3) illustrates a situation where a bridge supported by piers and footings is constructed across a channel that is subjected to long periods of low stage. When a river is subject to long periods of low flows, there is a tendency for the low flow to develop a new low water thalweg in the main channel. If the low water channel aligns itself with a given set of piers, it is possible that the depth of local scour resulting from this flow condition may be greater than the depth of local scour at high stage. There are several documented failures where bridges have been safe in terms of local scour at high stage, but have failed as a consequence of the development of greater local scour during low flow periods.

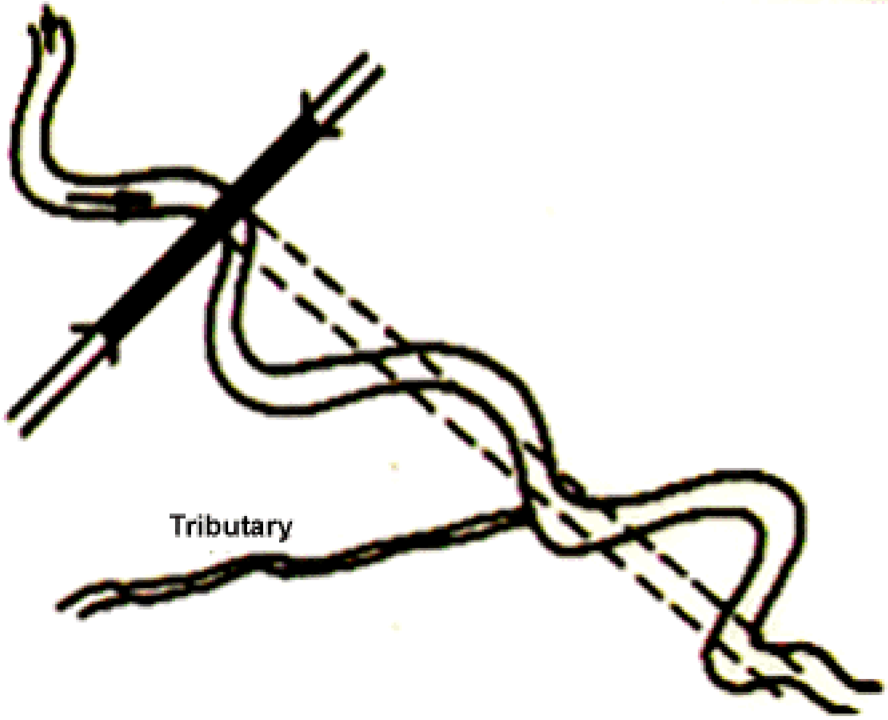
Case (4) illustrates a situation where artificial cutoffs have straightened the channel downstream of a particular crossing. Straightening the channel downstream of the crossing significantly increases the channel slope. This causes higher velocities, increased bed material transport, degradation and possible head cutting in the vicinity of the structure. This can result in unstable river banks and a braided streamform. The straightening of the main channel can drop the base level, adversely affecting tributary streams flowing into the straightened reach of the main channel, which was discussed in Case (2).

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location



(3) Channel characterized by prolonged low flows



(4) Cutoffs downstream of crossing

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
3	1-At low flow a low water channel develops in river bed 2-Increased danger to piers due to channelization and local scour 3-Bank caving	- - -	- - -

4	1-Steeper slope 2-Higher velocity 3-Increased transport 4-Degradation and possible headcutter 5-Banks unstable 6-River may braid 7-Danger to bridge foundation from degradation and local scour	See local effects	1-Depostion downstream of straightened channel 2-Increase in flood stage 3-Loss of channel capacity 4-Degradation in tributary

Case (5) illustrates the situation where a bridge is constructed across a river immediately downstream of the confluence with a steep tributary. The tributary introduces relatively large quantities of bed materials into the main channel. As a result, an island has formed in the main channel and divided flow exists. In order to reduce the cost of the bridge structure, the bridge is built across one subchannel to the island or bar formed by deposition, closing the secondary channel. Such a procedure forces all of the water and sediment to pass through a reduced width. <u>This contraction of the river in general increases the local velocity, increases general and local scour, and may increase bank instability.</u> In addition, the contraction can change the alignment of the flow in the vicinity of the bridge and affect the downstream channel for a considerable distance. A chute channel can develop across the second point bar downstream, adversely affecting several meander loops downstream. Upstream of the bridge, there is aggradation. The amount depends on the magnitude of water and sediment being introduced from the tributary. Also, there is increase in the backwater upstream of the bridge at high flows which in turn affects other tributaries farther upstream of the crossing (see Case (2)).

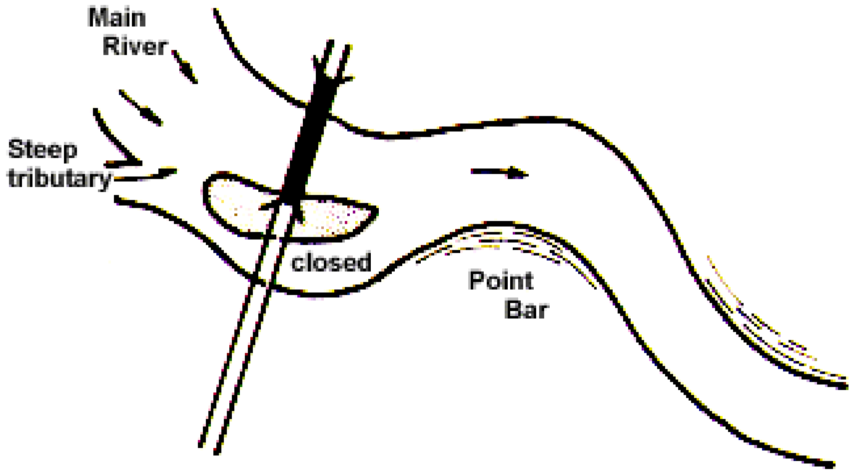
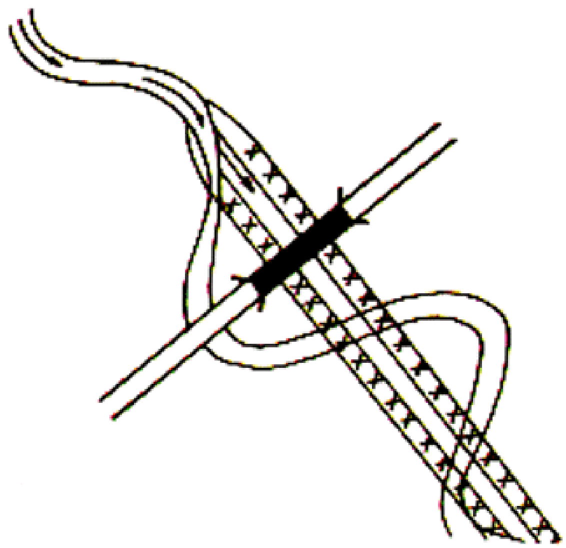
Case (6) illustrates a situation where the main channel is realigned in the vicinity of the bridge crossing. A cutoff is made to straighten the main channel through the selected bridge site. As discussed in Case (4), increased local gradient, local velocities, local bed material transport, and possible changes in the characteristics of the channel are expected due to the new conditions. As a result the channel may braid. On the other hand, if the straightened section is designed to transport the same sediment loads that the river is capable of carrying upstream and downstream of the straightened reach, bank stability is ensured. A carefully designed channel should not undergo significant change over either short or long periods of time.

<u>It is possible to build modified reaches of main channels that do not introduce major adverse responses due to local steepening of the main channel.</u> In order to design a straightened channel so that it behaves essentially as the natural channel in terms of velocities and magnitude of bed material transport, it is usually necessary to build a wider, shallower section.

Case (7) illustrates a bridge constructed across a main channel. Subsequently, the base level for the channel is raised by the construction of a dam. Whenever the base level of a channel is raised, a pool is created extending a considerable distance upstream depending on the size of dam and slope of the channel. As the water and sediment being transported by the river encounters this pool, most of the sediments drop out, forming a delta-like structure at the mouth. <u>If the bridge lies within the effects imposed by the new base level, the following effect at</u>

the crossing will be expected: a loss of waterway at the bridge site, significant changes in river geometry, and increased flood stages and lateral channel instability. This is similar to Case (1).

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location
 (5) Excess of sediment at bridge site due to upstream tributary  (6) River channel relocation at crossing site

Bridge Location	Local Effects	Upstream Effects	Downstream Effects

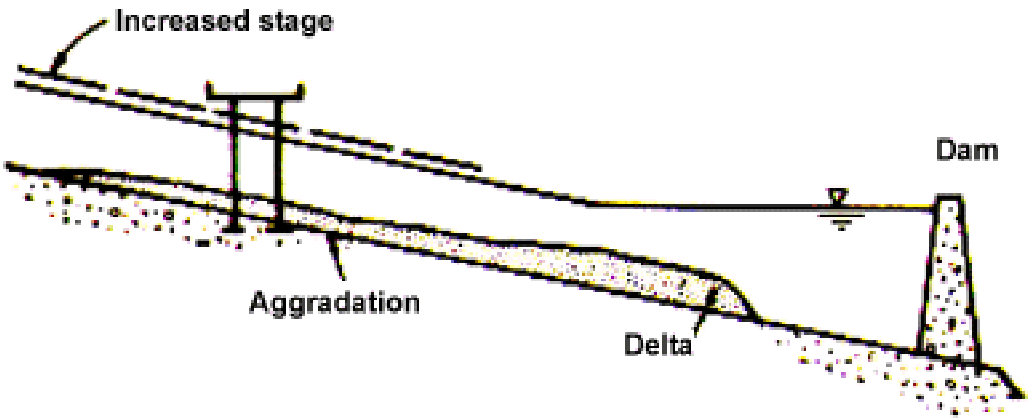
5	1-Contraction of the river 2-Increased velocity 3-General and local scour 4-Bank Instability	1-Aggradation 2-Backwater at flood stage 3-Changed response of the tributary	1-Deposition of excess sediment eroded at and downstream of the bridge 2-More severe attack at first bend downstream 3-Possible development of a chute channel across the second point bar downstream of the bridge
6	1-None if straight section is designed to transport the sediment load of the river and if it is designed to be stable when subjected to anticipated flow. Otherwise same as in case (4).	Similar to local effects	Similar to local effects

Case (8) considers the situation where the sediment load is reduced in the channel after a bridge has been constructed. This may happen due to the construction of a storage dam upstream of the crossing. As stated in the preceding case, the raising of the base level of a river, as in the development of storage by constructing a dam on a river, provides a sedimentation basin for the water flowing in the system. In most instances all of the sediment coming into a reservoir drops out within the reservoir. Water released from the reservoir is mostly clear. With sediment-free flow, the channel is too steep and bed sediments are entrained from the bed and the banks causing significant degradation. If the bridge is sufficiently close to the reservoir to be affected by the degradation in the channel, the depth due to general and local scour at the bridge may be significantly increased. Also, the channel banks may become unstable due to degradation and there is a possibility that the river, as its profile flattens, may change its form. In the extreme case, it is possible that the degradation may cause failure of the dam and the release of a flood wave.

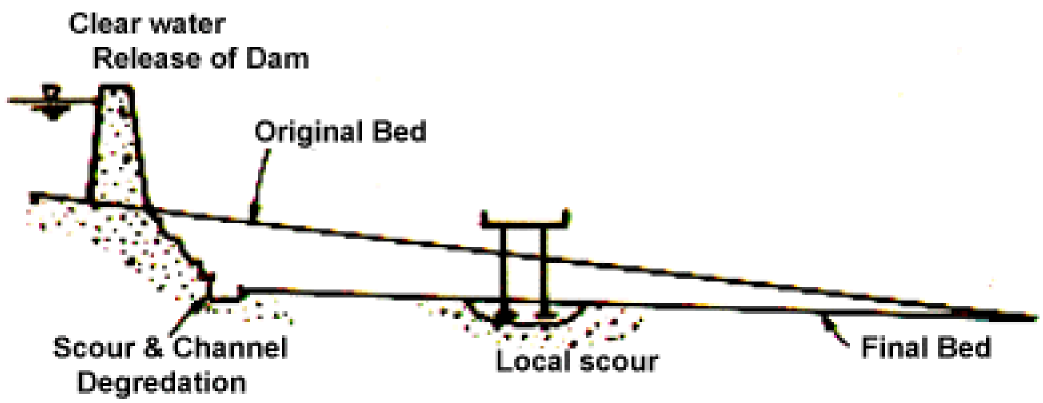
Case (9) through Case (13) illustrate a more complicated set of circumstances. These cases involve the interrelationship of Cases (1) through (8). In Case (9) the river crossing is affected by Dam A constructed upstream as well as Dam B constructed downstream. As documented in the preceding case, Dam A causes significant degradation in the main channel. Dam B causes aggradation in the main channel. The final condition at the bridge site is estimated by summing the effects of both dams on the main channel and the tributary flows. Normally, this analysis requires water and sediment routing techniques studying both long- and short-term effects of the construction of these dams and it is necessary to consider the extreme possibilities to develop a safe design.

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location



(7) Raising of river base level

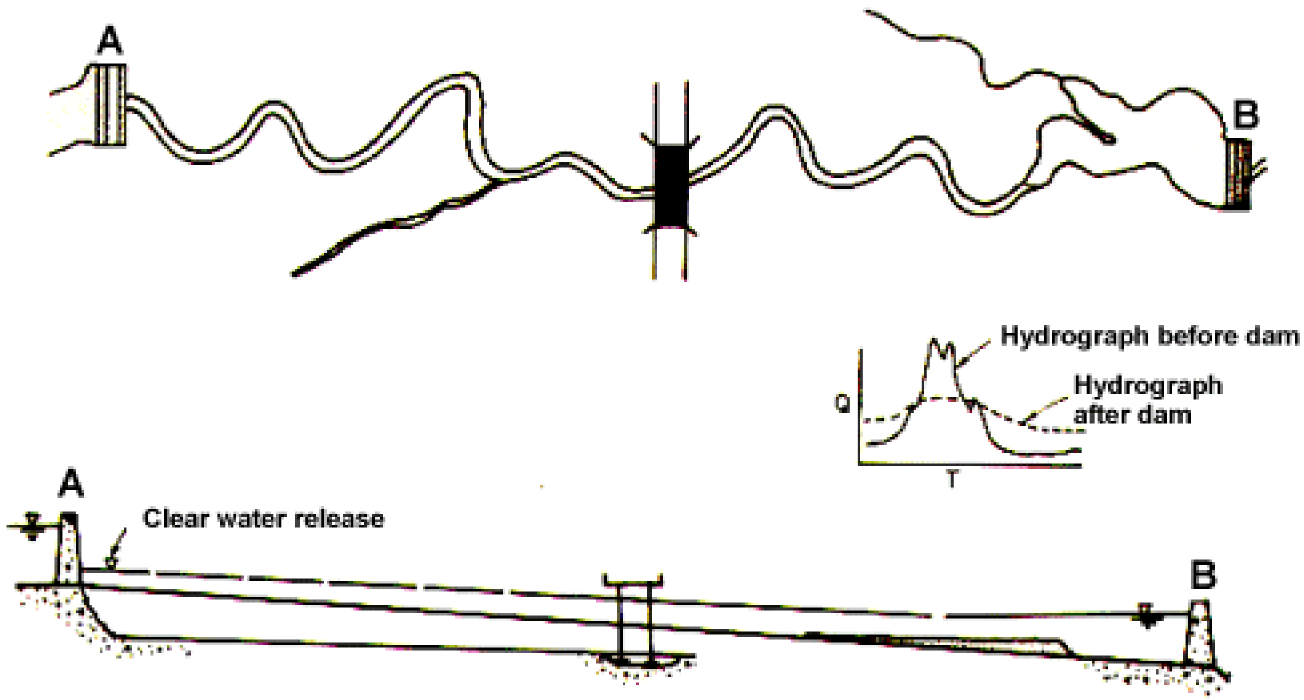


(8) Reduction of sediment load upstream

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
7	1-Aggradation of bed 2-Loss of waterway 3-Change in river geometry 4-Increased flood stage	1-See local effects 2-change in base level for tributaries 3-Deposition in tributaries near confluences 4-Aggradation causing a perched river channel to develop or changing the alignment of the main channel	1-See upstream effects

8	1-Channel degradation 2-Possible change in river form 3-Local scour 4-Possible bank instability 5-Possible destruction of structure due to dam failure	1-Degradation 2-Reduced flood stage 3-Reduced base level for tributaries, increased velocity and reduced channel stability causing increased sediment transport to main channel	1-Degradation 2-Increased velocity and transport in tributaries

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location
 (9) Combined increase of base level and reduction of sediment load upstream

Bridge Location	Local Effects	Upstream Effects	Downstream Effects

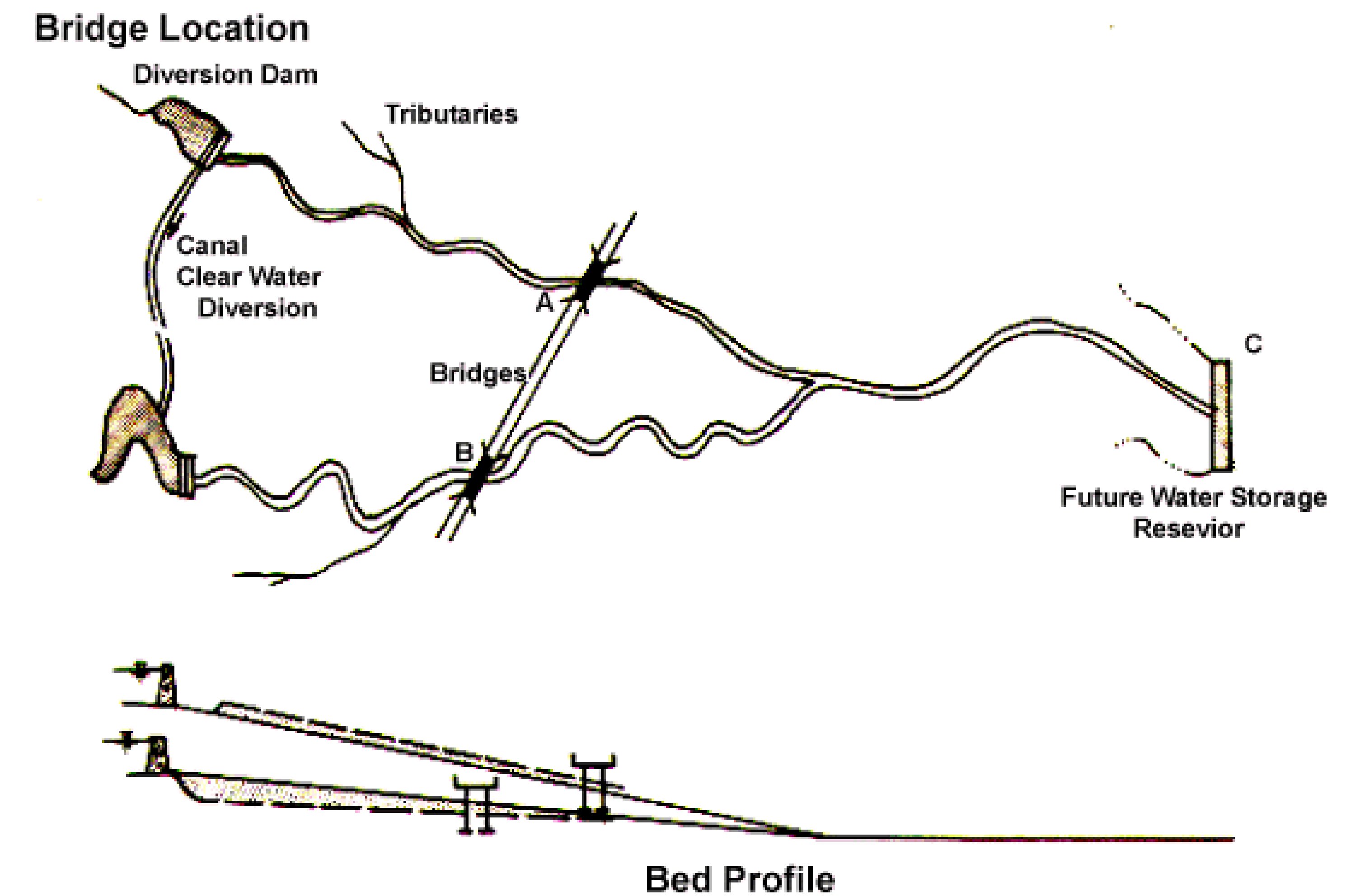
9	1-Dam a causes degradation 2-Dam B causes aggradation 3-Final condition at bridge site is the combined effect of (1) and (2). Situation is complex and combined interaction of dams, main chanel and tributaries must be analyzed using water and sediment routing techniques and geomorphic factors	1-Channel could aggrade or degrade with effects similar to cases (7) and (8).	1-See upstream effects

In Case (10) Bridges A and B cross two major tributaries a considerable distance upstream of their confluence. Upstream of Bridge A, a diversion structure is built to divert essentially clear water by canal to the adjacent tributary on which Bridge B has been constructed. Upstream of Bridge B the clear water diverted from the other channel enters the storage reservoir and the water from the tributary plus the transfer water is released through a hydro-power plant. It is anticipated that a larger storage reservoir may be constructed downstream of the confluence on the main stem at C. These changes in normal river flows give rise to several complex responses at bridge sites A and B, in the tributary systems as well as on the main stem. Bridge Site A may aggrade due to the excess of sediment left in that tributary when clear water is diverted. However, initially there may be lowering of the channel bed in the vicinity of the diversion structure because of the deposition upstream of the diversion dam and the release of essentially clear water for a relatively short period of time until the sediment storage capacity of the reservoir is satisfied. Bridge site B is subjected to degradation due to the increased discharge and an essentially clear water release. However, the degradation of the channel could induce degradation in the tributaries causing them to provide additional sediment to the main channel, see Case (8). This response would to some degree counteract the degrading situation in this reach of river. Such changes in river systems are not uncommon and introduce complex responses throughout the system. Complete analysis must consider the individual effects and sum them over time to determine a safe design for the crossings.

Case (11) shows a highway that crosses the main channel at Bridge A and its tributary at Bridge B. The confluence of the main channel and its tributary is downstream of both bridges. The alignment of the main channel is continually changing. The rate of change in the river system should be evaluated as part of the geomorphic and hydraulic analysis of the site. If the main channel shifts to the alternate position shown and moves the confluence closer to Bridge B, the gradient in the tributary is significantly increased causing degradation as well as channel instabilities and possible changes in river form. Excess sediment from the tributary causes aggradation in the main channel and possibly significant changes in channel alignment. Considering the possible changes in the position of the main channel, training works may be required at and upstream of Bridge A to assure a satisfactory approach of the flow to the bridge crossing. Otherwise, the river could abandon its present channel. A shift in the position of the main channel relative to the position of the confluence with the tributary also alternately flattens or steepens the gradient of the tributary causing corresponding aggradation or degradation in the tributary. This type of problem is difficult because of the continuously changing characteristics of such river systems. Rivers of this type are usually stable for several years at a time or at least between major flows. Consequently, if crossing locations are properly selected and appropriate stabilization techniques are taken, it may be possible to maintain the usefulness of the crossings for the life of the structures. However, the disadvantages associated with such locations will often require expensive solutions and these locations should be avoided if possible.

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location

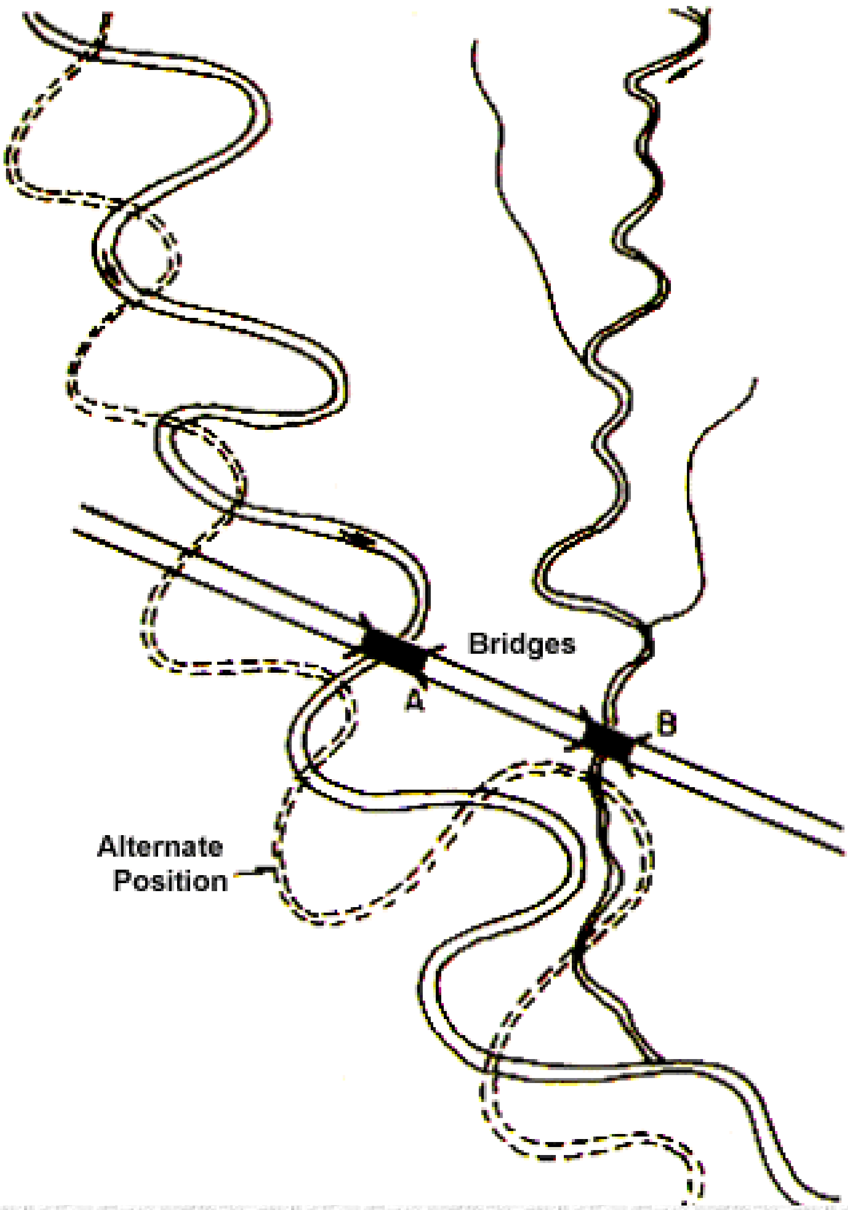


(10) Change in water discharge, no change in sediment load

Bridge Location	Local Effects	Upstream Effects	Downstream Effects

10	1-Bridge A may be subjected to aggradation due to excess sediment left in the channel by diversion of clear water. 2-Bridge B may be subjected to degradation due to increased discharge in the channel 3-If a storage reservoir was constructed at C it would induce aggradation in both main tributaries	1-Upstream of Bridge A - aggradation and possible change of river form 2-Upstream of Bridge B - degradation and change of river form 3-Channel instabilities 4-Significant effects on flood stage	1-See upstream effects 2-Construction of reservoir C could induce aggradation in the main channel and in the tributaries. Effects same as in case (7).

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location


(11) Naturally shifting river channel

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
11	1-Rivers are dynamic (ever changing) and the rate of change with time should be evaluated as part of the geomorphic and hydraulic analysis 2-Alignment of main channel continually changes affecting alignment of flow with respect to Bridge A 3-If the main channel shifts to the alternate position, the confluence shifts and the tributary gradient is significantly increased causing degradation in the tributary. Local effects on Bridge B same as 1, 2, 3, and 4 in Case (8) 4-Excess sediment from the tributary, assuming (3) causes aggradation in the main channel and possible significant changes in channel alignment	1-The river could bandon its present channel. Changing position of the main channel may require realignment of training works	1-See upstream effects 2-Shifts in the position of the main channel relative to the position of the confluence with the tributary alternatively flattens or steepens the gradient of the tributary causing corresponding aggradation 3-Shifts in the position of the main channel causes aggradation, degradation and instabilities depending upon direction magnitude of channel change

Case (12) illustrates a meandering channel with several tributaries and a major storage reservoir constructed on the main channel. Two crossings are shown on the main channel upstream of the reservoir. It is assumed that complete channelizing of the meandering river has been authorized. This shortens the path of travel of the water by an appreciable distance. If we consider local effects at the bridges, bridge site A is first subjected to possibly severe degradation and then aggradation. Bridge site B is primarily subjected to degradation. The magnitude of this degradation can be large. With the degree of straightening indicated in the sketch, severe head cutting may be initiated up the main channel as well as the tributaries. The whole system may be subjected to passage of sediment waves and the river form can dramatically change over time. The flood level in the system and the local and general scour in the vicinity of the bridges is greatly affected by the channelization.

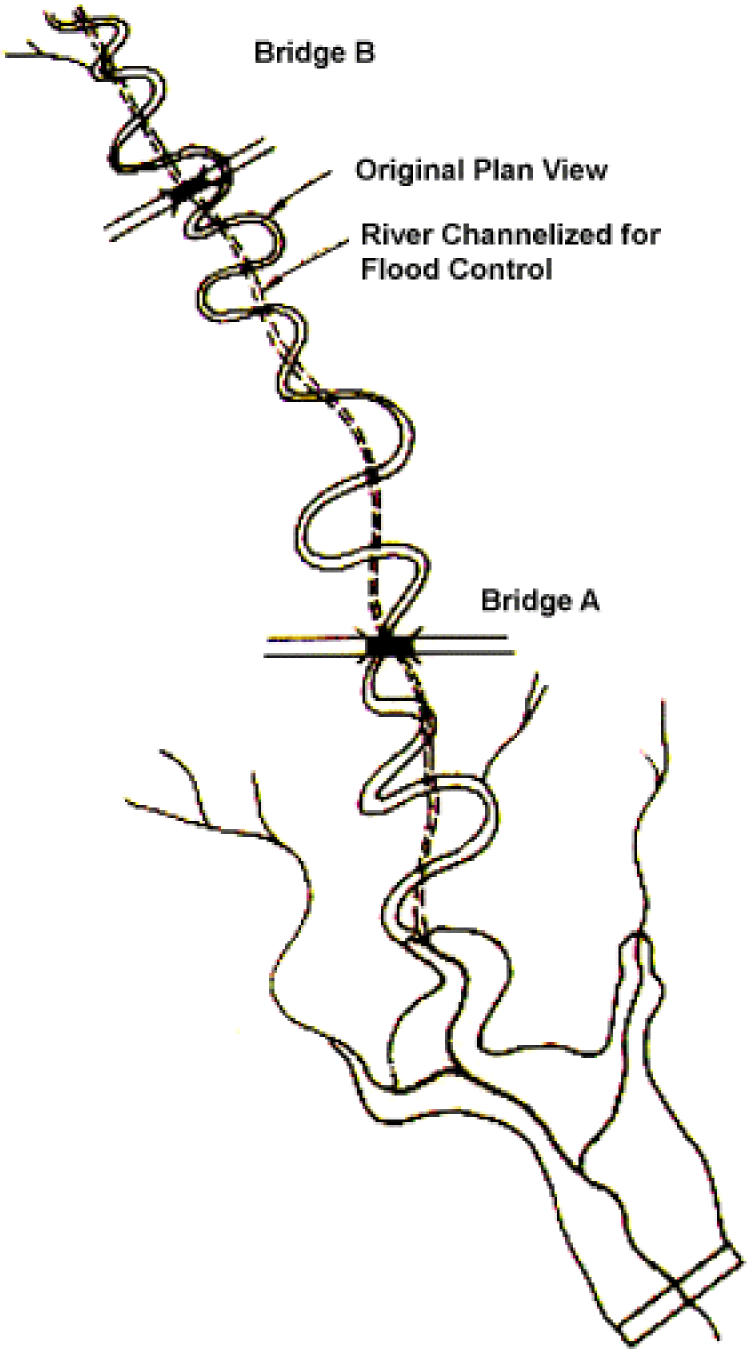
As a result of channelization, the river reach at bridge site B braids. Also, in this reach the rate of sediment transport is increased, head cutting is induced and flood stages are reduced. The tributaries in the upper reach are subjected to severe degradation. For the bridge at position A, the channel would probably degrade and then significantly aggrade. Significant reactions are possible when channelization is undertaken in a river system. A detailed analysis of all of the responses is necessary before it is possible to safely design crossings such as those at location A and B.

Case (13) is a series of situations, unrelated in some instances and combined in others, which can affect bridge crossings. Tidal flows, seiches and bores can have significant effects on scour and depth in the channel system. The tidal flows, seiches and bores, as well as wind waves, can rapidly and violently destroy existing bank lines. When considering earthquakes, it is of importance to examine a seismic probability map of the United States. Large portions of the United States are subjected to at least infrequent earthquakes.

Associated with earthquake activity are severe landslides, mud flows, uplifts in the terrain, and liquefaction of otherwise semi-stable materials, all of which can have a profound effect upon channels and structures located within the earthquake area. Historically, several rivers have completely changed their course as a consequence of earthquakes. For example, the Brahmaputra River in Bangladesh and India shifted its course laterally a distance of some 200 miles as a result of earthquakes that occurred approximately 200 years ago. Although it may not be possible to design for this type of natural disaster, knowledge of the probability of its occurrence is important so that certain aspects of the induced effects from earthquakes can be taken into consideration when designing the crossings and affiliated structures.

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location

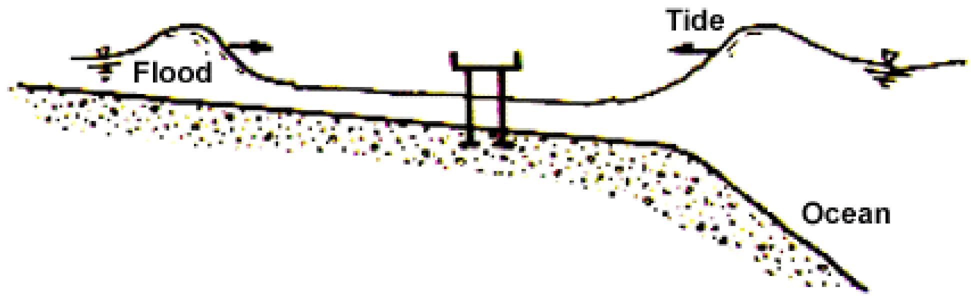


(12) Man-induced reduction
of channel length

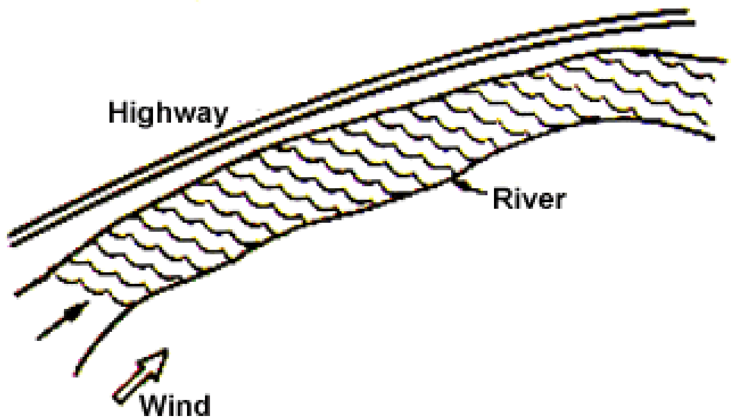
12	1-Bridge A is first subjected to degradation and then aggradation. Action can be very severe 2-Bridge B is primarily subjected to degradation. The magnitude can be large 3-The whole system is subjected to passage of sediment waves 4-river form could change to braided 5-flood level are reduced at B and inreased at A 6-Local and general scour is significantly affected	1-A change of river form from meandering to braided is possible 2-Rate of sediment transport is increased 3-head cutting is induced in the whole system upstream of B 4-Flood stage is reduced 5-Velocity increases 6-Tributaries respond to main channel changes	1-For Bridge B see upstream effects 2-For Bridge A the channel first degrades and then significantly aggrades 3-Large quantities of bed material and wash load are carried to the reservoir 4-Delta forms in the reservoir 5-Wash load may affect water quality in the entire reservoir 6-Tributaries respond to main channel changes

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

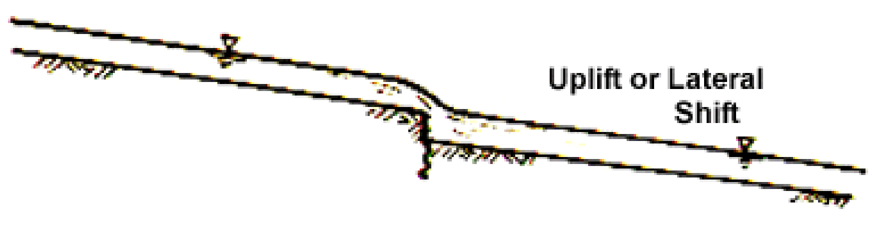
Bridge Location



a. - Tidal Flows, Seiches, Bores, etc.



b. - Wind (Hurricanes, Tornadoes)



c. - Earthquakes (See Seismic Probability Map of U.S.)

(13) Tectonics and other natural causes

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
a	1-Scour or aggradation 2-Bank erosion 3-Channel change 4-Bed form change	1-See local effects 2-Channel erosion 3-Changes in channel slope	1-See local effects 2-Beach erosion

b	1-Bank erosion 2-Inundated highway 3-Increase in Velocity	1-See local effects	1-See local effects
c	1-Channel changes 2-Scour or deposition 3-Decrease in bank stability 4-Landslides 5-Rockslides 6-Mudflows	1-See local effects 2-Slide lakes	1-See local effects 2-Slide lakes

Case (14) and (15) illustrate three examples of longitudinal encroachment. In Case (14) , a few bends of a meandering stream have been realigned to accommodate a highway. There are two problems involved in channel realignment. One, the length of realigned channel is generally smaller than the original channel and consequently results in a steeper energy gradient in the reach (Case 4). Two, the new channel bank material in the realigned reaches may have a smaller resistance to erosion. As a result of these two problems, the channel may suffer instability by the formation of a headcut from the downstream end and increased bank erosion. The realigned channel may also exhibit a tendency to regain the lost sinuosity and may approach and scour the highway embankment. To counter these local effects one could design the realignment to maintain the original channel characteristics (length, sinuosity). Another way would be to control the slope by a series of low check dams. In any case, bank protection by riprap, jacks or spurs will be needed. The upstream and downstream effects of the channel realignment will be the same as discussed for channel length reduction in Case (12). For example, as the degradation travels through the realigned reach, sediment load generation in the river by bed and bank erosion will cause aggradation downstream.

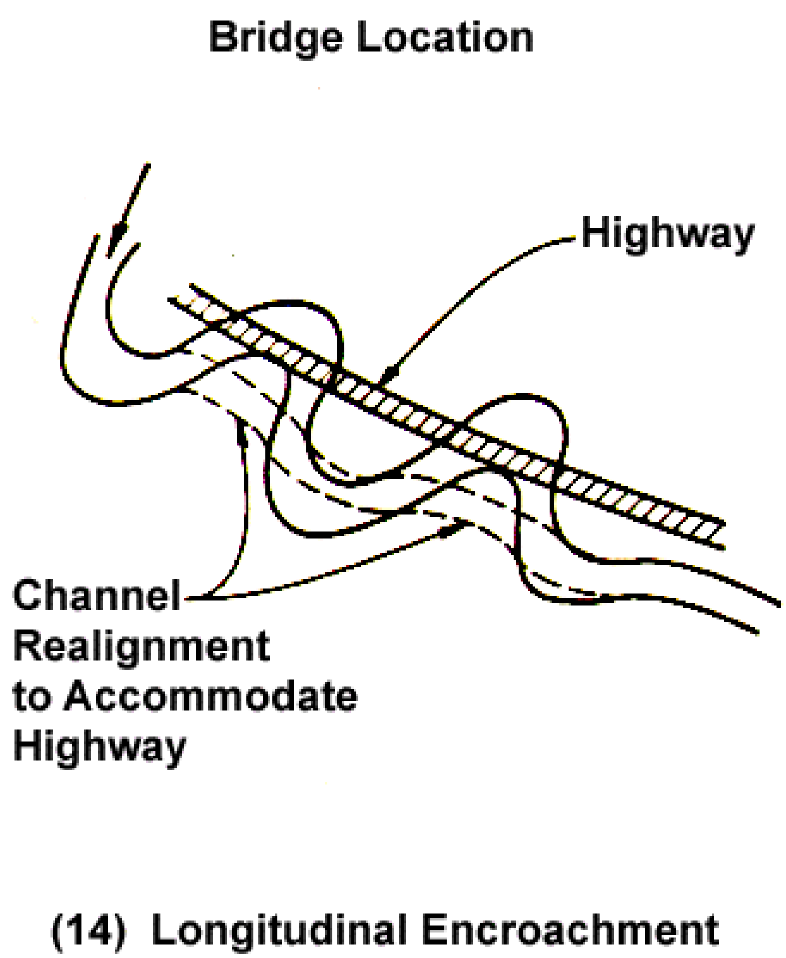
Case (15) illustrates encroachment on the waterway of an incised stream flowing through a narrow gorge. Locally, the effect is to reduce the waterway and to increase the velocities and bank and bed erosion potential. The erosion protection of the highway slope exposed to the flow and possibly on the opposite bank are important problems: The backwater induced by this obstruction may cause upstream aggradation and higher flood levels. On the downstream side, channel aggradation may be experienced if bed erosion occurs locally in the encroached reach.

Case (16) is a case of floodplain encroachment. It is assumed that during bankful and lower stages the highway does not interact with the flow. However, during high stages, the flow area is decreased by the encroachment. Locally, the highway is to be protected against inundation and erosion during flood. The effect on the river channel depends on the extent of encroachment on the waterway. If the highway significantly reduces the floodplain, it may increase river stages for a given flood. If the river channel is a shifting one, the highway encroachment may alter the interaction of floodplain flow and channel flow, affecting the channel floodplain flow pattern. Very often this type of encroachment has little or no effect on flood stages or on the stream upstream or downstream, however, possible adverse effects should be investigated.

In all cases of longitudinal encroachment, the lateral drainage into the river will be intercepted.

A main consideration in the design of encroachments will be to provide for this drainage.

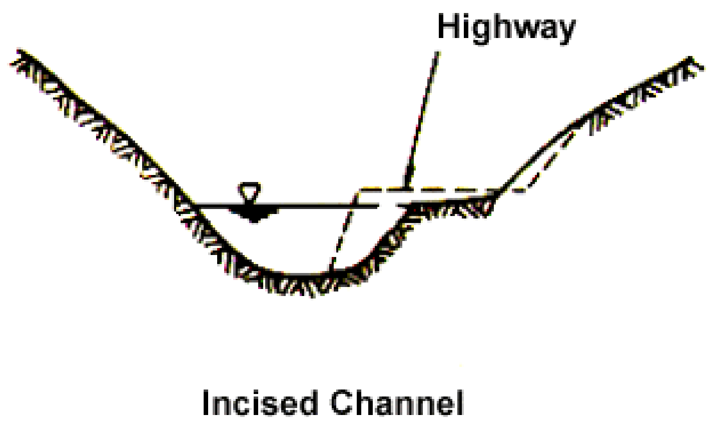
Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)



Bridge Location	Local Effects	Upstream Effects	Downstream Effects
14	1-Increased energy gradient and potential bank and bed scour 2-Highway fill is subject to scour as channel tends to shift to old alignment 3-Reach is subject to bed degradation as headcut develops at the downstream end and travels upstream 4-Lateral drainage into the river is interrupted and may cause flooding and reosion	1-Energy gradient also increased in the reach upstream and may cause change of river form from meandering to braided 2-Rate of sediment transport is increased. As the headcut travels upstream severe bank and bed erosion is possible 3-If tributaries in the zone of influence exist they will respond to lowering of base level	1-Channel will aggrade as the sediment load coming from bed and bank erosion is received 2-channel may deteriorate from meandering to braided.

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location

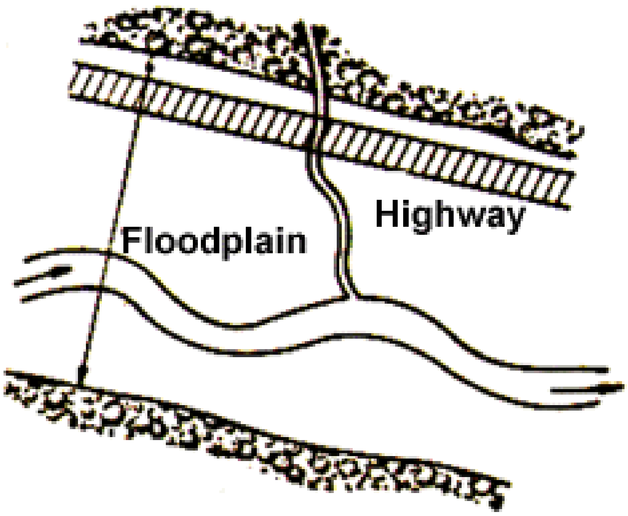


(15) Longitudinal Encroachment

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
15	1-Reduced waterway causes a local obstruction to flow and higher velocities 2-Significat erosion problem on the highway fill and induced bed degradation 3-Lateral drainage into the river is interrupted and may cause flooding and erosion	1-Backwater generated by the obstruction increases flood stage 2-Deposition induced by the backwater	1-Large sediment load may cause aggradation 2-Local scour at end of contracted section

Table 7.4.1 River Response to Highway Encroachments and to River Development (Continued)

Bridge Location



Floodplain Encroachment

(16) Longitudinal Encroachment

Bridge Location	Local Effects	Upstream Effects	Down Stream Effects
16	1-Erosion of highway fill and submergence possible durin floods 2-Pattern of overbank spell are affected by the encroachment and in highly shifting channels may change river course downstream 3-Lateral drainage into the river is interrupted and may cause flooding and erosion	1-If sgnificat encroachment on the floodplain waterway, backwater may be induced	1-If the river channel is highly shifting the channel alignment may change 2-If significant erosion experienced uptream, aggradation will occur

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Go to Chapter 7 (Part II)

Chapter 7 : HIRE

Design Considerations for Highway Encroachment and River Crossings Part II

Go to Chapter 7, Part III

7.5 Practical Examples of River Encroachments

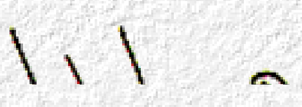
In the preceding paragraphs several possible conceptual cases were discussed. Each case considers the interactions between the river and the encroachment over a period of time. In general, the particular practical cases presented in this section are not as complex as some of the earlier conceptual cases. For example, there is no consideration of water resources development throughout the basins, including construction of reservoirs, transmountain diversions, and so forth.

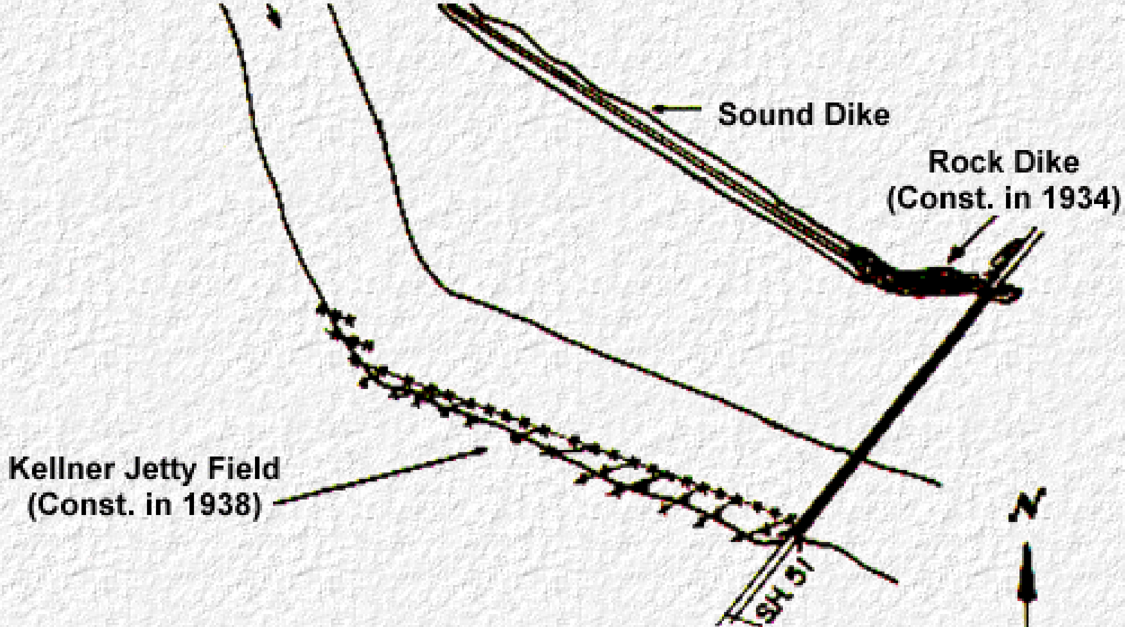
7.5.1 Cimarron River, East of Okeene, Oklahoma (Case 1)

The bridge and a rock dike on its north abutment were built in 1934 (Figure 7.5.1a). A high discharge year in 1938 caused the south bank to erode. A Kellner jetty field was installed to prevent further erosion of the bank. The jetty field was ineffective due to the lack of debris and suspended sediment load. A large flood in 1957 spread out over the floodplain in several places. After the flood, seven timber pile diversion units and a riprapped dike were installed in the old jetty field location to prevent future damage to the highway (Figure 7.5.1b). As of 1968, the south bank had been held in line by the timber pile diversion structures and the dike (Figure 7.5.1c).

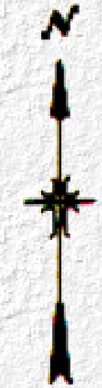
7.5.2 Arkansas River, North of Bixby, Oklahoma (Case 2)

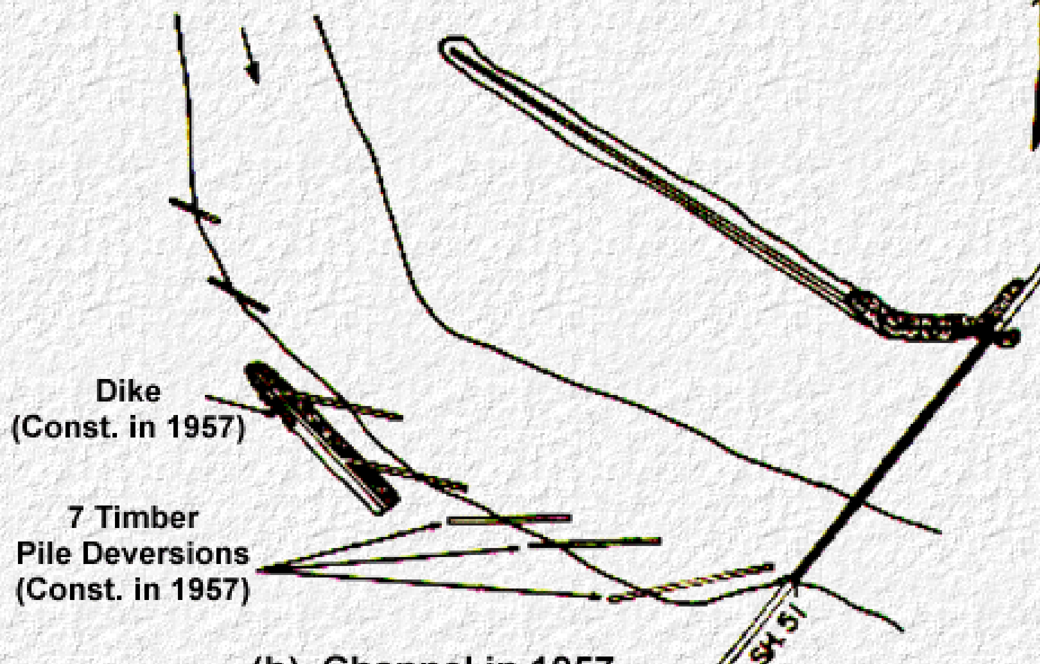
The bridge was built in 1938. A Kellner jetty field was installed on the north bank in 1939 to protect the north bridge abutment (Figure 7.5.2a). In 1948 minor floods eroded the south bank. A Kellner jetty field was installed to prevent further erosion (Figure 7.5.2b). Some time after, riprap was put on the south bank upstream and downstream of the jetty field (Figure 7.5.2c). In 1959, a 50-year frequency flood eroded the north bank and washed out a section of the north approach to the bridge. The flood also washed out two sections of roadway further north on the floodplain. The approach was rebuilt and riprap was installed on the embankment. A riprapped spur dike was also constructed just south of the north abutment. Five pile diversion structures were built to prevent further erosion of the north bank (Figure 7.5.2c). As of 1968, the south bank has remained stationary, and the north bank has filled (Figure 7.5.2d).



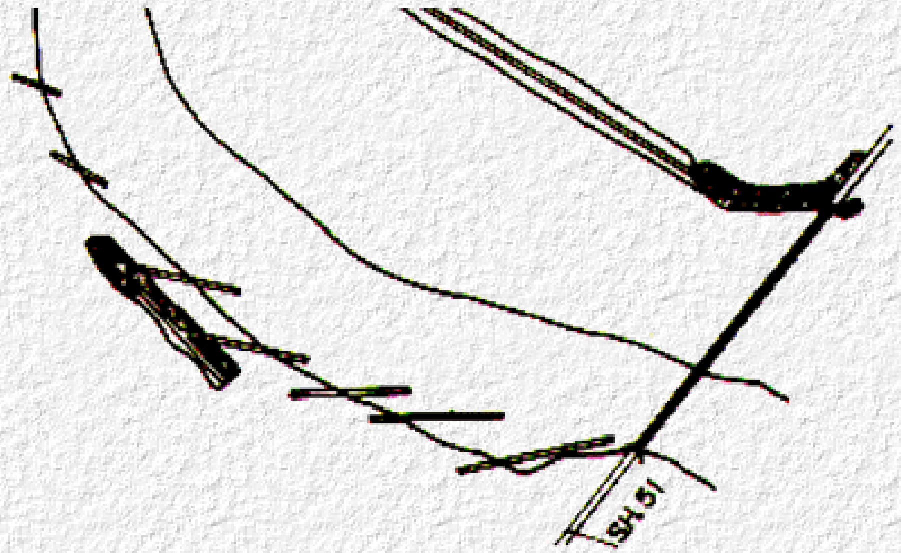


(a) Channel in 1938



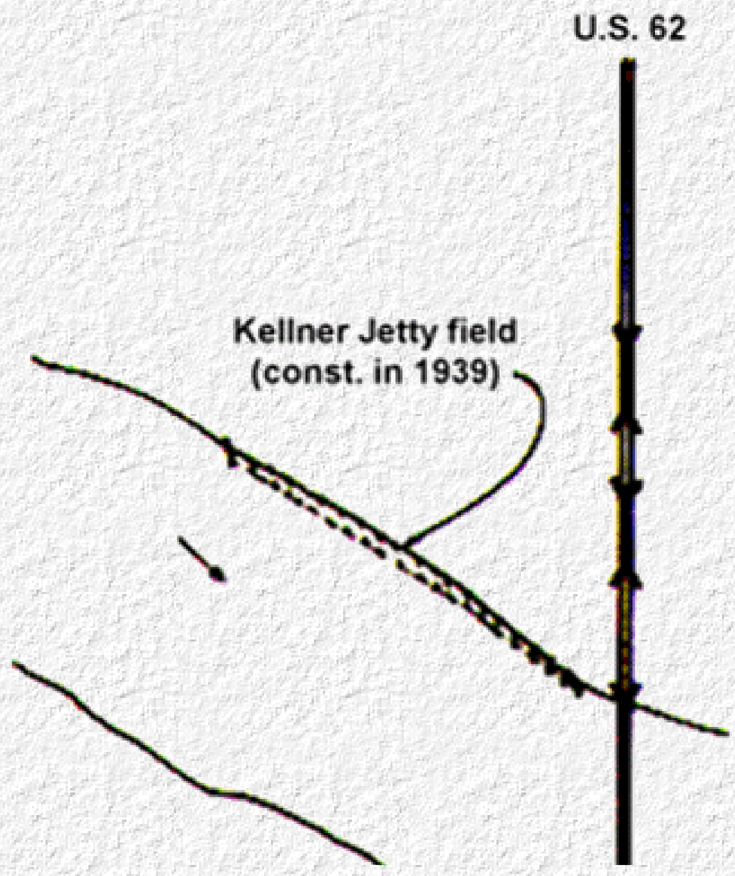


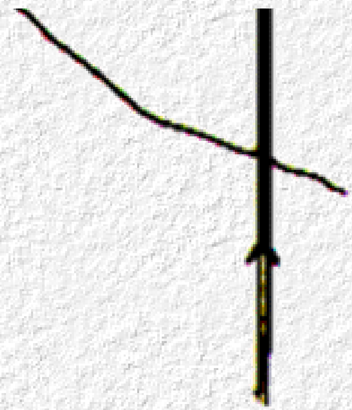
(b) Channel in 1957



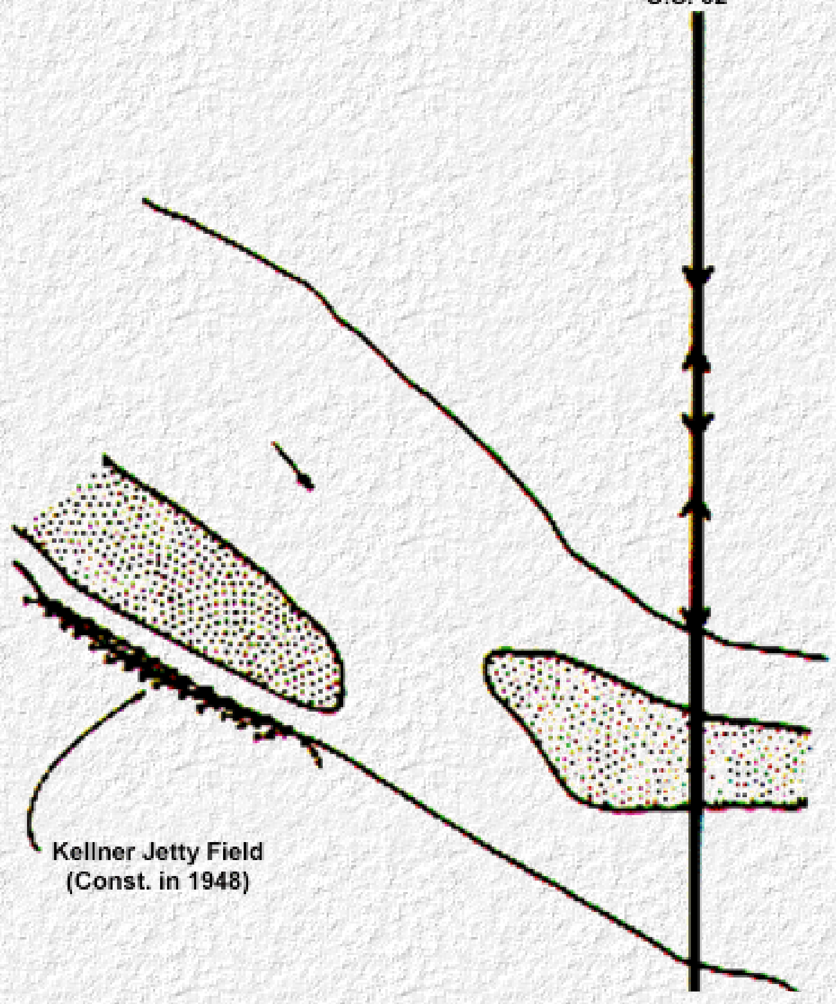
(c) Channel in 1968

Figure 7.5.1. Cimarron River, East of Okeene, Oklahoma

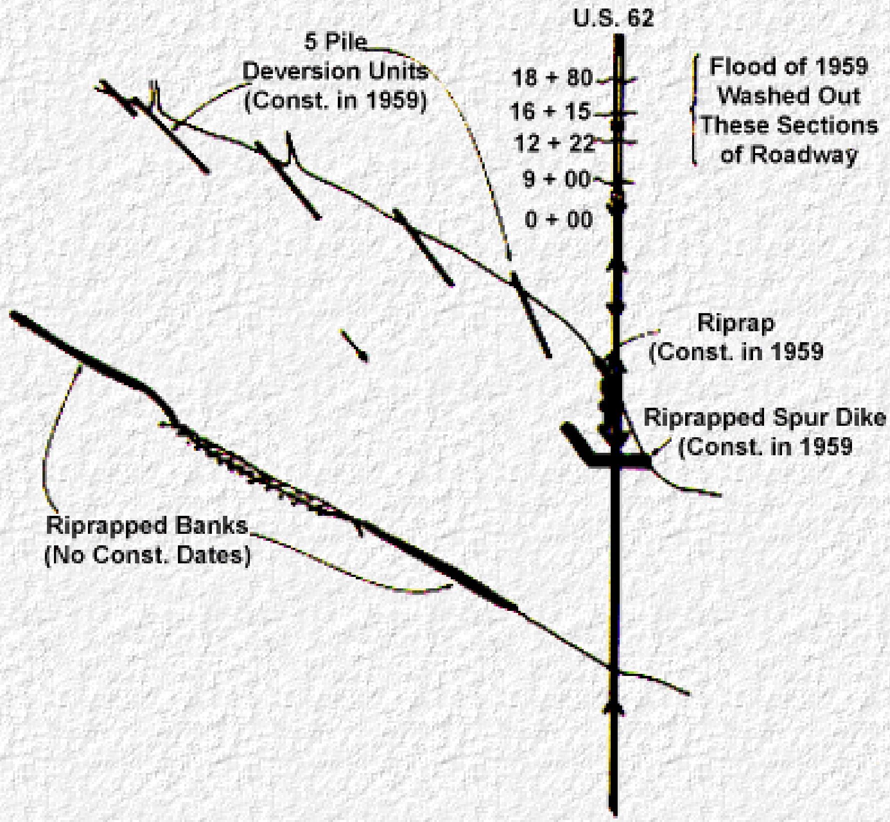




(a) Channel in 1939



(b) Channel in 1948



(c) Channel in 1959

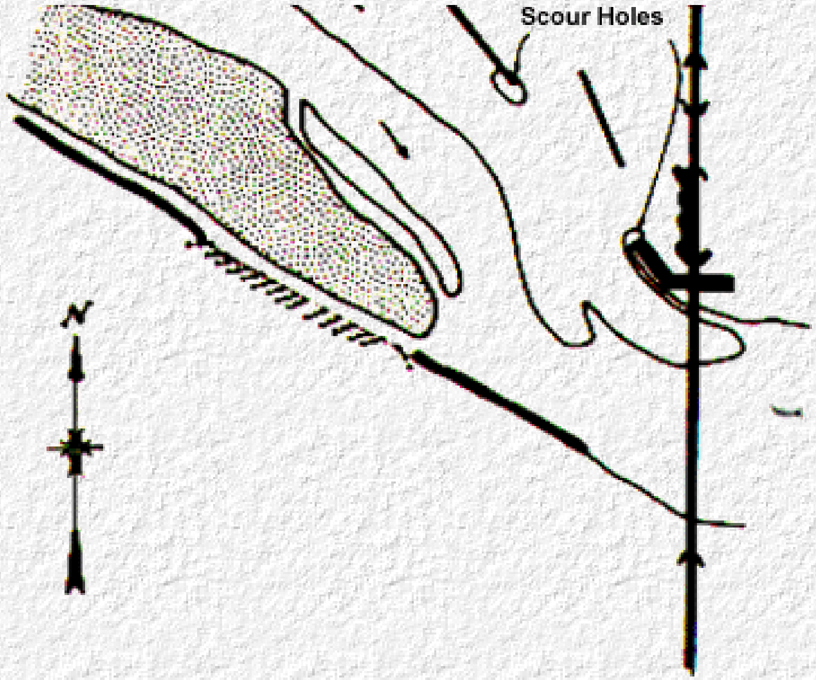


Figure 7.5.2. Arkansas River, North of Bixby, Oklahoma

7.5.3 Washita River, North of Maysville, Oklahoma (Case 3)

In 1949, floods washed out the north span of the bridge. Also, both banks upstream from the bridge were damaged. A temporary structure was installed in place of the north span of the bridge. In October of 1949, two Kellner jetty fields were completed upstream from the bridge to provide bank protection (Figure 7.5.3a). In 1950, a new bridge was constructed just downstream from the old bridge. State Highway 74 was realigned to conform to the new bridge. In eight months of operation, the Kellner jetty field on the northeast bank had completely silted in. This was largely due to the clay content in the suspended sediment and the large amount of drift in the stream (Figure 7.5.3b). The floods of 1957 did very little damage to the new bridge site or the banks. Floods in 1968 and 1969 have caused bank erosion on the north bank upstream of the jetty field which could eventually cut in behind the jettyfield (Figure 7.5.3c).

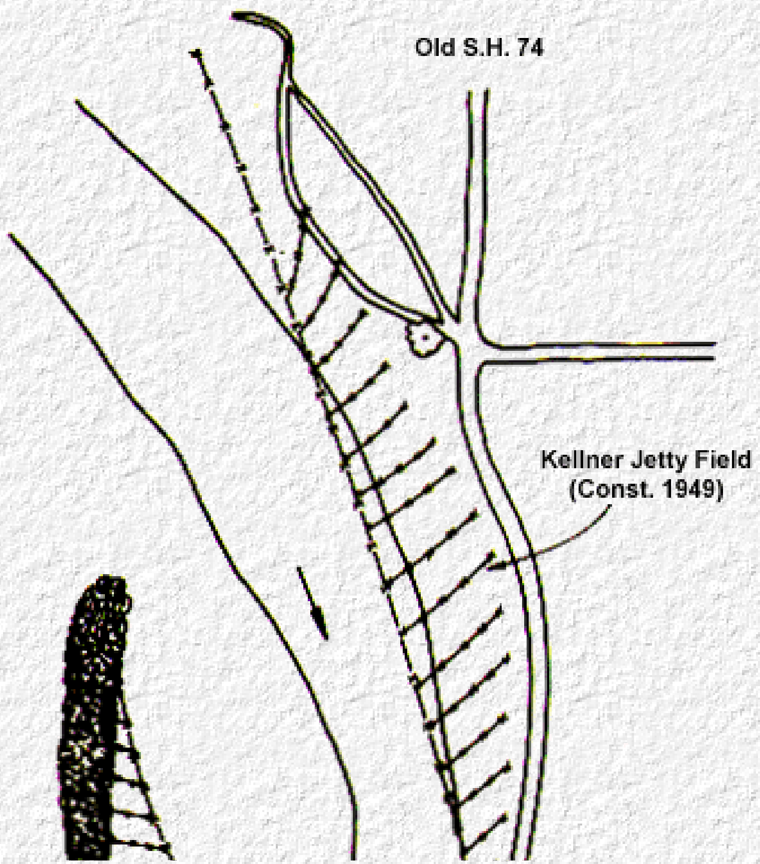
7.5.4 Beaver River, North of Laverne, Oklahoma (Case 4)

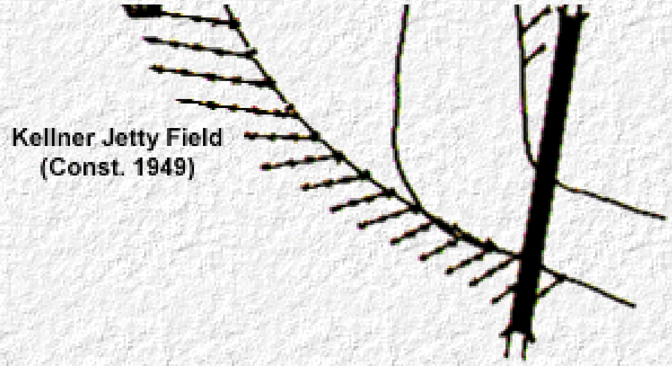
During high flows of 1938, the river washed over the south bank and damaged the approach roadway south of the bridge. The south end of the bridge was also damaged. Jetty fields were constructed in several locations upstream of the bridge in an attempt to reduce bank erosion. Two jetty lines were constructed in a side channel downstream of the bridge to discourage flow in that channel to prevent eddy currents from eroding the north embankment (see Figure 7.5.4a). A new longer bridge was constructed in

1941. High flows in 1946 caused severe erosion on the south bank upstream from the bridge. In 1949, an earth dike and jetty field were constructed on the south bank to prevent further erosion. In 1969, the river cut through a portion of the 1949 jetty field and eroded the earth dike (see Figure 7.5.4b). Car bodies were used as bank protection. However, car bodies are not environmentally acceptable and are difficult to hold in place unless anchored with cable or weighted down with concrete or rocks.

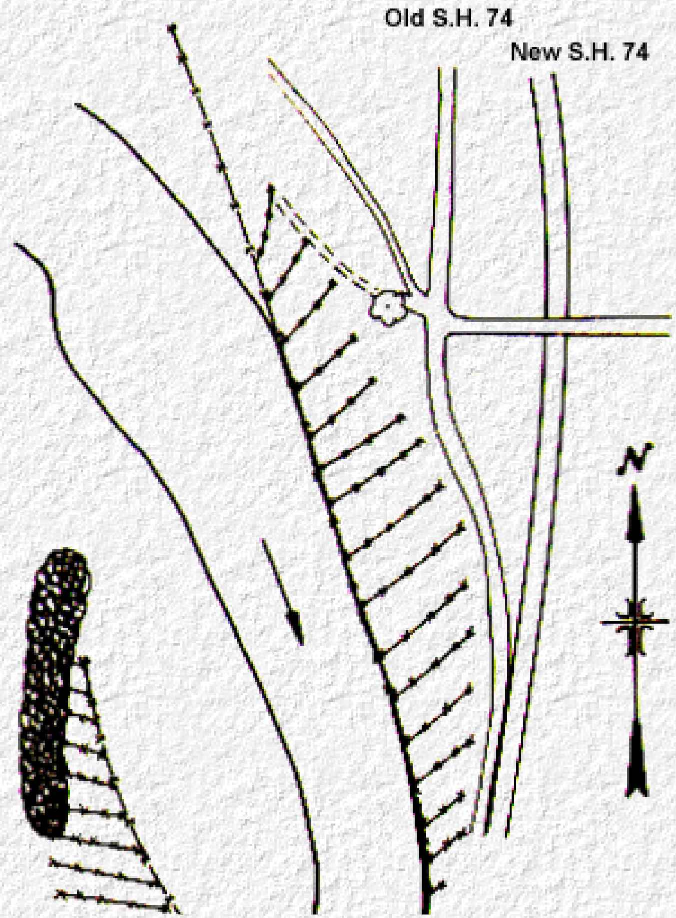
7.5.5 Powder River, 40 Miles East of Buffalo, Wyoming (Case 5)

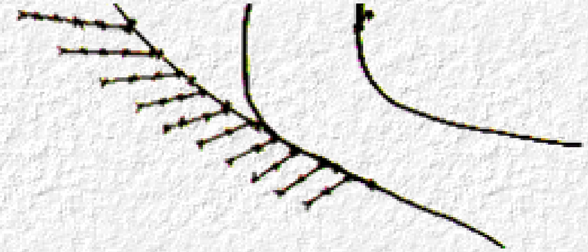
The Powder River has very fine bed material and a high sinuosity. The river contains dunes and antidunes at low flows. At the bridge site, there was a grove of cottonwood trees on the upstream east bank of the bridge and a dry draw coming in from the upstream right (see Figure 7.5.5a). Upon completion of the bridge, a large flood occurred. The river attempted to straighten out its meanders probably due to instream or flood plain mining in conjunction with the road construction. At the same time, the draw on the upstream right was bringing in a large amount of sediment, forcing the stream toward the upstream side of the (east) abutment. The flood flow uprooted the grove of cottonwoods which impacted on the sacked concrete riprapped spur dike at the west abutment and destroyed the dike (see Figure 7.5.5b). To restore the channel to its original alignment, a training dike was constructed from the bridge upstream to a nearby bluff. A jack jetty field was also constructed on the upstream meander to prevent the river from flowing across the point bar. The jetty field has begun to fail as a result of the streamside piles being undercut. The streamside anchors were rebuilt by driving the piles deeper in the center of small riprapped mounds.



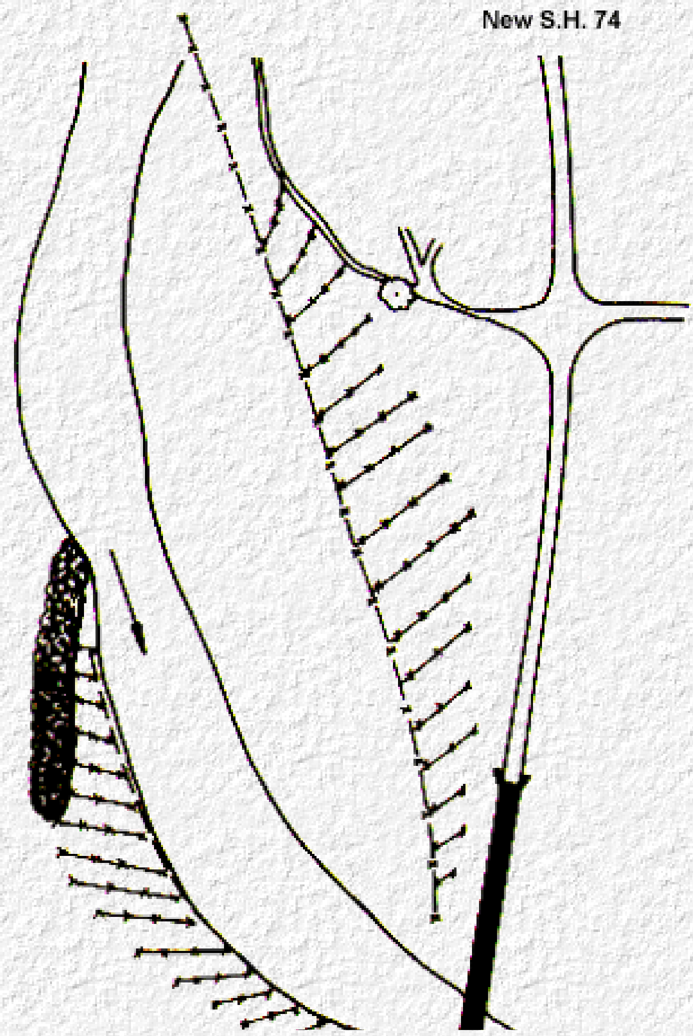


(a) Channel in October, 1949



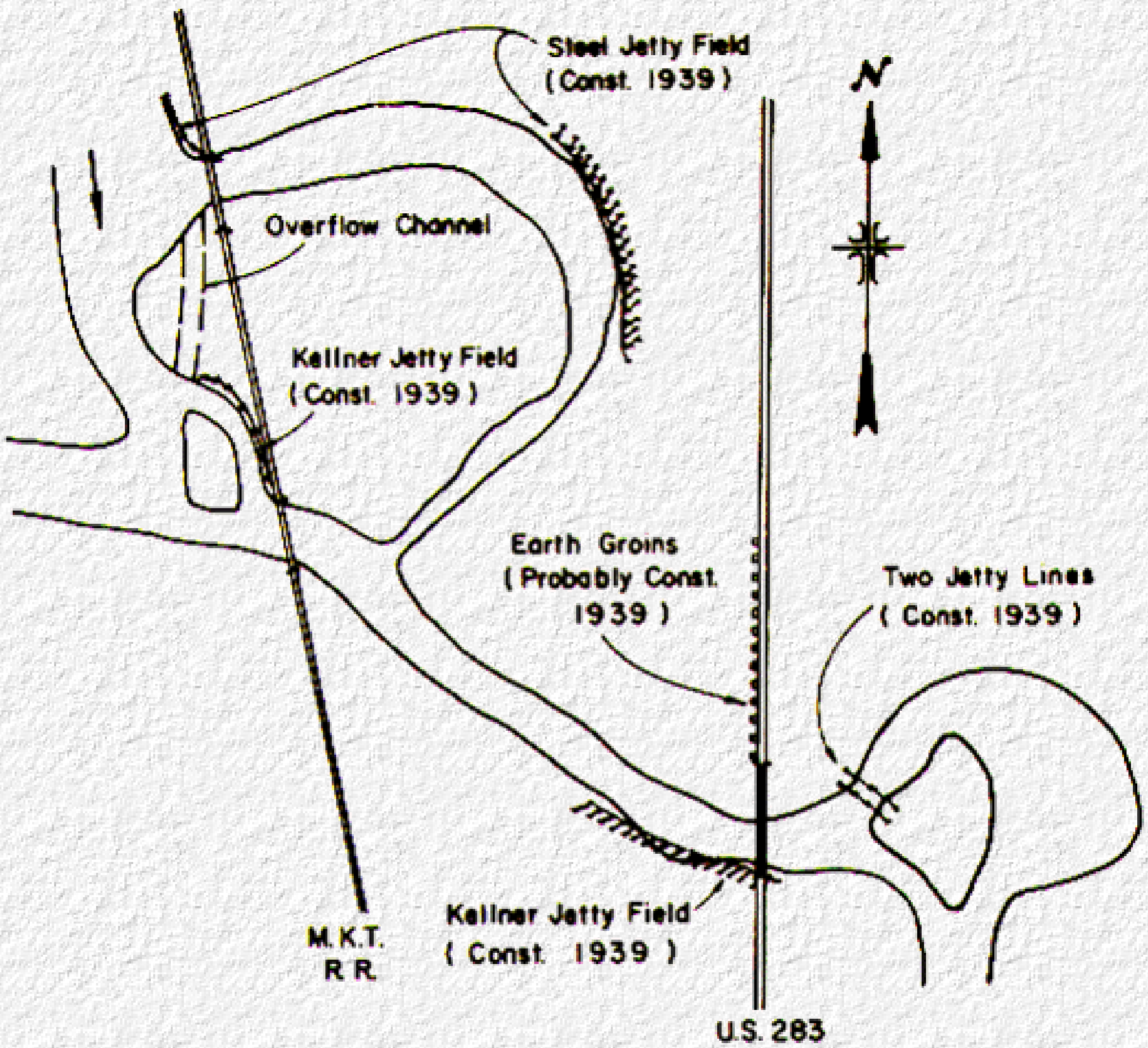


(b) Channel in May 1950

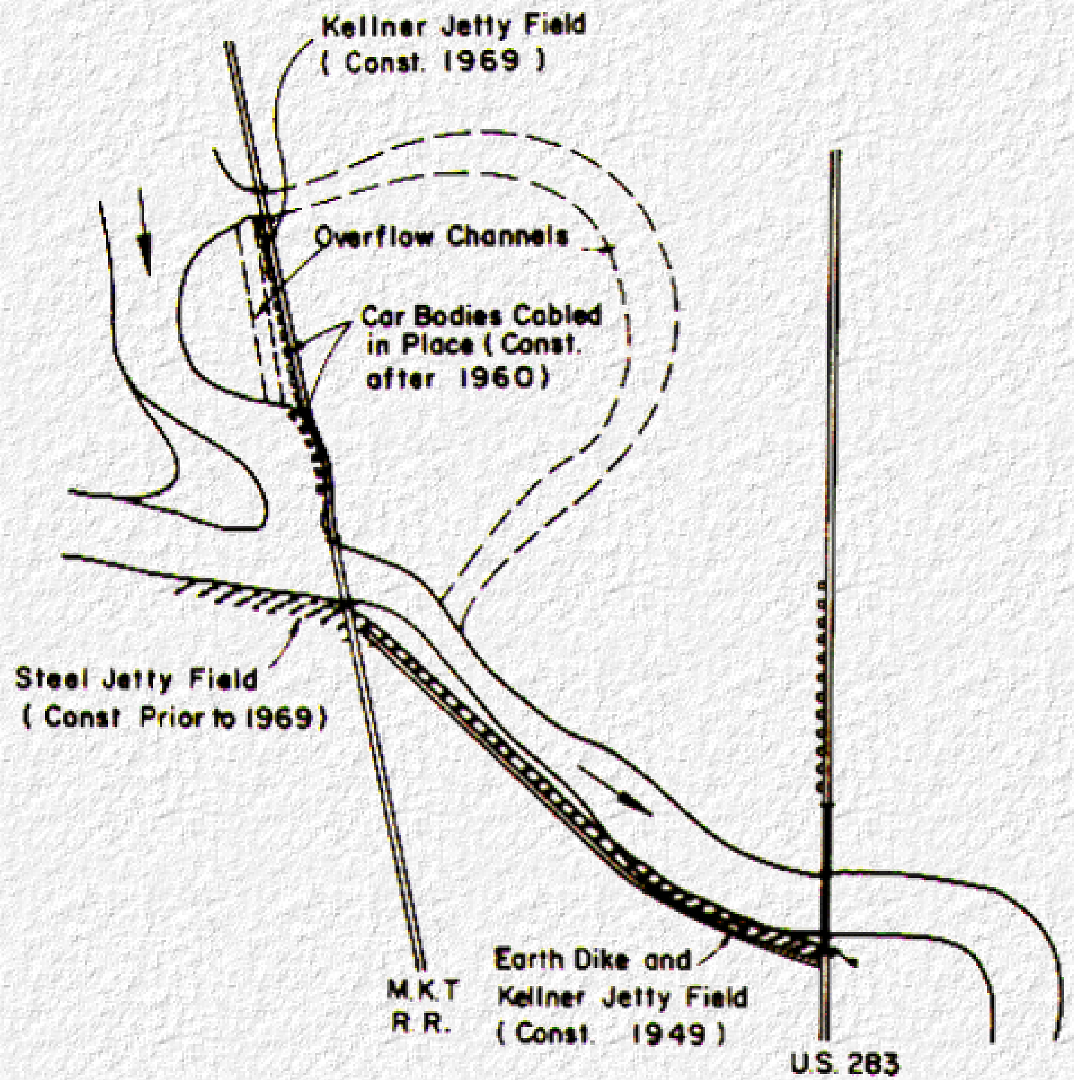


(c) Channel in 1969

Figure 7.5.3. Washita River, North of Maysville, Oklahoma



(a) Channel in 1939



(b) Channel in 1960

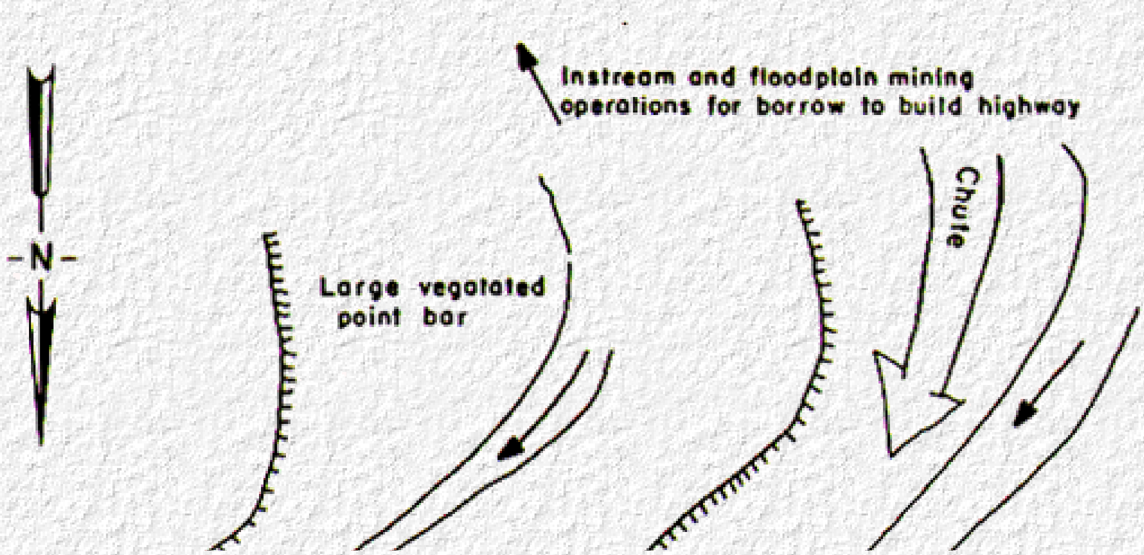
Figure 7.5.4. Beaver River, North of Laverne, Oklahoma

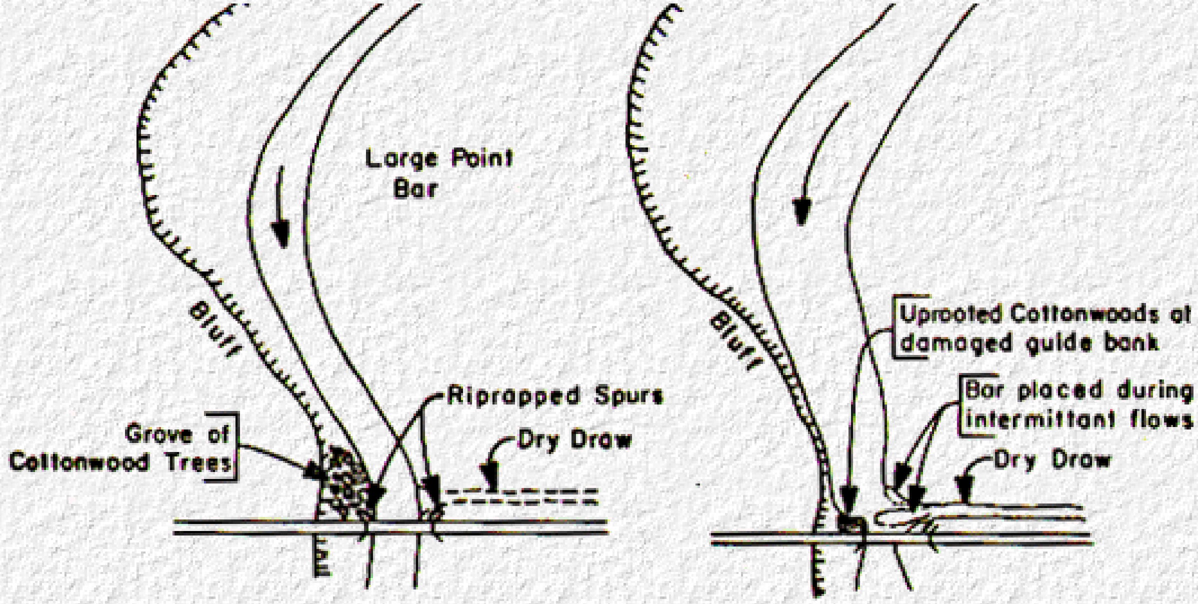
7.5.6 North Platte River, near Guernsey, Wyoming (Case 6)

The North Platte River is a fairly stable river in this reach as a result of reservoir control upstream (Figure 7.5.6). The bed is coarse granular material with cobbles in evidence. It was decided at this crossing to build the bridge over the main channel and part of the island and to block off the active overflow channel on the opposite side of the island (Figure 7.5.6a). Two situations can occur due to this choice for the bridge crossing. One situation is that the concave bank erodes due to the high velocities resulting from the decreased area of flow under the bridge (see Figure 7.5.6b). In fact, with extreme flows the river could erode a chute across the point bar on the first bend downstream. The other situation which may occur is that the high velocity flow carries increased sediment load and deposits this material in the eddies downstream (see Figure 7.5.6c), which in this case would not cause any problems. The county insisted on low construction costs so the training dike had minimal riprapping as shown in the figure. As a result the dike is now eroding badly. The high bank began eroding so the property owner dumped in some broken concrete, rock and debris. The channel degraded through the opening but as expected the coarse bed has armored quite well. Except for the eroding of training dike, everything has worked well for over 20 years.

7.5.7 Coal Creek, Tributary of Powder River, Wyoming (Case 7)

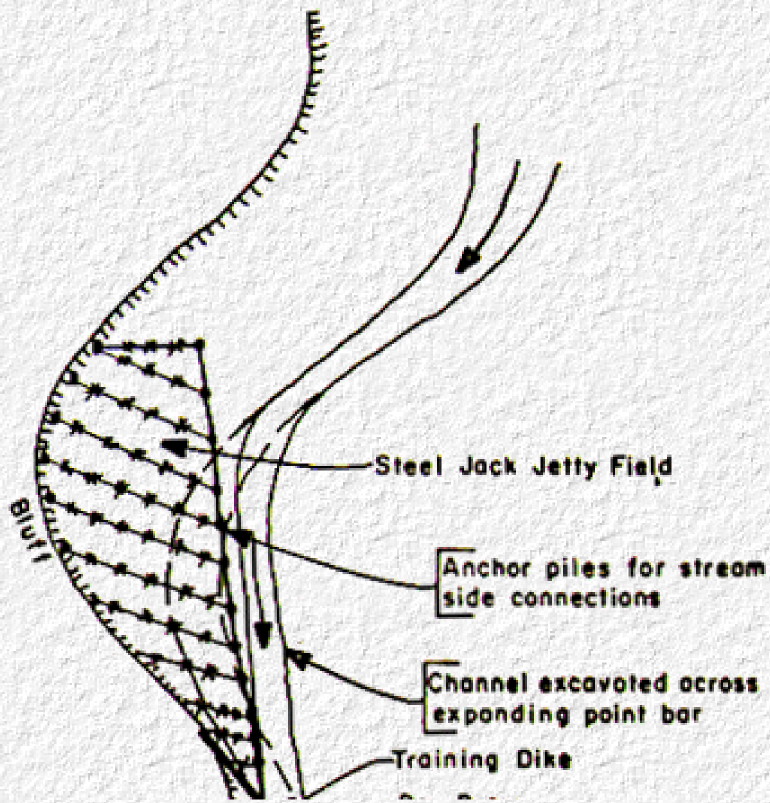
A small bridge was constructed over intermittent Coal Creek. Coal Creek is a dry draw that is incised. There existed head cuts downstream from the bridge at the time of its construction (see Figure 7.5.7a), some headcut were over a mile downstream. During a subsequent flood, at least one head cut moved upstream through the bridge site. This headcut almost undercut the midstream piles and it exposed some of the abutment piles. To prevent further degradation under the bridge when the remaining headcuts move through, chainlink enclosed riprap was placed as shown. The lower portion was placed as articulated riprap. The migration of the downstream headcuts has since resulted in some downward articulation. A heavy steel girder fence-like device to allow the lower portion of the replaced riprap to be stable at a steeper slope was necessary to establish a larger waterway opening. The original waterway was significantly smaller than the final configuration. Other alternatives would have been to excavate the head cuts in the channel through the bridge site before the bridge construction, allowing them to move naturally upstream from there; or to set the piles deeper in anticipation of the lowering of the bed elevation; or to construct a weir control structure at the location of the headcut to prevent upstream migration.

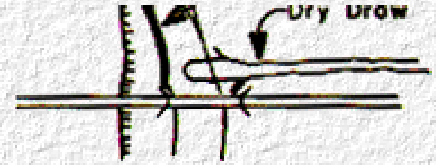




(a) Channel after completion of bridge

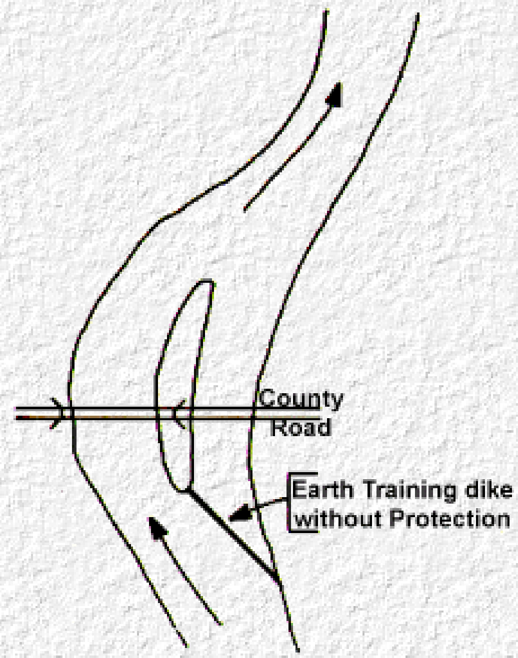
(b) Channel during flood



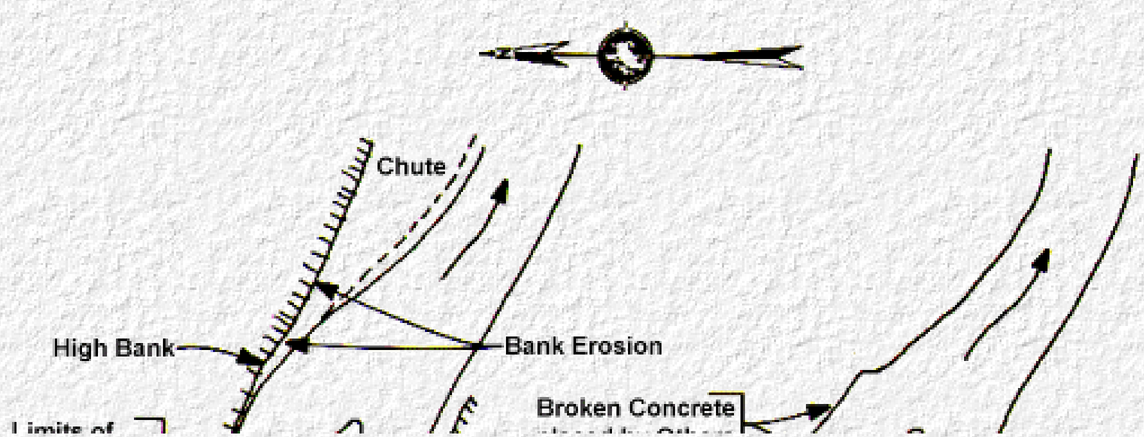


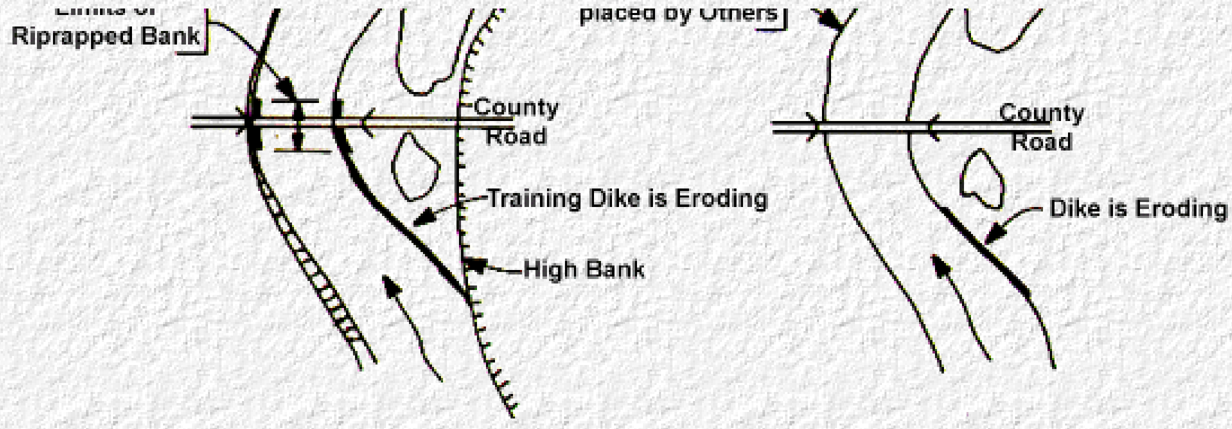
(c) Channel after restoration

Figure 7.5.5. Powder River, 40 Miles East of Buffalo, Wyoming



(a) Channel immediately after closure

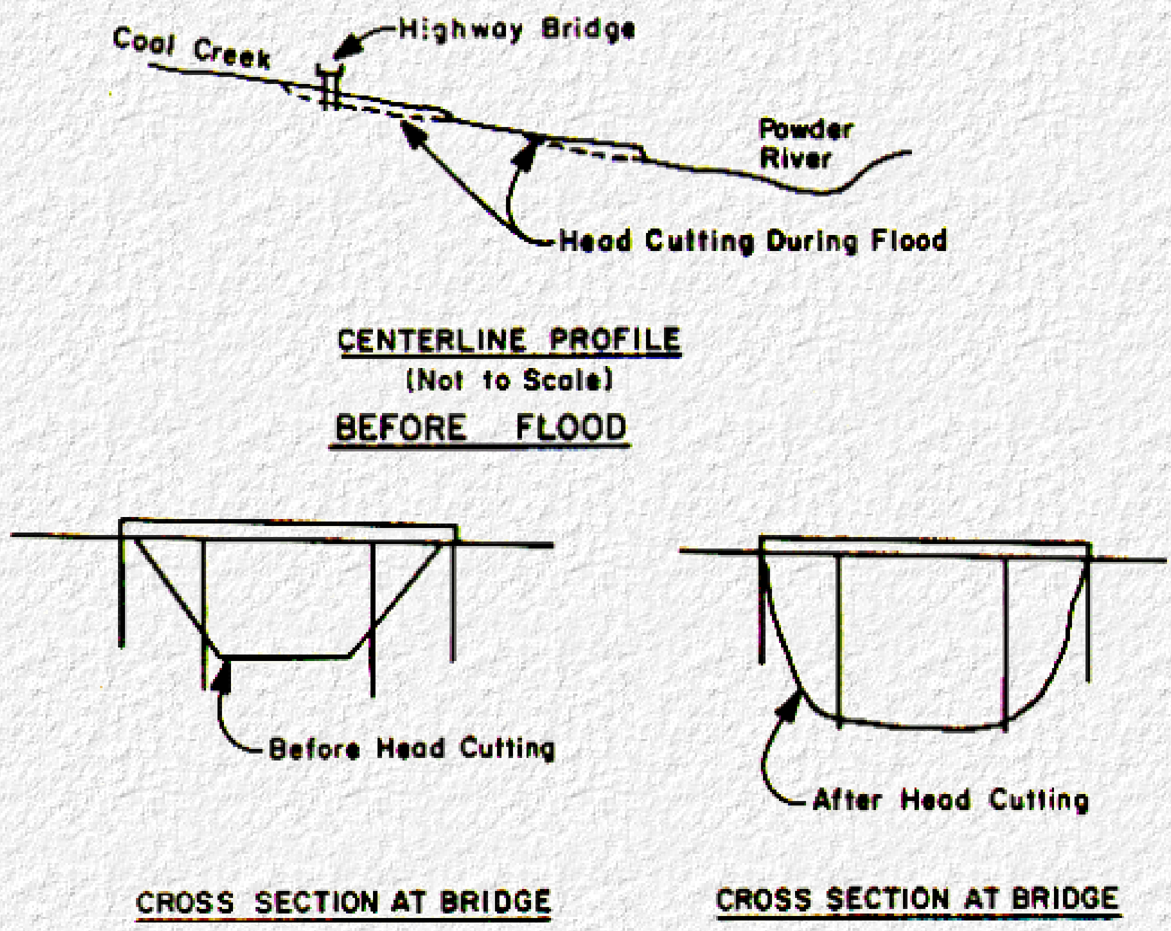




(b) Channel after first flood (situation 1)

(c) Channel after first flood (situation 2)

Figure 7.5.6. North Platte River near Guernsey, Wyoming



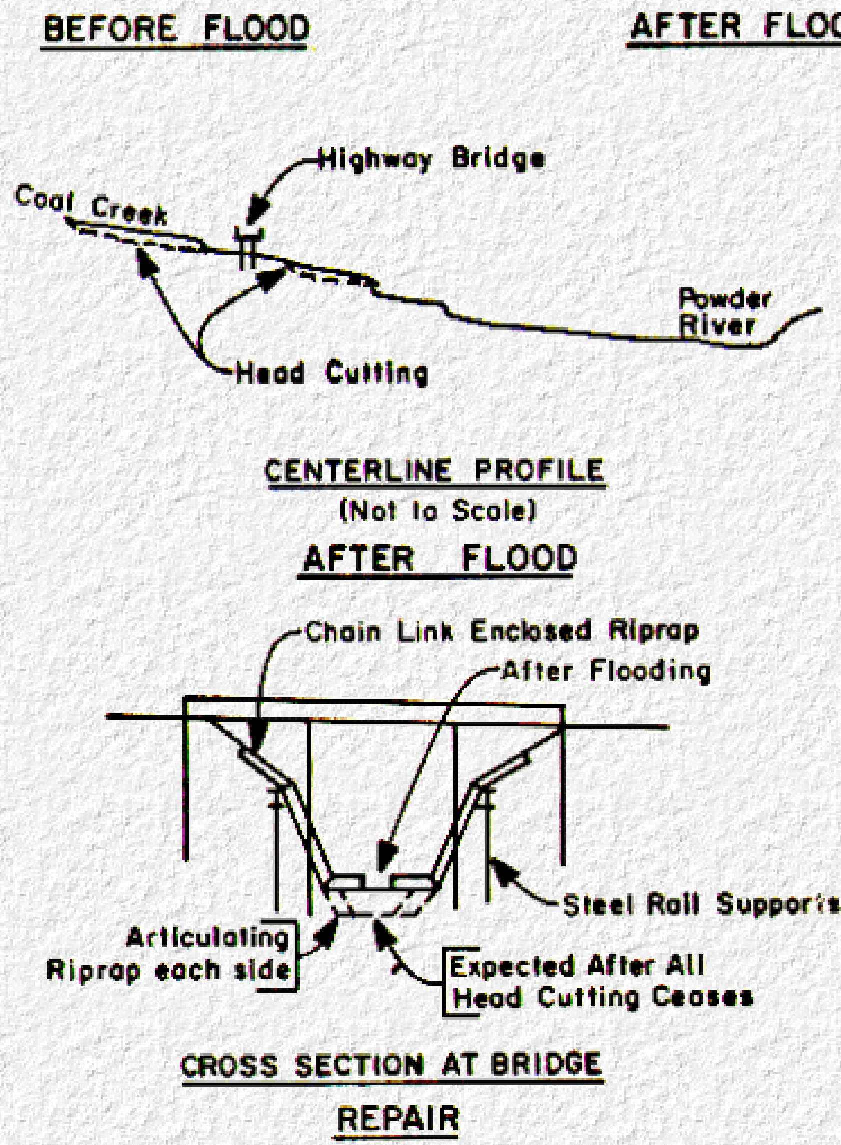


Figure 7.5.7. Coal Creek, Tributary of Powder River, Wyoming

7.5.8 South Fork of Forked Deer River at US-51 near Halls, Tennessee (Case 8)

Cases 8 through 11 were taken from Federal Highway Report Number FHWA/RD-80/038. In general, these cases are presented as in the original form. They have, however, been edited slightly and modified to fit the format of this text.

The location of interest is near the Dyer and Lauderdale County line as seen in Figure 7.5.8a (lat. 35°57', long. 89°24'). Dual bridges were built in 1963, with a 16 m main span supported by wall-type piers in the main channel, and thirty 8 m approach spans supported by concrete pile bents. In 1975, both bridges over the main channel were rebuilt, with a main span of 22.5 m supported by hammerhead piers. Spill-through abutments, set back from the main channel, were protected with sacked concrete in 1963 and have remained stable.

The drainage area is 2,688 km^2; the bankfull discharge is 28 m^3/s, and the width where bordered by natural vegetation is 24 m. The stream is perennial, alluvial, sand bed, in a valley of moderate relief and in a wide floodplain. The natural channel has a sinuosity of about 2.5 but the channel has been straightened; it is equiwidth, not incised, cut banks are rare, with silt-sand banks.

The channel was first straightened and enlarged in the 1920's by local drainage districts; but, probably because the natural floodplain forest was not cleared, the banks remained stable. In 1969, the Corps of Engineers straightened and enlarged a reach about 4.8 km in length downstream from the bridge, reducing the length about 20 percent. During the past few decades, and particularly in recent years, the floodplain has been cleared of trees for agricultural purposes.

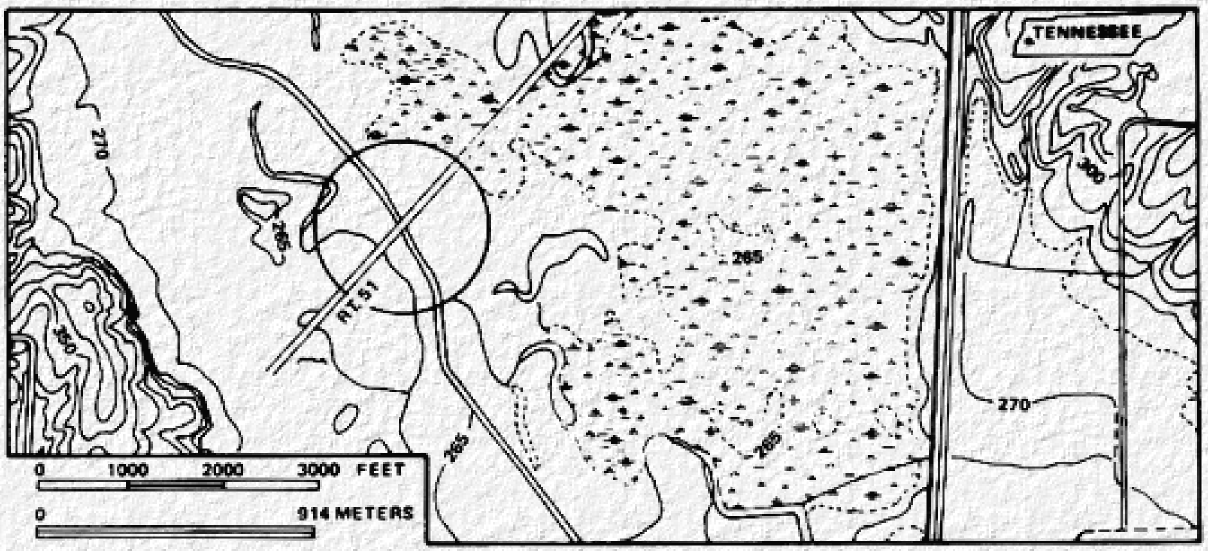


Figure 7.5.8a. Map Showing South Fork of Deer River at U.S. Highway 51 Crossing

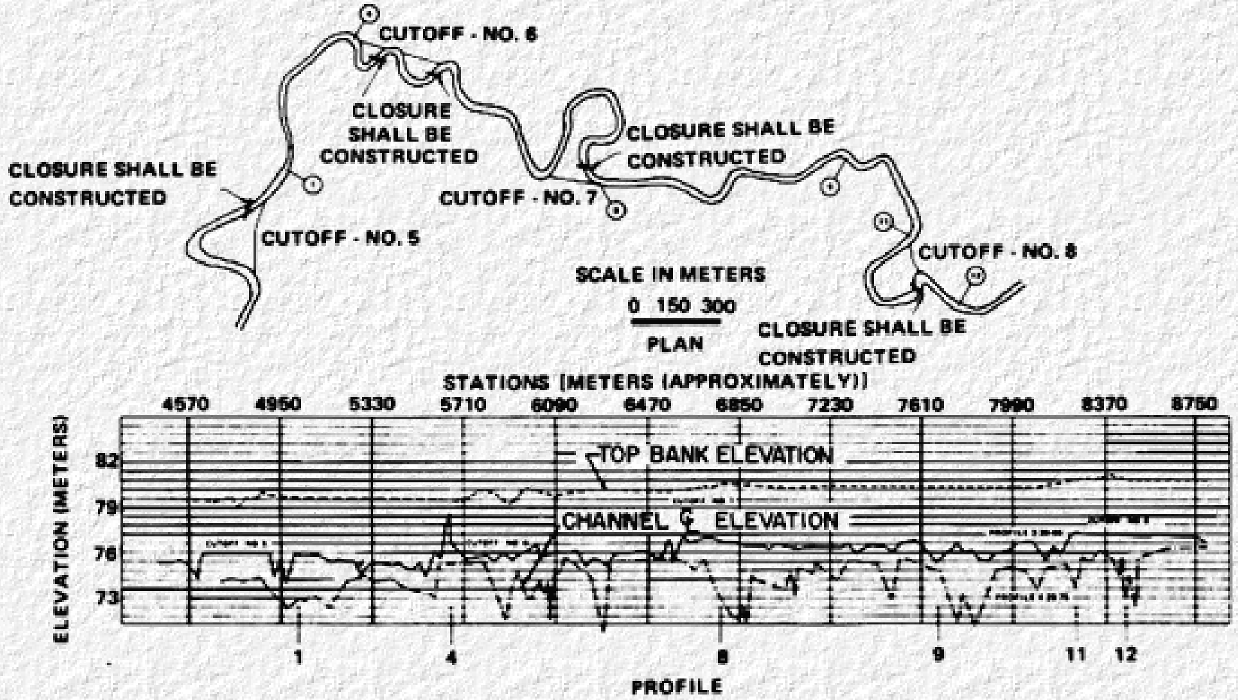


Figure 7.5.8b. Channel Modifications to South Fork of Deer River at U.S. Highway 51 Near Halls, Tennessee

Figure 7.5.8b illustrates the Corps of Engineers channel modifications that reduced the channel length and increased the channel slope. Figure 7.5.8b also provides an accurate profile of the channel before any modifications were made in 1975. Several cross-sections are illustrated at various longitudinal stationings showing the extent of the channel modifications.

Between 1970 and 1971 the left bank receded an average distance of 4 m. The peak discharge during this period was 215 m^3/s (R.I. of flood, 1.5 year). Timber pile retards were built at the left bank near bent 7 and a single row of pile with wood face planks extending from the downstream end of bent 7 for a distance of 37.5 m upstream.

Between 1971 and 1973 the peak discharge during this period was 751 m^3/s (R.I. 17 years). Bankfull stage occurred several times, high flows sustained for periods of weeks. The left bank continued to erode behind the retard, average distance of recession was about 2 m for the 3-year period. Bent 7 became exposed below the ground line. Concrete was poured at the base to prevent further erosion. Slumping from the left bank deflected flow toward the right bank, causing rapid erosion and failure of bent 8. The south lane of the bridge was closed. A detailed inspection was made in 1973, but little field data was collected. Data collected indicated a large local scour problem at the bridge. No profile data was available to evaluate the gradation problem.

In 1975 both lanes of the bridge were rebuilt, with new piers having deeper footings and less area normal to flow (Figure 7.5.8c). Single-row timber pile retards were built along both banks in the vicinity of the bridge. A large scour hole in the center of the channel downstream from the bridge, attributed to flow constriction during bridge construction, was filled with gravel.

In 1977 a detailed inspection was made and cross-section data at the bridge indicated that the local scour problem was somewhat corrected and the gradation problem was fairly stable. Effectiveness of the timber pile retard has not yet been tested, and the area between the retard and the bank is not accumulating sediment. The lowermost face plank on the retard is about 1.5 m above the streambed. From experience face planks should be extended to, or below, streambed elevation. In addition, the upstream end of the retard seems to be keyed into the bank for an insufficient distance. Vegetation is becoming re-established on the banks, which appear more stable now than in the recent past.

The bridge failed because of channel degradation and concurrent bank recession, which are directly attributable to straightening of the channel for drainage purposes and clearing of the banks and floodplain for agricultural purposes. Channel width increased by a factor of approximately 2, between 1969 and 1976. The clearing of vegetation is apparently one of the most critical factors, because channel straightening in the 1920's which was not accompanied by extensive clearing, did not result in significant bank instability. The timber pile retard installed in 1971 was of inadequate design, in view of the seriousness of the problem. Bank recession might have been controlled by an adequate retard or other countermeasure, but channel degradation is more difficult to control.

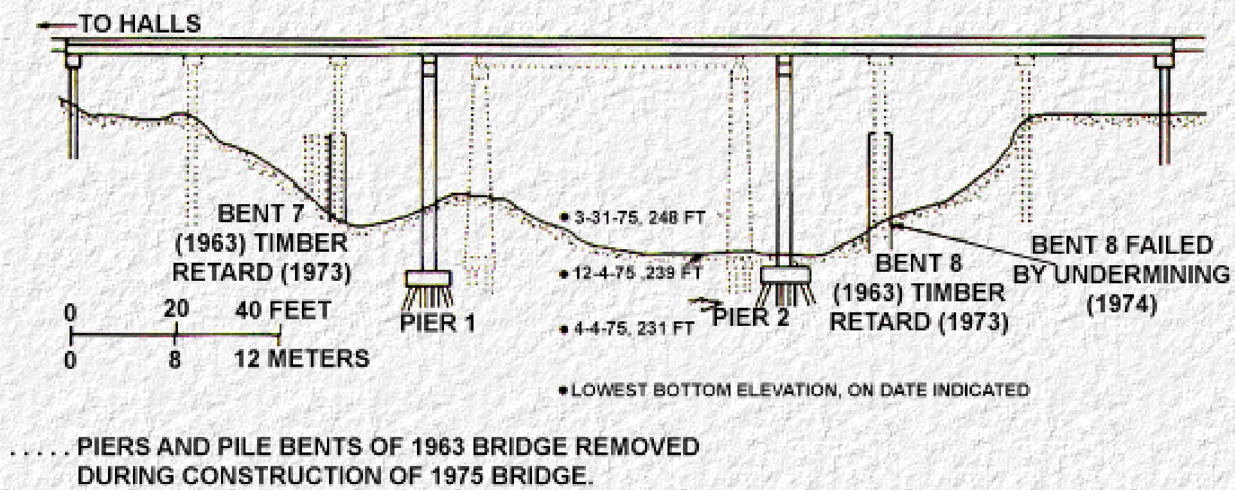


Figure 7.5.8c. Elevation Sketch of U.S. Highway 51 Bridge

The site was inspected in 1973, in 1974, four times in 1975 and in 1976 by numerous federal and state agency employees. It should be pointed out that little actual field data were collected to document the progressive channel change. All inspections have concluded that the channel changes were responsible for the gradation problems and other related hydraulic problems.

7.5.9 Elk Creek at SR-15 near Jackson, Nebraska (Case 9)

Elk Creek is located in Dakota County and is a tributary of the Missouri River. It flows into the Missouri River just upstream of Sioux City, Iowa. The State Road 15 Bridge just west of Jackson is of interest. The stream is perennial but flashy, alluvial, sand-silt bed and in a valley of low relief with a wide floodplain. The channel is sinuous, incised by degradation, and has silt-clay banks.

The stream bed has degraded at least 3 m since 1955. There are two primary reasons for this degradation. First, channel modifications have been made to improve and maximize agricultural production. As a result, the channel has been straightened and changed at isolated locations. Second, and probably more important, is the general degradation below Gavins Point Dam. Missouri river stage trends, for almost 100 years for eight of the key main stream gaging stations below Sioux City, indicated at least 3 m of degradation at Sioux City, as indicated in Figure 7.5.9. This degradation is probably due to three main reasons as follows:

1. Between 1890 and 1960 the Missouri River length from Sioux City to Omaha has been reduced 21 percent by the Corps of Engineers. As a result the stream bed slope was increased.
2. The sediment-free water released at Gavins' Point Dam is transporting the bed sediment that is available.
3. The rather high sustained flows of the regulated Missouri River System do not allow for any aggradation or filling.

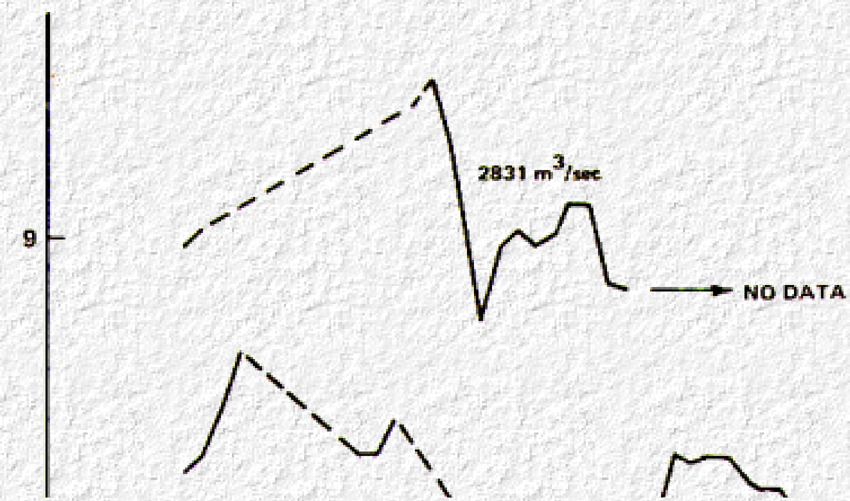
The degradation is primarily responsible for lateral instability as the channel has almost doubled in width. The degradation has exposed the pier footing.

Tributary degradation, resulting from degradation on the mainstream of the Missouri River is to be expected. The Missouri River has historically degraded, as indicated in the Missouri River stage trends. This condition should be evaluated on an annual basis and bridges inspected that are subject to this headcutting that is experienced by each tributary that is not protected by a grade control structure. Failure of this particular bridge due to degradation is not likely because of the great depth to which bridge foundations have been placed.

7.5.10 Big Elk Creek at I-90 near Piedmont, South Dakota (Case 10)

Big Elk Creek is located in Meade and Lawrence counties and is a tributary of the Cheyenne River. The headwaters of Big Elk Creek originate in the Black Hills National Forest. I-90 crosses the Big Elk Creek on an alluvial fan just outside of the National Forest, where there is a significant reduction in channel slope. The bridge, built in 1964, is 54 m long, has pile bents with square piles, spillthrough abutments, and a concrete deck. The creek is intermittent, flashy and alluvial with cobble and gravel bed. The drainage area above I-90 is 1300 km^2 and the design discharge was 85 m^3/sec.

The highway crossing is located on an alluvial fan. At this location there is insufficient slope (energy) to transport the cobble and gravel material. Since 1964, it has been necessary to excavate about 20,000 m^3 of deposited bed material on three occasions at an expense of hundreds of thousands of dollars. The excavation was necessary to pass the flow from the spring snowmelt runoff. The primary aggradation problem is insufficient flow area and is aggravated by too many piers in the channel as well as a bad alignment with a 67 degree skew.



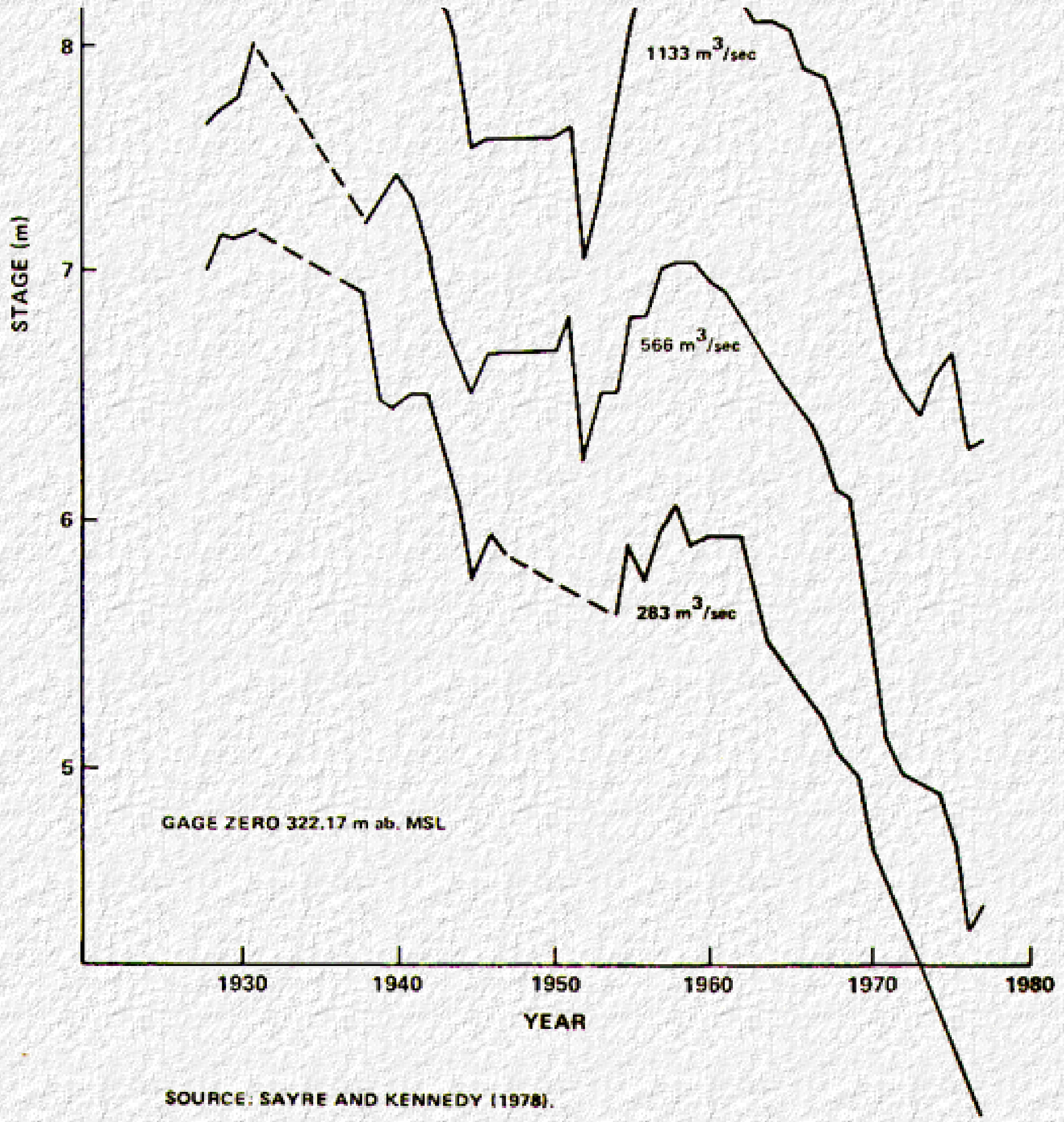


Figure 7.5.9. Stage Trends at Sioux City, Iowa, on the Missouri River

In 1966 several rock and wire basket flow deflectors were installed for several hundred meters upstream of the bridge to constrict the flow and increase the transport characteristics. Figure 7.5.10 illustrates the deflector arrangement as well as the alignment problem. The deflectors were not very effective. They did constrict the flow and increase the velocity to transport the gravel sizes, but the cobble bed material still deposited upstream of the bridge. The constriction was not enough compensation for the reduction in slope as the creek comes out of the Black Hills.

The channel is not well defined as it flows onto the alluvial fan. As a result, it is difficult to locate a bridge to take into account both significant lateral channel migrations and bed elevation changes. A possible solution to the problem would be to build a debris basin upstream of the bridge to trap the large-sized bed material. This would then provide a great source of gravel which is in demand in that area. This might be a site where a bedload transport model may be helpful in determining the solution to the aggradation problem.

7.5.11 Lawrence Creek at SR-16 Near Franklinton, Louisiana (Case 11)

Lawrence Creek is located in Washington Parish (lat 30° 19', long. 90° 19'). The bridge is 13 m in length and has pile bents with square concrete piles and spill-through abutments revetted with sacked concrete. The drainage area above SR-16 is 130 km^2, and the channel slope is 0.003. The stream is perennial, alluvial, sand-bed, in a valley of low relief and has a wide floodplain. The stream is sinuous, not incised, and has sand banks.

Sand and gravel mining downstream from the bridge has caused serious channel degradation. Headcutting moved upstream causing banks to cave along with large trees and debris. The trees partially blocked the main channel upstream of the bridge, such that at high flows the main flow was diverted to an old channel that flows almost parallel to SR-16 and then makes a 90° bend to flow under the bridge.

The channel has been lowered by an estimated 3 m at the bridge. The degradation, bank caving, and debris problems resulted in extensive lateral erosion of banks at the bridge.

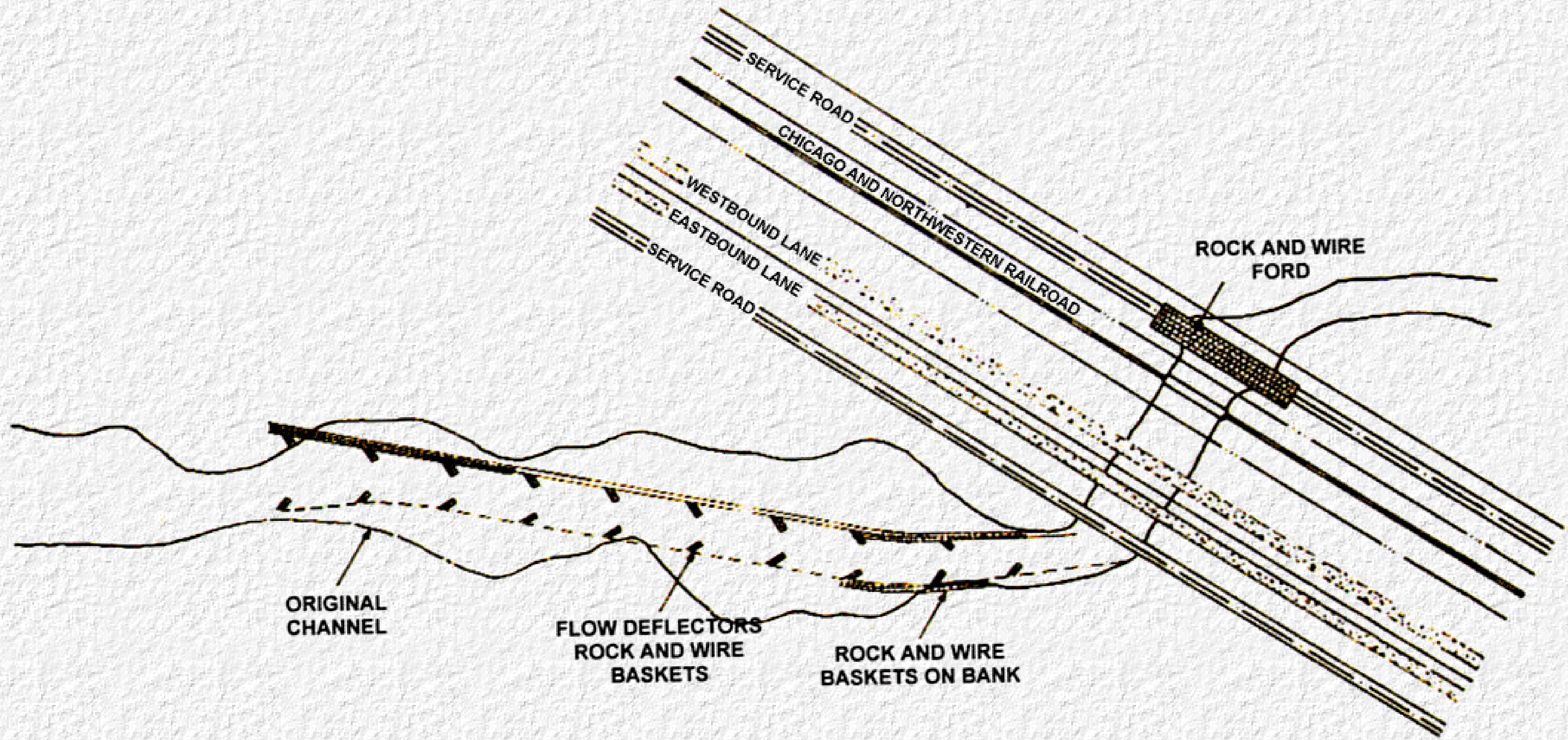


Figure 7.5.10. Deflector Arrangement and Alignment Problem on I-90 Bridge across Big Elk Creek near Piedmont, North Dakota

As a countermeasure, the bridge was lengthened by the addition of a span to the right end in 1967. In 1973, the approach embankment was raised and a curved spur dike of earth, revetted with broken concrete riprap, was built at the right abutment to protect the embankment. Timber cleared during construction of the spur dike was to be placed along the embankment but was placed too far from the embankment. During a flood in 1974, flow was diverted against the spur dike, eroding the tip and upstream side of the dike and impinging against the approach embankment.

Lateral instability of the channel is directly attributed to sand and gravel mining. Although the spur dike serves to direct flow through the bridge waterway, it does not seem to be an effective measure for protection of the approach embankment, which is subject to erosion or breaching during major floods. The main channel of Lawrence Creek was apparently too nearly parallel to the right approach embankment for the spur dike to function properly.

7.5.12 Outlet Creek at US-101 Near Longvale, California (Case 12)

Case 12 through Case 14 were taken from Federal Highway Administration Report No. FHWA/RD-80/158. These cases are good examples of situations where the river channel was relocated to accomodate highway encroachments and crossings. These cases are reprinted as in the original publication except for minor editing to conform with the format of this text.

In this case, the a channel has been shortened from 435 m in length to 335 m and relocated to avoid two crossings on the realigned highway curve (Figure 7.5.11). The stream is semi-alluvial, and resistant bedrock crops out in the bottom of the relocated channel. The highway embankment, which forms the right bank of the relocated channel, is heavily riprapped, and the main potential for instability is at the left bank, which is a steep (3/4:1) slope cut into colluvial material. However, no erosion or slumping was observed.

The site location is on US-101, 3.2 km south of Longvale, California. Outlet Creek is perennial with a drainage area of about 360 km^2 at the site and with an average discharge of 12 m^3/s. The channel width is 9-15 m, with a channel slope of 3.6 m/km. The bed material is gravel and cobble, bank material is gravel and sand where alluvial.

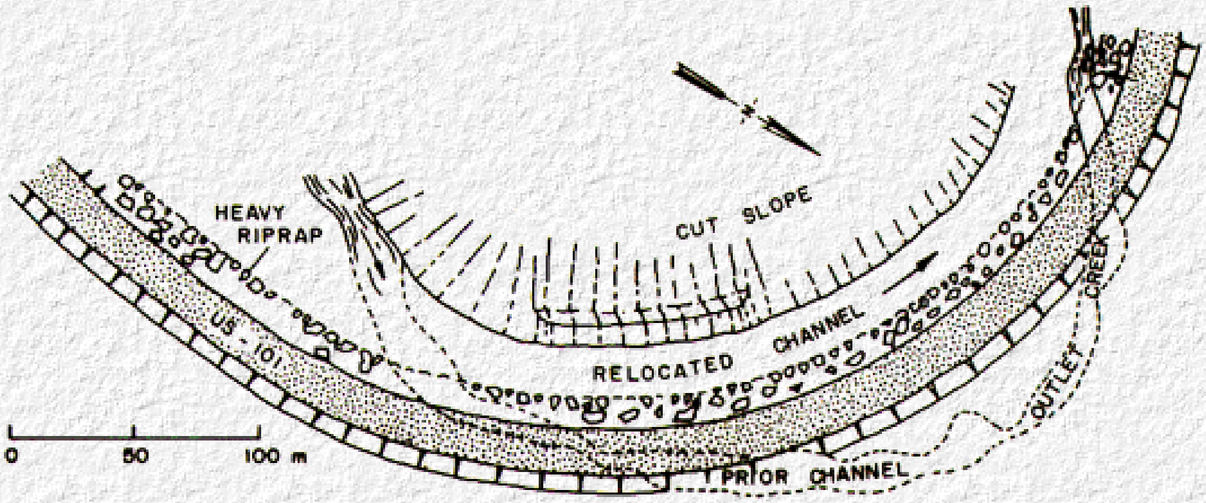


Figure 7.5.11. Plan Sketch of Channel Relocation, Outlet Creek

The length of a curve in the natural channel was shortened by a factor of 0.77, and room for the relocated channel was made by grading back a steep valley side-slope. The relocated channel lies between this graded slope and the riprapped highway embankment. Riprap on the embankment includes rocks weighing several tons, and erosion of the embankment is unlikely.

No maintenance work, following an extreme flood in 1964, was needed. Small trees have become established on the riprapped highway embankment and along the base of the cut slope.

Landsliding, the major potential for instability along the relocated channel, is very common in some California terrains, but is not evident here along the valley of the Outlet Creek. The lack of naturally occurring landslides, which is attributed to the resistance of the underlying bedrock, was an indication that the cut slope would not be particularly susceptible to failure by mass movement. Except for the bare upper part of the cut slope, the appearance of the relocated channel is not unnatural for a mountain stream in a narrow valley.

7.5.13 Nojoqui Creek at US-101 at Buellton, California (Case 13)

The lowermost 760 m of this creek was relocated to enter Santa Ynez River upstream from the US-101 bridge, for purpose of avoiding a stream crossing at an interchange (Figure 7.5.12). The performance period of 16 years (1964-1979), during which major floods occurred in 1969 and 1978, showed no evidence of degradation or lateral erosion in the relocated channel, but a sinuous low-water channel had developed in the wide bottom of the relocated channel. Severe bank erosion occurred in the natural channel at the bend upstream from the relocation during flood of 1978, but this is not attributed to the relocation.

The site is located on US-101 about 1.3 km south of Buellton. Nojoqui Creek is intermittent, with a drainage area about 39 km^2. The stream is ungaged, but an adjacent gaged stream of similar drainage area (Alisal Creek) has an average discharge of 0.15 m^3/s, with no flow 64 percent of the time. The channel width is 15-20 m, bank height to flood plain, 1.5 m, and channel slope at site 6.9 m/km. This point-bar braided stream is generally incised into terraces. Tree cover along the channel is less than 50 percent. Bed material is gravel, cobbles, and small boulders; bank material is moderately cohesive silt, clay and gravel.

The lowermost 760 m of natural channel was relocated into a straight artificial channel 640 m in length, resulting in a length change factor of 0.84. The width of the natural channel was in the range of 15-20 m. The relocated channel has a top width of 42 m, a bottom width of 31 m, and is bounded by riprapped dikes that rise about 1.5 m above the flat bottom. Slope of the natural channel was 4.4 m/km; of the relocated channel, 5.3 m/km, decreasing to 2.5 m/km at the lower end. The dikes bounding the relocated channel are riprapped in part with large (1 m) rock and in part, with 0.6 m rock, with the toe of riprap extending to a depth of 1.8 m below the channel bottom.

Floods having an estimated recurrence interval greater than 25 years occurred in 1969 and 1978. The banks of the natural channel at a bend upstream from the relocation were severely eroded in 1978, and the channel was subsequently realigned by the bulldozing of bed material against the banks. The flood apparently disrupted the riprap facing of dikes along the relocated channel, but there is no evidence that the dikes were broken. Young willows and other vegetation have become established along the dikes and, locally, in the channel bottom.

Because the bottom width of the relocated channel is more than twice that of the natural channel, a sinuous low-water channel has developed, which may eventually erode laterally against the bounding dikes. In addition, the wide bottom may become overgrown with willows, which will impair its transmission of floods.

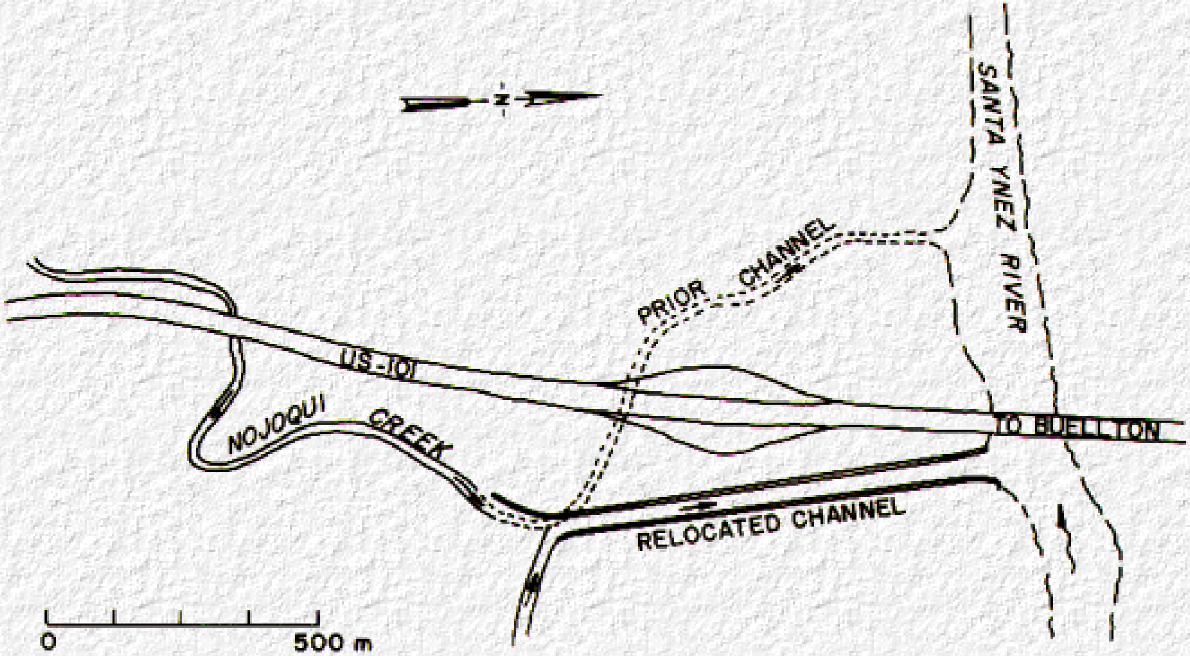


Figure 7.5.12. Plan Sketch of Nojoqui Creek Channel Relocation

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7.5.14 Turkey Creek at I-10 Near Newton, Mississippi (Case 14)

A reach, 625 m in length, was relocated and thereby shortened to 270 m, for the purpose of improving the channel alignment at the bridges and to accommodate the planned roadway location (Figure 7.5.13). The performance period was 15 years (1964-1979). At the U.S. Geological Survey gage on nearby Chunky River, major floods occurred in April 1974 and January 1975. As specified in the plans, the relocated channel was trapezoidal in cross section, with a bottom width of 9 m, and a side slope of 2:1. Channel slope as measured on the topographic map is about 1.6 m/km. No bank protection measures were applied. In 1979, the bottom width of the relocated channel between the interstate bridges was in the range of 3 - 4 m, resistant coherent clay was exposed in the channel bottom, and the banks were stable. The natural channel was generally stable (although choked with debris) upstream from the relocation; but downstream the bank was eroded and unstable at the outside of banks where, bordered by a pasture, the bottom width was in the range of 9 - 11 m. Bank instability at this point was more severe than in 1955, but causes other than the relocation may have contributed to this.

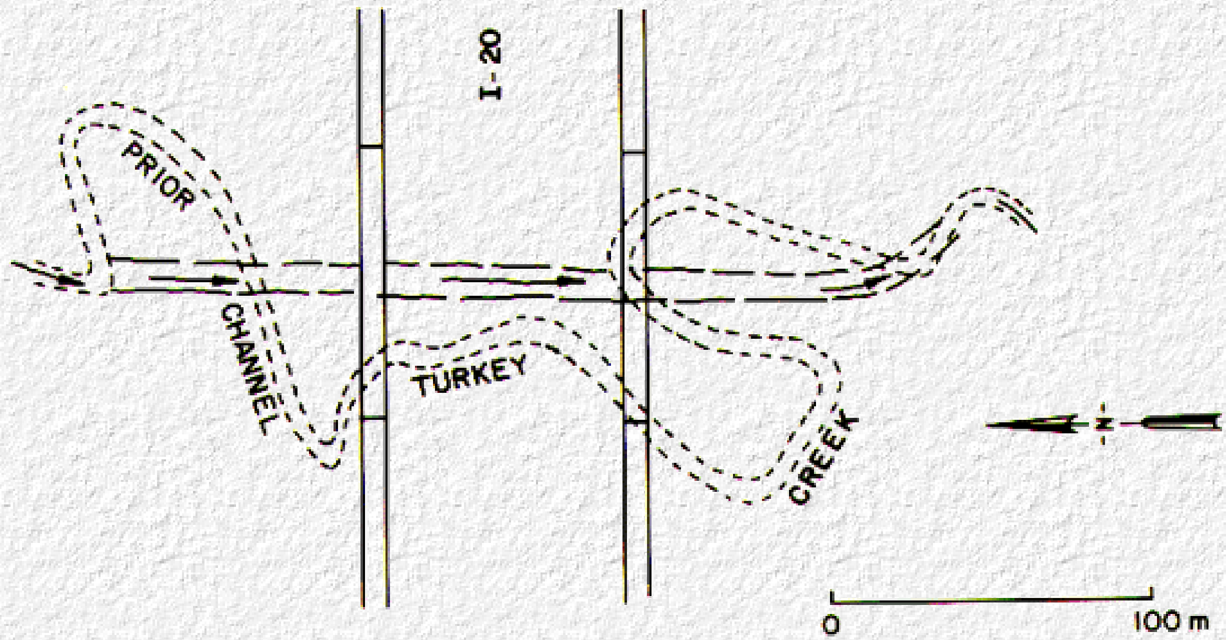


Figure 7.5.13. Plan Sketch of Turkey Creek Channel Relocation (Case 14)

Stability of the relocated channel, which is substantially more narrow than specified in the plans and also more narrow than the natural channel, is attributed to the resistant clay in the bed and lower banks. Evidently because of mowing of the median area, no trees have become established along the bankline.

7.5.15 Gravel Mining on the Russian River, California (Case 15)

It is essential to monitor and manage sand and gravel mining so as not to induce undesirable instabilities in the river system. In most cases, removal of sand and gravel has caused deepening and widening of the channel. These wider, deeper reaches act as sinks for the sediment loads and they may trap the finer clays and silts, altering the river environment. An interesting example of rivers where excessive removal of sands and gravels has caused significant changes is in the vicinity of the confluence of Dry Creek with the Russian River. The location map is shown in Figure 7.5.14a. The tributary (Dry Creek) enters the river just downstream of a small dam on the Russian River. Previous sand and gravel extraction in the tributary has not exceeded the calculated safe yield, however, extraction rates in the middle reach of the river significantly exceeded the safe yield for the period 1951 to 1964. This excessive extraction of sand and gravel induced a headcut that progressed upstream along the tributary, as shown in Figure 7.5.14b and Figure 7.5.14c. This headcut was curtailed by a rock outcrop acting as a control point near a bridge approximately seven miles upstream of the confluence and essentially stabilized around 1972-73. During the years 1946 to about 1955, the tributary channel widened (Figure 7.5.14d). Figure 7.5.14e shows the corresponding stage discharge relationships of the Russian River at the mouth of Dry Creek. Although in this example both rivers are bar-braided systems, they indicate very well the magnitude of possible adverse consequences on meandering streams.

7.5.16 Nowood River and Ten Sleep Creek Confluence, Wyoming (Case 16)

During a site visit a unique situation was uncovered at the confluence of Ten Sleep Creek and the Nowood River in the Big Horn Basin of Wyoming. While investigating the site it was noted that the Nowood River had become unstable and considerable meandering activity was posing a threat to a rancher's numerous hay meadows. There was considerable evidence that the rancher had constructed and armored several cutoffs as well as armored incipient bendway activity to try and protect his meadows. The river had unstable banks and displayed degradation of the load: obviously the river was in an unstable regime.

The rancher was contacted and could offer no explanation for the hyperactive bendway activity. He did say that prior to 1935 the Nowood River had been stable. Notably that was the same year a highway agency had constructed the new bridge across the Nowood River. The rancher, in passing, said the 1935 bridge had replaced two bridges; one on the Nowood River and one on Ten Sleep Creek: See Figure 7.5.15. The indiscriminate channel change employed to economize by constructing only one bridge had pushed the Nowood River past a stability threshold. The result was an unstable reach.

7.5.17 Middle Fork Powder River, Wyoming (Case 17)

Significant and rapid erosion occurred immediately downstream from the new bridge across the Middle Fork Powder River at Kaycee, Wyoming. The erosion is primarily bank migration in the bendways and threatens to cause a meander cutoff: See Figure 7.5.16. This cutoff would destroy the community's rodeo grounds.

The community blamed the highway agency's new bridge, claiming the bridge piers improperly directed flows into the downstream bendway. The highway agency's hydraulic engineers did not agree, as the new bridge was larger and more efficient than the previous structure. Aerial photos of the river reach were obtained dating back through many years. New photos were obtained showing the river's present planform. Together these photos displayed a planform history. These photos had been taken several years apart through this period by different agencies for various reasons.

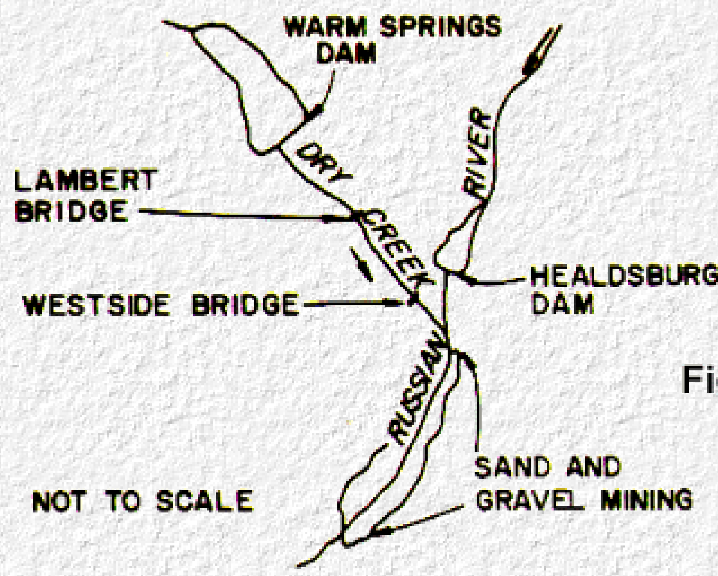
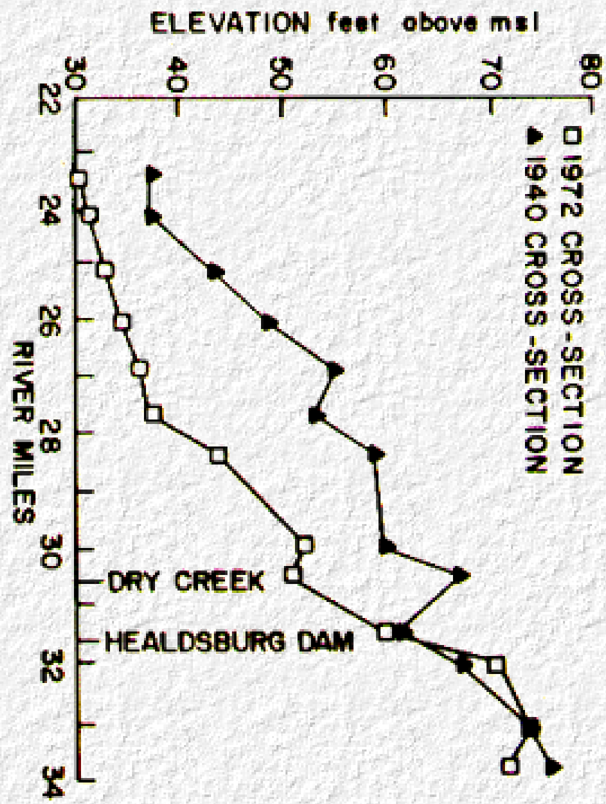


Figure a

Figure b



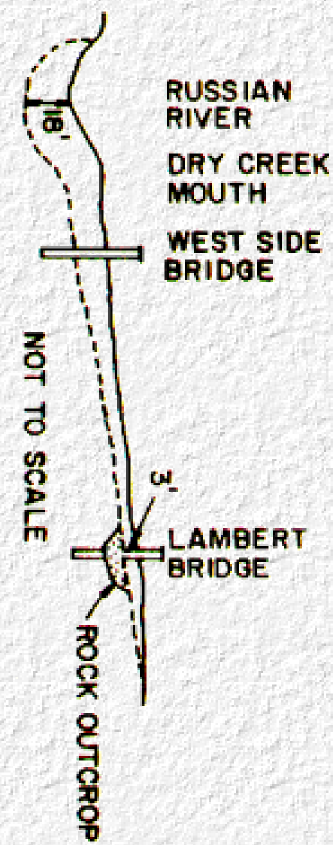
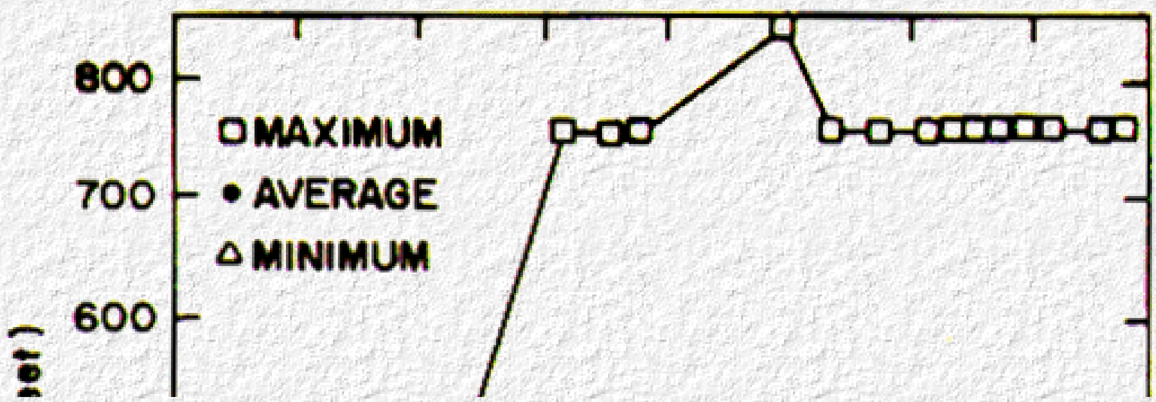


Figure c



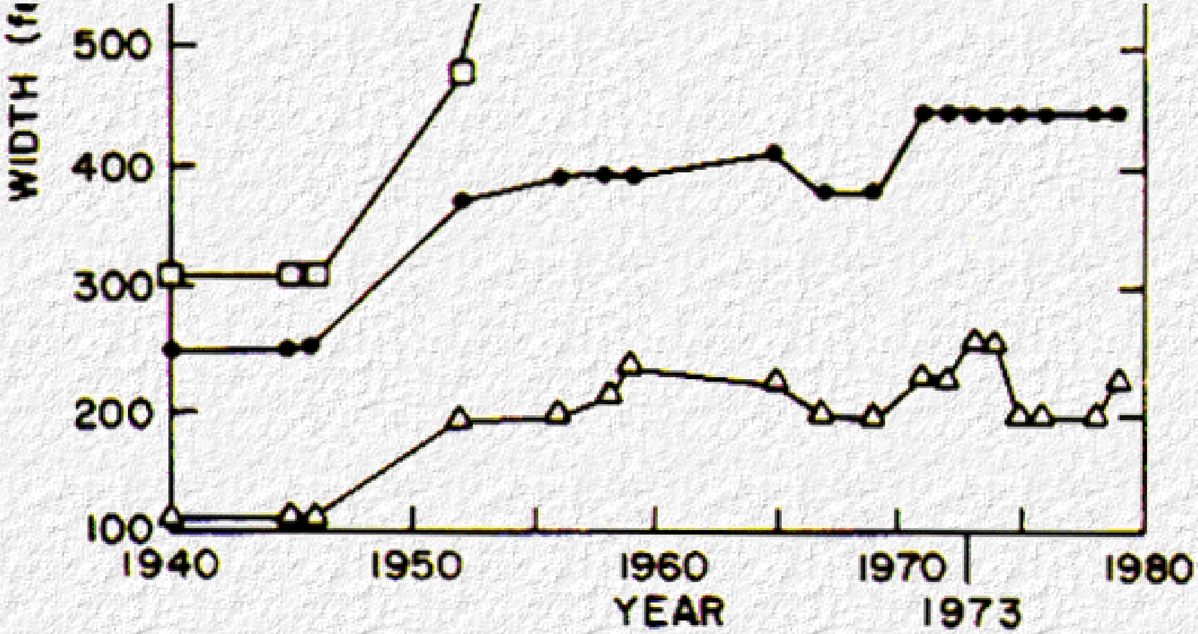
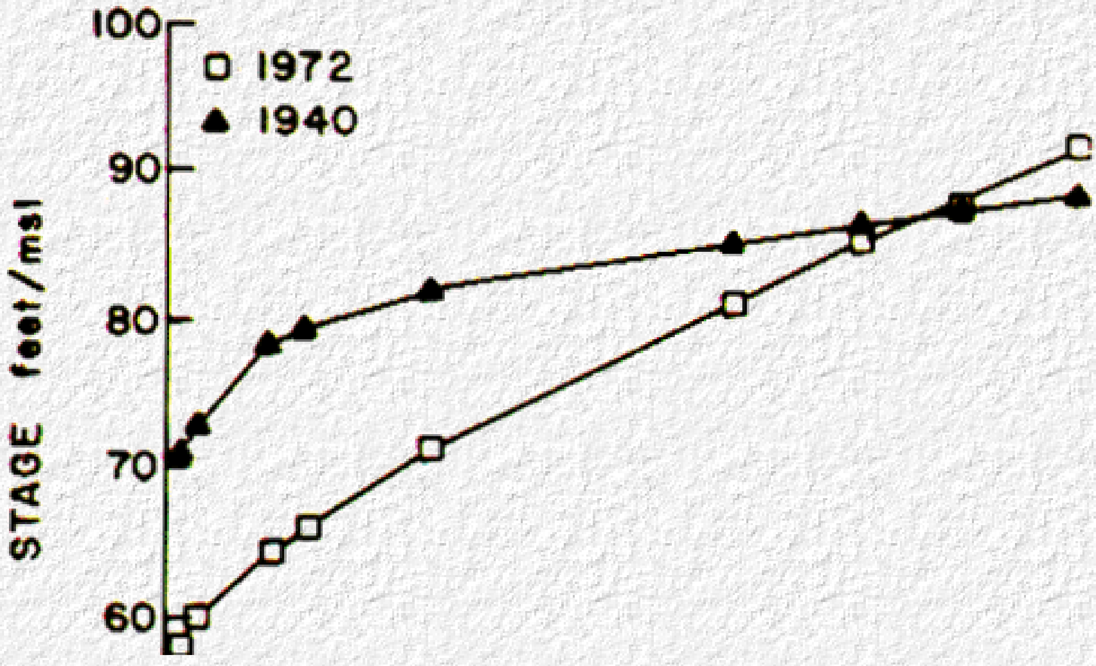

WIDTH (ft
500
400
300
200
100
1940
1950
1960
1970
1980
1973
YEAR

FIGURE d


1972
1940
100
90
80
70
60
STAGE feet/msl

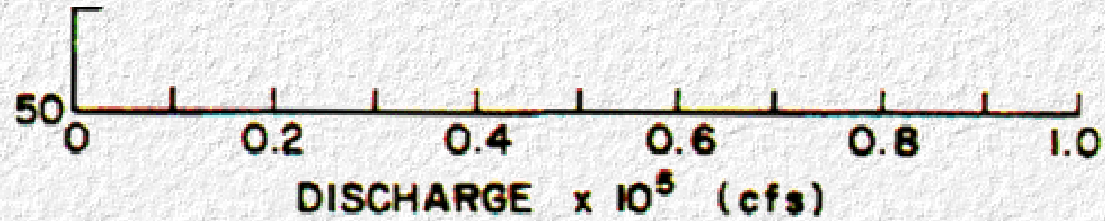
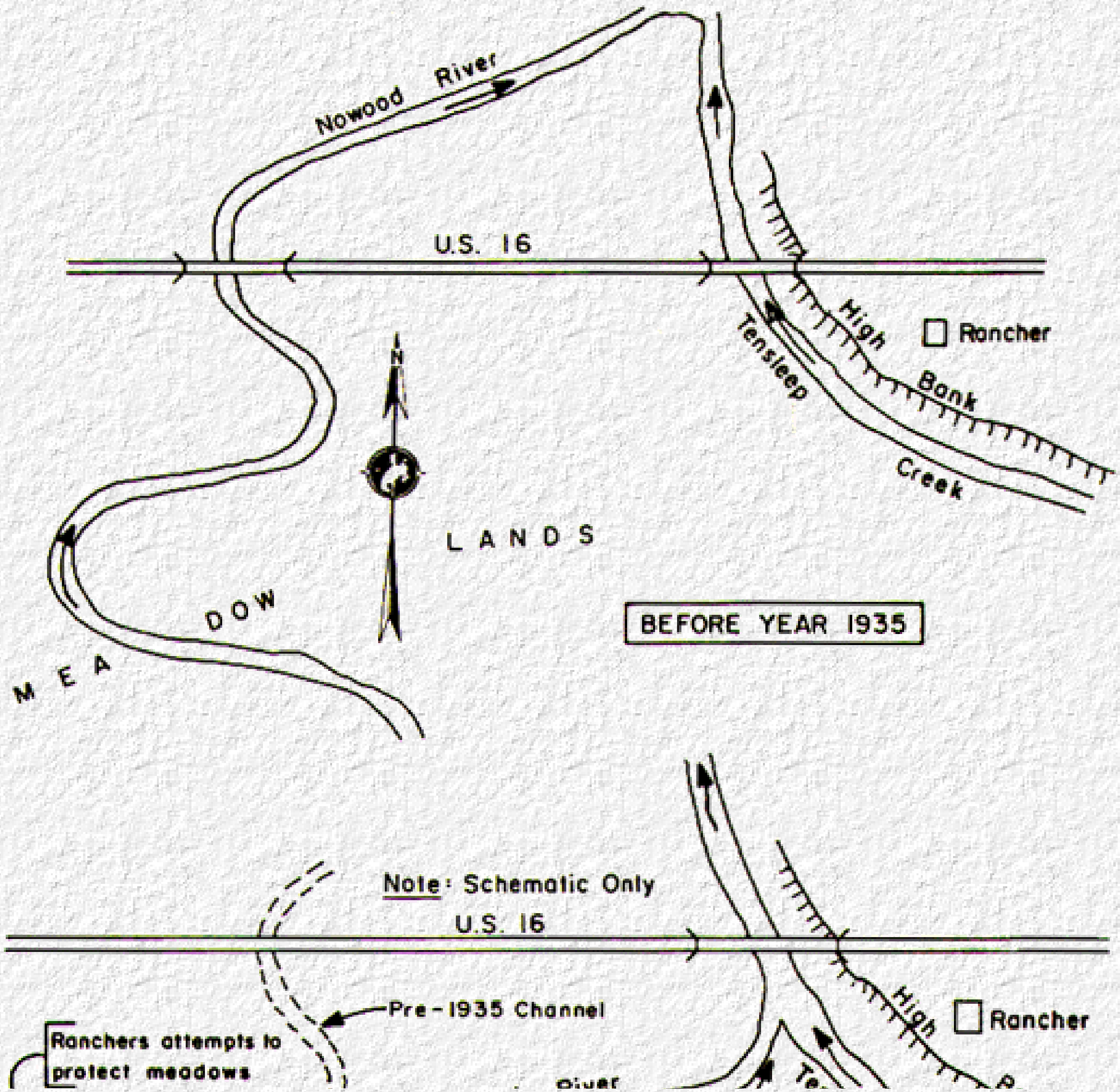


FIGURE e

Figure 7.5.14. Case Study of Sand and Gravel Mining (Case 15)



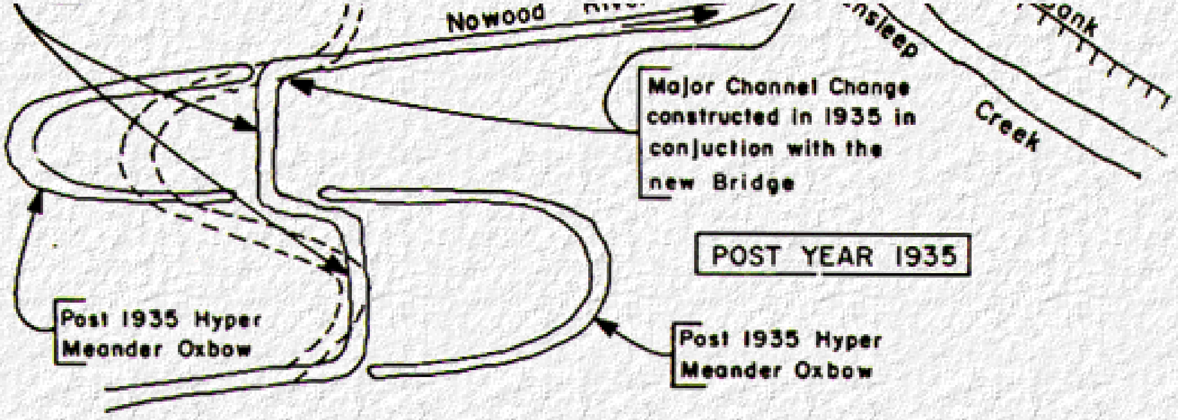


Figure 7.5.15. Nowood River near Ten Sleep, Wyoming (Case 16)

The river was noted to be relatively stable until such time as when a downstream rancher had constructed a major cutoff to gain additional pasture land on the side of the river where his operations were located. Unfortunately the bridge was constructed about the same time as the cutoff was completed. However, the evidence was overwhelming in attributing the sudden instability to the rancher's channel change. These findings were presented to the community of Kaycee and the complaints ceased.

The erosion problem immediately downstream from the bridge is expected to continue. Should the community fail to forestall the cutoff, the rodeo grounds will be destroyed and the bridge will be in jeopardy from potential headcutting, as will an upstream trailer park.

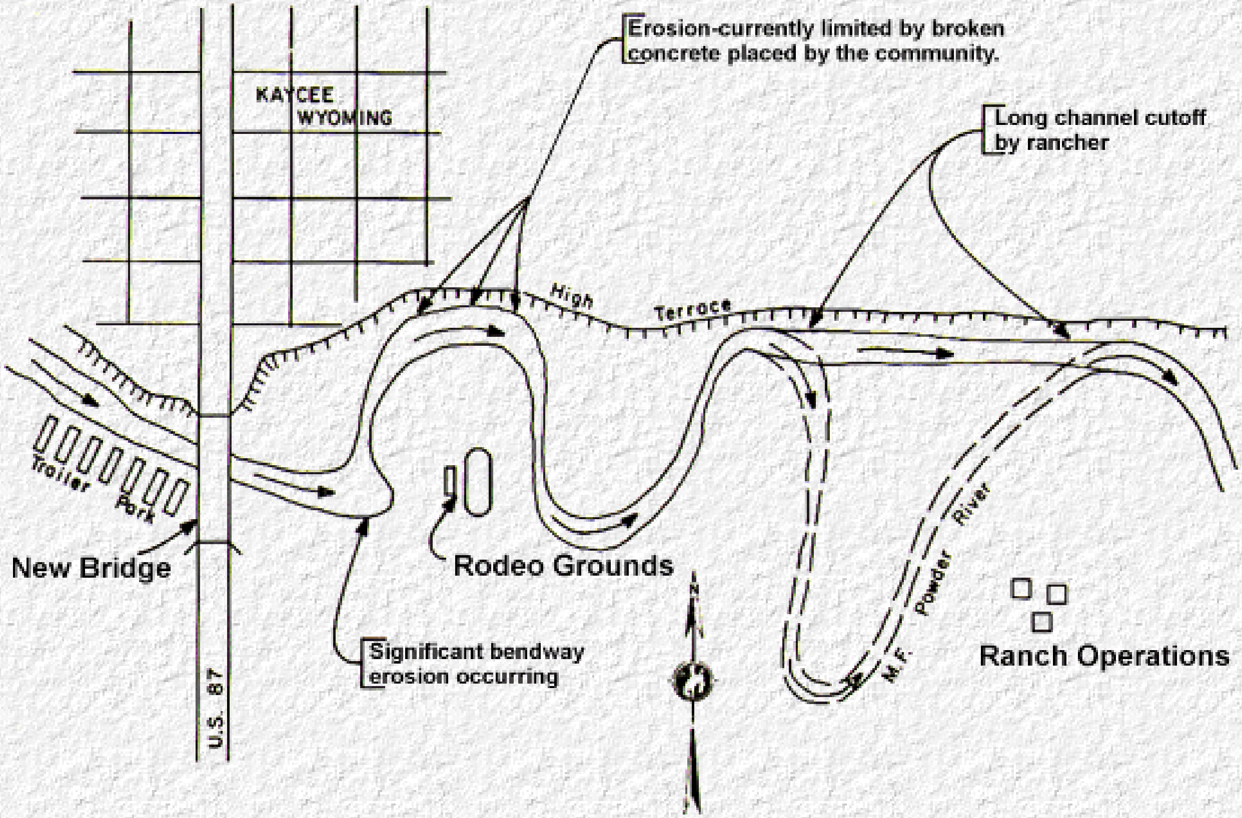


Figure 7.5.16. Middle Fork Powder River at Kaycee, Wyoming

7.6 Overview Example Application 1

In the following sections of the chapter, examples are given showing the application of the principles, methods and conceptions of previous chapters. The examples contain situations where design is determined by well established numerical procedures and also situations where design depends heavily on the judgment of the engineer. It would be wrong to treat these examples as approved design procedures. They are not intended to be examples of how a particular design problem is handled but rather as examples of how the concepts previously outlined in this manual find their application in design. For these reasons, <u>this section should be read and studied as an illustrative unit and not as a collection of individual design problems from which one can choose the correct prescription for the problem at hand. River problems are much too complex for a cookbook approach and it is hoped that the examples make this evident.</u>

7.6.1 General Situation

Overview example applications 1, 2 and 3 relate to the design of a crossing on the hypothetical "Mainstream River" shown in Figure 7.6.1. The flow is from top to bottom. An existing highway crossing can be seen in Figure 7.6.1 and an upgraded highway alignment is shown on the same figure. The old crossing is to be preserved for local travel. From first appearances, this proposed crossing seems to be located in a very unstable section of the river and more attractive locations are possible. But, assume that there are factors other than those associated with the bridge location that make this alternative worthy of scrutiny. Such other considerations actually dictate the location of many crossings.

There is a USGS gaging station several miles upstream of the crossing site with no intervening tributaries and only one minor diversion for irrigation. According to 43 years of record, the mean annual flow is 2900 cfs. Figure 7.6.2 is a hydrograph for the water year that included the flood of record. The summer flows are typically low, less than about 500 cfs, while the winter flows are much higher, punctuated by flood peaks lasting several days. The record flood overflowed the banks and inundated a considerable expanse of the extensive and flat floodplain. The peak instantaneous flow for this flood was 97,000 cfs, while the average flow for that day, shown on Figure 7.6.2, was only 77,000 cfs. Flood peaks are relatively short lived on this river. The daily flows of Figure 7.6.2 are replotted in Figure 7.6.3 in the form of the flow duration curve for one year. During that year, the flow exceeded the mean flow of 2900 cfs approximately 30 percent of the time. A flow of 1250 cfs was exceeded 50 percent of the time.

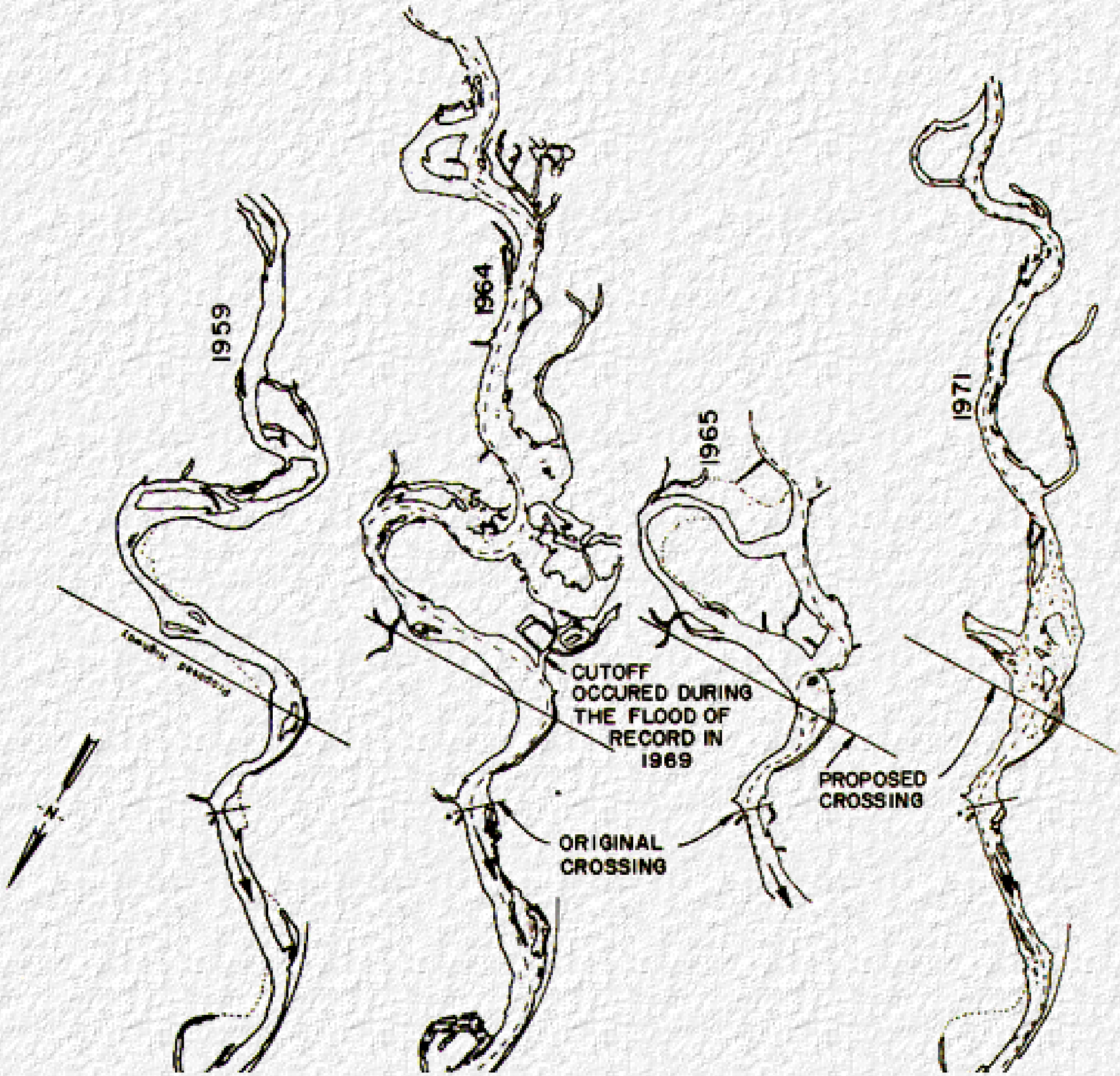


Figure 7.6.1. Recent Alignment Changes of Mainstream River

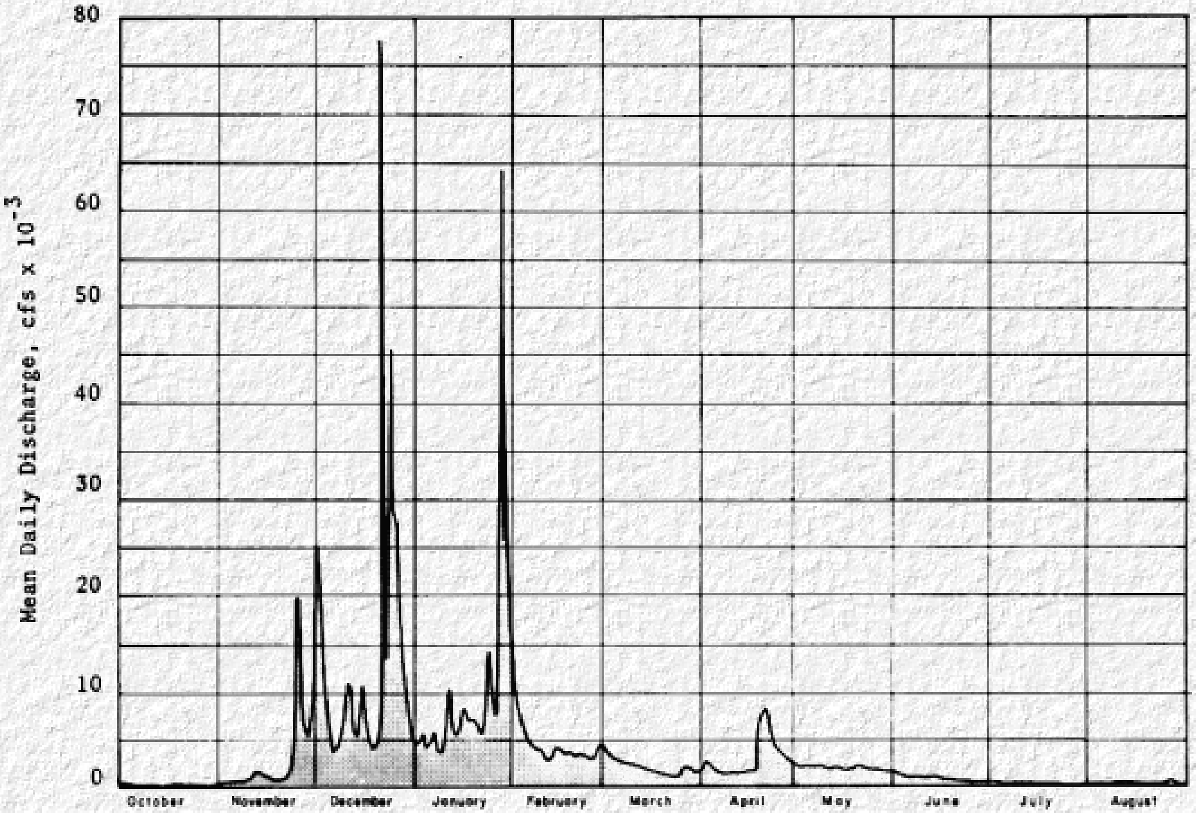


Figure 7.6.2. Hydrograph from Gaging Station on Mainstream River 12 Miles Upstream of Proposed Crossing

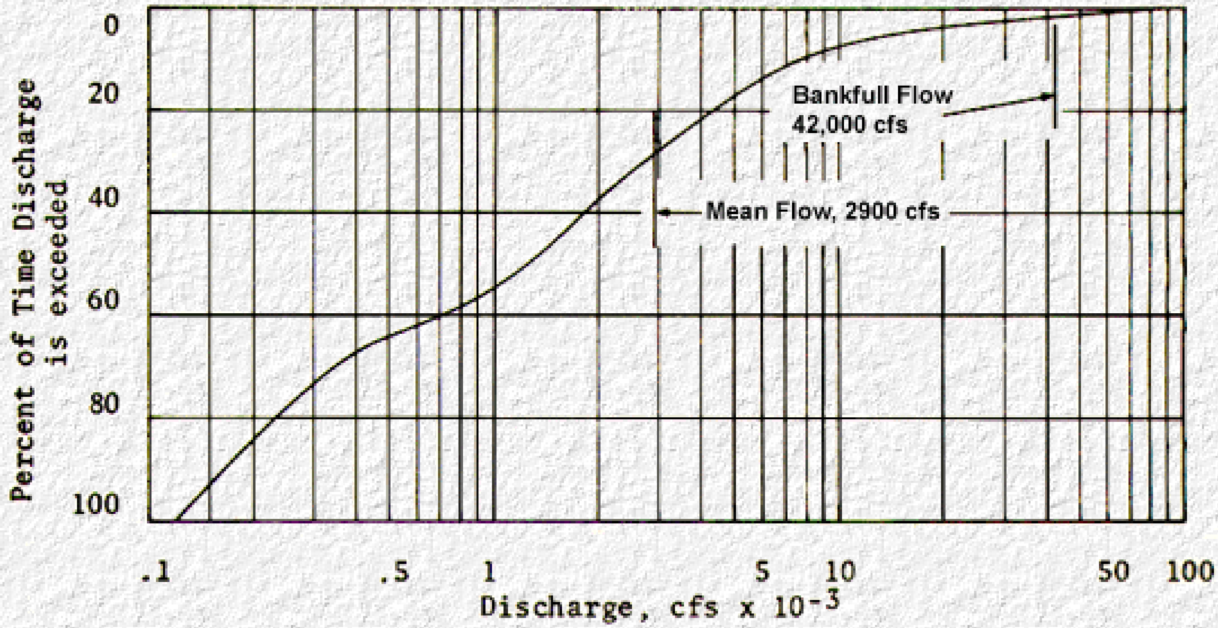


Figure 7.6.3. Flow Duration Curve for Mainstream River

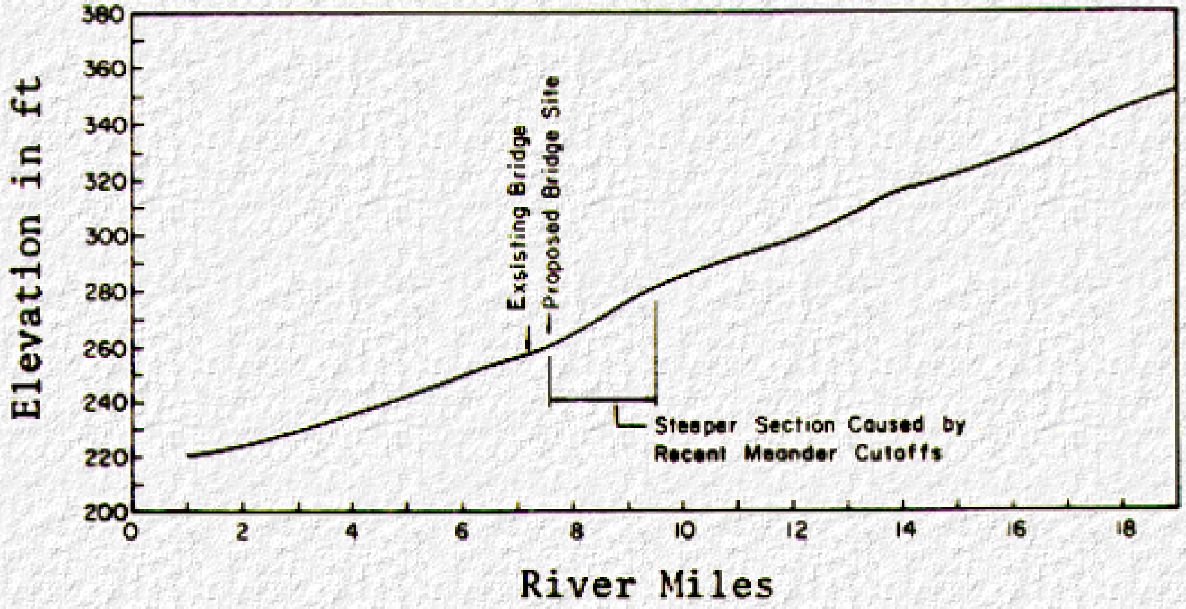


Figure 7.6.4. Profile for the Mainstream River

From the discharge data at the USGS gaging station several miles upstream the flood of record at the proposed crossing was estimated at 105,000 cfs and reached a stage of El. 272 in the vicinity of the crossing. The water levels on the floodplain went as high as El. 275, however, indicating that the channel flow and the floodplain flow are poorly connected in this area. Field estimates were 64,000 cfs channel flow and 4l,000 cfs overbank flow. The bankfull discharge for this river is about 42,000 cfs.

The floodplain has an overall slope of 0.00191 (10.09 ft/mile) in the direction of flow and the river is relatively unrestricted in lateral migration except at the localized revetment protections. Figure 7.6.4 shows a profile of the river with a steeper section just upstream of the proposed crossing. The river has an average slope of 0.00138 (7.29 ft/mile). In this reach, the river is classified as very mature. One question regarding this river is: what discharge dominates in defining the character of the river's morphology? As seen from the hydrograph of Figure 7.6.2 the flows are much higher than the mean annual flow of 2900 cfs in the winter and very much lower in the summer. The low summer flows contribute little to the morphology. The record flood, on the other hand, created an anomaly that only time will restore to equilibrium. An experienced river engineer might well consider the mean flow for the five winter months, approximately 7000 cfs, as representing the dominant discharge for this river. Applying the river slope of 0.00138 ft/ft and the mean discharge of 7000 cfs to Figure 4.4.3 of Chapter 4, the characteristics of this reach borders on the braided zone. Meanders are however, in evidence in the outlines of Figure 7.6.1. The braiding, particularly at the crossing site, appears to be a temporary condition caused by the steepening of the local slope due to cutoffs. The sinuosity of the river, 1.38, is low, more like that of a straight river. The bed material has a median size of 1.0 mm typical of coarse sandbed rivers. The D_{90} is about 15 mm. The USGS has recorded a sediment discharge of 223,000 tons/day during a flow of 41,300 cfs.

Aerial photographs of this stretch of the river date back as far as 1959 and Figure 7.6.1 shows outlines of the water's edge at four different dates. These outlines dramatize the activity of the river, particularly in the vicinity of the planned crossing. The convoluted meander existing in 1959 was cut off during the flood of record in l969, shortening and steepening this stretch of the river considerably. The cutoff was formed as a result of surface erosion when the flood overtopped the banks. Most of the large quantities of sediment removed in the formation of the cutoff deposited immediately downstream where the river rapidly aggraded. The result of the cutoff is a section of the river steeper than the rest. The river is slowly degrading its upper end and aggrading its lower end to restore its normal slope. As this process continues, the meander character of the river is once more taking over. The cutoff has created a localized reach that is highly unstable and this reach will lengthen upstream and downstream before equilibrium is restored.

7.6.2 Flows, Stage and Stability

In this example, a bridge crossing on the Mainstream River at the alignment shown in Figure 7.6.1 is discussed. The bridge is to be designed to pass the 100-year flood. The bridge will extend across 500 ft of the channel as shown in Figure 7.6.5. One abutment will extend into the main channel. The critical features of the design as far as river mechanics are concerned are as follows.

Design flows - The design flow selected for the bridge is 110,000 cfs which corresponds to the 100-year flood. From estimates reported earlier on the flood of record, this can be divided into 66,000 cfs channel flow and 44,000 cfs overbank flow.

Design stage - A stage of 272 ft was recorded in the field for the 105,000 cfs flood and this is approximately the design flood. This actual measured value of stage is superior to any that could be calculated by backwater methods.

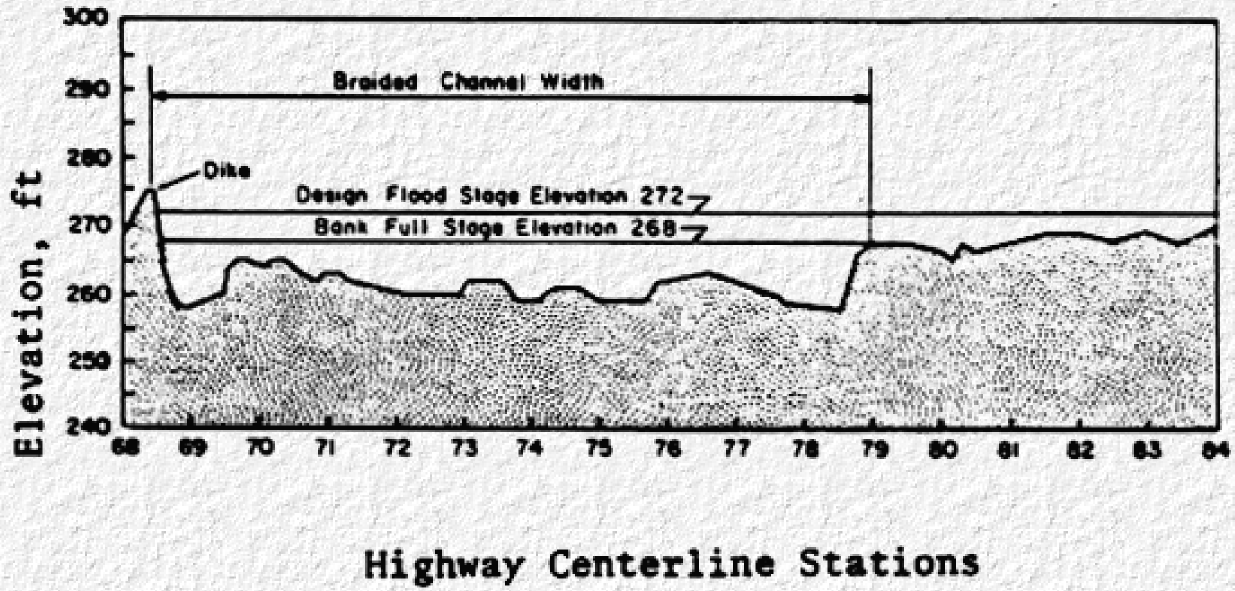


Figure 7.6.5. Cross Section of the Crossing on Mainstream River

River stability - Historically, the river has been very unstable and the recent cutoff just upstream of the crossing has produced an especially active situation. The braided section of the river is expected to revert to a meandering section in a matter of a few years and, indeed, is showing this tendency now. Figure 7.6.6 shows a prediction of future alignment changes as the river restores the sinuosity lost with the cutoff and as existing meanders migrate downstream. The tendency is for an attack on the right abutment and for the flow to approach the bridge obliquely from the right side. As the meander tendency is restored the channel will become narrower and deeper, even under the bridge crossing. Some parts of the bed will scour and some will fill. To determine the maximum depth of scour one can examine the river and determine visually a natural minimum width. As we anticipate the river will reform to its normal width of 500 ft in the future, a 500 -ft bridge opening is all that is required in the main channel. The wide braided reach is an anomaly which will disappear as the river meander redevelops. The design unit discharge (discharge per foot of width) for the estimated 66,000 cfs in the 500-ft wide channel is therefore 132 cfs/ft. Manning's equation can then be written in terms of unit discharge and solved for depth. That is,

$$y_o \simeq \left\{ \frac{qn}{1.486\, S_f^{1/2}} \right\}^{3/5}$$

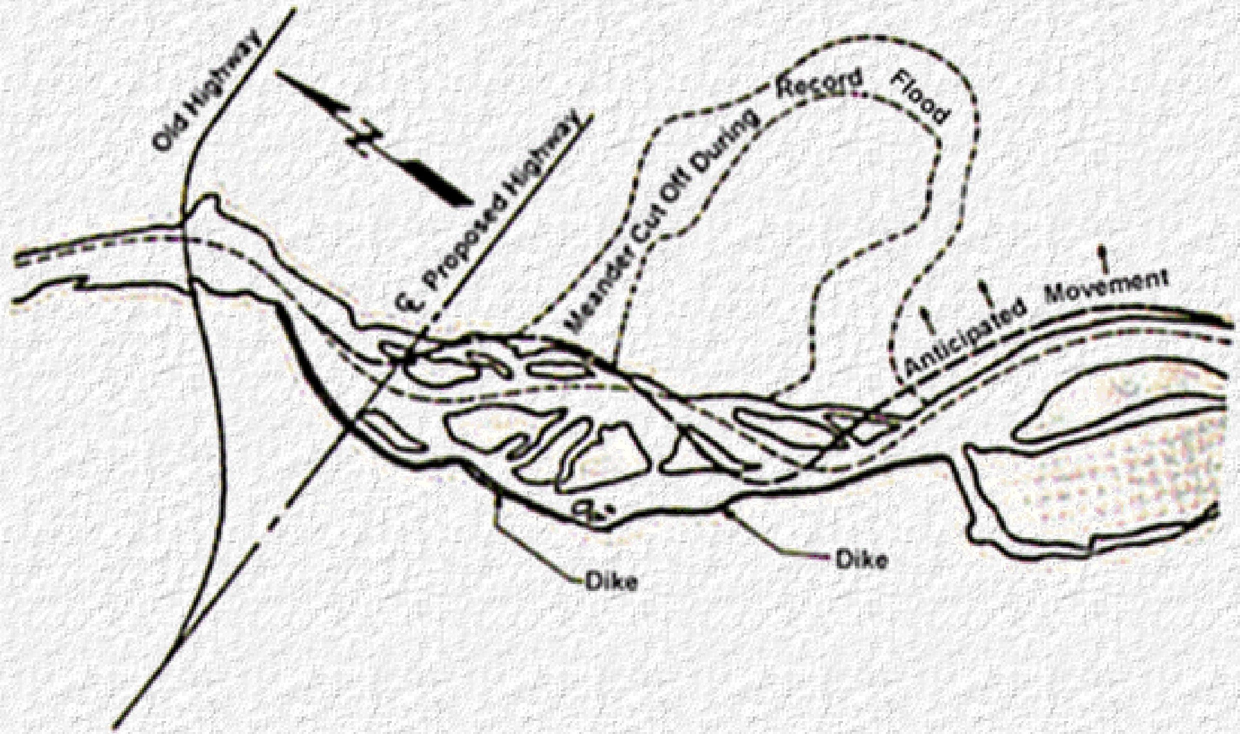


Figure 7.6.6. Existing and Anticipated Channel Alignments for Mainstream River

In Table 7.6.1 and in later calculations it is assumed that the depth is approximately equal to the hydraulic radius which is an acceptable assumption for wide rivers. The friction slope S_f can be assumed equal to the river slope 0.00138. As a first approximation, assume Manning's n value of 0.025 by referring to Table 2.3.1. Also, from Equation 3.4.2a and the D_{50} of the bed material equal to 1.0 mm Manning's n is 0.015; from Equation 3.4.2b and the D_{90} of the bed material equal to 15 mm Manning's n is 0.021 and Figure 7.6.1 indicates large bars in the channel which won't completely wash out (transition flow conditions). Thus, we expect Manning's n to be larger than for only grain roughness. Calculating normal depth

$$y_o = \left\{ \frac{(132)(0.025)}{(1.486)(0.00138)^{1/2}} \right\}^{3/5} = 11.6 \text{ ft}$$

and

$$V = \frac{q}{y_o} = \frac{132}{11.6} = 11.4 \text{ fps}$$

The average shear stress on the bed is (from Equation 3.5.12 with $R = y_o$)

$$\tau_o = \gamma R\ S_f = (62.4)\ (11.6)\ (0.00138) = 1.0 \text{ psf}$$

and the stream power is

$$\tau_o V = (1.0)\ (11.4) = 11.4 \text{ ft lb/sec/sq ft}$$

For 1 mm sand and a stream power of 11.4 ft lb/sec/sq ft, Figure 3.4.4 indicates that the flow should be in the upper flow regime. This bed form does not check our assumption that the flow is in transition. However, because the gravel bars won't completely wash out and because of the D_{90} we will still use n = 0.025.

The stage for the design discharge is 272 ft so the average bed level is approximately 260 ft for the flood discharge.

The Froude number for the channel at flood discharge is

$$F_r = \frac{V}{\sqrt{g y_o}} = \frac{11.4}{\sqrt{(32.2)(11.6)}} = 0.59$$

This value of the F_r number indicates that the bed would not plane out.

The width-to-depth ratio for the channel at the design flow is

$$\frac{W}{y_o} = \frac{500}{11.6} \simeq 43$$

Table 7.6.1 Data and Computations for Overview Example Application 1

Data:	
Valley slope, S_v = River slope, S_o =	0.0019 (10.0 ft/mile) .00138
Average discharge = Dominant flow - Average five-month winter discharge = Bankfull discharge = Record discharge: Channel = Overbank =	2,900 cfs 7,000 cfs 42,000 cfs 64,000 cfs 41,000 cfs 105,000 cfs

Design discharge: Channel = (100 year) Overbank =	 66,000 cfs 44,000 cfs 110,000 cfs
Design flood stage = Minimum channel width, W = Mean bed-material size, D_{50} =	272 ft 500 ft 1 mm
Conditions at Design Discharge:	
Unit discharge in channel, q = Manning's n = Average flow depth, y_o =	132 cfs/ft 0.025 11.6 ft
Average velocity, V = Channel Froude number, Fr = Average bed shear, t_o =	11.4 fps 0.59 1.0 psf
Average stream power, t_oV = Width-to-depth ratio, W/y_o = Flood stage elevation =	11.4 ft lb/sec/sq ft 43 272 ft
Conditions In Meander Pool:	
Maximum depth in pool, y_{max} = Bed elevation in deep part of pool: Average velocity in the pool: Froude number in the deep pool, Fr =	29 ft 243 ft 14.2 fps 0.46
Embankment Riprap for Present Conditions	
Angle of repose, ϕ = Side slope angle, θ = Shear stress on side slope, t_o =	37° 18.4° 1.8 psf
Specific weight of riprap, S_s = Safety factor, S.F. = Stability number, η = Effective grain size, D_s =	2.50 1.5 0.356 13.5 in
Recommended median size, D_{50} = Recommended maximum size D_{100} = Minimum thickness of riprap =	11 in 22 in 22 in
Pier Scour For Present Conditions:	
Width of round nose pier, a = Length of pier, ℓ = Skew angle = Approach Froude number, Fr_1 =	5 ft 20 ft 30° 0.59
Approach flow depth, y_1 = Maximum depth of scour, y_{smax} =	11.6 ft 28 ft
Pier Scour for Future Conditions:	

Skew angle = Approach flow depth, y_1 =	30° 29 ft
Approach Froude number, Fr_1 = Maximum depth of scour, y_s =	0.46 34 ft

This, supports the assumptions of a wide channel, i.e., $W/y_o > 10$. A summary of the calculations are given in Table 7.6.1

The depth computed above is an average depth. Deeper sections exist near the outside of meander bends. These pools can and will exist under the crossing. The anticipated depth of flow in the meander bend must be considered in design. At this point field measurements would be appropriate.

Suppose that the field measurements indicate a y_{max} of 29 ft. The elevation of the bed in the deep part of the pool is then

272-29 = 243 ft

The velocity in the bend will be greater than in the crossing. An estimate of this velocity can be made from the information given in Figure 2.6.2. For a long bend in a parabolic channel, the maximum velocity in the bend is only about l0 percent greater than the maximum velocity in the straight approach section. For most design problems, it is recommended that a 25 percent increase in the average velocity be used for the maximum velocity in the bend. Then, in the bendway

V = (1.25) (11.4) = 14.2 fps

and the Froude number in the deep part of the pool is

$$Fr = \frac{14.2}{\sqrt{(32.2)(29)}} = 0.46$$

A decision must be made regarding the substantial overbank flow associated with the design flood. If the highway grade is placed above the levels of the overbank flow, the entire flow would be forced under the crossing. The increase in the flow in the main channel would be 67 percent at the design discharge. Such an increase will surely magnify the flow problems downstream. The 44,000 cfs of overbank flow would have to flow laterally across the floodplain to return to the river upstream of the crossing. This is certain to increase the depth of inundation and worsen the flooding. If the highway were placed low enough not to obstruct the overbank flow, the roadbed would be flooded whenever the flow exceeded the 42,000 cfs bankfull flow or about every two years. One solution would be to provide openings under the highway (relief bridge) to handle this flow. These would have to be extensive but it is assumed in this example that they have been provided. Overview Example Application 2 examines the case where the total flow goes through the bridge.

7.6.3 Abutment Protection

The river alignment changes indicate the need for protection on the right embankment of the existing crossing as shown in Figure 7.6.7. The left abutment already has a protective dike that has withstood flows in excess of 100,000 cfs.

A guidebank is ideally suited for this embankment end. While this reach of river is in the braided form, the guidebank will help direct the flow from the right side of the channel through the bridge opening. After the river has narrowed, the guidebank will protect the embankment by holding the meander loops away from the embankment. In addition the guidebank will move the local scour away from the embankment end under the bridge to the nose of the spur.

The recommended guidebank configuration is shown in Figure 7.6.8. Here, a length of 200 ft is chosen because the guidebank will have to direct a substantial amount of channel flow while the approach river channel is braided. The guidebank bends outwards on a 1/4 ellipse which is terminated at (0.4) (200) = 80 ft back from the embankment end. The plan view of the spur is shown in Figure 7.6.8.

The top of the guidebank is set at El. 275, three feet above design stage. This grade elevation allows one foot for aggradation and two feet for wave wash. The toe of the riprap protection is set at El. 260, the anticipated average bed level for a design flood. This elevation is above the low summer flow stage so that construction would not require cofferdamming. Figure 7.6.7 shows areas of local scour to El. 250 in the natural river and computations mentioned above indicate scour in meander pools to reach El. 243. To protect against localized scour to these elevations, a rock-filled trench of riprap has been provided (see Figure 7.6.8) that will stop any local pockets of scour.

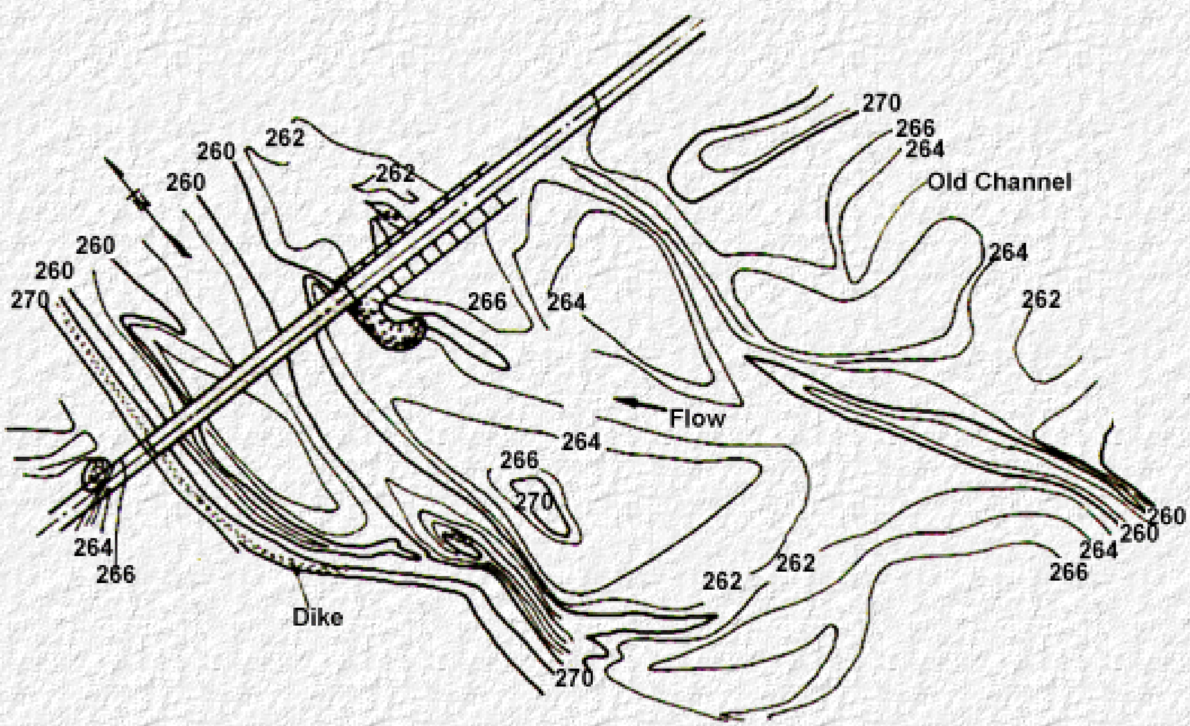


Figure 7.6.7. Right Embankment and Guidebank

The size of riprap required for the spur can be determined by the method in Chapter 5. The results of the computations are summarized in Table 7.6.1. As the embankments for this bridge will not constrict the river flow after the river narrows (relief bridges are provided on the floodplain), the flow through the bridge would not accelerate significantly and only the case of horizontal flow on a side slope is considered.

The side slope angle along the bankline is set at 3:1 to prevent slip circle failures in the soil behind the riprap on the guidebank. Therefore θ = 18.4°. The anticipated size of riprap is in the range between 1.0 and 10 in. Then, from Figure 3.2.4 the angle of repose for the dumped riprap will be approximately 37° or ϕ = 37°. For flow alignment as shown in Figure 7.6.7, the average shear stress on the bed

$$\tau_o = \gamma\, y_o S_f = (62.5)\,(11.6)\,(0.00138) = 1.0 \text{ psf}$$

where y_o is equal to R for wide floodplain crossing. However, the side slope shear stress is not equal to the bed shear stress. To determine the shear stress on the side slope riprap the method given in Chapter 5 can be used.

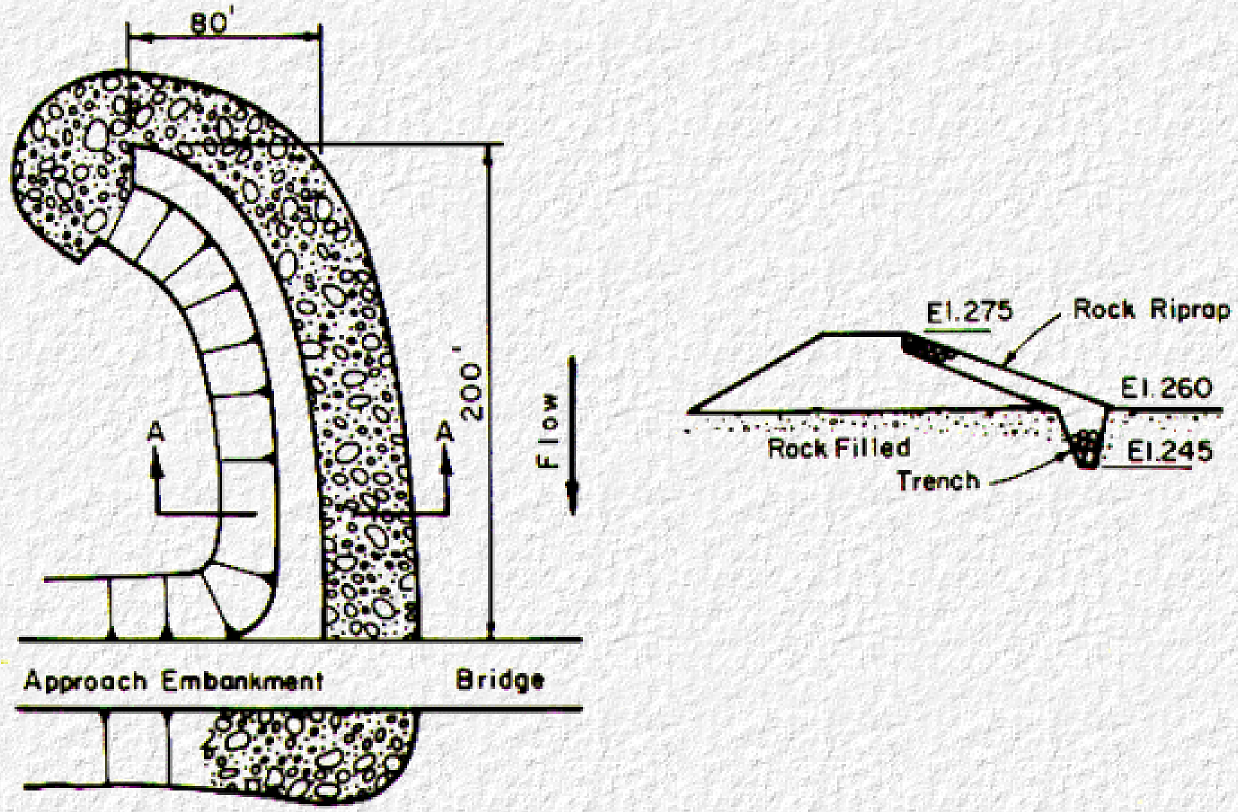


Figure 7.6.8. Spur Dike with a Rock Trench

The mean velocity of the flow is 11.4 ft/sec. The velocity relative to the stone V_r for the conditions assumed would be some percentage smaller. Assuming that V_r is 0.7 of the mean velocity and using Equation 5.2.16

$$\tau_o = \frac{\rho V_r^2}{72}$$

$$\tau_o = \frac{1.94(8.0)^2}{72} = 1.72\,\text{psf}$$

In this region, the rock available for riprap has a specific weight S_s = 2.50. The stability number for horizontal flow along a side slope is given by Equation 5.2.12, or

$$\eta = \left\{\frac{S_m^2 - (S.F.)^2}{(S.F.)S_m^2}\right\}\cos\theta$$

Here

$$S_m = \frac{\tan\phi}{\tan\theta}$$

$$S_m = \frac{\tan 37°}{\tan 18.4°} = 2.27$$

The recommended stability safety factor for design is in Section 5.2.0 S. F. = 1.5. Then,

$$\eta = \left\{\frac{(2.27)^2 - (1.5)^2}{(1.5)(2.27)^2}\right\}\cos 18.4° = 0.356$$

The rock size is related to stability number by Equation 5.2.13 or

$$D = \frac{21\tau_o}{(S_s - 1)\gamma\eta} = \frac{(21)(1.72)}{(2.50-1)(62.4)(0.356)} = 1.08\text{ ft}$$

giving the effective rock size of the required riprap as

$D_m \simeq 13.5$ in

The recommended gradation is given in Figure 5.2.10. This gradation is such that

$D_m = 1.25\ D_{50}$

or

$$D_{50} = \frac{13.5}{1.25} \simeq 11\,\text{in.}$$

and

$D_{100} = 2D_{50}$

or

$D_{100} = (2)\ (11) = 22$ in.

The recommended minimum thickness of the riprap blanket is $2D_{50}$ or D_{100} which ever is greater. Here, use a minimum thickness of 22 in.

If a meander bend reaches the spur, the bed will scour and the maximum flow depth in the bend at the design discharge becomes 29 ft. The average velocity in this deep pool is approximately 14.5 fps. How will the riprap be affected by these flow conditions? The attack on the bank protection on the right abutment will become severe when the spur is at the outside downstream end of the meander bend. From Richardson et al. (1974) velocity relative to the riprap can also be determined from the mean velocity by:

$$\frac{V_r}{V} = \frac{3.4}{\ell n\left(12.3\,\frac{y_o}{D_m}\right)}$$

Assuming $D_m = 1.26$ ft then $y_o/D_m = 23$ and

$$V_r = 14.5\left(\frac{3.4}{\ell n\, 12.3 \bullet 23}\right) = 14.5 \times .60 = 8.7\ \text{ft/sec}$$

From Equation 5.2.16

$$\tau_o = \frac{\rho V_r^2}{72}$$

$$\tau_o = \frac{1.94(8.7)^2}{72} = 2.0\ \text{psf}$$

The stability number for the 11 in. diameter riprap (D_m = 13.5 in.) on a 3:1 side slope with a shear stress of 2.0 psf is given by Equation 5.2.5 or

$$\eta = \frac{(21)(2)}{(2.50-1)(62.4)(1.13)} = 0.397$$

and the stability factor becomes (from Equation 5.2.9)

$$S.F. = \frac{S_m}{2}\left\{\left(S_m^2\eta^2\sec^2\theta + 4\right)^{1/2} - S_m\eta\sec\theta\right\}$$

or

$$S.F. = \frac{S_m}{2}\left\{\left(\zeta^2 + 4\right)^{1/2} - \zeta\right\}$$

where

$$\zeta = S_m\ \eta\ \sec\Theta = (2.27)\ (0.397)\ (1.054) = 0.950$$

and

$$S_m = \frac{\tan\phi}{\tan\theta}$$

Then

$$S.F. = \frac{2.27}{2}\left\{\left[(0.950)^2 + 4\right]^{1/2} - 0.950\right\} = 1.44$$

As the stability factor is less than 1.5 when flow in the meander bend attacks the guidebank protection determine the size of riprap required to give a 1.5 stability factor for the riprap under a shear stress of 2.0 psf. This is found by employing Equation 5.2.12 and Equation 5.2.13 in order. Accordingly, the stability number is still

$$\eta = 0.356$$

and

$$D_m = \frac{(21)(2)}{(2.50-1)(62.4)(0.356)} = 1.26 \text{ ft}$$

or

D_m = 15 in.

which corresponds to

D_{50} = 12 in.

and

D_{100} = 24 in.

Future river alignment conditions dictate that the riprap on the abutment have a D_m of 15 in. The recommended gradation requires that the D_{50} be approximately 12 in. and D_{100} be 24 in. Other riprap gradations can be used provided the D_m is equal to or greater than 15 in.

A filter is most likely required between the riprap and the spur embankment materials. A cloth filter is recommended. It would be prudent to go out in the field during a low-flow period and check the size and condition of the riprap on the left bank. This riprap has been subjected to a large flood. The inplace riprap should be at least 6 in. (D_{50}) in diameter.

7.6.4 Scour at the Piers

The scour to be expected around the piers can be computed with the equations in Section 5.5.5, Chapter 5. For the present-day conditions at the bridge crossing the approach flow depth is

$y_1 = y_o$ = 11.6 ft

and the corresponding Froude number is (from Table 7.6.1)

Fr_1 = 0.59

The pier width a is 5 ft, and the length ℓ is 20 ft. If the pier skew angle is zero degrees, then the equilibrium depth of scour is given by Equation 5.5.25 (round nose pier) or

$$y_s = 2.0\, y_1 \left(\frac{a}{y_1}\right)^{0.65} (Fr_1)^{0.43}$$

$$y_s = (2.0)(11.6)\left(\frac{5.0}{11.6}\right)^{0.65}(0.59)^{0.43} = 10.7 \text{ ft}$$

According to the plan view on Mainstream River shown in Figure 7.6.6, a pier skew angle (to the approach flow) of 30 degrees can be anticipated. From Table 5.5.2, the scour depth would increase. For

$$\frac{\ell}{a} = \frac{20}{5} = 4$$

the multiplying factor is 2.0 and the increased depth of scour is

y_s = (10.7) (2.0) = 21.4 ft

The maximum depth of scour can be 30 percent greater (Section 5.5.5) so the maximum depth of scour is

y_s = (21.4) (1.3) $\simeq$ 28 ft

The greatest contribution to this depth is the skewed approach flow. This is one of the main penalties to pay for the lack of river control. The maximum depth of scour corresponds to a scour hole bed elevation of

272-11.6-28 $\simeq$ 232 ft

This pier scour would be obtained if the bed material were sand only. However, the scour hole could armorplate if there is sufficient cobbles in the underlying bed deposits. The depth of scour would be decreased if there is underlying bedrock. A look at the laboratory test results on the core hole samples obtained in the bridge pier foundation explorations is very important.

In Mainstream River, the bed is alluvium at least down to an elevation of 220 ft but there is a considerable amount of cobbles in the underlying bed material. The scour hole may armor plate so that the scour hole bed elevation could be above El. 232 ft. However, a larger flood could remove the armor.

Future conditions may result in a lower bed elevation around the piers. The bed elevation in the deep part of the pool has been estimated at El. 243 ft (Table 7.6.1). Probably, the bed material in such pools is greater than 1 mm sand. The bed of the pool can also armorplate during a flood.

In the deep part of the pool the approach flow has a depth (from Table 7.6.1)

y_1 = 29 ft

and the approach Froude number of

Fr_1 = 0.46

Assuming a sandbed, the equilibrium scour at the round nose pier (a = 5 ft, ℓ = 20 ft) is given by Equation 5.5.25

$$y_s = (2.0)(29)\left(\frac{5}{29}\right)^{0.65}(0.46)^{0.43} = 13.2\text{ ft}$$

Again, the skew angle could be as great as 30° so the multiplying factor is 2.0 as before and the depth of scour is

y_s = (2.0) (13.2) = 26.4 ft

and the maximum depth of scour is

y_s = (26.4) (1.3) ≃ 34 ft

This depth of scour corresponds to a scour hole bed elevation of

272-29-34 = 209 ft

Based on future meandering considerations, it appears prudent to specify that scour could reach an elevation of 200 ft. The geotech engineers should use this elevation to design the pier foundations.

7.6.5 Further Consideration

In the scour computations for this example long-term aggradation and degradation were considered neglectable. Also, because there was no increase in the flow in the channel, the overbank flow went through the relief bridge, and there was no contraction scour.

The possiblity of debris and/or ice increasing the local pier scour would, also, need to be investigated.

7.7 Overview Example Application 2

7.7.1 General Situation

In Overview Example Application 1, a far reaching assumption was made that the overbank flood flow passed under the highway through relief bridges so that, for the 110,000 cfs design flood, only 66,000 cfs would pass under the bridge. In the flood of record, the channel carried 64,000 cfs without serious damage to the protected embankments or to the existing bridge, so it is not expected that the new crossing of Section 7.6 will affect the river behavior. In Overview Example Application 2 the assumption is made that the new highway completely obstructs the overbank flow so that all the design flow passes under the bridge.

There are several consequences to forcing all the flow under the main bridge. First, the highway obstruction increases the depth of the overbank flow immediately upstream and so worsens the local flooding problem. The concentrated overbank flow returning to the river just upstream of the highway could cause land erosion. The increased flow in the channel increases the channel scour and bank attack until the excess flow can return to the floodplain downstream. Also, the increased flow increases the stage under the bridge which decreases the clearance. Fortunately, the design flood is not long lasting as shown in the design hydrograph of Figure 7.6.2. This hydrograph was adapted from the flood of record measured at the USGS gauge simply by multiplying those flows by a constant to get a peak of 110,000cfs. The actual hydrograph at the crossing would be appreciably different in shape, however, because the USGS gage is in a narrow valley where there is no overbank flow. The peak at the crossing would be flattened as the first part of the flood goes into storage on the floodplain and later drains off to increase the flows towards the last of the flood.

The problems of the increased flooding and the returning flow along the highway are difficult to treat analytically but qualitative descriptions can be readily given. Figure 7.7.1 shows a schematic drawing of the overbank flow being impounded by a highway embankment and returning to the river. At the embankment, there is no overbank flow in the downstream direction and the slope of the longitudinal overbank profile is necessarily level there. The overbank profile gradually picks up slope until it parallels the main channel slope. The difference in the two profiles of Figure 7.6.4 provides the gradient that forces the overbank flow towards the river. The intensity and velocity of flow towards the river is related to this gradient so that this intensity is highest at the embankment and tapers off slowly in the upstream direction. While these lateral flows may become quite strong, the situation is clearly not one where all the overbank flow moves like a river along the embankment. The added depth of overbank flooding depends on how much gradient is required to direct the flow back to the river.

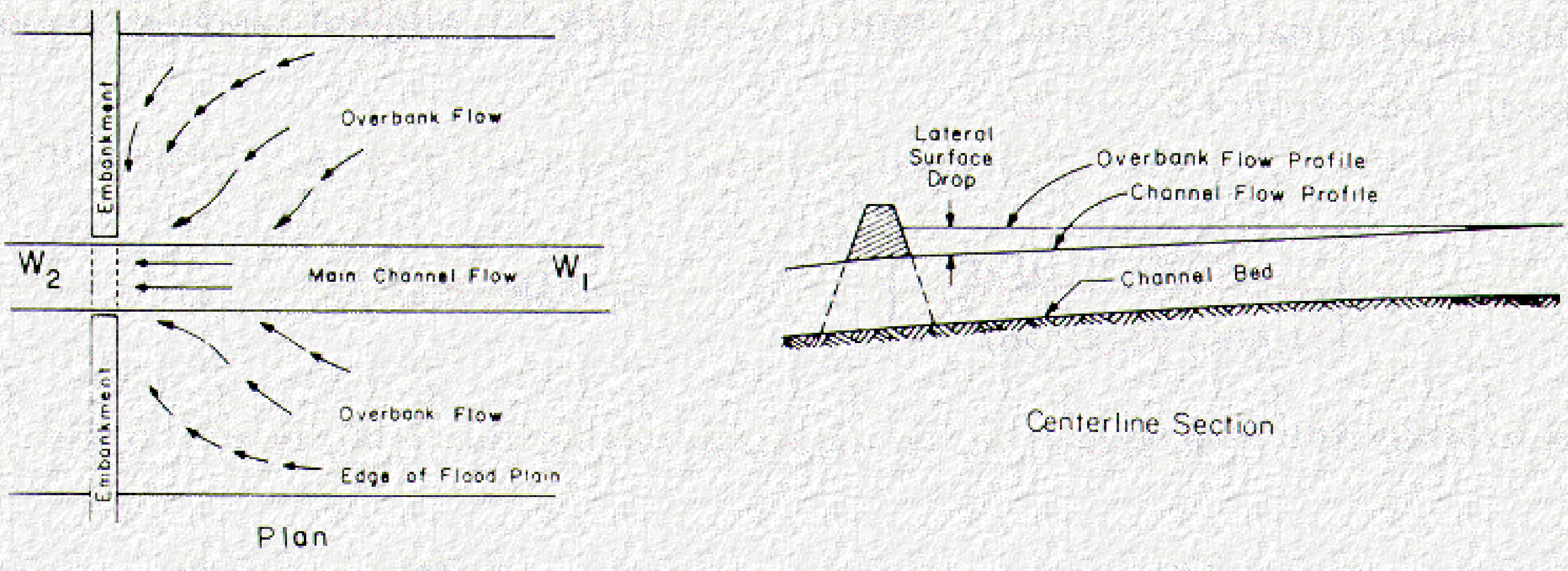


Figure 7.7.1. Schematic of Overbank and Main Channel Flow

The profile of the river channel flow is shown essentially constant in slope. This is not actually so because the overbank flow is entering the main channel with low momentum in the direction of flow and must pick up momentum at the expense of increasing the main channel slope. This increases the upstream main channel stage and consequently the amount of flooding.

The stage under the crossing is also increased over example 1 as a result of backwater due to increased channel flow. In this example, the banks are either protected or higher than average between the proposed crossing and the existing bridge so that the flow is expected to remain channelized for some 2000 ft before it can again spread onto the floodplain. As a first approximation, it is assumed that no appreciable bed scour has occurred in the several hours that it took for the flood to reach the design flow. Also, the return of the flood water to the floodplain takes some distance of the river to accomplish but it is assumed here that it all happens at one point.

What is the depth and velocity of flow in the main channel when the discharge is 110,000cfs?

The unit discharge is

$$q = \frac{Q}{W} = \frac{110{,}00}{500} = 220\ \mathrm{cfs/ft}$$

assume the friction slope is equal to the bed slope

$S_f = 0.00138$

With the higher discharge, a plane bed should exist with Manning's n 0.012 to 0.015. As a first approximation assume n = 0.015 (see Table 2.3.1, alluvial sandbed channels, plane bed). Then from Equation 2.3.20

$$y_o = \left\{ \frac{(220)(0.015)}{(1.486)(0.00138)^{1/2}} \right\}^{3/5} = 11.6\ \mathrm{ft}$$

and the average velocity is

$$V = \frac{q}{y_o} = \frac{220}{11.6} = 19.0\ \mathrm{fps}$$

The average shear stress on the bed is, from Equation 2.3.31, with $R = y_o$

$\tau_o = \gamma y_o S_f = (62.4)\ (11.6)\ (0.00138) = 1.0$ psf

and the stream power is

$\tau_o V = (1.0)\ (19.0) = 19.0$ ft lb/sec/sq ft.

For 1 mm sand and this stream power, Figure 3.4.1 indicates that the flow is in the upper flow regime; that is, the bed form is antidunes.

Check the Froude number of the flow in the main channel

$$Fr = \frac{19.0}{\{(32.2)(11.6)\}^{1/2}} = 0.98$$

A Froude number this large is not unacceptable for this channel. Therefore, use n = 0.015. The first approximation is that the flow is critical in the main channel. With this very high velocity flow in the main channel, the channel bed scours. The general scour decreases the main channel Froude number.

In the 2000-ft reach below the bridge, the scour is of the contraction type (see Section 5.5) and Laursen's equation (Equation 5.5.14) is applicable. The equation is

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{\frac{6(2+\varepsilon)}{7(3+\varepsilon)}} \left(\frac{n_1}{n_2}\right)^{\frac{6\varepsilon}{7(3+\varepsilon)}}$$

In this case, Q_t is the total design flood discharge or

Q_t = 110,000 cfs

Q_c is the flow in the main channel upstream of the bridge or

Q_c = 66,000 cfs

W_1 and W_2 are the approach channel and contracted channel reaches. Here

$W_1 = W_2$ = 500 ft

The term ε is dependent on the approach channel shear velocity and the fall velocity of the bed material. The approach channel shear velocity is

$$V_{*c} = \left(\frac{\tau_1}{\rho}\right)^{0.5}$$

Here τ_1 is the average bed shear in the upstream channel and is computed from Equation 2.3.31 or

$\tau_1 = \gamma y_1 S_f$

In the upstream approach channel the average depth is (from the previous example)

y_1 = 11.6 ft

and

$S_f \simeq 0.00138$

Then

$\tau_1 = (62.4)\ (11.6)\ (0.00138) = 1.0$ psf

and

$$V_{*c} = \left(\frac{1.0}{1.94}\right)^{1/2} = 0.72 \text{ fps}$$

The median diameter of the bed material is

$D_{50} = 1.0$ mm

Assume a shape factor of 0.7 (normal for sands) so that the fall velocity is (from Figure 3.2.1)

$\omega \simeq 12$ cm/sec = 0.39 fps

so that

$$\frac{V_{*c}}{\omega} = \frac{0.72}{0.39} = 1.85$$

With this value of V_{*c}/ω, the value of e can be found in the table accompanying Equation 5.6.14. Use

$\varepsilon \simeq 2.0$

Upstream of the bridge, Manning's n for the main channel is 0.025 (from previous example) so $n_1 = 0.025$; and downstream, $n_2 = 0.015$. Now, the flow depth downstream is

$$y_2 = (11.6)\left(\frac{110{,}000}{66{,}000}\right)^{6/7}\left(\frac{0.015}{0.025}\right)^{\frac{(6)(2)}{7(3+2)}} = 15.0 \text{ ft}$$

The uniform flow depth in the downstream section has been computed as 11.6 ft. Therefore, the contraction scour should be 3.4 ft for the design flood discharge. With this flow depth the downstream velocity is

$$V = \frac{q}{y_2} = \frac{220}{15} = 14.7 \text{ fps}$$

and the new Froude number is

$$Fr_1 = \frac{14.7}{\sqrt{(32.2)(15)}} = 0.67$$

In reality, flow conditions in the downstream reach have a Froude number somewhere between the Froude number 0.67 for the contraction scour assumption and the Froude number 0.98 for the no scour assumption. The Froude number of the flow at the peak of the design discharge hydrograph depends on the hydrograph rise time. If this time is long, the bed has time to scour out and contraction scour occurs. If the rise time is very short, the scour at the peak of the flood is much less than the contraction scour figure.

The flow and bed conditions during the passage of the hydrograph can be computed by applying the gradually varied nonuniform flow equations for water routing and a set of transport equations for routing sediment. Numerical programs are available to solve this water and sediment routing problem but their presentation and use is outside the scope of this manual.

According to Figure 7.6.2, the rise time can be very short. Then the flow in the downstream channel has a Froude number close to unity. With such flow conditions, large waves form in the channel and these waves could be very destructive to the channel and the downstream bridge. Also, the local depth of scour around the embankment ends and piers would be very large. The engineer probably would not confine all the flow under the bridge, but would provide relief bridges on the floodplain to take a portion of the flow.

Neither Example Applications 1 nor 2 are complete. At this stage, the engineer would decide on whether to have relief bridges, their number,location, and amount of flow they would take. These decisions would be based on economic, political, social and environmental factors that would exist for a particular site. The engineer would need to continue the analysis outlined in the two examples and, in addition, would need to make a backwater analysis of the most feasible designs before going to final design.

7.8 Overview Example Application 3

7.8.1 General Situation

This example is concerned with degradation of the channel. The same crossing and the same 500-ft bridge as in Example Application 1 are used but now a storage dam has been constructed 7 miles upstream for power generation, summer irrigation and flood control. Normal daily power releases are 10,000 cfs from the power plant for the six high demand hours and nominal releases for the remainder of the day to maintain fish stock. The irrigation diversion is far downstream of the crossing. Flood routing through the reservoir will reduce peak flow to the extent that the l00-year design flood is now only 40,000 cfs. The natural flow of sediment in the river has also been eliminated at the dam. The downstream control is 9 miles downstream where Mainstream River joins a much larger river.

The effect of the dam is that the time distribution of the flow is changed although the total volume is not. The flood peaks are reduced and the sediment transport is cut off. The average flow has been increased from 7000 cfs to about l0,000 cfs, ignoring the periods when the flows are very low. According to Figure 4.4.3, increasing the mean discharge shifts the river towards the braided stream classification. Such a shift is generally a destabilizing trend. The channel will

probably widen at this station and this effect may be estimated by Equation 4.4.1 which is

$W_1 \div W_2 = Q_1^{\,0.26} \div Q_2^{\,0.26}$

$W \sim Q^{\,0.26}$

The new width is

$$W_n = (500)\left(\frac{10{,}000}{7{,}000}\right)^{0.26} \simeq 550\text{ft}$$

The design flood is very nearly the bankfull discharge so that the design stage is approximately the bankfull stage or El. 268. The depth of flow of 40,000 cfs is computed from Manning's equation

$$y_o = \left\{\frac{Vn}{1.486\, S_f^{\,1/2}}\right\}^{3/2}$$

but since

$$V = \frac{q}{y_o}$$

$$y_o = \left\{\frac{qn}{1.486\, S_f^{\,1/2}}\right\}^{3/5}$$

The unit discharge is

$$q = \frac{Q}{W} = \frac{40{,}000}{550} = 72.6 \text{ cfs/ft}$$

The D_{90} value, the gravel bars and the smaller discharges (40,000 cfs vs 66,000 cfs) will increase the value of Manning's n to a value larger than that used in Example Application 1. Therefore, from our experience in working Example Applications 1 and 2, estimate Manning's n as 0.028. The friction slope is assumed equal to the bed slope, 0.00138. Then

$$y_o = \left\{ \frac{(72.7)(0.028)}{1.486\,(0.00138)^{1/2}} \right\}^{3/5} = 8.7 \text{ ft}$$

The average velocity is

$$V = \frac{q}{y_o} = \frac{72.7}{8.7} = 8.4 \text{ fps}$$

The average bed stress as before is

$\tau_o = \gamma\, y_o\, S_f$ = (62.4) (8.7) (.00138) = 0.75 psf

The Froude number for the channel flow is

$$Fr_1 = \frac{8.4}{\sqrt{(32.2)(8.7)}} = 0.50$$

and the average bed level is the bankfull stage minus y_o

268 - 9 = 259 ft

The bed level of El. 259 can be expected to degrade as a result of a cutoff of sediment by the dam. This degradation can be very extensive on a steep sloping river such as this one. Degradation starts at the dam and progresses downstream with time and stops only when it reaches a rock or gravel ledge or where the river enters a lake or confluences with a larger river as in this case. The river scours its bed to establish an ultimate gradient such that the shear is below the critical for transport of sediment. This is not necessarily the critical for D_{50} because the large sizes in the bed material tend to remain to armor the bed. The D_{90} size is sometimes considered as appropriate for armoring and a grain size analysis shows this to be about 15 mm for the Mainstream River.

The critical tractive force for the D_{90} material is given by Shields' diagram (Figure 3.5.1). Assume the flow is fully turbulent at the bed. Then

$$\frac{V_* D}{\upsilon} \geq 400$$

$$\frac{\tau_c}{(\gamma_s - \gamma) D_{90}} = \frac{\tau_c}{(s_s - 1)\gamma D_{90}} = 0.047$$

and

$$\tau_c = (0.047)(2.65 - 1)(62.4)\left(\frac{15}{304.8}\right) = 0.24 \text{ psf}$$

It will partially be the normal daily power release discharge that will degrade the channel. This flow is 10,000 cfs so

$$q = \frac{Q}{W} = \frac{10{,}000}{550} = 18.2 \text{ cfs/ft}$$

The Manning's n for the degraded bed will reflect the losses due to the remnant bed forms that were formed when the sediment was moving and the losses due to a higher grain roughness because the bed material becomes coarser. If there were grain roughness only, Manning's n is given by Equation 3.4.2a assuming the D_{90} is now the D_{50} size

$n = 0.04\ D_{50}^{1/6}$

$$n = (0.04)\left(\frac{15}{304.8}\right)^{1/6} = 0.024$$

Because there will be some form roughness from the gravel bars use

$n = 0.028$

Manning's equation

$$q = \frac{1.486}{n} y^{5/3} S_f^{1/2}$$

Here we know q, and n. Because

$\tau_c = \gamma\, y S_f = 0.24$ lb/ft^2

Then

$$S_f = \frac{0.24}{62.4y} = \frac{0.00385}{y}$$

Put this expression for S_f in Manning's equation so that

$$18.2 = \frac{1.486}{0.028} = (0.062y)^{7/6}$$

or

$y = 4.3$ ft

It follows that

$$S_f = \frac{0.00385}{4.3} = 0.00090$$

$S_f \simeq 4.8$ ft/mile

The slope before degradation is 7.3 ft/mile.

The existing profile and the ultimate profile are shown in Figure 7.8.1 along with an intermediate profile during the degradation process. These profiles are ultimately controlled at the larger river which is controlling the water surface level of the river at the point of confluence. If degradation proceeded to the limit shown, the scour at the crossing would be

$y_s = (9)\ (7.3\text{-}4.8) \simeq 22$ ft

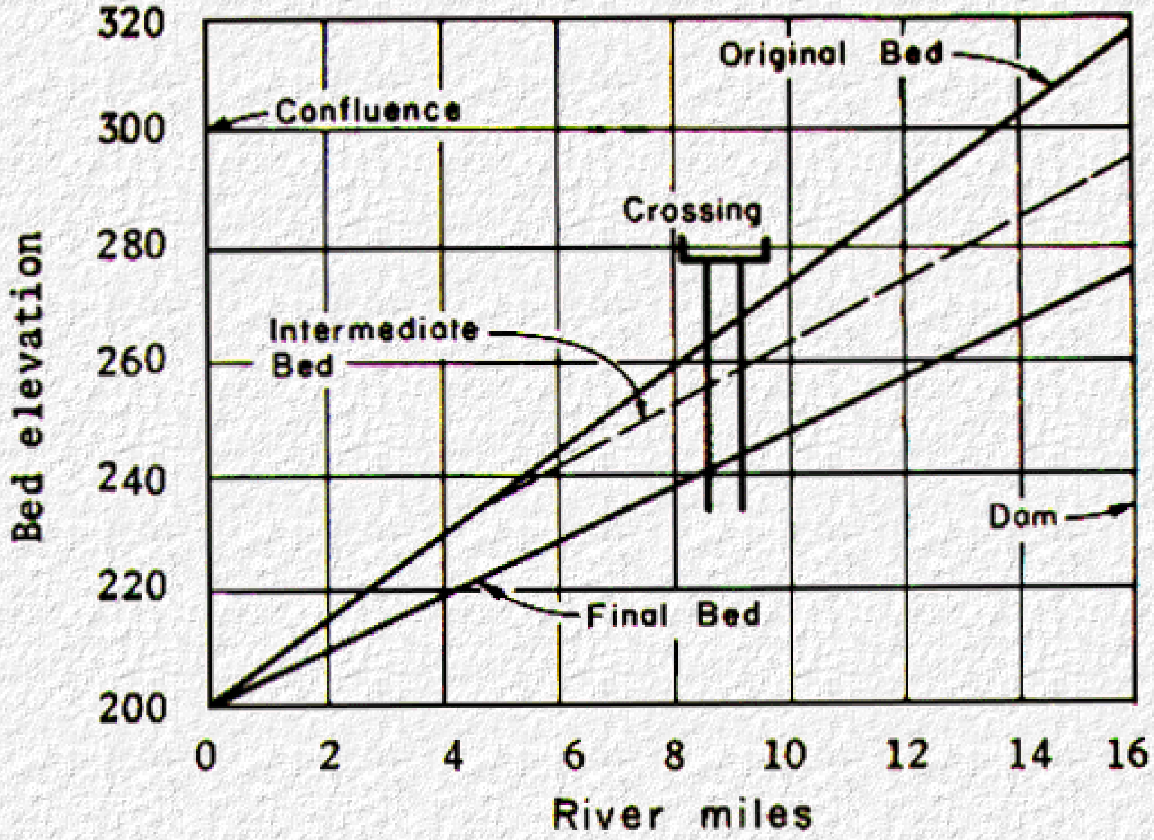


Figure 7.8.1. Degradation Due to Dam Upstream of the Crossing

This represents the removal of a substantial amount of material and, in the meantime, an equal amount is being captured by the upstream reservoir. The reservoir could well fill before the ultimate degradation is reached, in which case the flow of sediment would be restored.

A computation of the rate of sediment discharge will provide an estimate of the rate of degradation, especially if this computation is made in the undisturbed river just downstream of the degrading zone. A somewhat crude computation based on Figure 3.6.9 will serve this purpose. A discharge of 10,000 cfs at a slope of .00138 in a 550 ft channel will have a depth of approximately 4.3 ft and a velocity of 4.2 ft/sec. Using Figure 3.6.9 for 1 mm sand gives a transport rate of approximately 35 tons per day per foot of width, but since the release is for 6 hours only, the actual transport in Mainstream river is (6/24) (35) approximately nine tons per day per ft of width. If the bed material has a dry weight of 110 lbs/cu ft this represents 165 cu ft per day per foot of width. Now if the degradation zone has just reached the crossing seven miles from the dam, the average rate of degradation over this stretch of 37,000 ft is then 0.0045 ft per day or about 1.6 ft per year. When degradation has progressed to the confluence, which is nine miles downstream of the crossing and 16 miles downstream of the dam, the rate will be about one-half this or 0.002 ft per day. Thus, degradation proceeds at an ever decreasing rate. A data summary is given in Table 7.8.1.

Table 7.8.1 Data and Computations for Example Application 3

Data:

Bridge span = 500 ft Design discharge = 40,000 cfs Power plant release = 10,000 cfs for 6 hrs/day Original bed slope, S = 7.3 ft/mile
Final Degraded River Form:
Degradation discharge = 10,000 cfs Bed material size = 15mm Critical bed shear τ_c = 0.24 psf New width, w = 550 ft New slope, S_f = 4.8 ft/mile Degradation at crossing = 22 ft

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Go to Chapter 7 (Part III)

Chapter 7 : HIRE

Design Considerations for Highway Encroachment and River Crossings Part III

Go to Appendix 1

7.9 Overview Example Application 4

7.9.1 General Situation

The next two Example Applications 4 & 5 were presented by Simons and Li (1982). These examples were chosen to be incorporated into this text to illustrate a design of a bridge and a design of bank protection for a highway encroachment. These examples also illustrate the three level approach to river problems and were edited to correspond with the format of this text.

The Rillito River System in Tucson, Arizona provides an example of the problems encountered in bridge crossing design. The objective of this example is to illustrate to the designer the methodologies used in the analysis and design of a bridge crossing.

Two bridge sites are reviewed which provide insight into several outstanding problems characteristic of the Rillito system. These are the Sabino Canyon Road site with an existing bridge crossing (constructed 1936) and the Craycroft Road site with a dip crossing (where the roadway is at the same elevation as the channel bed). The study reach of Rillito River includes approximately 11.5 miles of channel extending from Dodge Boulevard to Agua Caliente Wash (see Figure 7.9.1). This included two miles on the Rillito River, six and one half miles upstream of Craycroft road on the Tanque Verde Creek, two miles upstream of Craycroft Road on Pantano Wash and one mile on Sabino Creek upstream of the confluence with Tanque Verde Creek.

The history of flood events and the recent geomorphology of the Rillito system has shown that it is very dynamic and illustrates the characteristics of a braided river. The channel is steep, dropping at the rate of 21 feet per mile. The bed material is predominantly in the medium to coarse sand sizes. The natural sinuosity of the river is low. Additionally, the river is generally unstable, changes alignment rapidly, carries large quantities of sediment, and is difficult to predict.

A large portion of the river system is in the metropolitan area of Tucson where man's activities in, and adjacent to, the river environment have induced a number of changes in the system. Primary impacts on the system have occurred due to encroachment by urban development and channelization of segments of the river. Uncontrolled sand and gravel extraction has also led to even more rapid and significant changes in the river system. A secondary effect of urbanization is a reduction in sediment supply from tributaries draining urban areas. Undeveloped land in the study area generally has little protective cover and supplies large quantities of sediment to the river system. However, extensive erosion control measures established for urban development and the creation of impermeable areas in these tributaries has reduced the sediment supply to the river system. As urbanization continues there will be a long-term decrease in sediment supply that will have a significant influence on the geomorphology of the river system.

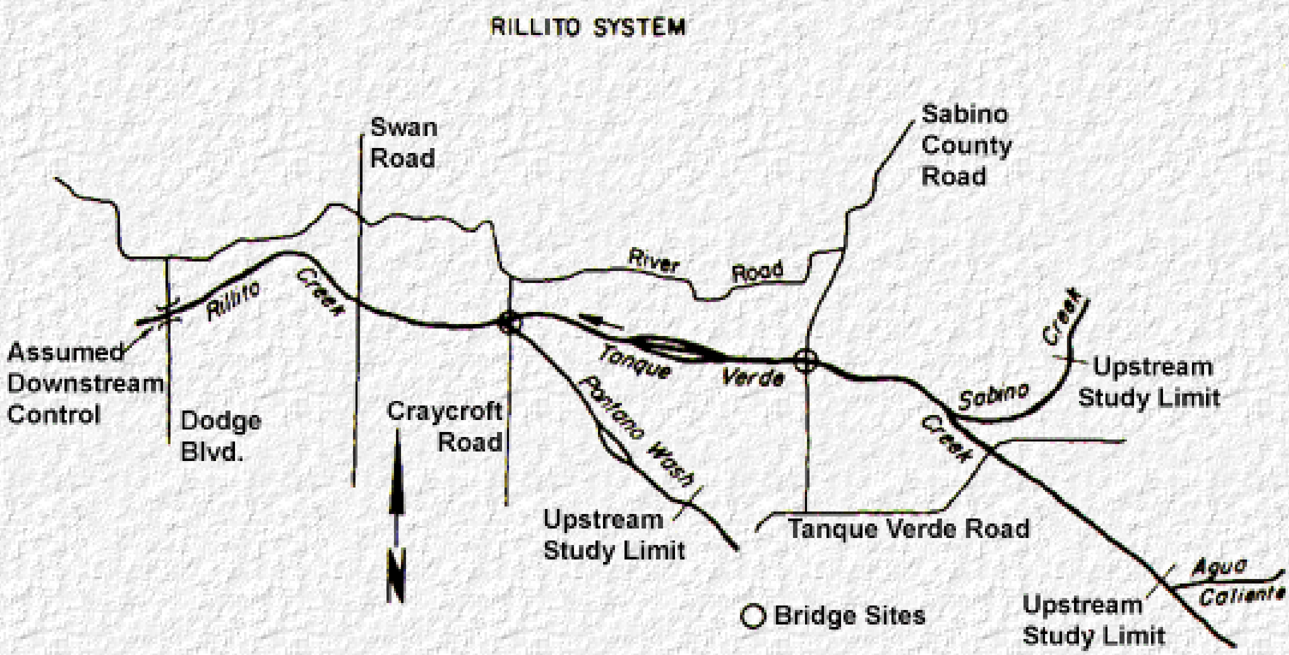

Figure 7.9.1. Rillito System Vicinity Map

A three-level analysis approach was applied to the river system to identify potential erosion and sedimentation problems associated with the bridge sites. First, a qualitative geomorphic analysis was performed documenting the history of the river system, the type of river form, the qualitative response of the system, and potential local problems at the bridge sites. The second level was an engineering geomorphic analysis which assesses the general quantitative response of the system, including determination of sediment supply, sediment transport rates, equilibrium slopes in the system for selected conditions, and lateral migration tendencies. The third level applied a detailed water and sediment routing procedure to evaluate the as-is conditions and various design alternatives. This three-level analysis provides the necessary information to:

1. Evaluate the stability of the bridge structures,
2. Determine the lateral migration tendencies of the channel,
3. Estimate the extent of expected general channel scour,
4. Determine the potential local scour around bridge piers and abutments,
5. Estimate the long-term effects of sediment degradation or aggradation on the bed and water surface profiles,
6. Determine the effects of debris on scour and water depth at the bridge sites.

Evaluation of both sites is based on the 100-year flood. Conceptual alternatives are presented and evaluated for each site.

7.9.2 Hydrology

The Rillito River is formed by the confluence of Pantano Wash and Tanque Verde Creek northeast of Tucson and flows west-northwest about 12 miles to its confluence with the Santa Cruz River. A summary of major tributaries with their drainage areas and estimated peak flow discharges is given in Table 7.9.1.

Precipitation in the Rillito River watershed is produced by three types of storms: general winter storms, general summer storms, and local thunderstorms. The general winter storms usually last for several days and result in widespread precipitation. General summer storms are often accompanied by relatively heavy precipitation over large areas for periods of up to 24 hours. Local thunderstorms can occur at any time of the year; however, they cover comparatively small areas and cause high-intensity precipitation for a few hours.

The flow in the Rillito River is intermittent; the creek is almost always dry, other than during or immediately after rain. The USGS gaging station on the Rillito River near Tucson kept daily discharge records from October, 1908 to September, 1975, after which it was converted to a crest-stage partial-record station.

Utilizing the USGS records at Rillito Station, all of the extreme events since 1915 are plotted in Figure 7.9.2. Based on these flood data the flood frequency curves are plotted on log-normal paper (Figure 7.9.3). The USGS log-Pearson Type III analysis is shown in Table 7.9.2. By reviewing historical floods which occurred in the Rillito River system, and judging from the physical characteristics of Tanque Verde Creek watershed and Pantano watershed, one may conclude that the flood peaks in Tanque Verde Creek are almost independent of those from Pantano Wash. The chance of simultaneous occurrence of both peaks is very small.

Table 7.9.1 Hydrology Summary

Location	Drainage (sq. mi.)	Estimated Peak Flow (cfs)			
		100 yr	50 yr	10 yr	2 yr
Agua Caliente Wash	42	13,000	11,000	6,300	2,400
Tanque Verde Creek (below Agua Caliente)	140	31,000	25,000	9,500	5,400
Sabino Creek	66	18,000	15,000	8,700	3,600
Tanque Verde Creek (below Sabino Creek)	215	35,000	29,300	16,400	6,000
Pantano Wash	608	32,000	36,900	15,200	5,500
Tanque Verde Creek (at Pantano Wash)	237	34,000	29,300	16,000	5,900
Ventana Canyon Wash	17	6,000	5,000	2,800	900
Alamo Wash	9	8,300	6,800	3,900	1,400

Table 7.9.2 Rillito River near Tucson, Arizona Log-pearson Type III Frequency Analysis by USGS

Exceedance Probability	Return Period (year)	Expected Discharge (cfs)	95% Confidence Limit (One-Sided Test)	
			Lower (cfs)	Upper (cfs)
0.5000	2	5,000	4,240	5,800
0.2000	5	9,300	7,670	11,100
0.1000	10	12,500	10,100	15,600
0.0400	25	17,200	13,300	22,000
0.0200	50	21,100	15,800	27,300
0.0100	100	25,200	18,400	33,000
0.0050	200	29,800	20,900	39,200
0.0020	500	35,700	24,400	47,800

The hydrograph of the 1965 flood observed at the Rillito River gage near Tucson (Figure 7.9.4) was used to establish the 100 year flood hydrographs for Tanque Verde Creek and Pantano Wash. The design hydrographs for the 100 year flood for Tanque Verde Creek, Sabino Creek, Pantano Wash, Ventana Wash and Alamo Wash are given in Figure 7.9.5.

7.9.3 Hydraulics

A complete rigid boundary hydraulic analysis was conducted prior to the moveable bed analysis. Water surface profile calculation from Dodge Boulevard to Agua Caliente Wash was conducted using the Corps of Engineers HEC-2 program. The cross sectional data were modified to reduce the number of cross-sections and makes the spatial resolution of the water surface computation compatible with the sediment routing model. The main channel roughness was reduced to near the lower limit of the river flow regime expected during the 100 year flood. The hydraulic conditions are predominantly subcritical up to Sabino Creek. The reach from Sabino Creek to Agua Caliente Wash increases its gradient and a mix of subcritical and supercritical hydraulic conditions is possible.

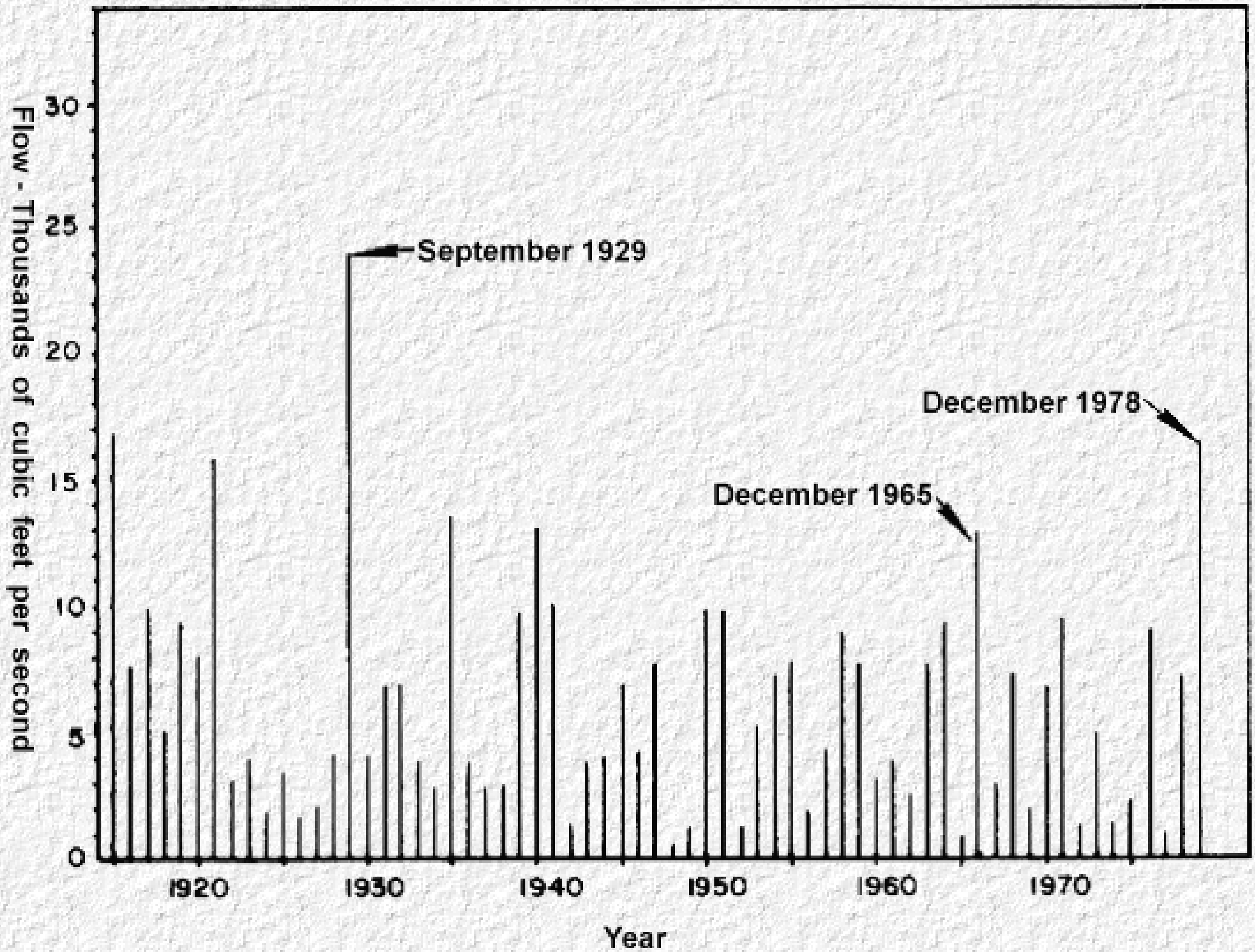

Figure 7.9.2. Flood Event at Rillito River near Tucson, Arizona (Drainage Area 915 Square Miles)

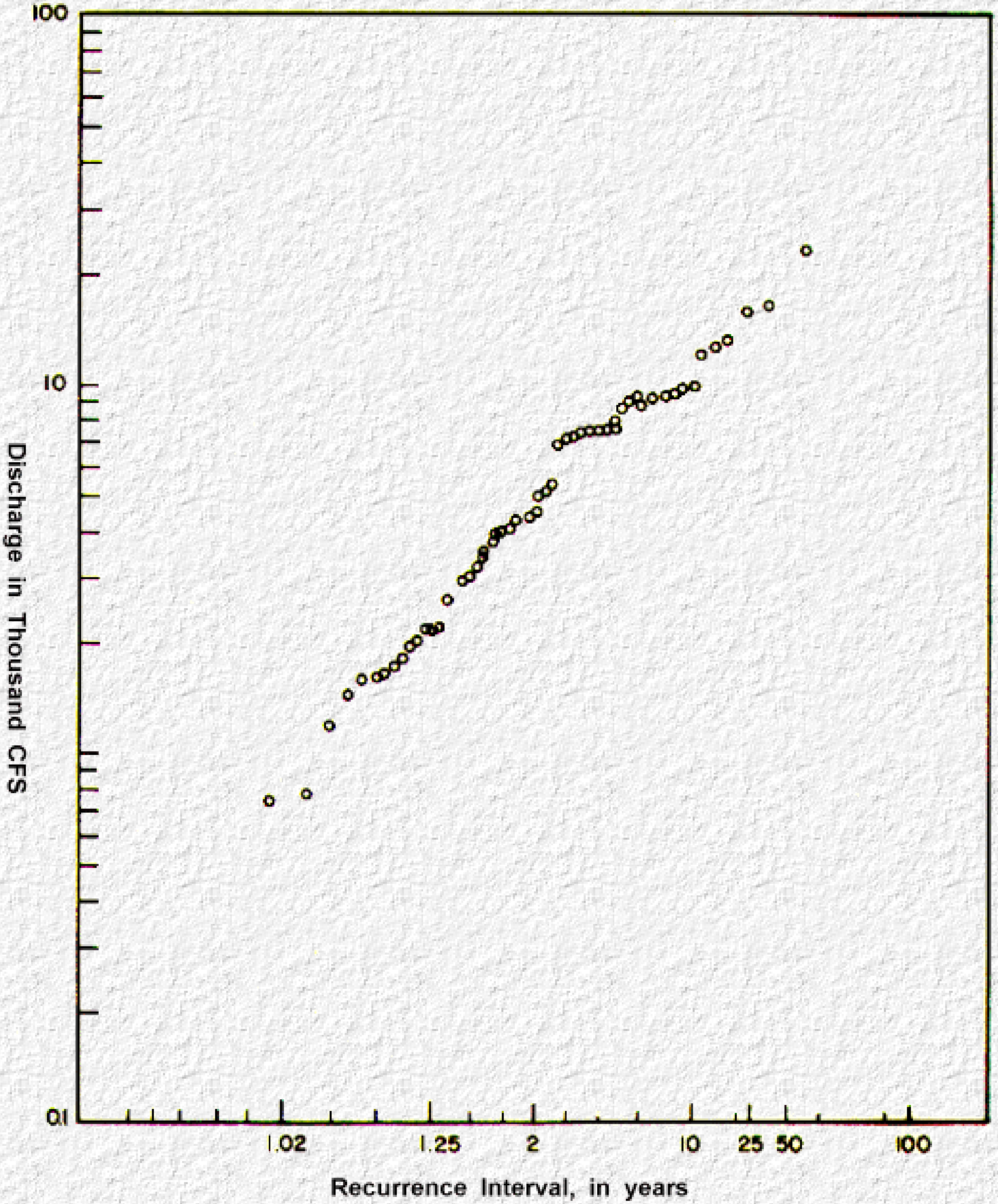

Figure 7.9.3. Log-Normal Frequency Analysis for Rillito River near Tucson

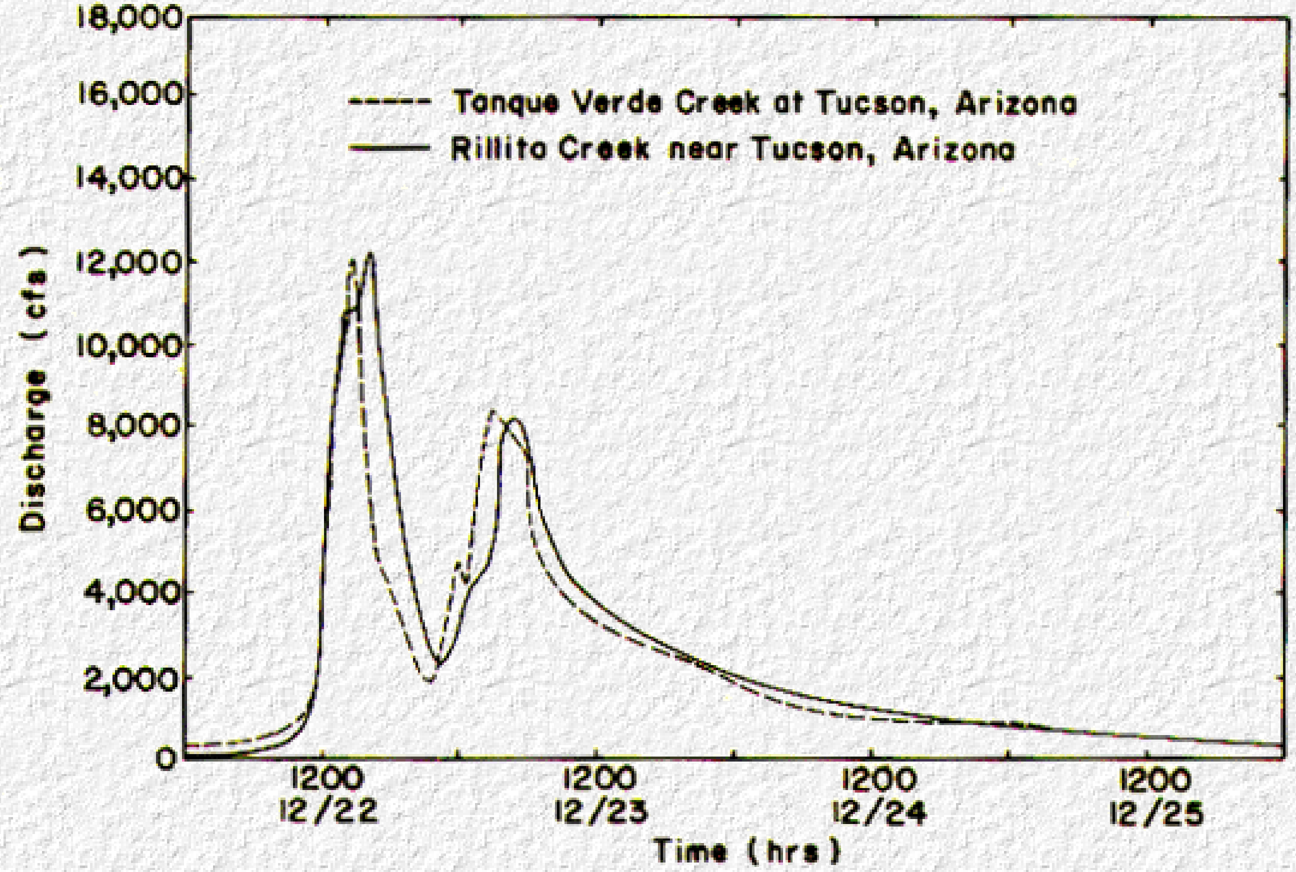

Figure 7.9.4. December, 1965 Flood in Rillito River and Tanque Verde Creek

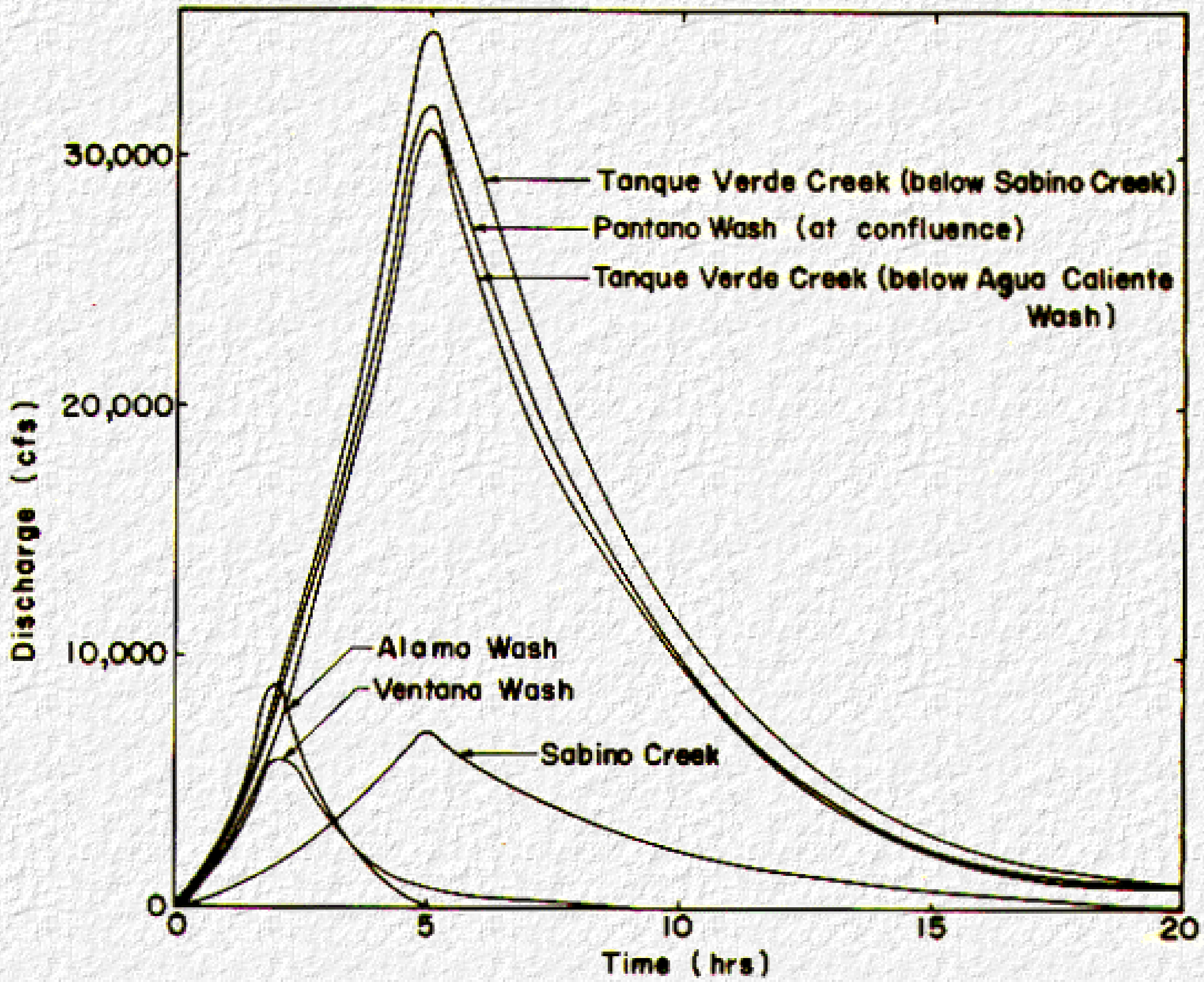

Figure 7.9.5. 100 Year Flood Design Hydrographs

The stage-discharge plot for the USGS gaging station on Rillito River near Tucson is shown in Figure 7.9.6. In this plot, the gage height values from the flood observations have been converted to equivalent stages at the present sites, based on the gage datum information given below. The shifting of the stage-discharge relationships from early years to 1970 is also indicated in this plot. For a given discharge the water-surface elevation drops about three feet from the 1956-65 curve to the 1966 curve and drops another two feet to the 1974-1978 curve. The decrease of the water surface elevation at this station is a result of the channel degradation since 1956.

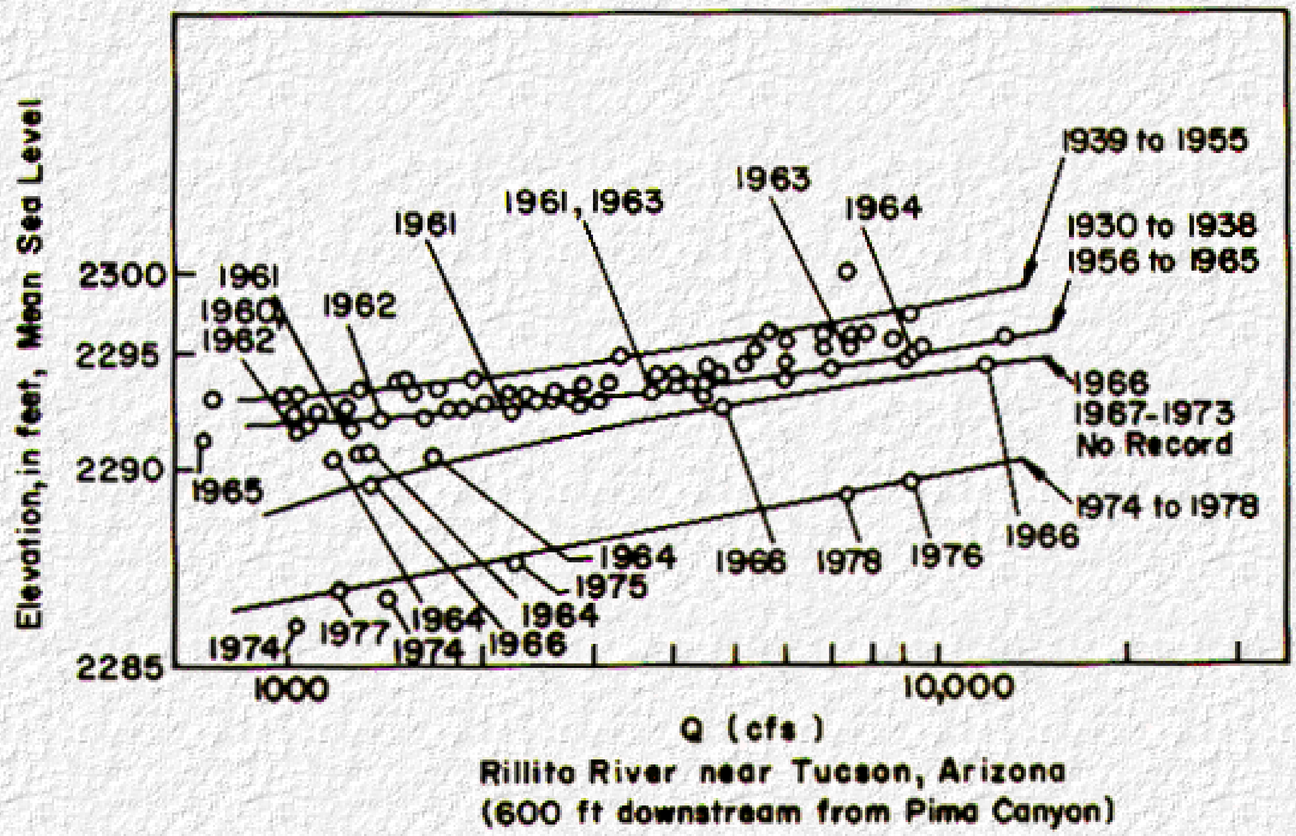

Figure 7.9.6. Stage-Discharge Plot for Rillito River near Tucson

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7.9.4 Spatial Design and Channel Geometry

In the quantitative geomorphic analysis groups of cross sections were aggregated to form a typical cross section for a study reach. The geometric properties (area, wetted perimeter, top width, and hydraulic radius) of this section are determined for a series of depths. Normal depth for a given discharge, channel roughness and slope are then calculated and the hydraulic parameters (velocity and hydraulic depth) are used in the sediment transport relationship to determine the transport capacity in the reach. An equilibrium slope analysis can then be made.

A water surface profile calculation is made based on the sections utilizing HEC-2. Hydraulic conditions in the main channel and overbank areas are based on output from HEC-2.

7.9.5 Bed Material Size Distribution

Sediment size is one of the most important parameters used in evaluating sediment transport. A thorough sediment sampling survey was conducted on the river system, consisting of 41 bed material samples. Variation of the size distribution within these segments of the river did not follow an identifiable trend, and therefore an average size distribution was used and the variation from the average size distribution was assumed to be sampling error. Three size distributions were used to cover the river segments from Dodge Boulevard to Pantano Wash (including Pantano Wash), Pantano Wash to Sabino Creek and Sabino Creek to Agua

Caliente Wash. Figure 7.9.7 shows the size distributions used for design on various segments of the river system. The size distribution of Pantano Wash is included to illustrate its similarity to the Rillito River size distribution.

A large percentage of sediment falls in the coarse sand and fine gravel range with less than 10 percent classified as medium gravel. Very coarse gravels are not present. From an analysis based on Shields' criteria, all sizes present can be easily transported by the mean annual storm. Formation of an armoring layer on the bed is unlikely since coarse, nontransportable particle sizes are missing from the distribution.

Subsurface bed material samples and bank material samples were also taken. The subsurface bed material is slightly coarser in most cases, but still lacked sizes in the nontransporting range. Bed material samples had more fine material and these distributions varied substantially from one location to the next.

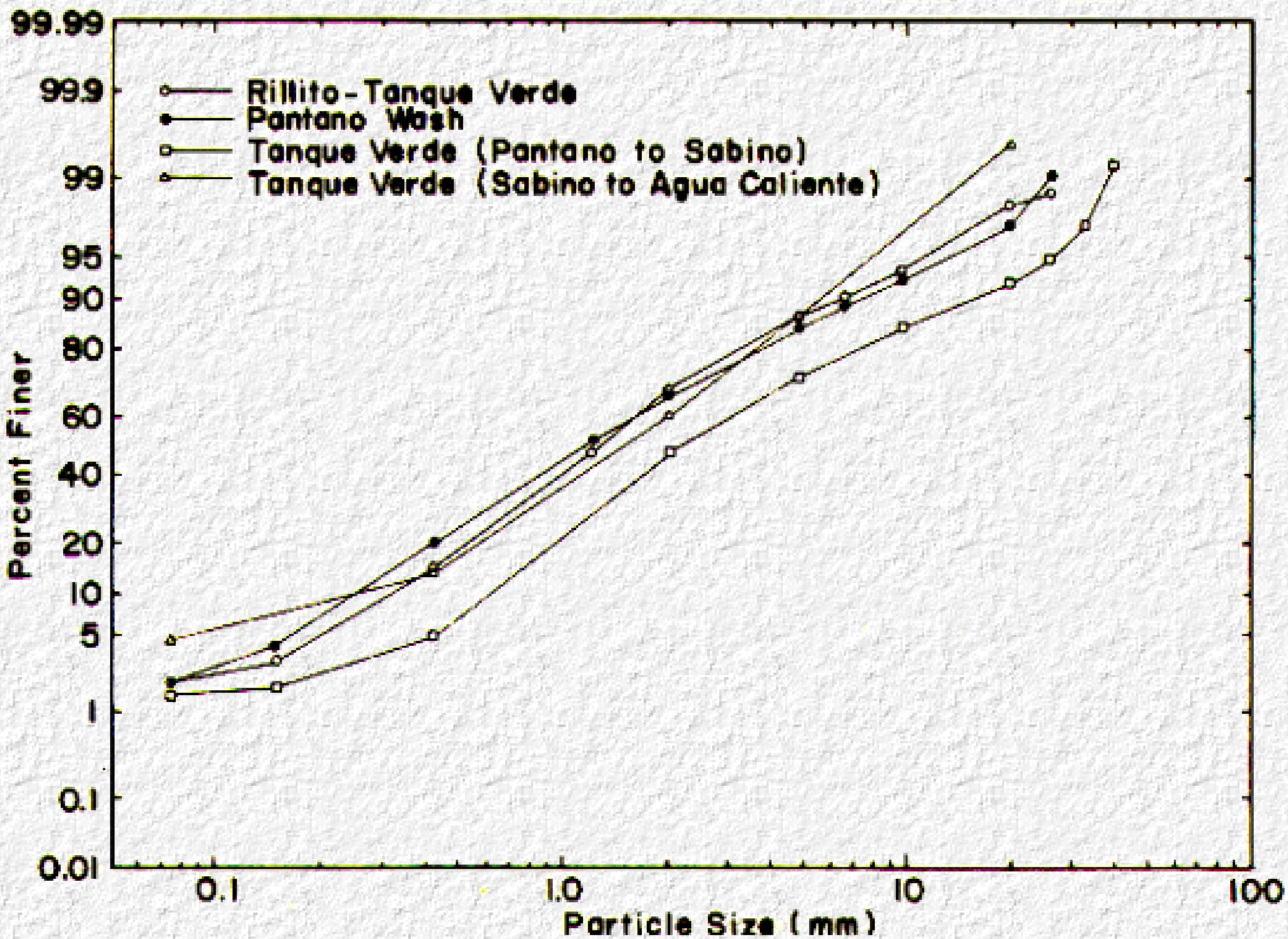

Figure 7.9.7. Rillito-Pantano-Tanque Verde Bed Sediment Distribution

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7.9.6 Existing Bridges and Other Hydraulic Structures

Four bridge sites exist in the study area at Dodge Boulevard, Swan Road, Sabino Canyon Road, and Tanque Verde Road. Measures have been taken at Craycroft Crossing to stabilize the crossing during the low flows and as a result the crossing is acting as a grade control on the Pantano Wash. Stabilization measures have not been successful on the north side of Craycroft and no grade control has formed. Complex hydraulic conditions exist at the confluence of Tanque Verde Creek and Pantano Wash during the 100 year flood. Divided flow occurs with flood water spilling laterally into Pantano Wash from Tanque Verde Creek during a flood from that watershed, or flood water spills to Tanque Verde Creek from Pantano Wash during a flood from the Pantano watershed.

The bridges across the Rillito River and Tanque Verde Creek span a variety of channel conditions. The sedimentation and erosion processes due to the proposed bridges will depend on the extent to which the bridge influences the hydraulic conditions in the river (primarily the velocity and depth). Conversely, the changing form of the channel due to lateral migration or long-term changes in the channel profile can alter the hydraulic conditions at the bridge. The Dodge Boulevard bridge is assumed to be the downstream control for the study reach. This assumption is valid because the bridge crosses a channelized section of the Rillito River which has little influence on the water surface elevation in the channel for the 100 year flood. The downstream boundary hydraulic condition is assumed to be uniform flow.

Swan Road also crosses the Rillito River at a channelized section and has little effect on upstream water surface. Craycroft Road is one of two bridges discussed in further detail later. As mentioned previously, hydraulic conditions at this bridge site are complex. Several alternatives for this site are developed along with the sedimentation and erosion analysis.

The next bridge considered is at the Sabino Canyon Crossing. This bridge crosses the defined channel on the Tanque Verde Creek. At the bridge site Tanque Verde Creek has also formed a sharp bend which will be unstable at high flow. The last bridge in the study area is at Tanque Verde Road. This bridge is on an undisturbed portion of the river system and crosses only the defined portion of the channel. Because the capacity of this channel is much less than the 100 year flood, the bridge creates a significant backwater for the 100 year flood. The roadway approaches would sustain heavy damage under these conditions.

7.9.7 Riparian Vegetation Information

The yield of debris to the bridge sites was determined by visual inspection of aerial photographs for the riparian zone of the system. A review of these photos indicated that large trees along the bank present the greatest problem. Trees along the banks were counted and the root zone size estimated. The root zone is the area of the tree capable of supporting the weight of the tree. This diameter is estimated as five to six feet for trees on the Tanque Verde.

Tree yield will be from the banks of the river for large floods. The accumulation of smaller debris at a bridge is assumed to occur only in conjunction with the trapping of larger debris. Actual debris yield and trapping at a bridge were analyzed by a probabilistic approach.

7.9.8 Resistance to Flow

During the December 1965 flood, the Rillito River was in upper regime, having antidunes with breaking waves. Similarly, the geomorphic analysis determines that the bed forms of the channels in the study system will be antidunes or standing waves during floods. Resistance to flow associated with antidunes depends on how often the antidunes form, the area of the reach they occupy, and the violence and frequency of their breaking. If many antidunes break, resistance to flow can be large because breaking waves dissipate a considerable amount of energy. With breaking waves, $C / \sqrt{g}$ may range from 10 to 20, and Manning's coefficient n ranges from about 0.019 to 0.038 for the flow depths being considered.

The existing channels will not contain all of the 100 year flood flows. Some overbank flow will occur. Sparse vegetation, brush, trees and houses are in the floodplain. These elements increase the resistance to flow. For a conservative erosion and sedimentation analysis (high channel velocity), a Manning's roughness of 0.025 for the main channels was assumed for this study. For overbank flows, a higher Manning's n value of 0.05 was used from Dodge Boulevard to Sabino Creek and a n value of 0.06 was used from Sabino Creek to Agua Caliente Wash.

7.9.9 Sediment Transport Rates

The rate of sediment transport is the most important factor in conducting a quantitative determination of aggradation and degradation in the channel. Since very little actual data were available to calibrate the sediment transport rate determinations, there is some uncertainty inherent in the procedure used to compute sediment supply rates. Fortunately, the uncertainty in the results was reduced by several factors. An indirect check of the sediment transport rate determinations was available on Tanque Verde Creek. This area has undergone the least change in river form of all the locations in the study area. The 1941 and present aerial photographs show this portion of the system to have remained nearly unchanged. Therefore, it is expected that these reaches must have sediment transporting capacities near equilibrium. The engineering geomorphic analysis is in agreement with this conclusion.

Another factor that helped provide a reliable determination of sediment supply was the grade control structure on Pantano Wash at the Craycroft Road bridge site. A channel will quickly come to equilibrium behind such a structure since the results of an excess or imbalance in sediment transport rate to the structure are corrected by removal or storage of material behind the structure. This process allows the channel to quickly reach equilibrium behind the structure by producing a channel bed slope that will result in a sediment transport rate equal to the incoming supply.

The method used to compute the sediment transport rate was the Meyer-Peter, Muller bed-load equation and the Einstein method for suspended bed material discharge. The shear stress on the bed of the channel was calibrated based on the grain resistance of the bed material. The method produced a total bed-material concentration which matched available data on the Rillito River and was consistent with similar sand-bed desert rivers.

7.9.10 Qualitative Geomorphic Analysis - Level One

Applications of the principles of qualitative geomorphic analysis (plus basic engineering relations), and quantitative analysis (sediment routing) are demonstrated for the Craycroft bridge site and the Sabino Canyon road bridge. Use of this three-level analysis gives a realistic bridge design for moveable bed and bank conditions resulting from the 100 year flood.

The purpose of qualitative geomorphic analysis is to identify the important physical processes which have been acting on the river system. General geomorphic relationships are used to classify the river system. Aerial photographs are compiled over a series of years as a means of constructing the recent history of the river system. Man's activities from gravel mining and river training are documented and the river's response noted. A qualitative prediction of river response is used, based on general geomorphic relationships such as the Lane relationship. The information gained by this level of analysis greatly aids in applying more rigorous methods of analysis.

The data base necessary for this type of analysis mainly includes aerial photographs, topographic maps (1" = 100', 2' contour intervals), and site observations. Thus, the basic characteristics of the river response can be understood quickly with limited information.

Much of the system has been significantly disturbed by human activities. Observed activities include channelization, sand and gravel mining, construction of bridges, construction of grade controls, road crossings, and encroachment by urbanization. Much of the system's shape and form, then, is dictated by man's activities rather than natural processes. This is especially true for the portions of the Rillito River and Pantano Wash within the study area. The Rillito River has been subjected to major channelization upstream and downstream of Swan Road. There is also a large instream gravel pit below Swan Road. The Pantano Wash system has a large instream gravel pit below, and bank stabilization works in the vicinity of Tanque Verde Boulevard. The results of these activities have been to change these systems from their natural braided forms to defined channels. Pantano Wash still possesses a stretch of over 3000 ft which is braided. Tanque Verde Creek, however, has experienced less impact from man's activities than the other two systems. The islands, bends and natural channel alignment observed in the 1941 aerial photographs of Tanque Verde Wash are still intact.

The Tanque Verde Creek system should be classified as a braided system. The evidence of multiple channels, islands, and shifting alignment support this conclusion.

A braided river can be identified by the equation

$$S\, Q^{1/4} \geq 0.01 \qquad (4.4.19)$$

in which S is the average bed slope and Q is the dominate discharge (cfs). The mean annual flood of 5,000 cfs is assumed to represent dominant conditions in the system. The average slope for 13.1 miles of the Tanque Verde Creek and Rillito River is 0.0044. This slope and discharge give a value of 0.037, which is well within the braided range. Pantano Wash has a slightly steeper grade, which would place it even further into the braided range. Even though much of the river has been channelized, it should be recognized that the river is in the braided range and, hence, is very dynamic.

Often, the general response of a river system to a flood event can be qualitatively assessed by studying its profile and plan view. This is especially true of a system which has been altered by man. This type of analysis is based on estimating the relative velocity along the system. In locations where a channel is constricted or the profile steepens, the velocity would be expected to increase. Since velocity is the dominant factor in determining sediment transport rate (when the sediment size does not change greatly), areas with large increases in velocity should degrade; areas where velocities are slowed considerably should experience aggradation. This is expressed in Lane's relationship, which can be written

$$Q_s\, D_{50}\, \alpha\, Q\, S \qquad (4.5.3)$$

In this relationship, Q_s is the sediment transport rate, D_{50} is the median sediment size, Q is the flow rate of water and S is the slope of the bed.

By noting where the average channel width or slope changes, a qualitative determination of the response of the three channels was made. Table 7.9.3 presents the results. The table shows that over half of the channel reaches are well balanced for sediment transport. None of the reaches has a great potential for either aggradation or degradation. In all, the system should not experience large bed elevation changes except for those related to increased development by man and localized flow conditions.

- Potential Local Problems at the Craycroft Road Bridge

  Each bridge site has possible problems associated with local erosion and sedimentation processes. These problems are identified below.

  Location of the bridge at the confluence of Tanque Verde and Pantano Wash (Figure 7.9.8) can cause several problems. First, the confluence of two sand-bed rivers is usually very dynamic and can shift upstream or downstream and laterally quite quickly. This is especially true when an abnormal sequence of events results in a shift in the relative balance of flows between the two rivers. To compound this problem, the grade control structure has created a situation in which Pantano Wash has a bed elevation several feet higher than Tanque Verde Creek at the same location. This provides an additional tendency for flows from Pantano Wash to migrate toward Tanque Verde Creek, creating a situation in which the flow could attack the bridge piers and abutments at angles other than designed. As a result, local scour around piers and abutments could be significantly increased.

  Neither Pantano Wash nor Tanque Verde Creek can contain a 100 year flood within its own channel. Since the two usually do not reach peak flows at the same time, the flow spills out of the flooding channel across the floodplain area between the two channels and into the opposite channel. In the process, the overflow deposits most of its sediment in the floodplain and the clear water entering the opposite channel causes degradation. There is also the problem of poor flow alignment past piers and abutments.

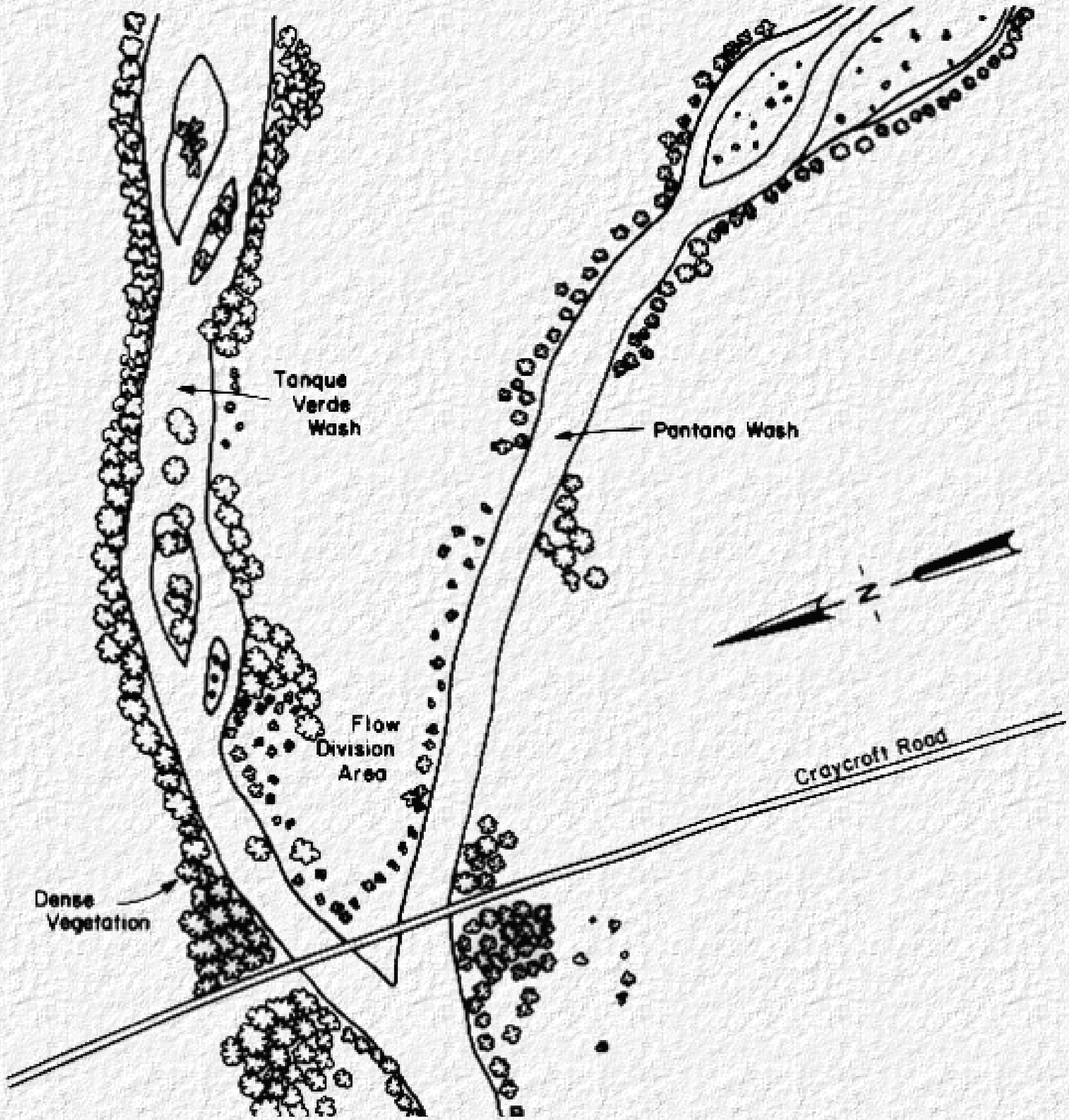
  These problems are analyzed further later. Engineering alternatives to eliminate the problems are also presented.

Table 7.9.3 Evaluation of Qualitative Response

Location	Width (ft)	Response Due to Width	Response Due to Profile	Overall Response
Rillito River between Dodge Blvd. and channelization	320	-	0	-
Rillito River channelization above & below Swan Road	450	+	0	+
Rillito River between channelization & Craycroft Road	300	0	0	0
Tanque Verde Creek from Craycroft Rd. to Sabino Canyon Rd.	310	0	0	0

Tanque Verde Creek above Supply Reach Sabino Canyon Rd.	320	Supply Reach (no determination)		
Pantano Wash from Craycroft to braided section	160	-	+	ND
Pantano Wash through braided section	600	0	0	0
Pantano Wash through gravel mining section	700	0	+	+
Pantano Wash above gravel Supply Reach mining section to Tanque Verde Road	250	Supply Reach (no determination)		

+ Corresponds to increase in slope or degradation
- Corresponds to decrease in slope or aggradation
0 Corresponds to no change in slope
ND No determination possible



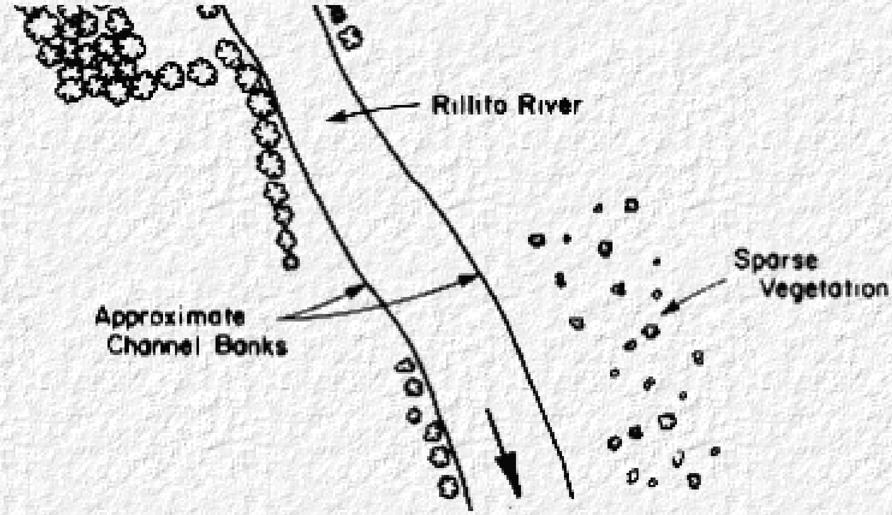


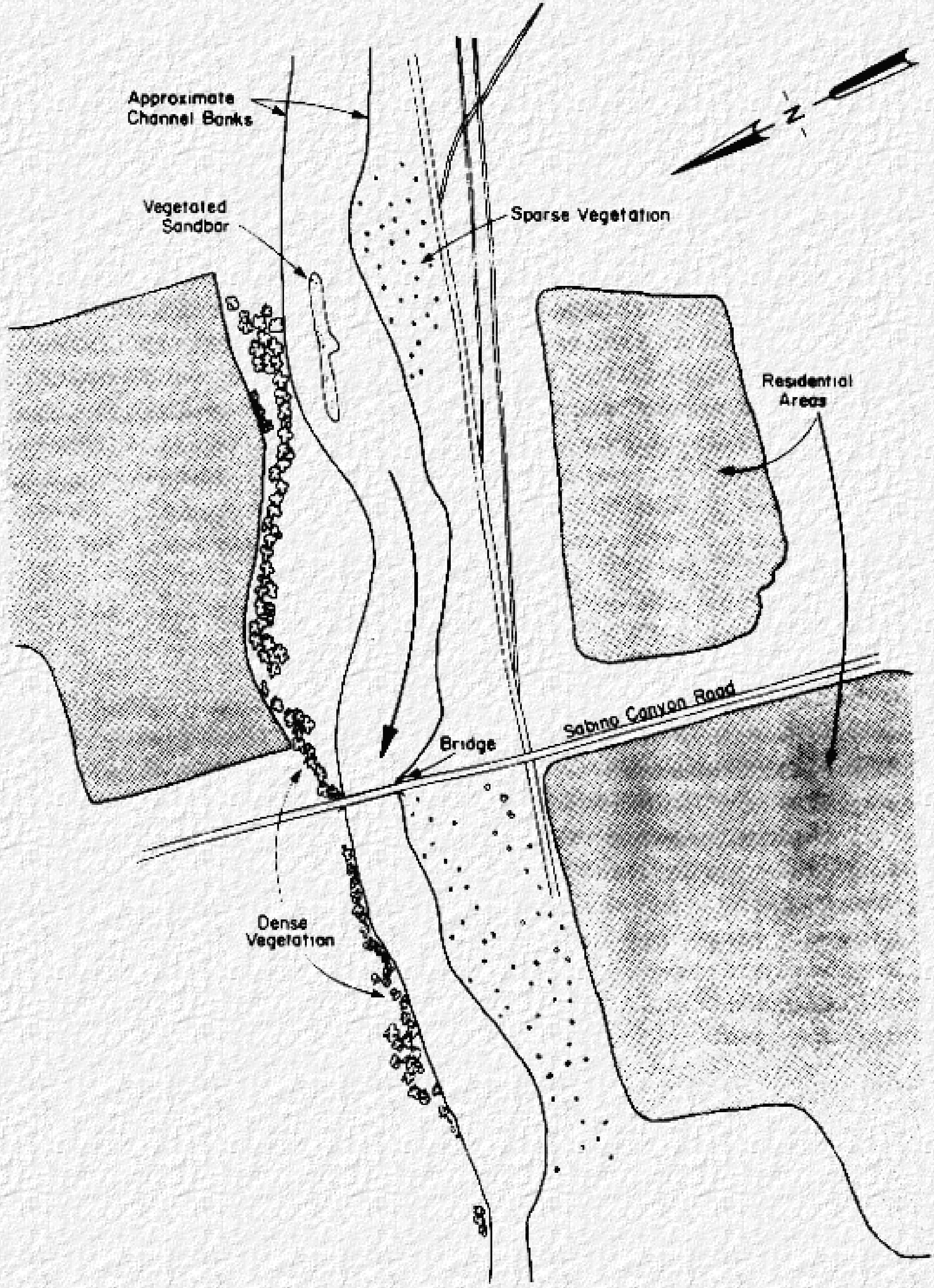
Figure 7.9.8. Sketch of Craycroft Road Bridge Crossing

- Potential Local Problems of the Sabino Canyon Road Bridge Site

  The Sabino Canyon Road bridge site (see Figure 7.9.9) has several potential erosion and sedimentation problems that should be considered in the bridge design. The present bridge has already experienced several such problems. The flow area of the bridge appears to be inadequate for the 100 year flood event. Over four feet of scour has occurred around the bridge piers and abutments. In addition, the channel is located on a reach that is migrating to the left (looking downstream). This is causing the left abutment to be attacked. The migration tendency of Tanque Verde Creek is largely due to its braided nature and its lack of confinement by bank stabilization or channelization works. The lateral migration tendency is studied in more detail in the quantitative engineering geomorphic analysis.

  Considerable scour is occurring on the left side of the channel under the bridge since it is located on the outside of the slight bend. This is the usual case with a river bend because high-velocity flow and secondary currents scour sediment from the outside of the slight bend.

  The final consideration is the gravel mining from the river. Currently, there is a mine approximately 3000 feet upstream of the bridge. The pit could act as a sediment trap and cause scour downstream of the pit near the bridge site as the water removes sediment from the bed to regain an equilibrium sediment transport rate. Because of the distance, the threat is not large from the present activity, considering the passage of the 100 year flood; however, gravel mining operations located closer to the bridge site could cause problems if not properly managed. In addition, over a long period of time the overextraction of sand and gravel can cause significant degradation for the entire reach downstream of the operating site, and possible headcuts upstream of the mining.



Approximate
Channel Banks
N
Vegetated
Sandbar
Sparse Vegetation
Residential
Areas
Sabino Canyon Road
Bridge
Dense
Vegetation

Figure 7.9.9. Sabino Canyon Road Crossing Site

7.9.11 Engineering Geomorphology - Level Two

To determine equilibrium channel slope and possible changes in channel alignment quantitatively, an engineering geomorphic analysis was performed. The analysis consisted of various river sections in the study areas for a series of water discharges ranging from 5000 cfs (mean annual flow) to 34,000 cfs (100 year flood). A multiple regression was developed for sediment transport as a function of velocity and depth. This equation was combined with Manning's equation and the water and sediment continuity equations to form a computational procedure for determining the equilibrium slope. Computation of equilibrium slopes was performed for the present sediment supply and reductions of 25 to 50 percent. Computations with the reduced sediment supplies were carried out to determine the effects of increased urbanization on the stability of the present channel system.

The possible extent of lateral migration was determined for Tanque Verde Creek by studying the channel alignment characteristics for 13 miles of Tanque Verde Wash above Craycroft Road.

The calculation of equilibrium slopes was accomplished by trial and error. The results show (Table 7.9.4) that for the present supply condition most of the study area is close to equilibrium. The exceptions occur at the bridge sites, all of which should degrade according to the equilibrium analysis. The accuracy of the calculations at the bridge sites is less than the other locations, since the normal depth assumption may not be accurate because of local hydraulic effects caused by the bridges. In all, the calculations reflect the fact that the system is near equilibrium and should not experience significant bed elevation changes if no further disturbances are introduced. This agrees with the results of the channel morphology analysis.

The sediment supply reduction cases result in degradation at all locations along the system. This is a crucial fact. It illustrates the severe consequences which would arise from a reduction in sediment supply by urbanization, sand and gravel mining, or other activities. The equilibrium conditions which presently exist would be destroyed and significant bed elevation changes would result. For example, a 0.0005 decrease in equilibrium slope resulting from a 25 percent reduction in supply would cause a degradation of 2.6 feet/mile. Lateral migration potential exists for Tanque Verde Wash at the Sabino Canyon road Bridge. In order to understand the migration process more thoroughly, 13 miles of Tanque Verde Wash above the confluence with Pantano Wash were studied closely to determine the range of channel plan geometry. Meander amplitudes, wave lengths and radii of curvature were measured.

Table 7.9.4 Equilibrium Slope Calculations, Dominant Discharge

Location	Slope in ft/ft			
	Actual	Present Supply	25 Percent Reduction	50 Percent Reduction
Cross-Sections 32-47 (Pantano)	0.0060	Too much gravel mining to accurately estimate		
Cross-Sections 31-24 (Pantano))	0.0035	Reach assumed in equilibrium at present slope (supply reach		
Cross-Sections 47-54 (Tanque Verde)	0.0033	Reach assumed in equilibrium at present slope (supply reach)		

Cross-Sections 46-45 (Tanque Verde-Sabino)	0.0048	0.0032	0.0026	0.0020
Cross-Sections 44-27 (Tanque Verde)	0.0036	0.0036	0.0030	0.0023
Cross-Sections 26-23 (Rillito Craycroft)	0.0043	0.0033	0.0027	0.0023
Cross-Sections 22-18 (Rillito)	0.0037	0.0032	0.0026	0.0020
Cross-Sections 17-14 (Rillito Swan Road)	0.0043	0.0032	0.0026	0.0020
Cross-Sections 13-1 (Rillito)	0.0039	0.0031	0.0025	0.0020

Meander wave length, channel width and radius of curvature (L, B and r_c) pairs were plotted for each meander loop. These points were plotted on logarithmic scales and on linear scales. Straight lines were fitted and the following equations were obtained and adopted for the lower 13 miles of the Tanque Verde Creek:

$$L = 2.6\ r_c$$

$$L = 2.75\ B^{1.35}$$

$$r_c = 1.06\ B^{1.35}$$

These relationships are used to determine appropriate channel widths and bend shapes at the Sabino Canyon Road Bridge site.

Sabino Canyon road Bridge is currently located on a bend with a curvature that creates several problems. At low flows, scour occurs at the outside of the bend (south side) because of high velocity and secondary currents. This phenomena is evident in the present channel cross section under the bridge. At flood flows, the problem is nearly the opposite. The north side of the bridge is attacked because of the tendency of the thalweg to straighten out the bend. The amplitude of the meander bend is 300 feet. Therefore, the lateral migration tendency is on the order of 300 feet.

These facts point to the necessity for engineering control measures to be taken at Sabino Canyon Road Bridge in order to prevent future failure of the structure from lateral migration. Possible measures are bank stabilization and channelization.

7.9.12 Sediment Routing 100 Year Flood - Level Three

The general scour of the Tanque Verde, Pantano, Rillito River system was determined using a sediment routing procedure developed by Simons, Li and Associates. This analysis determined the aggradation and degradation for subreaches in the respective river systems, sediment supply from the upstream reaches, and local hydraulic conditions throughout the system. Subcritical hydraulic conditions were assumed for all backwater computations carried out by U.S. Army Corps of Engineers HEC-2 computer program.

River and watershed information consists of upstream sediment loading information and the discretized hydrographs for the mainstem and tributaries. The mainstem hydrograph was discretized into 12 time steps with five time steps on the rising limb of the hydrograph. Time steps vary in length from 0.5 hours at the peak to four hours for the end of the recession limb. Time steps average typically 1.5 hours. The tributaries have the same time steps as the main stem. Sediment loading for the tributaries is based on normal depth hydraulic conditions in the tributaries. Channel geometries are represented by power relationships. The sediment transport rate in the tributaries is assumed to be at capacity. Two tributaries are considered in the model. These are Ventana Canyon Wash and Alamo Wash. The sediment load from the Pantano was based on a separate sediment routing procedure for this reach. The sediment output from the Pantano Wash was used as input to Tanque Verde at the Craycroft Road section. Two cases of sediment supply from Pantano were considered. The first case assumed stable conditions existed in the Pantano reach. This case corresponds to a no-fail grade control condition. The second case assumed the roadway grade control would fail during the 100 year flood. The sediment load resulting from the failure of the roadway was calculated independently of the routing procedure.

Four Tanque Verde sediment routing analyses were conducted for the as-is condition. The cases analyzed are:

> Case I. Tanque Verde floods with overflow to the Pantano Wash and no grade control failure.
>
> Case II. Tanque Verde floods with overflow to the Pantano Wash and the grade control fails.
>
> Case III. Pantano Wash floods with overflow to the Tanque Verde and no grade control failure.
>
> Case IV. Pantano Wash floods with overflow to the Tanque Verde and the grade control fails.

Figure 7.9.10 and Figure 7.9.11 show the bed level changes over the duration of the 100 year flood for both Tanque Verde flooding and Pantano flooding. The bridge sites show only slight aggradation/degradation for the 100 year flood. This agrees with the conclusion drawn from the geomorphic analysis. The major potential cause of aggradation is the failure of the existing roadway grade control. There is a maximum degradation just below the site of 4.6 feet. The erosion and/or headcut that may occur during Pantano flooding can supply significant amounts of sediment in a short period of time.

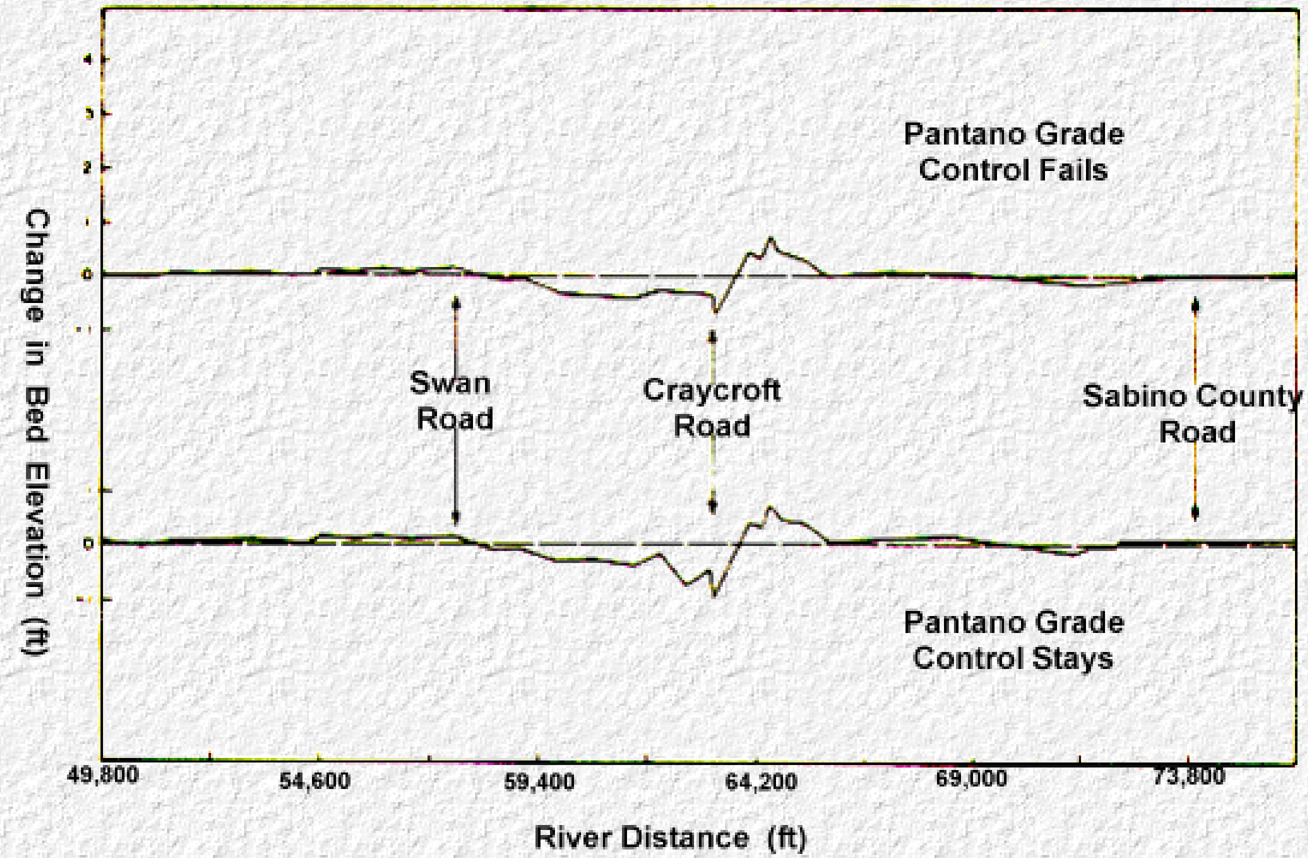


Figure 7.9.10. Bed Elevation Change of Rillito-Tanque Verde System (Tanque Verde Flooding)

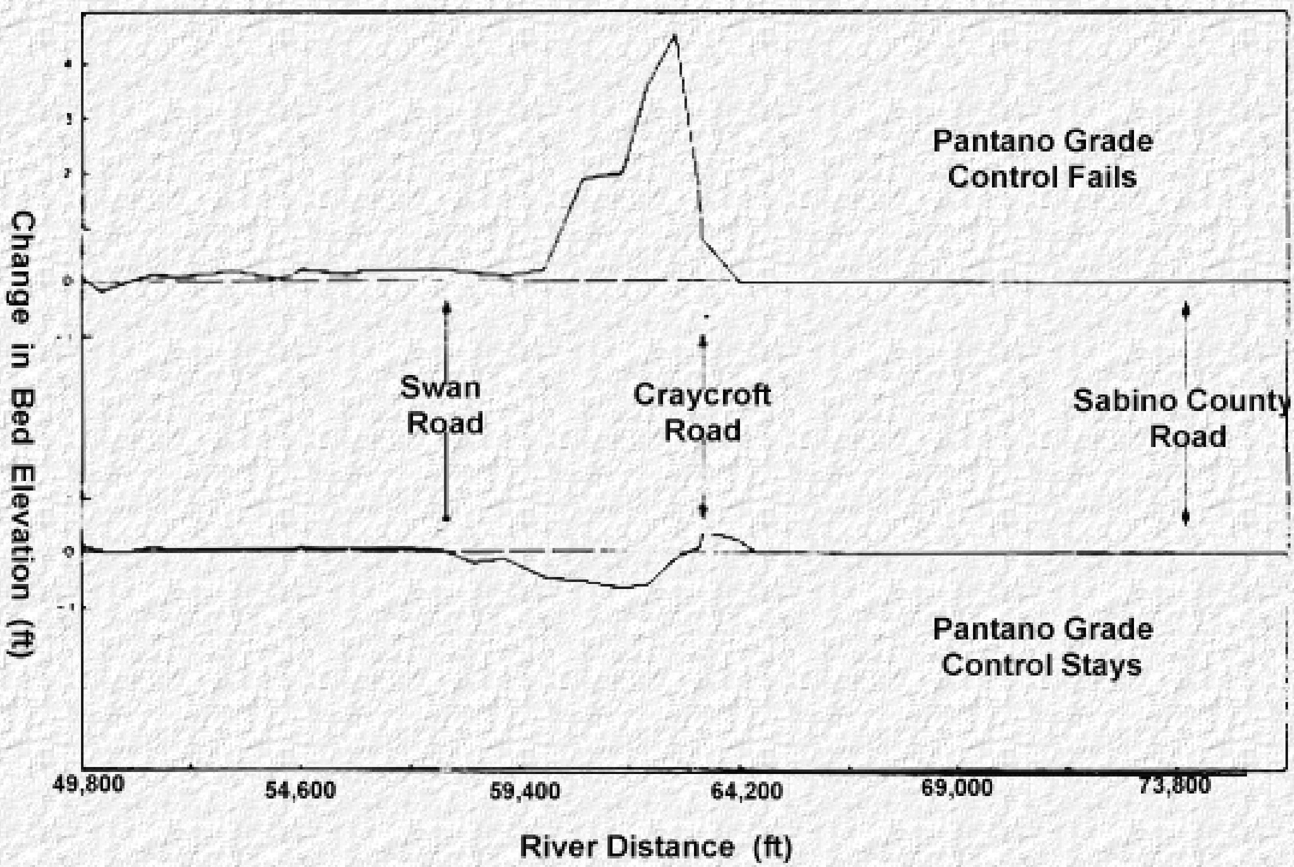


Figure 7.9.11. Bed Elevation Change of Rillito-Tanque Verde System (Pantano Flooding)

7.9.13 Results of Analysis

Each of the bridges has its own unique problems that must be considered in the formulation of alternative designs. This section presents the most practical conceptual alternatives based on the analysis of the as-is conditions. Analysis of the design alternative is broken into three areas, (1) low chord criteria, (2) total scour criteria and (3) other additional considerations.

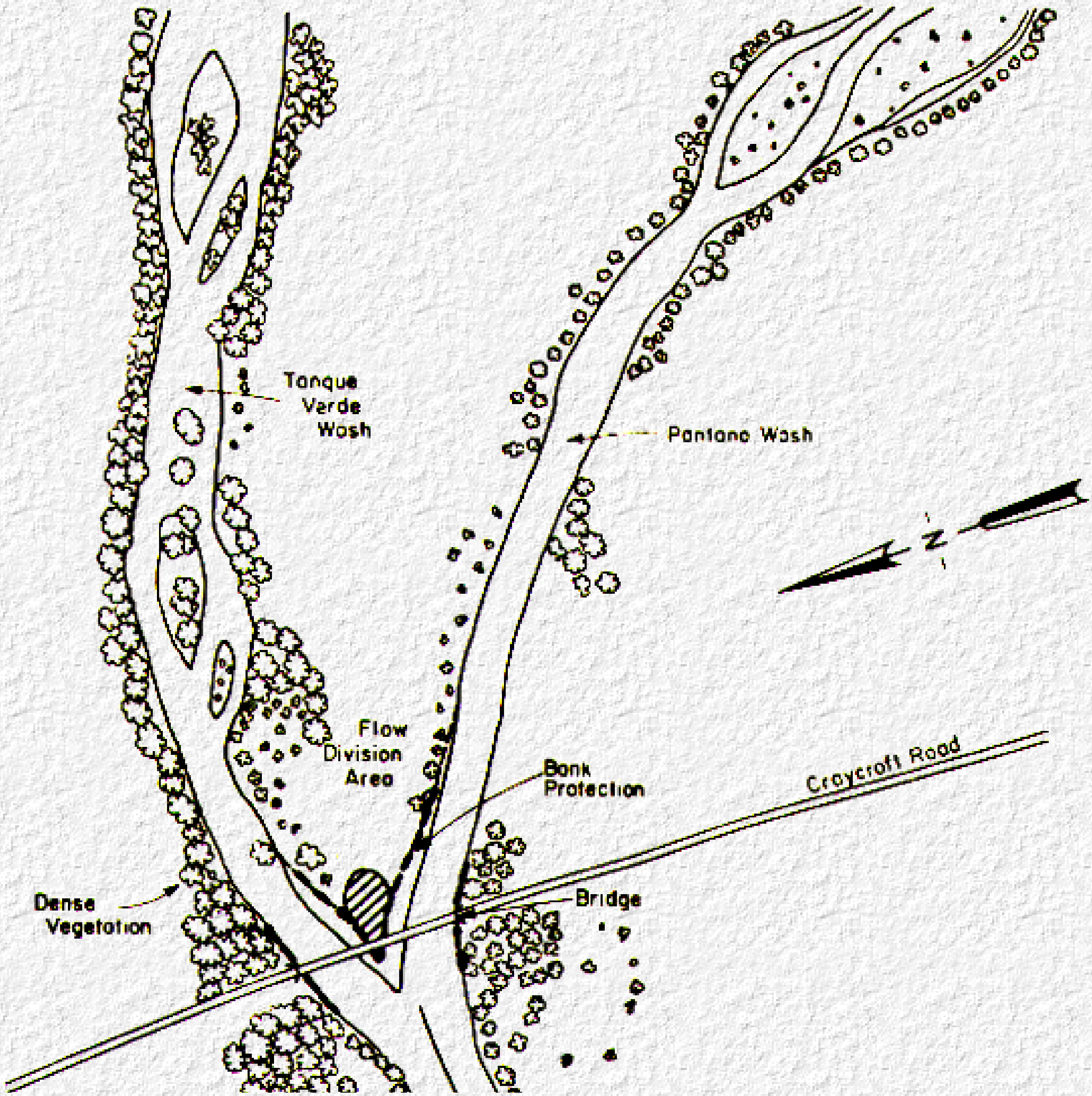
The criteria for an acceptable low chord elevation for this alternative is that the bridge can pass the 100 year flood peak with sufficient freeboard between the water surface elevation from a rigid-bed hydraulic analysis, plus additional components resulting from sand-wave movement, general aggradation, and superelevation caused by flow curvature. In addition, an increment of height is added to provide freeboard for debris passage.

When designing a bridge foundation, proper consideration of scour must be made to determine the required safe depth of piles or other supports. A design which gives adequate support for the structure when the channel bed is at its initial elevation may be inadequate after scour occurs and lowers the channel bed. The physical processes that must be considered are long-term changes in elevation, local scour, contraction scour and passage of sand waves. The total scour is the sum of these, and must be subtracted from the initial design elevation to establish the design depth for all supports. The supports must have a depth of burial below this elevation sufficient to support the structure.

- **Craycroft Road**

Three conceptual alternatives are presented which offer various channel alignment or bridge design configurations at this site. Figure 7.9.12a shows Alternative I. This alternative utilizes bank protection works, and the central embankment is designed to guide the flow through the bridge. The pier elevations are designed according to the worst scour potential of the two branches and are the same for both tributaries. The design should also consider the condition resulting from grade control failure on Pantano and should utilize circular piers to minimize problems due to adverse flow alignment.

The Alternative II conceptual design avoids some of the difficulties resulting from locating the bridge in the vicinity of the confluence. This alternative suggests relocating the bridge and road approximately 850 feet downstream of the current right-of-way (Figure 7.9.12b). This alternative would require only one bridge, and the piers could be properly aligned with the flow. Either circular piers or shaft piers could be used. However, the acquisition of the new right-of-way for this site may not be economically feasible.



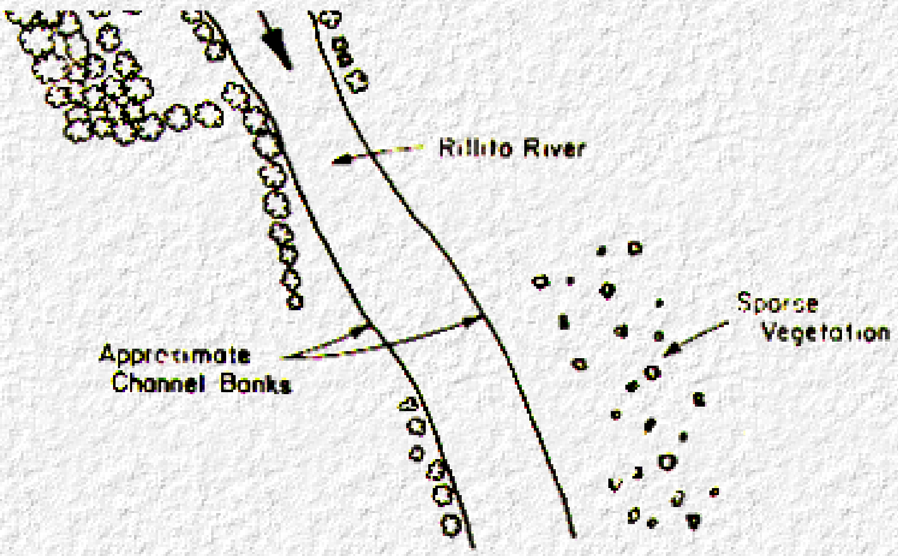
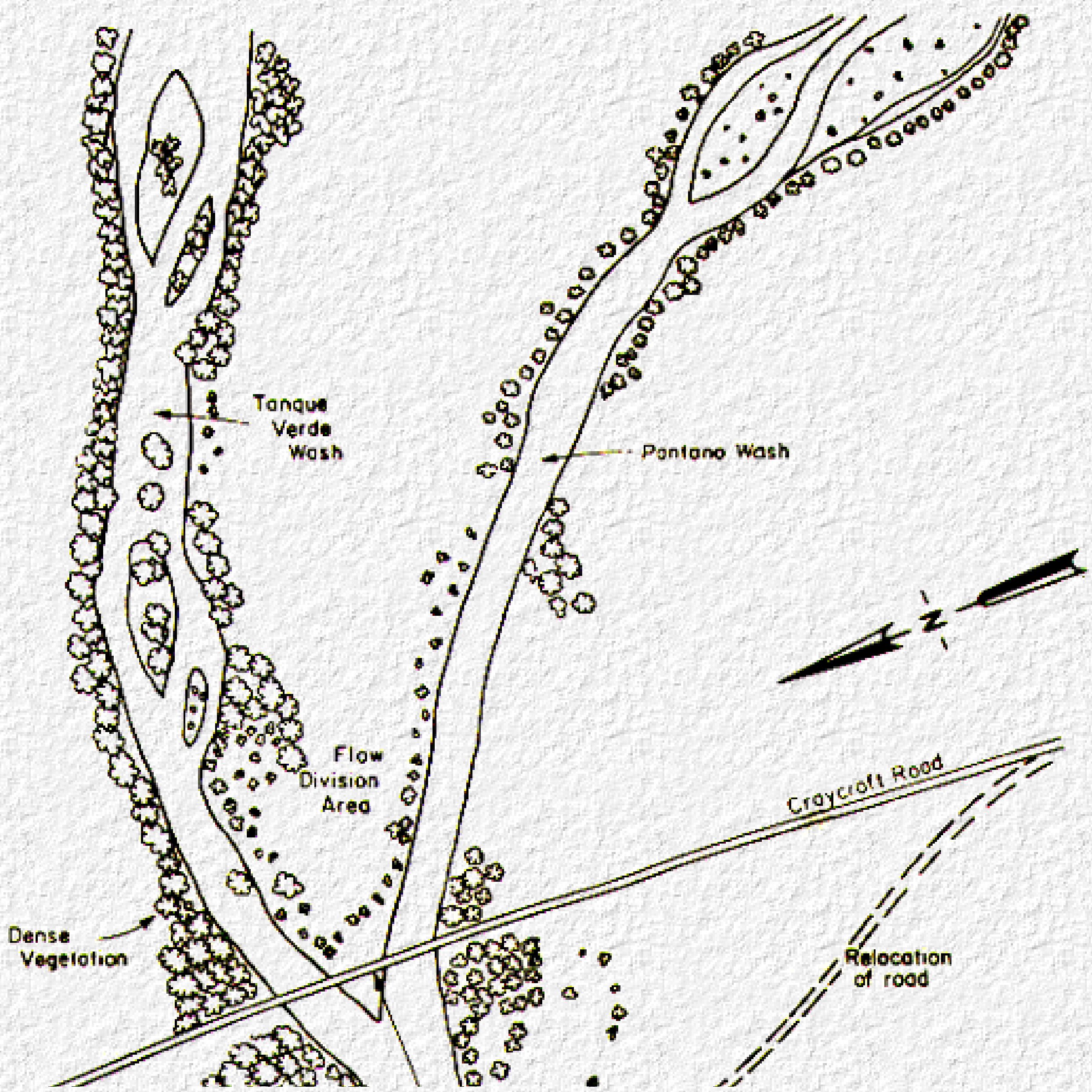


Figure 7.9.12a. Alternative I for Craycroft Road Bridge Crossing



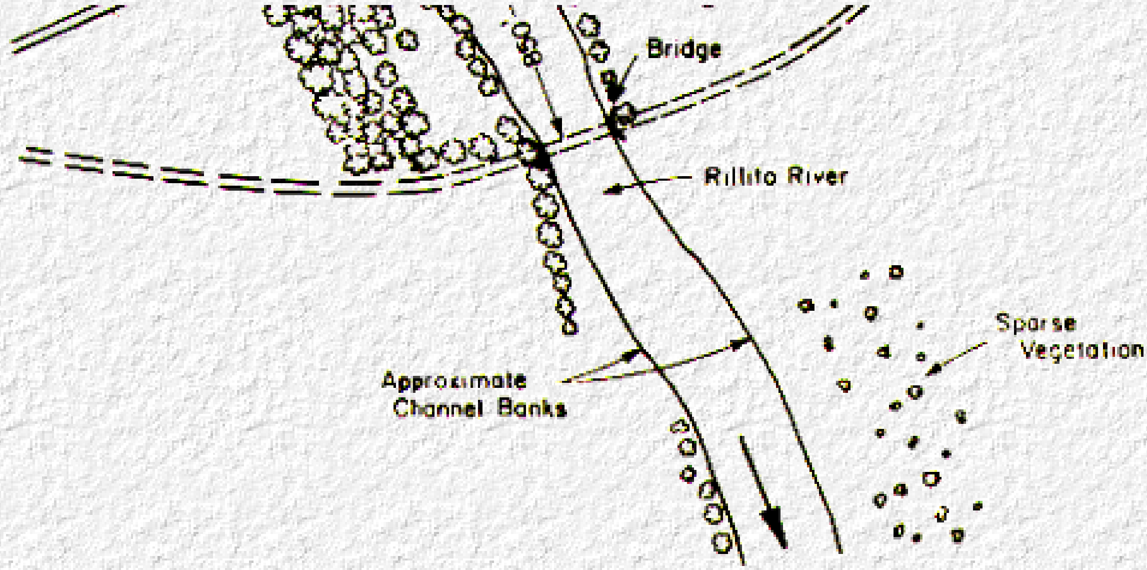


Figure 7.9.12b. Alternative II for Craycroft Road Bridge Crossing

Alternative III suggests physically moving the confluence approximately 700 ft upstream by modifying both branches of tributaries with excavation, fill and bank protection. The grade control and the deposited material behind the control would be removed. This conceptual design is shown in Figure 7.9.12c. This alternative has most of the advantages that are provided by Alternative II and does not require relocation of the road right-of-way. However, the cost of modifying the channels can be significant and may be prohibitive.

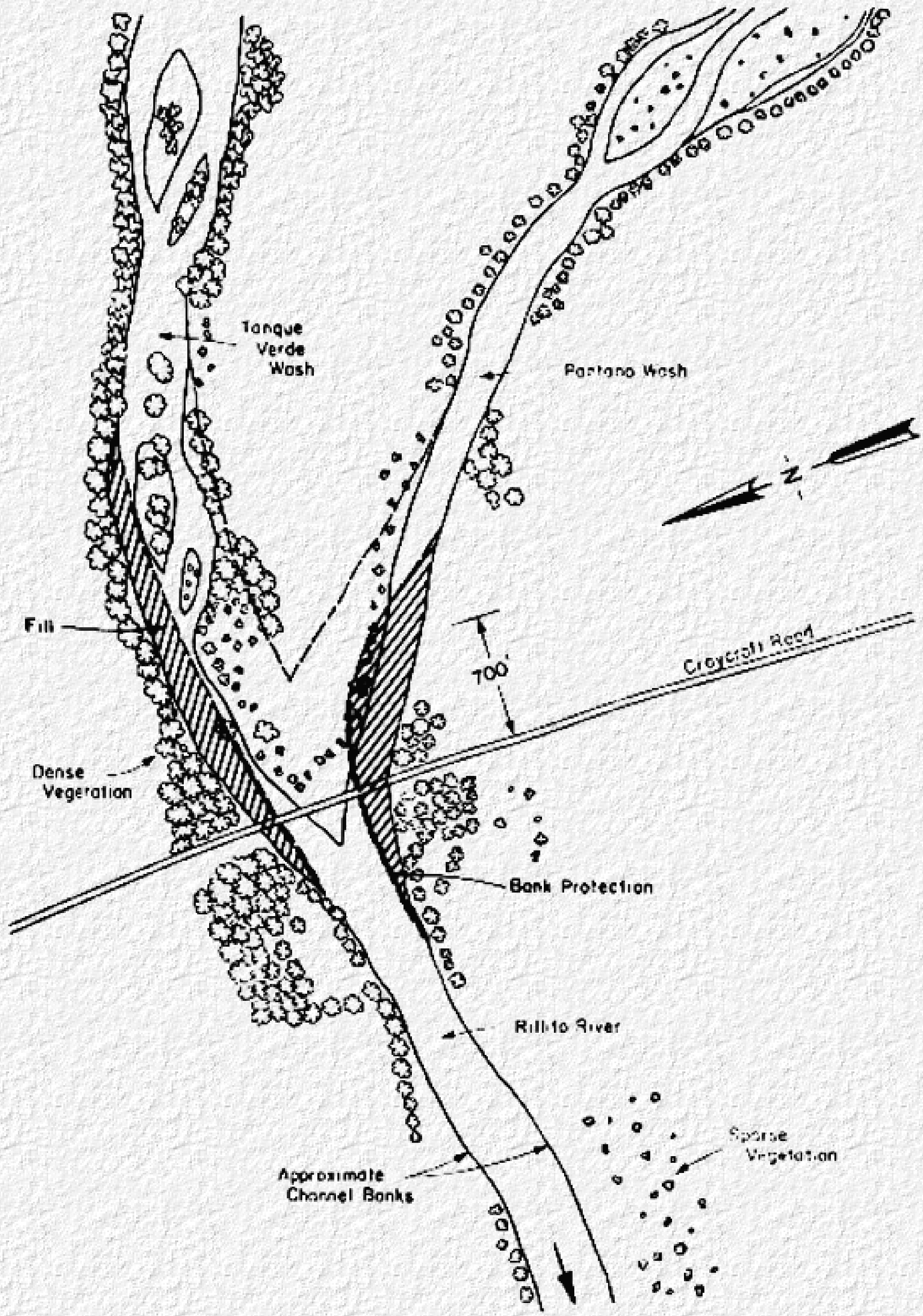
The result of low chord and total scour analysis are presented in Table 7.9.5. The freeboard requirement for debris was determined probabilistically. Pier scour was based on Neill's and Shen's equation with and without a vegetative debris accumulation. A maximum debris width of five feet was assumed for this analysis. A long-term general scour assuming a 25 percent reduction of sediment supply due to urbanization and gravel mining activities was added to the general scour calculated for the 100 year flood scour.

As mentioned earlier, the grade control on Pantano Wash near the confluence with Tanque Verde Creek is temporary in nature and is likely to fail during a major flood. Sudden failure of the grade control can cause instability of the banks and the new bridge. It is recommended that the grade control be removed, or at least the bridge design should consider the consequence of its sudden failure.

The alignment of the flow is a major concern when building a bridge in the vicinity of a confluence. Poor flow alignment can cause serious local scour problems. Circular piers are strongly recommended in this situation to minimize local scour.

Although the probability of simultaneous occurrence of the two peak flows at the bridge site is small, the impact on the proposed bridge due to such a rare event should be considered. One way to account for such an occurrence is to design the Craycroft Road Bridge with an extra margin of safety, both hydraulically and structurally.

In selecting the best alternatives for the Craycroft Road Bridge, economic, social and political constraints must be considered. Based upon this preliminary evaluation, both Alternatives I and III are more attractive than Alternative II. Alternative II has the disadvantage of a potential for both significant aggradation and degradation, which increases the cost of design. The acquisition of the right-of-way to realign the bridge in Alternative II would be costly.



Tanque Verde Wash
Pantano Wash
N
Fill
700'
Craycroft Road
Dense Vegetation
Bank Protection
Rillito River
Approximate Channel Banks
Sparse Vegetation

Figure 7.9.12c. Alternative III for Craycroft Road Bridge Crossing

Table 7.9.5 Low Chord and Total Scour Requirements at Craycroft Road

Alternative	Water Surface (ft)	Aggradation (ft)	General Scour (ft)		One-Half Antidune Height (ft)
			100 year Flood	25% Reduction in Sediment Supply	
I	2,435.6	1.7	0.5	2.6	1.4
II	2,431.0	4.6	0.8	2.6	2.1
III	2,434.4	0	0.8	2.6	2.1

Vegetative Debris Freeboard (ft)	Minimum Low Chord (ft)	Abutment Local Scour (ft)	Pier Local Scour (ft)		Total Abutment Scour (ft)	Total Pier Scour (ft)
			Without Debris	With Debris		
3.0	2,441.7	11.1	5.0	7.8	15.6	12.3
3.0	2,437.7	13.2	7.0	10.9	18.7	16.4
3.0	2,439.5	13.2	7.0	10.9	18.7	16.4

- **Sabino Canyon Road Bridge**

Four alternatives for the Sabino Canyon road bridge are analyzed. Alternative I is the present condition. The other three alternatives consider variations on a similar scheme. The obvious options involve channelization downstream of the bridge to reduce water surface elevations, widening of the bridge opening, protection of the south side of the bridge by a spur dike, and bank protection of key locations. Alternatives II, III and IV consider a channel bottom width of 300, 350, and 400 feet, respectively. A sketch of the basic design is presented in Figure 7.9.13. The drawing shows the 300-foot channel scheme of Alternative II.

The results of low chord and total scour analysis are presented in Table 7.9.5. The same approach is used for this analysis as in the Craycroft Road site. Although there is a bend, the radius of curvature is large enough to preclude any significant superelevation.

Because of the problems associated with a bridge located on a bend, Alternatives II, III, and IV include bank protection and stabilization 200 ft upstream and downstream of the bridge site. Also, additional bank protection should by provided along the south bank above Sabino Canyon Road. This latter protection will prevent the upstream bend from further migration that would cause flow alignment difficulties. Also, a guidebank should be constructed on the south side of Sabino to protect the bridge from southward channel migration and to assist in guiding the flow (see Figure 7.9.13).

By describing the radius of bend curvature as a function of channel width,

$$r_c = 1.06\ B^{1.35}$$

the appropriate radius of curvature for each alternative can be determined. The results show that for the channelization alternatives (II, III, IV), the radius of curvature is within the stable range. However, for the present condition (Alternative I), the radius of curvature is much too small. This is one cause of the present migration problem on the south bank. A very preliminary assessment concluded that Alternative II (300 ft bottom width channelization scheme) is probably the most feasible and practical solution for the new bridge. Very little is gained in terms of low chord and scour reduction by the wider channels. However, they would require a large amount of additional earthwork. The narrower channel would also cause less conflict with private property ownership.

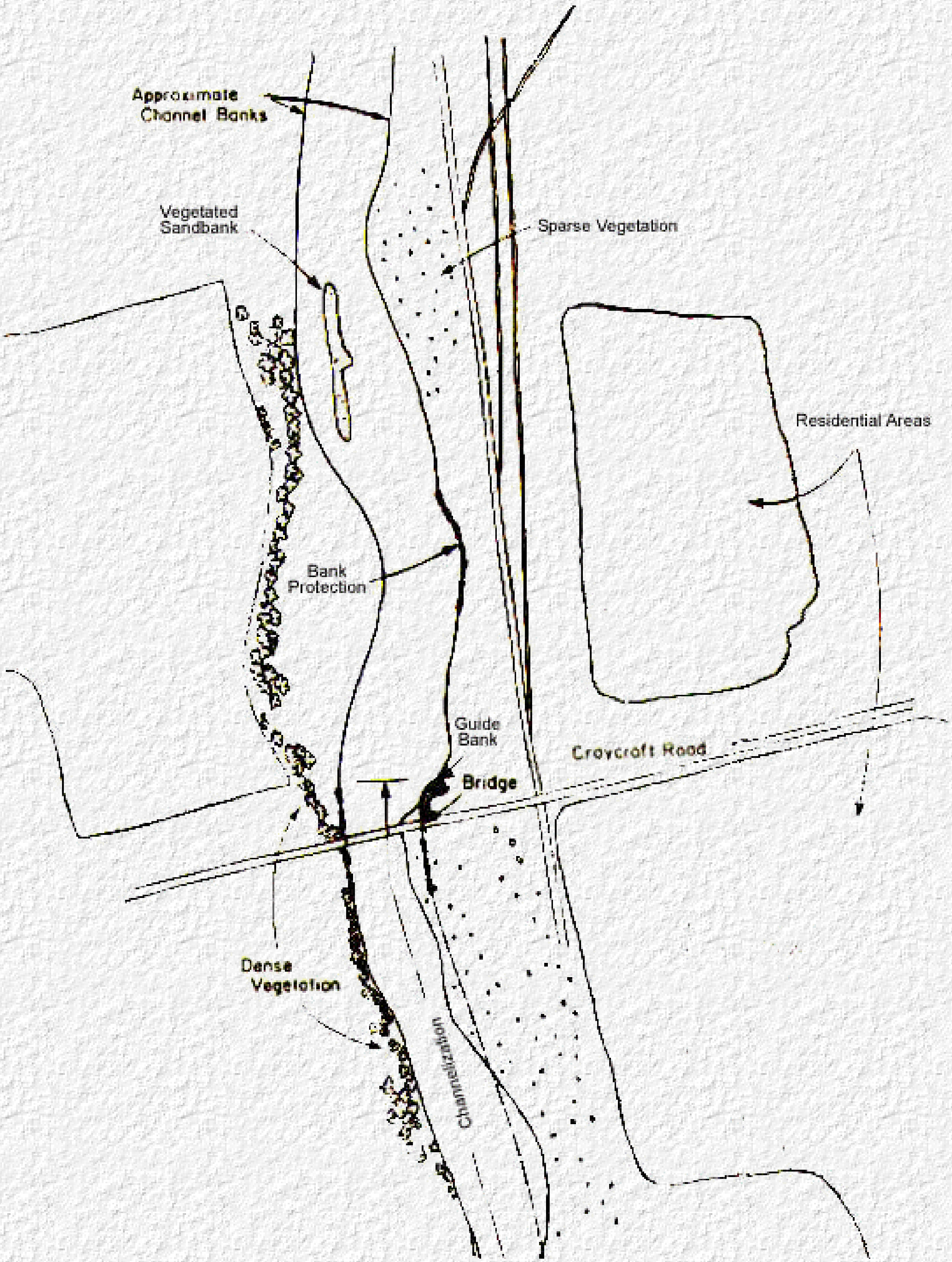


Figure 7.9.13. General Plan View for Alternative II, III, and IV (II Shown)

7.10 Overview Example Application 5

This section presents a hydrologic and hydraulic analysis of Bijou Creek, a tributary to the Narrows Unit near Fort Morgan, Colorado. The proposed location of the Union Pacific Railroad is approximately 1500 feet north of the creek (See Figure 7.10.1). A hydrologic and hydraulic analysis of the safety of the proposed railroad location is presented.

7.10.1 Hydrologic Analysis

The analysis identifies three design floods and corresponding water surface elevations with return periods of 50, 100 and 200 years. Bank protection is specified for each of the design floods and for five channel design alternatives. The design floods were estimated using the Gumbel Method of frequency analysis and all of the available hydrologic data. The design of riprap bank protection considers the design flood discharge, superelevation in the bend, bedform height, local scour, bed material size, and stability of riprap materials.

According to the analysis of streamflow data for the period 1939 to 1946 by the Corps of Engineers the maximum recorded mean daily flow is 30,900 cfs. This flow was assigned a frequency of once in 200 years. Based on the available streamflow data for Bijou Creek for the six-year period 1951 through 1956 the recurrence rate of this maximum daily flow was subsequently changed into once in 50 years. The historical records show that there were two extreme floods 282,900 cfs and 466,000 cfs observed at the streamflow gaging station of Bijou Creek near Wiggins, Colorado. According to the USGS, these two extreme floods are respectively 5.5 and 9.0 times the 50 year flood. In other words, the 50 year flood estimated by the USGS is about 51,600 cfs. Taking into consideration the two extreme floods that have occurred, the flood with a return period of 50 years is determined to be 51,600 cfs.

The floods and their return periods are as follows:

Return Period	Flood Discharge
5 years	13,400 cfs
10 years	23,000 cfs
20 years	33,000 cfs
30 years	41,400 cfs
40 years	47,100 cfs
50 years	51,600 cfs
100 years	62,000 cfs
200 years	72,500 cfs

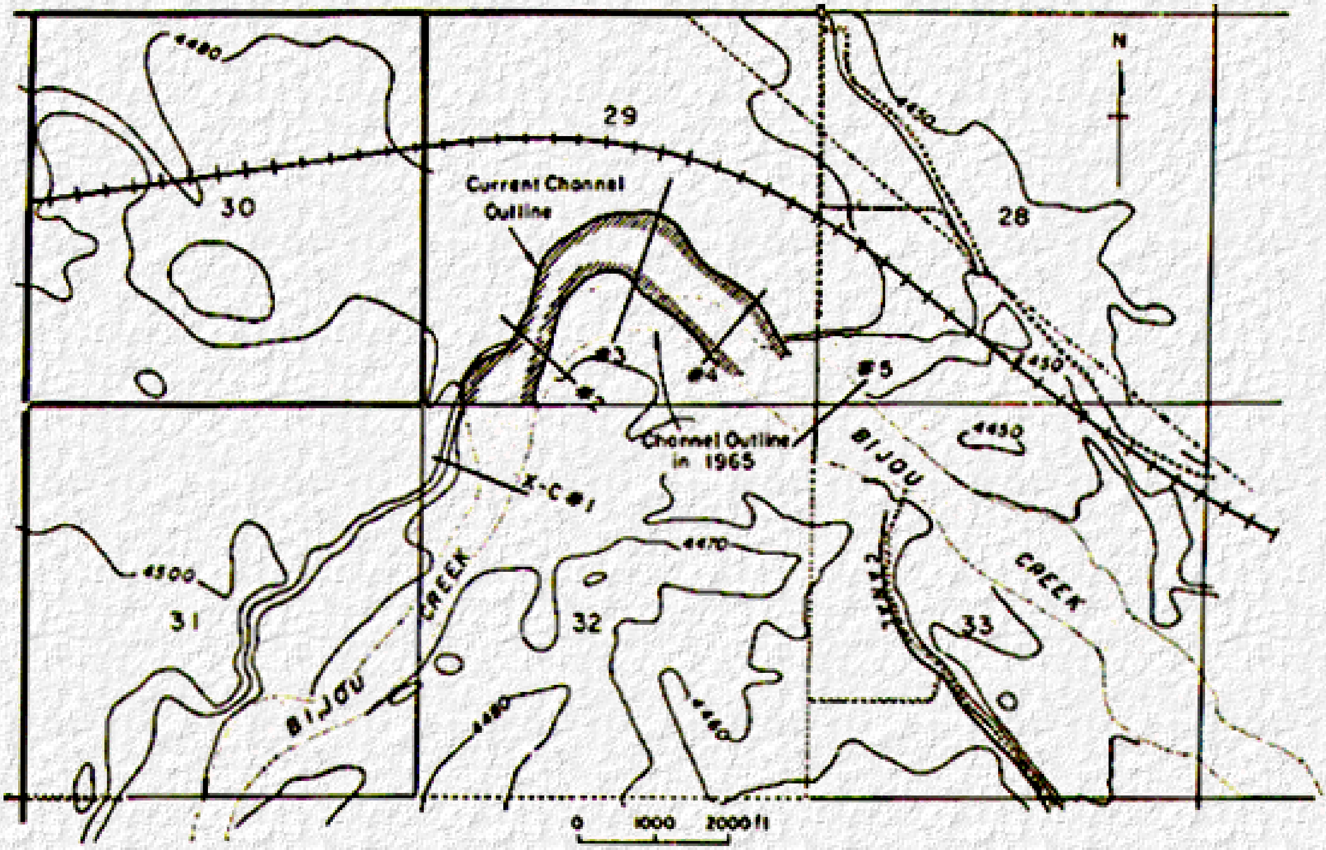


Figure 7.10.1. Topographic Map of the Bijou Creek Study Area

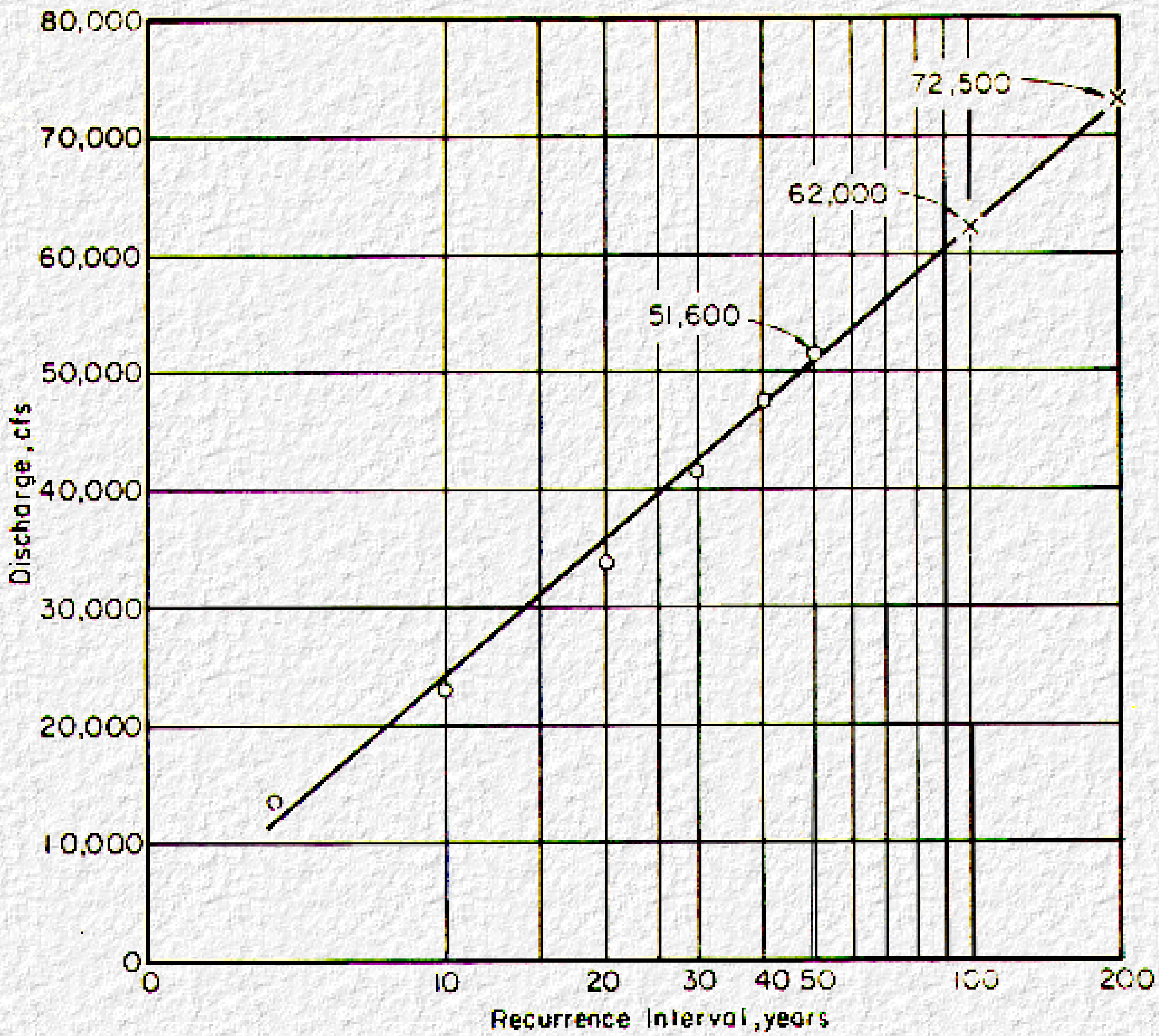


Figure 7.10.2 Gumbel's Method of Frequency Analysis. Annual Floods on Bijou Creek near Wiggins, Colorado

The above estimated floods were plotted in Figure 7.10.2 using a Gumbel probability paper. From this figure, the floods with various return periods can be interpolated or extrapolated. The estimated design floods with return periods of 100 and 200 years are respectively 62,000 cfs and 72,500 cfs.

7.10.2 Channel Morphology

A field study was conducted to obtain morphologic data for the hydraulic analysis. The information utilized in the analysis includes the representative cross sections of the channel, the representative bed material size, and the energy slope. With these data the flow resistance coefficient was estimated.

The representative cross section in the vicinity of the study site was developed by considering five selected cross sections established by the field study. Among them only two cross sections were similar in shape. They are representative of the cross sections of the study reach. The dimensions of the representative cross section were established by averaging these two cross sections. The results of this analysis are shown in Figure 7.10.3 and Figure 7.10.4.

Figure 7.10.3 shows the relationship between cross sectional areas and the flow depth as follows:

$$A = 242.6\, y_n^{\ 1.52}$$

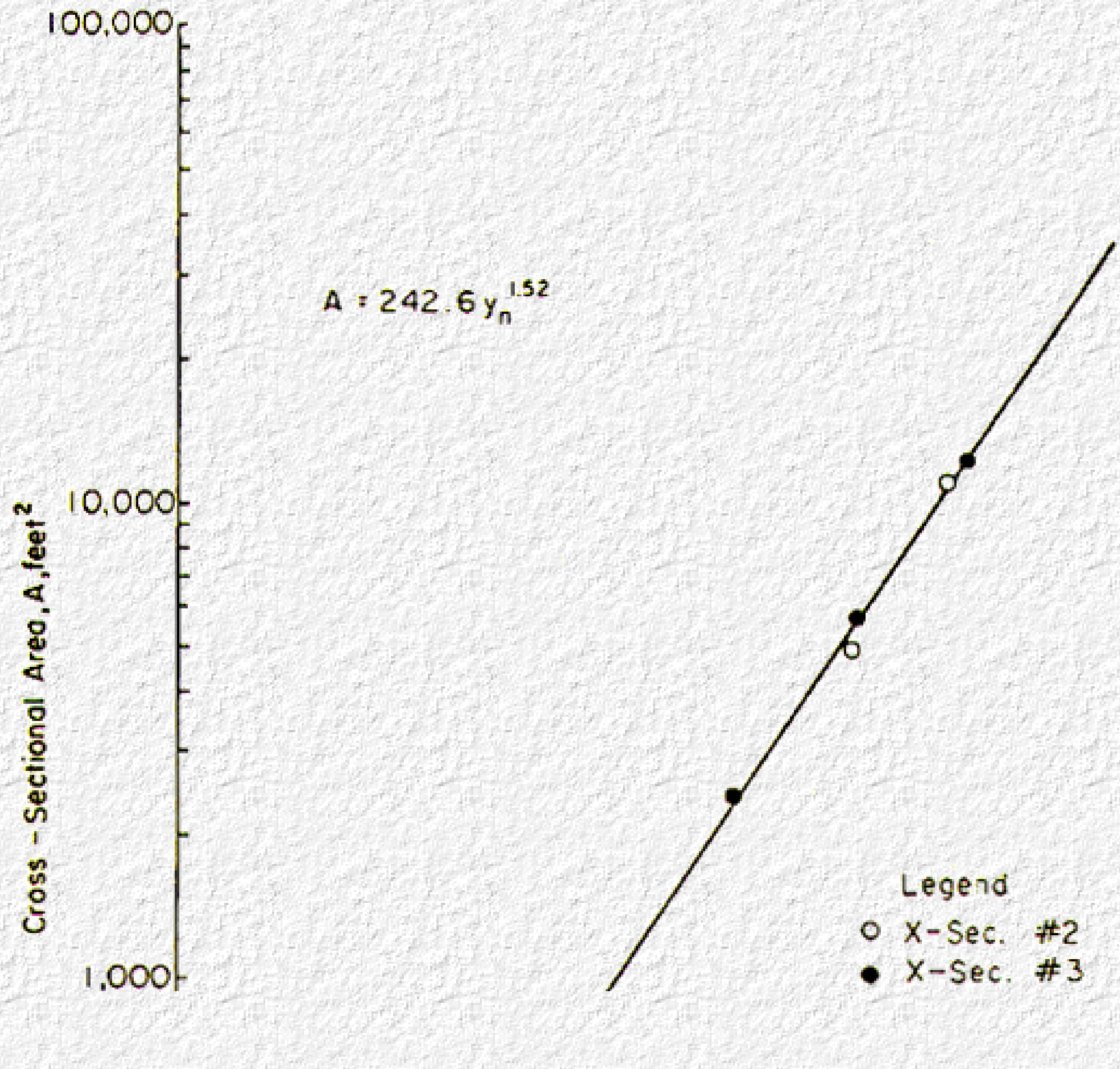
in which A is the cross sectional area and y_n is the normal depth of flow measured from the thalweg level.

Figure 7.10.4 gives the following relationship between the wetted perimeter and the flow depth.

$$P = 682.0\, y_n^{\ 0.31}$$

The average top width of channel as estimated by field survey and topographic maps is 1700 feet.

The results of sieve analyses of the bed material samples taken at cross sections are shown in Figure 7.10.5. The average median bed material size, D_{50}, is 0.45 mm. The average D_{16} size is 0.22 mm, and the average D_{84} size is 0.91 mm. The bank material has nearly the same distribution as that of the bed material, but contains lenses of silt and clay. The bank is highly stratified and can be easily eroded.



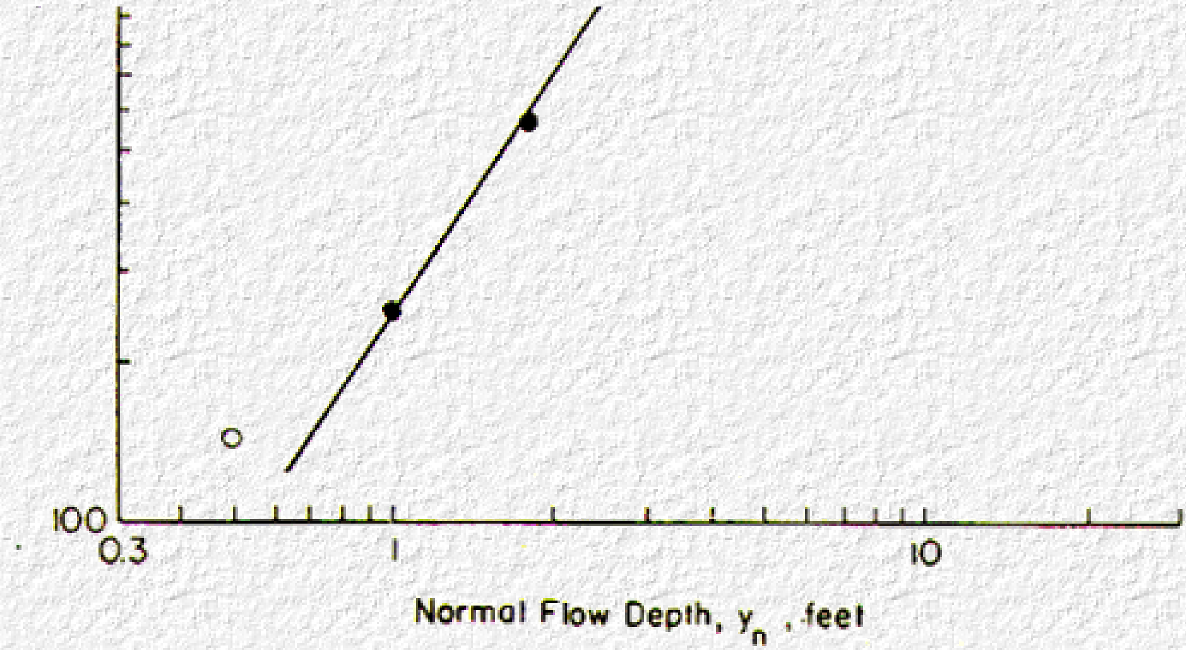


Figure 7.10.3. Cross Sectional Area versus Flow Depth Relation

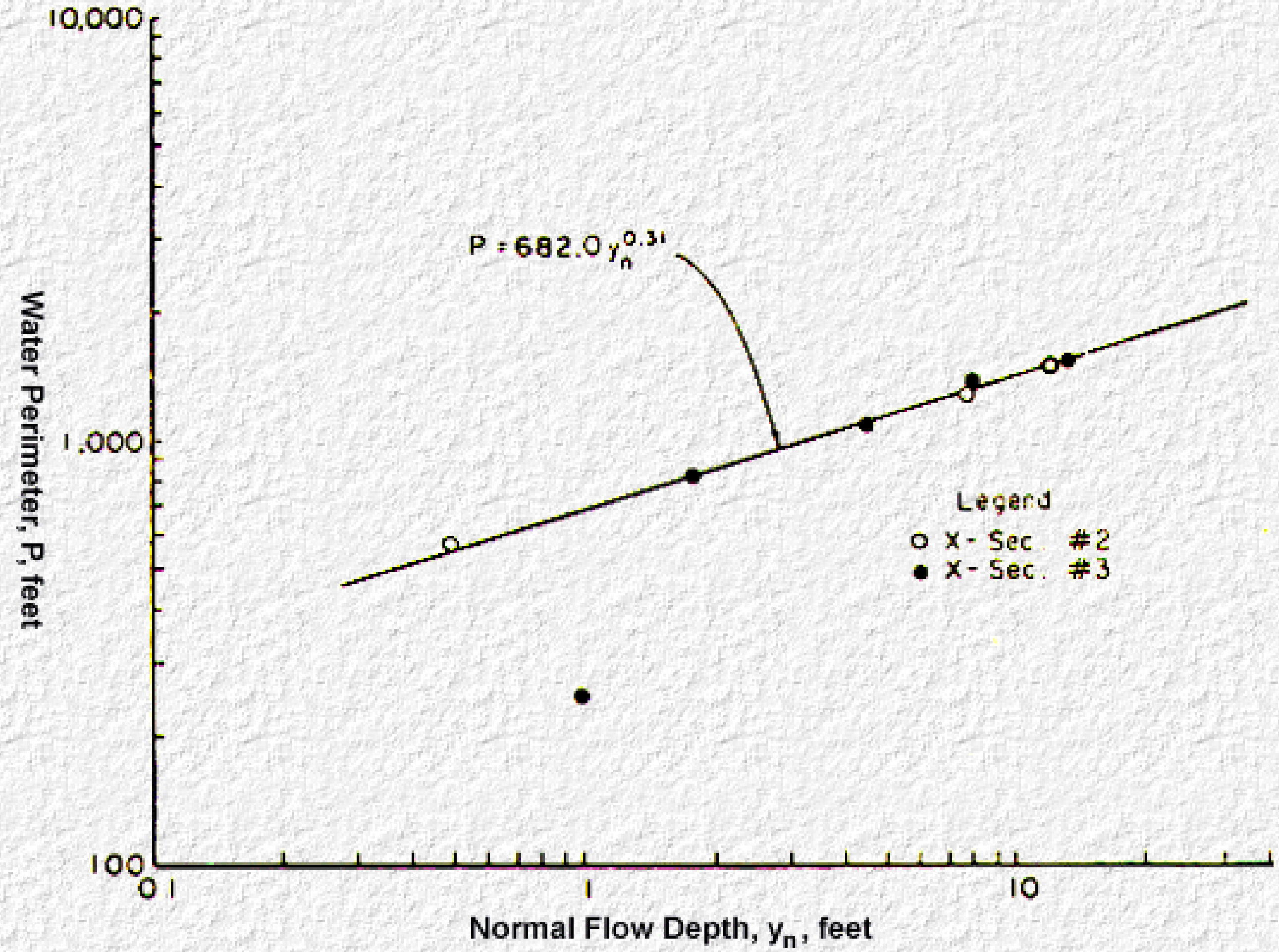


Figure 7.10.4. Wetted Perimeter versus Flow Depth Relation

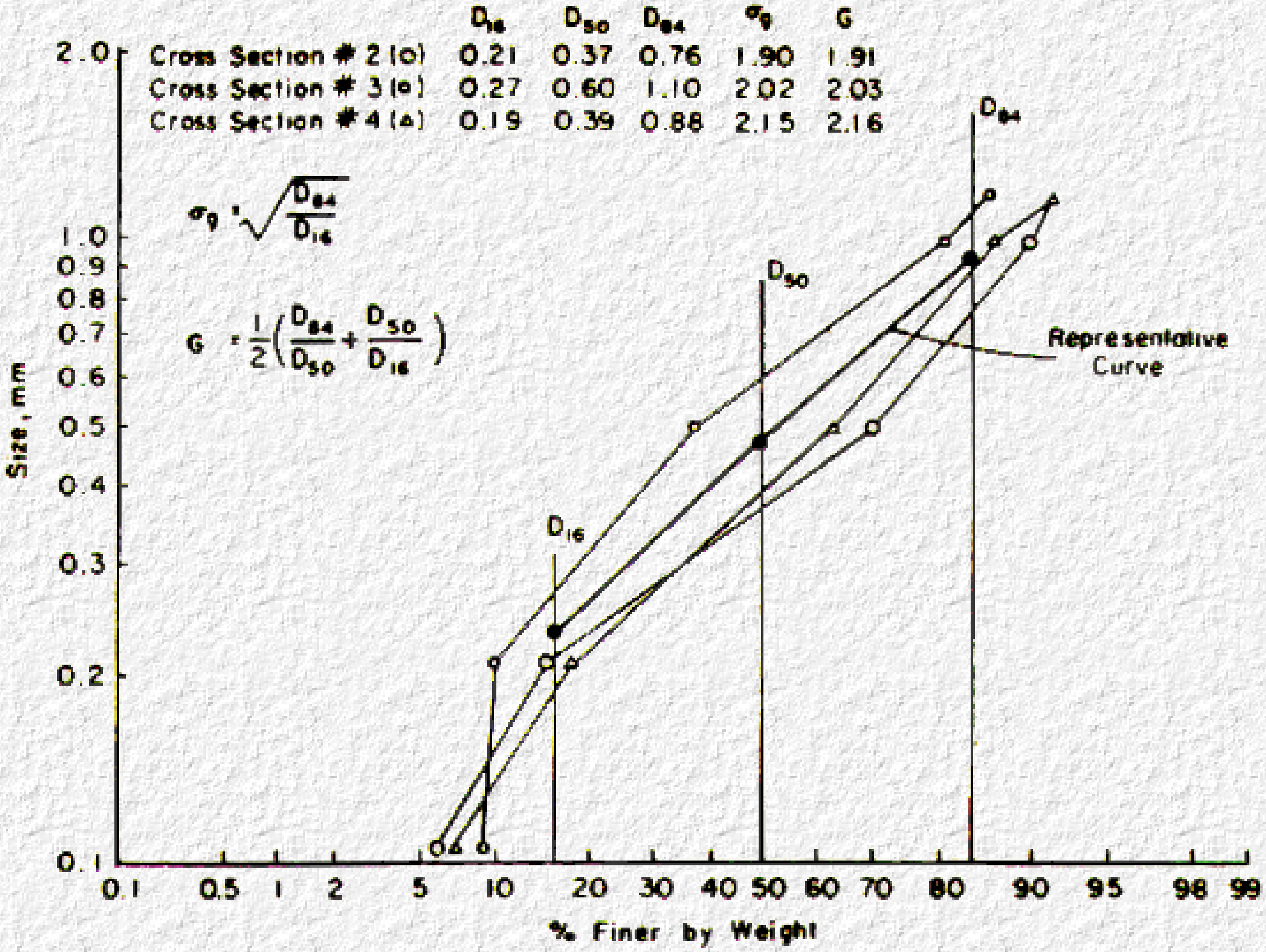


Figure 7.10.5. Analysis of Bed Material Size of Bijou Creek

The energy slope may be less than the channel bed slope. However, for a safer design, it is assumed that the energy slope is equal to the channel bed slope. The average channel bed slope from field surveys is 0.00252 ft/ft. This slope is adopted as the design energy slope.

The floods of June 1965 in South Platte River Basin, Bijou Creek were in upper regime, having antidunes with breaking waves. Similarly, computations show that the bed forms of Bijou Creek (D_{50} = 0.45 mm) will be antidunes or standing waves during floods. Resistance to flow associated with antidunes depends on how often the antidunes form, the area of the reach which they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as for a plane sand bed, and the discharge coefficient, $C/(g)^{1/2}$, (where C is the Chezy resistance coefficient and g is the gravitational acceleration) ranges from 14 to 23 (Manning's coefficient, n, is about 0.017 to 0.027 for the flow depths being considered). The acceleration and deceleration of the flow through the nonbreaking antidunes (frequently called standing waves) causes resistance to flow to be slightly more than that for flow over a plane bed. If many antidunes break, resistance to flow can be very large because the breaking waves dissipate a considerable amount of energy. With breaking waves, $C/(g)^{1/2}$ may range from 10 to 20, and Manning's coefficient n ranges from about 0.019 to 0.038 for the flow depths being considered.

From available data on the stage-discharge relation at the stream gaging station it is estimated that the Manning roughness coefficient, n, during the design floods is about 0.023. This value of Manning's n gives a computed mean flow velocity of 18.7 fps in Bijou Creek for the extreme flood of June 1965 (discharge 466,000 cfs).

7.10.3 Moveable Bed Hydraulic Analysis

The information on hydraulic conditions needed for designing bank protection includes the normal depth of flow, the cross sectional area of flow, the mean flow velocity, the Froude number, the bedform height, the local scour depth, the superelevation of the flow in the bend, the local depth, and the local velocity. Moreover, different design alternatives will result in different hydraulic conditions. In this study, five design alternatives were proposed; the first alternative was to protect the existing outer bank with riprap (see Figure 7.10.6), the second alternative was to realign the bend to its plan geometry before 1965 and to protect the outer bank with riprap (see Figure 7.10.7), and the third alternative was to realign the bend to that of a mild bend relative to the existing channel alignment and to protect the outer bank with riprap (see Figure 7.10.8). The fourth alternative was to determine the necessary buffer strip distance between the railroad and the present north river bank if bank protection is not utilized The fifth was to use rock riprap spur dikes to protect the existing river bank.

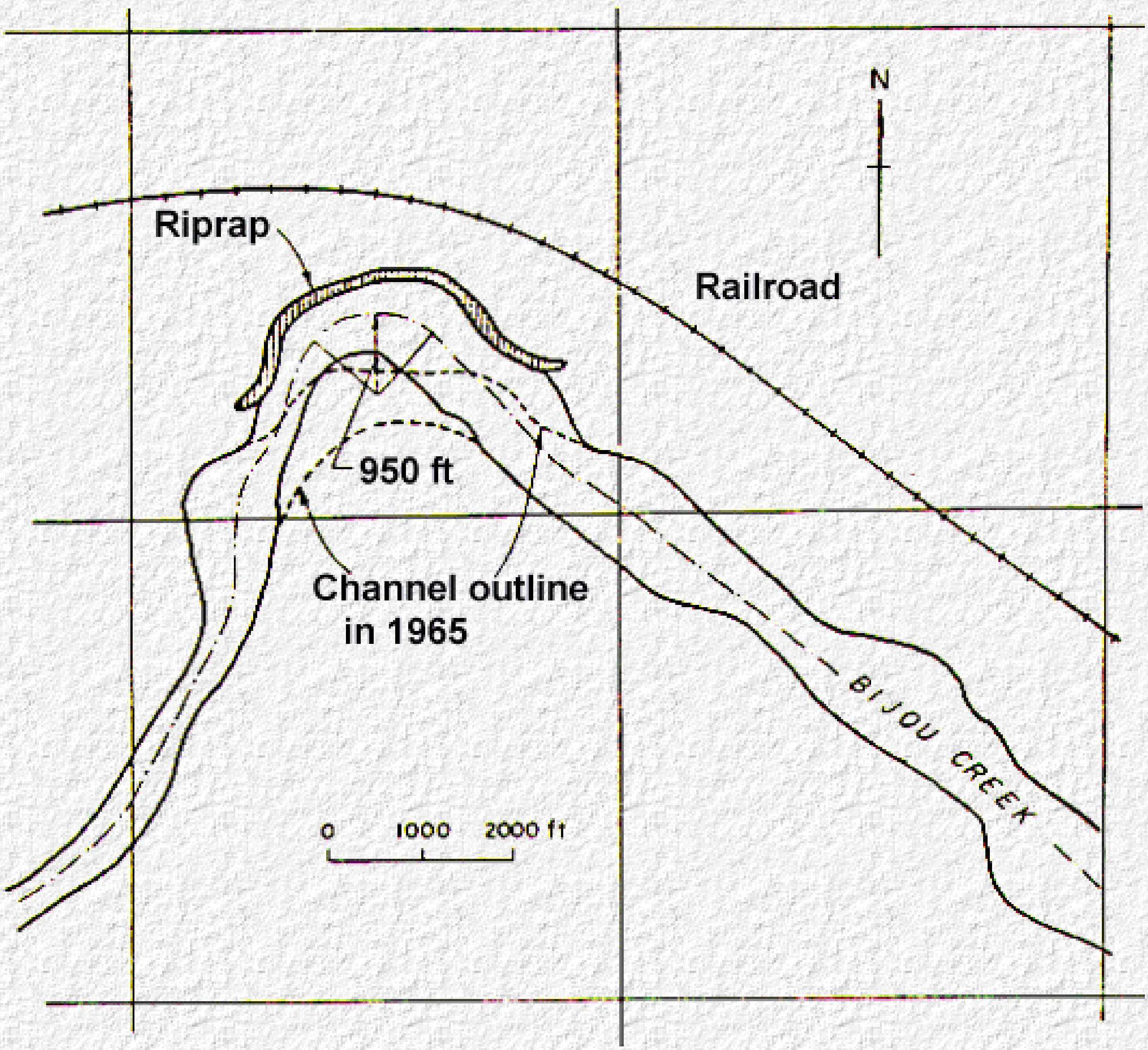


Figure 7.10.6. The Proposed First Alternative

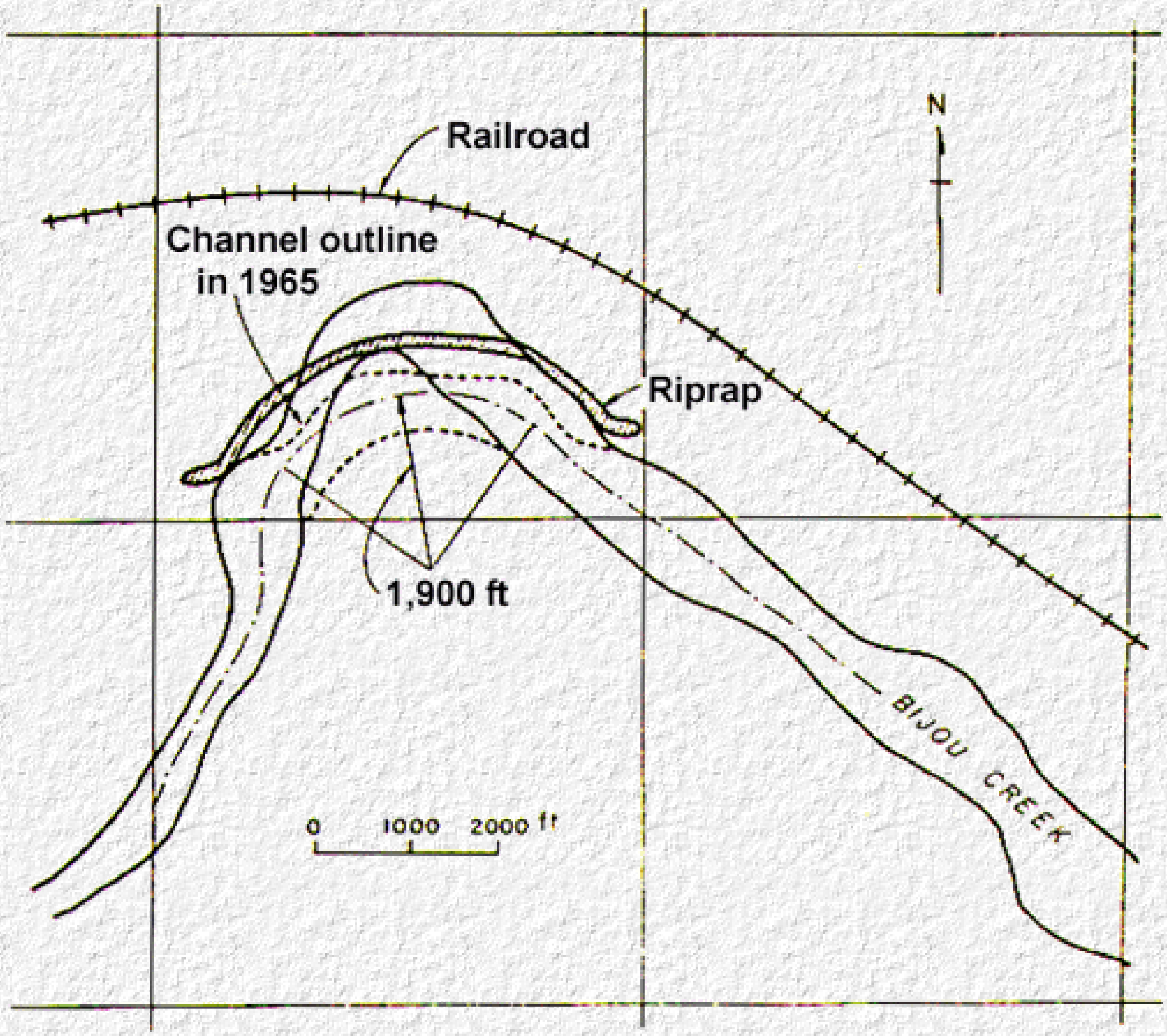


Figure 7.10.7. The Proposed Second Alternative

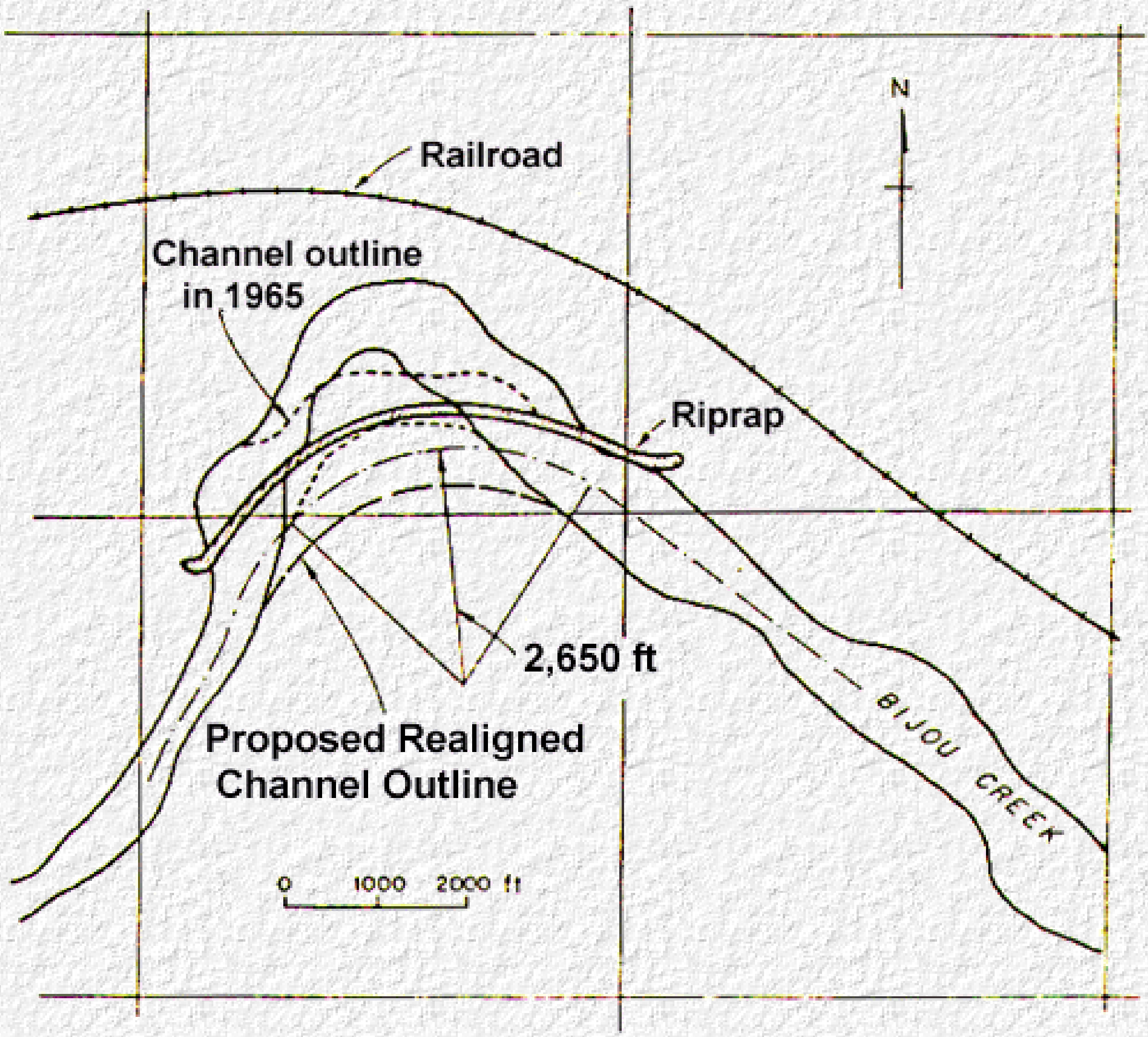


Figure 7.10.8. The Proposed Third Alternative

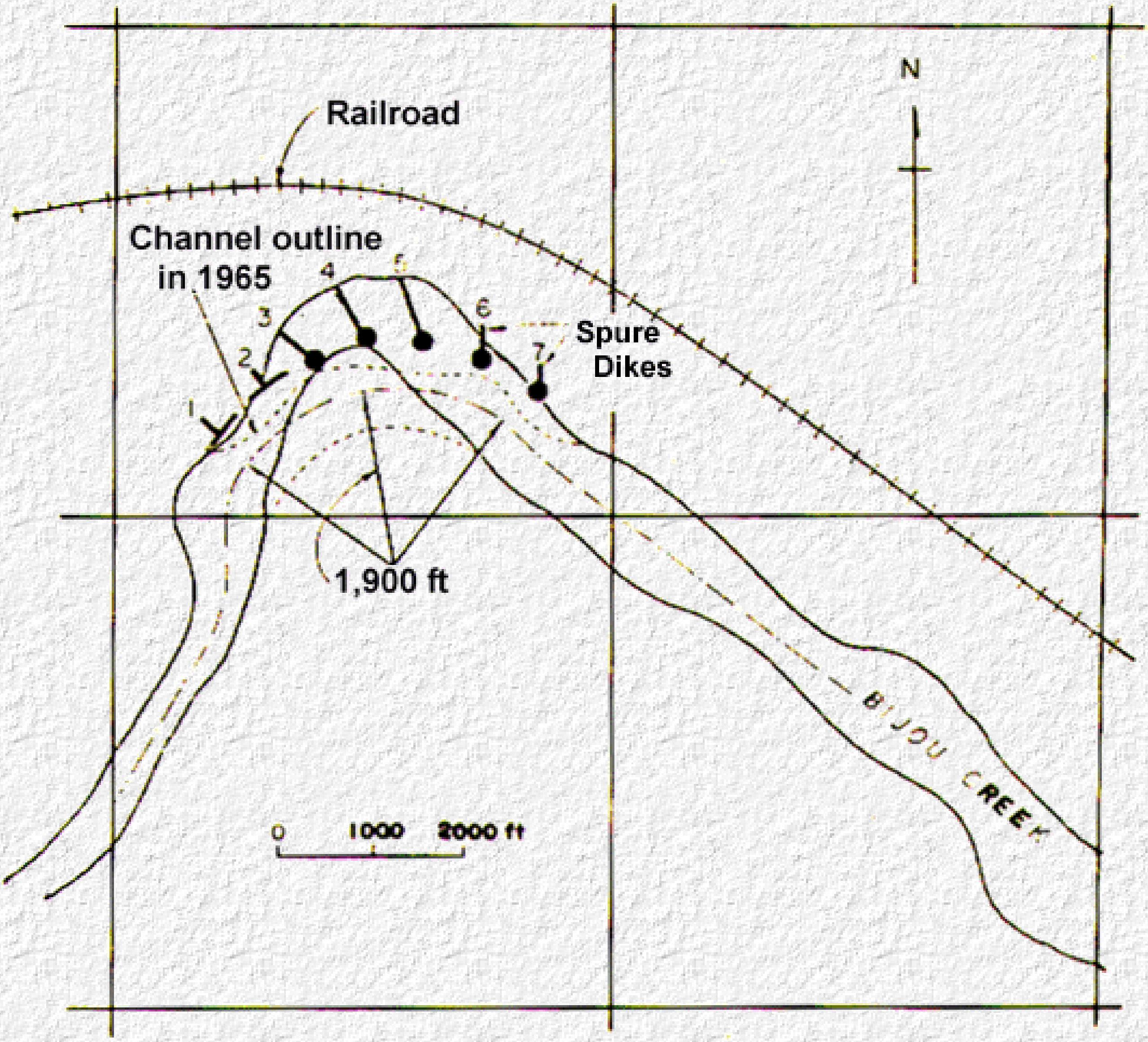


Figure 7.10.9. The Proposed Fifth Alternative

The fifth alternative, to construct a series of rock riprapped spur dikes to guide the flow away from the existing north bank (see Figure 7.10.9) has the same flow alignment as the second alternative (see Figure 7.10.7).

A summary of computed hydraulic conditions in the first three basic alternatives is given in Table 7.10.1. The calculations pertaining to determination of riprap size are briefly described.

The following normal depth-discharge relation is determined by using Manning's equation, using the slope of energy gradient S = 0.00252, and Manning's roughness coefficient n = 0.023.

$$Q = 395.02\, y_n^{2.32}$$

In this relation Q is the design discharge and y_n is the normal depth of flow measured from the thalweg level.

For design floods with return periods of 50, 100 and 200 years, the normal depths from thalweg level are determined. Then the cross sectional area and the wetted perimeter can be computed. Finally, the mean flow velocity, the hydraulic radius, and the Froude number are calculated.

As shown in Table 7.10.1, the average Froude numbers are approximately 0.74, which implies the maximum local Froude number in the center of the flow could be on the order of 1.0 to 1.2. According to Simons and Richardson (1971), these flow conditions should be in the upper regime with antidunes with breaking or nonbreaking waves. The estimated Manning's coefficient of 0.023 is correct for these flow conditions. In addition, the backwater effects are negligible because the flows are in the upper regime. The normal depth as computed should be satisfactory for design purposes.

There are many methods for determining riprap size (Simons and Senturk, 1977). Among them, the method developed by Stevens, et al. (1974) may be the most comprehensive and appropriate method to use. In applying their method, the depth of flow, the flow velocity and the angle between the horizontal and the velocity vector in the plane of the side slope are necessary as shown in Chapter 5.

Table 7.10.1 Summary of Computed Hydraulic Conditions

T yrs	Q cfs	Y_n ft	A ft	V fps	F	H ft	y_s ft	A_l	r_c ft	Δ_z ft	y_o ft	V_o fps
50	51,600	8.17	5,904	8.74	0.72	1.05	8.82	1	950	4.25	12.40	17.37
								2	1,900	2.12	10.28	15.30
								3	2,650	1.52	9.69	14.70
100	62,000	8.84	6,660	9.31	0.74	1.18	9.63	1	950	4.82	13.65	18.50
								2	1,900	2.41	11.20	16.20
								3	2,650	1.73	10.57	15.60
200	72,500	9.46	7,379	9.82	0.75	1.32	10.33	1	950	5.36	14.82	19.50
								2	1,900	2.68	12.14	17.11
								3	2,650	1.92	11.38	16.40

Note: T is the return period, Q is the design discharge, y_n is the normal depth from the thalweg level, A is the cross-sectional area of flow, V is the mean flow velocity, F is the Froude number, H is the antidune height, y_s is the scour depth at the leading portion of riprap bank protection, A_l is the alternative of designs, r_c is the radius at the center of a bend, D_z is the superelevation, and y_o and V_o are the depth and velocity, respectively, for designing riprap sizes.

As mentioned in Chapter 4, the forms of antidunes or standing waves constitute a series of inphase symmetrical sand and water waves. These waves are in the center of the channel. Thus, the depth of flow at the bank for designing riprap sizes is taken to be the sum of the normal depth from thalweg level and the superelevation, i.e., $y_o = y_n + \Delta z$; and the flow velocity for designing riprap size is computed by utilizing Manning's equation:

$$V_o = \frac{1.486}{n} y_o^{2/3} S_f^{1/2}$$

in which S_f is the energy slope.

The angle between the horizontal and the velocity vector in the plane of the side slope is assumed to be negligible.

Three design alternatives are considered in this study. Different hydraulic designs result for each of the three design alternatives and the three different design floods. The hydraulic design includes the determination of the total length of bank protection, the minimum buffer strip distance between the railroad and the river bank, the estimated volume of earthwork, the side slope of riprapped bank, the sizes of riprap material, the thickness of riprap, the size of the gravel filters, the thickness of the gravel filters, the height of riprap protection above the existing bed level, and the depth the riprap should extend below thalweg level. Using the hydraulic data, all these values can be computed. Table 7.10.2 provides a summary of the hydraulic designs for different design conditions. The methods of design are briefly described.

From Figure 7.10.6, Figure 7.10.7 and Figure 7.10.8 the length of bank protection, minimum buffer strip distance between the railroad and the river bank, and the excavation volumes for three different design alternatives can be estimated. The minimum buffer strip distance is a measure of how far the river bank must migrate due to bank erosion to endanger the railroad. A wider minimum buffer strip distance between Bijou Creek and the railroad will provide a larger factor of safety for the railroad. However, the length of bank protection and excavation volumes increase accordingly, which in turn increases the cost of construction.

Table 7.10.2 Summary of Hydraulic Designs

T	Q	A_ℓ	L	B_m	Ex 10^5	Z	K_{50}	G_K	t_r
yrs	cfs		ft	ft	yd^3		ft		ft
50	51,600	1 2 3	4,700 5,700 6,250	700 1,250 1,500	--- 6.26 9.42	2.5 2.5 2.5	1.5 1.1 1.0	1.3 1.3 1.3	3.3 2.4 2.2
100	62,000	1 2 3	4,700 5,700 6,250	700 1,250 1,500	--- 7.04 10.64	2.5 2.5 2.5	1.7 1.3 1.2	1.3 1.3 1.3	3.7 2.9 2.6
200	72,500	1 2 3	4,700 5,700 6,250	700 1,250 1,500	--- 7.85 11.91	2.5 2.5 2.5	2.0 1.4 1.3	1.3 1.3 1.3	4.4 3.1 2.9

T	Q	A_ℓ	f^1_{50}	G^1_f	t^1_f	f^2_{50}	G^2_f	t^2_f	h_a	h_b^l	h_b
yrs	cfs		mm		in	mm		in	ft	ft	ft
50	51,600	1 2 3	6.0 6.0 6.0	3.0 3.0 3.0	10.0 8.0 7.0	60.0 60.0 60.0	2.0 2.0 2.0	10.0 7.0 7.0	14.5 12.5 12.0	10.0 10.0 10.0	5.0 5.0 5.0
100	62,000	1 2 3	6.0 6.0 6.0	3.0 3.0 3.0	12.0 9.0 8.0	60.0 60.0 60.0	2.0 2.0 2.0	11.0 9.0 8.0	16.0 13.5 13.0	11.00 11.00 11.00	5.0 5.0 5.0
200	72,500	1 2 3	6.0 6.0 6.0	3.0 3.0 3.0	14.0 10.0 9.0	60.0 60.0 60.0	2.0 2.0 2.0	13.0 9.0 9.0	17.5 14.5 14.0	12.00 12.00 12.00	5.0 5.0 5.0

Note: T is the return period in years, Q is the design discharge, A_ℓ is the alternative of designs, L is the total length of riprap bank protection, B_m is the minimum buffer strip distance between the railroad and the riverbank, Ex is the estimated excavation volumes, Z is the ratio of the horizontal distance to the vertical distance for the sideslope of riprap bank, K_{50} is the design riprap size for which 50 percent is finer by weight, G_K is the gradation coefficient of riprap, t_r is the thickness of riprap, f^1_{50}, G^1_f, and t_f^2 are, respectively, the gravel size for which 50 percent is finer by weight, the gradation coefficient and the thickness of the first layer of gravel filter, f^2_{50}, G^2_{50}, and t^2_f are, respectively, the gravel size for which 50 percent is finer by weight, the gradation coefficient and the thickness of the second layer of gravel filter, h_a is the design height of riprap protection above the existing bed level, and h_b^l and h_b are, respectively, design riprap depths in and not in the leading portion of the bank protection.

- **Side Slope of Riprap Bank**

The side slope of the riprapped bank should be less than the angle of repose of the bank material. Analysis shows that the angle of side slope should be at least five degrees less than the angle of repose of the bank material. The bank material is medium sand with some clay and silt with average D_{50} of about 0.35-0.45 mm. Its angle of repose is about 29° (see Chapter 5). A side slope 2.5:1 (horizontal to vertical distance) is utilized and the corresponding angle of side slope is 21.75° (see Figure 7.10.10).

- **Riprap Design**

The size of riprap is determined using the method presented in Chapter 5. This is judged to be the most comprehensive and appropriate method available. The method developed by the Bureau of Reclamation (1958) to determine the maximum rock size in a riprap mixture is used for comparison. The flow conditions required for designing riprap sizes were discussed earlier and given in Table 7.10.1.

Using a stability factor of 1.3 and an angle of repose of riprap material of 41°, the required riprap sizes with a uniform gradation are determined. The results are shown in Table 7.10.3.

It is not acceptable to riprap a bank with uniform size rock, especially when large rock is required. Therefore, it is required to design a riprap mixture that includes an adequate range of sizes. In addition, the stability factor for determining the D_{50} of the riprap mixture is determined by assuming the stability factor of 1.1. According to Chapter 5, riprap gradation should follow a smooth size distribution curve with gradation coefficient of about 1.3. Riprap sizes are given in Table 7.10.3.

The Bureau of Reclamation (1958) developed a figure to determine the maximum rock size in a riprap mixture downstream from stilling basins. If the bottom velocity is assumed equal to the reference velocity on the top of the rock, the maximum rock sizes in the riprap mixtures for different design conditions can be estimated. The results utilizing both methods are given in Table 7.10.3 for comparison. The sizes obtained by use of the Stevens method given in Chapter 5 are recommended.

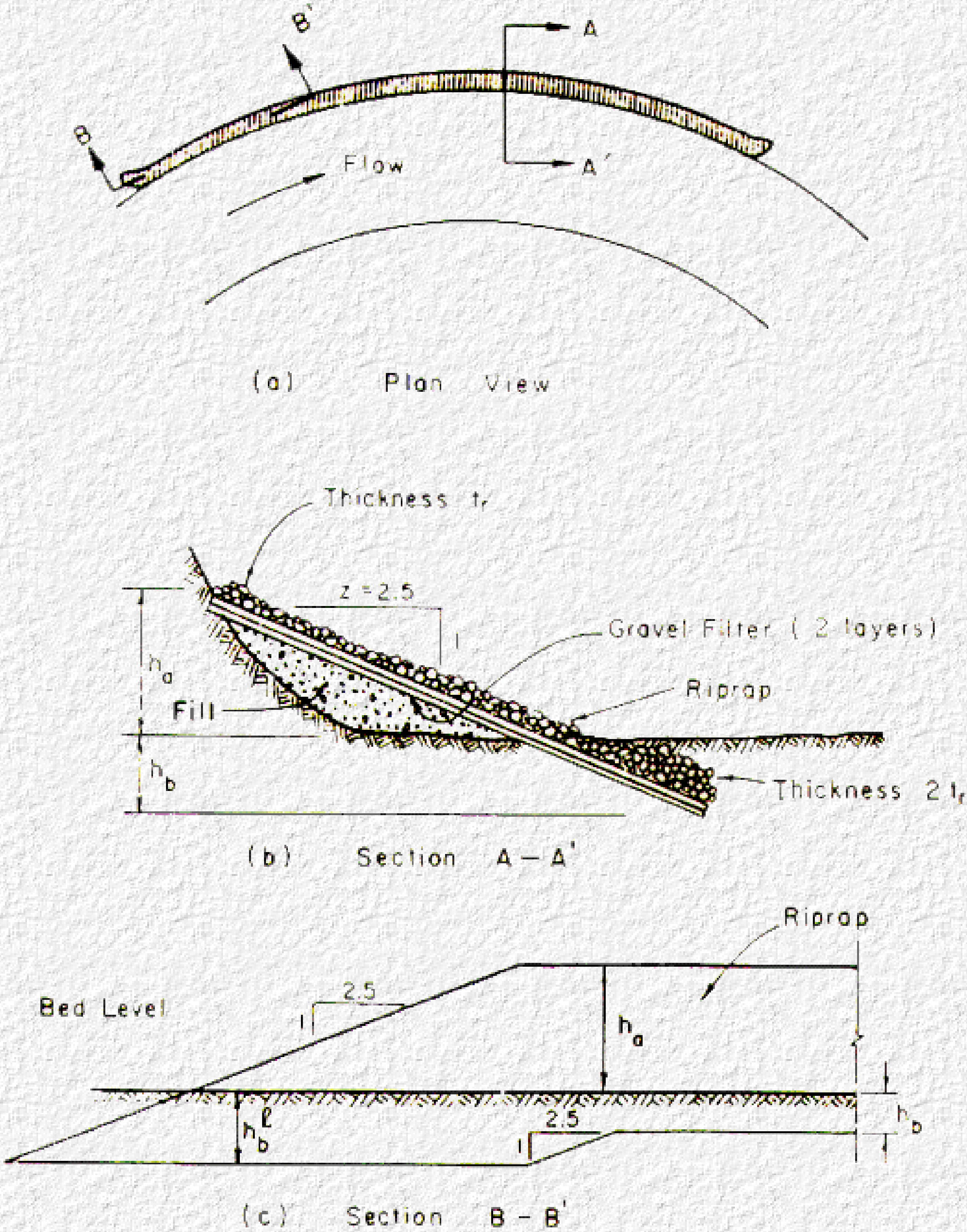


Figure 7.10.10. The Sketch of Proposed Riprap Design

Table 7.10.3 Design of Riprap Sizes

Return			Uniform Size by	Riprap Mixture by Stevens et al, (1974)	Maximum

Period (yrs)	Discharge (cfs)	Design Alternatives	Stevens et al., (1974) (ft)	Median Diameter (ft)	Maximum Size (ft)	Size by USBR (1958) (ft)
50	51,600	1 2 3	2.6 2.0 1.8	1.5 1.1 1.0	3.3 2.4 2.2	2.6 2.0 1.8
100	62,000	1 2 3	3.0 2.2 2.0	1.7 1.3 1.2	3.7 2.9 2.6	3.1 2.2 2.0
200	72,500	1 2 3	3.4 2.5 2.2	2.0 1.4 1.3	4.4 3.1 2.9	3.5 2.5 2.3

- **Gravel Filter Design**

  Filters should be placed under the riprap unless the material forming the core of the structure is coarse gravel or of such a mixture that it forms a natural filter. Two types of filters are commonly used: gravel and fabric filters.

  The sizes of gravel in the filter layer are calculated as explained in Chapter 5 and the results are given in Table 7.10.2. Two layers of filter material are required because the riprap sizes are large.

  Thickness of the filters may vary depending on the riprap thickness, but should not be less than six to nine inches. Filters that are one-half the thickness of the riprap are satisfactory and provide a great degree of safety.

- **Height and Depth of Riprap**

  The design height of riprap protection above the existing bed level (see Figure 7.10.10) must provide for freeboard, water depth, superelevation and wave height. The normal depth from the thalweg level and the superelevation were determined earlier. Because the forms of antidunes are a series of inphase symmetrical sand and water waves, the wave height at the water surface can be assumed equal to the antidune height. The minimum height required for riprap protection above the existing bed level can be determined by considering normal depth, superelevation, and antidune height. In order to provide additional protection against the breaking waves, an extra foot of freeboard is added to the required minimum height of riprap above bed level.

  The riprap (see Figure 7.10.10) must extend some distance below the thalweg level to provide safety against possible local scour, general scour, and troughs of passing sand waves. The general scour is assumed to be negligible because the proposed structures do not significantly constrict the flow. Some local scour is expected at the leading portion of the riprap revetment. Hence, a larger depth of riprap is required at this location. The minimum depth of riprap below thalweg level can be determined by considering the potential scour and the antidunes height.

  In general, the riprap should extend a minimum of five feet below thalweg level in order to protect against possible long-term degradation of the river reach. If the computed depth of riprap protection below thalweg level is less than five feet, the design depth is set at five feet.

  The fourth alternative is simply to provide sufficient buffer strip distance between the railroad and the existing river bank so that bank protection may not be required. The distances the north bank will migrate under different design flood conditions can be evaluated by utilizing sediment transport rates and the migration history of the 1965 flood. The estimated bank migration distances for the design floods are given in Table 7.10.4. In order to provide extra protection against slope failures and long-term bank migration due to smaller sizes of floods, the design buffer strip distances should be at least twice the computed migration distances (factor of safety - 2.0) because the accumulated bank migration distance due to smaller sizes of floods can be very significant.

The fifth alternative is to construct a series of rock riprapped spur dikes to guide the flow away from the existing north bank (see Figure 7.10.9). The design of the spurs includes: the form of spurs, the angle of spurs to the bank, the length of spurs, the spacing between spurs, the height or elevation of spurs, the construction materials, the crest width and slopes, and the local scour. The design of the spurs is summarized in Table 7.10.5, Table 7.10.6, and Table 7.10.7. Table 7.10.5 gives a summary of spur dike designs for different design floods including buffer strip distance, number of spurs, spacing of spurs, height of spur above the existing thalweg level, riprap size at the spur nose, and riprap size in the spur shank. Table 7.10.6 provides the suggested design dimension of spurs including spur form, spur length, angle of spur to bank, the portion requiring riprap the same size as used on the nose, and length requiring shank protection. Table 7.10.7 summarizes the sizes of riprap and filter design at the nose of the spurs and for the spur shanks.

Table 7.10.4 Bank Migration for Alternative Four

Return Period (yrs)	Discharge (cfs)	Bank Migration Distance (ft)	Design Buffer Strip Distance* (ft)
50 100 200	51,600 62,000 72,500	562 629 715	1124 1258 1430
*The eroded bank should be restored to its present alignment after each flood to maintain an adequate buffer zone.			

Figure 7.10.11 shows the suggested spur dike design. The methods of design are briefly described.

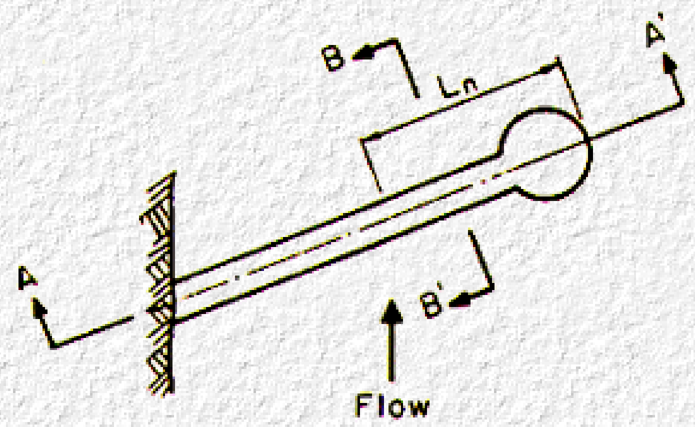
In Figure 7.10.9, Spurs No. 1 and 2 are T-head spurs, and the others are round-head spurs. The angle of spur to the bank is usually 60° to 120°. The available literature shows that the angle for T-head spurs is normally 90° but the angles for round spurs varies. Mamak (1964) states that the best results for deflecting flow and trapping sediment load are obtained with spurs inclined upstream from 100° to 110°. However, the study by Franco (1967) showed that for channelization the normal or angled downstream spurs (60°) performed better than the angled upstream spurs. Judging from the purpose of spurs and flow conditions being considered, it is determined that the angles of round spurs should be constructed about 70° to the bank angled downstream as shown.

The length of a spur depends on its location, amount of contraction of stream width, and purpose of the spur. The purpose of the spurs considered in this study is to guide the flow away from the bank and to provide a flow alignment similar to that of 1965. The lengths of spurs are determined to serve this purpose (see Figure 7.10.9 and Table 7.10.7).

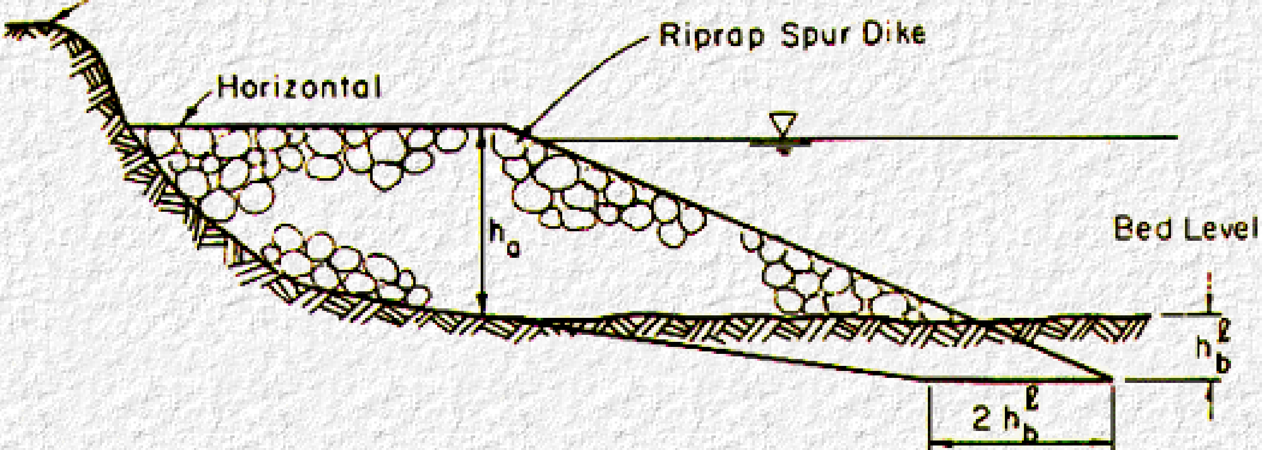
Table 7.10.5 Summary of Spur Dike Design

T yrs	Q cfs	B_m ft	N_s	S_d ft	h_a ft	h^{ℓ}_b ft	K^n_{50} ft	K^s_{50} ft
50	51,600	1,250	7	600	12.5	10.0	1.1	0.6
100	62,000	1,250	7	600	13.5	11.0	1.3	0.7
200	72,500	1,250	7	600	14.5	12.0	1.4	0.7

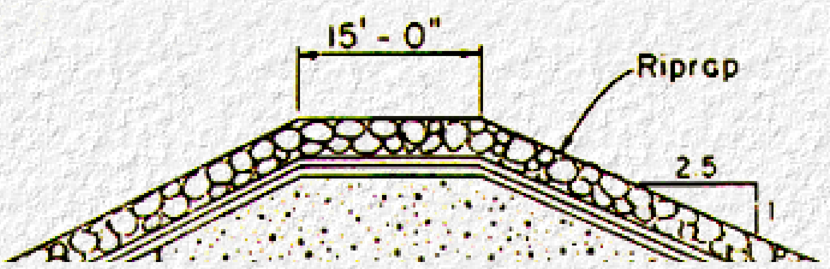
Note: T is the flood return period in years, Q is the design discharge, B_m is the minimum buffer strip distance between the railroad and the spur nose, N_s is the number of spurs, S_d is the spur spacing, h_a is the design height of spur above the existing thalweg level, h^{ℓ}_b is the design depth of spur at the spur nose below the existing thalweg level, K^n_{50} is the median size of riprap at the spur nose, and K^s_{50} is the size of riprap for the spur shank.

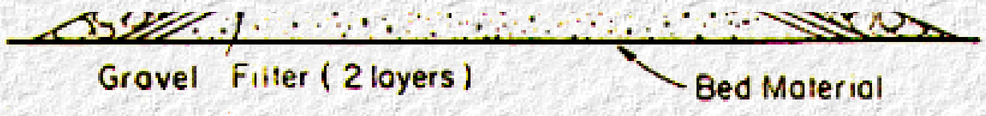


(a) Plan View



(b) Section A – A'





(c) Section B – B'

Figure 7.10.11. Sketch of Spur Dike Design

Table 7.10.6 Dimension of Spur Dikes

Spur No.	Spur Form	Spur Length (ft)	T-Head Length (ft)	Angle of Spur to Bank (degrees)	Length Requiring Nose Riprap Size (ft)	Length Requiring Shank Protection (ft)
1	T-head	200	400	90	20	180
2	T-head	200	400	90	20	180
3	Round head	500	N/A	70	200	300
4	Round head	750	N/A	70	200	550
5	Round head	750	N/A	70	200	550
6	Round head	400	N/A	70	200	200
7	Round head	200	N/A	70	200	0

Table 7.10.7 Size and Filter Design for Riprap at Spur Noses and Spur Shanks

T yrs	Q cfs	K_{50} ft	G_K	t_r ft	f^1_{50} mm	G^1_f	t^1_f in	f^2_{50} mm	G^2_f	t^2_f in
(a) At Spur Noses										
50	51,600	1.1	1.3	2.4	6.0	3.0	8.0	60.0	2.0	7.0
100	62,000	1.3	1.3	2.9	6.0	3.0	9.0	60.0	2.0	9.0
200	72,500	1.4	1.3	3.1	6.0	3.0	10.0	60.0	2.0	9.0
(b) At Spur Shank										
50	51,600	0.6	1.3	1.3	6.0	3.0	6.0	20.0	2.0	6.0
100	62,000	0.7	1.3	1.5	6.0	3.0	6.0	20.0	2.0	6.0
200	72,500	0.7	1.3	1.5	6.0	3.0	6.0	20.0	2.0	6.0

Note: T is the return period, Q is the design discharge, K_{50} is the design riprap size for which 50 percent is finer by weight, G_K is the gradation coefficient of riprap, t_r is the thickness of riprap, f^1_{50}, G^1_f, and t^1_f are, respectively, the gravel size for which 50 percent is finer by weight, the gradation coefficient, and the thickness of the first layer of gravel filter, and f^2_{50}, G^2_f, and t^2_f are, respectively, the gravel size for which 50 percent is finer by weight, the gradation coefficient, and the thickness of the second layer of gravel filter.

The spacing between spurs is primarily related to the length of the spur. In general, the recommended spacing is from one and one-half to six times that of the upstream projected spur length into the flow. For bank protection in a sharp bend, a smaller spacing should be used. For the design conditions, the spacing of spurs is taken as 600 ft.

The height or elevation of a spur is determined by considering the maximum flow depth above thalweg level. In order to provide additional protection against the breaking waves, an extra foot of freeboard was added to the design height of each spur above the existing thalweg level.

The local scour depth at the spur nose is the same as that at the leading portion of the continuous riprap revetment. The computed local scour depths for different design floods are given in Table 7.10.2. The minimum depth of riprap below thalweg level is determined by considering the local scour depth and the antidune heights, which also gives the design depth of spur at the spur nose below the existing thalweg level.

The construction materials are shown in Figure 7.10.11. The riprap design and the gravel filter design at the spur nose are the same as those used in the continuous revetment. The length requiring nose riprap size can be estimated by considering the flow separation zone. The riprap size in the shank inside this zone may use a smaller rock size. A reduction of fifty percent in rock size is determined by considering the decrease in flow velocity. The riprap at the downstream side of shank may be eliminated if a larger risk is accepted. This is because the downstream side of the shank is not expected to be subjected to a strong velocity. However, it may be subject to scour due to overtopping.

The crest width of rock riprap spurs usually ranges from three to 20 feet and the side slope from 1.25:1 to 5:1. Considering the convenience in hauling and placing rock riprap, the crest width of spurs is determined to be 15 ft. The side slope was determined to be 2.5:1.

A second set of alternatives would result from replacing the rock riprap with soil-cement riprap in design alternatives 1, 2, and 3. This alternative may be of interest because large rocks can be very difficult to obtain and soil-cement riprap may be manufactured at the site without much difficulty.

7.11 Concluding Remarks

This manual illustrates the many interactions between rivers and highway structures. Rivers are dynamic, moving back and forth across floodplains, lying dormant through years of low flows and then breaking out of their banks to recarve the form of the immediate landscape, including, at times, the highway structures.

The dynamic features of rivers and river systems and the natural beauty of the river scenery make the design of highways in the river environment one of the most challenging and stimulating of all engineering designs.

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Appendix 2 : HIRE

Solved Problems Chapter 2

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Problem 2.1 Evaluation of the Correction Factors α and β

Calculate the correction factors α and β for a cross-section given the discharge measurement during the peak flood event for the year. From Table 2.A1.1, the following values are obtained:

Q	=	5370 cfs
A	=	1485 sq ft
W	=	163 ft
$\Sigma V_i \Delta Q_i$	=	21,070 ft^4/sec^2
$\Sigma V_i^{\,2} \Delta Q_i$	=	85,500 ft^5/sec^3

The bed material at this gaging station[1] has a D_{50} of 0.33 mm and a D_{65} of 0.45 mm and a gradation coefficient G of 3.27. If the value of D_{65} is used for k_s, then for y_o = 12.8 ft (the maximum depth)

$$\frac{y_o}{k_s} = \frac{12.8}{0.45}(304.8) \simeq 8700$$

and for y_o = 1.1 ft (the smallest non-zero depth)

$$\frac{y_o}{k_s} = \frac{1.1}{0.45}(304.8) \simeq 750$$

Table 2.A1.1 Discharge Measurement Notes[1]

y_{oi} ft	Dz_i ft	V_i fps	DA_i sq ft	DQ_i cfs	V_iDQ_i ft^4/sec^2	$V_i^2DQ_i$ ft^5/sec^3
0.0	4.0	0.00	0.0	0.00	0.00	0.00
1.1	8.0	0.98	8.8	8.62	8.45	8.28
2.6	8.0	0.54	20.8	11.23	6.06	3.27
4.5	8.0	0.64	36.0	23.04	14.75	9.44
8.5	8.0	2.40	68.0	163.20	391.68	940.03
11.0	8.0	3.17	88.0	278.96	884.30	2803.24
11.6	8.0	4.02	92.8	373.06	1499.70	6028.80
12.0	8.0	4.06	96.0	389.76	1582.43	6424.65
12.8	8.0	3.78	102.4	387.07	1463.12	5530.61
12.6	8.0	3.74	100.8	376.99	1409.94	5273.19
12.4	8.0	3.78	99.2	374.98	1417.42	5357.86
11.6	8.0	4.71	92.8	437.09	2058.69	9696.45
11.4	8.0	4.30	91.2	392.16	1686.29	7251.04
10.8	8.0	4.90	86.4	423.36	2074.46	10164.87
10.6	8.0	4.63	84.8	392.62	1817.83	8416.56
10.9	8.0	4.32	87.2	376.70	1627.34	7030.13
11.4	8.0	3.89	91.2	354.77	1380.06	5368.42
11.8	8.0	3.10	94.4	292.64	907.18	2812.27
9.8	8.0	3.02	78.4	236.77	715.05	2159.44
6.4	7.0	1.69	44.8	75.71	127.95	216.24
3.8	5.5	0.00	20.9	0.00	0.00	0.00
0.0	2.5	0.00	0.0	0.00	0.00	0.00
TOTAL	163.0		1484.9	5368.74	21072.71	85494.77

1 Simons, D. B., Richardson, E. V., Stevens, M. A., Duke, J. H., and Duke, V. C., Stream flow, groundwater and ground response data, Hydrology Report, Vol. II, Venezuelan International Meteorological and Hydrological Experiment, Civil Engineering Dept., Colorado State University, August 1971.

If we use a mean y_o/k_s of approximately 5000, then from Figure 2.3.7, the average values for the energy and momentum coefficients are

α' = 1.024

and

β' = 1.008

As it has been assumed that α' and β' are constant across the river (for convenience), Equation 2.3.42 and Equation 2.3.43 become

$$\alpha = \alpha' \frac{A^2}{Q^3} \sum_i V_i^2 \Delta Q_i$$

and

$$\beta = \beta' \frac{A}{Q^2} \sum_i V_i \Delta Q_i$$

With the values in Table 2.A1.1

$$\alpha = \frac{1.024(1485)^2}{(5370)^3}(85500) = 1.247$$

$$\beta = 1.008 \frac{(1485)}{(5370)^2}(21070) = 1.094$$

These values for α (1.247) and β (1.094) differ from unity by appreciable amounts. The difference may be important in many river channel calculations. If no data are available, the assumptions that α = 1.25 and β = 1.1 should be used for river channels.

Problem 2.2 Superelevation in Bends

Calculate the superelevation of the water surface in a river bend given the velocity profile from Table 2.A1.1. The river radius of curvature r_i is measured equal to 350 feet and the outer radius of curvature r_o is 513 feet. The detailed calculations based on Equation 2.6.6 are presented in Table 2.A1.2.

$$\Delta z = \frac{1}{g} \sum_{r_i}^{r_o} \frac{V^2}{r} \Delta r$$

The total superelevation is 0.133 feet.

Based on approximate relationships, $\overline{V}$ = Q/A = 3.61 ft/s, we obtain:

- Woodward's method (Equation 2.6.8)

$$\Delta Z = \frac{\overline{V}^2}{g r_c}(r_o - r_i) = \frac{(3.61)^2}{(32.2)(431)}(513 - 350) = 0.153 \text{ ft}$$

- Ippen and Drinker's method (Equation 2.6.10)

$$\Delta Z = \frac{\overline{V}^2}{2g} \frac{2W}{r_c}\left[\frac{1}{1 - \left(\frac{W}{2r_c}\right)^2}\right] = \frac{3.61^2}{2 \times 32.2} \times \frac{2 \times 163}{431}\left[\frac{1}{1 - \left(\frac{163}{2 \times 431}\right)^2}\right] = 0.158 \text{ ft}$$

From Equation 2.6.11

$$\Delta Z = \frac{\overline{V}^2}{2g} \frac{2W}{r_c}\left\{\frac{1}{1 + \frac{W^2}{12 r_c^2}}\right\} = \frac{(3.61)^2}{2 \times 32.2} \times \frac{2 \times 163}{431}\left\{\frac{1}{1 + \frac{(163)^2}{12 \times (431)^2}}\right\} = 0.151 \text{ ft}$$

From combining free and forced vortices with V_{max} = 4.90 ft/s, Equation 2.6.13 gives

$$\Delta Z = \frac{V_{max}^2}{2g}\left\{2-\left(\frac{r_i}{r_c}\right)^2-\left(\frac{r_c}{r_o}\right)^2\right\} = \frac{4.90^2}{2\times 32.2}\left\{2-\left(\frac{350}{431}\right)^2-\left(\frac{431}{513}\right)^2\right\} = 0.236\ \text{ft}$$

As a result all the methods give comparable results ranging from 0.133 to 0.236 ft. Approximate methods slightly over-estimate the superelevation of the water surface.

Table 2.A1.2 Detailed Computation of Superelevation in Bends

Δr	r_i	V_i	$\Delta Z_i = \frac{V_i^2}{g r_i}\Delta r_i$
ft	ft	ft/s	ft
4.0	352	0.00	.0000
8.0	358	0.98	.0007
8.0	366	0.54	.0002
8.0	374	0.64	.0003
8.0	382	2.40	.0037
8.0	390	3.17	.0064
8.0	398	4.02	.0101
8.0	406	4.06	.0101
8.0	414	3.78	.0086
8.0	422	3.74	.0082
8.0	430	3.78	.0083
8.0	438	4.71	.0126
8.0	446	4.30	.0103
8.0	454	4.90	.0131
8.0	462	4.63	.0115
8.0	470	4.32	.0099
8.0	473	3.89	.0079
8.0	486	3.10	.0049
8.0	494	3.02	.0046
7.0	502	1.69	.0012
5.5	507	0.00	.0000
2.5	512	0.00	.0000
		Total ΔZ =	0.1330 ft.

Problem 2.3 Standard Step Method for Backwater Computation

Calculate the water surface elevation at an abrupt change in slope.

Consider an abrupt change of slope in a 100 ft. wide channel with a discharge of 5000 cfs. Upstream of the slope change, the flow is at the normal depth of 10 ft. The normal depth in the downstream reach is 3 ft. Manning's n for both reaches is 0.012.

In both the upstream and downstream reaches, the flow per unit width is

$$q = \frac{5000}{100} = 50\ \text{cfs/ft}$$

and the critical depth is (from Equation 2.5.7)

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{50}{32.2}\right)^{1/3} = 4.27\ \text{ft}$$

Upstream where the flow is at normal depth, $y = y_o > y_c$, the flow is tranquil.

The bed slope is obtained from Manning's equation for normal flow (from Equation 2.3.20a)

$$S_o = \frac{n^2 V_o^2}{2.21 R_o^{4/3}}$$

Here

$$V_o = \frac{q}{y} = \frac{50}{10} = 5.00\ \text{fps}$$

$$A_o = y_oW = 10\ (100) = 1000 \text{ sq ft}$$

$$P_o = 2y_o + W = 2\ (10) + 100 = 120 \text{ ft}$$

$$R_o = \frac{A}{P} - \frac{1000}{120} = 8.33 \text{ ft}$$

$$S_o = \frac{(0.012 \times 5)^2}{2.21(8.33)^{4/3}} = 0.000095$$

Downstream where the flow has attained its normal depth, $y = y_o < y_c$, the downstream reach flow is supercritical.

The bed slope in the downstream reach is obtained as follows:

$$V_o = \frac{q}{y} = \frac{50}{3} = 16.67 \text{ fps}$$

$$A_o = y_oW = 3 \times 100 = 300 \text{ sq ft}$$

$$P_o = 2y_o + W = 2\ (3) + 100 = 106 \text{ ft}$$

$$R = \frac{A}{P} = \frac{300}{106} = 2.83 \text{ ft}$$

$$S_o = \frac{n^2V_o^2}{2.21R_o^{4/3}} = \frac{(0.012 \times 16.67)^2}{2.21(2.83)^{4/3}} = 0.004523$$

At the change in slope the flow must pass through critical depth. Then, in the reach immediately upstream, $y_o > y > y_c$ so that the backwater curve in this reach is an M2 type (Table 2.7.1).

Downstream, $y_o < y < y_c$ so the backwater curve in this reach is a S2 type.

The two backwater curves are sketched in Figure 2.A1.1

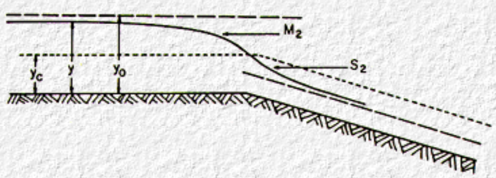

Figure 2.A1.1. Sketch of Backwater Curves

For the upstream reach, flow is subcritical so the standard step method computations start at the change in slope and proceed upstream.

At the change in slope

$$y = y_c = 4.27 \text{ ft}$$

$$A = yW = 4.27\ (100) = 427 \text{ sq ft}$$

$$V = \frac{q}{y} = \frac{50}{4.27} = 11.71 \text{ fps}$$

$$\frac{V^2}{2g} = \frac{11.71^2}{64.4} = 2.13 \text{ ft}$$

$$H = \frac{V^2}{2g} + y = 4.27 + 2.13 = 6.40 \text{ ft}$$

$$P = 2y + W = 2\ (4.27) + 100 = 108.54\ \text{ft}$$

$$R = \frac{A}{P} = \frac{427}{108.54} = 3.93\ \text{ft}$$

$$S_f = \frac{n^2V^2}{2.21R^{4/3}} = \frac{(0.012 \times 11.71)^2}{2.21(3.93)^{4/3}} = 0.001438$$

Let's compute the distance upstream to where the flow is 4.50 ft deep. The flow conditions at this section are computed with the same equations employed at the change in slope section, i.e.,

$$y = 4.50\ \text{ft}$$

$$A = 4.50\ (100) = 450\ \text{sq ft}$$

$$V = \frac{50}{4.50} = 11.11\ \text{fps}$$

$$\frac{V^2}{2g} = \frac{11.11^2}{64.4} = 1.92\ \text{ft}$$

$$H = 4.50 + 1.92 = 6.42\ \text{ft}$$

$$P = 2\ (4.50) + 100 = 109\ \text{ft}$$

$$R = \frac{450}{109} = 4.13\ \text{ft}$$

$$S_f = \frac{(0.012 \times 11.11)^2}{2.21(4.13)^{4/3}} = 0.001438$$

Now between these two sections where y = 4.27 ft and y = 4.50 ft the average friction slope is

$$S_{f_{ave}} = \frac{0.001438 + 0.001214}{2} = 0.001326$$

The distance between the two sections is (Equation 2.7.11)

$$\Delta L = \frac{H_2 - H_1}{S_o - S_{f_{ave}}} = \frac{6.40 - 6.42}{.000095 - 0.001326} = 16.2\ \text{ft}$$

That is, the section where the depth is 4.50 ft is 16.2 ft upstream of the section where the slope changes.

In a similar manner, the distance between the sections where the depths are 4.50 ft and 5.00 ft (arbitrary choice) is computed. The results are listed in Table 2.A1.3. The flow depth approaches normal depth at approximately 44,000 ft above the change in slope.

The backwater calculations for the downstream reach are also presented in Table 2.A1.3. Here the computations start at the change in section and proceed downstream because the flow is supercritical. The computations show that the normal depth is reached approximately 1600 ft below the change in slope.

Table 2.A1.3 Computation of the Backwater Curve

Flow Depth												Distance
y ft	A sq ft	V fps	$V^2/2g$ ft	H ft	P ft	R ft	n	S_f	$S_{f_{ave}}$	H_2-H_1 ft	ΔL ft	L ft
At the change in slope												
4.27	427	11.71	2.13	6.40	108.5	3.93	0.012	.001438	---	---	---	---
S2 backwater curve												
4.20	420	11.90	2.20	6.40	108.4	3.87	0.012	.001519	.001478	-0.00	0.0	0.0
4.00	400	12.50	2.43	6.43	108	3.70		.001779	.001649	-0.03	10.9	10.9
3.50	350	14.28	3.17	6.67	107	3.27		.002738	.002258	-0.24	105.96	116.86
3.00	300	16.67	4.31	7.31	106	2.83		.004523	.003630	-0.64	716.7	833.56
M2 backwater curve												

4.50	450	11.11	1.92	6.42	109	4.13	0.012	.001214	.001326	-0.02	16.2	16.2
5.00	500	10.00	1.55	6.55	110	4.54		.000867	.001040	-0.13	137.6	153.8
6.00	600	8.33	1.08	7.08	112	5.36		.000482	.000674	-0.53	914.6	1068.4
8.00	800	6.25	0.61	8.61	116	6.90		.000194	.000338	-1.53	6296.3	7364.7
10.00	1000	5.00	0.39	10.39	120	8.33		.000095	.000114	-1.78	36326.5	43691.2
A = Wy		$V=\frac{q}{y}$	P = 2y + W		$R=\frac{A}{P}$		$S_f=\frac{n^2V^2}{2.21R^{4/3}}$		$\Delta L=\frac{H_2-H_L}{S_o-S_{fsave}}$		L = ΣΔL	

Problem 2.4 Maximum Stream Constrictions without Causing Backwater (Neglecting Energy Losses)

A stream is rectangular in shape and 100 ft wide. The design discharge is 5000 cfs and the uniform depth for this discharge is 10 ft. Neglecting energy losses what is the maximum amount of constriction that can be imposed without causing backwater at the design discharge?

The upstream flow rate per unit width is

$$q=\frac{Q}{W}=\frac{5000}{100}=50\ \text{cfs/ft}$$

the average velocity is

$$\overline{V}=\frac{Q}{A}=\frac{5000}{1000}=5.00\ \text{fps}$$

and the specific head is (from Equation 2.5.3)

$$H=\frac{\overline{V}^2}{2g}+y=\frac{5^2}{64}+10=10.39\ \text{ft}$$

According to Section 2.5.3, the maximum unit discharge that can occur at y_c with this specific head is (from Equation 2.5.16)

$$y_c=\frac{\left(q_{max}^2\right)^{1/3}}{g}=\frac{2}{3}H=\frac{2\,V_c^2}{2g}$$

$$q_{max}=\left\{g\left(\frac{2}{3}H\right)^3\right\}^{1/2}=\left\{32.2\left(\frac{2}{3}\times 10.39\right)^3\right\}^{1/2}=103.4\ \text{cfs/ft}$$

Therefore, the width of channel which will accomodate this unit discharge is

$$W=\frac{Q}{q_{max}}=\frac{5000}{103.4}=48.3\ \text{ft}$$

and the amount of the constriction is 100 - 48.3 = 51.7 ft.

Note, that this contraction could cause an undulating hydraulic jump downstream. When energy losses are considered there will be some backwater at this constriction.

Problem 2.5 Water Surface Elevation Upstream of a Grade Control Structure

A small grade control structure is to be placed across a stream downstream of a highway bridge. The purpose of the structure is to head up water for diversion into a canal. At the bridge, the design flood discharge is 5000 cfs. The river is 100 ft wide and has a uniform flow depth of 10 ft for the design discharge.

a) What is the maximum height of the structure that will not cause backwater at the bridge?

The unit discharge in the river at design flood discharge is

$$q=\frac{Q}{W}=\frac{5000}{100}=50\ \text{cfs/ft}$$

the velocity is

$$V = \frac{q}{y_o} = \frac{50}{10} = 5.00 \text{ fps}$$

and the specific head is (from Equation 2.5.3)

$$H = \frac{V^2}{2g} + y = \frac{25}{64.4} + 10 = 10.39 \text{ ft}$$

As a first approximation assume no energy loss in the reach. Then at the dam, the elevation of the total energy line is 10.39 ft above the bed (see Figure 2.A1.2). At the dam,

$H_{min} + \Delta Z_{max} = 10.39$ ft

that is, the dam can be built to a height of WZ_{max} which decreases the specific head at the dam to H_{min}. From Equation 2.5.11

$H_{min} = 3/2\ y_c$

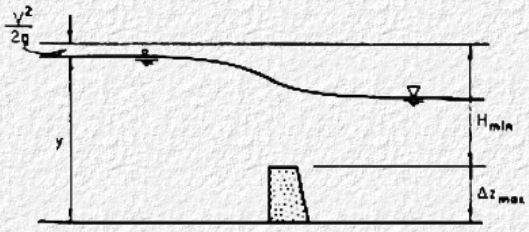

Figure 2.A1.2. Backwater Curve Upstream of the Structure

and from Equation 2.5.7

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{50^2}{32.2}\right)^{1/3} = 4.27 \text{ ft}$$

so

$$H_{min} = \frac{3}{2}(4.27) = 6.40 \text{ ft}$$

Thus

$\Delta Z_{max} = 10.39 - 6.40 = 4.0$ ft

If the structure is built to a crest elevation 4.0 ft above the bed, critical flow will occur at the dam for a flow of 5000 cfs and the dam will cause no backwater.

(b) How much backwater will the dam cause for a flow of 1000 cfs if the normal depth for this discharge is 5 ft and the dam height is 4.0 ft? Upstream of the dam,

$$q = \frac{Q}{W} = \frac{1000}{100} = 10 \text{ cfs/ft}$$

the velocity is

$$V_o = \frac{q}{y_o} = \frac{10}{5} = 2 \text{ fps}$$

At the dam the flow is critical so from Equation 2.5.7

$$y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{10^2}{32.2}\right)^{1/3} = 1.46 \text{ ft}$$

and from Equation 2.5.11

$$H_{min} = \frac{3}{2} y_c = \frac{3}{2}(1.46) = 2.19 \text{ ft}$$

The specific head upstream of the dam is then (assuming no energy loss)

$H = H_{min} + \Delta Z$

or

$H = 2.19 + 4.00 = 6.19$ ft

Also, the specific head upstream of the dam is (from Equation 2.5.4)

$$H = \frac{q^2}{2gy^2} + y$$

Therefore

$$\frac{q^2}{2gy^2} + y = 6.19$$

or

$y^3 - 6.19\, y^2 + 10^2/64.4 = 0$

The solution is

$y = 6.14$ ft

As the normal depth is only 5 ft, the backwater is

$\Delta y = 6.14 - 5.00 = 1.14$ ft

That is, the depth upstream of the dam is increased 1.14 ft by the 4.0 ft high dam when the flow is 1000 cfs.

Problem 2.6 Bridge Backwater Elevation

Calculate the backwater elevation for the bridge crossing illustrated below.

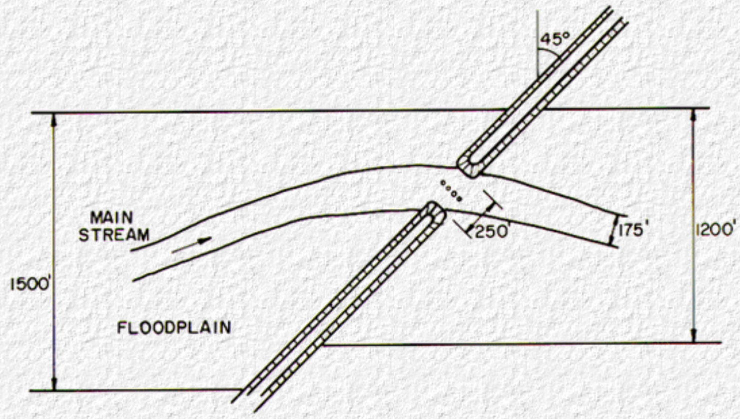

Figure 2.A1.3. Sketch of a Bridge Crossing

Total Discharge 25,000 cfs

Main Stream Discharge 15,000 cfs

Discharge on the Left Bank 2,000 cfs

Discharge on the Right Bank 8,000 cfs

Flow Depth at the Crossing 10 ft

Four circular piers with diameter d = 5 ft

The mean velocity at the bridge crossing is

$$V_{n2} = \frac{Q}{A} = \frac{25000}{250 \times 10} = 10\ \text{ft/sec}$$

The specific head H_2 is

$$H = y_{n2} + \frac{V_{n2}^2}{2g} = 10 + \frac{10^2}{2(32.2)} = 11.55\ \text{ft}$$

The maximum unit discharge (Equation 2.5.16) is

$$q_{max} = \left\{ g\left(\frac{2H}{3}\right)^3 \right\}^{1/2} = \left\{ 32.2 \times \left(\frac{2 \times 11.55}{3}\right)^3 \right\}^{1/2} = 121.29\ \text{ft}^2/\text{s}$$

the actual unit discharge is $q = Q/W = 25000/250 = 100\ \text{ft}^2/\text{s}$. Thus, the flow is subcritical at the crossing and the bridge crossing is of type I. The backwater elevation is calculated from Equation 2A.8.1 and Equation 2A.8.2.

$$h_1^* = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left\{ \left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2 \right\} \frac{V_{n2}^2}{2g} \qquad \text{(2A.8.1)}$$

and

$$K^* = K_b + \Delta K_p + \Delta K_e + \Delta K_s \qquad \text{(2A.8.2)}$$

with $A_4 \cong A_1 >> A_{n2}$, these equations approximate to

$$h_1^* = (K_b + \Delta K_p + \Delta K_e + \Delta K_s)\alpha_2 \frac{V_{n2}^2}{2g}$$

- To evaluate the backwater coefficients, the following parameters must be defined from Figure 2.8.6

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{15000}{2000 + 15000 + 8000} = 0.6$$

$$e_c = 1 - \frac{Q_a}{Q_c} = 1 - \frac{2000}{8000} = 0.75$$

$\phi = 45^o$, spillthrough abutment. Thus, from Figure 2.8.4, with M = 0.6 for spillthrough abutments, $K_b = 0.8$.

- From Figure 2.8.5 for four circular piers,

  $A_p = 4 \times 5 \times 10 = 200\ \text{ft}^2$
  $J = A_p/A_{n2} = 200/2500 = 0.08$
  $\Delta K = 0.32$
  $\sigma = 0.8$
  $\Delta K_p = \sigma \Delta k = 0.8 \times 0.32 = 0.26$
- From Figure 2.8.6, with M = 0.6 and $e_c = 0.75$,

  $\Delta K_e = 0$
- From Figure 2.8.7, with M = 0.6, $\phi = 45\text{-}^o$,

  $\Delta K_s = 0.1$

Taking $\alpha = 1.10$

$h_1^* = (0.8 + 0.26 + 0 + 0.1)\ (1.10)\ 10^2/(2 \times 32.2) = 1.98$ ft.

Problem 2.7 Overtopping of Low Water Stream Crossings

Consider the 20 ft wide gravel vented ford sketched below

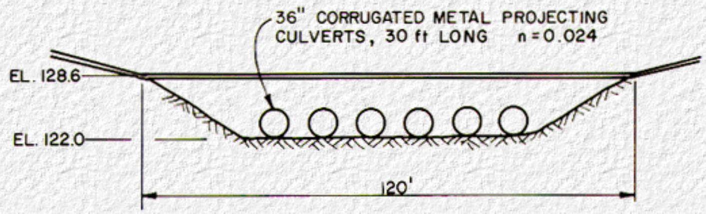

Figure 2.A1.4. Illustration of a Low Water Stream Crossing

a) Calculate the maximum stream discharge without overtopping of the roadway with inlet control.

The maximum head water depth HW without overtopping is 128.6 - 122 = 6.6 ft and the diameter D of the culverts is 3 ft. With the ratio HW/D = 2.2 and the diameter D = 36", one enters Figure 2.9.2b to obtain a discharge of 65 cfs per culvert. Thus, the total discharge is Q = 6 x 65 = 390 cfs prior to overtopping the roadway.

b) What is the total discharge when the flow is at elevation 130 ft, given the bed slope of the channel S_o = 0.0003 and the channel width 120 ft?

This problem is solved using an iterative procedure. Assuming the water surface elevation downstream of the roadway to be at elevation 126 ft, the culverts are flowing under full conditions with H = 130 - 126 = 4 ft. From Equation 2.9.3, the discharge in one culvert Q_c is

$$H = \left[1 + K_e + \frac{29n^2L}{R^{1.33}}\right]\frac{V^2}{2g}$$

$$Q_c = \frac{\pi D^2}{4}V = \frac{\pi D^2}{4}\left[\frac{2gH}{1 + K_e + \frac{29n^2L}{R^{1.33}}}\right]^{1/2}$$

in which, we assume

K_e = 0.9 for sharp edge corrugated metal pipe
n = 0.024
L = 30 ft
R = D/4 = 0.75 ft
H = 4 ft (inital trial)

Thus,

Q_c = 70 cfs

The overtopping discharge is calculated from Equation 2.10.1

$Q_o = k_t C_r L_s (HW_r)^{1.5}$

- without submergence, k_t = 1, from Figure 2.10.1c
- HW_r = 130 - 128.6 = 1.4 ft
- L_r = 20 ft
- HW_r/L_r = 1.4/20 = 0.7 < 0.15 thus C_r is obtained from Figure 2.10.1b.
- C_r = 2.90 from Figure 2.10.1b
- From Figure 2.A1.4, L_s = 120 ft

$Q_o = 1\text{x } 2.90 \text{ x } 120 \text{ x } (1.4)^{1.5} = 576$ cfs

then the total discharge is

$Q_T = Q_o + 6Q_c$ = 576 + 6 x 70 = 996 cfs

Then we need to check the water surface elevation downstream of the roadway crossing.

The hydraulic radius R or normal depth y_n for this wide channel (n = 0.03) is calculated for Equation 2.3.20a

$$y_n = \left[\frac{nq}{1.49\,S_f^{1/2}}\right]^{3/5} = \left[\frac{.03 \times 8.3}{1.49\ \ .0003}\right]^{0.6} = 3.9\text{ ft}$$

the downstream elevation is 125.9 ft which is very close to the assumed value of 126 ft. Therefore, the total discharge is around 1000 cfs, culverts are flowing full and the downstream flow elevation is 126 ft.

A trial and error procedure would be required if the calculated y_n would differ significantly from the assumed value. The same calculation procedure given in this example would have been repeated until the assumed and computed downstream elevations are comparable.

Problem 2.8 Velocity Distributions in Bends

Based on the discharge measurement data given in Table 2.A1.1 (y_{max} = 12.8 ft and R = 1485/163 ft) taken in a straight reach of the channel we would like to estimate the longitudinal and transverse velocity for a bend with an angle of 70â, and a bend centerline radius of 750 ft. The channel has a n-value of 0.024 and a total width of 163 ft.

(a) For the longitudinal velocity, Figure 2.6.3 is used with Equation 2.6.2

$$\Delta_o = 0.42\,\phi\,y_{max}\sqrt{g}\,/WC$$

and using Equation 2.3.21 C = 1.49 $R^{1/6}$/n

C = 1.49 (1484.0/163) $^{1/6}$/0.024 = 89.8

$$\Delta_o = (0.42)(70°)(12.8)\sqrt{32.2}\,/[(163)(89.8)] = 0.146$$

With Δ_o = 14.6 x 10^{-2}, Figure 2.6.3 is used to obtain 10 values of V/V_{max} at various values of x/W. The results are shown in Table 2.A1.4 using V_{max} = 4.9 ft/s from Table 2.A1.1.

Table 2.A1.4 Longitudinal Velocity in a Bend

x/W	x (ft)	V/V_{max}	V (ft/sec)
-.8	-65.2	0.48	2.35
-.6	-48.9	0.64	3.14
-.4	-32.6	0.79	3.87
-.2	-16.3	0.88	4.31
0	0	0.96	4.70
.2	16.3	1.03	5.05
.4	32.6	1.09	5.34
.6	48.9	1.06	5.19
.8	65.2	0.90	4.41

(b) To calculate the transverse velocity Figure 2.6.2 is used with Equation 2.6.1

$$V_r = \frac{1}{k^2}\,\overline{V}\,\frac{y}{r}\,F_1(\eta) - \frac{\sqrt{g}}{kC}\,F_2(\eta)$$

Five relative depths are calculated and the results are shown in Table 2.A1.5 using the value of 12.8 ft for y_{max}, a value of 3.62 ft/sec for $\overline{V}$, (from Table 2.A1.1, $\overline{V}$ = 5368/1489 ft/s) k = 0.4, C = 89.8, and r = 750 ft.

Table 2.A1.5 Transverse Velocity in a Bend

y/y_{max}	y (ft)	F_1 (h)	F_2 (h)	V_r (ft/sec)
.2	2.56	-0.8	-0.4	-0.057
.4	5.12	-0.2	0.4	-0.041
.6	7.68	0.4	0.3	0.081
.8	10.24	0.8	0.4	0.226
1.0	12.80	1.22	0.42	0.443

Footnote

[1] Simons, D. B., E. V. Richardson, M. A. Stevens, J. H. Duke, and V. C. Duke, "Geometric and Hydraulic Properties of the Rivers," Hydrology Report, Vol. I I I, Venezuelan International Meteorological and Hydrological Experiment, Civil Engineering Department, Colorado State University, October, 1971.

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Appendix 3 : HIRE

Solved Problems - Chapter 3

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Problem 3.1 Sediment Properties and Fall Velocities

(a) Sand Bed Channel

The bed sediment size distribution of a sand bed channel is shown in Table 3.A1.1.

Table 3.A1.1 Bed Material Size Distribution

Size Range mm	Percent of Total Weight in Size Range
0.002 - 0.0625	0.8
0.0625 - 0.125	4.4
0.125 - 0.250	14.2
0.250 - 0.500	74.9
0.500 - 1.00	5.0
1.00 - 2.00	0.5
2.00 - 4.00	0.2

From the grain size analysis we would like to calculate the following statistics: the geometric mean of each size range; effective diameter; the D_{16}, D_{50}, D_{84}, D_{90} sizes; the gradation coefficient; and the fall velocity of each size range.

The geometric mean is calculated as the square root of the product of the end points of a given size range.

For the first size range:

$D_1 = [(0.002)\ (.0625)]^{1/2} = 0.011$ mm.

Likewise for the remainder of the size ranges, the results are summarized in Table 3.A1.2.

The effective diameter of the sample distribution is calculated with the use of Equation 3.6.19:

$$D_m = \frac{\sum_{i=1}^{n} p_i D_i}{100} = \frac{(.8)(.011\,mm) + (4.4)(.088\,mm)\ldots(0.3)(2.83\,mm)}{100}$$

$$D_m = \frac{34.2}{100} = 0.34\ mm;\ \text{which lies in the sand size.}$$

Table 3.A1.2 Sand Size Bed Material Properties

Size Range mm	Geometric Mean Size, D_i mm	Percent of Bed Material in this Size p_i	Percent Finer	p_iD_i	Fall Velocity ft/sec
.002 - .0625	0.011	0.8	0.8	0.01	0.0004
.0625 - .125	0.088	4.4	5.2	0.39	0.02
.125 - .250	0.177	14.2	19.4	2.51	0.06
.250 - .500	0.354	74.9	94.3	26.51	0.15
.500 - 1.00	0.707	5.0	99.3	3.54	0.29
1.00 - 2.00	1.41	0.5	99.8	0.71	0.65
2.00 - 4.00	2.83	0.2	100.0	0.56	1.1
	TOTAL 100			34.2	

The sand size distribution is plotted on log-probability paper in Figure 3.A1.1 from which the following values can be obtained:

$D_{50} = 0.31$mm
$D_{16} = 0.24$mm
$D_{90} = 0.46$mm
$D_{84} = 0.42$mm

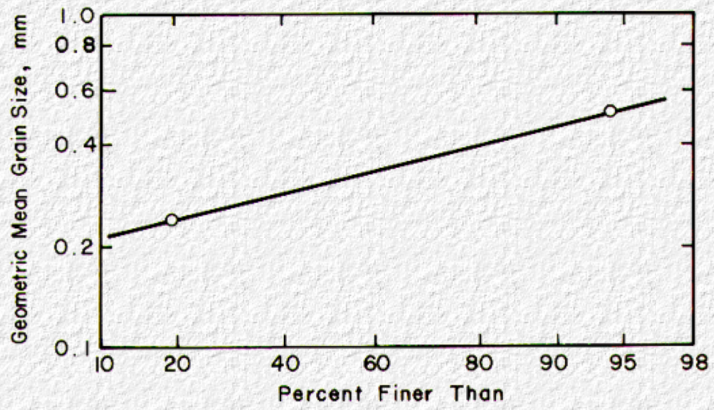

Figure 3.A1.1. Bed Material Size Distribution

The fall velocity is determined with the use of Figure 3.2.1 at 60ºF. For the geometric mean size of the first size range D_i = 0.011mm, ω = 0.0004 ft/s, similarly D_i = 0.707mm, ω = .29 ft/s etc. The results are shown in Table 3.A1.2.

Finally, the gradation coefficient G is calculated with Equation 3.2.9

$$G = \frac{1}{2}\left[\frac{D_{50}}{D_{16}} + \frac{D_{84}}{D_{50}}\right] = \frac{1}{2}\left[\frac{0.31\,\text{mm}}{0.24\,\text{mm}} + \frac{0.42\,\text{mm}}{0.31\,\text{mm}}\right]$$

$$G = 1.32$$

(b) Gravel Bed Channel

A gravel bed stream channel was sampled and the grain size analysis yielded the following results (Table 3.A1.3):

Table 3.A1.3 Gravel Bed Material Size Distribution

Size Range (mm)	Percent of Total Weight in Size Range
.125 - .250	0.1
.250 - .500	1.6
.500 - 1.00	1.7
1.00 - 2.00	6.3
2.00 - 4.00	31.8
4.00 - 8.00	58.5

From this size distribution we would like to calculate the following statistics: the geometric mean of each size range, the fall velocity of each size range, the D_{16}, D_{50}, D_{84}, D_{90} sizes and the gradation coefficient.

The geometric mean size is calculated as the square root of the product of the end points of a given size range. For the largest size range:

$D_i = [(4)\ (8)]^{½}$ = 5.66 mm.

Likewise the rest of the size ranges are calculated, with the results shown in Table 3.A1.4.

The fall velocity is calculated with the use of Figure 3.2.1 at 60°F, for each geometric mean size. For the largest geometric mean size: D_i = 5.66 mm, ω = 1.7 ft/s. Likewise the rest of the size ranges, the results are shown in Table 3.A1.4.

The gravel bed material size distribution was plotted on log-probability paper from which the following values were obtained:

D_{16} = 1.8mm	D_{84} =4.4mm
D_{50} = 3.1mm	D_{90} =4.9mm

The effective diameter of the sample distribution is calculated with Equation 3.6.19:

$$D_m = \frac{\sum_{i=1}^{n} p_i D_i}{100} = \frac{(0.1)(.177) + \ldots (58.5)(5.66)}{100}$$

$$D_m = \frac{431.9}{100} = 4.32\,\text{mm}$$

The gradation coefficient is calculated by Equation 3.2.9:

$$G = \frac{1}{2}\left(\frac{D_{50}}{D_{16}}\right) + \left(\frac{D_{84}}{D_{50}}\right) = \frac{1}{2}\ \frac{3.1\ \text{mm}}{1.8\ \text{mm}} + \frac{4.4\ \text{mm}}{3.1\ \text{mm}}$$

$$G = 1.57$$

Table 3.A1.4 Gravel Bed Material Properties

Size Range mm	Geometric Mean Size, D_i mm	Percent of Bed Material in this Size p_i	Percent Finer	p_iD_i	Fall Velocity ft/sec
.125 - .250	0.177	0.1	0.1	.018	0.06
.250 - .500	0.354	1.6	1.7	.566	0.15
.500 - 1.00	0.707	1.7	3.4	1.20	0.29
1.00 - 2.00	1.41	6.3	9.7	8.88	0.65
2.00 - 4.00	2.83	31.8	41.5	90.0	1.1
4.00 - 8.00	5.66	58.5	100.0	331.1	1.7
		100 Total		431.9	

Problem 3.2 Velocity Profiles and Shear Stress

A measurement is made on the Missouri River at Sioux City. The total discharge is 32,600 cfs. The average depth is 7.80 feet and the average velocity is 2.42 ft/sec in a vertical section at a point 800 feet from the right bank. The velocity measurements at various distances from the bottom (y) are shown in Table 3.A1.5.

Table 3.A1.5 Observed Velocity Data

y (ft)	Observed Velocity (ft/sec)
0.5	1.70
1.0	2.00
1.5	2.30
2.5	2.50
3.5	2.75
4.5	2.90
5.5	3.10
6.25	3.10
7.5	3.25

Also during this measurement the slope of the energy grade line was observed as 0.000206 and the bed material gradation was determined with D_{50} = 0.270 mm, D_{65} = 0.315 mm and G = 1.47.

To obtain a mathematical description of the velocity profile we will use both the log and power function velocity distribution.

For the log velocity relation we use Equation 2.3.14: $V/V_* = 1/k\ \ell n\ (y/y')$, we wish to determine the value of k and y', with $V_* = \sqrt{g y_o S_f}$ = 0.225 ft/sec.

To illustrate these calculations Table 3.A1.6 is prepared for both log and power forms with the values shown.

Using regression techniques, for the logarithmic form of the equation on the values of V/V_* and $\ell n\ y$ we obtain: $V/V_* = 2.583\ (\ell n\ y - \ell n\ 0.03)$ with R = 0.996 giving the values of k = 0.39 and y' = 0.03 and the final equation for the data is

$$V/V_* = 2.583\ \ell n\ (y/0.03)$$

Table 3.A1.6 Velocity Profile Calculations

y (ft)	V ft/sec	V/V_*	$V/\overline{V}$	y/y_o
0.50	1.70	07.55	0.702	0.064
1.00	2.00	08.89	0.826	0.128
1.50	2.30	10.22	0.950	0.192
2.50	2.50	11.11	1.033	0.321
3.50	2.75	12.22	1.136	0.449
4.50	2.90	12.89	1.198	0.577
5.50	3.10	13.78	1.281	0.705
6.25	3.10	13.78	1.281	0.801
7.50	3.25	14.44	1.340	0.962

For the power form of the equation we have $V/\overline{V} = a\ (y/y_o)^b$. To simplify the regression calculations we transform this by taking the logs: $\ln V/\overline{V} = \ln a + b \ln (y/y_o)$ so we can calculate the values of a and b by linear regression of $\ln V/\overline{V}$ vs $\ln (y/y_o)$ giving the following results:

a = 1.368

b = 0.240
R = 0.998

So that we can write the power equation as

$$\frac{V}{2.42} = 1.368\left(\frac{y}{y_0}\right)^{0.240}$$

Using the above information we would like to calculate the shear stress on the bed using several methods.

(a) By manipulation of the log velocity relation (Equation 2.3.15) which yields Equation 3.5.13

$$V_* = \frac{V}{2.583\ \ell n\,(y/0.03)} = \sqrt{\frac{\tau_0}{\rho}}$$ so

$$\tau_0 = \frac{\rho V^2}{[2.583\ \ell n(y/0.03)]^2};$$

using y = 1, V = 2.00 as a known point velocity.

$$\tau_0 = \frac{(1.92)(2)^2}{[2.583\ \ell n(1/0.03)]^2} = 0.095\,\text{lb}/\text{ft}^2$$

(b) By using Equation 3.5.12 and making R = y_o (for a wide channel), $\tau_o = \gamma R S_f$ = (62.4) (7.8) (0.000206) = 0.100 lb/ft^2.

The value in (a) is for a single vertical in the cross-section and the value in (b) is an average for the entire channel.

Problem 3.3 Beginning of Motion

To establish if the bed sediment of a channel is in motion the Shields' relationship is used. We would like to establish if the bed sediment in Problem 3.1 (a) and (b) is in motion for the hydraulic conditions given in Problem 3.2.

(a) For sand sized material, using Figure 3.5.1 and using the value of τ_o at a single vertical in the cross-section, i.e. τ_o = 0.095 lb/ft^2 and D_s = 0.3l mm = 0.001 ft we have:

$\tau_o/(\gamma_s - \gamma)\ D_s$ = 0.095/ (165 - 62.4) (.001) = 0.926

$V_*\ D_s/\upsilon$ = (0.095/1.94)^{½} (.001)/ 1.41x 10^{-5} = 15.69

This point plots above the line of incipient motion and indicates the bed of the channel is in motion.

(b) For the gravel sized material with the same values of τ_o = 0.095 lb/ft^2; D_s = 1.9 mm = 0.006 ft we have:

$\tau_o/(\gamma_s - \gamma)\ D_s$ = 0.095/ (165 - 62.4) (.006) = 0.154

$V_*\ D_s/\upsilon$ = (0.095/1.94)^{½} (.006)/ 1.41 x 10^{-5} = 95

This point plots above the line of incipient motion and indicates this channel bed is in motion.

Problem 3.4 Bedforms and Resistance to Flow in Sandbed Streams

A sandbed channel is observed to have an undulating water surface, a discharge of 850 cfs, an average velocity of 3.48 ft/sec, a channel width of 105 ft and a bed slope of 0.003. The stream bed has a D_{50} of 0.35 mm. An estimate of the bed form and n-value of the channel is desired.

Using Figure 3.4.4 and assuming the sieve diameter equals the fall diameter, and the bed slope equals the friction slope; we determine the stream power, $V\gamma y_o\ S_o$ = (3.48) (62.4) (850/105 x 3.48) (0.003) = 1.52 ft lb/sec ft^2.

This figure indicates that upper flow regime is expected. The bed configuration should be antidunes with standing waves.

Based upon the bedform the n-value is estimated to be 0.013.

Problem 3.5 Resistance to Flow in Gravel Bed Streams

Use the gravel bed material size analysis in Problem 3.1 (b) estimate the Manning's n-value of the stream.

Using Equations 3.4.2 through 3.4.6 we will get a range of n-values to evaluate for the channel

- n = 0.04 $D_{50}^{\ .167}$ = 0.04 (.0043) $^{.167}$ = 0.016

- $n = 1/44.4\, D_{90}^{\,.167} = (6.3 \times .00328 \times 12)^{.167} = 0.018$
- $n = 1/39\, D_{75}^{\,.167} = (1/39)\,(5.5 \times .0394)^{.167} = 0.020$
- $n = 0.038\, D_{90}^{\,.167} = 0.038\,(0.0063)^{.167} = 0.016$

(Assume y_o = 5.0 ft = 1.52m and $R = y_o$)

- $$n = \frac{0.113\, y_o^{1/6}}{1.16 + 2.0 \log\left(\frac{y_o}{D_{84}}\right)} = \frac{0.113\,(1.52)^{.167}}{1.16 + 2.0 \log\left(\frac{1.52}{0.006}\right)} = 0.020 \quad \frac{1}{\sqrt{f}} = 1.9\left(\frac{R}{D_{84}}\right)^{.25} = 1.9\left(\frac{1.52}{0.006}\right)^{.25} = 7.58$$
- $$n = 0.113 (R)^{.167} \sqrt{f} = \frac{0.113\,(1.52)^{.167}}{7.58} = 0.016$$

$$\frac{1}{\sqrt{f}} = 2.0 \log\left(\frac{R}{D_{84}}\right) + 1.1 = 2.0 \log\left(\frac{1.52}{0.006}\right) + 1.1 = 5.91$$

- $$n = 0.113 (1.52)^{.167} \sqrt{f} = 0.021$$

Based on the range of n-values indicated above a n-value of 0.018 is selected.

Problem 3.6 Sediment Concentration Profile

Data observed on the Missouri River for sediment discharge calculations are summarized in Table 3.A1.7

Table 3.A1.7 Sediment Discharge Data

y_o = 14.10 ft
V = 4.164 ft/sec
S = 0.000210
Temp = 72.5ºF
D_{50} = 0.000838 ft (0.255mm)
$\omega(D_{50})$ = 0.1109 ft/sec
υ = 1.017 x 10^{-5} ft^2/sec
C_a = 775,000 mg/ℓ (48.3 lb/ft^3)

We would like to calculate the vertical concentration profile of the D_{50} size sediment for this data.

From Equation 3.6.11

$$\frac{C}{C_a} = \left[\frac{y_o - y}{y} \quad \frac{a}{y_o - a}\right]^Z \text{ and } Z = \frac{\omega}{\beta \kappa V_*}$$

we calculate

$\beta = 1$
$\kappa = 0.4$
$a = 2\, D_{50} = 0.0017$ ft
$V_* = (g\, y_o S)^{1/2} = [(32.2)\,(14.1)\,(0.00021)\,]^{1/2} = 0.309$ ft/sec

giving

$$Z = \frac{0.1109\,\text{ft/sec}}{(.4)\,(.309)} = 0.898$$

and we can then calculate the following concentration versus distance data given in Table 3.A1.8.

Table 3.A1.8 Concentration vs. Elevation above the Bed

y (ft)	C (mg/ℓ)

0.3	7,210.
1	2,330.
2	1,170.
4	532.
8	181.
10	104.
12	48.
14	3.

Problem 3.7 Calculation of Bed Sediment Transport Using Meyer-Peter and Muller Method

Rapid Meyer-Peter, Muller computations.

If certain assumptions can be made the Meyer-Peter, Muller computations can be simplified considerably. If the channel is wide Q = Q_B and if the bed sediment distribution is log-normal we can use:

D_{50} = 1.9 mm
D_{90} = 2.8 mm
G = 1.41
n = 0.04
n_w = 1.5 n = 0.06
y_o = 9.8 ft
S_f = 0.0005 ft/ft
$D_m/D_{50} = \exp\{1/2\,(\ell n\, G)^2\}$
$D_m = 1.9 \exp\{1/2\,(\ell n\, 1.41)^2\} = 2.01$mm

and

$$n_b = (.04)\left\{1+\frac{19.6}{200}\left(1-\left(\frac{.06}{.04}\right)^{3/2}\right)\right\}^{2/3} = 0.038$$

Using the USBR form of Meyer-Peter and Muller's equation, Equation 3.6.18

$$q_B = 1.606\left[3.306\left(\frac{Q_b}{Q}\right)\left(\frac{D_{90}^{1/6}}{n_b}\right)^{3/2} y_o S_f - 0.627\, D_m\right]^{3/2}$$

$$q_B = 1.606\left[3.306(1)\left(\frac{2.8^{1/6}}{0.038}\right)^{3/2} (9.8)(.0005) - 0.627\,(2.01)\right]^{3/2}$$

q_B = 3.15 ton/day/ft

Q_B = 630 tons/day

Problem 3.8 Application of the Einstein Method to Calculate Total Bed Sediment Discharge

Description Of The Test Reach

A test reach, representative of the Big Sand Creek near Greenwood, Mississippi, was used by Einstein (1950) as an illustrative example for applying his bed-load function. His example is reproduced here. For simplicity, the effects due to bank friction are neglected. The reader can refer to the original example for the construction of the representative cross-section. The characteristics of this cross-section are as follows.

The channel slope was determined as S = 0.00105. The relation between cross-sectional area, hydraulic radius, and wetted perimeter of the representative cross-section and stage are given in Figure 3.A1.3. In the case of this wide and shallow channel, the wetted perimeter is assumed to equal the surface width. The averaged values of the four bed-material samples are given in Table 3.A1.9. Ninety-six percent of the bed material is between 0.589 and 0.147 mm, which is divided into four size fractions. The sediment transport calculations are made for the individual size fraction which has the representative grain size equal to the geometric mean grain diameter of each fraction. The water viscosity is

$\upsilon = 1.0 \times 10^{-5}$ ft^2/sec

and the specific gravity of the sediment is 2.65.

The calculation of important hydraulic parameters is performed in Table 3.A1.10. The table heading, its meaning, and calculation are explained with footnotes.

The bed sediment transport is calculated for each grain fraction of the bed material at each given flow depth. It is convenient to summarize the calculations in the form of tables. The procedure is given in Table 3.A1.11.

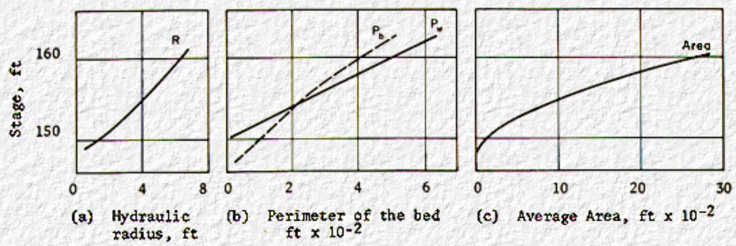

Figure 3.A1.2. Description of the Average Cross-Section (Einstein, 1950)

Table 3.A1.9 Bed Sediment Information for Sample Problem

Grain Size Distribution, mm	Average Grain Size			Settling Velocity	
	mm	ft	%	cm/sec	fps
D > 0.589	---	---	2.4	---	---
0.589 > D > 0.417	0.495	0.00162	17.8	6.25	0.205
0.417 > D > 0.295	0.351	0.00115	40.2	4.51	0.148
0.295 > D > 0.208	0.248	0.00081	32.0	3.23	0.106
0.208 > D > 0.147	0.175	0.00058	5.8	2.04	0.067
0.147 > D	---	---	1.8	---	---
D_{35} = 0.29 mm = 0.00094 ft					
D_{65} = 0.35 mm = 0.00115 ft					

Table 3.A1.10 Hydraulic Calculations for Sample Problem in Applying the Einstein Procedure (after Einstein,1950)

R'_b (1)	V'_* (2)	δ' (3)	k_s/δ' (4)	X (5)
0.5	0.129	0.00095	1.21	1.59
1.0	0.184	0.00067	1.72	1.46
2.0	0.259	0.00047	2.44	1.27
3.0	0.318	0.00039	2.95	1.18
4.0	0.368	0.00033	3.50	1.14
5.0	0.412	0.00030	3.84	1.10
6.0	0.450	0.00027	4.26	1.08

Δ (6)	V (7)	ψ (8)	V/V''_* (9)	V''_* (10)	R''_b (11)
0.00072	2.92	2.98	16.8	0.17	0.86
0.00079	4.44	1.49	27.0	0.16	0.76
0.00090	6.63	0.75	51.0	0.13	0.50
0.00097	8.40	0.50	87.0	0.10	0.30
0.00102	9.92	0.37	150.0	0.07	0.14
0.00104	11.30	0.30	240.0	0.05	0.07
0.00107	12.58	0.25	370.0	0.03	0.03

R_b (12)	y_o (13)	Z (14)	A (15)	P_b (16)	Q (17)
1.36	1.36	150.2	140	103	409
1.76	1.76	150.9	240	136	1,065
2.50	2.50	152.1	425	170	2,820
3.30	3.30	153.3	640	194	5,380
4.14	4.14	154.9	970	234	9,620
5.07	5.07	156.9	1,465	289	16,550
6.03	6.03	159.5	2,400	398	30,220

X (18)	Y (19)	B_x (20)	$(B/B_x)^2$ (21)	P_E (22)
0.00132	0.84	1.29	0.63	10.97
0.00093	0.68	1.19	0.85	11.10
0.00069	0.56	0.91	1.27	11.30
0.00076	0.55	0.91	1.27	11.50
0.00079	0.54	0.91	1.27	11.70
0.00080	0.54	0.91	1.27	11.90
0.00082	0.54	0.91	1.27	12.04

Table 3.A1.10 Hydraulic Calculation for Sample Problem in Applying the Einstein Procedure (after Einstein, 1950) (Continued)

See the following for explanation of symbols, column by column:

(1) R_b =	bed hydraulic radius due to grain roughness, ft.
(2) V'_* =	shear velocity due to grain roughness, fps = $\sqrt{gR'_bS}$ thickness of the laminar sublayer, ft =
(3) δ' =	$11.62u/V_*$
(4) k_s =	roughness diameter, ft = D_{65}
(5) X =	correction factor in the logarithmic velocity distribution, given in Figure 2.3.5
(6) Δ =	apparent roughness diameter, ft = k_s/X
(7) V =	average flow velocity, fps = $5.75V'_* \log (12.27 R'_b/D)$
(8) ψ' =	intensity of shear on representative particles
=	$\frac{\rho_s - \rho}{\rho}\frac{D_{35}}{R'_bS}$
(9) V/V''_* =	velocity ratio, given in Figure 3.6.8
(10) V''_* =	shear velocity due to form roughness, fps
(11) R''_b =	bed hydraulic radius due to form roughness, ft = V''^2_*/gS
(12) R_b =	bed hydraulic radius, ft = R, the total hydraulic radius if there is no additional friction = $R'_b + R''_b$
(13) y_o =	average flow depth, ft = R for wide, shallow streams
(14) Z =	stage, ft
(17) Q =	flow discharge = AV
(15) A =	cross-sectional area, ft^2
(16) P_b =	bed wetted perimeter, ft
(18) X =	charactertistic distance, ft = 0.77Δ for $\Delta/\delta' > 1.80$ = $1.39\delta'$ for $\Delta/\delta' < 1.80$
(19) Y =	pressure correction term, given in Figure 3.6.5
(20) B_x =	coefficient = log (10.6 X/D)
(21) B =	coefficient = log 10.6
(22) P_E =	Einstein's transport parameter
=	$2.303 \log \frac{30.2\, y_o}{\Delta}$

Table 3.A1.11 Bed-Material Load Calculations for Sample Problem in Applying the Einstein Procedure (Einstein, 1950)

D (1)	i_B (2)	R'_b (3)	ψ (4)	D/X (5)	ξ (6)	ψ_* (7)	ϕ_* (8)	i_Bq_B (9)	i_BQ_B (10)	Σi_BQ_B (11)	10^3E (12)	Z (13)	I_1 (14)	$-I_2$ (15)	$P_EI_1+I_2+1$ (16)	i_Tq_T (17)	i_TQ_T (18)	Σi_TQ_T (19)
.00162	.178	0.5	5.08	1.23	1.08	2.90	1.90	0.0267	119.00	400	2.38	3.78	0.078	0.44	1.42	0.03800	168	670
		1.0	2.54	1.74	1.00	1.73	4.00	0.0561	330.00	1,335	1.84	2.65	0.131	0.74	1.71	0.09580	561	3,928
		2.0	1.27	2.35	1.00	0.90	8.20	0.1150	845.00	3,771	1.30	1.88	0.240	1.27	2.44	0.28100	2,050	30,500
		3.0	0.85	2.16	1.00	0.60	12.80	0.1800	1,510.00	6,496	0.98	1.53	0.385	2.01	3.44	0.61700	5,170	113,000
		4.0	0.63	2.05	1.00	0.43	18.00	0.2530	2,560.00	10,745	0.78	1.33	0.560	2.80	4.75	1.20000	12,100	324,000
		5.0	0.51	2.03	1.00	0.35	22.50	0.3160	3,950.00	16,333	0.63	1.18	0.810	3.85	6.78	2.13000	26,500	800,000
		6.0	0.42	1.98	1.00	0.29	27.00	0.3800	6,350.00	27,142	0.54	1.08	1.090	4.90	9.20	3.48000	59,800	1,940,000
.00115	.402	0.5	3.38	0.82	1.36	2.44	2.45	0.0471	210.00		1.69	2.88	0.117	0.68	1.60	0.07540	335	
		1.0	1.69	1.16	1.10	1.27	5.50	0.1060	623.00		1.31	2.02	0.210	1.19	2.14	0.22700	1,330	
		2.0	0.85	1.57	1.01	0.61	12.60	0.2420	1,780.00		0.92	1.44	0.450	2.33	3.76	0.91000	6,660	
		3.0	0.56	1.44	1.04	0.41	19.00	0.3640	3,050.00		0.70	1.17	0.830	3.85	6.73	2.44000	20,400	
		4.0	0.42	1.37	1.05	0.30	26.00	0.5000	5,050.00		0.56	1.01	1.370	5.70	11.30	5.65000	57,100	
		5.0	0.34	1.35	1.05	0.25	31.50	0.6040	7,540.00		0.45	0.90	2.120	8.10	17.20	10.40000	129,000	
		6.0	0.28	1.32	1.05	0.20	39.00	0.7490	12,900.00		0.38	0.83	2.950	10.50	26.00	19.60000	335,000	

.00081	.320	0.5	2.54	0.6	2.25	3.03	1.75	0.0155	69.00		1.19	1.94	0.230	1.29	2.23	0.03450	153	
		1.0	1.27	0.87	1.26	1.09	6.80	0.0600	353.00		0.92	1.36	0.520	2.60	4.16	0.25000	1,460	
		2.0	0.63	1.17	1.10	0.49	15.80	0.1390	1,020.00		0.65	0.97	1.530	6.10	12.20	1.70000	12,500	
		3.0	0.42	1.08	1.12	0.33	23.50	0.2070	1,730.00		0.49	0.79	3.350	11.00	28.70	5.95000	49,700	
		4.0	0.32	1.04	1.15	0.25	31.50	0.2790	2,820.00		0.39	0.68	6.200	17.50	56.00	15.00000	157,000	
		5.0	0.25	1.01	1.17	0.20	39.50	0.3490	4,360.00		0.32	0.61	9.800	25.50	92.00	32.00000	397,000	
		6.0	0.21	0.99	1.19	0.17	46.00	0.4060	6,980.00		0.27	0.55	15.000	36.00	146.00	59.50000	1,020,000	
.00057	.058	0.5	1.80	0.43	5.40	5.15	0.58	0.00056	2.49		0.85	1.21	0.720	3.35	5.55	0.00312	14	
		1.0	0.90	0.61	2.28	1.39	5.10	0.00500	29.40		0.65	0.86	2.44	8.10	20.00	0.10000	587	
		2.0	0.45	0.83	1.37	0.44	17.50	0.01710	126.00		0.46	0.61	8.40	21.50	74.40	1.26000	9,350	
		3.0	0.30	0.76	1.52	0.32	25.00	0.02460	206.00		0.35	0.49	19.3	41.00	183.00	4.50000	37,600	
		4.0	0.22	0.72	1.60	0.25	31.50	0.03100	313.00		0.28	0.43	32.0	63.00	312.00	9.68000	97,800	
		5.0	0.18	0.71	1.65	0.20	39.50	0.03870	483.00		0.23	0.38	51.0	91.00	516.00	20.00000	248,000	
		6.0	0.15	0.70	1.70	0.18	43.50	0.04260	732.00		0.19	0.35	70.0	122.00	722.00	30.80000	526,000	

Table 3.A1.11 Bed-Material Load Calculations for Sample Problem in Applying the Einstein Procedure (Einstein, 1950)(Continued)

See the following for explanation of symbols, column by column:

1) D =	representative grain size, ft, given in Table 3.A1.9
(2) i_B =	fraction of bed material given in Table 3.A1.9
(3) R'_b =	bed hydraulic radius due to grain roughness, ft, given in Table 3.A1.10
(4) Ψ =	intensity of shear on a particle $\frac{\rho_s - \rho}{\rho}\frac{d}{R'_b S}$
(5) D/X =	dimensionless ratio, $\overline{X}$ given in Table 3.A1.10
(6) ξ =	hiding factor, given in Figure 3.6.4
(7) Ψ_* =	intensity of shear on individual grain size = $\xi Y(B/B_x)^2\Psi$, (values of Y and $(B/B_x)^2$ are given in Table 3.A1.10
(8) ϕ_* =	intensity of sediment transport for individual grain
(9) $i_B q_B$ =	bed load discharge per unit width for a size fraction, lb/sec/ft = $i_B \phi_* \rho_s (gD)^{3/2} \sqrt{(\rho_s/\rho)-1}$
(10) $i_B Q_B$ =	bed load discharge for a size fraction for entire cross-section, tons/day = 43.2 $W i_B q_B$, W = P_b given in Table 3.A1.10
(11) $\Sigma i_B Q_B$ =	total bed load discharge for all size fractions for entire cross-section, tons/day
(12) E =	ratio of bed layer thickness to water depth = $2D/y_o$, for values of y_o see Table 3.A1.10
(13) Z =	exponent for concentration distribution =$\omega/(0.4U'_*)$, for values of ω and U'_* see Table 3.A1.9 and Table 3.A1.10
(14) I_1 =	integral, given in Figure 3.6.6
(15) $-I_2$ =	integral, given in Figure 3.6.7
(16) $P_E I_1 + I_2 + I$=	factor between bed load and total load
(17) $i_T q_T$ =	bed material load per unit width of stream for a size fraction, lb/sec-ft = $i_B q_B$ $(P_E I_1 + I_2 + 1)$
(18) $i_T Q_T$ =	bed material load for a size fraction for entire cross-section, tons/day
(19) $\Sigma i_T Q_T$ =	total bed material load for all size fractions, tons/day

Problem 3.9 Calculation of Total Bed Sediment Discharge Using Coby's Method

For purposes of comparison the hydraulic and sediment data given in Problem 3.8 are used in the computation using Colby's method. The required data are taken from Table 3.A1.8 and Table 3.A1.9. In addition, the water temperature and the fine sediment concentration are assumed to equal 70º F and 10,000 ppm, respectively. For convenience, the calculations are summarized in the form of tables. Table 3.A1.12 contains the calculations over all size fractions using the median diameter of bed material; whereas, Table 3.A1.13 contains the calculations for individual fractions using the bed-material size distribution.

Table 3.A1.12 Bed Sediment Load Calculations for Sample Problem by Applying the Colby Method (Overall Computations)

y_o (1)	W (2)	V (3)	q_n (4)	k_1 (5)	k_2 (6)	k_3 (7)	q_T (8)	Q_T (9)
1.36	103	2.92	14.5	0.92	1.20	.99	15.6	1,610
1.76	136	4.44	50.0	0.91	1.21	.99	55.0	7,480
2.50	170	6.63	135.0	0.91	1.22	.99	150.0	25,500
3.30	194	8.40	220.0	0.90	1.23	.99	243.0	47,100
4.14	234	9.92	325.0	0.90	1.25	.99	365.0	85,410

(1) y_o =	mean depth ft taken from Table 3.A1.10
(2) W =	surface width ft taken from Table 3.A1.10
(3) V =	average velocity fps taken from Table 3.A1.10
(4) q_n =	uncorrected sediment discharge tons/day/ft width taken from Figure 3.6.9 for the given V, y_o and D_{50} by logarithmic interpolation.
(5) k_1 =	correction factor for temperature, given in Figure 3.6.10
(6) k_2 =	correction factor for fine sediment concentration, given in Figure 3.6.10
(7) k_3 =	correction factor for sediment size, given in Figure 3.6.10
(8) q_T =	true bed sediment discharge per unit width of stream tons/day/ft width given by Equation 3.6.43
(9) Q_T =	bed sediment discharge for all size fractions for entire cross-section tons/day given by Equation 3.6.44

Table 3.A1.13 Bed-Material Discharge Calculations for Sample Problem by Applying the Colby Method (Individual Size Fraction Computation)

D (1)	i_B (2)	y_o (3)	W (4)	V (5)	q_n (6)	k_1 (7)	k_2 (8)	k_3 (9)	q_T (10)	i_Bq_T (11)	i_BQ_T (12)
0.495	.178	1.36	103	2.92	12	0.92	1.20	.62	13	2.3	237
		1.76	136	4.44	40	0.91	1.21	.62	43	7.7	1,050
		2.50	170	6.63	112	0.91	1.22	.62	119	21.0	3,570
		3.30	194	8.40	193	0.90	1.23	.62	205	37.0	7,180
		4.14	234	9.92	265	0.90	1.25	.62	288	51.0	11,900
0.351	.402	1.36	103	2.92	15	0.92	1.20	.92	16	6.4	659
		1.76	136	4.44	45	0.91	1.21	.92	49	20.0	2,720
		2.50	170	6.63	120	0.91	1.22	.92	132	53.0	9,010
		3.30	194	8.40	210	0.90	1.23	.92	230	93.0	18,000
		4.14	234	9.92	290	0.90	1.25	.92	323	130.0	30,420
0.248	.320	1.36	103	2.92	18	0.92	1.20	1.00	20	6.4	659
		1.76	136	4.44	53	0.91	1.21	1.00	58	19.0	2,580
		2.50	170	6.63	140	0.91	1.22	1.00	155	50.0	8,500
		3.30	194	8.40	240	0.90	1.23	1.00	266	85.0	16,500
		4.14	234	9.92	345	0.90	1.25	1.00	388	124.0	29,000
0.175	.058	1.36	103	2.92	23	0.90	1.25	1.00	25	1.5	155
		1.76	136	4.44	64	0.91	1.21	.97	70	4.1	558
		2.50	170	6.63	163	0.91	1.22	.97	180	10.0	1,700
		3.30	194	8.40	305	0.90	1.23	.97	337	20.0	3,880
		4.14	234	9.92	420	0.90	1.25	.97	471	27.0	6,320

Table 3.A1.13 Bed-Material Discharge Calculations for Sample Problem by Applying the Colby Method (Individual Size Fraction Computation) (Continued)

See the following for explanation of symbols, column by column:

(1) D =	representative grain size, mm, given in Table 3.A1.9
(2) i_b =	fraction of bed material, taken from Table 3.A1.9
(3) y_o =	average flow depth, ft., taken from Table 3.A1.10
(4) W =	top width, ft, taken from Table 3.A1.10
(5) V =	average velocity, fps, taken from Table 3.A1.10.
(6) q_n =	uncorrect sediment discharge per unit width assuming the bed is composed entirely of one sand of size (d), tons/day/ft width, taken from Figure 3.6.9 by interpolation on logarithmic paper for the given V, D, and d
(7) k_1 =	correction factor for temperature, given in Figure 3.6.10
(8) k_2 =	correction factor for fine sediment, given in Figure 3.6.10
(9) k_3 =	correction factor for sediment size, given in Figure 3.6.10
(10) q_T =	corrected bed-material discharge per unit width by assuming the bed is composed entirely of one sand of size D, tons/day/ft width, given by Equation 3.6.43
(11) i_Bq_T =	bed-material discharge per unit width for a size fraction, tons/day/ft width
(12) i_BQ_T=	W i_Bq_T,the bed-material discharge for a size fraction for entire cross-section, tons/day

The results of bed sediment discharge calculations for the sample problem by using Einstein's (1950), and Colby's (1964) methods are shown in Figure 3.A1.4. The curves indicate that the sediment discharge increases rapidly with increasing water discharge. However, considerable deviations may result applying these two different methods.

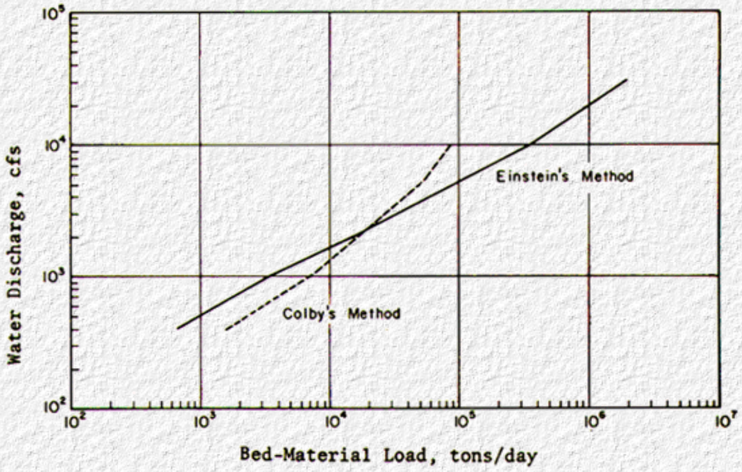

Figure 3.A1.3. Comparison of the Einstein and Colby Methods

Problem 3.10 Calculation of Total Bed Sediment Transport Using Power Relationships

Using the sediment characteristics in Problem 3.1a and the hydraulic data of Problem 3.2 we would like to calculate the total bed sediment rate with the power relations in Section 3.6.11. Referring to Table 3.6.1 we cross-interpolate for values of d_{50} = 0.31 mm and G = 1.32. The resulting values are:

$a_1 = 1.32 \times 10^{-5}$

$a_2 = 0.451$

$a_3 = 3.62$

$q_s = (1.32 \times 10^{-5})\ (7.8)^{.451}\ (2.42)^{3.62}$

$q_s = 0.001\ ft^3/sec/ft$

w = 1,727 ft

$Q_s = 1.727\ ft^3/sec = 12{,}310$ tons/day

In this example the range of parameters used to develop the regression equation is not the same as the application. This requires extrapolation, which is not recommended.

Problem 3.11 Armor Layer Formation

If the movement of the bed materials out of the river reach is not accompanied by an equal influx of bed material at the upstream end, the bed degrades and develops an armor layer. The size of the armor material is determined from the Meyer-Peter and Muller equation for no transport, Equation 3.6.13.

$$\frac{Q_b}{Q}\left(\frac{K_b}{K_r}\right)^{3/2}\gamma y_o S = 0.034(\gamma_s - \gamma)D_m$$

for zero sediment discharge. That is, the armor coat materials are of size D_m and do not move.

For the flow conditions described in Problem 3.7

$Q_b/Q = 0.847$

$K_b/K_r = 26.48/46.9 = 0.565$

by manipulation of Equation 3.6.13

$$D_m = \frac{(.847)(.565)^{3/2}(62.4)(9.8)(0.0005)}{0.034(165 - 62.4)} = 0.032\ \text{ft}$$

$$D_m = 0.032 \text{ ft} = 10\text{mm}$$

This size is larger than the largest bed material size so the bed must degrade substantially to become armored.

Armor formation can also be checked with use of the Shields' criteria which is manipulated from Equation 3.6.46 as

$$D_s = \frac{\tau_o}{(\gamma_s - \gamma)0.047} = \frac{\gamma y_o S_f}{(\gamma_s - \gamma)(0.047)}$$

$$D_s = \frac{(62.4)(9.8)(.0005)}{(102.6)(0.047)} = 0.063 \text{ ft}$$

D_s = 19 mm using Shields' criteria as the size of the armor material size, which also indicates that an armor layer will not likely form.

Problem 3.12 Sediment Transport in Culverts

For the culvert example in Chapter II we would like to calculate the maximum sediment transport rate. Using Equation 3.7.1

$$Q_{smax} = 3.78\sqrt{g}\, D_s^{-1.02} S_f^{2.52} R^{1.52} A$$

The applicable hydraulic data is:

Q = 35 cfs
D = 27 inches; whence, A = 3.97 ft^2 R = 0.563 ft
S_f = 7.5/120 = 0.063

We will check the sediment flow for the sediment characteristics in Problem 3.1 a and b

$$Q_s = 3.78\sqrt{32.2}\,(0.063)^{2.52}(.563)^{1.52}\, 3.97\, D_s^{-1.02}$$

$$Q_{smax} = 0.034\, D_s^{-1.02}$$

. for D_s = .31mm = 0.001 ft
Q_{smax} = 38.4 ft^3/sec

b. for D_s = 1.9mm = 0.006 ft
Q_{smax} = 6.04 ft^3/sec

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Appendix 4 : HIRE

Solved Problems - Chapter 4

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Problem 4.1 Meandering and Braiding

(a) Consider the sinuous point bar stream in Problem 4.1 (Figure 4.A1.1). Determine the following characteristics: meander wavelength λ; meander width W_m; mean radius of curvature r_c; meander amplitude A; and the bend deflection angle ϕ.

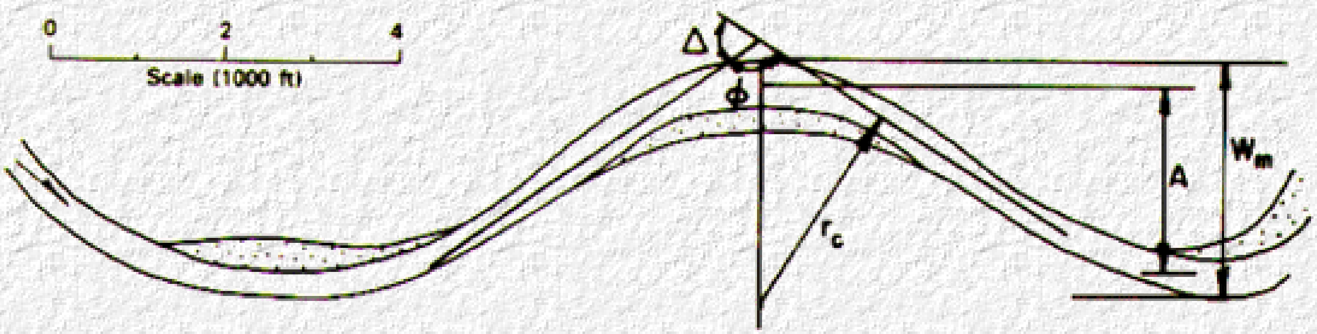

Figure 4.A1.1. Sinuous Point Bar Stream

> The meander wavelength λ is approximately 11,000 ft;
> The radius of curvature r_c is approximately 2400 ft;
> The channel width ranges from 250 ft to 850 ft at high stage;
> The meander amplitude A is 2200 ft;
> The meander width W_m is 2700 ft;
> The bend deflection angle ϕ is 105º, thus Δ is 75º; and
> The sinuosity is 1.2.

(b) Given the sand size D_{50} = 0.5mm, the bankfull discharge Q = 10,000 cfs and the slope S = $2x10^{-4}$, determine the effect of increasing slope, discharge, sediment size and sediment discharge on the planform geometry.

The result of increasing any or several of these variables (Figure 4.4.3 and Table 4.5.1) is to promote braiding of this channel. When locating this stream on Lane's diagram (Figure 4.4.3) it is shown that with $SQ^{1/4}$ = 0.002, this river is very close to the line $SQ^{1/4}$ = 0.0017 for meandering streams. Hence an increase in discharge or slope would be required to change the planform to a braided stream.

(c) Determine the effect of increased discharge of bed sediment size, bed sediment load and washload on channel stability, resistance to flow, energy slope and stage of the same river.

The Table 4.5.2 can be used to provide a qualitative response to these changes.

- An increase in discharge results in an increase in stage and a decrease in energy slope and channel stability.

- An increase in sediment size results in an increase in stage, energy slope and resistance to flow. The channel stability might not be changed.
- An increase in bed sediment load should increase the channel stability through a decreased resistance to flow, slope and stage.
- The effect of increasing washload is similar to that of increasing bed sediment load except for channel stability, which is uncertain.

Problem 4.2 Classification of Alluvial Reaches

Identify the three types of alluvial river reaches sketched below. Discuss the relative stability of each channel.

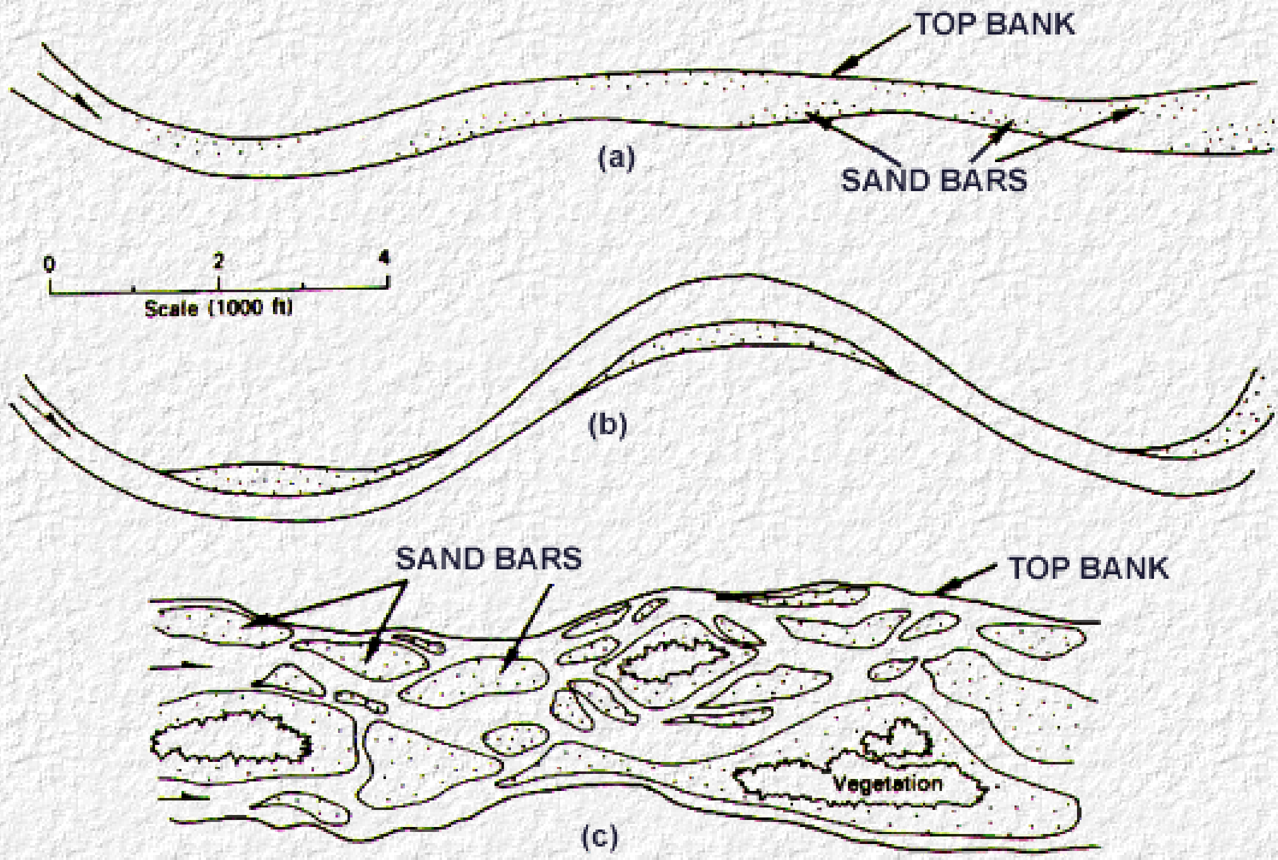

Figure 4.A1.1b. River Channels (after Petersen, 1986)

Based on the classification presented in Figure 4.3.1, Figure 4.3.2, and Figure 4.3.3. The first stream (a) is a straight channel with alternate bars and sinuous thalweg. This stream has a relatively low slope and low width-depth ratio. Figure 4.3.3 also indicates that it has an intermediate sediment size, sediment load and moderate ratio of bedload to total load (between 3-10%). This stream can be classified as relatively stable.

The second stream (b) is a sinuous point bar stream which is somewhat wider at bends. The meander bends are expected to shift gradually with possible neck cutoff. The stream is then relatively stable.

The third stream (c) is a braided sand-bed stream with multiple bars and channels. Bars are likely to be comprised of coarse sand and the large flow velocities and stream power will generate large sediment load with a large proportion transported as contact load. The overall stability of this braided channel is certainly very low, channel shifting, and convulsions are certainly common.

Problem 4.3 Channel Response to Changes in Watershed Condtitions

Determine the effect of watershed deforestation, bank erosion and headcutting on channel stability and gradation changes in an alluvial stream.

Referring to Table 4.7.1, deforestation generally causes aggradation problems and therefore channel instability. The reason for this is that deforestation increases runoff and peak runoff discharge as well as sediment transport from upland areas.

Headcutting is a degradation process. Upstream migration of headcuts induces bank failure and channel stability problems. Bank erosion has basically the same consequence on channel stability as that of headcutting. The gradation changes, however, are more difficult to assess because bank erosion changes the width-depth ratio.

Problem 4.4 Channel Migration Rate

(a) Determine the bank erosion rate and the erosion index of the sinuous point bar stream sketched in Figure 4.A1.1(b).

The best method to estimate the rate of bank erosion is to compare two sets of aerial photographs. However, a first assessment can be obtained from Figure 4.8.1. Entering the figure with an average width around 150m, the median erosion rate should be around 2 meters per year. Wide bend streams have slightly larger erosion rates than given by the dashed line. Entering Figure 4.8.3 with a sinuosity of 1.2, the erosion index might be as large as 18 indicating channel instability. Erosion is to be expected on the concave side of the bends. Other means for assessing lateral migration rates such as study of past aerial photographs, scroll formation and field studies should be undertaken if any bridge crossing was to be built across this river.

(b) Sketch the likely future changes in the meandering river shown in the sketch below.

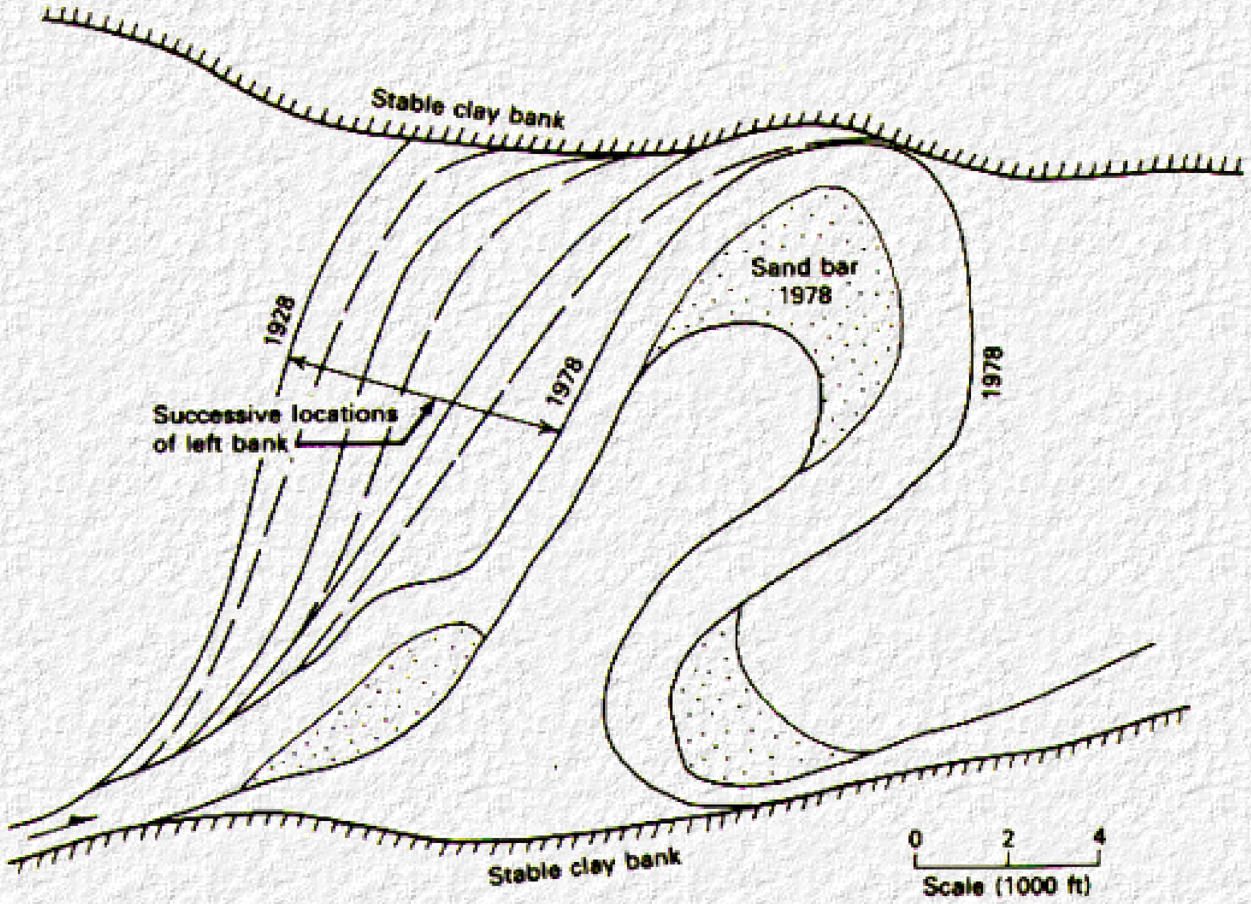

Figure 4.A1.2. Meandering River Sketch (after Petersen, 1986)

This meandering channel will most likely lead to a cutoff in the next 10 to 20 years. The estimated position of the channel is sketched below.

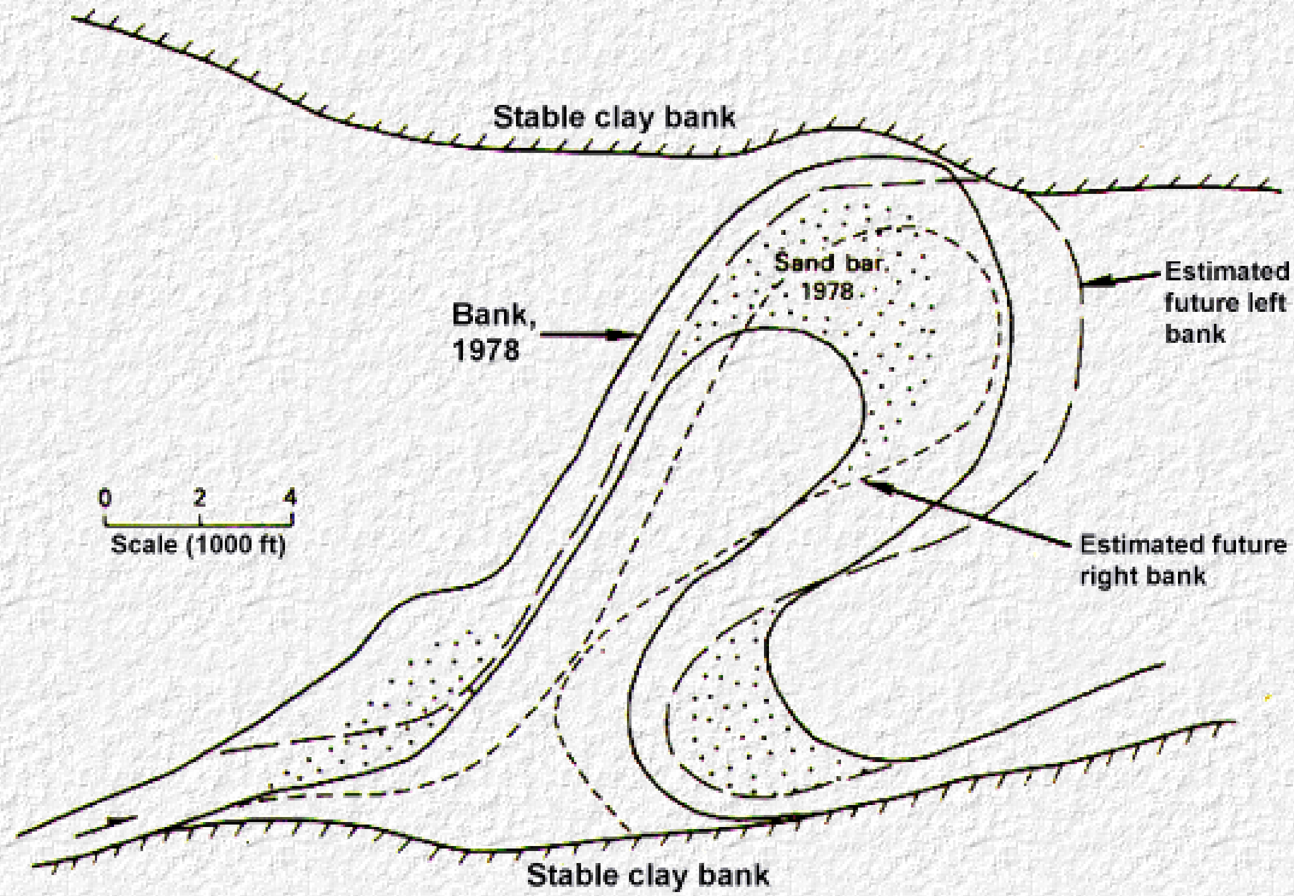

Figure 4.A1.3. Estimated Future Location of a Meandering River (after Petersen, 1986)

Problem 4.5 At-A-Station and Downstream Hydraulic Geometry Relationships

At bankfull discharge conditions Q_1 = 8000 cfs and the width of a sand-bed stream (D_{s1} = 0.6 mm) is W_1 = 250 ft, the maximum flow depth is y_{o1} = 8 ft, the slope is $S_{f1} = 2.5 \times 10^{-4}$, and the maximum velocity is V_1 = 5 ft/s.

(a) Estimate the width, W_2, depth y_{o2}, slope S_{f2} and velocity V_2 at the same station when the discharge Q_2 is 200 cfs if the cross-sectional geometry is unknown.

The at-a-station hydraulic geometry relationships (Equations 4.4.1 to 4.4.4) can be used when no specific field data is available.

For width

$$W_2 = W_1\left(\frac{Q_2}{Q_1}\right)^{0.26} = 250\times\left(\frac{200}{8000}\right)^{0.26} = 96\text{ ft}$$

For depth

$$y_{o2} = y_{o1}\left(\frac{Q_2}{Q_1}\right)^{0.40} = 8\left(\frac{1}{40}\right)^{0.40} = 1.8\ \text{ft}$$

Slope is unchanged

$S_{f1} = S_{f2} = 2.5 \times 10^{-4}$

For velocity

$$V_2 = V_1\left(\frac{Q_2}{Q_1}\right)^{0.34} = 5\times\left(\frac{1}{40}\right)^{0.34} = 1.4\ \text{ft/s}$$

b) Estimate the width, W_2. depth y_{o2}, slope S_{f2} and velocity V_2 in a gravel bed stream d_{s2} = 8 mm if its bankfull discharge Q_{b2} is 500 cfs.

The downstream geometry relationships must be used in this case. Two types of relationships are given in the text: Equations 4A.4.5 to 4A.4.8 are a function of discharge only whereas Equations 4A.4.9 to 4A.4.12 are a function of both discharge and sediment size. Both methods are compared in the following.

Flow Depth

$$y_{o2} = y_{o1}\left(\frac{Q_{b2}}{Q_{b1}}\right)^{0.46} = 8\times\left(\frac{500}{8000}\right)^{0.46} = 2.2\ \text{ft}$$ Equation 4A.4.5

$$y_{o2} = y_{o1}\left(\frac{Q_{b2}}{Q_{b1}}\right)^{0.40} = 8\times\left(\frac{1}{16}\right)^{0.40} = 2.6\ \text{ft}$$ Equation 4A.4.9

Channel width

$$W_2 = W_1\left(\frac{Q_{b2}}{Q_{b1}}\right)^{0.46} = 250\left(\frac{1}{16}\right)^{0.46} = 70\ \text{ft}$$ Equation 4A.4.6

$$W_2 = W_1\left(\frac{Q_{b2}}{Q_{b1}}\right)^{0.53}\left(\frac{D_{s2}}{D_{s1}}\right)^{-0.33} = 250\left(\frac{1}{16}\right)^{0.53}\left(\frac{8}{0.6}\right)^{-0.33} = 25\ \text{ft}$$ Equation 4A.4.10

Friction slope

$$S_{f2} = S_{f1}\left(\frac{Q_{b2}}{Q_{b1}}\right)^{-0.46} = 2.5\times10^{-4}\left(\frac{1}{16}\right)^{-0.46} = 9\times10^{-4} \qquad \text{Equation 4A.4.7}$$

$$S_{f2} = S_{f1}\left(\frac{Q_{b2}}{Q_{b1}}\right)^{-0.4}\left(\frac{D_{s2}}{D_{s1}}\right)^{1} = 2.5\times10^{-4}\left(\frac{1}{16}\right)^{-0.4}\left(\frac{8}{.6}\right) = 0.01 \qquad \text{Equation 4A.4.11}$$

Velocity

$$V_2 = V_1\left(\frac{Q_{b2}}{Q_{b1}}\right)^{0.08} = 5\left(\frac{1}{16}\right)^{0.08} = 4\ \mathrm{ft/s} \qquad \text{Equation 4A.4.8}$$

$$V_2 = V_1\left(\frac{Q_{b2}}{Q_{b1}}\right)^{0.07}\left(\frac{D_{s2}}{D_{s1}}\right)^{.33} = 5\left(\frac{1}{16}\right)^{0.07}\left(\frac{8}{0.6}\right)^{.33} = 9.7\ \mathrm{ft/s} \qquad \text{Equation 4A.4.12}$$

Equations 4A.4.9 to 4A.4.12 give a steeper, faster flowing and narrower channel as compared to the first set of equations (4A.4.5 to 4A.4.8). Unless the sediment size is markedly different for two streams, the resulting hydraulic geometry calculated from both sets of equations will be similar.

Problem 4.6 Downstream Sediment Size Distribution

Measurements of sediment size in the St.-Lawrence Seaway between Cornwall and Valleyfield are given in the following table.

Table 4.A1.1 Sediment Size Distribution in the St. Lawrence Seaway

River Mile Downstream of Cornwall (miles)	D_{50} mm
0	28
15	0.25
25	0.018
30	0.003
35	0.001

Estimate the mean sediment size D_{50} 10 miles and 20 miles downstream of Cornwall.

The gradual decrease in sediment size with downstream distance can be approximated by the following equation $D_{50} = 28 \times 10^{-0.132x}$ obtained by regression analysis based on Equation 4.4.17. At a distance of

10 and 20 miles, the expected mean sediment size D_{50} obtained by this relationship are respectively 1.34 mm and 0.064 mm.

Problem 4.7 Scale Ratios for Physical Models

A physical model is to be built in the Hydraulics Laboratory to simulate the flow pattern around a structure in a complex multiple channel stream. About 3600 ft^2 of space is available in the laboratory to model a 3 mile reach. Knowing that the same fluid is used for both the model and the prototype, determine the appropriate scale ratios for time, discharge and force. Also, required is the flow depth in the model at the location where flow depth reaches 100 ft in the prototype.

A fixed boundary model is considered and open channel flow modeling is scaled by similarity in Froude number. The scale ratios for γ and ρ equal unity and thus, scaling depends uniquely on the length scale L = 60/(3 x 5280) = 3.78 x 10^{-3}. The following scale ratios for time, discharge and force are calculated from the expressions shown in Table 4.6.1.

Parameter	Scale Ratio
Time	$(L\rho/\gamma)^{1/2} = L^{1/2} = 0.062$
Discharge	$L^{5/2} (\gamma/\rho)^{1/2} = L^{5/2} = 8.83 \times 10^{-7}$
Force	$L^3\gamma = L^3 = 5.43 \times 10^{-8}$

The flow depth of the model at h = 100 ft is given by the product hL = 0.38 ft. Distortion might not be necessary in this model because of the large flow depths involved.

Problem 4.8 Comparison of the Short Term Mathematical Models

It is intended to study aggradation and degradation in a 4.7 mile long reach of the San Lorenzo River in the Santa Cruz County in northern California. The reach is subdivided in two different subreaches: The relatively steep upper half and the 2.4 mile long lower half reach has a much smaller slope. Information on the thalweg profiles at the peak discharge of the February 16-20, 1980 flood are needed. Pre-flood channel cross-sectional profiles, rating curves by size fractions and bed material composition data are available. The peak discharge was 12,800 cfs and the median bed material size ranges from 0.34 mm to 0.93 mm. This problem involves the application of mathematical models for routing water and sediments in alluvial streams.

The thalweg profiles at the peak discharge computed by HEC2SR, UUWSR, and FLUVIAL-11 (Holly et al. 1984) are shown in Figure 4.A1.4 together with the initial thalweg profile. As seen in the figure, UUWSR and FLUVIAL-11 predicted significant changes in thalweg elevation compared with the HEC2SR prediction. The general agreement of predictions of thalweg elevations among the three models is seen to be limited to extremely small portions of the study reach. Longitudinal distributions of the total-sediment discharge, mean flow velocity, and median bed material size at peak flow were also found to differ significantly among the three models.

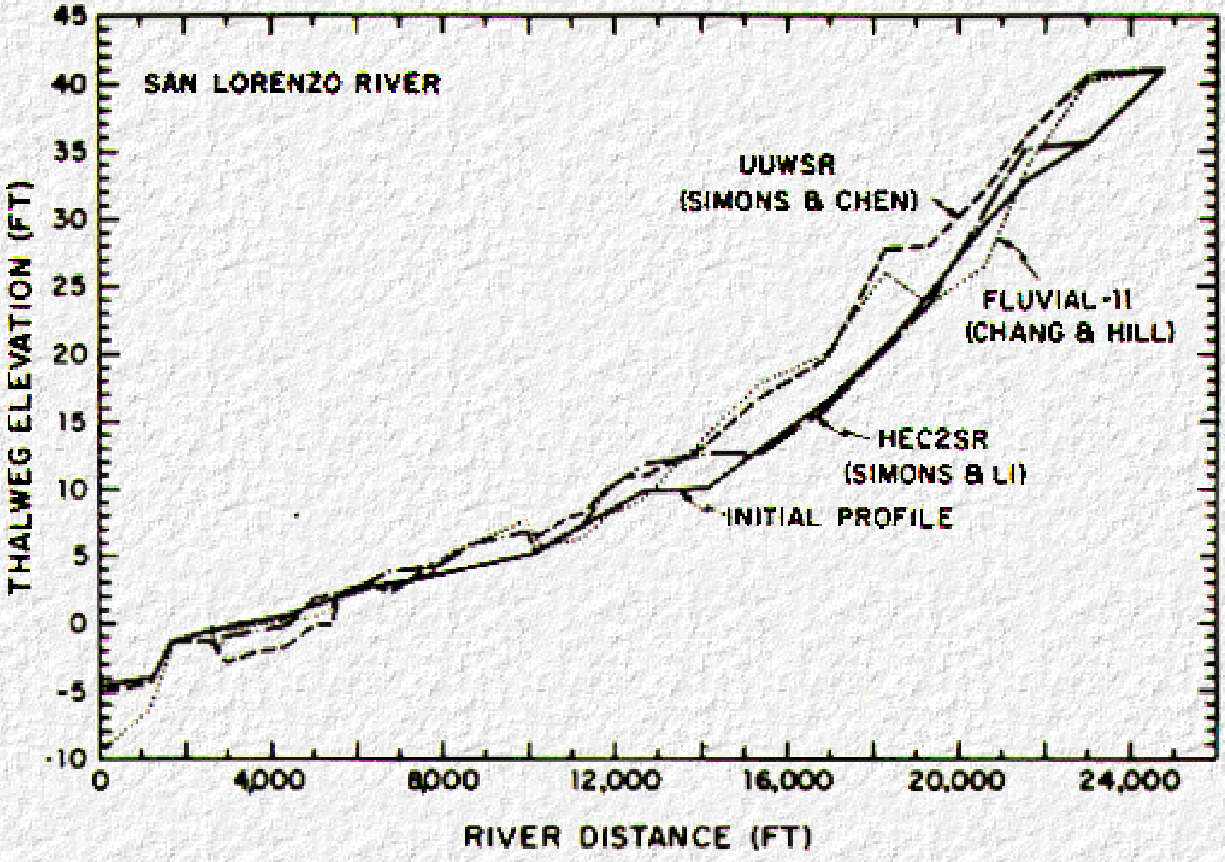

Figure 4.A1.4. Comparison of Computed Thalweg Elevations for the San Lorenzo River (after Holly et al., 1984)

Problem 4.9 Comparison of Long-Term Mathematical Models

Periodic high-cost maintenance dredging has been necessary to maintain the 9 ft depth along the passageway in the vicinities of Fox and Buzzard Islands (RM-355-6 and RM349-50) of pool 20 on the Mississippi River. To understand the basic mechanisms responsible for the shoaling problems, two field studies were conducted to obtain detailed information about the flow and sediment transport characteristics along the shoaling reaches. On the basis of the field data collected, detailed geometric, hydrologic, and sediment-input data were prepared and the three models, KUWASER, HEC-6, and CHAR II, were tested at Colorado State University, IIHR, and SOGREAH, respectively (Holly et al., 1984).

Simulation runs of these models were made for a 28-month period between May 1976 and August 1978. Figure 4.A1.5 shows the initial, computed and measured thalweg elevations. The degree of agreement between the computed and measured values is seen to be of almost the same order for each model. It should be noted that KUWASER used a 5-day time step for a water discharge over 100,000 cfs, a 10-day time step for a discharge between 50,000 cfs and 100,000 cfs, and a 30-day time step for a discharge below 50,000 cfs. HEC-6 used monthly averaged flow quantities, and CHAR II used a temporal computation interval ranging between 6 hours and 5 days.

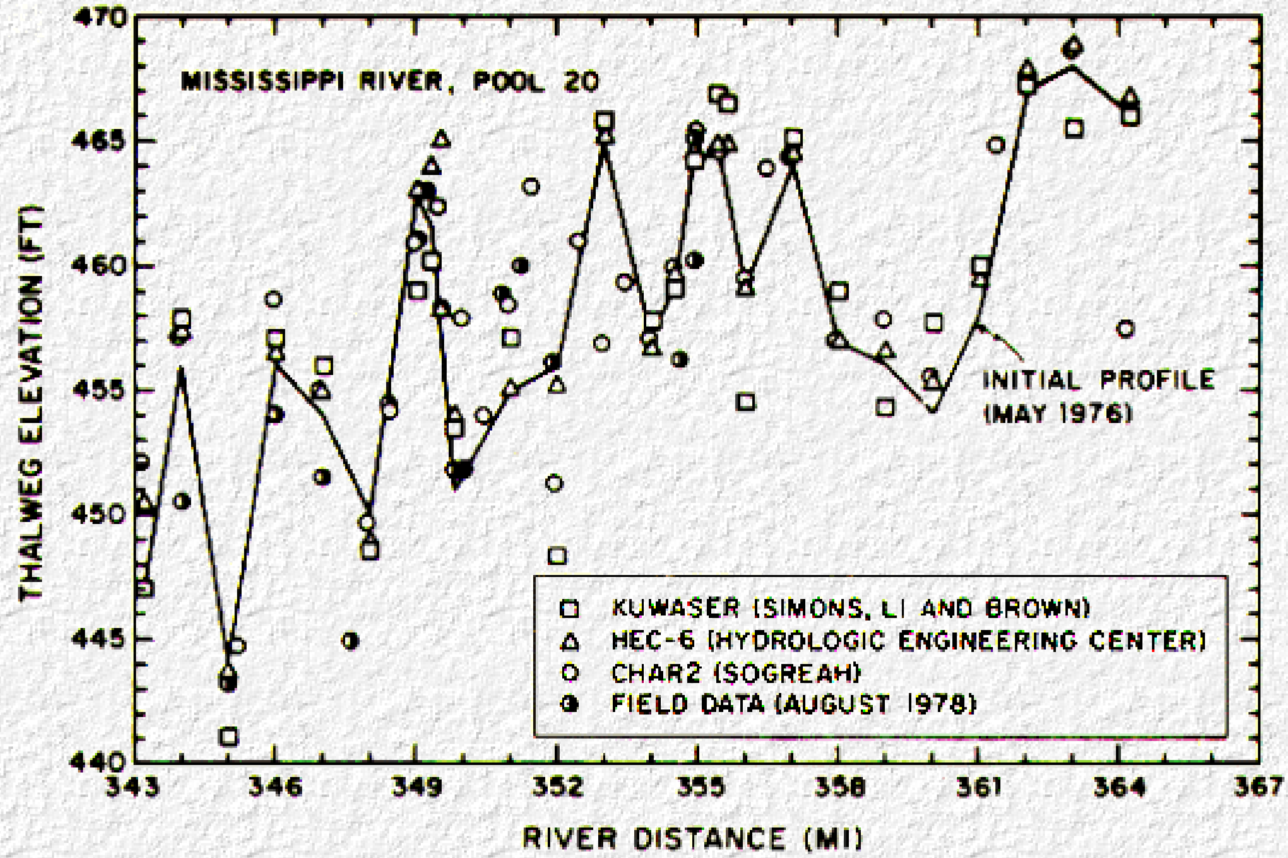

Figure 4.A1.5. Comparison of Computed Thalweg Elevations and Several Spot Measurements for Pool 20, Mississippi River (after Holly, et al., 1984)

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Appendix 5 : HIRE

Solved Problems - Chapter 5

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Problem 5.1 Stability of Particles under Downslope Flow

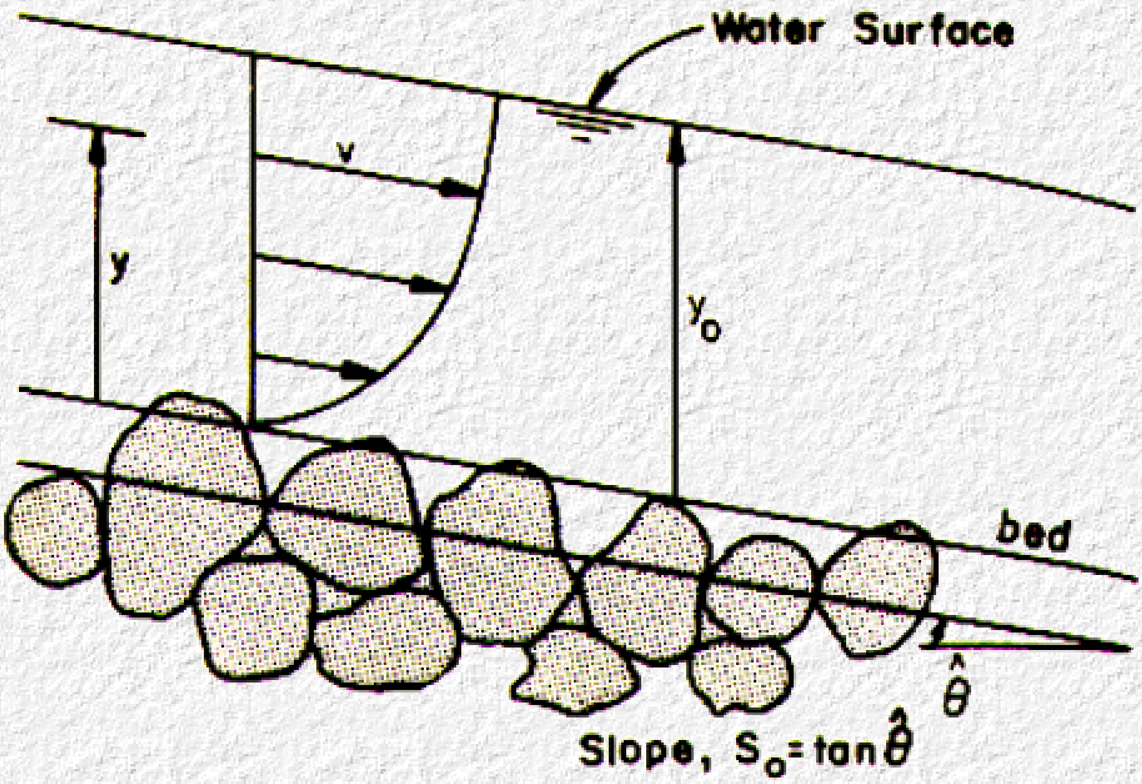

Figure 5.A1.1. Definition Sketch for Riprap on a Channel Bed

Downslope flow over a plane bed inclined at an angle $\hat{\theta}$ shown in Figure 5.A1.1 is equivalent to an oblique flow on a side slope with Θ = $\hat{\theta}$ and λ = 90°. Then, according to Equation 5.2.4, β = 0, and from Equation 5.2.6,

$$\eta' = \eta\left(\frac{1 + \sin 90° + 0}{2}\right) = \eta$$

It follows from Equation 5.2.3 that the stability factor is

$$S.F. = \frac{\cos\hat{\theta}\tan\phi}{\eta\tan\phi + \sin\hat{\theta}}$$

Alternatively, solving for η yields

$$\eta = \cos\hat{\theta}\left[\frac{1}{S.F.} - \frac{\tan\hat{\theta}}{\tan\phi}\right]$$

Using this information we wish to calculate if, ϕ = 40°, what is the maximum bed angle $\hat{\theta}$ at which η will be 5% different from that of a horizontal bed. Solving for $\hat{\theta}$ with η = 0.95, S. F. = 1 and ϕ = 40° yields $\hat{\theta}$ = 2.35° or 4.1%.

Problem 5.2 Riprap Design on Embankment Slopes

In a design situation water flows parallel to an embankment built of crushed rock riprap, (S_s = 2.65), at a sloping angle θ = 20º.

(a) If the design shear stress is τ_o = 2 psf, calculate the riprap size that gives a stability factor equal to 1.5.

Assuming ϕ = 40º, the stability number η is obtain from Equation 5.2.3 or Figure 5.2.2 for θ = 20º (η = 0.36). The stone size is then calculated from Equation 5.2.13

$$D_m = \frac{21\tau_0}{(S_s - 1)\gamma\eta} = \frac{21 \times 2}{(2.65 - 1)62.4 \times 0.36} = 1.13 \text{ ft}$$

(b) For the same design shear stress τ_o = 2 psf, determine the stability factor of particle sizes D_m = 6 inches. From Equation 5.2.5

$$\eta = \frac{21\tau_0}{(S_s - 1)\gamma D_m} = \frac{21 \times 2}{(2.65 - 1)62.4 \times 0.5} = 0.82$$

Then, Equations 5.2.9, 5.2.10 and 5.2.11 are used to calculate the stability factor

$$S_m = \frac{\tan\phi}{\tan\theta} = \frac{\tan 40°}{\tan 20°} = 2.31$$

$$\zeta = S_m\, \eta \sec\theta = 2.31 \times 0.82 \sec 20 = 2.02$$

$$S.F. = \frac{S_m}{2}\left(\left(\zeta^2 + 4\right)^{1/2} - \zeta\right) = \frac{2.31}{2}\left(\left(2.02^2 + 4\right)^{1/2} - 2.02\right) = 0.95 < 1$$

This size fraction (D_m = 6 inches) is unstable.

Problem 5.3 Stability Factors for Riprap Design

For the case shown in Figure 5.2.5, the angle between the horizontal and the velocity vector at the point "P" is $\lambda = 20°$. The method of computing the streamlines and velocities around an embankment end is given in Section 5.2.4. If the drawdown through the bridge is large, the reference velocity v_r in the vicinity of the riprap can be as large as 6 fps.

Suppose that the reference velocity is v_r = 6 fps and the embankment side slope angle is $\theta = 18.4°$ which corresponds to a 3:1 side slope. If the embankment is covered with dumped rock having a specific weight $S_s = 2.65$ and an effective rock size D_m = 1.0 ft, determine the stability factor.

From Equation 5.2.17

$$\eta = \frac{0.30\, v_r^2}{(S_s - 1)gD_m} = \frac{(0.30)(6)^2}{(2.65-1)(32.2)(1.0)} = 0.203$$

This dumped rock has an angle of repose of approximately 35° according to Figure 3.2.4. Therefore, from Equation 5.2.4

$$\beta = \tan^{-1}\left\{\frac{\cos\lambda}{\dfrac{2\sin\theta}{\eta\tan\phi} + \sin\lambda}\right\} = \tan^{-1}\left\{\frac{\cos 20°}{\dfrac{2\sin 18.4°}{0.203\tan 35°} + \sin 20°}\right\} = 11°$$

and from Equation 5.2.6

$$\eta' = \eta\left\{\frac{1+\sin(\lambda+\beta)}{2}\right\} = 0.203\left\{\frac{1+\sin(20^\circ+11^\circ)}{2}\right\} = 0.154$$

The stability factor for the rock is given by Equation 5.2.3

$$S.F. = \frac{\cos\theta\tan\phi}{\eta'\tan\phi+\sin\theta\cos\beta} = \frac{\cos 18.4^\circ\tan 35^\circ}{0.154\tan 35^\circ+\sin 18.4^\circ\cos 11^\circ} = 1.59$$

Thus, with a stability factor of 1.59, this rock is more than adequate to withstand the flow velocity.

By repeating the above calculations over the range of interest for D (with ϕ = 35°), the curve given in Figure 5.A1.2 is obtained. This curve shows that the incipient motion rock size is approximately 0.35 ft and the maximum stability factor is less than 2.0 on the 3:1 side slope.

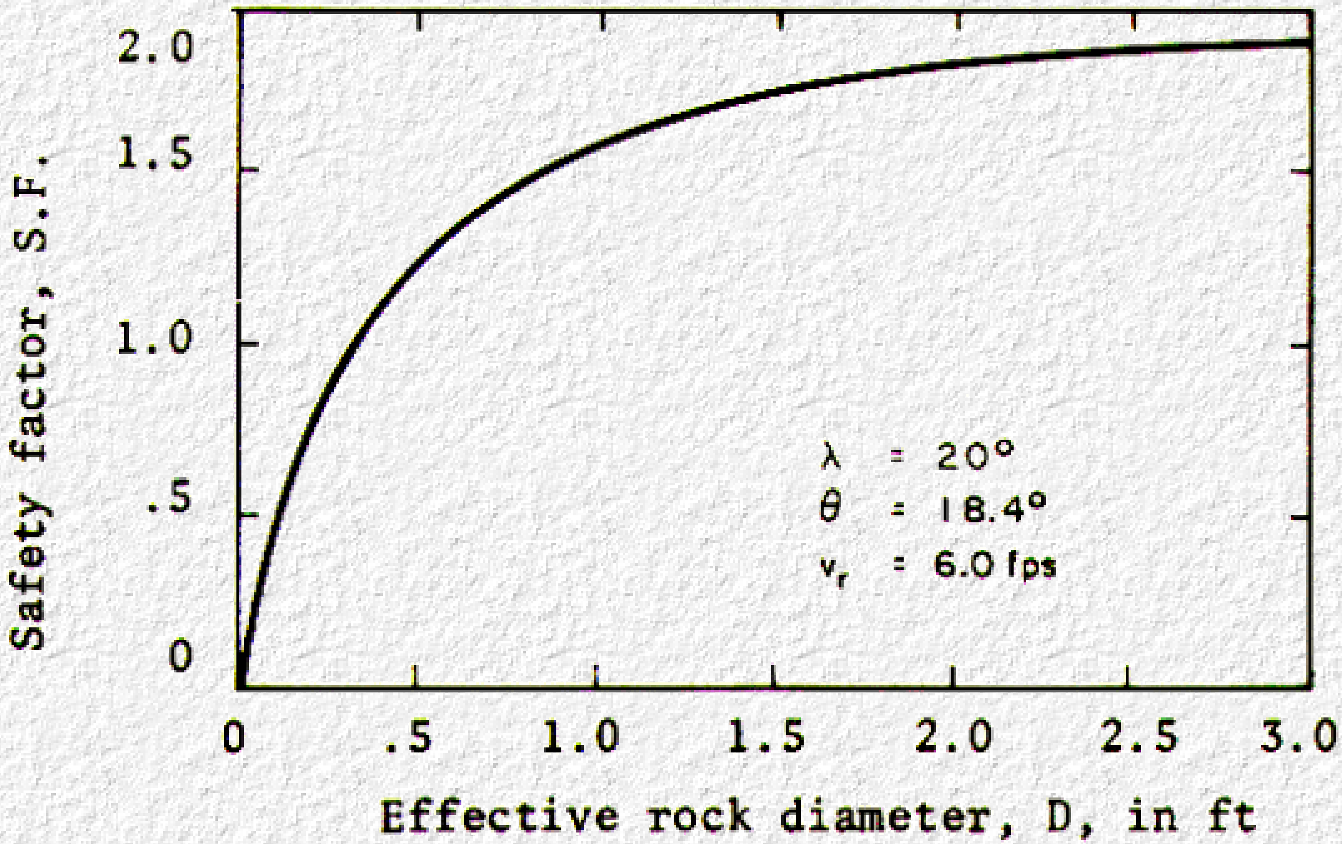

Figure 5.A1.2. Stability Factors for Various Rock Sizes on a Side Slope

The stability factor of a particular side slope riprap design can be increased by decreasing the side slope angle θ. If the side slope angle is decreased to zero degrees, then Equation 5.2.3 is applicable and

$$S.F. = \frac{1}{\eta} = \frac{1}{0.203} = 4.93$$

The curve in Figure 5.A1.3 relates the stability factor and side slope angle of the embankment (for λ = 20°, D = 1.0 ft and V_r = 6.0 fps). The curve is obtained by employing Equations 5.2.3, 5.2.4 and 5.2.6 for various values of θ.

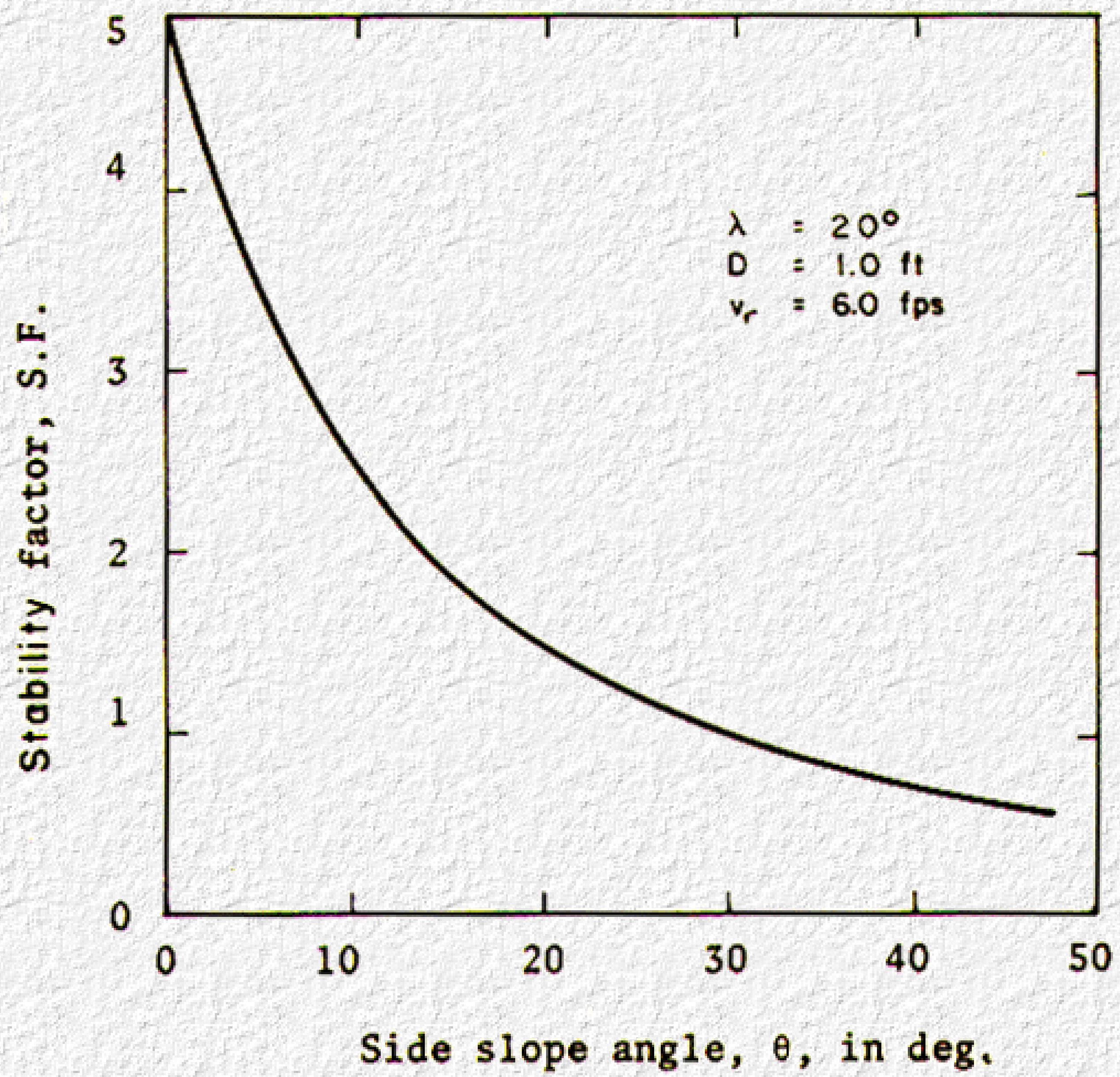

Figure 5.A1.3. Safety Factors for Various Side Slopes

Problem 5.4 Riprap Design on an Abutment

Consider the following abutment characteristics: Upstream velocity V_1 = 3 ft/s, α_1 = 1.2; the water surface drop is 2 ft along a distance of 50 ft; and the abutment slope angle θ = 18.5º. Determine the size of riprap required to resist the erosive forces at the toe of the abutment.

The velocity around the upstream spill slope V_p is determined from Figure 5.2.8. The drop ratio $\Delta h/L_a$ = 2/50 = 0.04 and V_p/V_d = 0.5. From Equation 5.2.19

$V_p = 0.5\ V_d = 0.5\ (\ \alpha_1\ V_1^2 + 2g\ \Delta h\ )^{1/2}$

$V_p = 0.5\ (\ 1.2 \times 3^2 + 2 \times 32.2 \times 2)^{1/2} = 5.91$ ft/s.

The angle λ for $\Delta H/L = 0.04$ is 20° as obtained from Figure 5.2.9.

The spill slope riprap size is obtained by iteration from assuming D_m and calculating successively Equation 5.2.16, Equation 5.2.4, Equation 5.2.6 and Equation 5.2.3 until the stability factor, S.F. equals 1.5. Using Figure 5.2.5 as a first approximation for a 3:1 side slope, a stone size of 0.3 ft and a velocity V_p of 6 ft/s is first used in the calculation. From Figure 3.2.4, $\phi \simeq 40°$.

From Equation 5.2.17,

$$\eta = \frac{0.3 \times 6^2}{(2.65 - 1) \times 32.2 \times 0.3} = 0.68$$

From Equation 5.2.4,

$$\beta = \tan^{-1}\left\{\frac{\cos 20}{\dfrac{2\sin 18.5}{0.68\tan 40} + \sin 20}\right\} = 32.8$$

From Equation 5.2.6,

$$\eta' = 0.68\left\{\frac{1 + \sin(20 + 32.8)}{2}\right\} = 0.61$$

From Equation 5.2.3,

$$S.F. = \frac{\cos 18.5 \tan 40}{0.61 \tan 40 + \sin 18.5 \cos 32.8} = 1.02$$

The procedure is repeated with increasing stone size until S. F. = 1.5. A riprap size of 9 inches would give a S. F. slightly over 1.5.

The riprap size at the toe of the abutment is determined in a similar manner from the velocity obtained from Figure 5.2.8 and Equation 5.2.19

$V_t = 0.8\ V_d = 0.8\ (\alpha_1\ V_1^2 + 2g\ \Delta h)^{1/2} = 0.8\ (1.2 \times 3^2 + 2 \times 32.2 \times 2)^{1/2} = 9.45$ ft/s

It is assumed that $\lambda = 0$, and taking $\phi = 40°$, the riprap size is obtained either by the same interative procedure or with the aid of the method described in Section 5.2.2.

From Equation 5.2.11 S_m = tanφ/tanθ = tan 40/tan 18.5 = 2.51 and from Equation 5.2.12

$$\eta = \left(\frac{S_m^2 - S.F.^2}{S.F.\,S_m^2} \right) \cos\theta = \left(\frac{(2.5)^2 - 1.5^2}{1.5 \times 2.5^2} \right) \cos 18.5 = 0.40$$

the riprap size is obtained from Equation 5.2.17

$$D_m = \frac{0.3\,V_t^2}{(S_s - 1)g\eta} = \frac{0.3 \times (9.45)^2}{(2.65 - 1) \times 32.2 \times 0.4} = 1.26 \text{ ft}$$

This is the size recommended.

Problem 5.5 Filter Design

The requirements for a gravel filter are given in Section 5.2.6. The gradation of a filter should be such that

$$\frac{D_{50}(\text{FILTER})}{D_{50}(\text{BASE})} < 40 \qquad \text{5A.2.22}$$

$$5 < \frac{D_{15}(\text{FILTER})}{D_{15}(\text{BASE})} < 40 \qquad \text{5A.2.23}$$

$$\frac{D_{15}(\text{FILTER})}{D_{85}(\text{BASE})} < 5 \qquad \text{5A.2.24}$$

Filter design example 1

Consider a riprap blanket resting on a base material. The properties of the riprap and base material are given in Table 5.A1.1. Design a filter to be placed between the riprap and the base material.

Table 5.A1.1 Sizes of Materials

Base Material Sand	Riprap Gravel
D_{85} = 1.50 mm D_{50} = 0.75 mm D_{15} = 0.38 mm	D_{85} = 24 mm D_{50} = 12 mm D_{15} = 6 mm

In accordance with the recommended sizes for filters, we note that

$$\frac{D_{50}(RIPRAP)}{D_{50}(BASE)} = \frac{12}{0.75} = 16$$

which satisfies expression 5A.2.22. Also

$$\frac{D_{15}(RIPRAP)}{D_{15}(BASE)} = \frac{6}{0.38} = 16$$

which satisfies the requirement 5A.2.23. Moreover

$$\frac{D_{15}(RIPRAP)}{D_{85}(BASE)} = \frac{6}{1.5} = 4$$

which satisfies the requirement 5A.2.24. The riprap itself satisfies the requirements for the filter so no filter is needed.

Filter Design Example 2

The following filter design is taken from Anderson et al. (1968). The properties of the base material and the riprap are given in Table 5.A1.2.

Table 5.A1.2 Sizes of Materials

Base Material Sand	Riprap Rock
D_{85} = 1.5 mm D_{50} = 0.5 mm D_{15} = 0.17 mm	D_{85} = 400 mm D_{50} = 200 mm D_{15} = 100 mm

The riprap does not contain sufficient fines to act as the filter because

$$\frac{D_{15}(RIPRAP)}{D_{85}(BASE)} = \frac{100}{1.5} = 67$$

which is much greater than 5, the recommended upper limit (expression 5A.2.24). Also

$$\frac{D_{15}(RIPRAP)}{D_{15}(BASE)} = \frac{100}{0.17} = 600$$

which is much greater than 40, the recommended upper limit (expression 5A.2.23).

The properties of the filter to be placed adjacent to the base are as follows:

$$\frac{D_{50}(\text{FILTER})}{D_{50}(\text{BASE})} < 40 \qquad (1)$$

so D_{50} (Filter) < (40) (0.5) = 20 mm

$$\frac{D_{15}(\text{FILTER})}{D_{15}(\text{BASE})} < 40 \qquad (2)$$

so D_{15} (Filter) < (40) (0.17) = 6.8 mm

$$\frac{D_{15}(\text{FILTER})}{D_{85}(\text{BASE})} < 5 \qquad (3)$$

so D_{15} (Filter) < (5) (1.5) = 7.5 mm

$$\frac{D_{15}(\text{Filter})}{D_{15}(\text{Base})} > 5 \qquad (4)$$

so D_{15} (Filter) > (5) (.17) = 0.85 mm

Thus, with respect to the base

0.85 mm < D_{15} (Filter) < 6.8 mm

and

D_{50} (Filter) < 2 mm

The properties of the filter to be placed adjacent to the riprap are as follows:

$$\frac{D_{50}(\text{RIPRAP})}{D_{50}(\text{FILTER})} < 40 \qquad (1)$$

so D (Filter) > $\frac{200}{40}$ = 5 mm

$$\frac{D_{15}(\text{RIPRAP})}{D_{15}(\text{FILTER})} > 5 \qquad (2)$$

so D (Filter) > $\frac{100}{5}$ = 20 mm

$$\frac{D_{15}(\mathrm{RIPRAP})}{D_{15}(\mathrm{FILTER})} < 40 \qquad (3)$$

so D (Filter) > $\frac{100}{40}$ = 2.5 mm

$$\frac{D_{15}(\mathrm{RIPRAP})}{D_{85}(\mathrm{FILTER})} < 5 \qquad (4)$$

so $D_{85}(\mathrm{FILTER}) > \frac{100}{5} = 20\ \mathrm{mm}$

Therefore, with respect to the riprap, the filter must satisfy these requirements

2.5mm < D_{15} (Filter) < 20 mm

D_{50} (Filter) > 5 mm

D_{85} (Filter) > 20 mm

These riprap filter requirements along with those for the base material are shown in Figure 5.A1.4. Any filter having sizes represented by the double cross-hatched area is satisfactory. For example, a good filter could have these sizes:

D_{85} = 40 mm

D_{50} = 10 mm

D_{15} = 4 mm

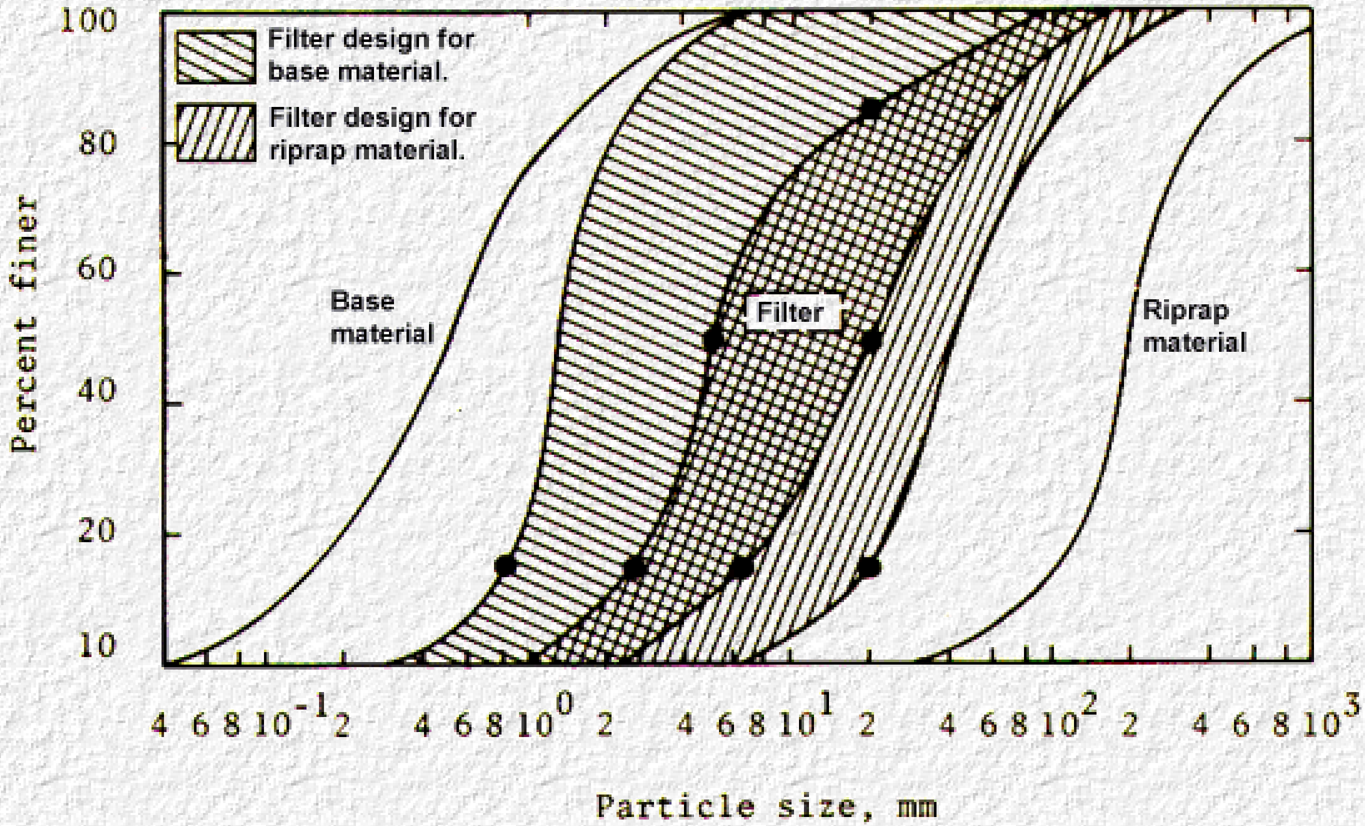

Figure 5.A1.4. Gradations of Filter Blanket for Example 2 (after Anderson et al., 1968)

Problem 5.6 Abutment Scour

Consider that the embankments encroach right to the edge of the main channel. Determine abutment scour depth for a 1000 ft embankment (transverse encroachment) under the following conditions obtained from water surface profile computations:

y_1 = main channel = 15'
y_o = overbank flow = 5'
q_{mc} = main channel = 150 cfs/ft
q_o = overbank = 30 cfs/ft (say 5' depth x 6 ft/sec vel)
a = 1000 ft each embankment
$Q_o = q_o \times a$ = 30,000 cfs
D_{50} = 5 mm
D_{75} = 6.0 mm
n_c = .030 main channel
n_o = .040 overbank
τ_{co} = 3.70 lb/ft^2 overbank with class A vegetative cover

Calculate:

$$\tau_o = \frac{f}{8}\rho V^2 = \frac{n^2}{2.22}\gamma V^2 R_h^{-1/3}$$

where

f = Darcy-Weisbach friction factor
ρ, γ = are density and specific weight of water
n = Manning roughness factor
R_h = hydraulic radius = depth of flow in a wide open channel.

$$\tau_o = \frac{(0.03)^2}{2.22} \times 62.4 \times 10^2 \times 15^{-1/3} = 1.02\,\text{lb/ft}^2 \text{ (for main channel)}$$

$$\tau_o = \frac{(0.04)^2}{2.22} \times 62.4 \times 6^2 \times 5^{-1/3} = 0.95\,\text{lb/ft}^2 \text{ (for overbank flow)}$$

τ_c = 0.0164 d_{75} = .0164 x 6.0 = 0.10 lb/ft^2 (for main channel)

Check main channel and overbank

$t_o > t_c$ for main channel	O.K.
$t_o < t_c$ for overbank flow	O.K.

Estimate of scour

From Equation 5.5.16

$$\frac{y_s}{y_o} = 1.1\left(\frac{1000}{y_o}\right)^{0.40}\left(\frac{V_o}{\sqrt{g y_o}}\right)^{0.33}$$

Use y_o = 5'; V_o = 6 ft/sec; then

$$\frac{y_s}{y_o} = 1.1\left(\frac{1000}{5}\right)^{0.40}\left(\frac{6}{\sqrt{32.2\times 5}}\right)^{0.33} = 7.15$$

Using Laursen's method (Equation 5.5.22)

$$\frac{Q_o}{q_{mc}y_o} = \frac{30{,}000}{150\times 5} = 40$$

$$40 = 2.75\frac{y_s}{y_o}\left[\left(\frac{y_s}{4.1y_o}+1\right)\right]^{7/6} - 1$$

By trial and error, one obtains

$$\frac{y_s}{y_o} = 6.8$$

Both methods are found to be in good agreement for this example.

Problem 5.7 Scour Around Piers

Calculate the equilibrium scour around a circular pier 4 ft wide in a sandbed channel D_{50} = 1 mm if the flow velocity during floods V = 8 ft/s and the flow depth y_1 = 15 ft.

From Equation 5.5.25

$$y_s = 2.0y_1\left(\frac{a}{y_1}\right)^{0.65} F_{r1}^{\;0.43}$$

$$y_s = 2.0\times 15\times\left(\frac{4}{15}\right)^{0.65}\left[\frac{8}{\sqrt{32.2\times 15}}\right]^{0.43} = 8.23\text{ ft}$$

Using Jain and Fischer's method, F_c must be determined first from Figure 3.5.2.

$$\tau_c \simeq 0.04\text{ lb/ft}^2$$

$$V_{*c} = \left(\frac{\tau_c}{\rho}\right)^{1/2} = \left(\frac{0.04\,\text{lb ft}^3}{62.4\,\text{lb ft}^2}\right)^{1/2} \times \left(\frac{32.2\,\text{ft}}{\text{Sec}^2}\right)^{1/2} = 0.144\ \text{ft/s}$$

$$\delta = \frac{11.6 \times 1.08 \times 10^{-5}\,\text{ft}^2/\text{sec}}{0.144\,\text{ft/sec}} = 8.7 \times 10^{-4}\,\text{ft}$$

$$\frac{D_{50}}{\delta} = \frac{1\,\text{mm} \times 1\,\text{ft}}{305\,\text{mm} \times 8.7 \times 10^{-4}\,\text{ft}} = 3.75$$

From Figure 2.3.5, X = 1.2

$$V_c = V_{*c}\,[2.5\,\ell n\,(11\,y_1 X/d_{50})\,]$$

$$= 0.144\,[2.5\,\ell n\,(11 \times 15 \times 1.2 \times 305/1)]$$
$$= 3.96\ \text{ft/sec}$$

$$F_c = \frac{3.96}{\sqrt{32.2 \times 15}} = 0.18$$

$$F_{r1}\ \frac{8}{\sqrt{32.2 \times 15}} = 0.36$$

$$F_{r1} - F_c = .18$$

From Equation 5.5.27, the scour depth is

$y_s = a\ 1.84\ F_c^{\ 0.25}\ (y_1/a)^{\ 0.3}$

$y_s = 4 \times 1.84 \times (0.18)^{.25}\ (15/4)^{\ 0.3}$

$y_s = 7.13$ ft

This value is comparable to that computed using Equation 5.5.25.

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16. Abstract
The federal Highway Administration document "Highways in the River Environment - Hydraulic and Environmental Design Considerations" was published in 1975. It has proven to be a singularly authoritative cocument for the design of highway associated hydraulic structures in moveable boundary waterways. This revised document incorporates many technological advances that have been made in this discipline since 1975. Hydraulic problems at stream crossings are described in detail and the hydraulic principles of rigid and moveable boundary channels are discussed.
In the United States, the average annual damage related to hydraulic problems at highway facilities on the Federal-aid system is $40 million. Damages by streams can be reduced signigicantly by considering channel stability. The types of rever changes to be carefully considered relate to: 1) lateral bank erosion; 2) degradation and aggradation of the streambed that continues over a period of years, and 3) natural short-term fluctuations of streambed elevation that are usually associated with the passage of floods. Sediment transport, natural and manmade causes of waterway responses, stream stabilization (bed and banks), computerapplications and countermeasures are some of the many issues

covered. Case histories of typical manmade and natural impacts on waterways are analyzed.

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HEC 7	8 Superseded by HY 6 and HDS 5			
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RES	Streambank Stabilization Measures for Highway Stream Crossings, Exec. Sum.	1985	RD-84-099	
RES	Streambank Stabilization Measures for Highway Engineers	1985	RD-84-100	
RES	Design of Spur-Type Streambank Stabilization Structures	1985	RD-84-101	

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A

Abrasion

Removal of streambank material due to entrained sediment, ice, or debris rubbing against the bank.

Afflux

Backwater or height by which water levels are raised at a stated point, owing to presence of a constriction or obstruction, such as a bridge.

Aggradation

General and progressive upbuilding of the longitudinal profile of a channel by deposition of sediment.

Aggradation (bed)

A progressive buildup or raising of the channel bed due to sediment deposition. Permanent or continuous aggradation is an indicator that a change in the stream's discharge and sediment load characteristics is taking place.

Alluvial

Soil and rock material deposited from flowing water.

Alluvial Channel

A channel wholly in alluvium, no bedrock exposed in channel at low flow or likely to be exposed by erosion during major flow.

Alluvial Fan

A landform shaped like a fan in plan view and deposited where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope.

Alluvium

Unconsolidated clay, silt, sand, or gravel deposited by a stream in a channel, flood plain, fan or delta.

Alternating Bars

Elongated deposits found alternately near the right and left banks of a channel.

Anabranch

Individual channel of an anabranched stream.

Anabranched Stream

A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars. The width of individual islands or bars is greater than three times water width.

Analysis

A term that means "to break apart" and that is applied to methods used to break down hydrologic data in order to develop a hydrologic model or design method (see synthesis).

Angle of Repose

The angle of slope formed by particulate material under the critical equilibrium condition of incipient sliding.

Annual Flood

The highest peak discharge in a water year.

Apron

Protective material laid on a streambed to prevent scour.

Apron, Launching

An apron designed to settle and protect the side slopes of a scour hole after settlement.

Armor

Artificial surfacing of channel beds, banks, or embankment slopes to resist streambed scour and/or lateral bank erosion. Compare with Apron, Blanket, Channel Lining, and Revetment.

Armoring

Armoring is a natural process whereby an erosion-resistant layer of relatively large particles is formed on a streambank and/or streambed due to the removal of finer particles by streamflow; i.e., the concentration of a layer of stones on the bed of the stream which are of a size larger than the transport capability of the recently experienced flow -- the winnowing out of smaller material capable of being transported while leaving the larger sizes as armor that, for discharges up to that point in time, can not be transported. Armoring may also refer to the placement of a covering on a streambank and/or streambed to prevent erosion.

Articulated Concrete Mattress or Mass

Rigid concrete slabs, which can move as scour occurs without separating, usually hinged together with corrosion-resistant wire fasteners; primarily placed for lower bank protection.

Asphalt Block

Precast or broken pieces of asphalt that can be hand-placed or dumped on a streambank or filter for protection against erosion.

Asphalt (bulk)

Mass of uncompacted asphalt usually dumped from a truck (upper bank protection) or a barge (lower bank protection) that is placed to protect the bank against erosion.

Attribute File

A computer file that assigns descriptive characteristics to map or georeferenced features. For example, a symbol might be plotted on a computer screen to show the location of a land cover feature. The attribute file would define characteristics such as the land cover type and percent of imperviousness represented by the symbol.

Autocorrection

The degree of association between values in a time or space series, such as the annual maximum flood series. Watershed changes, such as urbanization, can cause autocorrected flood series.

Average Velocity

Velocity at a given cross section determined by dividing discharge by cross-sectional area.

Avulsion

A sudden change in the course of a channel, usually by breaching of the banks during a flood.

Axial Flow Pumps

Pumps that lift the water up a vertical riser pipe; flow is parallel to the pump axis and drive shaft; commonly used for low head, high discharge applications.

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E

Eddy Current

A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.

Effective Duration

The time in a storm during which the water supply for direct runoff is produced. Also used to mean the duration of excess rainfall.

Effective Particle Size

The diameter of particles, spherical in shape, equal in size and arranged in a given manner, of a hypothetical sample of granular material that would have the same transmission constant as the actual material under consideration.

Embankment End Slope

Conical slope at end of road approach embankment.

Emergency Spillway

Structure designed to allow controlled release of storm flows in excess of the design discharge from a detention facility.

End Section

A structure, commonly made of concrete or metal, that is attached to the end of a culvert for such purposes as retaining the embankment from spilling into the waterway, improving the appearance, providing anchorage, improving the discharge coefficient and limiting some scour at the outlet compare with Inlet, Flared.

Energy Dissipation

The phenomenon whereby energy is dissipated or used up.

Energy Grade Line

A line joining the elevation of energy heads of a stream; a line drawn above the hydraulic grade line a distance equivalent to the velocity head of the flowing water at each cross section along a stream or channel reach or through a conduit. An inclined line representing the total energy of a stream flowing from a higher to a lower elevation. For open channel flow the energy grade slope is located (or plotted) a distance equal to the velocity head (V2/2g) plus the flow depth above the water surface V = velocity and g = acceleration due to gravity. Slope of the foregoing line joining the elevations of total energy through the reach of a stream or channel, or through a conduit of flowing water.

Energy Grade Slope

An inclined line representing the total energy of a stream flowing from a higher to a lower elevation. For open-channel flow the energy grade slope is located a distance of $V^2/2g$ above the water surface (V = velocity and g = acceleration due to gravity).

Energy Gradient

The slope of the energy line with reference to any plane or, more simply, the slope of the energy grade line. The slope of this line represents the rate of loss of head and it must always slope downward in the direction of flow. Equivalent to Energy Gradient. Compare with Hydraulic Gradient and Friction Slope.

Engineering fabric

Permeable textile (or filter fabric) used below riprap to prevent piping and permit natural seepage to occur.

Entrenched Stream

Stream cut into bedrock or consolidated deposits.

Envelope curves

Bounds defined approximately by the maximum observed values. The peak discharge envelope curve, which is placed on a graph of peak discharge versus drainage area, is the upper bound of observed peak discharges for any drainage area. The envelope curves are usually established for homogeneous hydrologic regions.

Environmental

Pertaining to the effects of engineering works on their surroundings and on nature.

Ephemeral Stream

A stream or reach of a stream that does not flow continuously for most of the year.

Equalizer

A culvert or opening placed where it is desirable to equalize the water head on both sides of the embankment.

Equivalent Cross Slope

An imaginary straight cross slope having a conveyance capacity equal to that of the given compound cross slope.

Erosion

Displacement of soil particles on the land surface due to such things as water or wind action. The wearing away or eroding of material on the land surface or along channel banks by flowing water or wave action on shores. Compare with Abrasion, Scour, Mass Wasting, and Sloughing.

Erosion Control Matting

Fibrous matting (e.g. jute, paper, etc.) placed or sprayed on a streambank for the purpose of preventing erosion or providing temporary stabilization until vegetation is established.

Estuary

Tidal reach at the mouth of a river.

Evapotranspiration

Plant transpiration plus evaporation from the soil. Difficult to determine separately, therefore used as a unit for study (see Consumptive Use) National Engineering Handbook. The combined loss of water from a given area by evaporation from the land and transpiration from plants Groundwater Subcommittee. The sum of evaporation plus transpiration Fetter.

Water withdrawn from a land area by evaporation from water surfaces and moist soil and plant transpiration. It is a coined word; probably the first recorded use is on page 296 of the Transactions of the American Geophysical Union, part 2,1934 Langbein and Iseri.

Exceedence Probability

The probability that the magnitude of the random variable (e.g., annual maximum flood peak) will be equalled or exceeded in any one time period, often one year.

Excess Rainfall

The water that enters the stream channels during a storm or soon after, forming a runoff hydrograph. May consist of rainfall on the stream surface, surface runoff, and seepage of infiltrated water (rapid subsurface flow).

Exfiltration

The process by which stormwater leaks or flows to the surrounding soil through openings in a conduit.

Extended Detention Dry Ponds

Depressed basins that temporarily store a portion of the stormwater runoff following a storm event. The extended detention time of the stormwater provides an opportunity for urban pollutants carried by the flow to settle out.

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F

Fabriform

Grout-filled fabric mattress used for streambank protection.

Fascine

A streambank protection technique consisting of wire mesh or timber attached to a series of posts, sometimes in double rows; the space between the rows may be filled with rock, brush, or other materials. Fences may be placed either parallel to the bank or extended into the stream; in either case these structures decrease the stream velocity and encourage sediment deposition as the flow passes through the fence.

Fetch

The effective distance the wind blows over water in generating waves. The area in which waves are generated by wind having a rather constant direction and speed; sometimes and incorrectly used synonymously with "fetch length". The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

Fetch Length

The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

Field

A character or group of characters that is a component of a record. Each field holds a single data value such as a character representing a land cover type or a group of characters that name a stream.

File

A source from which data can be obtained or a destination to which data can be sent.

Fill-Slope

Side or end slope of an earth-fill embankment. Where a fill-slope forms the streamward face of a spill-through abutment, it is regarded as part of the abutment.

Filter

Layer of synthetic fabric, sand, gravel, and/or graded rock placed (or developed naturally where suitable in-place materials exist) between the bank revetment and soil for one or more of three purposes (A) To prevent the soil from moving through the revetment by piping, extrusion, or erosion (exfiltrating); (B) To prevent the revetment from sinking into the soil; (C) To permit natural seepage from the streambank, thus preventing buildup of excessive hydrostatic pressure. Also may be a device or structure for removing solid or colloidal material from stormwater and floodwater or preventing the migration of fine-grained soil particles as water passes through soil; i.e., the water is passed through a filtering medium -- usually a granular material or finely woven or non-woven geotextile. Depending on context, may be used to remove material other than soils from a substance.

Filter Cloth, Fabric

Synthetic fabric that serves the same purpose as a granular filter blanket.

Filter Blanket

One or more layers of graded non-cohesive material placed below riprap to prevent soil piping and permit natural drainage.

Filter, Granular

A filter consisting of one or more layers of well- graded granular material.

Filter, Fabric

A filter consisting of one or more layers of permeable textile. Also referred to as geotextiles and engineering fabrics.

Filtration

The process of passing water through a filtering medium consisting of either granular material of filter cloth for the removal of suspended or colloidal matter.

Fine Sediment Load (or washload)

That part of the total sediment load that is composed of particle sizes finer than those represented in the bed. Normally the fine-sediment load is finer than 0.062 mm for a sand-bed channel. Silts, clays and sand could be considered as wash load in coarse gravel and cobble bed channels.

Flanking

Erosion resulting from streamflow between the bank and the landward end of a river-training or a grade-control structure.

Flanking Inlets

Inlets placed on either side of a low point inlet. Flanking inlets limit the spread of water onto the roadway if the low point inlet becomes clogged or is exceeded in its capacity. The purpose of these inlets are to intercept debris as the slope decreases and to act in relief of the inlet at the low point

Flap Gates

A gate which restricts water from flowing back into the discharge pipe and discourages entry into the outfall line.

Flared Inlet

A specially fabricated pipe appurtenance or a special feature of box culverts. This type of inlet is effective in reducing the calculated headwater.

Flared Wingwalls

The part of a culvert headwall which serves as a retaining wall for the highway embankment. The walls form an angle to the centerline of the culvert.

Flashy Stream

Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Most flashy streams are ephemeral but some are perennial.

Flexible Lining

A channel lining material having the capacity to adjust to settlement; typically constructed of a porous material that allows infiltration and exfiltration.

Flocculating Agent

A coagulating substance which, when added to water, forms a flocculant precipitate which will entrain suspende matter and expedite sedimentation; examples are alum, ferrous sulfate, and lime.

Flood

In common usage, an event that overflows the normal flow banks or runoff that has escaped from a channel or other surface waters see Normal Flow, and Bank. In frequency analysis it can also mean an annual flood that may not overflow the normal flow banks. In technical usage, it refers to a given discharge based, typically, on a statistical analysis of an annual series of events.

An overflow or inundation that comes from a river or other body of water and causes or threatens damage. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a channel. A relatively high flow as measured by either gage height or discharge quantity.

An overflow or other body of water that causes or threatens damage Barrows. Any relatively high streamflow overtopping the natural or artificial banks in any reach of a stream Leopold and Maddock. A relatively high flow as measured by either gage height or discharge quantity Jarvis.

See Floodwaters and Flood, Annual.

Flood Envelope Curve

An empirical relationship developed between the maximum flood discharge and drainage area for a given region.

Flood Frequency

The average time interval between occurrences of a hydrological event of a given or greater magnitude, usually expressed in years. May also be called recurrence interval. Novak.

The average time interval, in years, in which a given storm or amount of water in a stream will be exceeded. Also, referred to as exceedance interval, recurrence interval or return period. May be stated as the (A) average time interval between actual occurrences of a hydrological event of a given or greater magnitude; (B) percent chance of occurrence in any one year period, e.g., a 2% chance of flood. The chances that a specific flood magnitude (discharge) will be exceeded each year expressed as a percent; i.e., a 100-year flood has a flood probability of 1% of being exceeded each year. In the analysis of hydrologic data the flood frequency is simply called frequency and has years as a unit of measure. Note that flood frequency is not hyphenated when referring to a specific flood's frequency, but is when referring to such things as a "flood-frequency" curve.

An expression or measure of how often a hydrologic event of given size or magnitude should, on an average, be ...exceeded. For example a 50-year frequency flood should be ...exceeded in size, on the average, only once in 50 years. In drought or deficiency studies it usually defines how many years will, on the average, be ...less than a given size or magnitude Langbein and Iseri. Note, this reference incorrectly stated "equalled or exceeded", and "equal to or less than" where the three periods (...) appear (Ed.).

Flood-Frequency Curve

Langbein and Iseri offer two definitions (A) A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are equaled or exceeded; (B) A similar graph but with recurrence intervals [frequency] of floods plotted as the abscissa. A graph indicating the probability that the annual flood discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude. Compare with Frequency Curve, and Flood Frequency.

According to Dalrymple (A) A graph showing the number of times per year on the average, plotted as abscissa, that floods of magnitude, indicated by the ordinate, are equaled or exceeded; (B) A similar graph but with recurrence intervals of floods plotted as abscissa.

Note that Flood-Frequency is hyphenated when referring to a flood-frequency (flood versus frequency) curve or relationship, and not hyphenated when referring to a specific flood's frequency.

Floodplain

Any plain which borders a stream and is covered by its waters in time of flood. Topographic area adjoining a channel that is covered by flood flows as well as those areas where the path of the next flood flow is unpredictable, such as a debris cone, alluvial fan or braided channel. A nearly flat, alluvial lowland bordering a stream and commonly formed by stream processes, that is subject to inundation by floods.

Bryan provides A strip of relatively smooth land bordering a stream, built of sediment carried by the stream and dropped in the slack water beyond the influence of the swiftest current. It is called a living floodplain if it is overflowed in times of highwater; but a fossil floodplain if it is beyond the reach of the highest flood.

The lowland that borders a river, usually dry, but subject to flooding Hoyt and Langbein. That land outside of a stream channel described by the perimeter of the Maximum Probable Flood White.

Compare with Flood Plane, Flood Zone, and Backwater Area.

Flood Pool

Floodwater storage elevation in a reservoir. In a floodwater retarding reservoir, the temporary storage between the crests of the principal and emergency spillways.

Flood Of Record

Reference to the maximum estimated or measured discharge that has occurred at a site.

Flood Routing

The process of determining progressively the timing and shape of a flood wave at successive points along a river Carter and Godfrey.

Determining the changes in a flood wave as it moves downstream through a valley or through a reservoir (then sometimes called reservoir routing). Graphic or numerical methods are used National Engineering Handbook.

Floodwater Retarding Structure

A dam, usually with an earth fill, having a flood pool where incoming floodwater is temporarily stored and slowly released downstream through a principal spillway. The reservoir contains a sediment pool and sometimes storage for irrigation or other purposes.

Flow Concentration

A preponderance of the streamflow.

Flow-Control Structure

A structure, either within or outside a channel, that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

Flow, Critical

Flow conditions at which the discharge is a maximum for a given specific energy, or at which the specific Flow, Nonuniform energy is minimum for a given discharge.

Flow Distribution

The estimated or measured spatial distribution of the total streamflow from the landward edge of one floodplain or stream bank to the landward edge of the other floodplain or stream bank. Usually shown as a percent of accumulated flow from one edge (0%) to the other edge (100%). Same as the cumulative conveyance only in terms of discharge rather than conveyance compare with Cumulative Conveyance.

Flow-Duration Chart

A graph indicating the percentage of time during which a given discharge is exceeded.

Flow, Gradually Varied

Flow in which the velocity or depth changes gradually along the length of the channel.

Flow Hazard

Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.

Flow Line

The bottom elevation of an open channel or closed conduit.

Flow, Nonuniform

Flow in which the velocity vector is not constant along every streamline.

Flow Rapidly Varied

Flow in which the velocity or depth change rapidly along the length of the channel.

Flow Slide

Saturation of a bank to the point where the soil material behaves more like a liquid than a solid; the soil/water mixture may then move downslope, resulting in a bank failure.

Flow, Steady

Flow in which the velocity is constant in magnitude or direction with respect to time.

Flow, Subcritical

Flow conditions below critical; usually defined as flow conditions having a Froude Number less than 1.

Flow, Supercritical

Flow conditions above critical; usually defined as flow conditions having a Froude Number greater than 1.

Flow, Uniform

Flow in which the velocity vector is constant along every streamline.

Flow, Unsteady

Flow in which velocity changes in magnitude and direction with respect to time.

Flow, Varied

Flow in which velocity or depth change along the length of the channel.

Flume

An open or closed channel used to convey water. An open conduit of such things as wood, concrete, or metal on a prepared grade, trestle, or bridge. A flume holds water as a complete structure. A concrete lined canal would still be a canal without the lining, but the lining supported independently would be a flume. A large flume is also termed an aqueduct. Compare with Bench-Flume.

Ford

A location where a highway crosses a channel by allowing high annual or larger flows to pass over the highway and lower flows to pass through a culvert(s). Often used with cutoff walls, roadway lane markers, and paved roadway embankments and traveled way (and shoulders). Warning signs may be included, also.

Free Outlet

Those outlets whose tailwater is equal to or lower than critical depth. For culverts having free outlets, lowering of the tailwater has no effect on the discharge or the backwater profile upstream of the tailwater.

Freeboard

The vertical distance between the level of the water surface, usually corresponding to design flow and a point of interest such as a low chord of a bridge beam or specific location on the roadway grade.

French Drain

An underground passagewat for water through interstices among stomed placed loosely in a trench.

Frequency

1. Also referred to as exceedance interval, recurrence interval or return period; the average time interval between actual occurences of a hydrological event of a given or greater magnitude; the reciprocal of the percent chanfce of occurrence in any one year period.
2. In analysis of hydrologic data, the recurrence interval is simply called frequency.

Fresh Water Ridge

(Ground Water Mound) - A mound or ridge-shaped feature of a water table or piezometric surfac, usually produced by downward percolation of water to water-bearing deposits. Also called groundwater hill.

Frontal Flow

The portion of flow which passes over the upstream side of a grate.

Froude number

The ratio of inertia forces to gravity forces, usually expressed as the ratio of the flow velocity to the square root of the product of gravity and a linear dimension, i.e., $V/(gL)^{0.5}$. The Froude (rhymes with food) number is used in the study of fluid motion.

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I

Icing

Masses or sheets of ice formed on the frozen surface of a river or floodplain. When shoals in the river are frozen to the bottom or otherwise dammed, water under hydrostatic pressure is forced to the surface where it freezes.

Impermeable Strata

A strata in which texture is such that water cannot move perceptibly through it under pressures ordinarily found in subsurface water.

Impervious

Impermeable to the movement of water.

Improved Inlet

Flared, depressed or tapered culvert inlets which decrease the amount of energy needed to pass the flow through the inlet and thus increase the capacity of culverts.

Incipient Motion

The condition that exists just prior to the movement of a particle within a flow field. Under this condition, any increase in any of the factors responsible for particle movement will cause motion.

Incised Reach

The stretch of river with a incised channel that only rarely overflows its banks.

Incised Stream

A stream that flows in an incised channel with high banks. Banks that stand more than 15 ft above the water surface at normal stage are regarded as high.

Index-Flood Method

A peak discharge estimation method that quantifies a peak discharge for a specific exceedence probability by the product of a peak discharge estimated with a regression equation for the index flood and an index ratio.

Infiltration

1. The flow of a fluid into a substance through pores or small openings. It connotes flow into a substance in contradistinction to the word percolation, which connotes flow through a porous substance Horton, 1942.

The downward entry of water into the soil or rock Groundwater Subcommittee.

Rainfall minus interception, evaporation, and surface runoff. The part of rainfall that enters the soil National Engineering Handbook.

That part of rainfall that enters the soil. The passage of water through the soil surface into the ground.

Compare with Percolation.

2. The process of water entering the upper layers of the soil profile.

Infiltration Basins

An excavated area which impounds stormwater flow and gradually exfiltrates it through the basin floor.

Infiltration Capacity

The maximum rate at which the soil, when in a given condition, can absorb falling rain or melting snow.

Infiltration Drainage

Disposal of storm water by infiltration into the soil.

Infiltration Pond

A small natural or man-made surface reservoir for collection and infiltration of storm water.

Infiltration Rate

The rate at which water enters the soil under a given condition. The rate is usually expressed in inches per hour, feet per day, or cubic feet per second.

Infiltration System

The storm drain system with features designed for the purpose of infiltrating storm water into the surrounding soil.

Infiltration Trenches

Shallow excavations which have been backfilled with a coarse stone media. The trench forms an underground reservoir which collects runoff and exfiltrates it to the subsoil.

Infiltration Well

See Wells.

Inflow

The rate of discharge arriving at a point (in a stream, structure, or reservoir).

Initial Abstraction (Ia)

1. When considering surface runoff, Ia is all the rainfall before runoff begins. When considering direct runoff, Ia consists of interception, evaporation, and the soil-water storage that must be exhausted before direct runoff may begin. Sometimes called "initial loss."

2. The portion of the rainfall that occurs prior to the start of direct runoff.

Injection Well

A deep vertical well used to dispose of liquid wastes under pressure.

Inlet

Consider four definitions (A) A surface connection to a closed drain; (B) A structure at the diversion end of a conduit; (C) The upstream end of any structure through which water may flow; (D) An inlet structure for capturing concentrated surface flow. Inlets may be located in such places as along the roadway, a gutter, the highway median, or a field.

Inlet Chamber

A typically cast-iron, welded steel, or formed concrete compartment that is beneath an inlet. It is usualy set into athe bridge deck, but is sometimes only an open hole in the deck.

Inlet Efficiency

The ratio of flow intercepted by an inlet to the total flow.

Inlet Time

The time required for stormwater to flow from the most distant point in a drainage area to the point at which it enters a storm drain.

Instantaneous Discharge

A discharge at a given moment.

Instantaneous Unit Hydrograph

The hydrologic response of the watershed to 1-cm of rainfall excess concentrated in an infinitesimally small period of time.

Intensity

1. The rate of rainfall typically given in units of millimeters per hour (inches per hour).
2. Volume Per Unit Time

Intensity-Duration-Frequency Curve

1.A graph or mathematical equation that relates the rainfall intensity, storm duration, and exceedence frequency.

2.IDF curves provide a summary of a site's rainfall characteristics by relating storm duration and exceedence probability (frequency) to rainfall intensity (assumed constant over the duration).

Interception

The process and the amount of rain or snow stored on leaves and branches and eventually evaporated back to the air. Interception equals the precipitation on the vegetation minus stemflow and throughfall Hoover. See Stemflow and Throughfall.

Precipitation retained on plant or plant residue surfaces and finally absorbed, evaporated, or sublimated. That which flows down the plant to the ground is called stemflow and not counted as true interception National Engineering Handbook

Invert

The flow line in a channel cross section, pipe, or culvert. The lowest point in the channel cross section or at flow control devices such as weirs or dams. The floor, bottom, or lowest part of the internal cross section of a conduit. Compare with Soffit.

Inverted Syphon

A structure used to convey water under a road using pressure flow. The hydraulic grade line is above the crown of the structure.

Isohyet

A line on a map of equal rainfall depth for the same duration, usually the duration of a storm.

Island

A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Some islands originate by establishment of vegetation on a bar, and other originate by channel avulsion or at the junction of minor tributaries with a stream.

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J

Jack

A device for flow control and protection of banks against lateral erosion; it has six mutually perpendicular arms rigidly fixed at the center. Steel jacks are strung with wire; Kellner jacks are made of three steel struts; concrete jacks are made of three reinforced concrete beams bolted together at the midpoints.

Jack Field

Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.

Jetty

An elongated obstruction projecting into a stream to control shoaling and scour by deflection of currents and waves. They may be permeable or impermeable.

Junction Boxes

Formed control structures used to join sections of storm drains.

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K

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L

Lag Time, TL

The difference in time between the centroid of the excess rainfall (that rainfall producing runoff) and the peak of the runoff hydrograph. Often estimated as 60 percent of the time of concentration (TL = 0.6Tc)

Land Cover/Land Use

Most conventional definitions have land cover relating to the type of feature on the surface of the earth such as roof-top, asphalt surface, grass and trees. Land use associates the cover with a socio-economic activity such as factory or school, parking lot or highway, golf course or pasture and orchard or forest. In hydrologic modeling, the terms land cover and land use are often used interchangeably because the inputs to the models require elements from each definition.

Land Use

A term which relates to both the physical characteristics of the land surface and the human activities associated with the land surface Alley. A highway facility to accommodate land uses is termed a land use structure or facility. See Land Use Facility.

A land classification. Cover, such as row crops or pasture, indicates a kind of land use. Roads may also be classified as a separate land use National Engineering Handbook.

Compare with Development, and Land Treatment Measure.

Land Treatment

Application of wastewater to land surface that uses plants and soil to remove contaminates from wastewater.

Lateral Drainage

Movements of water though soil in the horizontal direction.

Lateral Erosion

Erosion in which the removal of material has a dominantly lateral component, as contrasted with scour in which the component is dominantly vertical.

Launching

Release of undercut material (stone riprap, rubble, slag, etc.) downslide; if sufficient material accumulates on the streambank face, the slope can become effectively armored.

Least Squares Regression

A procedure for fitting a mathematical function such that the sum of the squares of the differences between the predicted and measured values are minimized.

Levee

An embankment, generally landward of a top bank, that confines flow during high water periods, thus preventing overflow into lowlands. A linear embankment outside a channel for containment of flow. Longer than a dike. Compare with Dike.

Level of Significance

A statistical concept that equals the probability of making a specific error, namely of rejecting the null hypothesis when, in fact, it is true. The level of significance is used in statistical decision making.

Lining, Composite

Combination of lining materials in a given cross section (e.g., riprap in low-flow channel and vegetated upper banks).

Lining, Flexible

Lining material with the capacity to adjust to settlement typically constructed of a porous material that allows infiltration and exfiltration.

Lining, Permanent

Lining designed for long term use.

Lining, Rigid

Lining Material with no capacity to adjust to settlement constructed of nonporous material with smooth finish that provides a large conveyance capacity (e.g., concrete, soil cement).

Lining, Temporary

Lining designed for short term utilization, typically to assist in development of a permanent vegetative lining.

Littoral Drift

The transport of material along a shoreline (also 'long-shore sediment transport').

Littoral Transport

The movement of sediments in the near shore zone by waves and currents. The movement can be parallel to the shore (long shore transport) or perpendicular to the shore (onshore-offshore transport).

Load (or Sediment Load)

Amount of sediment being moved by a stream.

Local Scour

Scour in a channel or on a flood plain that is localized at a pier, abutment or other obstruction to flow. The scour is caused by the acceleration of the flow and the development of a vortex system induced by the obstruction to the flow.

Longitudinal Profile

The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

Longitudinal Slope

The rate of change of elevation with respect to distance in the direction of travel or flow.

Lower Bank

That portion of a streambank having an elevation less than the mean water level of the stream.

Glossary

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M

Major System

This system provides overland relief for stormwater flows exceeding the capacity of the minor system and is composed of pathways that are provided, knowingly or unknowingly, for the runoff to flow to natural or manmade receiving channels such as streams, creeks, or rivers.

Manhole

A structure by which one may access a drainage system.

Manning's "n"

A coefficient of roughness, used in a Manning's Equation for estimating the capacity of a channel to convey water. Generally, "n" values are determined by inspection of the channel National Engineering Handbook. The roughness coefficient, n, in the Manning equation for determination of a discharge. Compare with Hydraulic Roughness. See Manning's Equation.

Mass Inflow Curve

A graph showing the total cumulative volume of stormwater runoff plotted against time for a given drainage area.

Mass Rainfall Curve

The cumulative precipitation plotted over time.

Mathematical Model

A symbolic representation of a flow situation using mathematical equations.

Mattress

A covering of concrete, wood, stone or other material used to protect a streambank against erosion.

Maximum Likelihood Estimation

A mathematical method of obtaining the parameters of a probability distribution by optimizing a likelihood function that yields the most likely parameters based on the sample information.

Maximum Probable Flood

The maximum probable flood is the greatest flood that may reasonably be expected, taking into collective account the most adverse flood related conditions based on geographic location, meteorology, and terrain.

Mean Daily Discharge

The average of mean discharge of a stream for one day. Usually given in cfs.

Meander

One curved portion of a sinuous or winding stream channel, consisting of two consecutive loops, one turning clockwise, and the other counterclockwise.

Meander Belt

The distance between lines drawn tangent to the extreme limits of successive fully developed meanders.

Meander Loop

An individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.

Meander Ratio

The ratio of meander width to meander length.

Meander Scrolls

Low, concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.

Meander Width

The amplitude of swing of a fully developed meander measured from midstream to midstream.

Meandering Channel

A channel exhibiting a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.

Meandering Stream

A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops.

Mean Velocity

In hydraulics, the discharge divided by the cross sectional area of the flowing water.

Median Diameter

The midpoint in the size distribution of sediment such that half the weight of the material is composed of particles larger than the median diameter and half is composed of particles smaller than the median diameter.

Method-of-Moments Estimation

A method of fitting the parameters of a probability distribution by equating them to the sample moments.

Mid-Channel Bar

A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.

Middle Bank

That portion of a streambank having an elevation approximately the same as that of the mean water level of the stream.

Migration

Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.

Migration (of Bed Forms or Meanders)

Systematic shifting in the direction of flow.

Migration, Channel

Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.

Minor System

This system consists of the components of the storm drainage system that are normally designed to carry runoff from the more frequent storm events. These components include curbs, gutters, ditches, inlets, manholes, pipes and other conduits, open channels, pumps, detention basins, water quality control facilities, etc.

Mixed Flow Pumps

Mixed flow pumps are very similar to axial flow except they create head by a combination of lift and centrifugal action. An obvious physical difference is the presence of the impeller "bowl" just above the pump inlet.

Mounding

The condition that exists when the water table rises to the elevation of the bottom of the infiltration system. When this occurs, percolation rates are controlled by the groundwater gradient laterally away from the system rather than vertical infiltration rates.

Moving-Average Smoothing

A statistical method of smoothing a time or space series in which the nonsystematic variation is eliminated by averaging adjacent measurements. The smoothed series represents the systematic variation.

Mud

A soft, saturated mixture mainly of silt and clay.

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N

Natural Levee

A low ridge along a stream channel, formed by deposition during floods, that slopes gently away from the channel.

Natural Scour

Scour which occurs along a channel reach due to an unstable stream, no exterior causes.

Nominal Sediment

Equivalent spherical diameter of a hypothetical sphere of the same volume as a given stone.

Nonalluvial Channel

A channel whose boundary is completely in bedrock.

Nonhomogeneity

A characteristic of time or space series that indicates the moments are not constant throughout the length of the series.

Nonparametric Statistics

A class of statistical tests that do not require assumptions about the population distribution.

Normal Depth

The depth of a uniform channel flow.

Normal Stage

The average water stage prevailing during the greater part of the year. The water surface elevation corresponding to the Normal Flow. See Normal Flow and Depth, Normal.

Glossary

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O

One-Dimensional Profile

An estimated water surface profile which accommodates flow only in the up-Water Surfacestream-downstream direction.

Open Channel

A natural or manmade structure that conveys water with the top surface in contact with the atmosphere.

Open Channel Flow

Flow in an open conduit or channel that is driven by gravitational forces.

Order-Theory Statistics

A class of statistical methods in which the analysis is based primarily on the order relations among the sample values.

Ordinary High Water

A term for defining a regulatory related water surface for a natural channel or the shore of standing waters. This intersection reflects the highest level water reaches in an average runoff year as indicated by such things as erosion, shelving, change in the character of soil, destruction of terrestrial vegetation or its inability to grow, the presence of litter and debris; or in the absence of such evidence, an arbitrarily estimated water surface might be used such as that associated with the mean annual flood. For the purposes of this glossary, in no instance will the Ordinary High Water (OHW) be considered as exceeding the estimated water surface level of the mean annual flood unless so mandated by the cognizant regulatory agency(ies). The sum of the water right, flood right and mean annual flood may be used to arbitrarily determine the maximum OHW for irrigation channels intercepting runoff.

Organic Compound

Amount of organic material such as discrete particles of wood, leaf matter, spores, etc., present on the surface or between layers of clay particles.

Orifice Equation

An equation based on Bernoulli's equation that relates the discharge through an orifice to the area of the orifice and the depth of water above the center of the orifice.

Orifice Flow

Flow of water into an opening that is submerged. The flow is controlled by pressure forces.

Outfall

The point location or structure where drainage discharges from a channel, conduit or drain.

Outlet Pipe

The pipe that leads that water away from an inlet chamber or drop inlet.

Outlier

An extreme event in a data sample that has been proven using statistical methods to be from a population different from the remainder of the data.

Overbank Flow

Water movement over top bank either due to a rising stream stage or to inland surface water runoff.

Overland Flow

Runoff which makes its way to the watershed outlet without concentrating in gullies and streams (often in the form of sheet flow).

Oxbow

The abandoned bow-shaped or horseshoe-shaped reach of a former meander loop that is left when the stream cuts a new shorter channel across the narrow neck between closely approaching bends of the meander.

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P

Parametic Statistics

A class of statistical tests in which their derivation involved explicit assumptions about the underlying population.

Partial-Duration Frequency Analysis

A frequency method that uses all floods of record above a threshold to derive a probability function to represent the data.

Pathogenic Bacteria

Bacteria which may cause disease in the host organisms by their parasitic growth.

Pavement

Streambank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on streambanks are concrete, compacted asphalt, and soil-cement.

Paving

Covering of stones on a channel bed or bank (used in the Manual with reference to natural covering).

Peak Discharge

The highest value of the stage or discharge attained by a flood; thus, peak stage or peak discharge. Flood crest has nearly the same meaning, but since it connotes the top of the flood wave, it is properly used only in referring to stage thus, crest stage, but not crest discharge Langbein and Iseri.

Maximum discharge rate on a runoff hydrograph for a given flood event. The instantaneous, maximum discharge of a particular flood at a given point along a stream.

In a frequency study of annual floods, it is the maximum instantaneous discharge rate reached during the year.

Peak Runoff

See Peak Discharge

Peaked Stone Dike

Riprap placed parallel to the toe of a streambank (at the natural angle of repose of the stone) to prevent erosion of the toe and induce sediment deposition behind the dike.

Pearson Correlation Coefficient

An index of association between paired values of two random variables. The value assumes a linear model.

Perched Groundwater Table

Groundwater that is separated from the main body of groundwater.

Percolation

The flow of a fluid through a substance via pores or small openings. Two definitions are offered by the Groundwater Subcommittee (A) The downward movement of water through the unsaturated zone; (B) The downward flow of water in saturated or nearly saturated porous medium at hydraulic gradients of the order of 1.0 or less.

Movement of water through the interstices of a substance, as through soils. The movement or flow of water through the interstices or the pores of a soil or other porous medium.

The movement, under hydrostatic pressure, of water through the interstices of a rock or soil, except the movement through large openings such as caves Meinzer, 1923. Compare with Infiltration.

Perennial Stream

A stream or reach of a stream that flows continuously for all or most of the year.

Perimeter of a Grate

The sum of the lengths of all sides of the grate, except that any side adjacent to a curb is not considered a part of the perimeter in weir flow computations.

Permeability

The property of a material or substance which describes the degree to which the material is penetrable by liquids or gases. Also, the measure of this property.

Permissible Shear Stress

Defines the force required to initiate movement of the channel bed or lining material.

Permissible Velocity

The velocity which will not cause serious erosion of the channel lining material.

Pervious Soil

Soil containing voids through which water will move under hydrostatic pressure.

pH

The reciprocal of the logarithm of the hydrogen ion concentration. The concentration is the weight of hydrogen ions, in grams per liter of solution. Neutral water, for example, has a pH value of 7 and a hydrogen ion concentration of 10^{-7}.

Phreatic Line

The upper boundary of the seepage water surface landward of a streambank.

Physically-based Hydrologic Models

That family of models that estimate runoff by simulating the behavior and watershed linkages of individual processes such as infiltration, depression and detention storage, overland and channel flows, etc.

Pier Shaft

The main part of a pier above the footing or foundation.

Pile

An elongated member, usually made of timber, concrete, or steel, that serves as a structural component of a river-training structure.

Pile Bin (or Pile Pier)

A pier composed of piles capped or decked with a timber grillage or with a reinforced-concrete slab forming the bridge foundation.

Pile Dike

A type of permeable structure for the protection of banks against caving; consists of a cluster of piles driven into the stream, braced and lashed together.

Piping

Removal of soil material through subsurface flow of seepage water that develops channels or 'pipes' within the soil bank.

Pixel

An array of picture elements on a color screen of a personal computer.

Plotting Position Formula

An equation used in frequency analysis to compute the probability of an event based on the rank of the event and the sample size.

Point Bar

An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop usually somewhat downstream from the apex of the loop.

Point Rainfall

Rainfall at a single rain gage.

Poised Stream (stable stream)

A stream which, as a whole, maintains its slopes, depths, and channel dimensions without any noticeable raising or lowering of its bed. Such condition may be temporary from a geological point of view, but for practical engineering purposes, the stream may be considered stable.

Pollutants

Harmful of objectionable contaminants in water.

Positive System

A storm drain system which pipes discharge directly into a stream river canal, pond, or lake.

Power Loss Methodology

A method used to determine the energy lost at an access hole or junction box during a storm drainage design procedure.

Power Model

A mathematical function that relates the criterion (dependent) variable, y, to the predictor (independent) variable,x, raised to an exponent, i.e., $y = ax^b$.

Precipitation

The process by which water in liquid or solid state falls from the atmosphere. The total measurable supply of water received directly from clouds, as rain, snow, and hail; usually expressed as depth in a day, month, or year, and designated as daily, monthly, or annual precipitation. Not synonymous with Rainfall compare with Rainfall.

As used in hydrology, precipitation is the discharge of water, in liquid or solid state, out of the atmosphere, generally upon a land or water surface. It is the common process by which atmospheric water becomes surface or subsurface water. The term "precipitation" is also commonly used to designate the quantity of water that is precipitated Meinzer, 1923. Precipitation includes rainfall, snow, hail, and sleet, and is therefore a more general term than rainfall Langbein and Iseri.

Precision

A measure of the nonsystematic variation. It is the ability of an estimator to give repeated estimates that are close together.

Pressure Flow

Flow in a conduit that has no surface exposed to the atmosphere. The flow is driven by pressure forces.

Pressure Head

The head represented by the expression of pressure over weight (p/γ); where p is pressure, and γ is weight. When p is in pounds per square foot and γ is the weight of the liquid per cubic foot, h becomes head in feet.

Pressure Ridges

Ridges on an ice-sheet over a body of water caused by expansion and consequent upheaval of the ice.

Principal Spillway

Conveys all ordinary discharges coming into a reservoir and all of an extreme discharge that does not pass through the emergency spillway.

Probability Paper

A graph paper in which the ordinate is the value of a random variable and the abscissa is the probability of the value of the random variable being equaled or exceeded. The nature of the probability scale depends on the probability distribution.

Project Flood

A flood discharge value adopted for the design of projects such as dams and flood control works.

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Q

Quarry-Run Stone

Natural material used for streambank protection as received from a quarry without regard to gradation requirements.

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R

Radial Flow Pump

Pumps that utilize centrifugal force to move water up the riser pipe. They will handle any range of head and discharge, but are the best choice for high head applications. Radial flow pumps generally handle debris quite well.

Radial Flows

Flow both inward and outward in all directions from a given location, such as a well.

Railbank Protection

A type of countermeasure composed of rock-filled wire fabric and supported by steel rails or posts driven into the streambed.

Rainfall Excess

The portion of rainfall that causes direct flood runoff. It equals the total rainfall minus the initial abstraction and losses.

Rainfall Intensity

Amount of rainfall occurring in a unit of time, converted to its equivalent in inches per hour at the same rate.

Random Access

Access to stored data in which the data can be referred to in any order whatever, instead of just in the order in which they are stored.

Rapid Drawdown

Lowering the water against a bank more quickly than the bank can drain which can leave the bank in an unstable condition.

Raster Database

A method for displaying and storing geographic data as a rectangular array of characters where each character represents the dominant feature, such as a land cover or soil type, in a grid cell at the corresponding location on a map.

Rating Curve

A graph of the discharge of a river at a particular point as a function of the elevation of the water surface Hydrology Subcommittee. A graphic (or tabular) representation of rating; a calibration; a curve (table) relating stage to discharge. Compare with Stage-Discharge Curve.

Reach

A segment of stream or valley, selected with arbitrary bounds for purposes of study. A comparatively short length of a stream or channel.

Langbein and Iseri offer five definitions (A) The length of channel uniform with respect to discharge, depth, area, and slope; (B) The length of a channel for which a single gage affords a satisfactory measure of the stage and discharge; (C) The length of a river between two gaging stations; (D) More generally, any length of a river; (E) A length of stream or valley, selected for convenience in a study. See Damage Reach, [and] Stream Reach.

Real-time Modeling

Hydrologic modeling in which a calibrated model is used with data for a storm event in progress to make predictions of streamflow for the remainder of the storm event.

Recession Curve

[That portion of] a hydrograph showing the decreasing rate of runoff following a period of rain or snowmelt. Since direct runoff and base runoff recede at different rates, separate curves, called direct runoff recession curves and base runoff recession curves, respectively are generally drawn. The term "depletion curve" in the sense of base runoff recession is not recommended Langbein and Iseri.

The receding portion of a hydrograph, occurring after excess rainfall has stopped National Engineering Handbook.

Recharge

Addition of water to the zone of saturation from precipitation or infiltration Groundwater Subcommittee. The process of adding water to the saturated zone; also the water added. For man-made recharge facilities see Basin, Recharge.

Recharge Basin

A basin excavated in the earth to receive the discharge from streams or storm drains for the purpose of replenishing groundwater supply.

Recharge Well

See Wells.

Record

A string of characters or groups of characters (fields) that are treated as a single unit in a file.

Recurrence Interval (R.I.); Return Period; Exceedance Interval

The reciprocal of the annual probability of exceedance of a hydrologic event.

Refusal

Erosion-resistant material placed in a trench (excavated landward) at the upstream end of a revetment to prevent flanking.

Reinforced-Earth Bulkhead

A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a natural or artificial streambank (a specific type of retaining wall).

Reinforced Revetment

A streambank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank.

Regime

General pattern of variation around a mean condition, as in flow regime, tidal regime, channel regime, sediment regime, etc.; used also to mean a set of physical characteristics of a river.

Regime Change

A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads or slope.

Regime Channel

Alluvial channel that has attained more or less a state of equilibrium with respect to erosion and deposition.

Regime Formula

A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.

Regional Analysis

Flood-frequency [relationships] lines for gaged watersheds in a similar [homogeneous physiographic] area or region are used to develop a flood-frequency line for an ungaged watershed in that [same] region. Also used with other types of hydrologic data. Method is a simple (usually graphical and freehand) form of "regression analysis" used by statisticians National Engineering Handbook.

A regional study, statistically based of gaged stream data from a homogeneous physiographic region which produces regression equations relating various watershed and climatological parameters to such things as discharge frequency for application on ungaged streams. Used to formulate methods of predicting flood-frequency relationships for the hydraulic design of drainage facilities in hydrologically similar ungaged watersheds having characteristics similar to those used in the regression analysis.

Regulatory Flood

Means the 100-year flood, which was adopted by the Federal Emergency Management Agency (FEMA), as the base flood for flood plain management purposes.

Regulatory Floodway

The floodplain area that is reserved in an open manner by Federal, State, or local requirements, i.e., unconfined or unobstructed either horizontally or vertically, to provide for the discharge of the base flood so that the cumulative increase in water surface elevation is no more than a designated amount.

Reinforced-Earth Bulkhead

A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a streambank.

Relief Bridge

An opening in an embankment on a floodplain to permit passage of overbank flow.

Representative Channel Cross-Section

A cross-section that is selected for use in a model because the flow characteristics through that section are considered to be typical or representative of the flow conditions along a given length of a river or stream.

Retaining Wall

A structure used to maintain an elevation differential between the water surface and top bank while at the same time preventing bank erosion and instability.

Retention System

A facility designed for the purpose of storing storm water.

Reservoir Routing

Flood routing through a reservoir National Engineering Handbook. Flood routing of a hydrograph through a reservoir taking into account reservoir storage, spillway and outlet works discharge relationships.

Retard

A channel bank protection technique consisting of such things as wire mesh, chain-link, steel rails, or timber framed fence attached to a series of posts, sometimes in double rows; the space between the rows may be filled with rock, brush, or other suitable permeable materials. Fences may be placed either parallel to the bank and/or extended into the channel; in either case these structures decrease the stream velocity and encourage sediment deposition as the flow passes through the fence. Explained anther way, a frame structure, filled with earth or stone ballast, designed to absorb energy and to keep erosive channel flows away from a bank. A retard is designed to decrease velocity and induce sediment deposition or accretion. Retard type structures are permeable structures customarily constructed at, and parallel to the toe of a highway fillslope and/or channel banks. A permeable or impermeable linear structure in a channel, parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.

Retardance Classification

Qualitative description of the resistance to flow offered by various types of vegetation.

Retention Basin

A basin or reservoir wherein water is stored for regulating a flood. It does not have an uncontrolled outlet. The stored water is disposed by a means such as infiltration, injection (or dry) wells, or by release to the downstream drainage system after the storm event. The release may be through a gate-controlled gravity system or by pumping.

Retention/Detention Facilities

Facilities used to control the quantity, quality, and rate of runoff discharged to receiving waters. Detention facilities control the rate of outflow from the watershed and typically produce a lower peak runoff rate than would occur without the facility. Retention facilities capture all of the runoff from the watershed and use infiltration and evaporation to release the water from the facility.

Return Period

A concept used to define the average length of time between occurrences in which the value of the random variable is equaled or exceeded.

Revetment

1. A rigid or flexible armor placed on a bank or embankment as protection against scour and lateral erosion.

2. A channel bank lining designed to prevent or halt bank erosion.

Revetment Toe

The lower terminus of a revetment blanket; the base or foundation of a revetment.

Riffle

A natural shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.

Rigid Lining

A lining material with no capacity to adjust to settlement; these lining materials are usually constructed of non-porous material.

Riparian

Pertaining to anything connected with or adjacent to the banks of a stream.

Riprap

A well graded mass of durable stone, or other material that is specifically designed to provide protection from flow induced erosion.

Risk

The probability that an event of a given magnitude will be equaled or exceeded within a specific period of time.

River Training

Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel. Also, any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.

River Training Structure

Any configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.

Roadway Channel

Stabilized drainage-way used to collect water from roadway and adjacent areas and to deliver it to an inlet or main drainage-way

Roadway Cross-Slopes

Transverse slopes and/or superelevation described by the roadway section Slopes geometry. Usually provided to facilitate drainage and/or resist centrifugal force.

Rock-and-Wire Mattress

A flat or cylindrical wire cage or basket filled with stone, or other suitable material; and placed on a streambank with a filter used as protection against erosion.

Rock Well

See Wells.

Rock Windrow

An erosion control technique that consists of burying or piling a sufficient supply of erosion-resistant material below or on the existing land surface along the bank, then permitting the area between the natural riverbank and the rock to erode until the erosion reaches and undercuts the supply of rock.

Roughness

The estimated measure of texture at the perimeters of channels and conduits. Usually represented by the "n-value" coefficient used in Manning's channel flow equation.

Roughness Coefficient

Numerical measure of the frictional resistance to flow in a channel, as in the Manning or Strickler formulas.

Routing

The process of transposing an inflow hydrograph through a structure and determining the outflow hydrograph from the structure.

Rubble

Broken fragments of rock or debris resulting from the decay or destruction of a building.

Runoff

Surface Water, Stream Water, and Floodwater as defined in the AASHTO legal guide. That part of the precipitation which runs off the surface of a drainage area after accounting for all abstractions. The portion of precipitation that appears as flow in streams; total volume of flow of a stream during a specified time.

That part of the precipitation that appears in surface streams. It is the same as streamflow unaffected by artificial diversions, storage, or other works of man in or on the stream channels Langbein and Iseri.

The total amount of water flowing in a stream. It includes Overland Flow, Return Flow, Interflow, and Base Flow Fetter.

Runoff Coefficient

A factor representing the portion of runoff resulting from a unit rainfall. Dependent on terrain and topography.

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S

Sack Revetment

Streambank protection consisting of sacks (e.g. burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material placed on a bank to serve as protection against erosion.

Salt-Water Intrusion

The invasion of a body of fresh water by a body of salt-water.

Saltation Load

Sediment bounced along the stream bed by energy and turbulence of flow and by other moving particles.

Sand

Granular material that is smaller than 2.0mm and coarser than 0.062mm.

Sand Filters

Filters that provide stormwater treatment when runoff is strained through a sand bed before being returned to a stream or channel.

Saturated Flow

See Saturated Permeability.

Saturated Mound

Upward movement of water in soil near a point of recharge.

Saturated Permeability

Movement of water through soil under hydrostatic pressure with all void spaces of soil filled with water.

Saturated Soil

Soil that has its interstices or void spaces filled with water to the point at which runoff occurs.

Scanner

A device that measures the light passing through or the reflectance of light from a map or other document to convert the data into a computer compatible raster format file. Subsequent operations can then translate the raster data into vector formats, land cover files, etc.

SCS County Soil Map

A book prepared by the Soil Conservation Service of the USDA that describes and discusses the soil related environment and presents maps showing the distribution of soil characteristics for a county.

Scour

The displacement and removal of channel bed material due to flowing water; usually considered as being localized as opposed to general bed degradation or headcutting. The result of the erosive action of running water which excavates and carries away material from a channel bed. Compare with Erosion, Abrasion, Lateral Erosion, Mass Wasting, and Sloughing.

Scoured Depth

Total depth of the water from water surface to a scoured bed level (compare 'depth of scour').

Scupper

1. A vertical hole through a bridge deck for the purpose of deck drainage. Sometimes a horizontal opening in the curb or barrier is called a scupper.

2. A small opening (usually vertical) in the deck, curb, or barrier through which water can flow.

Sediment (or fluvial sediment)

Fragmental material transported, suspended, or deposited by water.

Sediment Concentration (by weight or by volume)

Weight or volume of sediment relative to quantity of transporting or suspending fluid or fluid-sediment mixture.

Sediment Discharge

The quantity of sediment, by weight or volume, that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

Sediment Load

Amount of sediment being moved by a stream.

Sediment Pool

Reservoir storage provided for sediment, thus prolonging the usefulness of floodwater or irrigation pools.

Sediment Yield

The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

Sedimentation

The process involving the deposition of soil particles which have been carried by flood waters.

Sedimentation Basin

A basin or tank in which stormwater containing settleable solids is retained to remove by gravity or filtration a part of the suspended matter.

Seepage

The slow movement of water through small cracks and pores of the bank material.

Seepage Pit

A small pit extending into porous strata and lined with open-jointed stone, concrete block, precast concrete, or similar walls, capped, and provided with an access cover. It serves to introduce into the ground, by seepage, partly-treated wastewater effluent.

Seiche

Long-period oscillation of a lake or similar body of water.

Separator

A device placed between native soil and aggregate backfill in infiltration systems to prevent migration of fine soil particles during periods of high groundwater. Also see Filter, Filtration.

Set-Up

Raising of water level due to wind action.

Shallow Concentrated Flow

Flow that has concentrated in rills or small gullies.

Shallow Water (for waves)

Water of such a depth that waves are noticeably affected by bottom conditions; customarily, water shallower than half the wavelength.

Shear Stress

The force developed on the wetted area of the channel that acts in the direction of the flow, usually measured as a force per unit wetted area.

Sheet Flow

A shallow mass of runoff on a planar surface or land area in the upper reaches of a drainage area.

Shoal

A submerged sand bank. A shoal results from natural deposition on a streambed which has resisted all erosion; thus, the water is of necessity compelled to pass over it.

Significant Wave

A statistical term denoting waves with the average height and period of the one-third highest wave of a given wave group.

Sheet Flow

Shallow flow on the watershed surface that occurs prior to the flow concentrating into rills.

S-hydrograph

The cumulative hydrograph that results from adding an infinite number of T-hour unit hydrographs, each lagged T-hours.

Side-flow Interception

Flow which is intercepted along the side of a grate inlet, as opposed to frontal interception.

Side Slope

Slope of the sides of a channel; usually referred to by giving the horizontal distance followed by the vertical distance. For example, 1.5 to 1, or 1.5 1.0, meaning a horizontal distance of 1.5 feet (.46 m) to a 1 foot (.3 m) vertical distance.

Sieve Diameter

The size of sieve opening through which the given particle will just pass.

Sill

(a) A structure built under water, across the deep pools of a stream with the aim of changing the depth of the stream; (b) A low structure built across an effluent stream, diversion channel or outlet to reduce flow or prevent flow until the main stream stage reaches the crest of the structure.

Silt

Material finer than 0.62mm and coarser than 0.004mm that is nonplastic or very slightly plastic and exhibits little or no strength when air-dried (Unified Soil Classification System).

Sinuosity

The ratio between the thalweg length and the valley length of a sinuous stream.

Skew

1. A measure of the angle of intersection between a line normal to the roadway centerline and

the direction of the streamflow at flood stage on the lineal direction of the main channel.
2. The third statistical moment, with the mean and variance being the first and second statistical moments. The skew is a measure of the symmetry of either data or a population distribution, with a value of zero indicating a symmetric distribution.

Skewness

When data are plotted in a curve on log-normal paper, the curvature is skewness.

Slope (of channel or river)

Fall per unit length along the channel centerline.

Slope-area method

A method of estimating discharge rates using basic equations of hydraulics, such as Manning's equation and the continuity equation.

Slope Protection

Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.

Slotted Inlets

A section of pipe cut along the longitudinal axis with transverse bars spaced to form slots.

Slotted Drain Inlets

Drainage inlet composed of a continuous slot built into the top of a pipe which serves to intercept, collect and transport the flow.

Sloughing

Shallow transverse movement of a soil mass down a streambank as the result of an instability condition at or near the surface (also called slumping). Conditions leading to sloughing are: bed degradation, attack at the bank toe, rapid drawdown, and slope erosion to an angle greater than the angle of repose of the material.

Slump

A sudden slip or collapse of a bank, generally in the vertical direction and confined to a short distance, probably due to the substratum being washed out or having become unable to bear the weight above it.

Soffit

The inside top of the culvert or storm drain pipe.

Soil-Cement

A designed mixture of soil and portland cement compacted at a proper water content to form a veneer or structure that can prevent streambank erosion.

Soil Permeability

The property of soil that permits water to pass through it when it is saturated and movement is actuated by hydrostatic pressure of the magnitude normally encountered in natural subsurface water. Also see Permeability, Percolation.

Soil Piping

The process by which soil particles are washed in or through pore spaces in filters.

Soil Porosity

The percentage of the soil (or rock) volume that is not occupied by solid particles, including all pore space filled with air and water.

Soil-Water-Storage

The amount of water the soils (including geologic formations) of a watershed will store at a given time. Amounts vary from watershed to watershed. The amount for a given watershed is continually varying as rainfall or evapotranspiration takes place.

Solubility

The degree with which a particular substance will react under a given condition and go into solution.

Sorting

Progressive reduction of size (or weight) of particles of the load carried down a stream.

Spatial Concentration

The dry weight of sediment per unit volume of water-sediment mixture in place or the ratio of dry weight of sediment or total weight of water-sediment mixture in a sample or unit volume of the mixture.

Spearman Correlation Coefficient

An index of association between paired values of two random variables. It is computed using the ranks of the data rather than the sample values. It is the nonparametric alternative to the Pearson correlation coefficient.

Specific Energy

The energy head relative to the channel bottom. The total energy head measured above the channel bed. The sum of the velocity head and the depth of flow.

Specific Surface

The particle area contained in a unit volume of soil solids. The particle surface area includes only the external particle surface (the internal porosity of individual particles is neglected). Used as an indirect method for determining soil permeability.

Spill-through Abutment

A bridge abutment having a fill slope on the streamward side. The term originally referred to the 'spill-through' of fill at an open abutment but is now applied to any abutment having such a slope.

Splash-Over

That portion of frontal flow at a grate which splashes over the grate and is not intercepted.

Spread

A measure of the transverse lateral distance from the curb face to the limit of the water flowing on the roadway.

Spread Footing

A pier or abutment footing that transfers load directly to the earth.

Spur

A structure, permeable or impermeable, projecting into a channel from the bank for the purpose of altering flow direction, inducing deposition, or reducing flow velocity along the bank.

Spur Dike

A dike placed at an angle to the roadway for the purpose of shifting the erosion characteristics of stream flow away from a drainage structure. Often used at bridge abutments.

Stable Channel

A condition that exists when a stream has an appropriate bed slope and cross-section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or deposited or streambank erosion.

Stage

Height of water surface above a specified datum. Water surface elevation of a channel with respect to a reference elevation. The elevation of a water surface above its minimum; also above or below an established "low-water" plane; hence above or below any datum of reference; gage height.

The height of a water surface above an established datum plane (See also gage height) Langbein and Iseri. The depth of water in a river or stream above the gage datum, or 0.0 level Hydrology Subcommittee.

Stage-Discharge

Sometimes referred to as the Rating Curve of a stream cross-section. A correlation between stream flow rates and corresponding water surface elevations.

Stage-Storage-Discharge Relationships

A relationship between stage, storage, and discharge used in storage routing methods. It is usually computed from the stage-storage and stage-discharge relationships.

Standard Error

A measure of the sampling variation of a statistic.

Standard Error of Estimate

The standard deviation of the residuals in a regression analysis. It is based on the number of degrees of freedom associated with the errors.

Standing Waves

Curved symmetrically shaped waves on the water surface and on the channel bottom that are virtually stationary.

Steady Flow

Flow that remains constant with respect to time.

Steady-State Seepage

Steady seepage or flow occurring when there is equilibrium between the discharge and the source.

Stilling Basin

A device or structure placed at, or near the outlet of a structure for the purpose of inducing energy dissipation where flow velocities are expected to cause unacceptable channel bed scour and bank erosion.

Stochastic Methods

Frequency analysis used to evaluate peak flows where adequate gaged stream flow data exist. Frequency distributions are used in the analysis of hydrologic data and include the normal distribution, the log-normal distribution, the Gumbel extreme value distribution, and the log-Pearson Type III distribution.

Stone Riprap

Natural cobbles, boulders, or rock dumped or placed on a streambank or filter as protection against erosion.

Stop-Logs

Devices used for temporary closure of an opening in a hydraulic structure.

Storage Basin

A basin excavated in the earth for detention of retention of water for future flow.

Storage-Indication Method

A flood-routing method, also often called the modified Puls method.

Storm Drain

A particular storm drainage system component that receives runoff from inlets and conveys the runoff to some point. Storm drains are closed conduits or open channels connecting two or more inlets.

Storm Drainage Systems

Systems which collect, convey, and discharge stormwater flowing within and along the highway right-of-way.

Storm Duration

The period or length of storm.

Storm Frequency

The recurrence of two floods equaling or exceeding a specific discharge.

Storm Intensity

See Rainfall Intensity.

Storm Surge

Oceanic tidelike phenomenon resulting from wind and barometric pressure changes.

Stream

A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.

Stream Contraction/Constriction

A narrowing of the natural stream waterway. Usually in reference to a drainage facility installed in the roadway embankment.

Stream Reach

A length of stream channel selected for use in hydraulic or other computations.

Streambank Failure

Sudden collapse of a bank due to an instability condition such as removal of the bank by scour.

Streambank Protection

Any technique used to prevent erosion or failure of a streambank.

Streambank Erosion

Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to streambank erosion.

Sub-Bed Material

Material underlying that portion of the stream bed which is subject to direct action of the flow.

Subcritical Flow

Flow characterized by low velocities, large depths, mild slopes, and a Froude number less than 1.0.

Subcritical, Supercritical Flow

Open channel flow conditions with Froude Number less than and greater than unity, respectively.

Submeander

A small meander contained within the banks of a perennial stream channel. These are caused by relatively low discharges after the flood has subsided.

Submerged Inlets

Inlets of culverts having a headwater greater than about 1.2 D.

Submerged Outlets

Submerged outlets are those culvert outlets having a tailwater elevation greater than the soffit of the culvert.

Supercritical Flow

Flow characterized by high velocities, shallow depths, steep slopes, and a Froude number greater than 1.0.

Superelevation

Local increases in water surface on the outside of a bend.

Superflood

Flood used to evaluate the effects of a rare flow event; a flow exceeding the 100-year flood. It is recommended that the superflood be on the order of the 500-year event or a flood 1.7 times the magnitude of the 100-year flood if the magnitude of the 500-year flood is not known.

Surface Runoff

Total rainfall minus interception, evaporation, infiltration, and surface storage, and which moves across the ground surface to a stream or depression.

Surface Storage

Natural or man-made roughness of a land surface, which stores some or all of the surface runoff of a storm. [such things as] Natural depressions, contour furrows, and terraces are usually considered as producing surface storage, but stock ponds, reservoirs, stream channel storage, etc. are generally excluded National Engineering Handbook.

Stormwater that is contained in surface depressions or basins. Compare with Storage, Basin.

Surface Water

Water appearing on the surface in a diffused state, with no permanent source of supply or regular course for a considerable time; as distinguished from water appearing in water courses, lakes, or ponds.

Suspended-Sediment Discharge

The quantity of suspended sediment by weight or volume, passing through a stream cross section above the bed layer in a unit of time.

Swale

A wide, shallow ditch usually grassed or paved and without well defined bed and banks. A slight depression in the ground surface where water collects, and which may be transported as a stream. Often vegetated and shaped so as to not provide a visual signature of a bank or shore.

Synthesis

The term means "To put together" and is applied to the problem of hydrologic estimation using a known model (see analysis).

Synthetic

A graph developed for an ungaged drainage area, based on known physical Hydrograph characteristics of the watershed basin.

Synthetic Hydrograph

A hydrograph determined from empirical rules. Usually based on the physical characteristics of the basin.

Synthetic Mattress, Matting or Tubing

A grout, or sand-filled, manufactured, semiflexible casing placed on a streambank to prevent erosion.

Synthetic Unit Hydrograph

A unit hydrograph not directly based on measured rainfall and runoff data.

Synthetic Rainfall Events

Artificially developed rainfall distribution events.

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T

Tailwater, TW

Tailwater is the depth of flow in the channel directly downstream of a drainage facility. Often calculated for the discharge flowing in the natural stream without the highway effect (but may include other local effects from development), unless there is a significant amount of temporary storage that will be (or is) caused by the highway facility; in which case, a flood routing analysis may be required. The tailwater is usually used in such things as culvert and storm drain design and is the depth measured from the downstream flow line of the culvert or storm drain to the water surface. May also be the depth of flow in a channel directly downstream of a drainage facility as influenced by the backwater curve from an existing downstream drainage facility. With such things as releases from a dam, the water just downstream from a structure.

Tetrahedron

Component of river-training works made of six steel or concrete struts fabricated in the shape of a pyramid.

Tetrapod

Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5 degrees with the other three.

Thalweg

Line following the deepest part of a streambed or channel.

Tidal Amplitude

Generally, half of tidal range.

Tidal Cycle

One complete rise and fall of the tide.

Tidal Inlet

A body of water with an opening to the sea, but otherwise enclosed.

Tidal Passage

A tidal channel connecting with the sea at both ends.

Tidal Period

Duration of one complete tidal cycle.

Tidal Prism

Volume of water contained in a tidal inlet or estuary, between low and high tide levels.

Tidal Range

Vertical difference between specified low and high tide levels.

Tides, Astronomical

Variations in sea level due to motion of heavenly bodies.

Tieback

Structure placed between revetment and bank to prevent flanking.

TIGER/Line Files

Topologically Integrated Geographically Encoding and Referencing (TIGER) system available on CD-ROM from the U.S. Bureau of Census. The files store vector segments that when connected form line features such as streets and streams. The files also provide the names of the individual streets and streams and the street addresses between intersections.

Timber or Brush Mattress

A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream and weighted with ballast.

Time-Area Curve

The relationship between runoff travel time and the portion of the watershed that contributes runoff during that travel time.

Time of Concentration

The time for runoff to travel from the hydraulically most distant point in the watershed to a point of interest within the watershed. This time is calculated by summing the individual travel times for consecutive components of the drainage system.

Toe

That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.

Toe Protection

Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.

Toe-Fill

Break in slope between the bank and the overbank area.

Total Dynamic Head

The combination of static head, velocity head, and various head losses in the discharge system caused by friction, bends, obstructions, etc.

Total Sediment Discharge

The sum of suspended-sediment discharge and bedload discharge or the sum of bed material discharge and washload discharge of a stream.

Total Sediment Load (or total load)

The sum of suspended load and bedload or the sum of bed material load and washload of a stream.

Tractive Force

Force developed at the channel bed as a result of the resistance to flow created by the channel section. This force acts in the direction of flow, and is equal to the shear stress on the channel section multiplied by the wetted perimeter.

Transmission Zone

A moisture zone during infiltration of water which is characterized by an essentially constant moisure content. Nearly the entire depth of the profile of the wetted soil will be in this zone.

Trash Rack

A device used to capture debris, either floating, suspended, or rolling along the bed, before it enters a drainage facility.

Travel Lane

Portion of the traveled way for the movement of a single lane of vehicles, normally 12 feet.

Travel Time

The average time for water to flow through a reach or other stream or valley length that is less than the total [stream or valley] length. A travel time is part of a TC [Time of Concentration] but never the whole TC National Engineering Handbook.

The average time for water to flow through a reach or other stream or valley length.

Not synonymous with Time of Concentration compare with Time of Concentration.

Trench-Fill Revetment

Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding streambank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion.

Tributaries

Branches of the watershed stream system.

Tsunamis

Waves created by earthquakes or other tectonic disturbance on the ocean bottom.

Turbulence

Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.

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U

Uncontrolled Crossing

A bridge crossing that imposes no constraints on the natural width of the stream or on its ability to shift its channel.

Uncontrolled Spillway

A facility at a reservoir at which flood water discharge is governed only by the inflow and resulting head in the reservoir. Usually the emergency spillway is uncontrolled.

Ungaged Stream Sites

Locations at which no systematic records are available regarding actual stream-flows.

Uniform Flow

The flow condition where the rate of head loss due to friction is equal to bed slope of the channel.

Unit Discharge

Discharge per unit width (may be average over a cross section, or local at a point).

Unit Hydrograph

The direct runoff hydrograph produced by a storm of given duration such that the volume of excess rainfall and direct runoff is 1 cm.

Unit Peak dPDischarge

The peak discharge per unit area, with units of $m^3/sec/km^2$.

Unit Shear Force (shear stress)

The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually expressed in units of stress, force per unit area.

Unsaturated Flow

Flow of water though unsaturated or dry soil, that is predominantly controlled by capillary condution.

Unsteady Flow

Flow that changes with respect to time.

Upper Bank

The portion of a streambank having an elevation greater than the average water level of the stream.

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V

Varied Flow

Flow in an open channel where the flow rate and depth change along the length of the channel.

Vector Database

A method for displaying and storing geographic data as a distribution of vector segments that, when connected, form polygons that enclose homogeneous areas such as a defined land cover or form lines representing features such as roads or streams.

Vegetation

Woody or nonwoody plants used to stabilize a streambank and retard erosion.

Velocity

A measure of the speed or a moving substance or particle given in feet per second (m/s).

Velocity, Cross-Sectional Average

Discharge divided by cross-sectional area of flow.

Velocity, Local Average

Local discharge intensity divided by depth of flow.

Velocity-Weighted Sediment Concentration

The dry weight of sediment discharged through a cross section during unit time.

Vertical (full-height) Abutment

An abutment, usually with wingwalls, that has no fill slope on its streamward side.

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W

Wandering Channel

A channel exhibiting a more or less non-systematic process of channel shifting, erosion, and deposition, with no definite meanders or braided pattern.

Wandering Thalweg

A thalweg whose position in the channel shifts during floods and typically serves as an inset channel that transmits all or most of the streamflow at normal or lower stages.

Wash Load

Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.

Water Table

The upper surface of a zone of saturation. No water table exists where that surface is formed by an impermeable body Meinzer; as well as Langbein and Iseri.

The upper surface of a zone of saturation except where that surface is formed by a confining unit. The upper surface of the zone of saturation on which the water pressure in the porous medium equals atmospheric pressure. Means that surface in a groundwater body at which the water pressure is atmospheric. Upper surface of a zone of saturation, where the body of groundwater is not confined by an overlying impermeable zone.

The surface in an unconfined aquifer or confining bed at which the pore water pressure is atmospheric. It can be measured by installing shallow wells extending a few feet into the zone of saturation and then measuring the water level in those wells Fetter.

The upper surface of groundwater National Engineering Handbook.

The upper surface of a zone of saturation in soil or in permeable strata or beds. The upper surface of the zone of saturation, except where that surface is formed by an impermeable body compare with Groundwater, Perched.

Watercourse

A channel in which a flow of water occurs, either continuously or intermittently, with some degree of regularity.

Water Quality Inlets

Pre-cast storm drain inlets (oil and grit separators) that remove sediment, oil and grease, and large particulates from paved area runoff before it reaches storm drainage systems or infiltration BMPs.

Watershed

The catchment area for rainfall which is delineated as the drainage area producing runoff. Usually it is assumed that base flow in a stream also comes from the same area.

Waterway Opening Width (area)

Width (area) of bridge opening at (below) a specified stage, measured normal to principal direction of flow.

Water Year

October 1 to September 30, with the water year number taken as the calendar year of the January 1 to September 30 period.

Wave Attack

Impact of waves on a streambank.

Wave Downrun

The down slope flow of water experienced immediately following a wave runup as the water flows back to the normal water elevation.

Wave Period

Time period between arrivals of successive wave crests at a point.

Wave Runup

The movement of water up a channel bank as a result of the breaking of a wave at the bank line; The extent and magnitude of the wave runup is a function of the energy in the wave.

Weephole

A hole in an impermeable wall or revetment to relieve the neutral stress or porewater pressure.

Weighted Skew

An estimate of the skew based on both the station skew and a regionalized value of skew.

Weir Flow

1. Flow over a horizontal obstruction controlled by gravity. 2. Free surface flow over a control surface which has a defined discharge vs. depth relationship.

Well Screen

A special form of slotted or perorated well casing that admits water from an aquifer consisting of unconsolidated granular material while preventing the granular material from entering the well.

Wells

Shallow to deep vertical excavations, generally with perforated or slotted pipe backfilled with selected aggregate. The bottom of the excavation terminates in pervious strata above the water table.

Wet-Pit Stations

Pump stations designed so that the pumps are submerged in a wet well or sump with the motors and the controls located overhead.

Wet Ponds

A pond designed to store a permanent pool during dry weather.

Wetted Perimeter

The boundary over which water flows in a channel, stream, river, swale, or drainage facility such as a culvert or storm drain. The boundary is taken normal to the flow direction of the discharge in question. The length of the wetted contact between a stream of water and its containing conduit, measured along a plane at right angles to the flow in question; that part of the periphery of the cross section area of a stream in contact with its container. See Hydraulic Radius.

Wet Well Sump

The feature in a pump station in which runoff waters are temporarily stored.

Windrow Revetment

A row of stone (called a windrow) placed on top of the bank landward of an eroding streambank. As erosion continues the windrow is eventually undercut, launching the stone downslope, thus armoring the bank face.

Wind Set-Down

Corresponding fall in level at the windward side.

Wind Set-Up

Rise in level at the leeward side of a body of water, due to wind stresses on the surface.

Wire Mesh

Wire woven to form a mesh, the openings of which are of suitable size and shape to enclose rock or broken concrete, or to be used on fence-like spurs and retards.

Wire-Enclosed Riprap

Consists of wire baskets filled with stone, connected together and anchored to the channel bottom or sides.

Work Station

A combination of hardware and software normally used by one person to interact with a computer system and perform computer supported tasks.

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X

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Symbols

A, B, C, D, E, F, G, H, I, J, K, L, M, N, , Q, R, S, T, U, V, W, X, Y, Z, MISC.

To jump to a specific part of the alphabet, click on the above HotLinks!
Click the Back button to return to the top of this page.
(If the letter you are looking for does not appear in the HotLink list below, then there are no glossary entries for that letter!)

A

A = surface area

A_1 = cross-sectional area for the fluid entering the control volume

A_2 = cross-sectional area for the fluid leaving the control volume

A_{n2} = normal cross-sectional area at the bridge crossing

A_p = projected area of a pier normal to the flow

A_s = area of a sediment particle

a = acceleration or pier width or embankment length

a = thickness of the bed layer, the unmeasured zone of flow

a_n = embankment length for relief bridge

a_n = acceleration component normal to a streamline

a_m = embankment length for main bridge

a_o = amplitude of wave

$\hat{a}$ = coefficient of width relationship

B

B, B' = dimensionless constants in Meyer-Peter, Muller equations

B_x = correction term in Einstein's Method

b_s = bridge opening length

$\hat{b}$ = exponent of width relationship

C

C = sediment concentration

C = Chezy coefficient

C_o = Chezy coefficient for steady uniform flow

C_1 = constant for free vortex flow

C_2 = constant for forced vortex flow

C_D = drag coefficient on a particle

C_f = concentration of fine material

C_r = overtopping discharge coefficient

C_T = bed material sediment concentration

C_s = coefficient of shear

$\bar{C}$ = average suspended sediment concentration

c = wave celerity

$\hat{c}$ = coefficient of the flow depth relationship

D

D = culvert diameter or sediment size

D_{50} = sediment riprap size for which 50% by weight of the particles are smaller; similarily D_{65}, D_{84}, D_{90} represent sizes for which 65, 84, and 90% of the particles are smaller

D_{50_x} = mean sediment size a distance x downstream from reference location

D_{50_o} = mean sediment size at the reference section

D_m = effective mean diameter of sediment or riprap

D_s = sediment size or riprap size

dA_1 = differential area for the fluid entering the control volume

dA_2 = differential area for the fluid leaving the control volume

d_{rr} = thickness of riprap layer

ds = displacement of a fluid particle during time dt

d_s = scour depth

dt = differential time interval

E

E = energy of a system

E = ratio of bed layer thickness to flow depth a/y_o

e = energy per unit mass

e_1, e_2,

e_3,e_4 = moment arms of forces acting on a rock particle

e_c = eccentricity coefficient

F

F_1, F_2 = pressure force acting at section one and two

$F_1(\eta)$, $F_2(\eta)$ = functions for the transverse velocity distribution in bends

F_B = buoyant force on a particle

F_b = force exerted at the boundary

F_c = critical Froude number for the beginning of motion of sediment

F_D = drag force on a particle

F_d = drag force on a rock particle

F_f = fluid force on a particle

F_g = gravitational force on a particle

F_ℓ = lift force on a rock particle

F_n = external forces between particles

Fr = Froude number

F_s = shear force

F_v = shear force on a particle

F_x = force acting in the x direction

f = Darcy-Weisbach friction factor

f'_b = Darcy-Weisbach friction factor for the grain roughness

f_s = seepage force

$\hat{f}$ = exponent of the flow depth relationship

G

G = gradation coefficient

g = acceleration due to gravity

H

H = total energy

H_e = entrance loss of head

H_e = average head loss over a cross-section

H_f = friction loss of head

H_{min} = minimum total energy at a critical section

H_T = total head

H_v = velocity head

HW_2 = headwater at a culvert

H_s = horizontal side slope related to one unit vertically

h_L = head loss in a hydraulic jump

h_{e*} = head loss

h_1 = total backwater elevation

Δh = drop in water surface elevation through bridge opening

I

I_1, I_2 = integrals in the Einstein method

i_b = fraction of the bed load for given equation

i_s = fraction of the suspended load for a given equation

i_T = fraction of total load for a given equation

J

J = ratio of projected area of piers to the gross constricted area A_{n2}

$\hat{j}$ = exponent of the bed sediment discharge relationship

K

K = coefficient of the Meyer-Peter and Muller equation

K_1, K_2 = coefficient for the area and volume of sediment particles

K_B, K_r = coefficient in Meyer-Peter, Muller equation

K_b = base backwater coefficient

K_e = entrance loss coefficient for culverts

ΔK = correction coefficient for piers

ΔK_e = incremental correction coefficient for excentricity

ΔK_p = incremental correction coefficient for piers

ΔK_s = incremental correction coefficient for skewed flow

k_s = height of roughness elements

$\hat{k}$ = coefficient of the velocity relationship

k* = total backwater coefficient for subcritical flow

L

L = length of control volume

L = length of culvert

L_1, L_2, L_3 = lengths for sloping sill drop structure

L_a = length of abutment

L_c = length of a channel along the thalweg

L_e = length of embankment

L_j = length of hydraulic jump

L_s = upstream length of a guidebank

L_s = length of the roadway crest for overspill

ℓ = pier length

ℓ = mixing length

ℓ_a = longest axis of particle

ℓ_b = intermediate axis of particle

ℓ_c = shortest axis of particle

M

M = bridge opening ratio

m = exponent of the velocity relationship

M/N = ratio of lift to drag moments on a particle

N

n = Manning's resistance coefficient

n_b = Manning's resistance coefficient of the bed

n_o = Manning's resistance coefficient for steady uniform flow

n_w = Manning's resistance coefficient of the walls

P

P = wetted perimeter of the flow

p_1, p_2 = pressure of fluid at sections one and two

P_b = wetted perimeter of the bed

P_E = coefficient in the Einstein's total sediment discharge method

P_i, P_o = pressure of fluid inside and outside the bend

P_w = wetted perimeter of the banks

p_i = percentage by weight of a size fraction of sediment

$\hat{p}$ = coefficient of the total bed sediment discharge relationship

$\bar{P}$ = average pressure over a cross section

Q

Q = discharge (flow rate)

$\dot{Q}$ = rate at which heat is added to the system

Q_B = bed load discharge in weight units

Q_b = water discharge quantity determining the bed load transport

Q_c = approach channel flow discharge

Q_e = approach flow discharge on the embankment

Q_f = formative discharge for a particular phenomena

Q_n = uncorrected bed sediment discharge

Q_o = overtopping discharge

Q_s = total suspended sediment discharge

Q_{smax} = maximum suspended sediment discharge

Q_T = total bed sediment discharge

Q_T = total discharge

q = unit discharge

q_b = unit bed load discharge

q_n = uncorrected unit bed sediment discharge

q_s = unit suspended sediment discharge

q_T = unit total bed sediment discharge

R

R = hydraulic radius

R'_b = hydraulic radius due to grain roughness

Re = Reynolds number

Re_p = Reynolds number of falling particle

$\bar{R}$ = vector R of the direction of particle movement

r = radius of curvature

r_c = radius of curvature at the center of the stream

r_i, r_o = radius of curvature for inner and outer banks in a bend

$\hat{r}$ = coefficient of Manning's n relationship

S

S = channel slope

S_c = shape factor of a cross-section

S_f = friction slope (also called the energy slope)

S'_f = friction slope to overcome grain resistance

S_m = ratio of tangents of friction angle to side slope angle

S_n = channel sinuosity

S_o = bed slope

S_p = shape factor of sediment particle

S_R = shape factor of a river reach

S_s = specific gravity of sediment particle

S_x = bed slope at a distance x downstream of a reference location

S_w = slope of water surface

s = direction tangental to streamline

T

T = wave period

T = time

T^o = temperature

$\hat{t}$ = coefficient of the friction slope relationship

U

u = internal energy associated with fluid temperature

V

V = velocity vector

$\forall$ = volume

V_c = critical velocity

V_d = depth-average velocity at the vena contracta

V_m = mean stream velocity

V_{max} = maximum velocity

V_{n2} = average velocity at the bridge opening

V_o = tangential velocity in a bend

V_p = volume of a particle, or average velocity on a spillslope

V_r = reference velocity

V_s = velocity against the stone

V_T = depth-average velocity at the toe of an embankment

V_w = velocity of the wave

V_* = shear velocity

V_{*c} = shear velocity in the channel

V'_* = shear velocity due to grain roughness

V''_* = shear velocity due to form roughness

v_1, V_1 = velocity of the fluid entering a control volume

v_2, V_2 = velocity of the fluid leaving a control volume

v_s, v_n = velocity components in the s and n directions

v_s, v_y = velocity components along x and y

v_x, v_y= average velocity components along x and y

v'_x, v'_y=velocity fluctuations

W

W = width of control volume or channel free surface width

$\dot{W}$ = rate at which a fluid system does work on its surroundings

$\dot{W}_p$ = rate of pressure work done by system

W_s = Buoyant weight of sediment particle

$\dot{W}_\tau$ = rate of stress work

ΔWS = drop in water surface elevation at a drop structure

w = lateral location in a cross-section

X

X = Einstein's multiplication factor

$\overline{X}$ = function of Δ/δ' in Einstein's method

Y

Y = pressure correction factor, a function of D_{65}/δ' in Einstein's method

y = vertical distance

y_c = critical depth of flow

y_{max} = maximum depth of flow

y_n = normal depth of flow

y_o = total flow depth

y_s = scour depth due to contraction

y' = vertical distance above bed at which the velocity is zero

$\hat{y}$ = exponent of the Manning's n relationship

Z

Z = exponent of the Rouse equation

ΔZ = superelevation of flow in bends

Δz = drop in water surface elevation along a channel

z = elevation above arbitrary reference level

z_i, z_o = water surface elevation at inside and outside of bend

$\hat{z}$ = exponent of the friction slope relationship

MISC.

Greek Symbols

α = energy correction factor

α' = energy correction factor for unit width

$\hat{\alpha}$ = coefficient of the downstream decrease in slope

β = momentum correction factor

β = angle between the particle movement direction and the vertical

β_1 = angle of the hydraulic jump in bends

β' = momentum correction factor for a unit width

$\hat{\beta}$ = coefficient of the downstream decrease in sediment size

γ = specific weight of the fluid

γ_s = specific weight of the sediment

Δ = apparent roughness of the bed

Δ_o = dimensionless bend angle

δ = angle between the drag force and the particle movement direction

δ' = thickness of the laminar sublayer in turbulent flow

ε = exponent in Laursen's equation

η = stability number for particles on a plane bed

η_1 = stability number for particles on side slopes

θ = side slope angle

θ = inclination angle of a channel

θ_1 = bend angle in supercritical flows

θ_e = angle of orientation of the embankment

κ = von Karman velocity constant

λ = wave length

λ = angle between the horizontal and the drag force vector

μ = dynamic viscosity of a fluid

υ = kinematic viscosity of a fluid

ξ = correction factor, a function of $D_s/\bar{\chi}$ in Einstein's method

ξ = function of other parameters (Equation 5.3.31)

ρ = fluid density

ρ_1 = density of a fluid entering a control volume

ρ_2 = density of a fluid leaving a control volume

ρ_s = density of sediment particles

σ = correction coefficient for piers

τ = shear stress

τ_o = bed shear stress

τ_c = critical shear stress

ϕ_* = dimensionless sediment transport function

ψ = uncorrected entrainment function

ψ_* = entrainment function

ω = fall velocity of a sediment particle

ω_a = angle of the abutment