HIGHWAYS IN THE RIVER ENVIRONMENT
HYDRAULIC AND ENVIRONMENTAL DESIGN CONSIDERATIONS

Training and Design Manual

Chapters VI , VII and VIII

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Chapter VI

RIVER STABILIZATION, BANK PROTECTION AND SCOUR

6.1.0 INTRODUCTION

From a study of the chapters on river mechanics and geomorphology it should be clear that both short- 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 VIII.

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 extensive treatise on the subject of bank and shore protection was prepared by the California Division of Highways (1970)*. 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, it is the purpose 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.

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 panacea to all problems associated with highway encroachments on rivers. Handbook analyses and designs usually lead to poor solutions of specific problems. Also, solution to a particular problem may generate problems elsewhere in the river system.

6.2.0 CHANNEL IMPROVEMENT

There are some circumstances when 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 Fig. 6.2.1a. Assume that the alignment is fixed by constraints in the acquisition of the right-of-way.

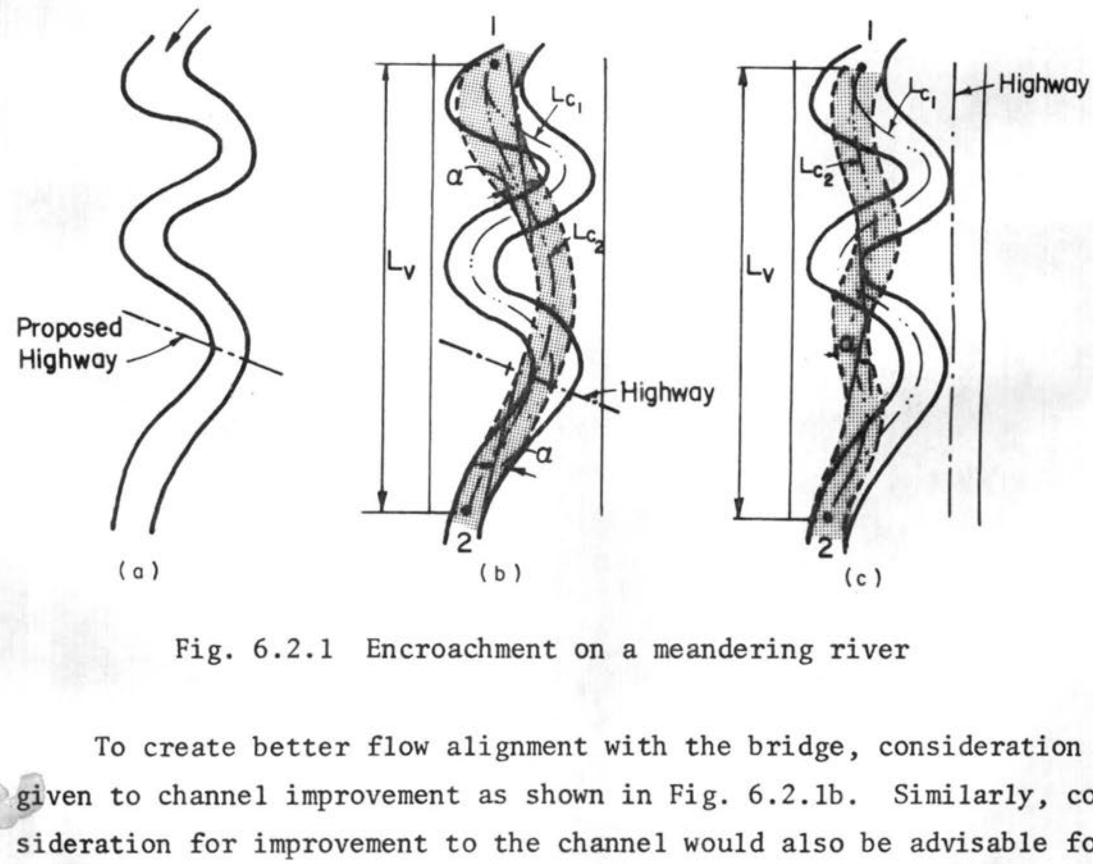


Fig. 6.2.1 Encroachment on a meandering river

To create better flow alignment with the bridge, consideration is given to channel improvement as shown in Fig. 6.2.1b. Similarly, consideration for improvement to the channel would also be advisable for a hypothetical lateral encroachment of a highway as depicted in Fig. 6.2.1c. In either case, the designer's questions are how to realign the channel, and what criteria to use to establish the cross-sectional dimensions.

In realigning a river channel, the 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 improvement. It is difficult to state generalized criteria for channel improvement 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. The angle α shown in Fig. 6.2.1 between a line drawn tangent to the inside of two successive bends and the bank line in the crossing should be approximately 20 degrees. This enables a sufficient crossing length for the thalweg to swing 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 Fig. 6.2.1 are established by existing conditions. Hence, the total drop in bed elevation for the new channels (subscript 2) and the old channels (subscript 1) are the same.

$$\Delta z_1 = \Delta z_2 = \Delta z \qquad 6.2.1$$

The length of channel measured along the thalweg is labeled L_c. Thus the mean slope of the channel bed before rechannelization is

$$S_1 = \frac{\Delta z}{L_{c_1}} \qquad 6.2.2$$

and after rechannelization is

$$S_2 = \frac{\Delta z}{L_{c_2}} \qquad 6.2.3$$

Sinuosity is defined by the ratio of the length of channel to length of the valley, or

$$P = \frac{L_c}{L_v} \geq 1 \qquad 6.2.4$$

Clearly,

$$P_1 = \frac{L_{c_1}}{L_{v_1}} \qquad 6.2.5$$

$$P_2 = \frac{L_{c_2}}{L_{v_2}} \qquad 6.2.6$$

but

$$L_{v_1} = L_{v_2} = L_v \qquad 6.2.7$$

and

$$\frac{\Delta z_1}{L_{v_1}} = \frac{\Delta z_2}{L_{v_2}} = \frac{\Delta z}{L_v} \qquad 6.2.8$$

Thus,

$$P_1 S_1 = \frac{L_{c_1}}{L_v} \cdot \frac{\Delta z}{L_{c_1}} = \frac{L_{c_2}}{L_v} \cdot \frac{\Delta z}{L_{c_2}} = P_2 S_2 \qquad 6.2.9$$

The new channel slope and channel sinuosity are inversely related. If $P_2 < P_1$ then $S_2 > S_1$. The new channel alignment, hence P_2, 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 Eq. 6.2.9, and the relationship (from Eq. 4.4.4)

$$S_2 Q^{1/4} \leq 0.0017 \qquad 6.2.10$$

should be satisfied. If S_1 is of such magnitude that Eq. 6.2.10 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 increase in sediment transport which could cause degradation and the effect would be extended both upstream and downstream of the rechannelized 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 plan 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 (Eq. 4.4.3), $QS \sim Q_s D_{50}$ tends toward equilibrium by slight increases in bed material 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 in the old and new channels. That is,

$$(\tau_o V)_1 = (\tau_o V)_2. \qquad 6.2.11$$

With substitution of $\tau_o = \gamma RS$, $V = Q/A$ and $R = A/P \simeq A/W$, Eq. 6.2.11 leads to

$$W_2 = \frac{P_1}{P_2} W_1 \qquad 6.2.12$$

The 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.

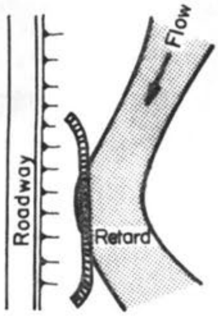
The depth of flow in the channel is dependent on discharge, effective channel width, sediment transport rate (because it affects bed form and channel roughness) and channel slope. Methods for determining flow depth are discussed in Chapter III.

The foregoing discussion pertains to alluvial channels with fine-sized bed materials (sands and silts). For streams with gravel and cobble beds, the 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.

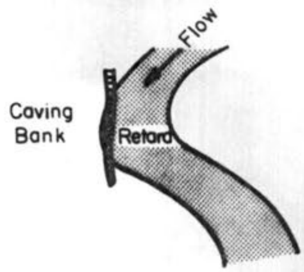
6.3.0 RIVER TRAINING AND STABILIZATION

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 with hydraulic model studies. *Rock riprap is probably the most widely used material* to stabilize river banks and protect the side slopes of embankments. Because of its wide use and importance in highway practice a separate section (Section 6.4.0) is devoted to rock riprap. Other materials useful in highway practice are discussed in this section.

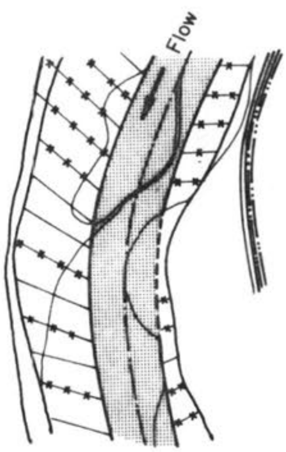
Dikes, retards, and jetties are devices used to guide the river flow and to protect the banks. Their use is illustrated in Fig. 6.3.1. In all instances, the intent of all the devices shown in Fig. 6.3.1 is to cause resistance or obstruction to flow along the channel bank, thereby creating lower velocities to stop bank erosion, contain the *thalweg of* the stream to the center portions of the channel, and if possible cause deposition of sediment along the bank where erosion previously occurred.



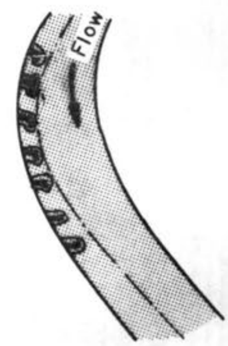
(a) Retards to protect highway embankment



(b) Retard to prevent further bank caving



(c) Jetties to train the flow and protect the bank



(d) Dikes to train the flow and protect the bank

Fig. 6.3.1 Retards, jetties and dikes to protect embankments and train channel flow.

6.3.1 Dikes

In general dikes extend outward from the bank into the channel at right angles or angled thereto, depending upon the circumstances and particular success achieved in past experiences. Along straight reaches, dikes should be perpendicular to the bank. Along sharp curves the dikes should be angled slightly downstream so as to deflect the flow toward the center of the channel. Some of the dikes are terminated with extensions parallel to the flow, forming L or T shapes, and are correspondingly referred as L or T heads.

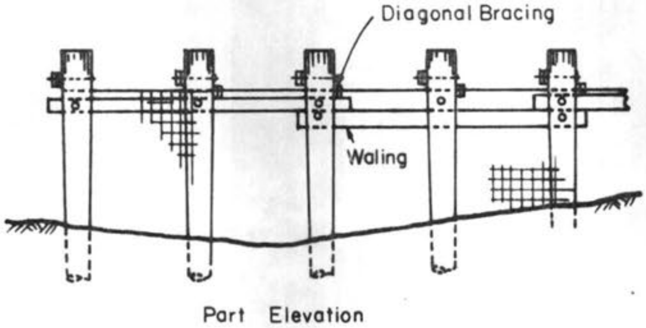
There are two principle types of dikes, permeable and impermeable. Permeable dikes are those which permit flow through the dike but at reduced velocities, thereby preventing further erosion of the banks and causing deposition of suspended sediment from the flow.

Timber pile dikes - Timber 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 timber piles can be placed together to form timber cribs, and rocks may be used to fill the space between the piles. Timber pile dikes are vulnerable to failure through scour. 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 with sufficient amounts of rock to form a combination permeable and impermeable dike. The various forms of timber pile dikes are illustrated in Fig. 6.3.2.

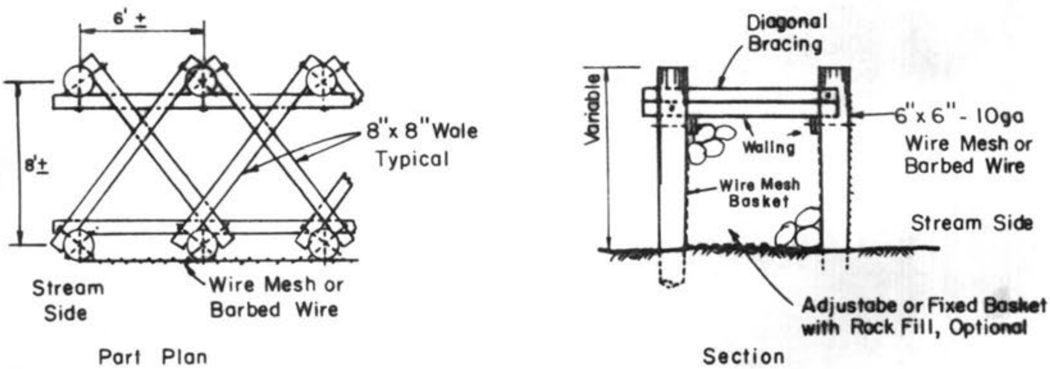
The arrangement of timber piles depends upon the velocity of flow, quantity of suspended sediment transport, and depth and width of river. If the velocity of flow is large, timber pile dikes are not likely to be very effective. Stabilization of the bank by other methods should be considered. On the other 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 timber pile dikes will be only partially 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 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.

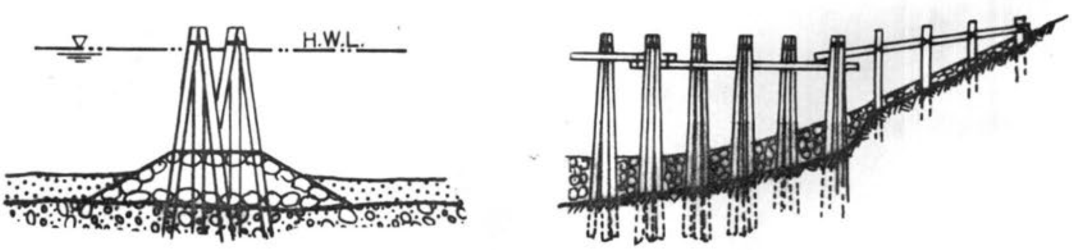
Stone-fill dikes - Stone-fill dikes are classed as impermeable dikes and do not depend on deposition of sediment between dikes nearly as much as permeable dikes. The principal function is to deflect the flow away from the bank and the dikes must be long enough to accomplish this purpose. The dikes may be angled downstream, angled upstream, or constructed normal to the bank. Variations such as a sloping dike, with



(a) Single row timber pile with wire fence



(b) Double row timber piles with rocks and wire fence



(c) Pile clusters

Fig. 6.3.2 Timber pile dikes (retards would be similar)

declining top elevation away from the bank, L or T head dikes, and curved dikes have been used. Stone-fill dikes are illustrated in Fig. 6.3.3.

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.

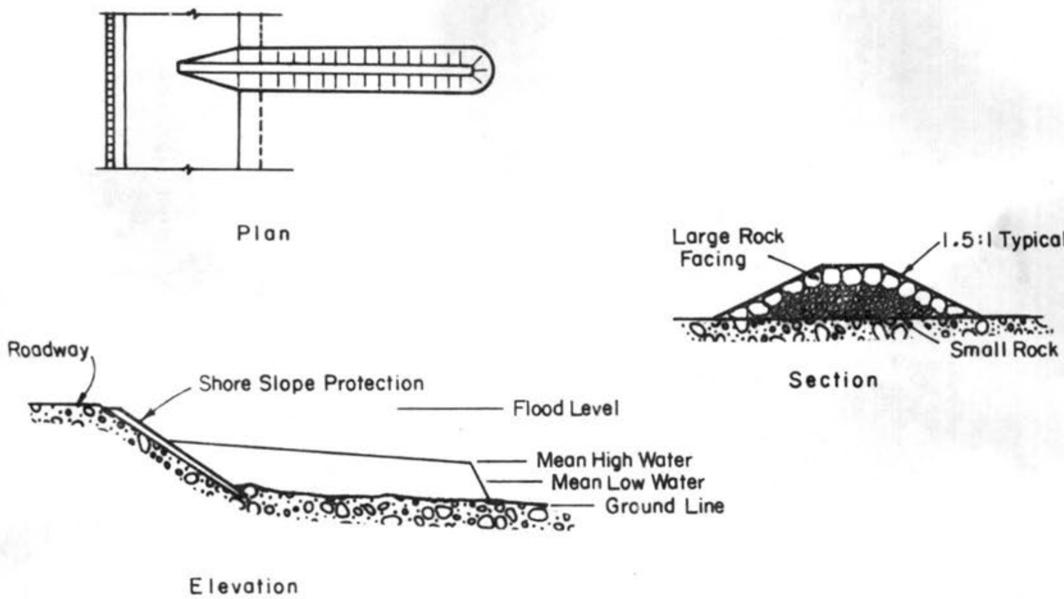


Fig. 6.3.3 Typical stone fill dike

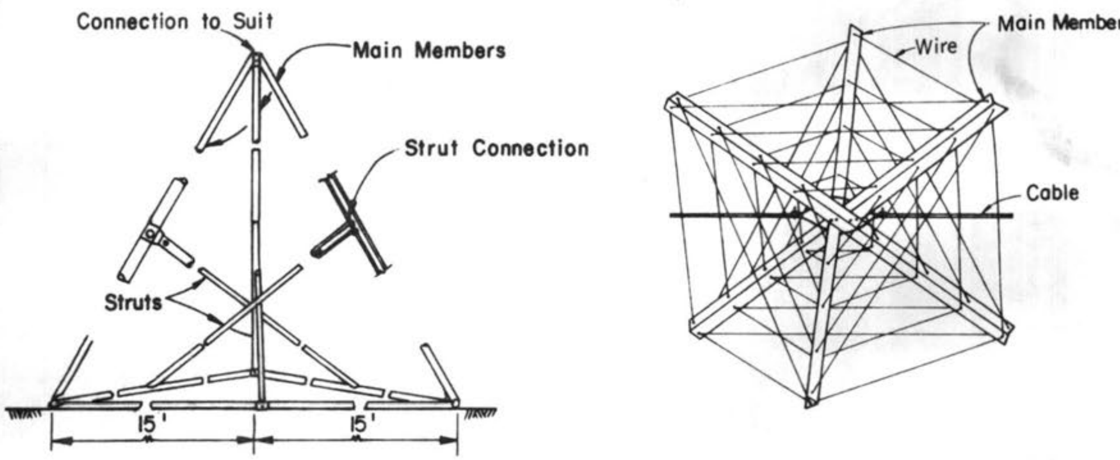
The ends of the dikes are subjected to local scour and appropriate allowance should be made for loss of dike material into the scour hole. The size of rock to be used for the dike depends on availability of material. Large rocks are generally used to cover the surface, while the internal section may be constituted with smaller rocks or earth-fill. Side slopes of 1.5:1 and 2:1 are common.

6.3.2 Retards

Retards are permeable devices placed parallel to embankments and river banks to decrease the stream velocities and prevent erosion.

Timber pile retards - The design of timber pile retards is essentially the same as timber pile dikes discussed in the previous section, and shown in Fig. 6.3.1. They may be used in combination with bank protection works such as riprap. The retard then serves to reduce the velocities sufficiently so that the riprap behind the retard is stable.

Steel jacks - These devices are 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 Fig. 6.3.4. Jacks must be tied together with cable and must have tiebacks to deadmen set in the bank. Tiebacks should be spaced every 100 ft and space between jacks should not be greater than their width.



(a) Typical Tetrahedron (b) Kellner Jack

Fig. 6.3.4 Steel jacks

6.3.3 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 Fig. 6.3.5.

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 200 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 Fig. 6.3.5.

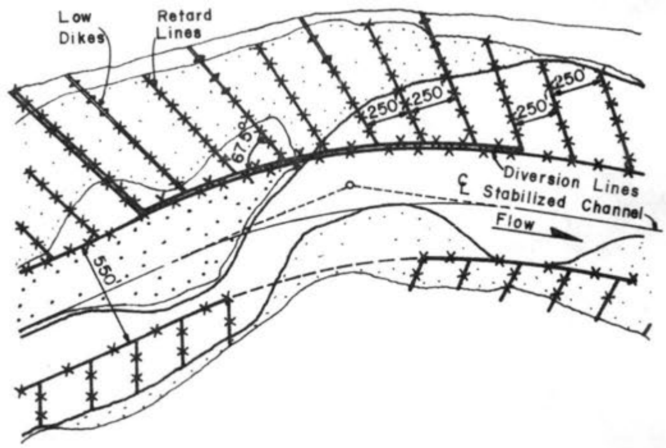


Fig. 6.3.5 Typical jetty-field layout

6.3.4 Spur dikes

To river engineers, spur dikes are dikes placed laterally from the river bank. These were discussed previously in this chapter and were simply called dikes. Spur dikes, to highway engineers, have acquired a special meaning because of their localized interest with rivers. *In highway engineering, spur dikes are guide banks 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 eliminated 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 spur dikes is a function of river geometry, quantity of flow on the floodplain, and size of bridge opening. A typical spur dike at the end of an embankment is shown in Fig. 6.3.6.

The recommended shape of a spur dike 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 spur dike 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 spur dikes for a skewed crossing is shown in Fig. 6.3.7.

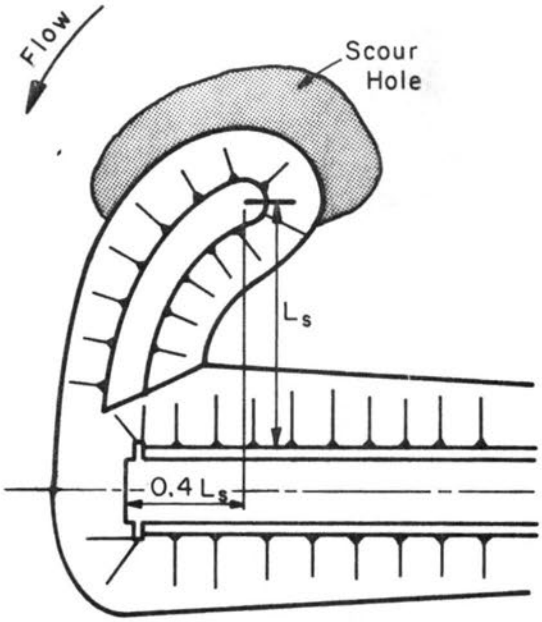


Fig. 6.3.6 Typical spur dike

Fig. 6.3.7 Spur dikes at skewed highway crossing

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. Generally, the U.S. practice has been to use spur dikes about 150 ft. Shorter spur dikes may be used where floodplain flow is small or scour potential at piers and embankment ends are small. In other countries, much longer spurs have been employed.

6.3.5 Bank protection

The term "bank protection" implies that the bankline has or is about to fail. In order to design bank protection properly, we must know how the bank fails. There are four principal ways in which a bank fails. They are:

(1) the erosion of soil particles on the bank either by the river currents or by waves.

(2) sloughing banks caused by excessive internal hydrostatic pressure in the bankline materials.

(3) slip-circle failures caused by the undermining of the toe.

(4) liquefaction and subsequent movement of the soil mass (called a flow slide).

The most common method of bank protection is with rock riprap. The sides of the bank or embankment are lined with large rocks to prevent erosion along the bank and at the toe. Section 6.4.0 in this chapter is devoted to riprap and further discussion is deferred. Other methods and devices are discussed in this section.

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 Fig. 6.3.8.

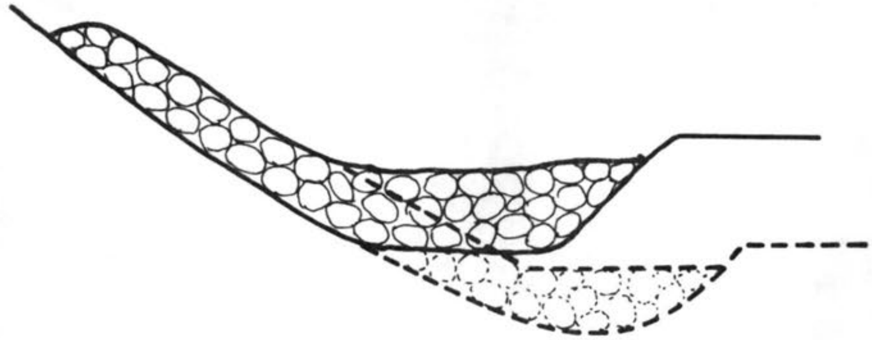


Fig. 6.3.8 Rock-fill trench

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 fill 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.

Rock-and-wire riprap - 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 extending the mattresses into the channel bed as shown in Fig. 6.3.9. As the bed along the toe is scoured, the mattress drops into the scour hole. Special wire baskets of manageable sizes 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 rather rapidly, which will destroy the intended protection along the base of the bank. Rusting of the wire mesh may also be a problem.

Mats can be made up in large sizes in the field. The mats are flexible and can conform to scour holes which threaten the stability of the banks. The mats should be linked together to prevent separation as subsidence takes place.

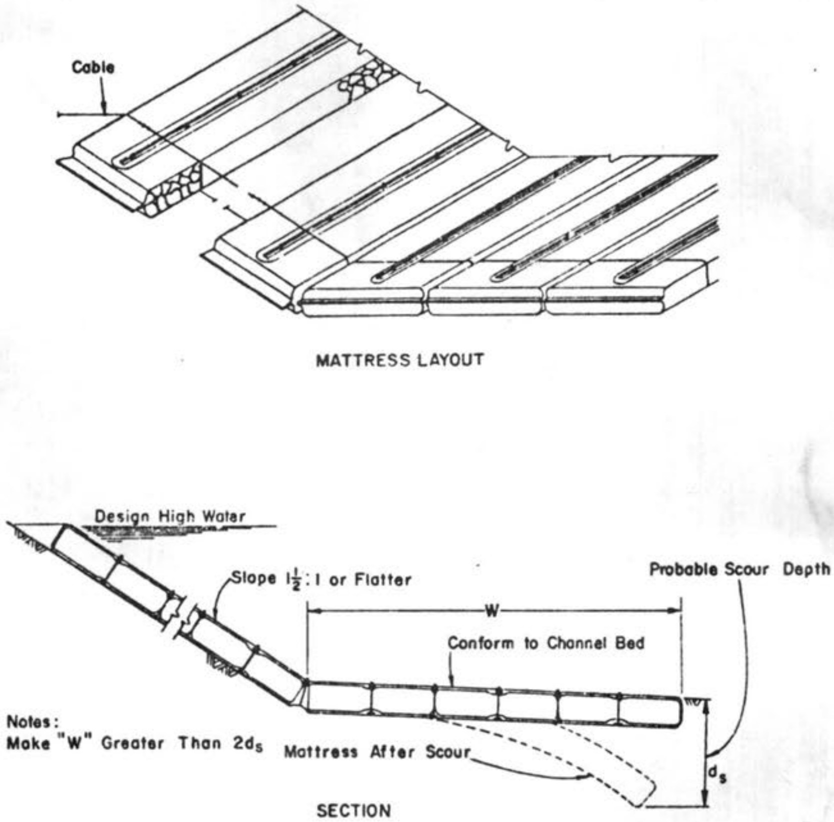


Fig. 6.3.9 Rock and wire mattress

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 Fig. 6.3.10.

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

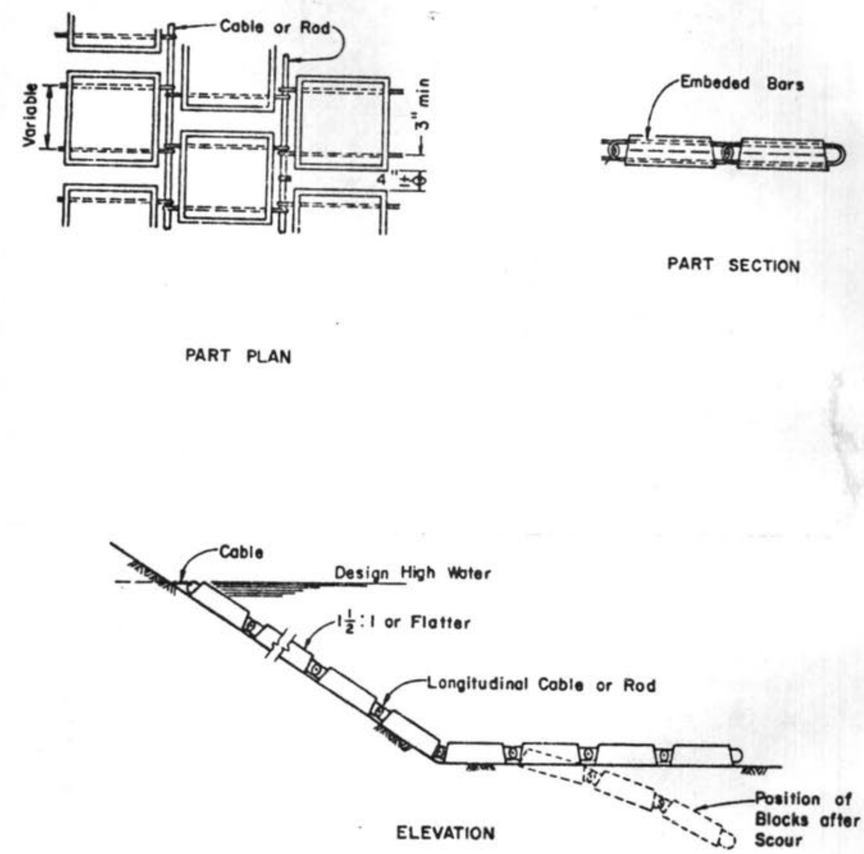


Fig. 6.3.10 Articulated concrete mattress

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.

Other types of mattresses - Woven willow, brush, woven lumber, asphalt, and soil cement mattresses are other types that can be utilized. In modern highway construction practice however, occasion to use these types arises only rarely. Concrete paving slabs are occasionally used. While paving slabs may be satisfactory along high water lines, they are not satisfactory for use below normal water levels because with even minor scour of the bank or toe, the rigid paving cannot conform to the scour hole and soon becomes undermined and the paving may break up. If the paving is extended well below the stream bed, or if the entire cross section is lined with concrete paving, (especially for small channels) this form of bank paving is satisfactory.

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 Fig. 6.3.11. 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.

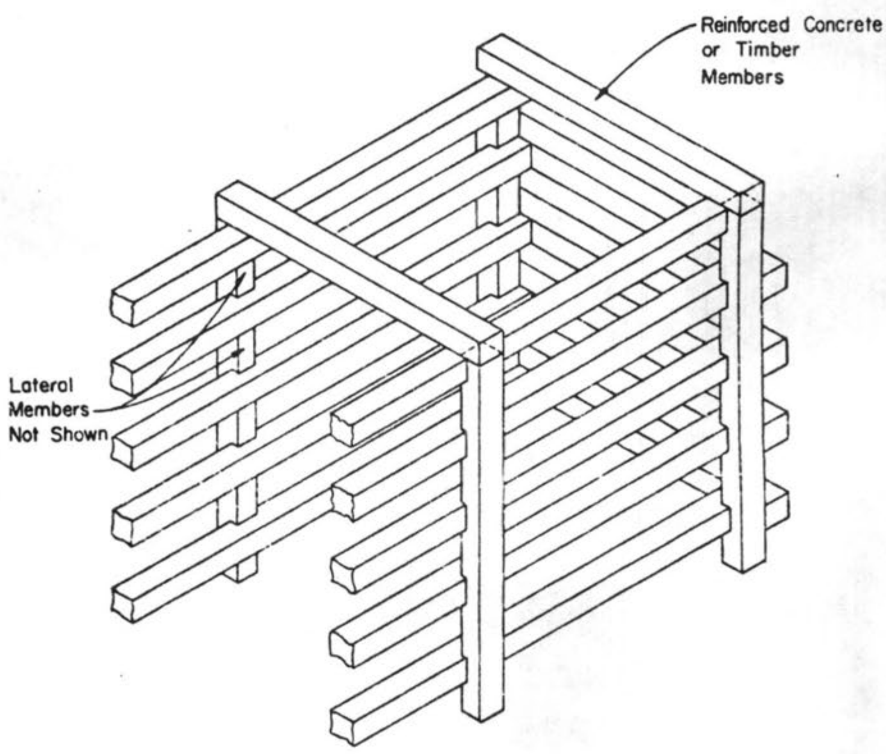


Fig. 6.3.11 Concrete or timber cribs

6.4.0 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. Rock riprap protection is flexible and local damage is easily repaired. Construction must be accomplished in a prescribed manner but is not complicated. Although the riprap must be placed to the proper level in the bed, there are no foundation problems. The appearance of rock riprap is natural and after a period of time vegetation will grow between the rocks. Wave runup on rock slopes is usually less than on other types. Finally, when the usefulness of the protection is finished, the rock is salvable.

The important factors to be considered in designing rock riprap protection are:

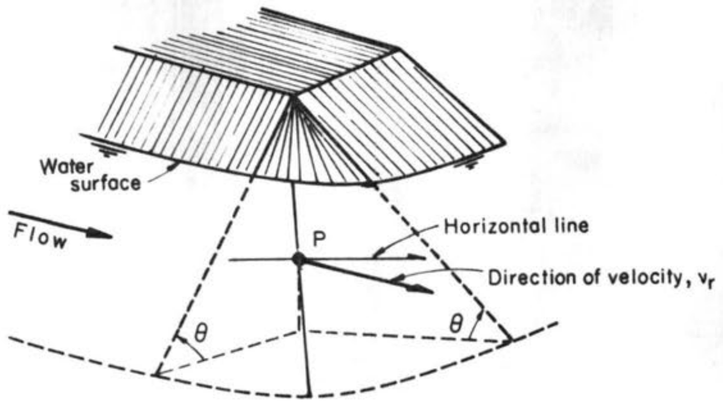
(1) The *durability* of the rock.
(2) The *density* of the rock.
(3) The *velocity* (both magnitude and direction) of the flow in the vicinity of the rock.
(4) The *slope* of the bed or bankline being protected.
(5) The *angle of repose* for the rock.
(6) The *size* of the rock.
(7) The *shape and angularity* of the rock.

The theoretical development of the relations between these important factors is presented in the Chapter VI appendix. A summary of the equations for riprap design is given below.

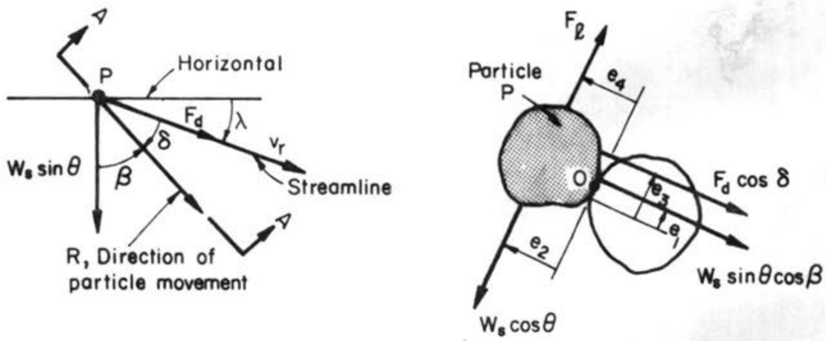
6.4.1 Oblique flow on a side slope

Consider flow along the nose of an embankment as shown in the diagrams of Fig. 6.4.1.

The forces on the rock particle are lift force, F_ℓ, drag force F_d, and weight of the particle W_s. Rock particles on side slopes tend to roll, rather than slide, so it is appropriate to consider stability of rock particles in terms of moments about a contact point 0 about which rotation must take place. The components of forces relative to the plane of motion are shown in Fig. 6.4.1b.



(a) General view



(b) View normal to the side slope (c) Section A-A

Fig. 6.4.1 Diagrams for the riprap stability analysis

At incipient motion, there is a balance of moments such that

$$e_2 W_s \cos\theta = e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell \tag{6.4.1}$$

where e is the moment arm of each force.

The factor of safety, S.F., of particles against rotation is then determined by the ratio of the moments.

$$\text{S.F.} = \frac{e_2 W_s \cos\theta}{e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell} \qquad 6.4.2$$

By following the development given in the Chapter VI appendix, the following equations relating the safety factor for rock riprap on side slope where the flow has a non-horizontal velocity vector are obtained.

$$\text{S.F.} = \frac{\cos\theta \tan\phi}{\eta' \tan\phi + \sin\theta \cos\beta} \qquad 6.4.3$$

where

$$\eta' = \eta \left\{\frac{1 + \sin(\lambda+\beta)}{2}\right\} \qquad 6.4.4$$

$$\eta = \frac{21\ \tau_o}{(S_s - 1)\gamma D} \qquad 6.4.5$$

$$\beta = \tan^{-1}\left\{\frac{\cos\lambda}{\frac{2\ \sin\theta}{\eta\ \tan\phi} + \sin\lambda}\right\} \qquad 6.4.6$$

The angle λ shown in Fig. 6.4.1 is the angle between the horizontal and the velocity vector (drag force) measured in the plane of the side slope.

The angles θ and β are defined in Fig. 6.4.1, ϕ is the angle of repose (given in Fig. 3.7.3 for dumped riprap), τ_o is the bed shear stress, D is the representative rock size, S_s is the specific weight of the rock, and η' and η are stability numbers.

In full-scale experiments with rock riprap below culvert outlets, Stevens (1969) developed the expression

$$D = \left\{\frac{\sum_{i=1}^{10} D_i^3}{10}\right\}^{1/3} \qquad 6.4.7$$

where $\quad D_i(i=1) = \dfrac{D_0 + D_{10}}{2}$

$$D_i(i{=}2) = \frac{D_{10} + D_{20}}{2}$$

$$\vdots$$

$$D_i(i{=}10) = \frac{D_{90} + D_{100}}{2}$$

for the *representative grain size of riprap*. The terms D_0, D_{10}, ... D_{100} are the sieve diameters of the rock for which zero percent, 10 percent, ..., 100 percent of the material by weight is finer. The value of D is greater than D_{50} except for uniform materials. If all the rock are one size, $D = D_{50}$. The representative grain size for riprap is discussed further in the Chapter VI appendix.

6.4.2 Horizontal flow on a side slope

In many circumstances the flow angularity with the horizontal is small, (i.e. $\lambda \approx 0$), and Eq. 6.4.6 reduces to

$$\tan\beta = \frac{\eta \tan\phi}{2 \sin\theta} \qquad 6.4.8$$

and Eq. 6.4.3 solved for η becomes

$$\eta = \left(\frac{S_m^2 - (S.F.)^2}{(S.F.)S_m^2}\right) \cos\theta \qquad 6.4.9$$

where S_m is the safety factor of rock particles from rolling down the slope with no flow. Accordingly

$$S_m = \frac{\tan\phi}{\tan\theta} \qquad 6.4.10$$

Alternatively, the safety factor may be expressed as

$$S.F. = \frac{S_m}{2} \{(S_m^2\eta^2 \sec^2\theta + 4)^{1/2} - S_m\eta \sec\theta\} \qquad 6.4.11$$

6.4.3 Flow on a sloping bed

When considering flow along a plane bed sloping at an angle $\bar{\alpha}$ with respect to the horizontal (see Fig. 6.4.2) the equations describing the stability of the riprap on the bed reduce to

$$S.F. = \frac{\cos\alpha \tan\phi}{\eta \tan\phi + \sin\alpha} \qquad 6.4.12$$

or $\eta = \cos\alpha\,[\frac{1}{S.F.} - \frac{\tan\alpha}{\tan\phi}]$ 6.4.13

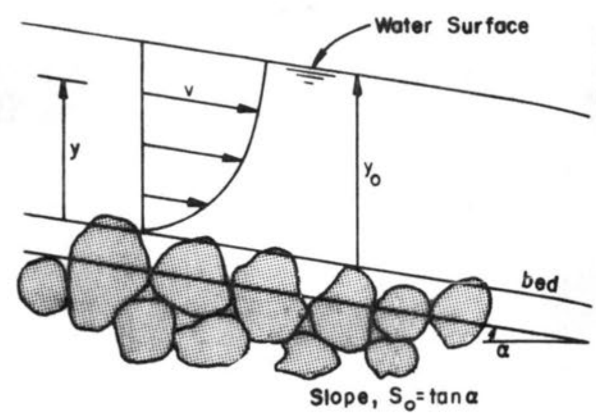


Fig. 6.4.2 Definition sketch for riprap on a channel bed

6.4.4 Flow on a horizontal bed

The most simple case is flow on a plane horizontal bed. In that case

$$S.F. = \frac{1}{\eta} \quad 6.4.14$$

If the particles on a plane horizontal bed are in incipient motion, S.F. = 1 so $\eta = 1$ and from Eq. 6.4.5

$$\frac{\tau_0}{(S_s - 1)\gamma D} = 0.047 \quad 6.4.15$$

which is the Shields' criteria for incipient motion in fully developed rough turbulent flow. The use of foregoing analysis for determining riprap size is illustrated by numerical examples in the appendix at the end of this chapter.

6.4.5 University of Minnesota method for riprap size determination

A method to determine size of riprap to line the entire channel of small to intermediate sizes (6 to 1000 cfs) has been proposed by Anderson, et al. (1970). *The method applies to channels that are trapezoidal or* *triangular in shape and which are essentially straight in alignment.* The proposed equation relating size of riprap to discharge and channel geometry is

$$Q = \frac{1}{118} \frac{D_{50}^{5/2}}{S_o^{13/6}} \frac{P}{R} \qquad 6.4.16$$

in which P is the wetted perimeter and R is hydraulic radius. Equation 6.4.16 is based on maximum shear stress related to rock diameter and Manning's equation of flow. It can be seen that for fixed channel size, P/R, the riprap size is a function of Q and S_o so that a family of design curves can be made for fixed P/R values.

6.4.6 Riprap gradation and placement

Riprap gradation should follow a smooth size distribution curve such as that shown in Fig. 6.4.3. The ratio of maximum size to median size,

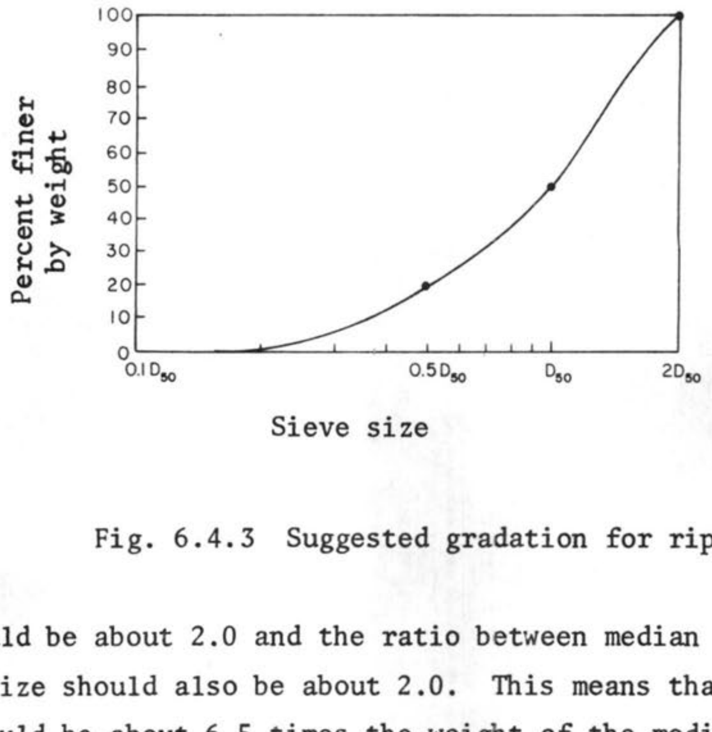


Fig. 6.4.3 Suggested gradation for riprap

D_{50}, should be about 2.0 and the ratio between median size and the 20 percent size should also be about 2.0. This means that the largest stones would be about 6.5 times the weight of the median size and small sizes would range down to gravels. The representative rock size D for the gradation shown in Fig. 6.4.3 is 1.25 D_{50} (calculated using Eq. 6.4.7) which is approximately equal to the D_{67}.

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 are more suitable than 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 of bed material. The representative grain size of the riprap is determined by Eq. 6.4.7 and the filter is designed in accordance with the criteria given in the next section.

Riprap should be hard, dense and durable to withstand long exposure to weathering. Visual inspection is most often adequate to judge quality, but laboratory tests may be made to aid the judgment of the field inspector.

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 never 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 require 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 advantageously 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.

The thickness of riprap should be sufficient to accommodate the largest stones in the riprap. With a well-graded riprap with no voids, this thickness should be adequate. If strong wave action is of concern, the thickness should be increased by 50 percent.

6.4.7 Filters for riprap

Filters underneath the riprap are recommended to protect the fine embankment or riverbank material from washing out through the riprap. Two types of filters are commonly used; gravel filters and plastic 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 3/16 in. to an upper limit depending on the gradation of the riprap with maximum sizes of about 3 to 3-1/2 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:

$$(1)\quad \frac{D_{50}\ \text{(Filter)}}{D_{50}\ \text{(Base)}} < 40$$

$$(2)\quad 5 < \frac{D_{15}\ \text{(Filter)}}{D_{15}\ \text{(Base)}} < 40$$

$$(3)\quad \frac{D_{15}\ \text{(Filter)}}{D_{85}\ \text{(Base)}} < 5$$

Plastic filter cloths - Plastic filter cloths are being used beneath riprap and other revetment materials such as articulated concrete blocks with considerable success. The cloths 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. Some amount of care must be exercised in placing riprap over the plastic cloth filters to prevent damage. Experiments and results with various cloth filters were reported by Calhoun, Compton and Strohm (1971) in which specific manufacturers and brand names are listed. Stones weighing as much as 3000 lbs have been placed on plastic filter cloths with no apparent damage.

Filters can be placed subaqueously by using steel rods as weights fastened along the edges. Additional intermediate weights would assist in sinking the cloth in place. Durability of filter cloths has not yet been established because they have been in use only since about 1967. However, inspections at various installations indicate little or no deterioration had occurred in the few (1 to 4) years that have elapsed for test installations.

6.4.8 Guide to channel improvement, river training and bank stabilization

The type of channel improvement and devices used for training and bank stabilization depend upon the size of river with regard to width, depth and discharge; type of rivers, 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. The decisions for 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 6.4.1 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 meander systems, stabilization and improvements may be required. This is the interpretation to be used in the table. The X in the box indicates that consideration could be given to use of the particular device. The absence of a check mark in the box indicates that the devices are not often used, but consideration could be given to them in special circumstances. Additional remarks are noted on the table.

6.5.0 SCOUR AND DEGRADATION

Changes in bed level which affect highway construction and structures may be described by three types of interrelated phenomena. They are:

(1) *Local scour*, caused by local disturbances in the flow such as vortices and eddies. Examples are scour at the base of piers, dikes and other obstructions in a stream.

(2) *General scour* due to contractions causing increasing velocities across the entire contracted width. Examples are scour at contracted stream channels caused by spur dikes, embankments, and accumulation of debris at bridge openings.

Table 6.4.1 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 and Wire	Concrete	Other	Cribs
Large	Meandering		X	X	*	*		X		X	X	*	X	X	
	Braided	X	X	X	*	*		X		X	X	*	X	X	
	Straight		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

(3) *Degradation or aggradation* of a stream channel over long lengths and time due to changes in controls, such as dams, changes in sediment content, and changes in river geomorphology, as a consequence of changing from a meandering river to a braided stream.

These effects are in general additive, so that local scour can occur while scour due to contraction is occurring and degradation or aggradation of the stream is taking place. The respective time scales for scour are progressively larger.

6.5.1 Degradation and aggradation

Degradation and aggradation of the river bed has been discussed previously. A long reach of river channel may be subjected to a general lowering or raising of the bed level over a long period of time. Prediction of ultimate degradation or aggradation of a stream was discussed in Chapters IV and V. Degradation or aggradation at a bridge opening must be anticipated. Otherwise, pier and abutment foundation depths may be inadequate when degradation occurs, or the road and bridge deck levels may be too low in a few years with an aggrading stream.

6.5.2 General scour

Scour at contractions occur because the flow area becomes smaller than the normal stream, the average velocity and bed shear stress increase, hence there is increase in stream power locally at the contraction and more bed material is transported through the contracted section than is transported into the section. As the bed level is lowered, the velocity decreases, shear stress decreases and equilibrium is restored when the transport rate of sediment through the contracted section is equal to the incoming rate.

Flows confined to the channel - Consider a situation where a normal river channel is narrowed by a contraction. Scour due to the contraction may be determined in the following way (Nordin, 1971). 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

$$q_2 = \frac{W_1}{W_2} q_1 \qquad 6.5.1$$

Here $q_1 = y_1 V_1$ and $q_2 = y_2 V_2$ and the subscript 2 refers to conditions in the contracted section. The sediment transport rate at the contracted section after equilibrium is established must be

$$q_{s2} = \frac{W_1}{W_2} q_{s1} \qquad 6.5.2$$

The relationships of y and V at sections 1 and 2 are shown in Fig. 6.5.1 for constant q_1 and q_2.

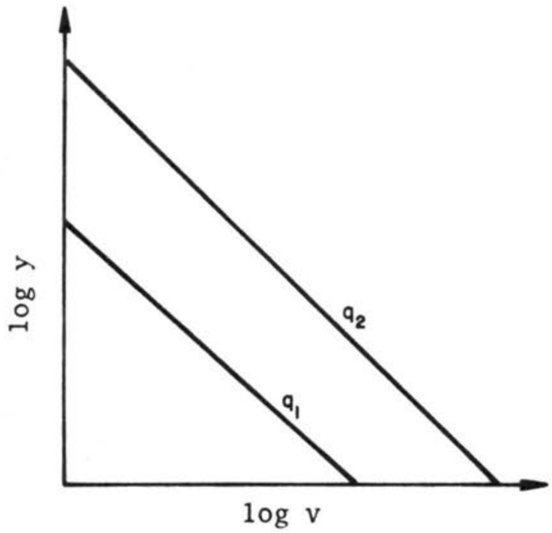


Fig. 6.5.1 Unit discharge as a function of depth and velocity

From one of the sediment transport equations given in Chapter III 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 Fig. 6.5.2 using the method of Colby.

Now overlap Fig. 6.5.1 with Fig. 6.5.2. The result is shown in Fig. 6.5.3.

The depth of scour due to the contraction is then

$$y_s = y_2 - y_1 \qquad 6.5.3$$

Overbank flow with flow in the channel - Laursen (1960) developed an equation for scour at a contraction, where in addition to channel flow there is overbank flow concentrating into the contracted channel

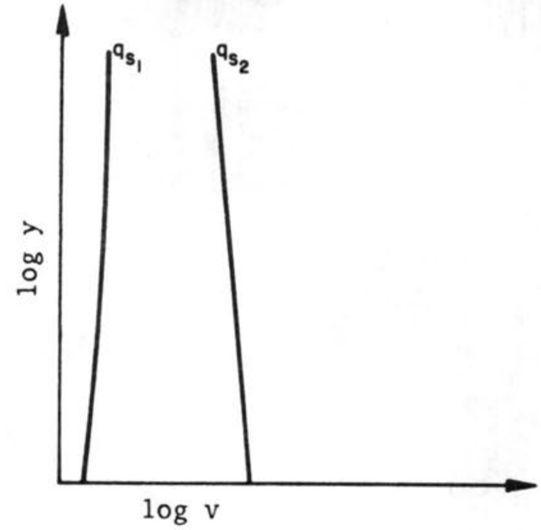


Fig. 6.5.2 Sediment transport rate as a function of depth and velocity

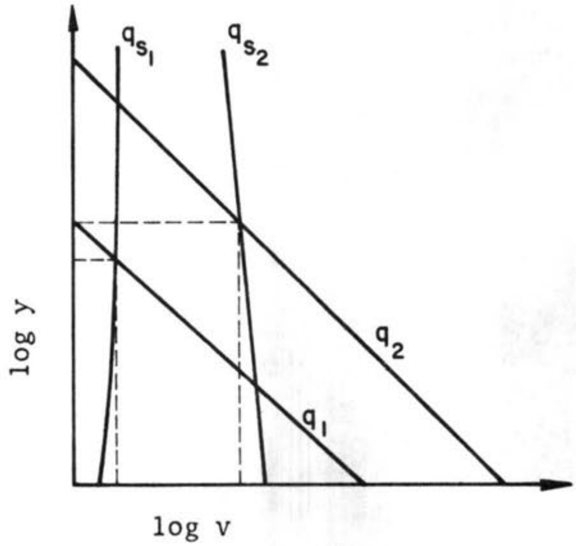


Fig. 6.5.3 Determination of scour depth

(designated by subscript 2). The equation to predict depth of flow at section 2 is

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{\frac{6(2+f)}{7(3+f)}} \left(\frac{n_2}{n_1}\right)^{\frac{6f}{7(3+f)}} \qquad 6.5.4$$

in which Q_c is the approach channel flow rate, Q_t is the contracted channel flow rate which is greater than the approach channel flow rate by the amount of flow on the floodplain. The variable n is the Manning roughness coefficient, W is the channel width and the exponent f is given below.

V_{*c}/ω	f
< 0.5	0.25
1	1
> 2	2.25

Here V_{*c} is the shear velocity in the approach channel and ω is the fall velocity of the bed material.

Overbank flow only - For scour at bridges on a floodplain where there is no sediment transport from upstream, Laursen (1963) proposed that

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{V_1^2}{120\ y_1^{1/3}\ D_{50}^{2/3}}\right)^{3/7} \qquad 6.5.5$$

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.

6.5.3 Local scour

Local scour occurs in the bed of the channel around piers and embankments due to the actions of vortex systems induced by the obstructions to the flow. Local scour occurs in conjunction with or in the absence of degradation, aggradation, and scour due to contractions. There is need to understand the mechanism of local scour and calculation of potential scour depths, after which means may be considered in the design to eliminate or reduce its magnitude by suitable protective methods.

Mechanism of local scour - The basic mechanism causing local scour is the vortex of fluid resulting from the pileup of water on the upstream edge and subsequent acceleration of flow around the nose of the pier or embankment. The action of the vortex is to erode 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 is increased, the strength of the vortex reduces, the transport rate reduces and equilibrium is reestablished and scouring ceases.

The flow field and vortex systems around a circular pier and approach embankment are illustrated in Figs. 6.5.4 and 6.5.5. Although the vortex system is known to be the cause of local scour, it has not been possible as yet to calculate the strength of the vortex and relate the velocity field with subsequent scour. Until further research and study makes this possible, average velocity and local depth of flow are used in the equations to predict local scour depths.

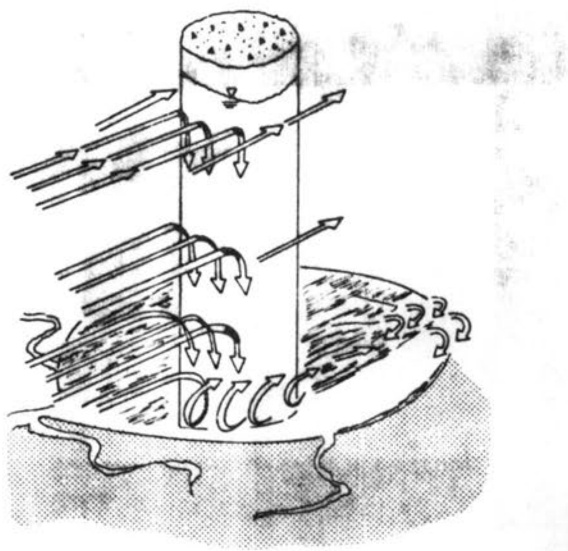


Fig. 6.5.4 Schematic representation of scour at a cylindrical pier

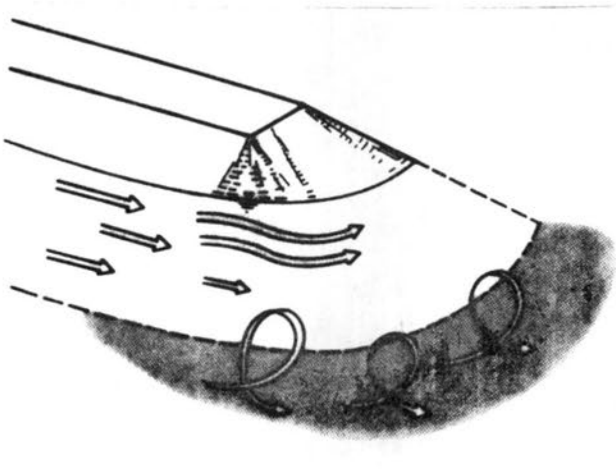


Fig. 6.5.5 Schematic representation of scour at an embankment

Local scour around embankments - A typical scour hole at an embankment and adjacent pier is illustrated in Fig. 6.5.6.

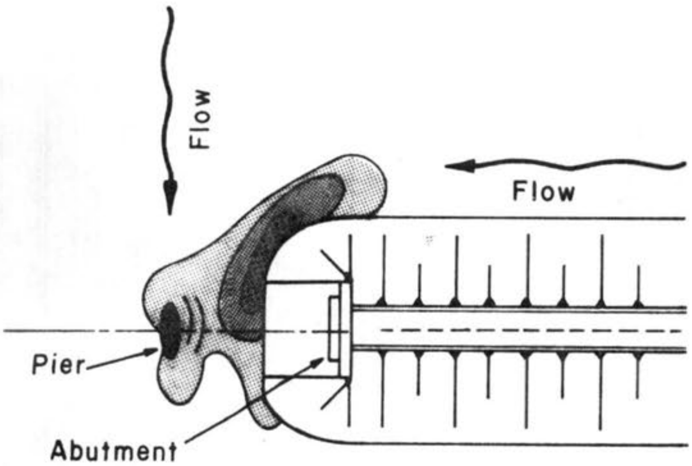


Fig. 6.5.6 Typical scour at an embankment and adjacent pier

The depth of scour varies with time because the sediment transported into the scour hole from upstream varies, depending upon the presence or absence of dunes. The time required for dune motion is much larger than the time for local scouring action. Thus, even with steady state conditions, the depth of scour is likely to fluctuate with time when there are dunes traveling on the channel bed. The depth of the scour hole is more variable with larger dunes. When the crest of the dune reaches the local scour area, the transport rate into the hole increases, the scour hole fills and the scour depth temporarily decreases. When a trough approaches, there is less sediment supply and the scour depth increases to try to reestablish equilibrium in sediment transport rates. A mean scour depth between these oscillations is referred to as equilibrium scour depth. It is not uncommon (as determined in laboratory tests) to find maximum depths to be 30 percent greater than equilibrium scour depths. The depth that would be reached if no sediment was transported into the scour hole is the "clear-water" scour depth.

Detailed studies of scour around embankments have been made mostly in laboratories. There are very few case studies for scour at field installations. According to the studies of Liu et al. (1961) *the equilibrium scour depth for local scour in sand at a spill slope when the flow is subcritical is determined by the expression*

$$\frac{y_s}{y_1} = 1.1 \left(\frac{a}{y_1}\right)^{0.40} F_{r1}^{\ 0.33} \qquad 6.5.6$$

where y_s is the equilibrium depth of scour measured from the mean bed level to the bottom of the scour hole, a is the embankment length (measured normal to the wall of a flume), y_1 is upstream depth and F_{r1} is the upstream Froude number determined as

$$F_{r1} = \frac{V_1}{\sqrt{gy_1}} \qquad 6.5.7$$

If the embankment terminates at a vertical wall and has a vertical wall on the upstream side then the scour hole depth in sand nearly doubles (Liu, 1961 and Gill, 1972). That is,

$$\frac{y_s}{y_1} = 2.15 \left(\frac{a}{y_1}\right)^{0.40} F_{r1}^{\ 0.33} . \qquad 6.5.8$$

The lateral extent of the scour hole is nearly always determinable from the depth of scour and the natural angle of repose of the bed material.

Field data for scour at embankments for various size rivers are scarce, but data collected at rock dikes on the Mississippi indicate that

$$\frac{y_s}{y_1} = 4\ F_{r1}^{\ 0.33} \qquad 6.5.9$$

determines the equilibrium scour depth for large a/y_1. 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 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.

Accordingly, it is recommended that Eq. 6.5.6 be applied for embankments with $0 < a/y_1 < 25$ and Eq. 6.5.9 be used for $a/y_1 > 25$. The equations are shown in graphical form in Fig. 6.5.7. In applying Eq. 6.5.6, the embankment length a is measured from the high water line at the valley bank perpendicularly to the end of the embankment at the bridge. If $a/y_1 > 25$, then scour depth is independent of a/y_1 and depends only on the approach Froude number and depth of flow. For $a/y_1 < 25$, as for example at dikes and short highway embankments, Eq. 6.5.6 would apply.

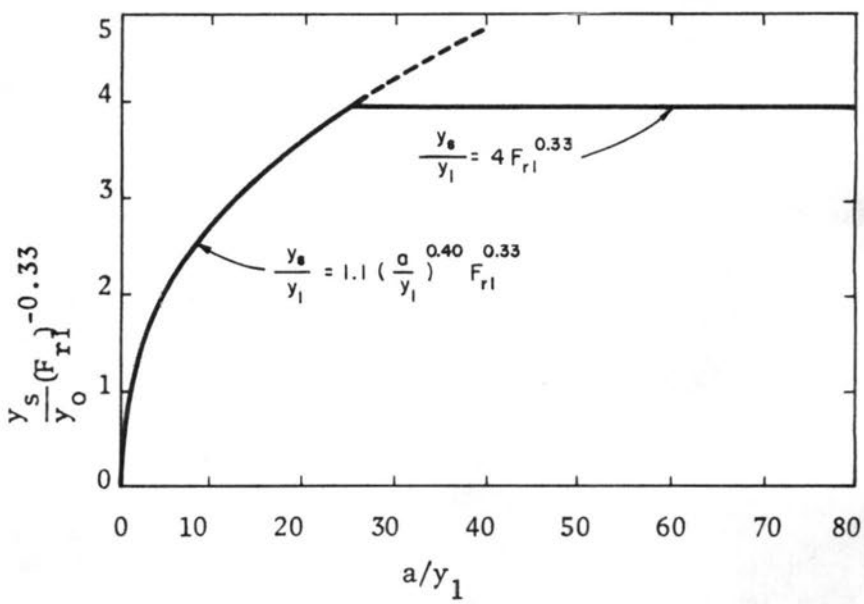


Fig. 6.5.7 Recommended prediction equation for embankment scour

It should be recalled that maximum depth of scour is about 30 percent greater than equilibrium scour depth. The lateral extent of scour can be determined from the angle of repose of the material and scour depth.

If the embankment is angled downstream, the depth of scour is reduced because of streamlining effect. Embankments that are angled upstream have deeper scour holes. The calculated scour depth should be adjusted in accordance with the curve of Fig. 6.5.8 which is patterned after Ahmad (1953).

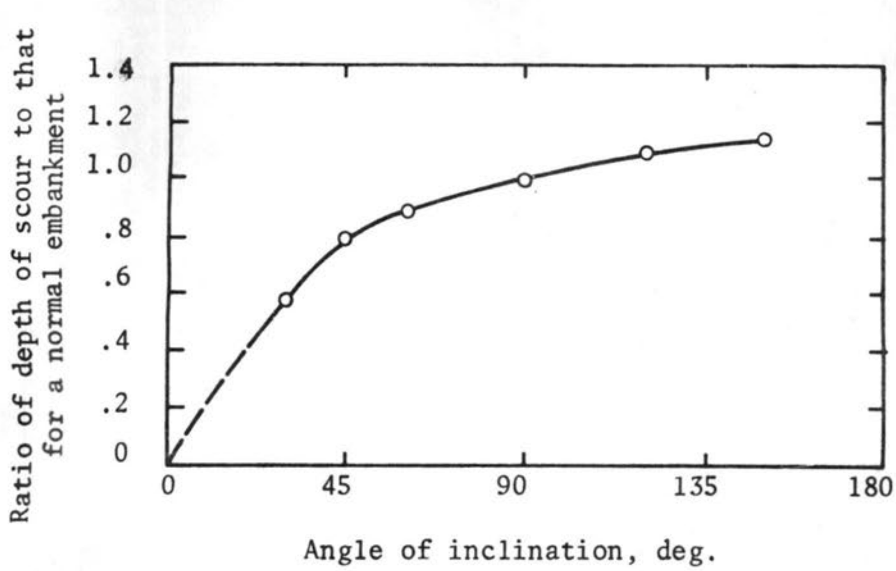


Fig. 6.5.8 Scour reduction due to embankment inclination

Local scour around piers - Local scour at bridge piers is a result of vortex systems developed at the pier. The so-called *"horseshoe" vortex system is dominant at piers* causing deepest scour at the nose of the pier. The axis of this vortex, or the vortex line, is horizontal, and wraps around the base of the pier in the shape of a horseshoe. The high velocities scour the bed. The wake-vortex system has vertical axes and develops because of blockage of the flow by the pier. The wake vortices are commonly seen as "eddies". This vortex system suspends the scoured material and carries it downstream with the flow. Large wake vortices or eddies are set up downstream of embankments which scour the downstream sides of the embankments, the river bank and the stream bed. Wake vortices downstream of piers may create sufficient velocities to cause bed scour if the piers are wide. For most piers however, very little additional scouring is caused by wake vortices.

The shape of the pier is very significant with respect to scour depth because it reflects the strength of the horseshoe vortex at the base of the pier. A blunt-nose pier causes the greatest scour depth. Streamlining the front end of the pier reduces the strength of the horseshoe vortex reducing the scour. Streamlining the downstream end of piers reduces the strength of wake vortices. Common shapes of piers are shown in Fig. 6.5.9. The width of piers are labeled "a" and length designated ℓ.

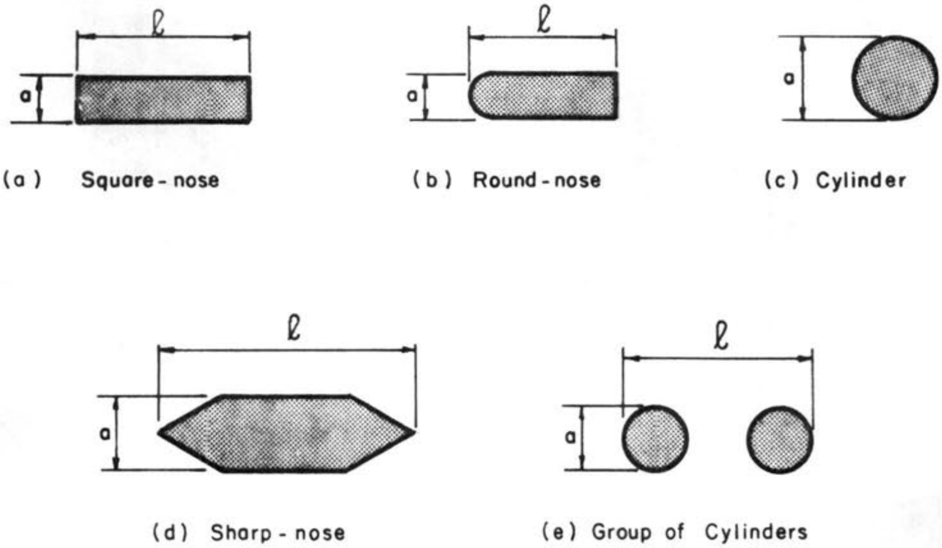


Fig. 6.5.9 Common pier shapes

The equilibrium of scour at the nose of square-nosed piers can be estimated with the equation

$$\frac{y_s}{y_1} = 2.2 \left(\frac{a}{y_1}\right)^{0.65} F_{r1}^{\;0.43} \qquad 6.5.10$$

where y_1 is upstream depth of flow and F_{r1} is the Froude number. The scour depth for circular cylinders is

$$\frac{y_s}{y_1} = 2.0 \left(\frac{a}{y_1}\right)^{0.65} F_{r1}^{\;0.43} \qquad 6.5.11$$

The form of Eq. 6.5.10 for square-nosed piers and Eq. 6.5.8 for vertical-wall embankments are similar although the variable "a" takes on different meanings. This similarity exists because the scour mechanisms for the two cases are similar.

Cylindrical piers have been widely investigated in the laboratory. The exponents in Eq. 6.5.11 were determined from laboratory data shown in Fig. 6.5.10. In this figure, the abscissa is labeled $(a/y_1)^3 F_{r1}^2$ to spread the data. Note that

$$\{(\frac{a}{y_1})^3 F_{r1}^2\}^{0.215} = (\frac{a}{y_1})^{0.65} F_{r1}^{0.43} \qquad 6.5.12$$

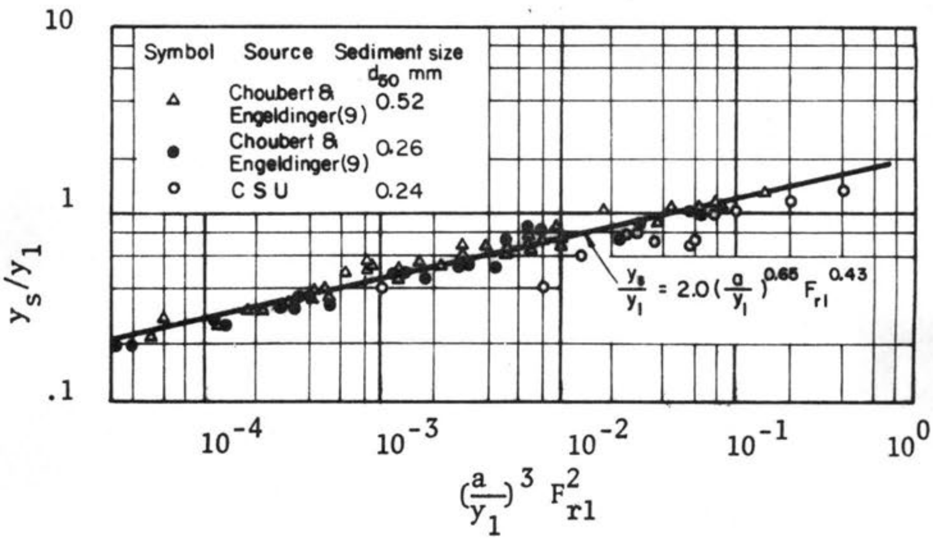


Fig. 6.5.10 Results of laboratory experiments for scour at circular piers

The scour depth decreases as a consequence of streamlining. The reduction in scour depth can be estimated from the value given in Table 6.5.1.

Table 6.5.1 Reduction in scour depths for equal projected width of pier

Type of Pier	y_s/y_s (square nose)
Square nose	1.0
Cylinder	0.9
Round nose	0.9
Sharp nose	0.8
Group of cylinders	0.9

Pier alignment other than parallel with flow direction will create deeper scour holes because in effect, the dimension "a" increases. In Table 6.5.2 multiplying factors to Eq. 6.5.10 are given. Scour at single cylindrical piers are invariant to flow direction, but groups of cylindrical piers would be affected by flow angularity.

Table 6.5.2 Multiplying factors for scour depths with skewed flow direction

Angle of skew in degrees	$\ell/a = 4$	$\ell/a = 8$	$\ell/a = 12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.5	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

The concept of equilibrium scour depth also applies to scour around piers. *It should be recalled that maximum scour depth at piers could be as large as 30 percent greater than equilibrium scour depth.*

The base level to which scour depths can be referenced is not a trivial question. In some rivers, dune heights may be as large as 20 to 30 feet, and y_1 would normally be measured from some level closer to the tops of the dunes. Scour depths on the other hand should be referenced nearer the trough of the dunes. The bed level for scour due to contractions is first calculated. This establishes y_1 and F_{r1}. The local scour depth is then referenced to that bed level.

6.5.4 Protection of structures for local scour

Three basic methods may be used to protect structures from damage due to local scour. The first is to prevent damaging vortices from developing and the second is to provide protection at some level at or below the stream bed to arrest development of the scour hole. The third is to place the foundations of structures at such depth that the deepest scour hole will not threaten the stability of the structure. The last method is often very expensive, and risk is involved because of the uncertainty associated with estimating the additive effects of scour due to contraction and channel degradation.

Vortex reduction - As previously mentioned in Section 6.5.3, streamlining the piers can reduce scour depth by 10 to 20 percent. Another method of reducing the vortex strength at the pier is to construct barriers upstream of bridge piers, as for instance with a cluster of piles. While the piles are subjected to scour, failure of these piles are 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. Spur dikes can be placed at the ends of approach embankments to reduce local scour at the bridge. Spur dikes were discussed in Section 6.3.4.

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. 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 Eqs. 6.5.10 and 6.5.6. 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.

A *structural concrete shelf* placed at about 0.5 y_s, where y_s is calculated from Eq. 6.5.10, 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.

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.

6.6.0 SUMMARY

Ordinarily, highway agencies cannot justify extensive control installations. As a consequence most highway installations will suffer damages at some time. Usually, it is less expensive to repair damages at some future time than to invest large amounts of money in the initial structures.

The information in this chapter is guidance on the principles of training or controlling relatively short reaches of river channels. Over long periods of time, normal river behavior can result in the river outflanking or destroying training structures in short reaches. Often, in the initial design, some thought as to how the river will attack a structure can result in efficient maintenance plans.

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APPENDIX

6.A1.0 SAFETY FACTORS FOR RIPRAP - THEORETICAL DEVELOPMENT

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; (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).

6.A1.1 Oblique flow on a side slope

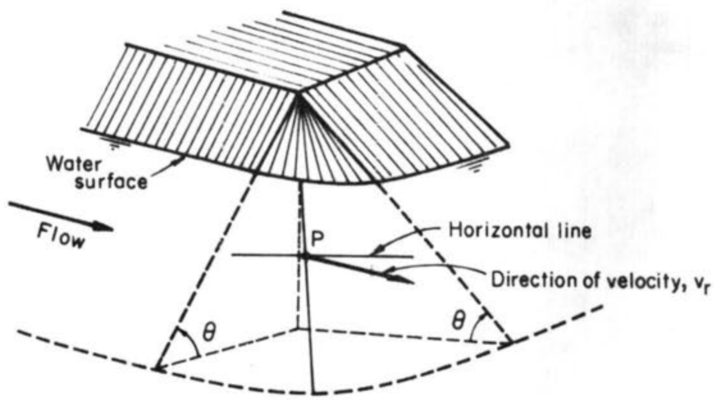
Consider flow along an embankment as shown in Fig. 6.A1.1. The fluid forces on a rock particle identified as P in Fig. 6.A1.1a result primarily from fluid pressures around the surface of the particles. The lift force F_ℓ is defined herein as the fluid force normal to the plane of the embankment. The 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 Fig. 6.A1.1b the direction of movement is defined by the vector R. The point of contact about which rotation in the R direction occurs is identified as point "0" in Fig. 6.A1.1c.

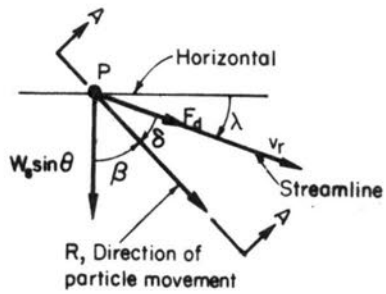
The forces acting in the plane of the side slope are F_d and $W_s \sin\theta$ as shown in Fig. 6.A1.1b. The angle θ is the side slope angle. The lift force acts normal to the side slope and the component of submerged weight $W_s \cos\theta$ acts normal to the side slope as shown in Fig. 6.A1.1c.

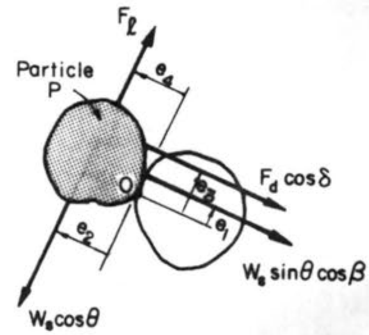
At incipient motion, there is a balance of moments about the point of rotation such that

$$e_2 W_s \cos\theta = e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell \qquad 6.A1.1$$



(a) General view





(b) View normal to the side slope (c) Section A-A

Fig. 6.A1.1 Diagram for the riprap stability conditions

The moment arms e_1, e_2, e_3 and e_4 are defined in Fig. 6.A1.1c and the angles δ and β are defined in Fig. 6.A1.1b.

The factor of safety S.F. of the particle P against rotation 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

$$\text{S.F.} = \frac{e_2 W_s \cos\theta}{e_1 W_s \sin\theta \cos\beta + e_3 F_d \cos\delta + e_4 F_\ell} \qquad 6.\text{A1.2}$$

If there is no flow and the side slope angle is increased to the angle of repose ϕ for the rock particles, the safety factor becomes unity. The no flow condition causes F_d and F_ℓ to become zero. Then,

$$\begin{aligned} \text{S.F.} &= 1.0 \\ \theta &= \phi \\ \beta &= 0 \text{ deg} \\ \lambda &= 0 \text{ deg} \\ \delta &= 90-\lambda-\beta \text{ (See Fig. 6.A1.1b)} \\ &= 90 \text{ deg} \end{aligned}$$

With these values, Eq. 6.A1.2 reduces to

$$\frac{e_2 W_s}{e_1 W_s} \frac{\cos\theta}{\sin\theta} = 1 \qquad 6.\text{A1.3}$$

or

$$\tan\phi = \frac{e_2}{e_1} \qquad 6.\text{A1.4}$$

That is, the ratio of the moment arms e_2/e_1 is characterized by the natural angle of repose ϕ. Further, it is assumed that the ratio e_2/e_1 is invariant to the direction of particle motion indicated by the angle β.

Dividing both numerator and denominator by $e_2 W_s$, Eq. 6.A1.2 is transformed to

$$\text{S.F.} = \frac{\cos\theta \tan\phi}{\eta' \tan\phi + \sin\theta \cos\beta} \qquad 6.\text{A1.5}$$

in which

$$\eta' = \frac{e_3 F_d}{e_2 W_s} \cos\delta + \frac{e_4 F_\ell}{e_2 W_s} \qquad 6.\text{A1.6}$$

The variable η' is called the stability number for the particles on the embankment side slope and is related to the Shields' parameter

$$\frac{\tau_o}{(S_s - 1)\gamma D}$$

Here τ_o is the average tractive force on the side slope in the vicinity of the particle P, γ is unit weight of water and D is the diameter of the rock particles.

The angle λ shown in Fig. 6.A1.1b is the angle between the horizontal and the velocity vector (or drag force) measured in the plane of the side slope. Then

$$\delta = 90-\lambda-\beta \qquad 6.A1.7$$

so

$$\cos\delta = \cos(90-\lambda-\beta)$$

$$= \sin(\lambda+\beta) \qquad 6.A1.8$$

Also $$\sin\delta = \sin(90-\lambda-\beta)$$

$$= \cos\lambda \cos\beta - \sin\lambda \sin\beta \qquad 6.A1.9$$

It is assumed that the moments of the drag force F_d and the component of submerged weight $W_s \sin\theta$ normal to the path R are balanced so that the direction of particle motion will be along R. Thus

$$e_3 F_d \sin\delta = e_1 W_s \sin\theta \sin\beta \qquad 6.A1.10$$

It follows then from Eqs. 6.A1.9 and 6.A1.10 that

$$\sin\beta = \frac{e_3 F_d \sin\delta}{e_1 W_s \sin\theta}$$

$$= \frac{e_3 F_d(\cos\lambda \cos\beta - \sin\lambda \sin\beta)}{e_1 W_s \sin\theta} \qquad 6.A1.11$$

or

$$\tan\beta = \frac{\cos\lambda}{\frac{e_1 W_s}{e_3 F_d} \sin\theta + \sin\lambda} \qquad 6.A1.12$$

The stability number η for particles on a plane bed ($\theta = 0$) with $\delta = 0$ would be

$$\eta = \frac{e_3 F_d}{e_2 W_s} + \frac{e_4 F_\ell}{e_2 W_s} \qquad 6.A1.13$$

according to Eq. 6.A1.6. Also, Eq. 6.A1.5 becomes

$$S.F. = \frac{1}{\eta} \qquad 6.A1.14$$

for flow over a plane flat bed.

For incipient motion conditions for flow over a plane flat bed, S.F. = 1.0 by definition so from Eq. 6.A1.14, $\eta = 1.0$. When the flow along the bed is fully turbulent, the Shield's parameter for incipient motion has the value 0.047 according to Gessler (1971). That is, with $\eta = 1.0$,

$$\frac{\tau_o}{(S_s - 1)\lambda D} = 0.047 \qquad 6.A1.15$$

For flow conditions other than incipient η is the ratio

$$\frac{1}{0.047} \frac{\tau_o}{(S_s - 1)\lambda D}$$

or

$$\eta = \frac{21\ \tau_o}{(S_s - 1)\gamma D} \qquad 6.A1.16$$

For convenience, let

$$M = \frac{e_4 F_\ell}{e_2 W_s} \qquad 6.A1.17$$

and

$$N = \frac{e_3 F_d}{e_2 W_s} \qquad 6.A1.18$$

In terms of these new variables, Eq. 6.A1.6 becomes

$$\eta' = M + N \cos\delta \qquad 6.A1.19$$

and Eq. 6.A1.13 becomes

$$\eta = M + N \qquad 6.A1.20$$

Thus η' and η are related by the expression

$$\frac{\eta'}{\eta} = \frac{\frac{M}{N} + \cos\delta}{\frac{M}{N} + 1} \qquad 6.A1.21$$

Eq. 6.A1.21 is represented graphically in Fig. 6.A1.2.

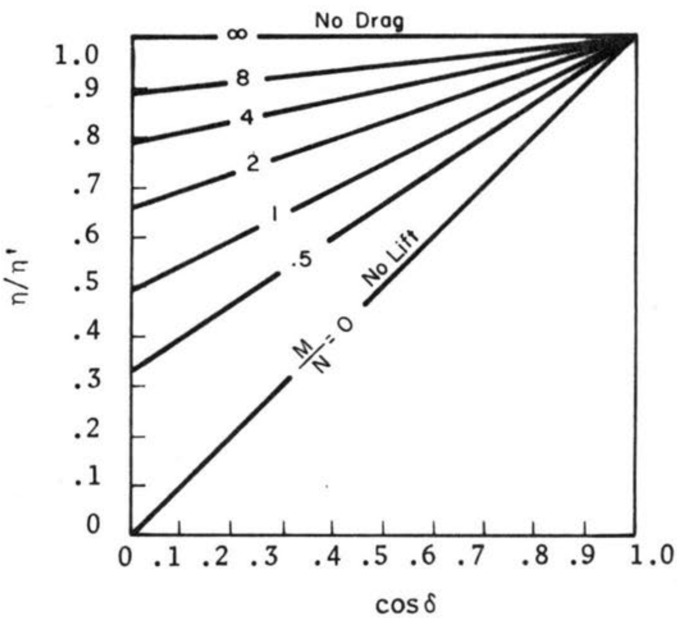


Fig. 6.A1.2 Ratio of stability factors

The problem is to select the proper value of the ratio M/N so that the stability factor on a side slope η' can be related to the stability factor on a plane horizontal bed, η, which in *turn is* related to the Shields' parameter. The assumption that the drag force F_d is zero means M/N is infinite, β is zero and $\eta' = \eta$. The assumption of zero lift force F_ℓ means M/N is zero and $\eta'/\eta = \cos\delta$.

In considering incipient motion of riprap particles, the ratios F_ℓ/F_d and e_4/e_3 vary depending on the turbulent conditions of the flow and the interlocking arrangement of the rock particles. In referring to Fig. 6.A1.1c assume that

$$\frac{e_4}{e_3} \simeq 2 \qquad 6.A1.22$$

then a choice for F_ℓ/F_d is

$$\frac{F_\ell}{F_d} \simeq \frac{1}{2} \qquad 6.A1.23$$

so that

$$\frac{M}{N} = \frac{e_4}{e_3}\,\frac{F_\ell}{F_d} \simeq 1. \qquad 6.A1.24$$

With $M/N = 1$, Eq. 6.A1.21 becomes

$$\frac{\eta'}{\eta} = \frac{1 + \cos\delta}{2} \qquad 6.A1.25$$

or by using Eq. 6.A1.8

$$\frac{\eta'}{\eta} = \frac{1 + \sin(\lambda+\beta)}{2} \qquad 6.A1.26$$

In Eq. 6.A1.12, the term $e_1W_s/e_3/F_d$ can be written

$$\frac{e_1W_s}{e_3F_d} = \frac{e_2W_s}{e_3F_d}\,\frac{e_1}{e_2}$$

$$= \frac{1}{N}\,\frac{1}{\tan\phi} \qquad 6.A1.27$$

according to Eqs. 6.A1.18 and 6.A1.4. For $M/N = 1$, Eq. 6.A1.20 becomes

$$N = \frac{\eta}{2} \qquad 6.A1.28$$

If we substitute Eqs. 6.A1.27 and 6.A1.28 into Eq. 6.A1.12, the expression for β becomes

$$\tan\beta = \frac{\cos\lambda}{\frac{2\sin\theta}{\eta\tan\phi} + \sin\lambda} \qquad 6.A1.29$$

In summary, the safety 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:

$$S.F. = \frac{\cos\theta \tan\phi}{\eta'\tan\phi + \sin\theta \cos\beta} \qquad 6.A1.30$$

in which

$$\beta = \tan^{-1}\left\{\frac{\cos\lambda}{\frac{2\sin\theta}{\eta\tan\phi} + \sin\lambda}\right\} \qquad 6.A1.31$$

$$\eta = \frac{21\ \tau_o}{(S_s - 1)\gamma D} \qquad 6.A1.32$$

and

$$\eta' = \eta\ \left\{\frac{1 + \sin(\lambda+\beta)}{2}\right\} \qquad 6.A1.33$$

Given a rock size D 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 (Eqs. 6.A1.30, 6.A1.31, 6.A1.32, and 6.A1.33) can be solved to obtain the safety factor S.F. *If S.F. is greater than unity, the riprap is safe from failure; if S.F. is unity, the rock is at the condition of incipient motion; if S.F. is less than unity, the riprap will fail.*

6.A1.2 <u>Horizontal flow on a side slope</u>

In many circumstances, the flow angularity with the horizontal is small; i.e., $\lambda \simeq 0$. Then Eqs. 6.A1.31 and 6.A1.32 reduce to

$$\beta = \tan^{-1} \{\frac{\eta \tan\phi}{2 \sin\theta}\} \qquad 6.A1.34$$

and

$$\eta' = \eta \{\frac{1 + \sin\beta}{2}\} \qquad 6.A1.35$$

When Eqs. 6.A1.34 and 6.A1.35 are substituted into Eq. 6.A1.30, the expression for the safety factor for horizontal flow on a side slope is

$$S.F. = \frac{S_m}{2} \{\sqrt{\xi^2 + 4} - \xi\} \qquad 6.A1.36$$

in which

$$\xi = S_m \eta \sec\theta \qquad 6.A1.37$$

and

$$S_m = \frac{\tan\phi}{\tan\theta} \qquad 6.A1.38$$

If we solve Eqs. 6.A1.36 and 6.A1.37 for η, then

$$\eta = \{\frac{S_m^2 - S.F.^2}{S.F.\ S_m^2}\} \cos\theta \qquad 6.A1.39$$

The term S_m is the safety factor for riprap on a side slope with no flow. Unless the flow is up the slope, the safety factor for the riprap cannot be greater than S_m.

6.A1.3 Flow on a plane sloping bed

Flow over a plane bed at a slope of α degrees in the downstream direction is equivalent to oblique flow on a side slope with $\theta = \alpha$, and $\lambda = 90°$.

Then, according to Eq. 6.A1.31, $\beta = 0°$ and from Eq. 6.A1.33.

$$\eta' = \eta \{\frac{1 + \sin(90° + 0°)}{2}\} = \eta \qquad 6.A1.40$$

It follows from Eq. 6.A1.30 that

$$S.F. = \frac{\cos\alpha \tan\phi}{\eta\tan\phi + \sin\alpha} \qquad 6.A1.41$$

for flow on a plane bed sloping α degrees to the horizontal. Alternatively solving for η in Eq. 6.A1.41

$$\eta = \cos\alpha \left\{\frac{1}{S.F.} - \frac{\tan\alpha}{\tan\phi}\right\} \qquad 6.A1.42$$

6.A1.4 Flow on a horizontal bed

For fully developed rough turbulent flow over a plane horizontal bed ($\alpha = 0$) of rock riprap, Eq. 6.A1.41 reduces to

$$S.F. = \frac{1}{\eta} \qquad 6.A1.43$$

If the riprap particles are at the condition of incipient motion, S.F. = 1 so $\eta = 1$ and from Eq. 6.A1.32

$$\frac{\tau_o}{(S_s - 1)\gamma D} = 0.047 \qquad 6.A1.44$$

which is Shields' criteria for the initiation of motion.

6.A2.0 THE REPRESENTATIVE GRAIN SIZE FOR RIPRAP

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 for riprap is larger than the median rock size, D_{50}.

In studies of scour below culvert outlets, Stevens (1968) obtained the following expression for the representative grain size of well-graded materials:

$$D = \left\{\frac{\sum_{i=1}^{10} D_i^3}{10}\right\}^{1/3} \qquad 6.A2.1$$

where

$$D_i(i=1) = \frac{D_0 + D_{10}}{2}$$

$$D_i(i=2) = \frac{D_{10} + D_{20}}{2}$$

$$\vdots$$

$$D_i(i=10) = \frac{D_{90} + D_{100}}{2}$$

The terms D_0, D_{10}, ..., D_{100} are the sieve diameters of the riprap for which 0 percent, 10 percent, ..., 100 percent of the material is finer by weight.

Eq. 6.A2.1 is equivalent to taking the arithmetic average of the sum of the weights of the individual rock sizes. In Stevens' studies (1968), the ratio of the representative size to the median size D/D_{50} was varied between 1.005 and 2.25.

6.A2.1 The recommended representative grain size for riprap

In Fig. 6.4.3, the recommended gradation for riprap is illustrated in terms of D_{50}. The computations of the representative grain size D for the recommended gradation is given in Table 6.A2.1.

Table 6.A2.1 Data for suggested gradation

Percent finer	Sieve Dia.	i	D_i
0	0.25 D_{50}	--	--
10	0.35 D_{50}	1	0.28 D_{50}
20	0.5 D_{50}	2	0.43 D_{50}
30	0.65 D_{50}	3	0.57 D_{50}
40	0.8 D_{50}	4	0.72 D_{50}
50	1.0 D_{50}	5	0.90 D_{50}
60	1.2 D_{50}	6	1.10 D_{50}
70	1.6 D_{50}	8	1.50 D_{50}
90	1.8 D_{50}	9	1.70 D_{50}
100	2.0 D_{50}	10	1.90 D_{50}

The rock sizes in the last column in Table 6.A2.1 are used in Eq. 6.A2.1 to find the representative grain size, D. The computed D is

$$D = \left\{\frac{\sum_{i=1}^{10} D_i^3}{10}\right\}^{1/3}$$

$$= 1.25\ D_{50}$$

This effective grain size of the mixture corresponds to the size D_{65} of the riprap.

6.A2.2 Comparison of representative grain sizes

In scour investigations, Stevens (1968) compared the scour produced *in two widely* different gradations of riprap having the same median diameter, D_{50} = 1.2 in. The riprap gradations are shown in Fig. 6.A2.1 as curves 1-2 and 3-4.

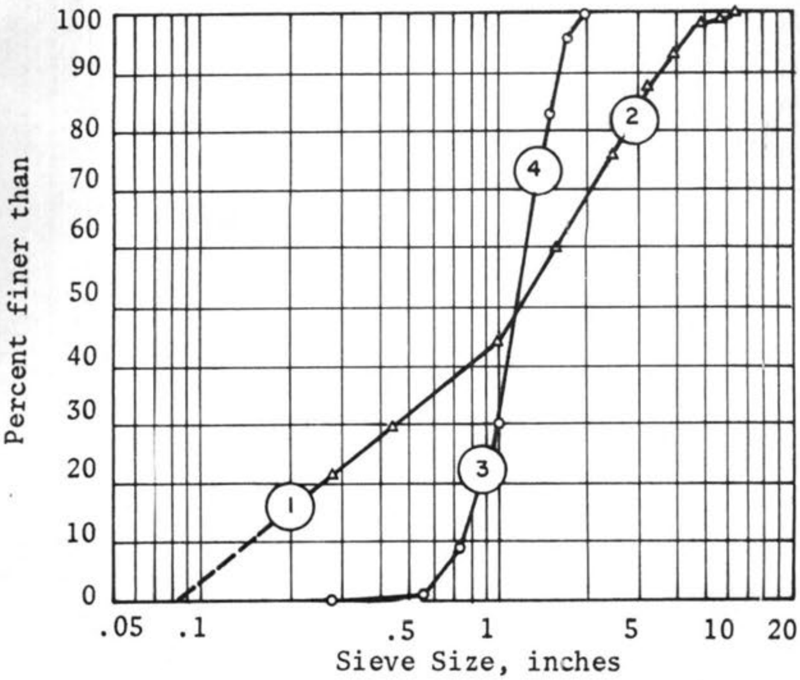


Fig. 6.A2.1 Gradation curves

The sieve sizes of four segments of the two gradation curves (segments 1, 2, 3 and 4) are listed in Table 6.A2.2.

Table 6.A2.2 Data for selected gradations

Percent Finer	Curve 1 Dia. in.	Curve 2 Dia. in.	Curve 3 Dia. in.	Curve 4 Dia. in.
0	0.08	--	0.28	--
10	0.15	--	0.75	--
20	0.26	--	0.88	--
30	0.46	--	1.00	--
40	0.80	--	1.10	--
50	1.20	1.20	1.20	1.20
60	--	1.60	--	1.28
70	--	2.15	--	1.35
80	--	2.80	--	1.45
90	--	3.70	--	1.60
100	--	6.60	--	2.00

From these data, the representative grain sizes for the four gradations are computed using Eq. 6.A2.1. The representative grain size D and the ratio of representative diameter to median diameter D/D_{50} are given in Table 6.A2.3 for the four gradations (refer to Fig. 6.A2.1).

Table 6.A2.3 Representative diameter for various gradations

Curve	D in.	D/D_{50}
1-2	2.70	2.25
3-2	2.71	2.26
1-4	1.21	1.01
2-4	1.27	1.06

Inspection of the values in Table 6.A2.3 shows that the larger sizes in the gradation have a dominant effect in the determination of the representative grain size. Thus, the large sizes of a gradation are the important sizes for stability. The fines should not be neglected, however, since lack of fines necessitates using a filter under the riprap.

If a riprap gradation has a wide range of sizes, the thickness of the riprap layer must be large enough to permit the loss of some fines (armorplating) without uncovering the protected materials (filter or bank material). The recommended thickness for the recommended gradation (Fig. 6.4.3) is $2D_{50}$ which is also D_{100}. For gradations with larger gradation coefficients the thickness must be at least the D_{100} size. For large gradation coefficients ($G > 3.0$), the thickness can be increased to $1.5D_{100}$ to provide enough material for armorplating.

6.A2.3 Effective size for log-normal gradations

In section 3.A1.3, an expression was given for the mean size of the bed material based on the assumption that the bed material has a log-normal distribution. The cumulative distribution function of the log-normal distribution can be expressed as

$$F_X(x) = \int_0^x \frac{1}{\sigma_y x(2\pi)^{1/2}} \exp\{-1/2(\frac{\ln x - \mu_y}{\sigma_y})^2\}dx \qquad 6.A2.2$$

where exp means raising the base of the natural log e to the power $(-1/2((\ln x-\mu_y)/\sigma_y)^2)$.

where X = particle diameter

Y = a normally (Gaussian) distributed variable

= $\ln X$

μ_y = mean of Y

σ_y = standard deviation of Y

The median value of X can be written

$$X_{med} = D_{50} = \exp(\mu_y)$$

Thus

$$\mu_y = \ln D_{50} \qquad 6.A2.3$$

Similarly

$$\sigma_y = \ln G \qquad 6.A2.4$$

where G is the gradation coefficient. It follows that the expression for the mean size of bed material is

$$D_m = D_{50} \exp\{\frac{1}{2} (\ln G)^2\} \qquad 6.A2.5$$

Mahmood[1] has found that the distribution of other properties of the bed material such as the projected area of the particle, the volume of the particle, or the surface area of the particle can also be represented by a log-normal distribution. The volume of a particle properties can be written

$$V = bX^3 \qquad 6.A2.6$$

where V = the volume of the particle

b = constant

X = particle diameter

[1] Mahmood, Khalid, 1973, Lognormal size distribution of particulate matter: Jour. of Sedimentary Petrology, vol. 43, no. 4, December, pp. 1161-1166.

Assuming a constant specific weight of the sediment, the weight of the particles can be represented by Eq. 6.A2.6.

The weight of a bed-material particle is important to the stability of the particle. Thus, it is more meaningful to compute the representative particle size based on the weight of the particle than on its diameter. From the expressions given by Mahmood, the representative size of the bed material based on the weight of the particles is given by

$$D = D_{50} \exp\{\frac{3}{2} (\ln G)^2\} \qquad 6.A2.7$$

The ratio of the mean weight size D to the mean diameter size D_m is given by

$$\frac{D}{D_m} = \frac{\exp\{\frac{3}{2} (\ln G)^2\}}{\exp\{\frac{1}{2} (\ln G)^2\}} \qquad 6.A2.8$$

which is always greater than one for nonuniform grain size distributions. For uniform distributions, there is only one particle size and the means are identical.

6.A3.0 RELATION BETWEEN VELOCITY AND SHEAR

In order to design riprap, it is necessary to be able 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 relation for the local velocity v at a distance y above the bed is given by Eq. 2.3.14 (with $y' = \frac{D}{30.2}$ for the hydraulically rough boundary) or

$$v = 2.5\, V_* \,\ell n\, (30.2\, \frac{y}{D_c}) \qquad 6.A3.1$$

in which V_* is the shear velocity which is

$$V_* = (\frac{\tau_o}{\rho})^{1/2} \qquad 6.A3.2$$

by definition. This velocity distribution equation was employed by Einstein (1950) in his bed-load function research.

If we select the velocity at a distance $y = D$ above the bed as the reference velocity v_r, then

$$v_r = 2.5\, V_* \,\ell n\, 30.2$$

or

$$v_r = 8.5\, V_* \qquad 6.A3.3$$

From Eqs. 6.A3.2 and 6.A3.3, the relation between v_r and τ_o is

$$\rho v_r^2 = 72\, \tau_o \qquad 6.A3.4$$

This relation is strictly valid only for uniform flow in wide prismatic channels in which the flow is fully turbulent. For the purposes of riprap design, Eq. 6.A3.4 can be employed when the flow is accelerating, for example on the nose of a spur dike. 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 would be calculated by Eq. 6.A3.4.

By substituting Eq. 6.A3.4 into Eq. 6.4.5, the expression for the stability factor η becomes

$$\eta = \frac{0.30\ v_r^2}{(S_s-1)gD} \qquad 6.A3.5$$

The average velocity in the vertical V is obtained from Eq. 2.3.16 with $x = 1$ for the large riprap sizes. The expression can be written

$$V = 2.5\ V_* \ \ell n\ (12.3\ \frac{y_o}{D}) \qquad 6.A3.6$$

in which y_o is the depth of flow. The ratio of the reference velocity v_r to the depth-averaged velocity is

$$\frac{v_r}{V} = \frac{2.5\ V_*\ \ell n\ (30.2)}{2.5\ V_*\ \ell n\ (12.3\ \frac{y_o}{D})}$$

or

$$\frac{v_r}{V} = \frac{3.4}{\ell n(12.3\ \frac{y_o}{D})} \qquad 6.A3.7$$

Eq. 6.A3.7 is obtained from Eqs. 6.A3.3 and 6.A3.6. Now the expression for the stability factor η can be written in terms of the depth-averaged velocity. From Eqs. 6.A3.5 and 6.A3.7

$$\eta = \frac{\beta V^2}{(S_s-1)gD} \qquad 6.A3.8$$

in which

$$\beta = 0.30\ \{\frac{3.4}{\ell n(12.3\ \frac{y_o}{D})}\}^2 \qquad 6.A3.9$$

As shown in Fig. 6.A3.1 the value of β varies from 0.30 for relatively shallow flows to a value of 0.04 for $y_o/D = 1000$.

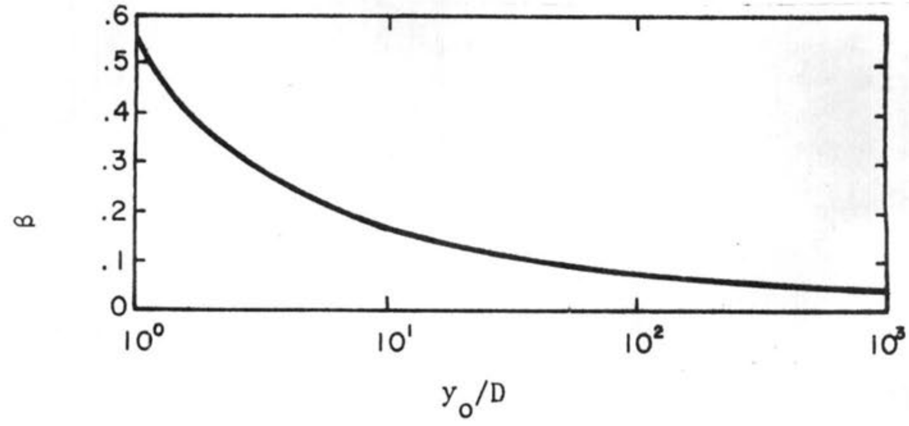


Fig. 6.A3.1 Relation between β and y_o/D.

The Bureau of Public Roads,[1] Hydraulic Engineering Circular No. 11 uses the expression

$$\frac{v_s}{\overline{V}} = \frac{1}{0.958 \log(\frac{y_o}{D}) + 1} \qquad 6.A3.10$$

to determine the velocity "against the stone". Here v_s is the velocity against the stone, $\overline{V}$ is the mean velocity in the channel and log is the logarithm to the base 10.

In wide channels, the depth-averaged velocity and the mean velocity in the channel are nearly equal; i.e., $V \approx \overline{V}$. Then the velocity against the stone is related to the reference velocity by the expression

$$\frac{v_r}{v_s} = \frac{v_r}{V}\frac{V}{v_s}$$

or

$$\frac{v_r}{v_s} = \frac{3.4\{.958 \log(\frac{y_o}{D}) + 1\}}{\ln(12.3\,\frac{y_o}{D})} \qquad 6.A3.11$$

[1] Searcy, J. K., Use of riprap for bank protection, Hydraulic Engineering Circular No. 11, Hydraulics Branch, Bridge Division, Office of Engineering and Operations, Bureau of Public Roads, Washington, D.C., June, 1967.

according to Eqs. 6.A3.7 and 6.A3.10. For values of y_o/D between $1x10^0$ and $1x10^6$, the value of the v_r/v_s is nearly 1.4. By letting

$$\frac{v_r}{v_s} = 1.4 \qquad 6.A3.12$$

the expression for the stability factor η (Eq. 6.A3.5) becomes

$$\eta = \frac{0.60\ v_s^2}{(S_s - 1)gD} \qquad 6.A3.13$$

In summary then the following expressions for η are equivalent:

$$\eta = \frac{21\ \tau_o}{(S_s - 1)\gamma D} \qquad 6.4.5$$

$$\eta = \frac{0.30\ v_r^2}{(S_s - 1)gD} \qquad 6.A3.5$$

$$\eta = \frac{0.60\ v_s^2}{(S_s - 1)gD} \qquad 6.A3.13$$

and $$\eta = \frac{\beta V^2}{(S_s - 1)gD} \qquad 6.A3.8$$

in which

$$\beta = 0.30\ \left\{\frac{3.4}{\ell n(12.3\ \frac{y_o}{D})}\right\}^2 \qquad 6.A3.9$$

6.A4.0 RIPRAP DESIGN ON AN EMBANKMENT

When the drawdown through a bridge opening is large there is an appreciable downslope component to the velocity vector on the nose of an embankment end or spur dike. This downslope component of velocity is illustrated in the model embankment shown in Fig. 6.A4.1. The model study was reported by Simons and Lewis[1].

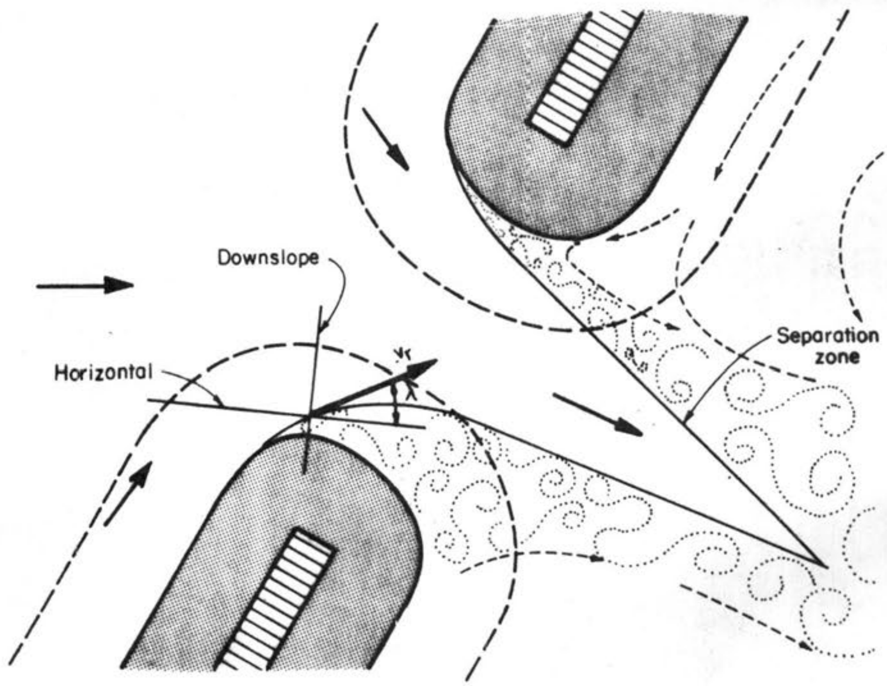


Fig. 6.A4.1 Flow around an embankment end

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[1] Simons, D. B. and Lewis, G. L., "Flood protection at bridge crossings," CER71-72DBS-GLL10, Colorado State University, Fort Collins, Colorado, prepared for the Wyoming State Highway Department Planning and Research Division, 1971.

For the case shown in Fig. 6.A4.1, 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 by Lewis[1]. If the drawdown through the bridge is large, the reference velocity μ_r in the vicinity of the riprap can be as large as 6 fps.

Suppose that the reference velocity is

$$\mu_r = 6 \text{ fps}$$

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 $D = 1.0$ ft, the safety factor is determined in the following manner.

From Eq. 6.A3.5

$$\eta = \frac{0.30\ v_r^2}{(S_s - 1)gD} = \frac{(0.30(6)^2}{(2.65 - 1)(32.2)(1.0)} = 0.203$$

This dumped rock has an angle of repose of approximately 35° according to Fig. 3.7.3. Therefore, from Eq. 6.4.6

$$\beta = \tan^{-1} \left\{\frac{\cos\lambda}{\frac{2 \sin\theta}{\eta \tan\phi} + \sin\lambda}\right\} = \tan^{-1}\left\{\frac{\cos 20°}{\frac{2 \sin 18.4°}{0.203 \tan 35°} + \sin 20°}\right\} = 11°$$

---

[1]Lewis, G. L., Riprap protection of bridge footings, thesis presented in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Ft. Collins, Colo., 1972.

and from Eq. 6.4.4

$$\eta' = \eta\{\frac{1 + \sin(\lambda+\beta)}{2}\} = 0.203\{\frac{1 + \sin(20° + 11°)}{2}\} = 0.154$$

The safety factor for the rock is given by Eq. 6.4.3 or

$$S.F. = \frac{\cos\theta \tan\phi}{\eta'\tan\phi + \sin\theta \cos\beta} = \frac{\cos 18.4° \tan 35°}{0.154 \tan 35° + \sin 18.4° \cos 11°} = 1.59$$

Thus, with a safety 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 Fig. 6.A4.2 is obtained. This

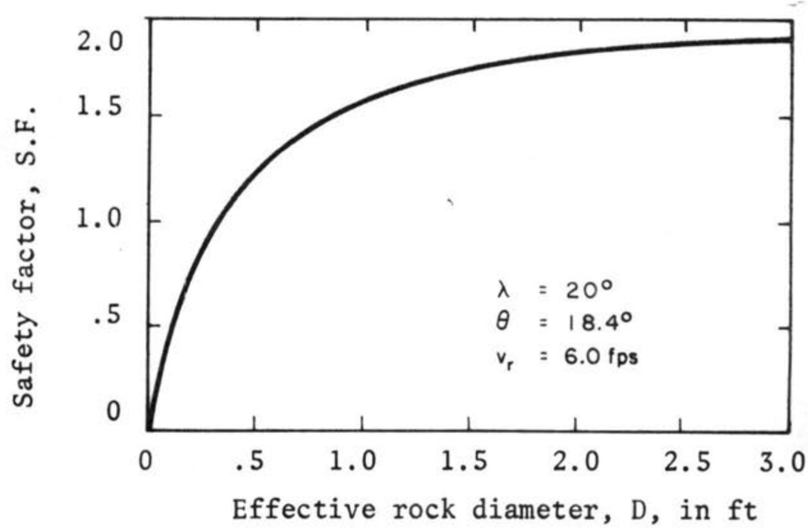

Fig. 6.A4.2 Safety factors for various rock sizes on a side slope

curve shows that the incipient motion rock size is approximately 0.35 ft and that the maximum safety factor is less than 2.0 on the 3:1 side slope.

The safety 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 Eq. 6.4.14 is applicable and

$$S.F. = \frac{1}{\eta} = \frac{1}{0.203} = 4.93$$

The curve in Fig. 6.A4.3 relates the safety factor and side slope angle of the embankment (for $\lambda = 20°$, $D = 1.0$ ft and $v_r = 6.0$ fps). The curve is obtained by employing Eqs. 6.4.6, 6.4.4 and 6.4.3 for various values of θ.

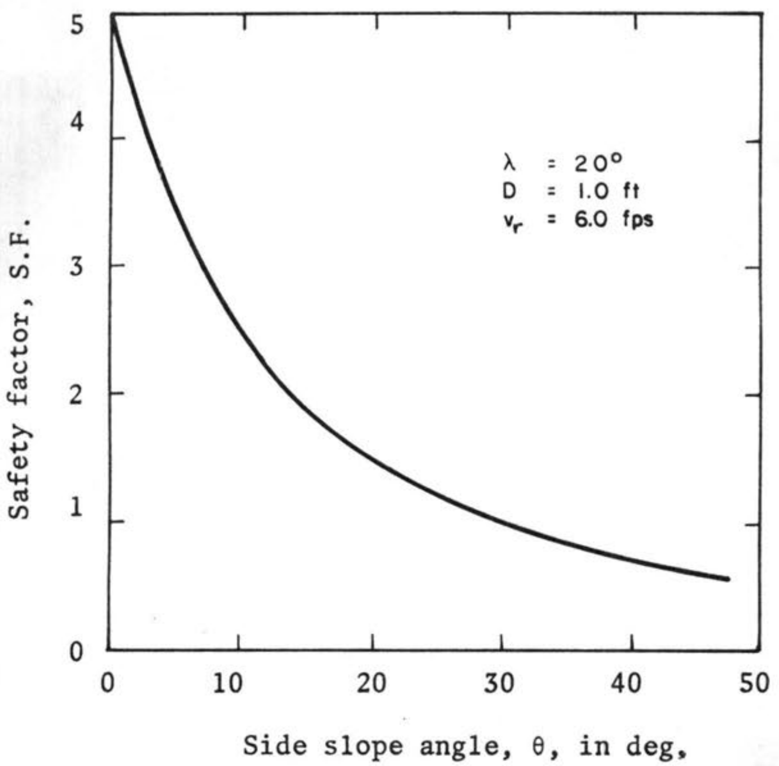

Fig. 6.A4.3 Safety factors for various side slopes

6.A5.0 DESIGN AID FOR SIDE SLOPE RIPRAP

When the velocity along a side slope has a downslope component, the sizing of the riprap is accomplished by employing the complex equations given in Section 6.4.1. An example of sizing riprap using these equations is given in Appendix Section 6.A4.0. When the velocity along a side slope has no downslope component (i.e., the velocity vector is along the horizontal), some simple design aids can be developed.

For horizontal flow along a side slope, the equations relating the safety factor, the stability number, the side slope angle, and the angle of repose for the rock are (from Section 6.4.2)

$$\eta = \left\{\frac{S_m^2 - (S.F.)^2}{(S.F.)\ S_m^2}\right\} \cos\theta \qquad 6.A5.1$$

and

$$S_m = \frac{\tan\phi}{\tan\theta} \qquad 6.A5.2$$

The interrelation of the variables in these two equations is represented in Fig. 6.A5.1. Here, the specific weight of the rock is taken as 2.65 and a safety factor of 1.5 is employed.

The recommended safety factor for the design of riprap is

$$S.F. = 1.5$$

This recommendation is the result of studies of the riprap embankment model data obtained by Lewis. These studies were reported by Simons and Lewis[1].

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[1]Simons, D. B. and Lewis, G. L., Flood protection at bridge crossings, CER71-72DBS-GLL10, Colorado State University, Fort Collins, Colorado, report prepared for the Wyoming State Highway Department, Planning and Research Division, 1971.

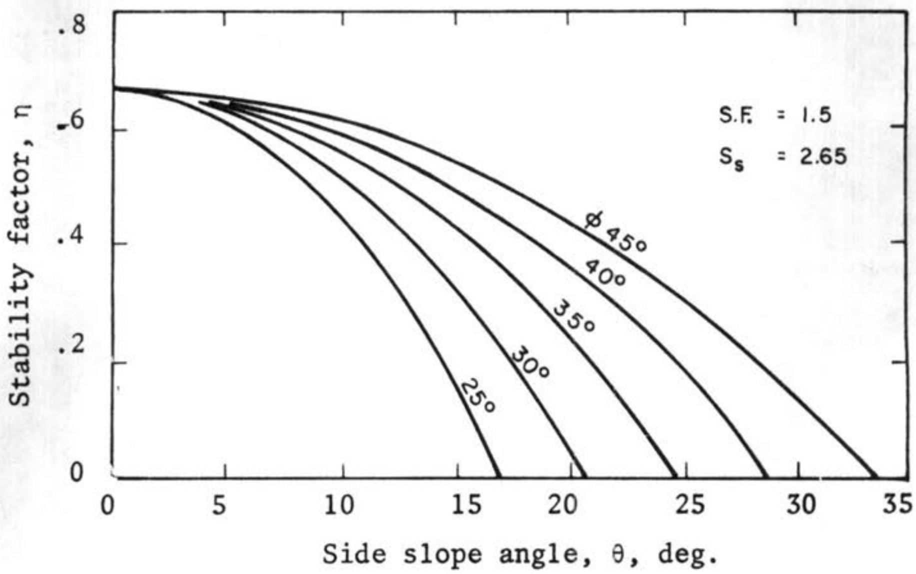

Fig. 6.A5.1 Stability factors for a 1.5 safety factor for horizontal flow along a side slope

The curves in Fig. 6.A5.1 are computed in the following manner.

(1) Select an angle of repose ϕ. For example

$$\phi = 45°$$

(2) Select a side slope angle. For example

$$\theta = 25°$$

(3) Compute S_m from Eq. 6.A5.2

$$S_m = \frac{\tan 45°}{\tan 25°} = 2.14$$

(4) Compute η from Eq. 6.A5.1 with S.F. = 1.5

$$\eta = \{\frac{(2.14)^2 - (1.5)^2}{(1.5)(2.14)^2}\} \cos 25° = 0.29$$

(5) Repeat the above steps for the full range of interest for ϕ and θ.

In a design problem, the side slope angle is known and the angle of repose can be estimated (See Fig. 3.7.3). With these values the stability factor is obtained from Fig. 6.A5.1. For example, if

$$\phi = 30°$$

and

$$\theta = 10°$$

then

$$\eta = 0.52$$

If the shear stress on the side slope is known, then (from Eq. 6.4.5)

$$D = \frac{21\ \tau_o}{(S_s - 1)\gamma\eta} \qquad 6.A5.3$$

$$= \frac{21\ \tau_o}{(2.65 - 1)(62.4)\eta}$$

$$= 0.20\ \frac{\tau_o}{\eta}$$

In our example if

$$\tau_o = 2 \text{ psf}$$

then

$$D = \frac{(0.20)(2)}{(0.52)} = 0.77 \text{ ft.}$$

If the reference velocity on the side slope is known, then (from Eq. 6.A3.5)

$$D = \frac{0.30\ v_r^2}{(S_s - 1)g\eta}$$

$$= \frac{(0.30\ v_r^2}{(2.65 - 1)(32.2)\eta}$$

$$= .0056\ \frac{v_r^2}{\eta}$$

In our example, if

$$v_r = 6 \text{ fps}$$

$$D = \frac{(.0056)(6)^2}{0.52} = 0.40 \text{ ft.}$$

If the average velocity V and the depth y_o are known, more design aids should be prepared from Eqs. 6.A3.8 and 6.A3.9.

6.A6.0 FILTER DESIGN

The requirements for a gravel filter are given in Section 6.4.7. The gradation of a filter should be such that

$$\frac{D_{50}(\text{Filter})}{D_{50}(\text{Base})} < 40 \qquad 6.A6.1$$

$$5 < \frac{D_{15}(\text{Filter})}{D_{15}(\text{Base})} < 40 \qquad 6.A6.2$$

and

$$\frac{D_{15}(\text{Filter})}{D_{85}(\text{Base})} < 5 \qquad 6.A6.3$$

6.A6.1 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 6.A6.1. *Design a filter to be placed between the riprap and the base material.*

Table 6.A6.1 Sizes of materials

Base Material Sand	Riprap Gravel
D_{85} = 1.50 mm	D_{85} = 24 mm
D_{50} = 0.75 mm	D_{50} = 12 mm
D_{15} = 0.38 mm	D_{15} = 6 mm

In accordance with the recommended sizes for filters, we note that

$$\frac{D_{50}(\text{Riprap})}{D_{50}(\text{Base})} = \frac{12}{0.75} = 16$$

which satisfies expression 6.A6.1. Also

$$\frac{D_{15}(\text{Riprap})}{D_{15}(\text{Base})} = \frac{6}{0.38} = 16$$

which satisfies the requirement 6.A6.2. Moreover

$$\frac{D_{15}\,(\text{Riprap})}{D_{85}\,(\text{Base})} = \frac{6}{1.5} = 4$$

which satisfies the requirement 6.A6.3. The riprap itself satisfies the requirements for the filter so no filter is needed.

6.A6.2 Filter design example 2

The following filter design is taken from Anderson et al.[1] The properties of the base material and the riprap are given in Table 6.A6.2.

Table 6.A6.2 Sizes of materials

Base Material Sand	Riprap Rock
D_{85} = 1.5 mm	D_{85} = 400 mm
D_{50} = 0.5 mm	D_{50} = 200 mm
D_{15} = 0.17 mm	D_{15} = 100 mm

The riprap does not contain sufficient fines to act as the filter because

$$\frac{D_{15}\,(\text{Riprap})}{D_{85}\,(\text{Base})} = \frac{100}{1.5} = 67$$

which is much greater than 5, the recommended upper limit (expression 6.A6.3). Also

$$\frac{D_{15}\,(\text{Riprap})}{D_{15}\,(\text{Base})} = \frac{100}{0.17} = 600$$

which is much greater than 40, the recommended upper limit (expression 6.A6.2).

[1] Anderson, A.G., Paintal, A.S., and Davenport, J.T., Tentative Design Procedure For Riprap Lined Channels, Project Report No. 96, St. Anthony Falls Hydraulic Laboratory, University of Minnesota, Minneapolis, Minnesota, June, 1968.

The properties of the filter to be placed adjacent to the base are as follows:

(1) $$\frac{D_{50}(\text{Filter})}{D_{50}(\text{Base})} < 40$$

so $$D_{50}(\text{Filter}) < (40)(0.5) = 20 \text{ mm}$$

(2) $$\frac{D_{15}(\text{Filter})}{D_{15}(\text{Base})} < 40$$

so $$D_{15}(\text{Filter}) < (40)(0.17) = 6.8 \text{ mm}$$

(3) $$\frac{D_{15}(\text{Filter})}{D_{85}(\text{Base})} < 5$$

so $$D_{15}(\text{Filter}) < (5)(1.5) = 7.5 \text{ mm}$$

(4) $$\frac{D_{15}(\text{Filter})}{D_{15}(\text{Base})} > 5$$

so $$D_{15}(\text{Filter}) > (5)(.17) = 0.85 \text{ mm}$$

Thus, with respect to the base

$$0.85 \text{ mm} < D_{15}(\text{Filter}) < 6.8 \text{ mm}$$

and $$D_{50}(\text{Filter}) < 20 \text{ mm}$$

The properties of the filter to be placed adjacent to the riprap are as follows:

(1) $$\frac{D_{50}(\text{Riprap})}{D_{50}(\text{Filter})} < 40$$

so $$D_{50}(\text{Filter}) > \frac{200}{40} = 5 \text{ mm}$$

(2) $$\frac{D_{15}(\text{Riprap})}{D_{15}(\text{Filter})} > 5$$

so $$D_{15}(\text{Filter}) < \frac{100}{5} = 20 \text{ mm}$$

(3) $$\frac{D_{15}(\text{Riprap})}{D_{15}(\text{Filter})} < 40$$

so $$D_{15}(\text{Filter}) > \frac{100}{40} = 2.5 \text{ mm}$$

(4) $$\frac{D_{15}(\text{Riprap})}{D_{85}(\text{Filter})} < 5$$

so $$D_{85}(\text{Filter}) > \frac{100}{5} = 20 \text{ mm}$$

Therefore, with respect to the riprap, the filter must satisfy these requirements

$$2.5 \text{ mm} < D_{15}(\text{Filter}) < 20 \text{ mm}$$

$$D_{50}(\text{Filter}) > 5 \text{ mm}$$

$$D_{85}(\text{Filter}) > 20 \text{ mm}$$

These riprap filter requirements along with those for the base material are shown in Fig. 6.A6.1. Any filter having sizes represented by the double cross-hatched area is satisfactory. For example, a good filter could have these sizes:

$$D_{85} = 40 \text{ mm}$$

$$D_{50} = 10 \text{ mm}$$

$$D_{15} = 4 \text{ mm}$$

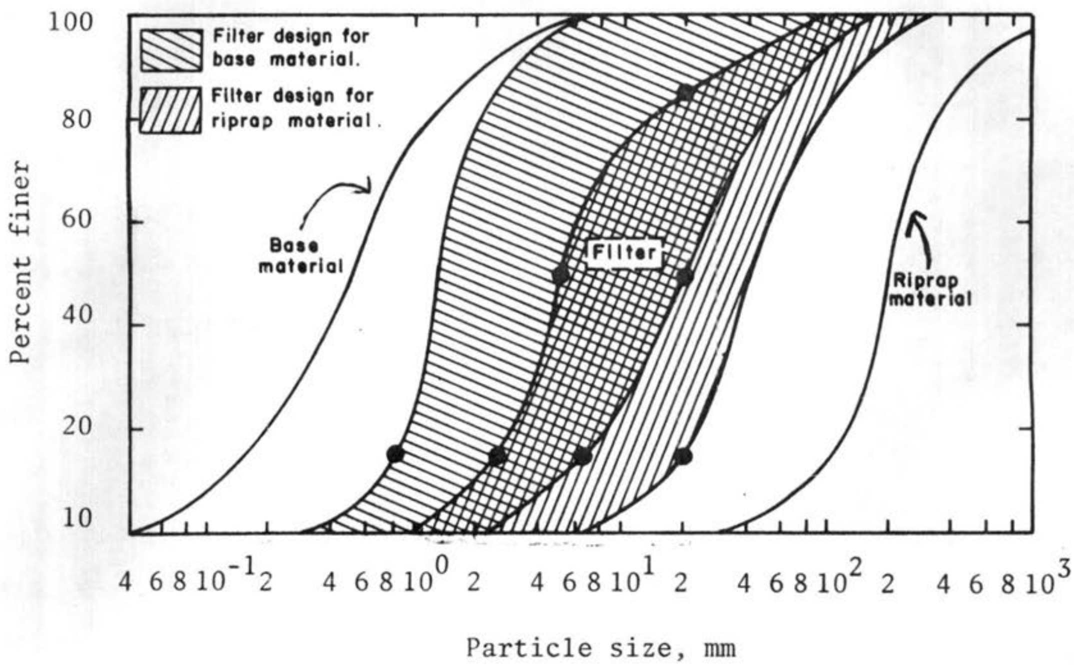

Fig. 6.A2.1 Gradations of filter blanket for example 2 (after Anderson et al., 1968)

6.A7.0 STRAUB'S EQUATION FOR CLEAR-WATER SCOUR

Consider the long contraction shown in Fig. 6.A7.1. 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

$$Q = V_1 y_1 W_1 \qquad 6.A7.1$$

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

$$Q = V_2 y_2 W_2 \qquad 6.A7.2$$

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.

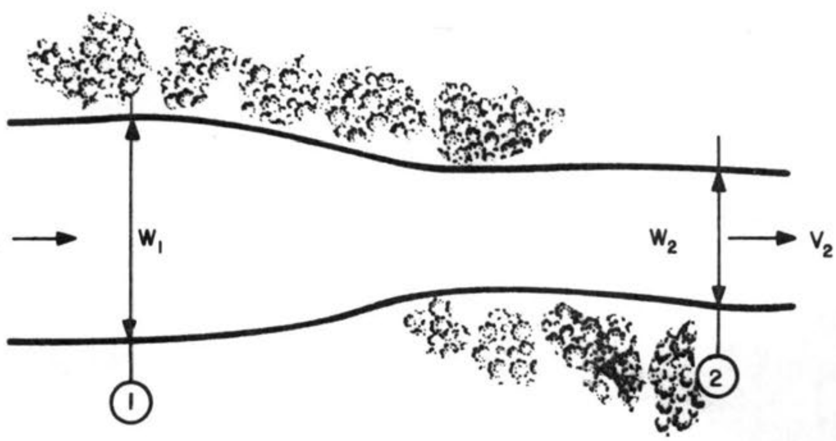

Fig. 6.A7.1 Plan view of the long contraction

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,

$$\tau_1 = \gamma y_1 S_{f_1} < \tau_c \qquad 6.A7.3$$

Here S_{f_1} 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

$$\tau_2 = \tau_c = \gamma y_2 S_{f_2} \qquad 6.A7.4$$

When this condition is reached there is no longer 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 Eqs. 6.A7.3 and 6.A7.4, the depth ratio is

$$\frac{y_2}{y_1} = \frac{S_{f_1}}{S_{f_2}} \frac{\tau_c}{\tau} \qquad 6.A7.5$$

Manning's equation (Eq. 2.3.20) can be employed to determine the friction slope ratio. Accordingly

$$\frac{S_{f_1}}{S_{f_2}} = \left(\frac{n_1}{n_2}\right)^2 \left(\frac{V_1}{V_2}\right)^2 \left(\frac{y_2}{y_1}\right)^{4/3} \qquad 6.A7.6$$

so

$$\frac{y_2}{y_1} = \left(\frac{n_1}{n_2}\right)^2 \left(\frac{V_1}{V_2}\right)^2 \left(\frac{y_2}{y_1}\right)^{4/3} \left(\frac{\tau_c}{\tau}\right) \qquad 6.A7.7$$

The velocity ratio V_1/V_2 is obtained by equating Eqs. 6.A7.1 and 6.A7.2 (constant discharge) or

$$\frac{V_1}{V_2} = \frac{y_2}{y_1} \frac{W_2}{W_2} \qquad 6.A7.8$$

By putting this ratio into Eq. 6.A7.7, the expression

$$\frac{y_2}{y_1} = \left(\frac{n_2}{n_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{\tau}{\tau_c}\right)^{3/7} \qquad 6.A7.9$$

is obtained for clear-water scour. If it is assumed that

$$n_1 = n_2$$

then Eq. 6.A7.9 reduces to

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{6/7} \left(\frac{\tau}{\tau_c}\right)^{3/7} \qquad 6.A7.10$$

which is the form of the clear-water scour equation first developed by Straub[1].

[1] Straub, L.G., 1940, Approaches to the study of mechanics of bed movement, Iowa State Univ. Studies in Engin. Bul. 20.

6.A8.0 DETERMINATION OF FLOW PARAMETERS ON AN EMBANKMENT

To apply the equations presented in Section 6.4 for the design of riprap bank protection, the flow parameters τ_o , v_r , v_s or V , and λ must be evaluated. Lewis[1], developed an analytical model for determination of the complete velocity and depth distribution in the vicinity of embankments 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 occurs 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 embankment toe through the constriction. Riprap losses initially occurred on the upstream spill-slope for small constrictions ($\Delta h/L$ is small) and along the embankment toe for the severe constrictions ($\Delta h/L$ is large) as shown in Fig. 6.A8.1. Here Δh is the drop in water surface elevation through the bridge opening and L is the horizontal distance shown in Fig. 6.A8.2 with ω = 45 deg.

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 embankment. The flow parameters for determining the required riprap at midway around the upstream spill-slope and along the embankment toe through the constriction are determined below. The sizes of riprap required to produce a safety factor of 1.5 at those two positions are computed and the larger size rock is chosen for the embankment protection.

6.A8.1 Determination of flow parameters at the embankment toe

The flow at the toe of the embankment is approximately horizontal and parallel to the side slope (see Fig. 6.A8.2). For design purposes

[1] Lewis, G. L., "Riprap protection of bridge footings," thesis presented in partial fulfillment of the requirements for the degree of Doctor of Philosophy, Colorado State University, Fort Collins, Colorado, 1972.

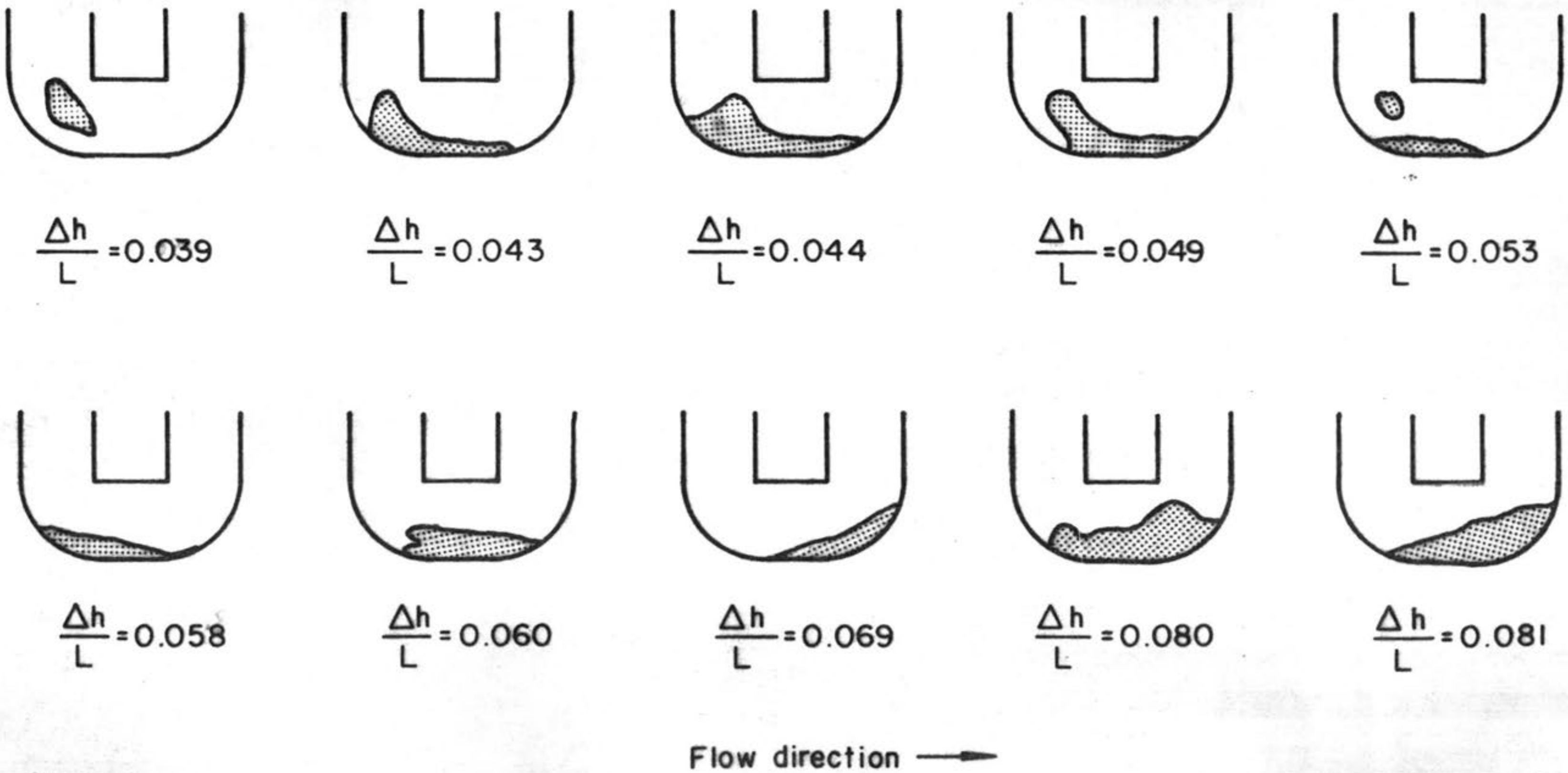

Fig. 6.A8.1 Zones of failure on riprapped embankments

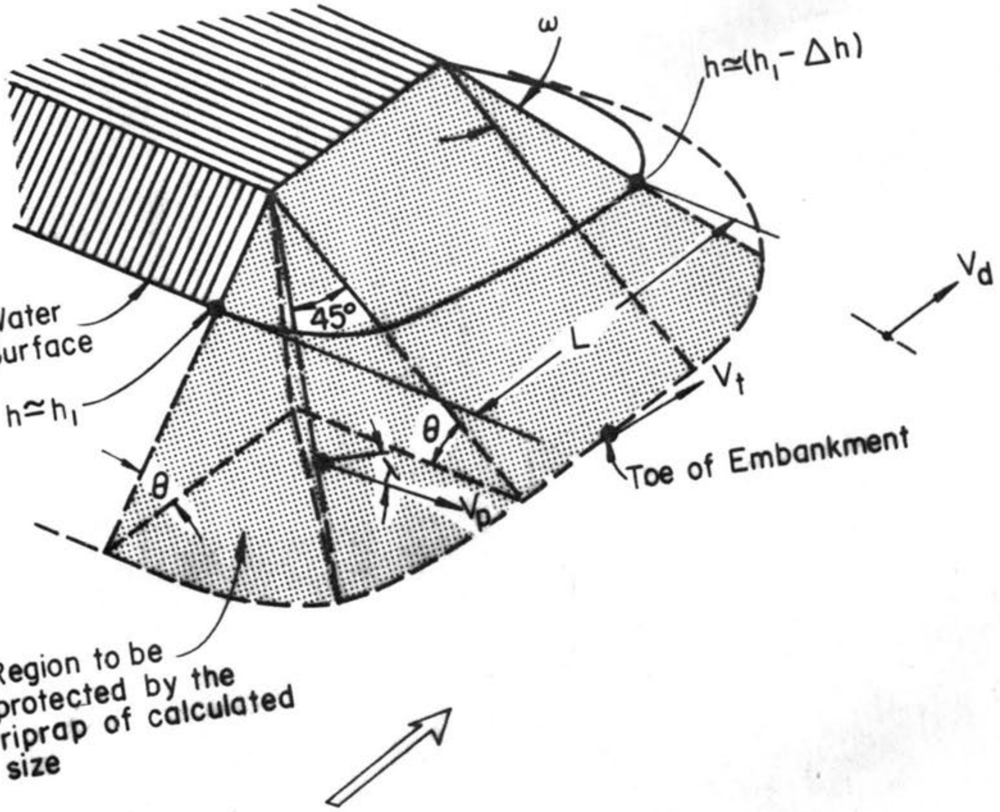

Fig. 6.A8.2 Flow around a spill-through embankment nose

it is assumed that $\lambda = 0$ deg. 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 Fig. 6.A8.3. The quantity V_d is found by using the expression

$$V_d = (\alpha_1 V_1^2 + 2g\Delta h)^{1/2} \qquad 6.A8.1$$

where α 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 by the Bureau of Public Roads[1]. The quantity L in Fig. 6.A8.2 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 velocity is then found from Fig. 6.A8.3.

6.A8.2 Determination of flow parameters midway around the upstream spill-slope

The depth-averaged velocity midway around the upstream spill-slope V_p (see Fig. 6.A8.2) is related to the vena contracta velocity V_d and can be determined for given values of V_p and $\Delta h/L$ by using the relation in Fig. 6.A8.3. The angle between the velocity vector and the horizontal line, λ, is given in Fig. 6.A8.4.

6.A8.3 Application and limitations

After the values of V_t, V_p and λ are estimated from Figs. 6.A8.3 and 6.A8.4, the riprap sizes for a given safety factor are determined at the toe and midway around the upstream spill-slope by employing Eqs. 6.A3.8, 6.A3.9, 6.4.3, 6.4.4, and 6.4.6. The quantity

[1] Bureau of Public Roads, "Hydraulics of bridge waterways," Hydraulics Branch, Bridge Division, Office of Engineering and Operations, reported by J. N. Bradley, Consultant, 2nd edition, Hydraulic Design Series No. 1, U. S. Government Printing Office, Washington, D. C., 1970.

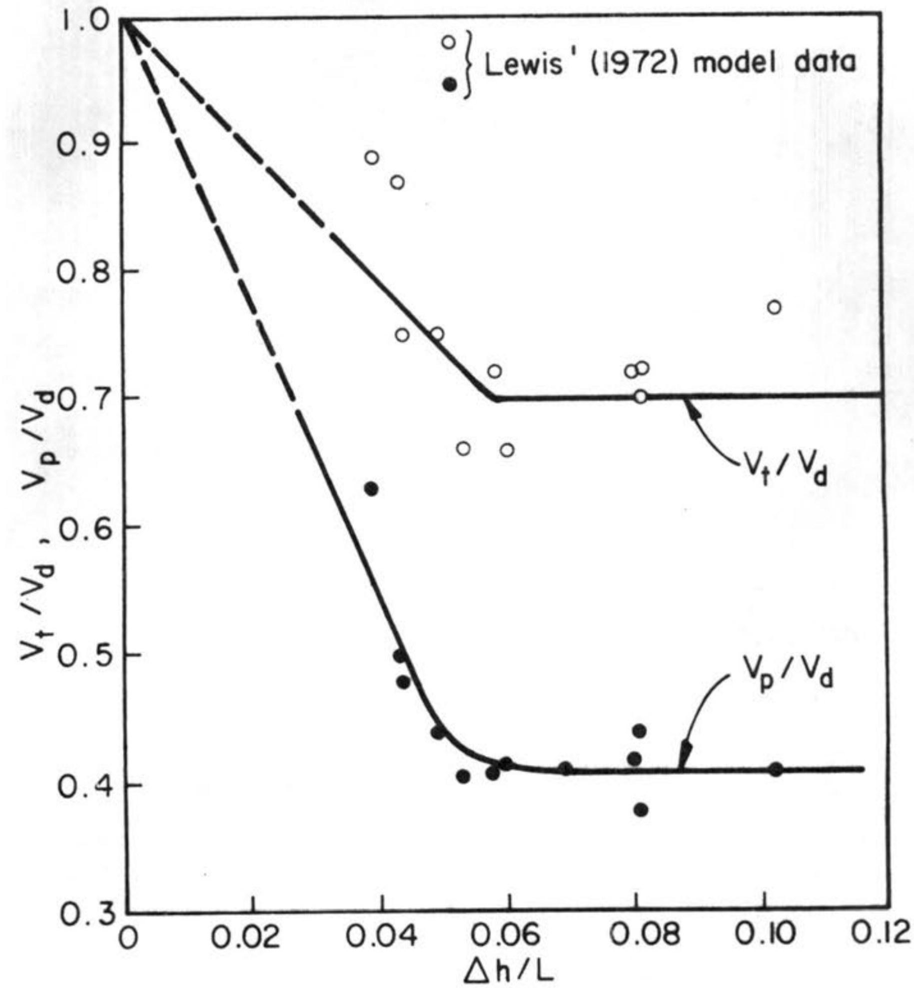

Fig. 6.A8.3 Relation between relative velocities and the drop ratio

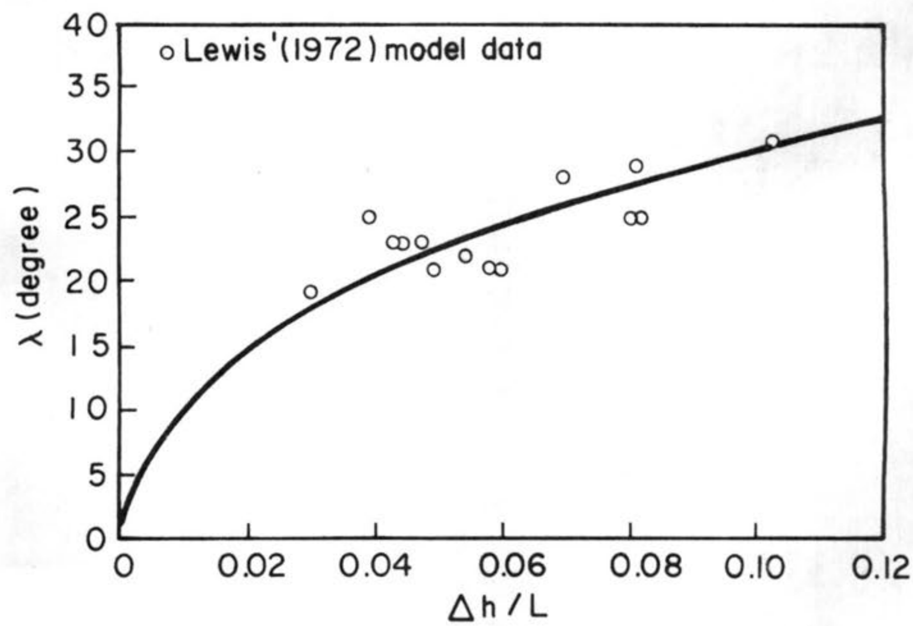

Fig. 6.A8.4 Relation between λ at midway around the upstream spill-slope and the drop ratio

y_o in Eq. 6.A3.9 is the local flow depth. The larger of the two calculated riprap sizes should be used for the embankment protection.

The design procedure given above has been applied to Lewis' model data. It was found that the procedure resulted in a built-in safety factor by approximately 1.2. However, the larger safety factor of S.F. = 1.5 is recommended for embankments to allow for error in the hydraulic predictions and for unknown scale effects which may exist in the models.

The region around the embankment nose requiring the calculated size of riprap protection is shown in Fig. 6.A8.2. The angle ω varies according to the relation given in Table 6.A8.1. The other region of the embankment may be protected by a smaller size of riprap than the calculated one. A method for computing this size is given by Simons and Lewis[1].

Table 6.A8.1 Termination angle for riprap protection

$\frac{\Delta h}{L}$	ω degree
0 to .05	45
.06	55
.07	65
.08	75
.09	85
.10	90

Figs. 6.A8.3 and 6.A8.4 are developed from the model data on spill-through embankments normal to the flow and require modifications when applied to other types of embankments.

[1] Simons, D. B. and Lewis, G. L., "Flood protection at bridge crossings," CER71-72DBS-GLL10, Colorado State University, Fort Collins, Colorado, prepared for the Wyoming State Highway Department Planning and Research Division, 1971.

Chapter VII

NEEDS AND SOURCES FOR DATA

7.1.0 DATA NEEDS

The objective and purpose in 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.

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.

7.2.0 BASIC DATA NEEDS

7.2.1 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, and cities, mountain ranges, railroads and other highways and roads. The area map should be large enough to identify river systems and tributaries.

7.2.2 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 reach 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 coarse 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 and picnic grounds, bathing 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 indicated.

7.2.3 Site maps

Site maps are needed to determine details for hydraulic 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 alignment 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.

7.2.4 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 vegetation, sizes and heights of sandbars, river thalwegs, river controls and geologic formations, existing bank protection works, old meander channels, and other features. Detailed contours can 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 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 pertaining data should be noted and briefly discussed. High water marks are useful to record photographically along with dates of occurrence. *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.

7.2.5 Hydrologic data

The purpose of hydrologic data is to determine the stream discharge, flood magnitudes, duration and frequencies of floods preparatory 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 the purpose of preparing construction plans and evaluating performance 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.

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.

Sometimes a highway crossing and/or encroachment may have a significant effect on flood hydrographs. Sufficient hydrologic analysis should be made to determine the significance. This would involve hydrograph development and valley routings within the zone of influence of such highway structures.

7.2.6 Geologic map

A geologic vicinity map, on which geophysical features are indicated is of basic need. *The basic rock formations, outcroppings, and glacial* *and river deposits which form control points on rivers are valuable in analysis of rivers.* Soil type has important effects on sediment transport material, 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 U.S. They may be helpful in selecting layouts and assessing the suitability of fill materials.

7.2.7 Field inspection

A field inspection of potential highway encroachment sites of rivers should be made prior to or during 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 personal inspection by the entire design staff.

7.2.8 Environmental data

In making environmental impact analysis of highway projects on streams and rivers, it is necessary to obtain water quality and biological data for the streams. Such data are available for many rivers but are not readily available for many others. 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 fish and fish habitat in the river 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 embankment would be helpful in this regard.

7.3.0 AUXILIARY DATA

The basic data needed for hydrologic, hydraulic and environmental analyses have been indicated. In addition, depending upon the nature of the highway project, it may be desirable to obtain additional data.

7.3.1 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.

7.3.2 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 would be helpful in sediment transport and backwater studies. It would also be helpful to know water temperatures. Direct measurement of flood flows should be made when 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.

7.4.0 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 data. Approximate methods for extrapolating the range of flood-frequency curves are available but are not discussed in detail here. (See Guidelines for Hydrology (1973) for references).

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 others) paper. The recurrence interval, RI is calculated from

$$RI = \frac{n+1}{m} \qquad 7.4.1$$

in which n is the number of years of record, 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 (1967) 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 the gaging station to the project site, the flood peaks are 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 are also 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. An excellent reference relating n to channel conditions is presented pictorially in USGS Water Supply Paper 1849 which is published in book form. 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.

7.5.0 CHECKLIST OF DATA NEEDS

As an aid to check data needs preparatory to analysis of rivers and highway encroachment of rivers, the relevant types of data have been listed in Table 7.5.1. There may be more data items included in this table than are needed for a given project site, and some judgment is required in delineation. For data which are not available, the checklist should be helpful for planning a field investigation or other data acquisition program.

7.6.0 DATA SOURCES

The best data sources are national data centers where the principle function is to disseminate data. But it probably will be necessary to collect data from a variety of other sources such as a field investigation, interviews with local residents, and search through library materials. 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) Plats 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 plats--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; National Oceanic and Atmospheric Administration; and mapping firms.
(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.

Transportation Maps:

(1) State Highway Agency.

Triangulation and Benchmarks:

(1) State Engineer
(2) State Highway Agency.

Geologic Maps:

(1) U.S. Department of the Interior, Geologic Survey, Geologic Division; and State geological surveys or departments. (Note--some regular quadrangle maps show geological data also).

Soils 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.

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) Owners of Reclamation.
(7) Corp of Engineers, U.S. Army, Flood control 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 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.

Table 7.5.1 Checklist of data needs

Description of data or needed information	Basic Data	Auxiliary Data	Check Whether Available	Needed
Maps and charts:				
Geographic	*			
Topographic	*			
Geologic	*			
Navigation charts		*		
Potamology surveys		*		
County and city plats		*		
Aerial and other photos:				
Large scale photos for working plans	*			
Small scale stereo pairs of river and surrounding terrain	*			
Color infrared photos for flow patterns, scour zones, and vegetation		*		
Ground photos	*			
Underwater photos		*		
Information on existing structures bridges, dams, diversions or outfalls:				
Plans and details	*			
Construction details		*		
Alterations and repairs		*		
Foundations	*			
Piers and abutments	*			
Scour		*		
Dikes	*			
Field investigations	*			
Investigating bridge structure & repairs to bridge & approach	*			
Damage due to ice or debris	*			

Table 7.5.1 Checklist of data needs (continued)

Description of data or needed information	Basic Data	Auxiliary Data	Check Whether Available	Needed
Hydraulic, Hydrology and Soils:				
Discharge records	*			
Stage-discharge records	*			
Flood frequency curves for stations near site	*			
Flow duration curves (hydrographs)		*		
Newspaper, radio, television, accounts of large floods		*		
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)	*			
Sediment discharge	*			
Size distribution	*			
Bed and bank material sizes	*			
Roughness coefficient n	*			
Ice:				
Recorded thickness		*		
Dates of freeze up and break up		*		
Flow patterns and jams		*		

Table 7.5.1 Checklist of data needs (continued)

Description of data or needed information	Basic Data	Auxiliary Data	Check Whether Available	Needed
Damage		*		
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 ground-water levels in areas		*		
Install gaging stations		*		
Soils Information:	*			
Excavation data	*			
Borrow pits	*			
Gravel pits	*			
Cuts	*			
Tunnels	*			
Core boring logs	*			
Well drilling logs		*		
Soil tests	*			
Permeability		*		
Rocks - riprap	*			
Planned and anticipated water resources projects	*			
Lakes, tributaries, reservoirs or side channel impoundments	*			
Field surveys:	*			
Onsite inspections and photographs	*			

Table 7.5.1 Checklist of data needs (continued)

Description of data or needed information	Basic Data	Auxiliary Data	Check Whether Available Needed
Sample 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:			
Natural weather service records for precipitation	*		
Wind	*		
Temperatures	*		
Land Use:			
Zoning maps	*		
Recent Aerial Photographs	*		
Planning Committee Records	*		
Urban areas	*		
Industrial areas	*		
Recreational areas	*		
Primitive areas	*		
Forests	*		
Vegetation	*		

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Chapter VIII

HYDRAULIC AND ENVIRONMENTAL CONSIDERATIONS OF HIGHWAY RIVER CROSSINGS AND ENCROACHMENTS

8.1.0 INTRODUCTION

The objective of this chapter is to *present application of the basic principles of hydraulics, hydrology, fluvial geomorphology and river mechanics to the hydraulic and environmental design of river crossings and highway encroachments.* In general, the design of complex problems in river engineering can be facilitated by a qualitative estimation followed by a quantitative analysis. For this reason, this chapter includes thirteen hypothetical cases of river environments and their response to river crossings and encroachments based on the geomorphological principles given in Chapter IV. These cases indicate the trend of change in river morphology for given initial conditions. The case studies are followed by twelve case histories of actual river crossings in the United States. These histories document river response to highway crossings and encroachments and serve the basic purpose of illustrating qualitative river response.

Finally, this chapter presents the main design considerations related to river crossings and highway encroachment. The application of basic principles developed in Chapters I through VI is illustrated by specific numerical examples related to the subject matter of this manual. It is believed that *the systematic approach of qualitative assessment of channel response followed by quantitative estimation, will enable a meaningful analysis of complex river response problems.*

8.2.0 CASE HISTORIES

8.2.1 Introduction

To initiate analysis of case histories first consider some of the problems normally encountered in river crossings and encroachments. Several hypothetical cases are tabulated in Table 8.2.1 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, upstream effects, and downstream effects 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 8.2.1. *The relation* $QS \sim Q_s D_{50}$ *is valuable in determining qualitative river response. Having identified the qualitative response that can be anticipated, water and sediment routing techniques coupled with river mechanics relations* given in Chapters IV and V, *can be used to predict the possibility of change in river form and to estimate the magnitudes of local, upstream and downstream river response.* This approach should be kept in view as one considers each of the fourteen cases outlined in Table 8.2.1.

The initial river conditions in Table 8.2.1 are sometimes given in terms of storage dams, water diversions, etc. 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 from rivers 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 stretch, often occur due to river development projects upstream from the proposed crossings* or due to natural causes.

Case (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. *The change in gradient of the tributary stream in most cases 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, even if the water continues to flow under the bridge, the waterway is significantly reduced thus endangering the usefulness and stability of the structure. In general, streams on alluvial fans shift laterally so that the future direction of the approach flow to the bridge is uncertain.

Table 8.2.1 River response to highway encroachments and to river development

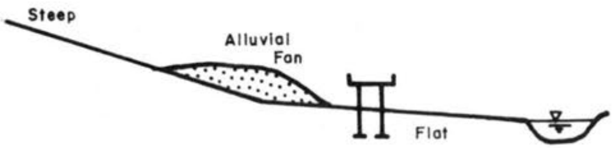
Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 (1) *Crossing downstream of an alluvial fan.*	1 - Fan reduces waterway 2 - Direction of flow at bridge site is uncertain 3 - Channel location is uncertain	1 - Erosion of banks 2 - Unstable channel 3 - Large transport rate	1 - Aggradation 2 - Flooding 3 - Development of tributary bar in the main channel
 (2) *Lowering of base level for the channel.*	1 - Headcutting 2 - General scour 3 - Local scour 4 - Bank instability 5 - High velocities	1 - Increased 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

Table 8.2.1 River response to highway encroachments and to river development (continued)

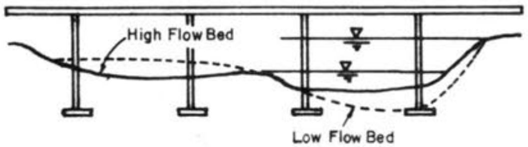
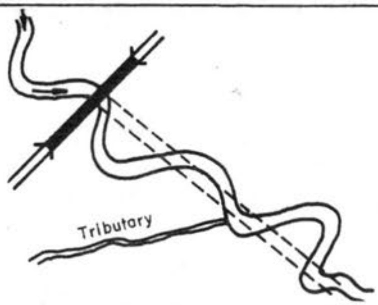
Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 (3) *Channel characterized by prolonged low flows.*	1 - At low flow a low water channel develops in river bed 2 - Increased danger to piers due to channelization and local scour 3 - Bank caving	---	---
 (4) *Cutoffs downstream of crossing.*	1 - Steeper slope 2 - Higher velocity 3 - Increased transport 4 - Degradation and possible head-cutting 5 - Banks unstable 6 - River may braid 7 - Danger to bridge foundation from degradation and local scour	See local effects	1 - Deposition downstream of straightened channel 2 - Increased flood stage 3 - Loss of channel capacity 4 - Degradation in tributary

Table 8.2.1 River response to highway encroachments and to river development (continued)

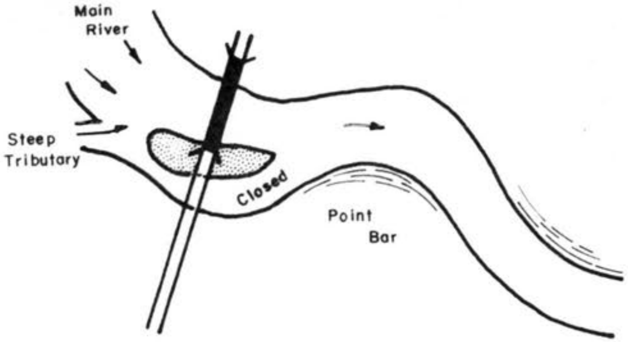
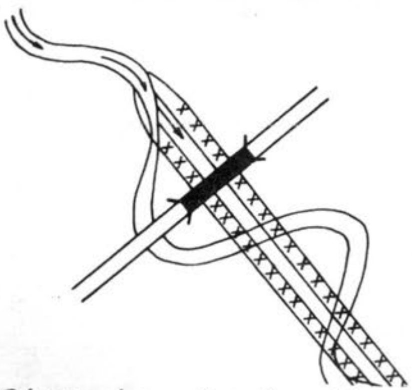
Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 (5) *Excess of sediment at bridge site due to upstream tributary*	1 - Contraction of the river 2 - Increased velocity 3 - General and local scour 4 - Bank instability	1 - Aggradation 2 - Backwater at flood stage 3 - Changed response of the tributary	1 - Deposition of excess sediment eroded at and down-stream of the bridge 2 - More severe attack at first bend downstream 3 - Possible development of a chute channel across the second point bar downsteam of the bridge
 (6) *River channel relocation at crossing site*	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).	1 - Similar to local effects	1 - Similar to local effects

Table 8.2.1 River response to highway encroachments and to river development (continued)

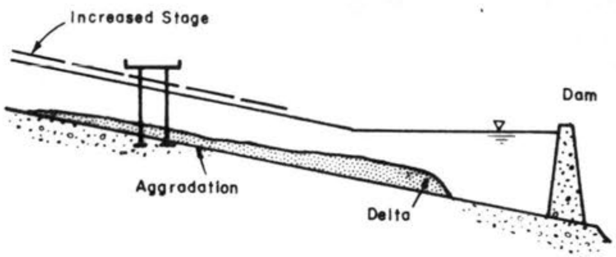
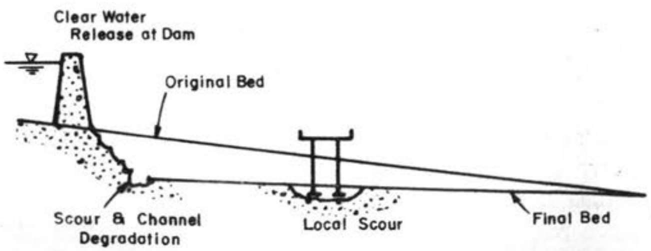
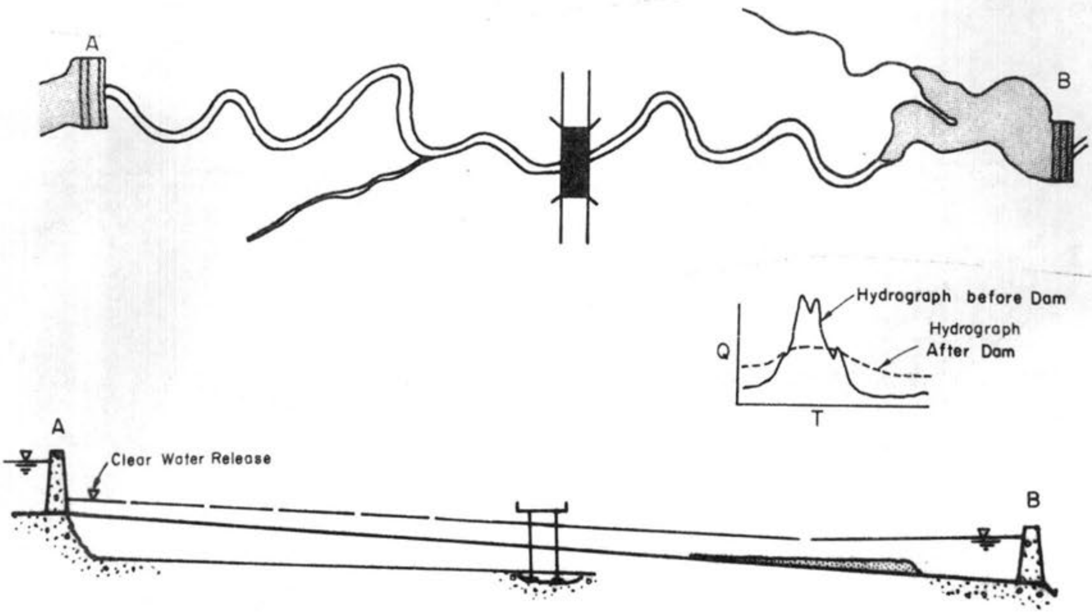
Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 (7) *Raising of river base level.*	1 - Aggradation of Bed 2 - Loss of waterway 3 - Change in river geometry 4 - Increased flood stage	1 - See local effects 2 - Change in base level for tributarics 3 - Deposition in tributaries near confluences 4 - Aggradation causing a perched river channel to develop or changing the alignment of the main channel	1 - See upstream effects
 (8) *Reduction of sediment load upstream*	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 8.2.1 River response to highway encroachments and to river development (continued)

Bridge location

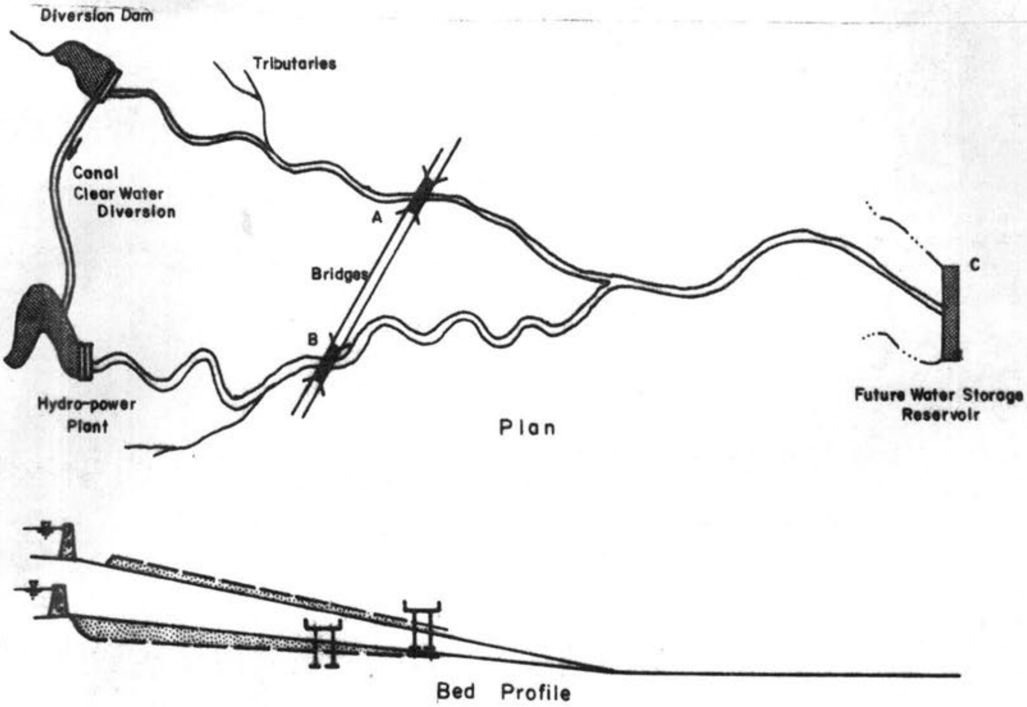


(9) *Combined increase of base level and reduction of sediment load upstream*

Local Effects	Upstream Effects	Downstream Effects
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 channel 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)	See upstream effects

Table 8.2.1 River response to highway encroachments and to river development (continued)

Bridge location



(10) *Change in water discharge, no change in sediment load*

Local Effects	Upstream Effects	Downstream Effects
1 - Bridge A may be subjected to aggradation due to excess sediment left in the channel by diversion of clear water and degradation in tributaries caused by lowering of their base level 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 8.2.1 River response to highway encroachments and to river development (continued)

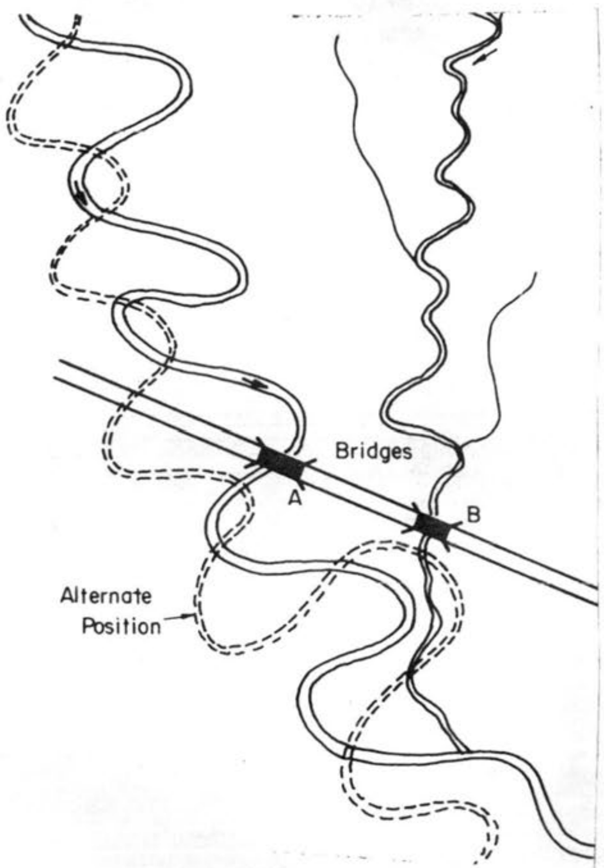
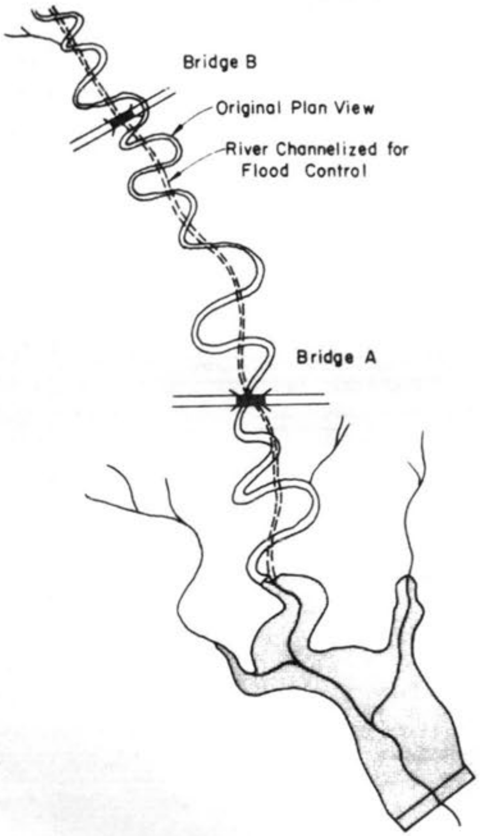
Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 (11) *Naturally shifting river channel*	1 - Rivers are dynamic (ever changing) and the rate of change with time should be evaluated as part of the geomorphic and hydraulic analysis 2 - Alignment of main channel continually changes affecting 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 abandon 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 and degradation. 3 - Shifts in the position of the main channel causes aggradation, degradation and instabilities depending upon direction and magnitude of channel change

Table 8.2.1 River response to highway encroachments and to river development (continued)



Bridge Location	Local Effects	Upstream Effects	Downstream Effects
(12) *Man-induced reduction of channel length*	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 levels are reduced at B and increased at A 6 - Local and general scour is significantly affected	1 - A change of river form from meandering to braiding 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 8.2.1 River response to highway encroachments and to river development (continued)

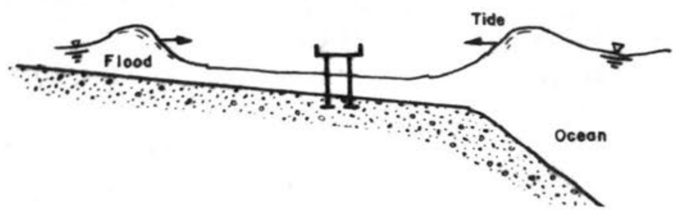
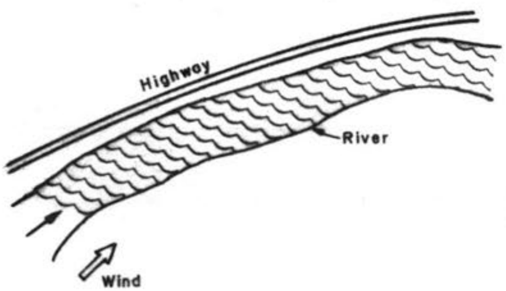
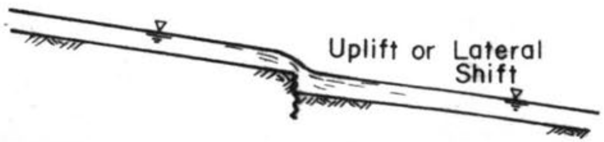
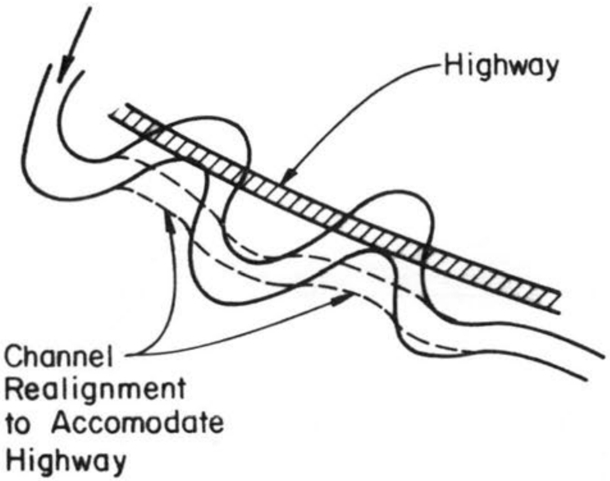
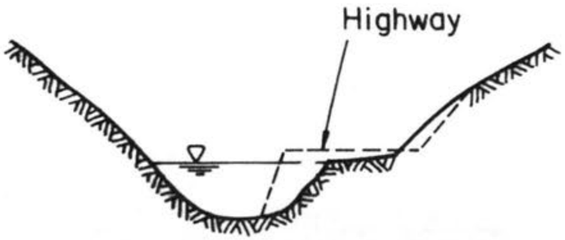
Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 a. - Tidal Flows, Seiches, Bores, etc.	1 - Scour or aggradation 2 - Bank erosion 3 - Channel change 4 - Bed form change	1 - See local effects 2 - Channel erosion 3 - Changes in shaping slope	1 - See local effects 2 - Beach erosion
 b. - Wind (Hurricanes, Tornadoes)	1 - Bank erosion 2 - Inundated highway 3 - Increase in velocity 4 - Wave action	1 - See local effects	1 - See local effects
 c. - Earthquakes (see Seismic Probability Map of U.S.)	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
(13) *Tectonics and other natural causes*			

Table 8.2.1 River response to highway encroachments and to river development (continued)

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 a. - Meandering Channel	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 erosion.	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.

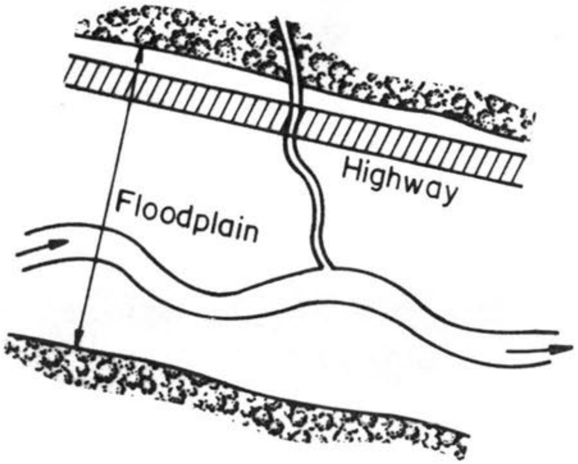
(14) *Longitudinal Encroachment*

Table 8.2.1 River response to highway encroachments and to river development (continued)

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 b. - Incised Channel	1 - Reduced waterway causes a local obstruction to flow and higher velocities. 2 - Significant 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

(14) *Longitudinal Encroachment* (continued)

Table 8.2.1 River response to highway encroachments and to river development (continued)

Bridge Location	Local Effects	Upstream Effects	Downstream Effects
 c. - Floodplain Encroachment	1 - Erosion of highway fill and submergence possible during floods 2 - Pattern of over-bank spill are affected by the encroachment and in highly shifting channels may channel river course downstream. 3 - Lateral drainage into the river is interrupted and may cause flooding and erosion.	1 - If significant encroachment on the floodplain waterway, back-water may be induced.	1 - If the river channel is highly shifting, the channel alignment may change. 2 - If significant erosion experi-enced upstream, aggradation will occur.

(14) *Longitudinal Encroachment* (continued)

Case (2) 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 imposed 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.

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 channel in the bed of 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 have been *several examples documented where bridges have been entirely safe in terms of local scour at high stage, but have failed or have partially 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. It is obvious that *straightening the channel* downstream of the crossing *significantly increases the channel slope. In general, 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 brings about a drop in base level and any tributary streams flowing into the affected reach of the main channel are subjected to conditions discussed in Case (2).

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. Such a procedure forces all of the water and sediment to pass through a reduced width. This *contraction of the river in general increases the local velocity, increases general and local scour, and may significantly increase bank instability.* In addition, the contraction can change the alignment of the flow in the vicinity of the bridge and thus would affect the downstream main channel for a considerable distance. A chute channel can develop across the second point bar downstream and its effect may extend several meander loops downstream. Upstream of the bridge there is aggradation and its amount depends on the magnitude of water and sediment being introduced from the tributary. Also, there is significant increase in the backwater upstream of the bridge at high flows which in turn affects other tributaries farther upstream of the crossing.

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 imposed 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 the bank stability is ensured. Such a channel should not undergo significant change over either short or long periods of time.

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. 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 necessary, in general, 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 amount of raise. 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. The deposition of sediment at the entrance to the pool induces aggradation in the channel upstream. This aggradation may extend many miles upstream after a long period of time. *If the bridge lies within the effects imposed by the new base level, the following effect at the crossing will be expected: a loss of waterway at the bridge site, significant changes in river geometry, and increased flood stages.* In the extreme it is possible that the river may become sufficiently perched that at some high flow it could abandon the old channel and adopt a new one.

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 base level of a river as in the development of storage by constructing a dam on a river provides a desilting 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. The existing river channel is the result of its interaction with normal water-sediment flows over a long period of time. *With the sediment-free flows the channel is too steep and sediments are entrained from the bed and the banks bringing about 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 plan 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) illustrates a more complicated set of circumstances. In this case 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 affects 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 possiblities to develop a safe design.

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. Ultimately, 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. Any 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. It is assumed that the alignment of the main channel is continually changing. The rate of change in the river system will have been 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 therein 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 as shown in Table 8.2.1. 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 rather difficult because of the ever 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 and measures 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.

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 in effect 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 the 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 but 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, bores, as well as wind waves, can rapidly and violently destroy existing bank lines. When considering earthquakes it is of interest to look at 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.

Case (14) illustrates three examples of longitudinal encroachment. In example (a), 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. 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 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.

Example (b) illustrates encroachment on the waterway of an incised stream flowing through a narrow gorge. This is just one of many possible reasons that the highway may need to encroach on the main channel. 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 locally occurs in the encroached reach.

Example (c) 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 occupied 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 occupies a significant portion of 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 direction and pattern of spill onto the floodplain and back into the channel. Very often this type of of encroachment has little or no effect on flood stages or on the stream upstream or downstream.

In all cases of longitudinal encroachment, the lateral drainage into the river will be intercepted. A main consideration in the design of encroachment will be to provide for this drainage.

8.2.2 Actual case histories of river encroachments

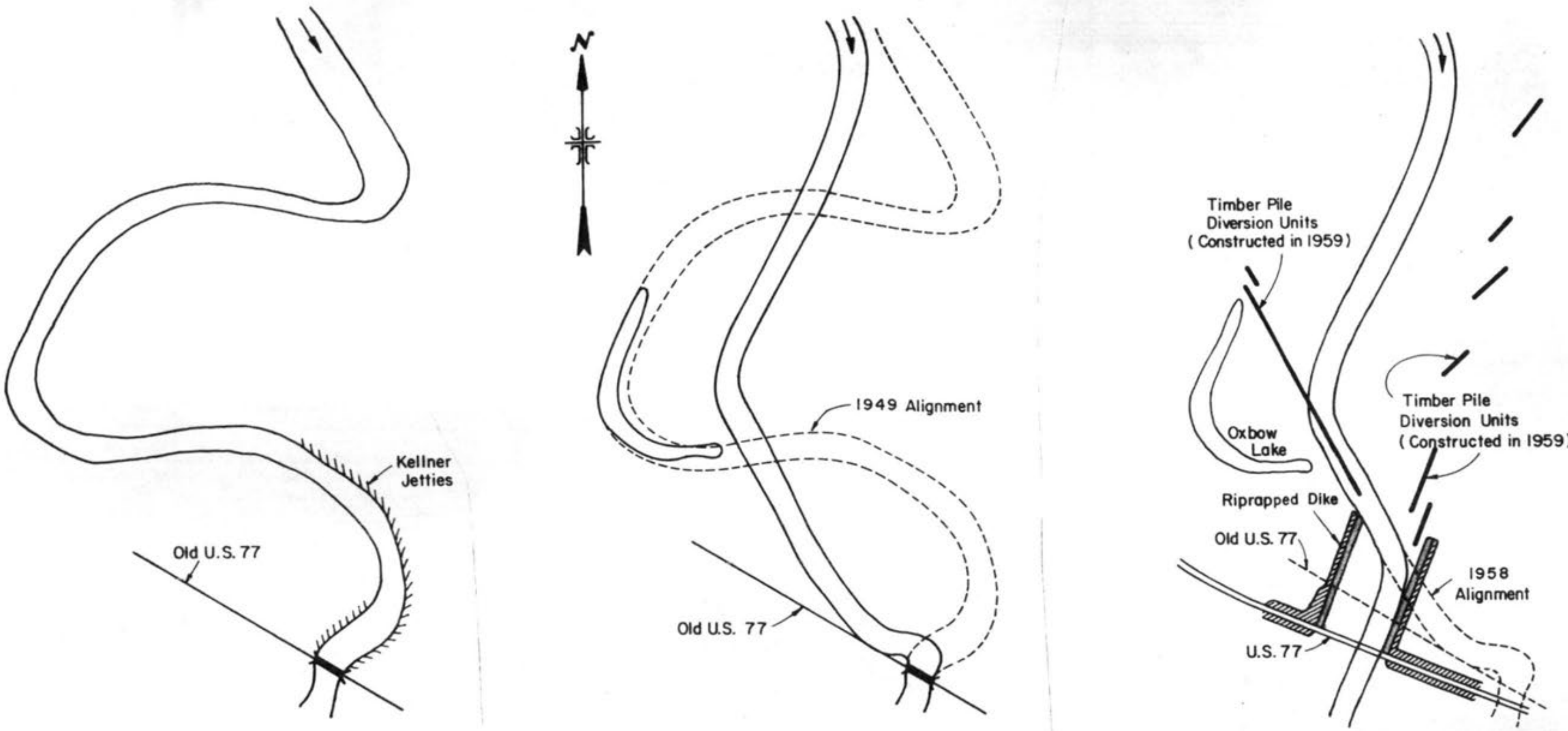
In the preceding paragraphs several possible cases were discussed that the engineer dealing with river encroachments might encounter. As a follow up some actual river encroachments are presented. Each case considers the interactions between the river and the encroachment over a period of time. In general these particular cases are not as complex as some of the foregoing hypothetical cases. For example there is no consideration of water resources development throughout the basins, including construction of reservoirs, transmountain diversions, and so forth.

Washita River, west of Wynnewood, Oklahoma

During a large flood in 1949, the concave bank immediately upstream of the old bridge began eroding at an excessive rate. After the subsidence of the high flows, Kellner jetties were installed as shown in Fig. 8.2.1a to prevent further erosion of the banks. The jetty field was successful. Then, in a 50 to 60 year return period flood in 1958, *the channel developed a new alignment by cutting across several meander loops* and washed out a section of the old U.S. 77, (see Fig. 8.2.1b). In 1959, a new bridge was built over the more stable river alignment (Fig. 8.2.1c) and *riprap dikes were constructed to protect the structure.* Six timber pile diversions were constructed on the east edge of the floodplain and have been dormant since. Two timber pile diversion structures were constructed on the southwest bank of the river upstream of the bridge to prevent a meander loop from developing and encroaching on the approach to the bridge. These structures proved to be ineffective as the river flows under the timber pile structures at all stages. Nevertheless, in 1968, the river was still being held in a proper channel location relative to the bridge.

Cimarron River, south of Perkins, Oklahoma

At some point in time, AT&SF Railroad constructed Kellner jetties on the south bank of the river to prevent it from encroaching on the



(a) Channel in 1949

(b) Channel in 1958

(c) Channel in 1968

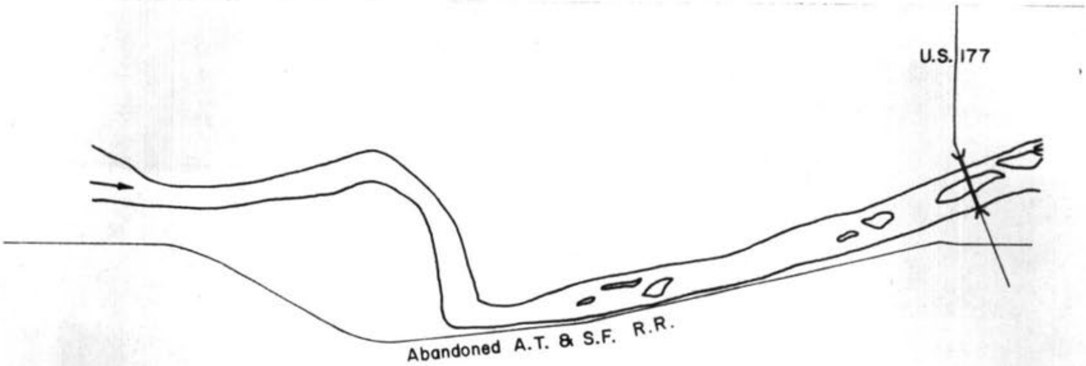
Fig. 8.2.1 Washita River, west of Wynnewood, Oklahoma road bed. Figure 8.2.2a shows the alignment of the river in 1938. Note the location of the bend in the river. In 1949, flood waters eroded the south bank immediately upstream from the old bridge. As a result, in 1950 five pile diversion units were installed at this location (Fig. 8.2.2b). A new bridge was built in 1953. Floods of 1957 eroded the south bank immediately upstream of the new bridge. After subsidence of the flood waters, riprap was installed upstream of the bridge to prevent further erosion. Figure 8.2.2c shows this installation as well as subsequent training and stabilization that became necessary. Floods of 1959 damaged the five pile diversion units on the south bank and eroded some of the north bank immediately upstream of the bridge. *High flows were recorded from 1959 to 1962 causing a rapid movement of the meander loop downstream.* In 1963, five pile diversion structures were built on the north bank to prevent the river from encircling the north abutment of the new bridge. These pile diversions and the dike immediately upstream of the bridge have held the river in the same location. The south bank downstream of the bridge began eroding in 1971.

Cimarron River, east of Okeene, Oklahoma

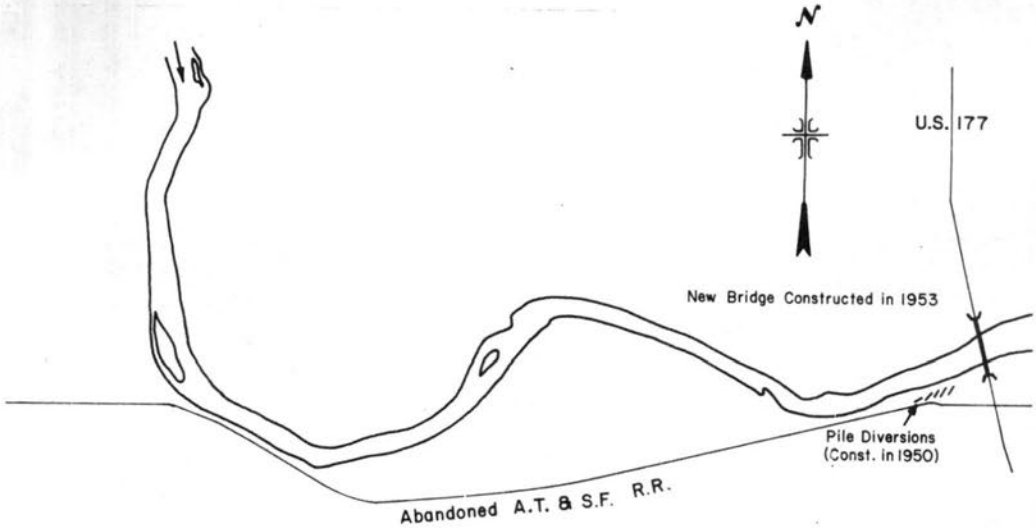
The bridge and a rock dike on its north abutment were built in 1934 (Fig. 8.2.3a). A high discharge year in 1938 caused the south bank to erode. A Kellner jetty field was installed to prevent further erosion of the bank. The jetty field was ineffective due to the lack of debris and suspended sediment load and 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. Riprap was also placed at the south abutment (Fig. 8.2.3b). As of 1968, the south bank had been held in line by the timber pile diversion structures and the dike (Fig. 8.2.3c).

Cimarron River, south of Waynoka, Oklahoma

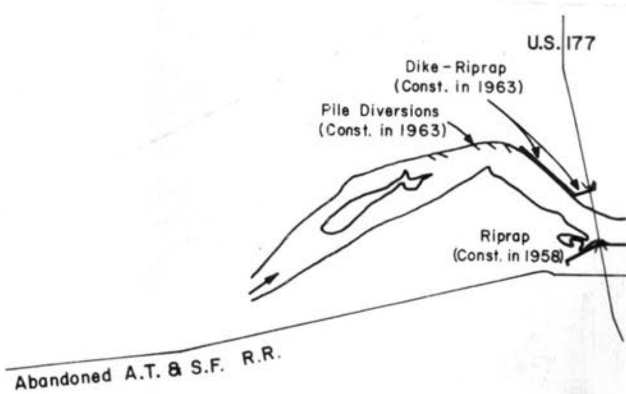
The river alignment prior to about 1942 is shown in Fig. 8.2.4a. Between 1942 and 1952, a series of above normal flows were reported. At some time in this period, a Kellner jetty field was installed on the north bank (Fig. 8.2.4b). A new bridge was begun in 1955 just upstream from the old bridge. 1955 was a year of high flow and several timber piles were installed on the north bank as well as two piles on



(a) Channel in 1938



(b) Channel in 1956



(c) Channel in 1969

Fig. 8.2.2 Cimarron River, south of Perkins, Oklahoma

(a) Channel in 1938

(b) Channel in 1957

(c) Channel in 1968

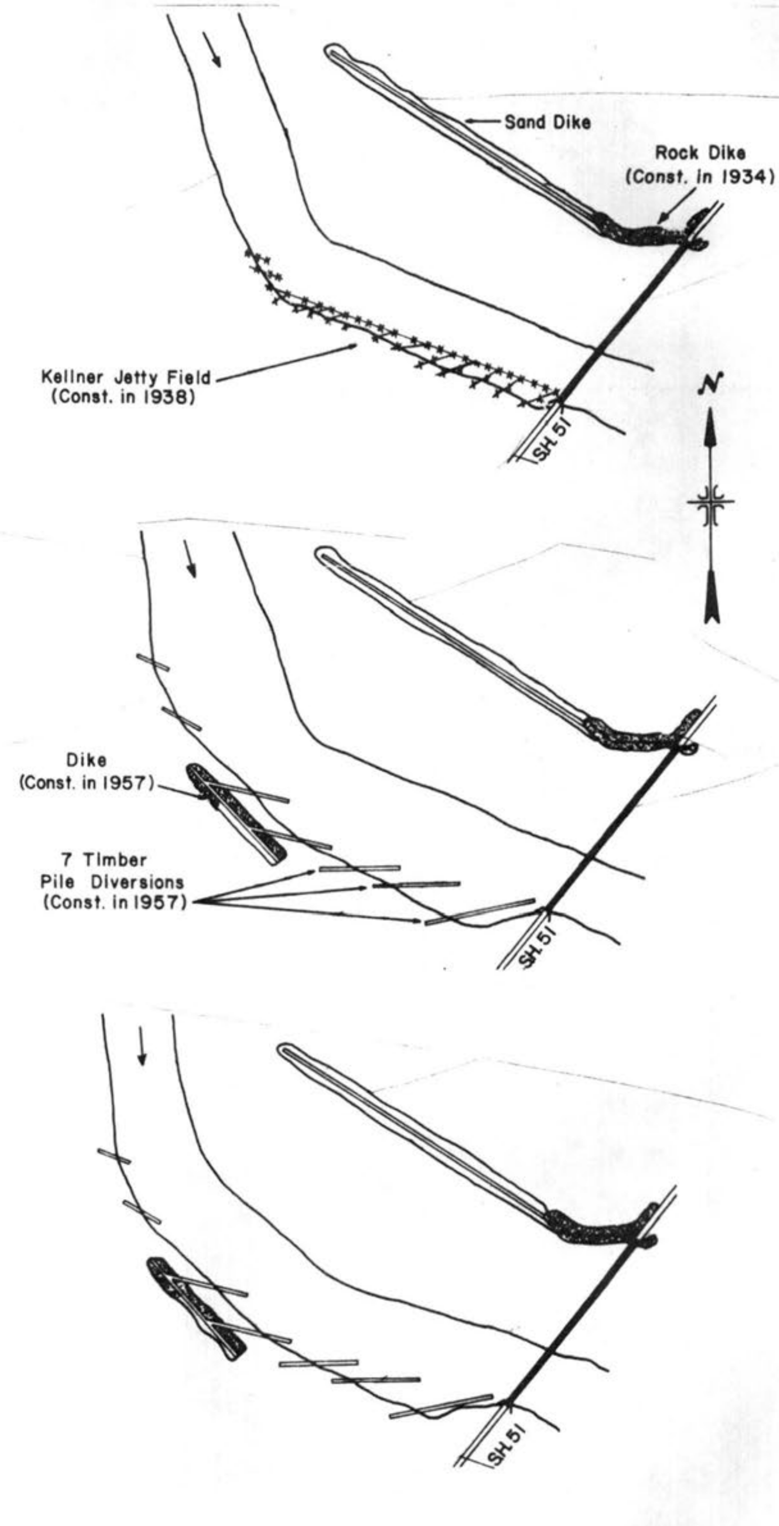


Fig. 8.2.3 Cimarron River, east of Okeene, Oklahoma

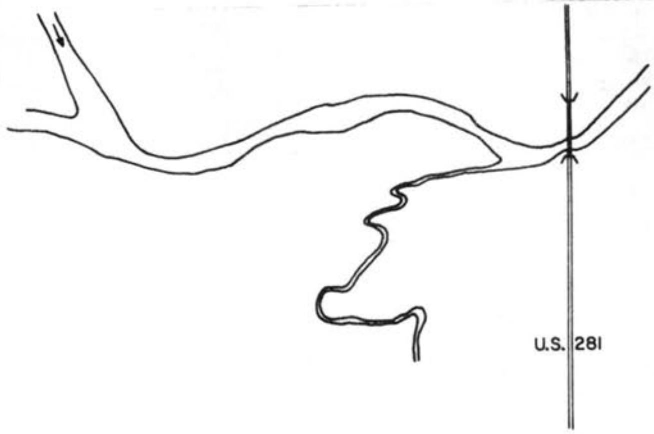
the south bank (Fig. 8.2.4b). *The highest flood on record occurred in 1957 and overturned two piers of the new bridge dropping three spans into the water.* Several damaged pile diversion structures were repaired and pile diversion structures were added to the north bank. Riprap was placed around the north abutment and upstream along the north bank in 1958. The channel has essentially retained this alignment through 1968 (Fig. 8.2.4c). No data on channel alignment subsequent to this date were available to evaluate additional changes that may have occurred.

Arkansas River, north of Bixby, Oklahoma

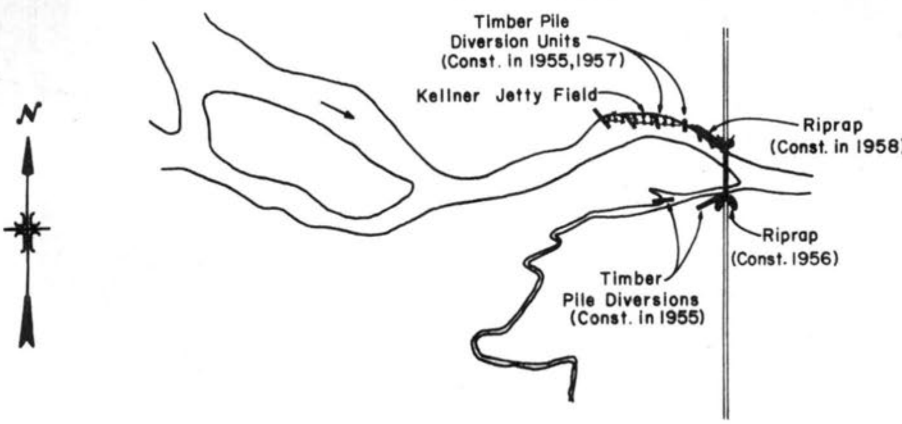
The bridge was built in 1938. A Kellner jetty field was installed on the north bank in 1939 to protect the north bridge abutment (Fig. 8.2.5a). In 1948 minor floods eroded the south bank. A Kellner jetty field was installed to prevent further erosion (Fig. 8.2.5b). Some time after, riprap was put on the south bank upstream and downstream of the jetty field (Fig. 8.2.5c). *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 (Fig. 8.2.5c). As of 1968, the south bank has remained stationary, and the north bank has filled in to some extent (Fig. 8.2.5d).

Washita River, north of Maysville, Oklahoma

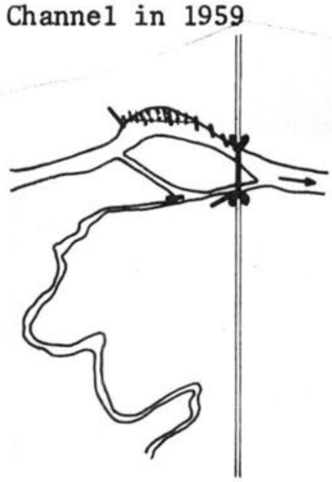
In 1949, floods washed out the north span of the bridge. Also, the banks upstream from the bridge were damaged. A temporary structure was installed in place of the north span of the bridge. In October of 1949, two Kellner jetty fields were completed upstream from the bridge to provide bank protection (Fig. 8.2.6a). 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* (Fig. 8.2.6b). The floods of 1957 did very little damage to this bridge site or the banks. Floods



(a) Channel in 1942

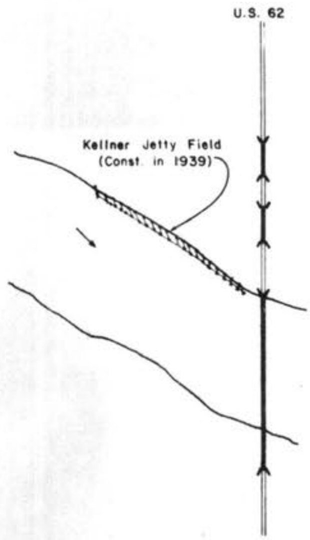


(b) Channel in 1959

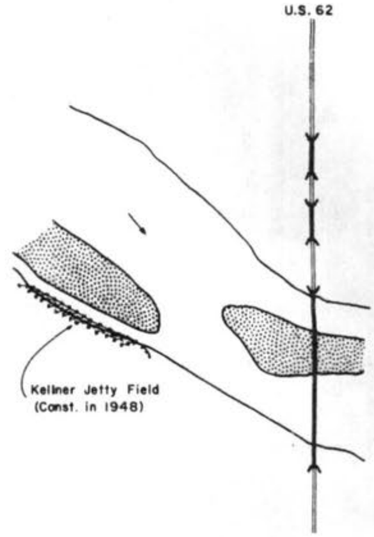


(c) Channel in 1968

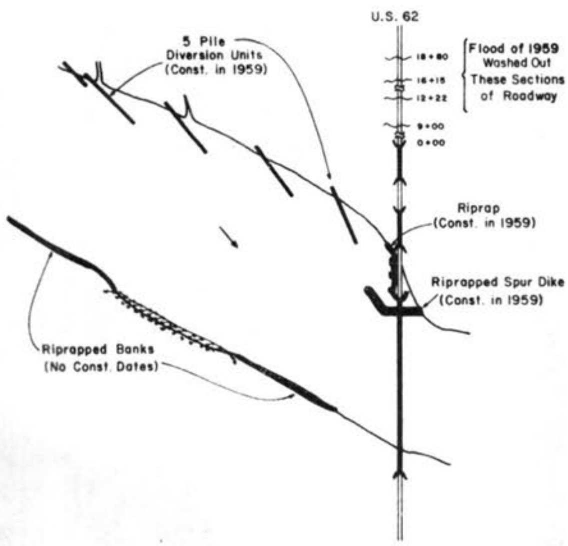
Fig. 8.2.4 Cimarron River, south of Waynoka, Oklahoma



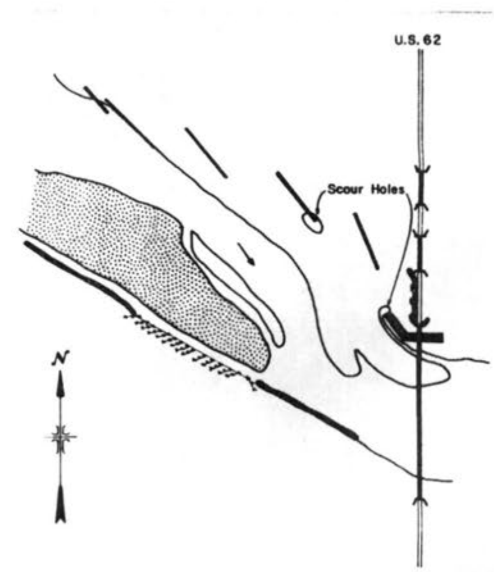
(a) Channel in 1939



(b) Channel in 1948



(c) Channel in 1959



(d) Channel in 1968

Fig. 8.2.5 Arkansas River, north of Bixby, Oklahoma in 1968 and 1969 have caused bank erosion on the north bank upstream of the jetty field which could eventually cut in behind the jetty field (Fig. 8.2.6c).

Cimarron River, south of Crescent, Oklahoma

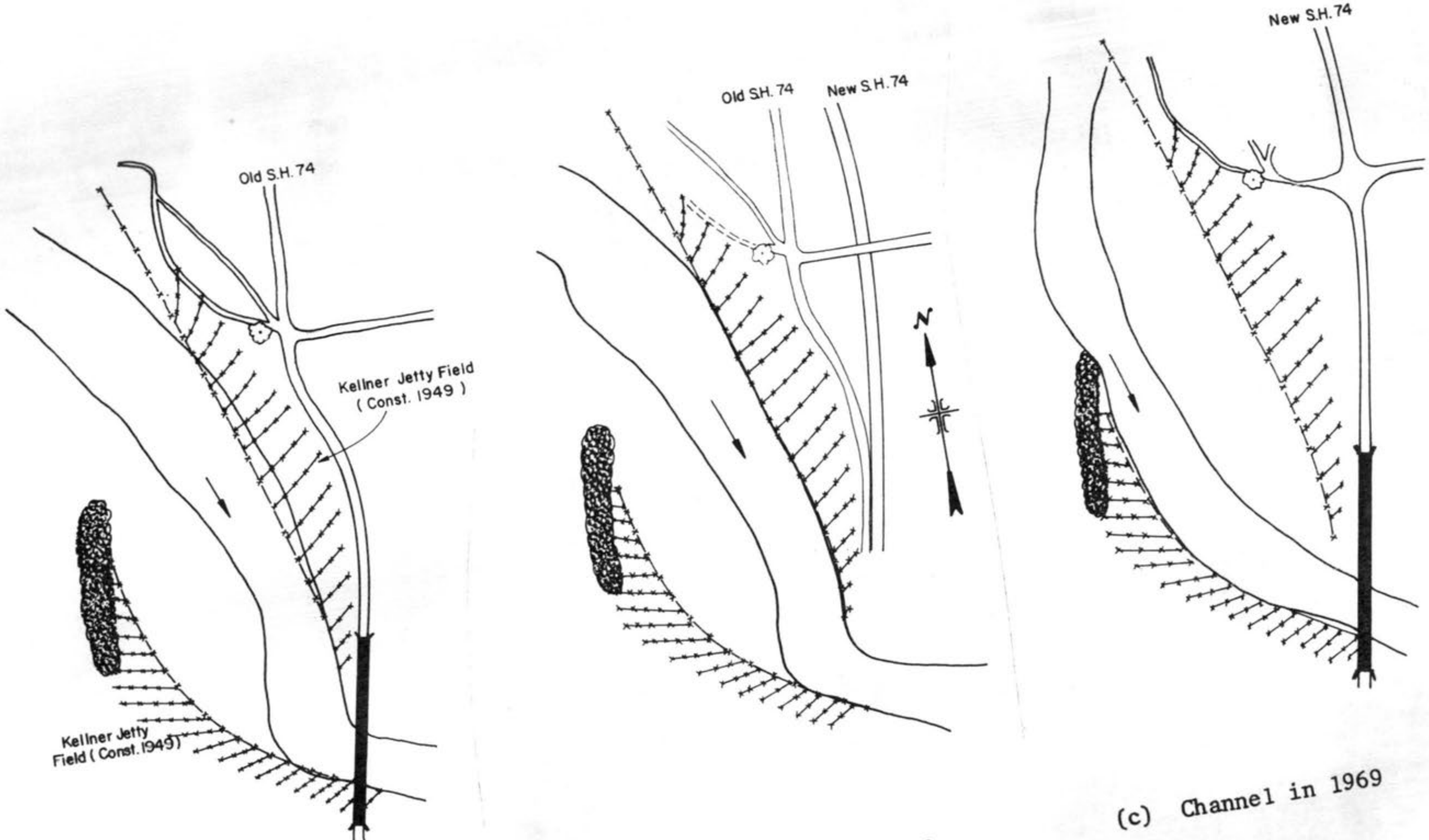
The water year 1937-38 was one of high flow. A Rayfield jetty field was constructed to control the river and stabilize the bank to protect Old State Highway 74 (see Fig. 8.2.7a). In 1942-43, high flows caused the east bank to erode closer to the embankment of Old State Highway 74. Sixteen hundred feet of embankment was riprapped for protection. In 1956, a new bridge was built to replace the old bridge. Earth dikes were constructed on the upstream and downstream side of the south abutment. Riprap was also placed around the south abutment (see Fig. 8.2.7b). The north abutment is located on a solid rock bluff. High flows in 1957 caused the river to overtop Old State Highway 74 in an attempt to cut off the bend to the north. *An earth dike and riprap were placed along the east bank to prevent further shifting eastward* (Fig. 8.2.7b). No further damage has occurred through 1971 (see Fig. 8.2.7c).

Washita River, south of Davis, Oklahoma

Prior to 1967, the river shifted several times during high flows, gradually encroaching on the future location of I-35 (see Figs. 8.2.8a and b). High flows in 1967 caused the two meander bends closest to the road to approach even closer. Riprap was placed on the concave bank of the upstream meander bend and plans were made for a Kellner jetty field at the downstream bend (see Fig. 8.2.8c). High flows prevented the construction of the Kellner jetty field until the fall of 1968. After placement a new channel was excavated and the old channel was filled in. Thus, the jetty field was more for the purpose of bank protection than realignment. Both bank protection methods have been successful in the few years of their operation.

Beaver River, north of Laverne, Oklahoma

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*

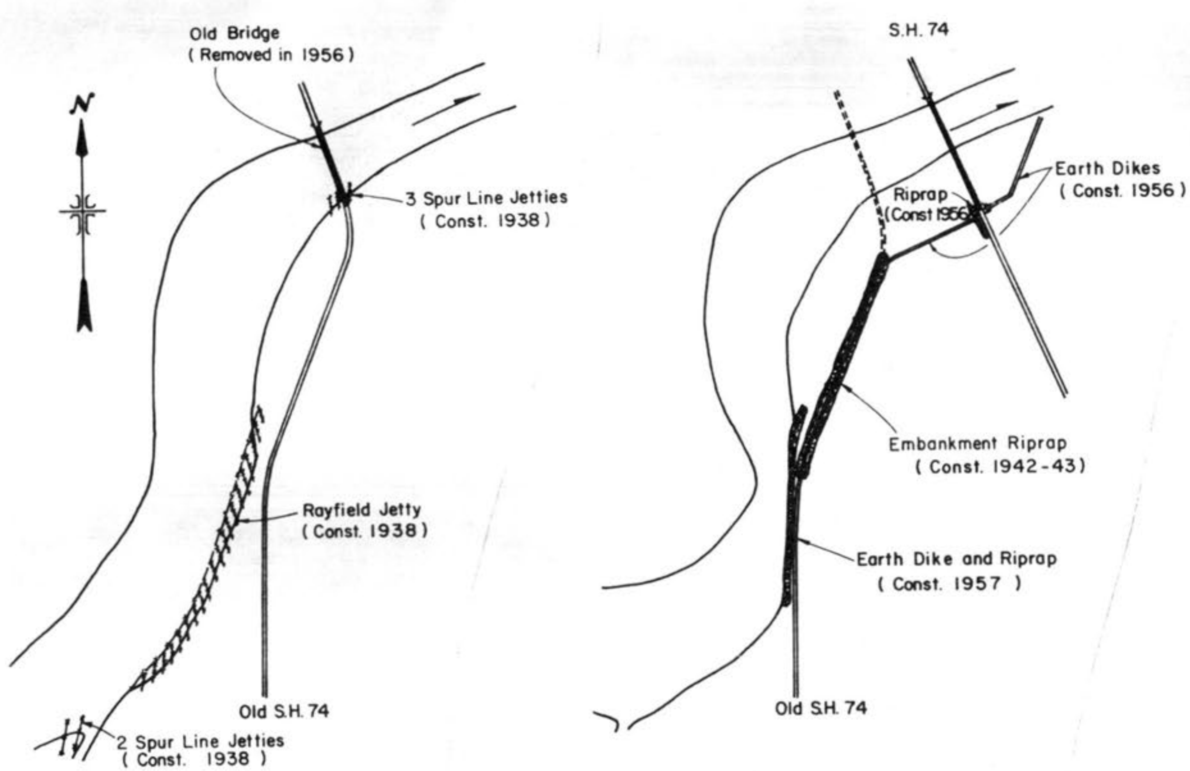


(a) Channel in October, 1949

(b) Channel in May 1950

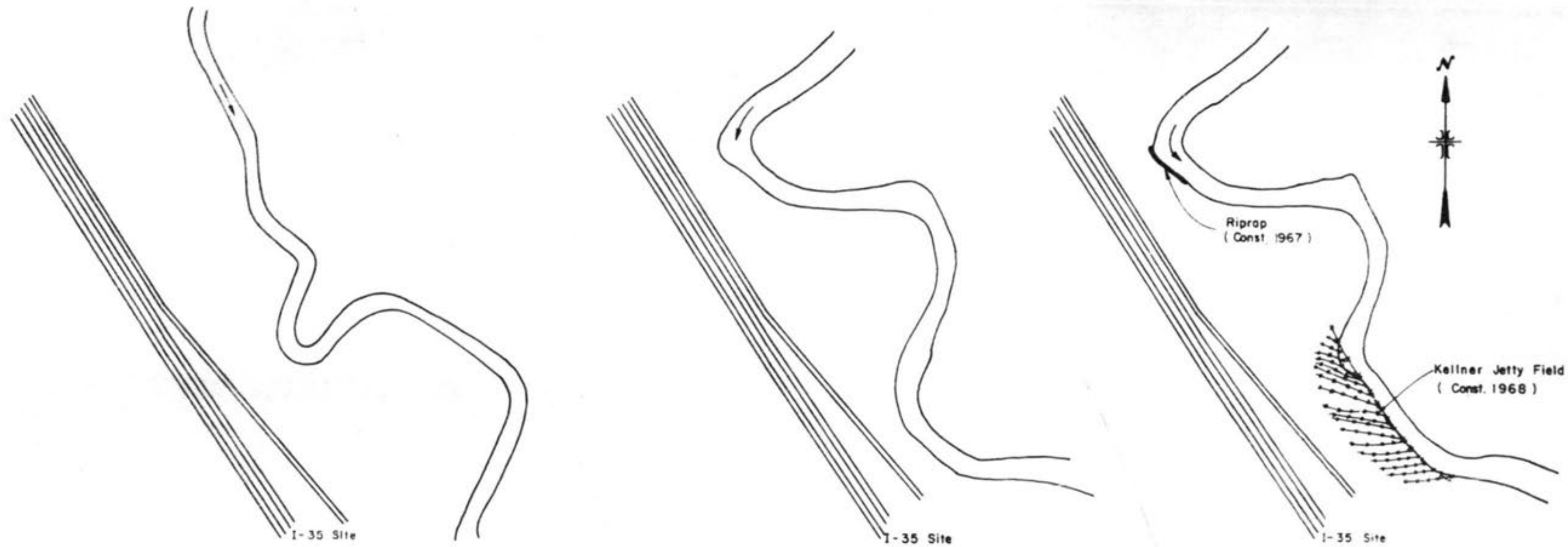
(c) Channel in 1969

Fig. 8.2.6 Washita River, north of Maysville, Oklahoma



(a) Channel in 1938 (b) Channel in 1957 (c) Channel in 1968

Fig. 8.2.7 Cimarron River, south of Crescent, Oklahoma



(a) Approximate channel location in 1940

(b) Approximate channel location in 1966

(c) Approximate channel location in 1968

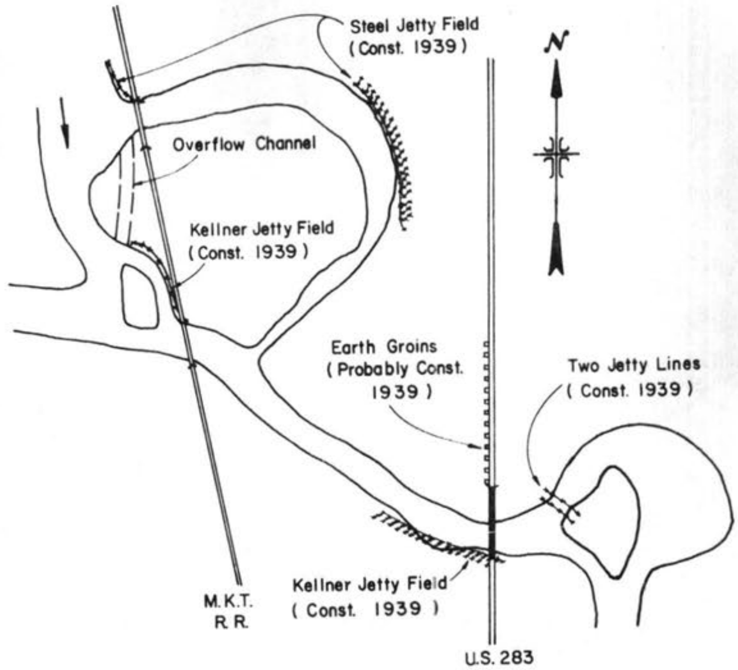
Fig. 8.2.8 Washita River, south of Davis, Oklahoma *of the bridge to discourage flow in that channel to prevent eddy currents from eroding the north embankment* (see Fig. 8.2.9a). A new longer bridge was constructed in 1941. High flows in 1946 caused severe erosion on the south bank upstream from the 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 Fig. 8.2.9b). *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.*

Powder River, 40 miles east of Buffalo, Wyoming

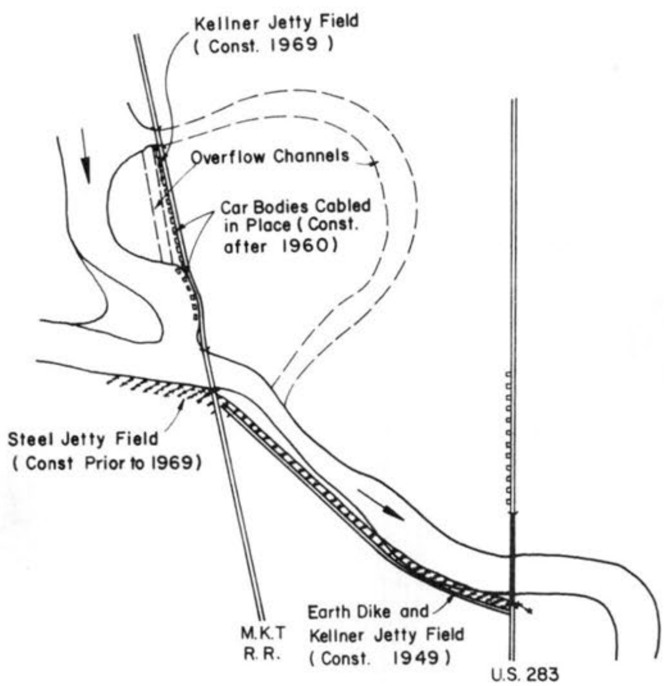
The Powder River has very fine bed material and a high sinuosity. The river contains dunes at low flows. At the bridge site, there was a grove of cottonwood trees on the upstream left of the bridge and a dry draw coming in from the upstream right (see Fig. 8.2.10a). Upon completion of the bridge, a large flood occurred. The river attempted to straighten out its meanders. 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 left (west) abutment. The flood flow uprooted the grove of cottonwoods (estimated to be 50 years old) and carried them downstream. Some of the cottonwoods hung up on the riprapped spur dike at the west abutment and destroyed the dike completely (see Fig. 8.2.10b). 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. This has been fairly successful to date (1974), being in place approximately six years (see Fig. 8.2.10c).

North Platte River, Wyoming

The North Platte River is a fairly stable river in this reach as a result of reservoir control upstream. It was decided at this crossing to build the bridge over the main channel and part of the island and to block off the overflow channel on the opposite side of the island. Downstream from the bridge, a farmer had located his irrigation pump on the concave bank of a meander. The pump was protected by a man-made

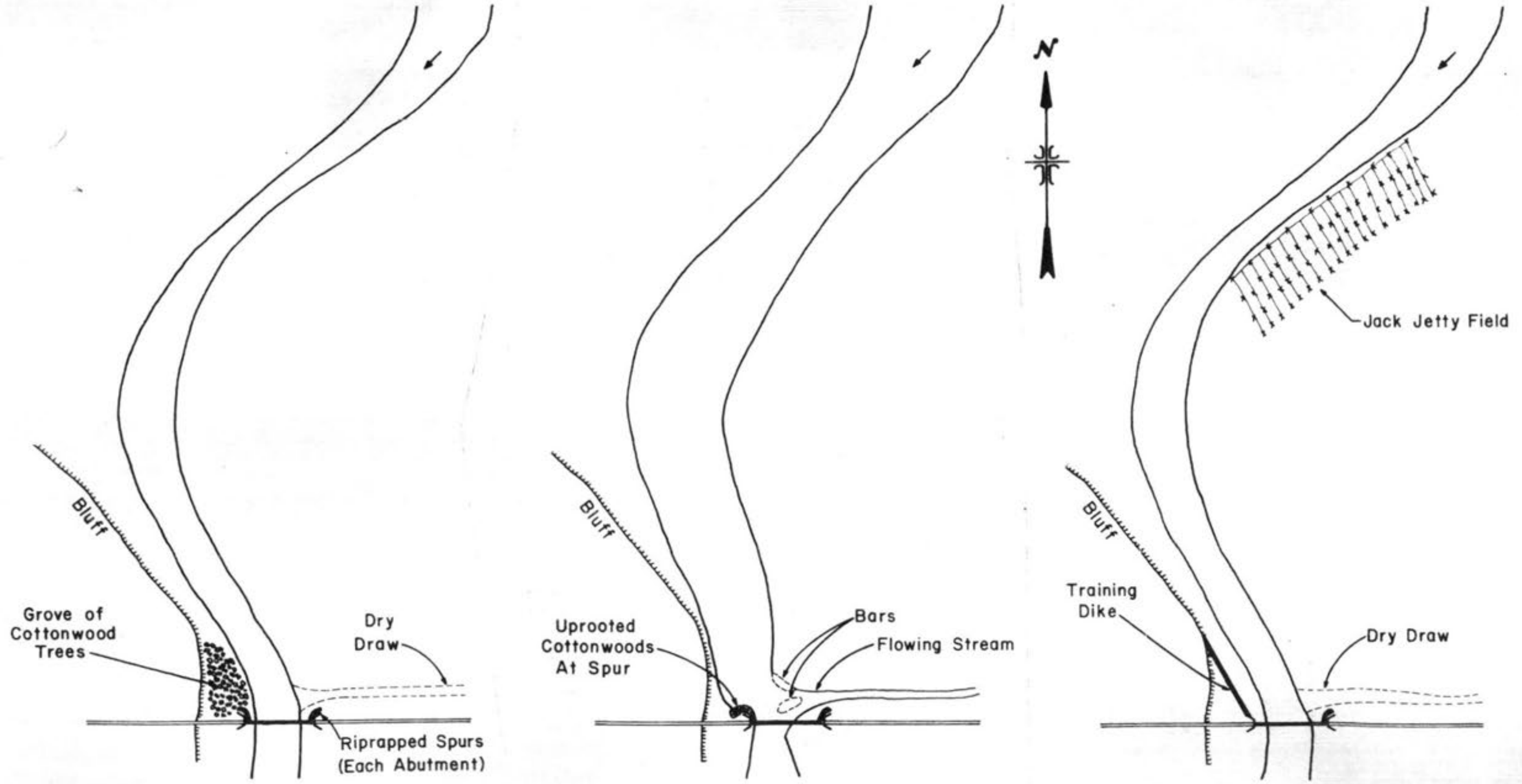


(a) Channel in 1939



(b) Channel in 1960

Fig. 8.2.9 Beaver River, north of Laverne, Oklahoma



(a) Channel after completion of bridge.

(b) Channel during flood

(c) Channel after restoration

Fig. 8.2.10 Powder River, 40 miles east of Buffalo, Wyoming armored bar (Fig. 8.2.11a). *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. This would wash out the bar and irrigation pump (see Fig. 8.2.11b). The other situation which may occur is that the high velocity flow carries increased sediment load and deposits this material downstream in the eddies caused by the bar protecting the pump. Thus, the pump pit fills and the pump becomes inoperable (see Fig. 8.2.11c). Either action is possible. In fact, with extreme flows the river could erode a chute across the point bar on the first bend downstream.

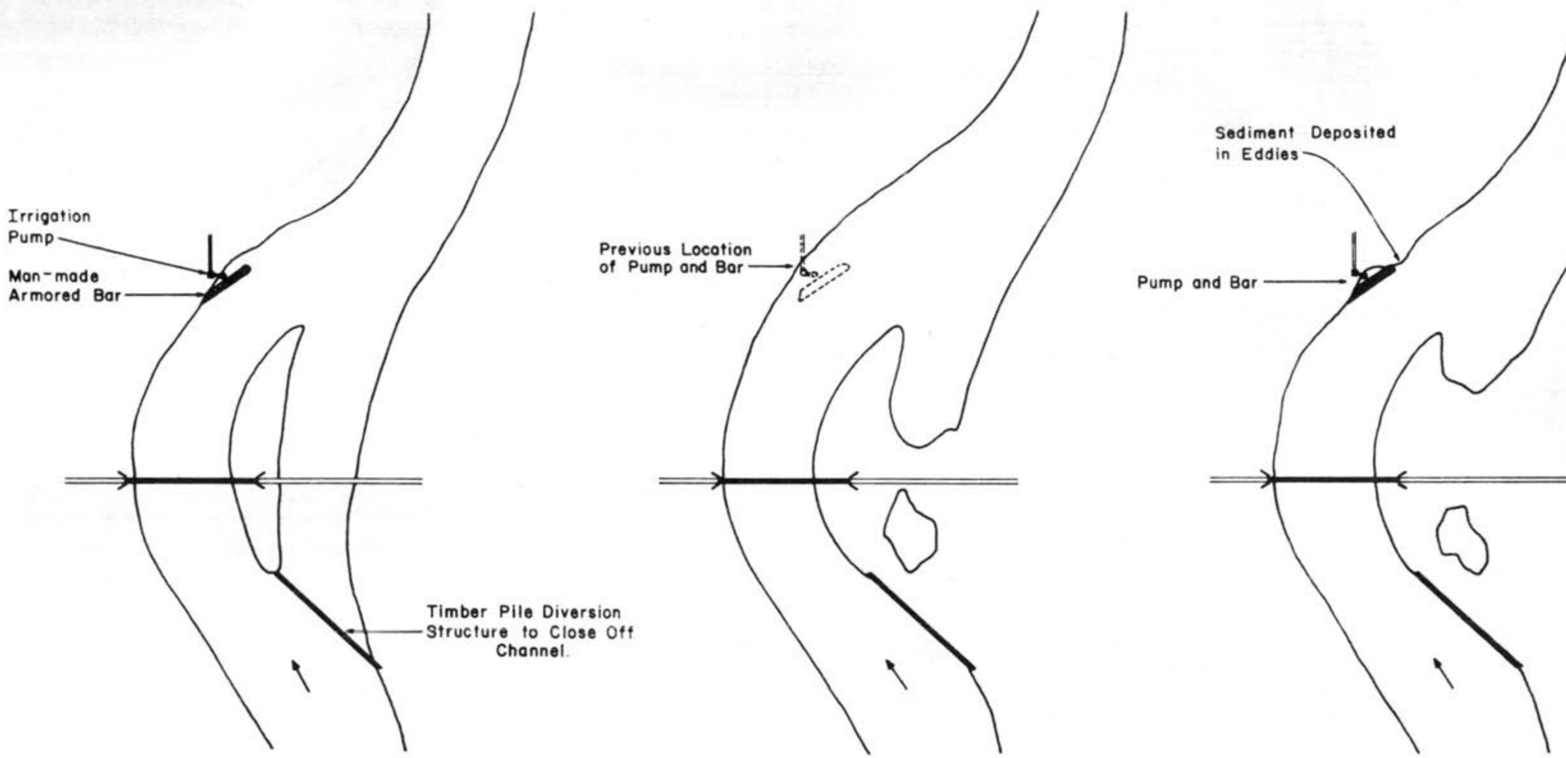
Coal Creek, tributary of Powder River, Wyoming

A small bridge was constructed over intermittent Coal Creek. Coal Creek was a dry draw at the time of construction. *There existed some head cuts downstream from the bridge at the time of its construction* (see Fig. 8.2.12a). During a subsequent flood, one head cut moved upstream through the bridge site. This head cut almost undercut the midstream piles and it exposed some of the abutment piles. To prevent further degradation under the bridge when the second head cut moves through, rock filled baskets were placed on the bed under the bridge (see Fig. 8.2.12a). When the second head cut moved through, the rock filled baskets settled slowly, but prevented the undermining of the piles (see Fig. 8.2.12b). Other alternatives would have been to excavate the head cuts in the channel through the bridge site before constructing the bridge, allowing them to move naturally upstream from there, or to set the piles deeper in anticipation of the lowering of the bed elevation.

8.3.0 PRINCIPAL FACTORS TO BE CONSIDERED IN DESIGN

8.3.1 Introduction

The following paragraphs identify the principal factors that should be considered in the design of crossings and longitudinal encroachments. Because of the differences in river size and forms at different locations, it is not possible to outline a single system that is applicable to all problems. On the other hand, *it is essential that the application of the fundamentals to these problems be understood.*

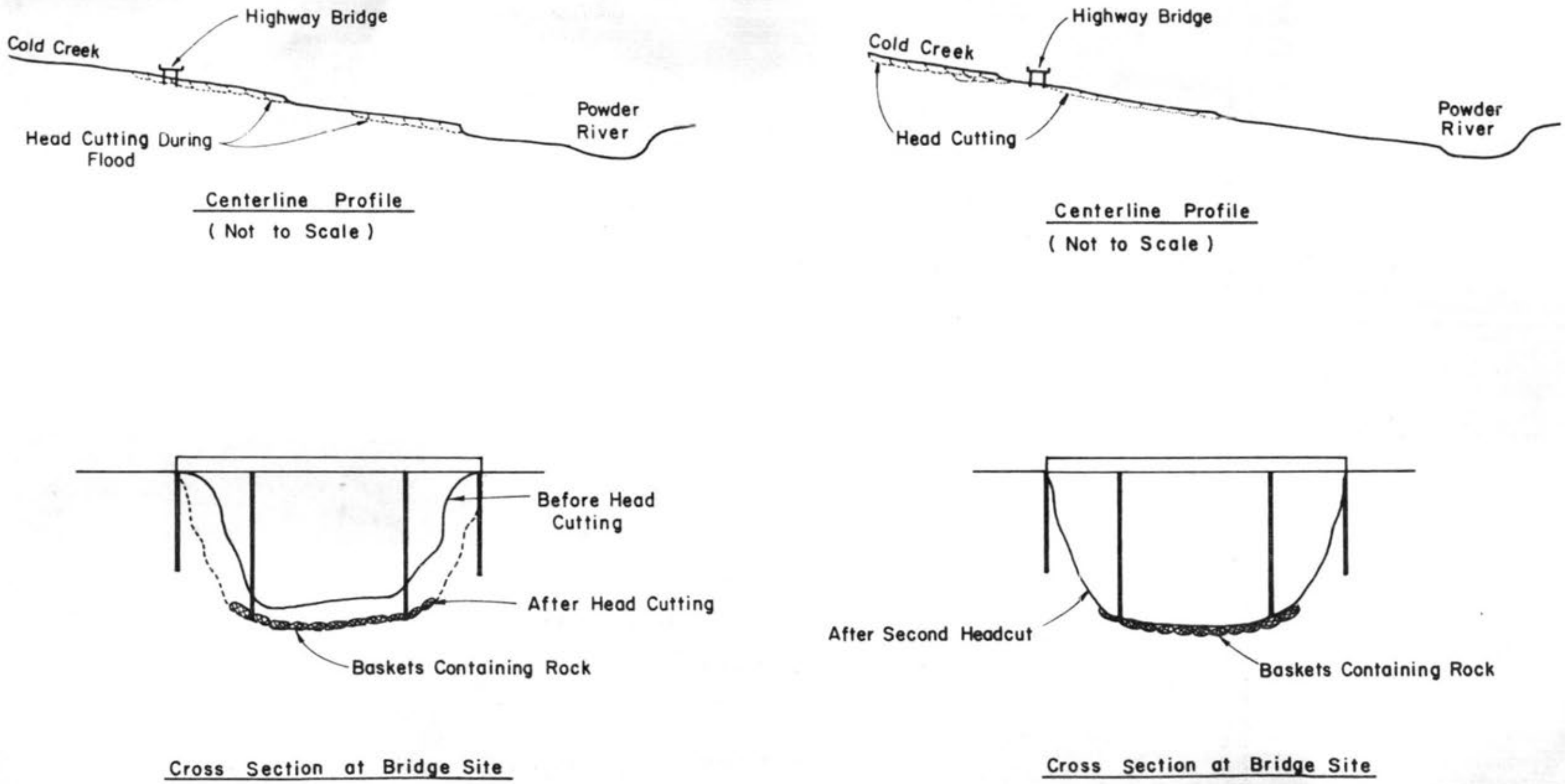


(a) Channel immediately after closure

(b) Channel after first flood (situation 1)

(c) Channel after first flood (situation 2)

Fig. 8.2.11 North Platte River, Wyoming



(a) First head cut

(b) Second head cut

Fig. 8.2.12 Coal Creek, tributary of Powder River, Wyoming

8.3.2 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 subdivisions 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.

8.3.3 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 encroachmennt must mesh with the transportation system in the area.* Secondly, *the environmental considerations cited in Chapter 1 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 bridge, 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 quality 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.

8.3.4 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 Fig. 1.3.2 of Chapter I 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 IV for example Fig. 4.4.3 or Chapter III, Fig. 3.4.1 can be used to predict this sensitivity. A meandering stream whose slope and discharge plot close to the braided river line in Fig. 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 sand banks; it may have a sand bed and gravel banks; it may have a sand bed and cohesive banks; it may be formed entirely in cohesive materials; it may be formed in gravel; it may be formed in cobbles; or other combinations of these material. 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.

8.3.5 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 V, 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.

8.3.6 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 VII, 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". The basic flood is a 100 year flood and the 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. As pointed out in Table 8.2.1, Case (3), 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.

8.3.7 Hydraulic data

Both 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 in 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 training works 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 that the river operates in upper regime over its whole range of discharge. These possibilities must be considered in the detailed design.

8.3.8 Characteristics of the watershed feeding the river system

The water flowing in the river system and the sediment transported therein is 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 use including recreation, industrial development, agricultural, grazing etc., may all be important factors. Similarly we need to consider the vegetative cover on the watershed and the response of vegetative cover to its utilization by man and to climatic changes. Significant changes in vegetative 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 those areas that 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. In addition such information can be used to determine the feasibility of watershed engineering to help control the water and sediment yield from the watershed to the river system.

8.3.9 Flow alignment

In order to appropriately and safely design a crossing or longitudinal encroachment it is necessary to consider in detail flow alignment. *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 VI to design against excessive general and local scour, in order to make the highway functional with minimum maintenance over the life of the project.

8.3.10 Flow on the floodplain

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 (refer Case 14 in Table 8.2.1). 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 to develop chute channels across the point bars. Often the water spills over the outsides of the bends onto the floodplain. As has been illustrated in the preceding chapters flow conditions on the floodplain and in the main channel can be greatly different at flood stage than at low flow and these factors must be taken into consideration. A case in point is the new bridge being constructed across the Mississippi near Caruthersville, Missouri. In this instance the flow on the floodplain was sufficiently intense and the alignment of the approaches in relation to the flow on the floodplain was such that a large channel was scoured paralleling the upstream side of the Missouri approach by the overbank flow. Ultimately a large segment of the Missouri approach 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 probability of such an occurrence.

8.4.0 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 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.

8.5.0 CHANNEL STABILITY INVESTIGATIONS

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 VI 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.

8.6.0 SHORT-TERM RESPONSE

Having completed the tentative design of the crossing or the encroachment based on river form, channel geometry, hydrologic and hydraulic data etc., 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.

8.7.0 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 course. Nevertheless, sufficient advances have been made pertaining to the mathematical modeling of river systems considering both their short- and long-term response that this approach is worth considering on important projects.

8.8.0 DESIGN EXAMPLES

8.8.1 Introduction

In this concluding section 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, *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 cook-book approach and it is hoped that the examples make this evident.*

The examples relate to the design of a crossing on the "Mainstream River" shown in the aerial photograph of Fig. 8.8.1. The flow is from right to left. An existing highway crossing can be seen in the photograph but the highway and its alignment are to be upgraded and one alternative for the crossing is drawn on the photograph about 2200 ft upstream. 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 one must assume that there are factors other than those associated with the bridge location that make this alternative worthy of scrutiny. Indeed, such 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*

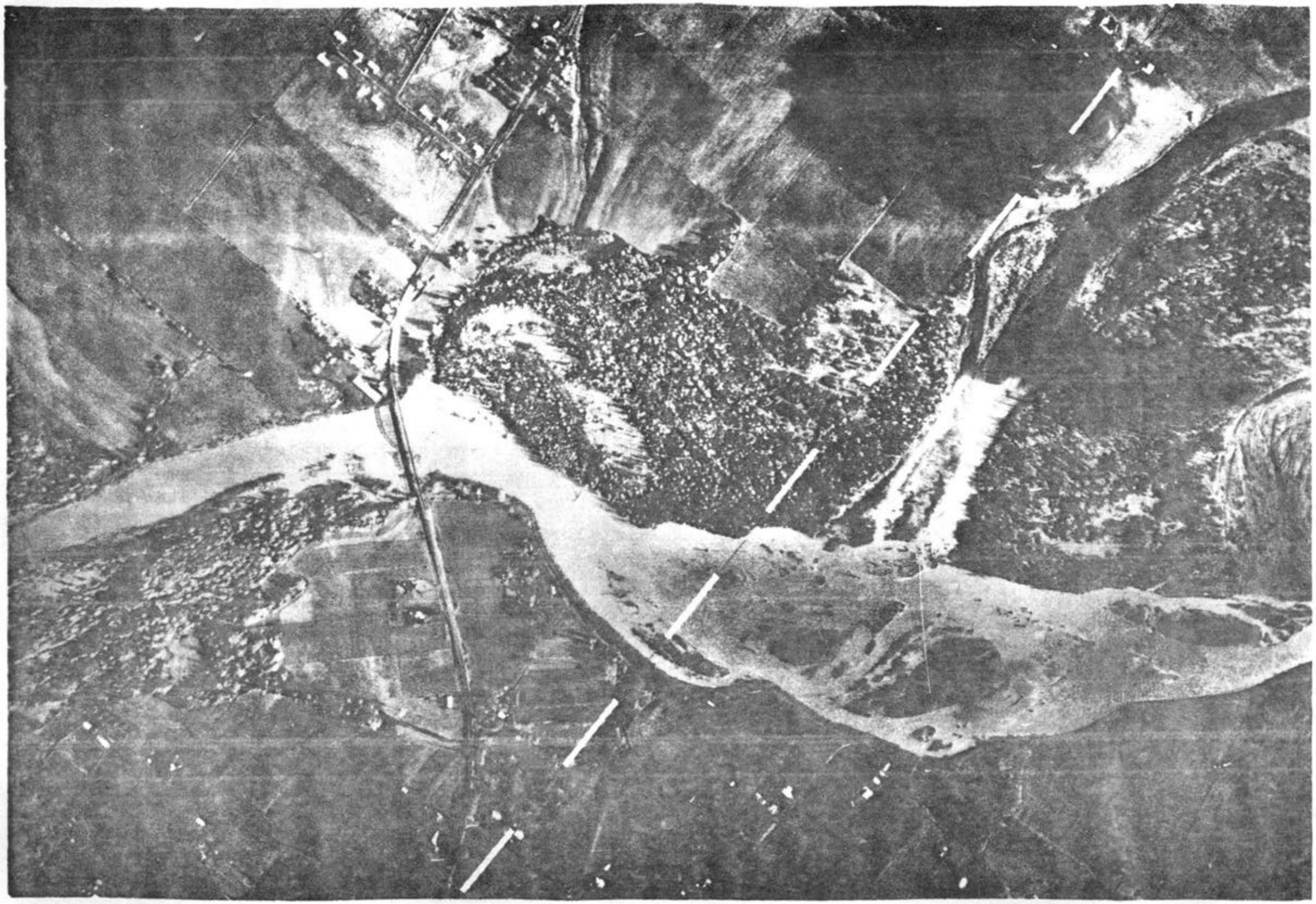


Fig. 8.8.1 Mainstream River showing existing crossing, newly proposed crossing, recently cutoff meander, and broad flat floodplain. Flow is from right to left and the scale is approximately 1 inch = 1000 ft.

river flow is 2900 cfs. Fig. 8.8.2 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 Fig. 8.8.2, *is only 77,000 cfs.* Flood peaks are relatively short lived on this river. The daily flows of Fig. 8.8.2 are replotted in Fig. 8.8.3 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.

The flood of record, estimated at 105,000 cfs at the proposed *crossing 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 41,000 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. Fig. 8.8.4 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 Fig. 8.8.2 the mean flow of 2900 cfs is almost never realized. The flows are much higher in the winter and very much lower in the summer. One cannot imagine the low summer flows as contributing much to the morphology. The record flood, on the other hand, created an anomaly that only time will restore to equilibrium. At this stage of the development of knowledge *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.*

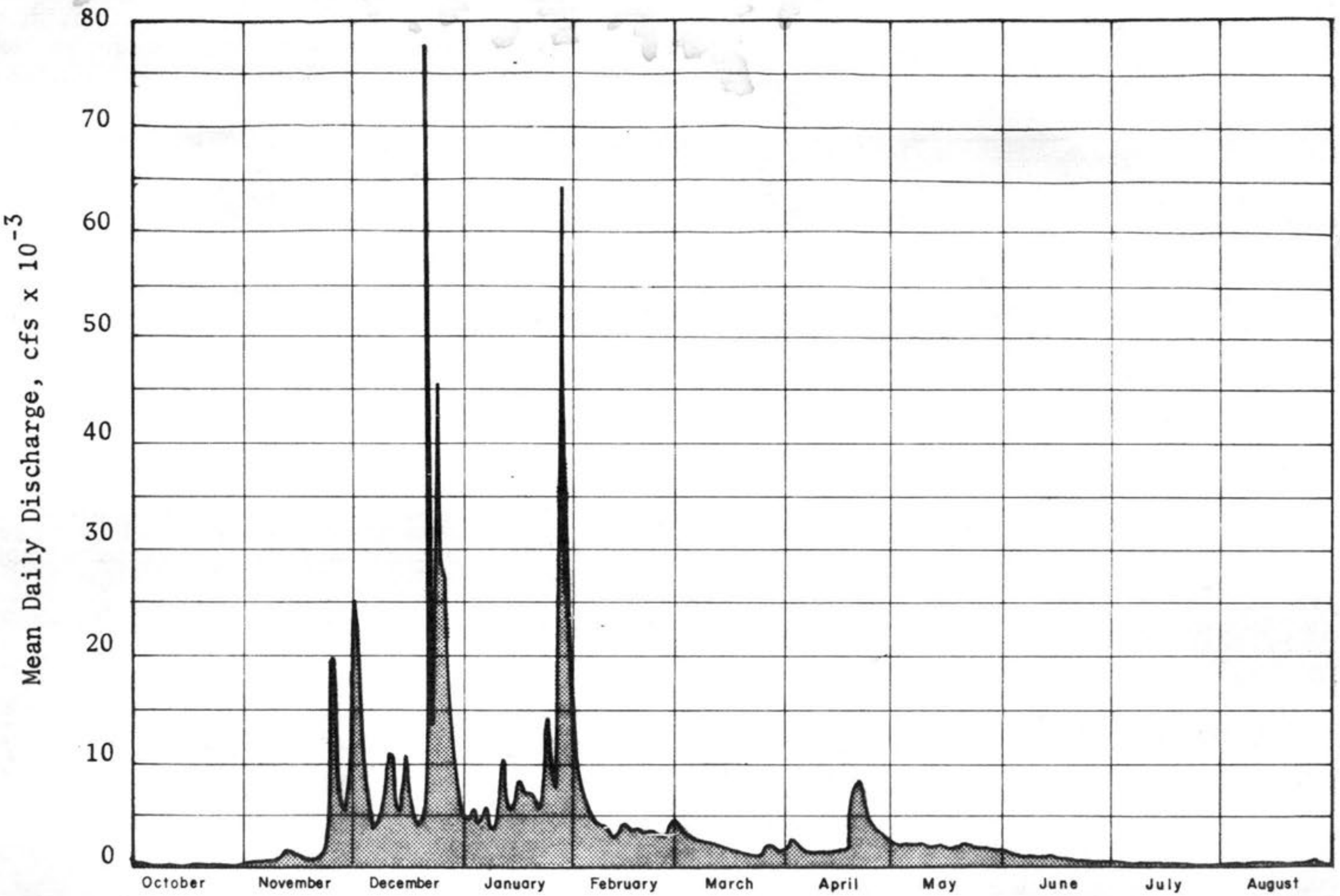


Fig. 8.8.2 Hydrograph from gaging station on Mainstream River 12 miles upstream of proposed crossing

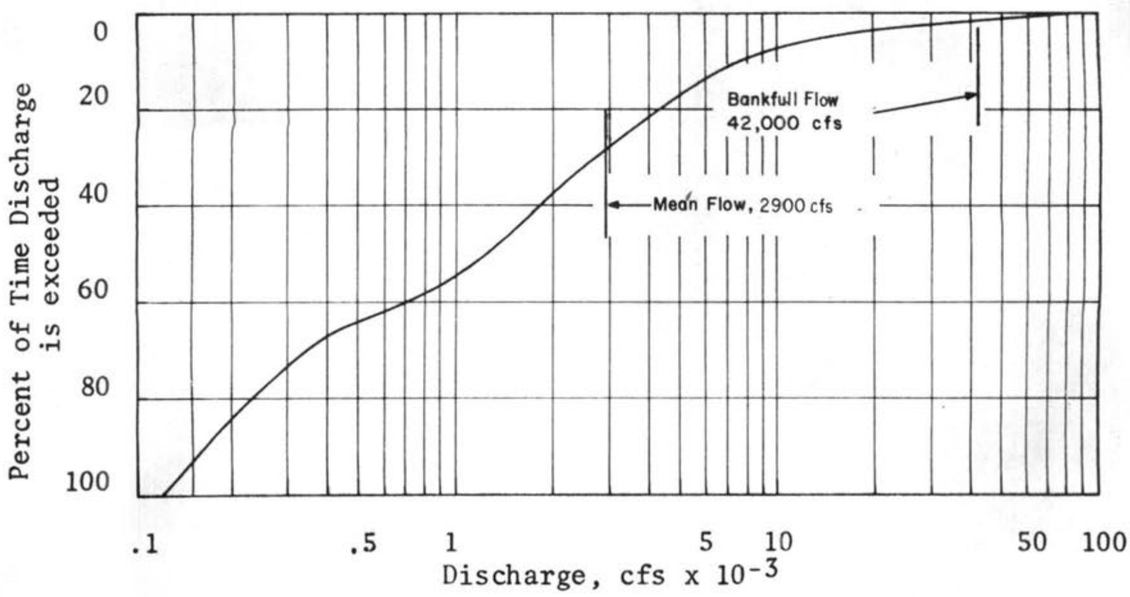


Fig. 8.8.3 Flow duration curve for Mainstream River

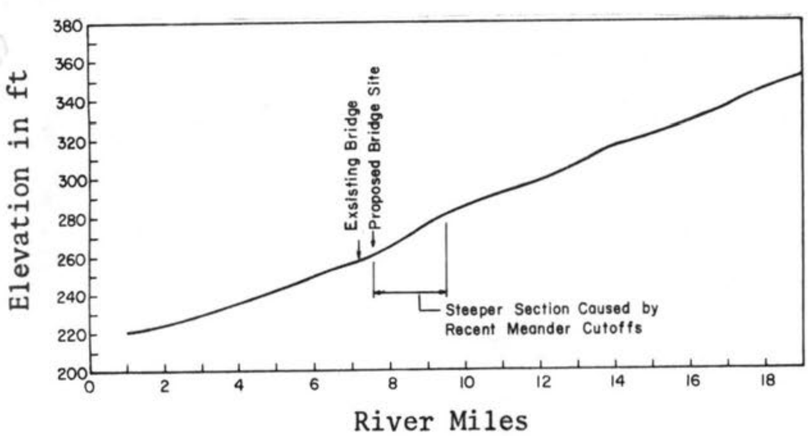


Fig. 8.8.4 Profile for the Mainstream River

Applying the river slope of 1.38 ft/1000 ft and the mean discharge of 7000 cfs to Fig. 4.4.3 of Chapter IV, the characteristics of this border on the braided zone. Meanders are, however, very much in evidence in the outlines of Fig. 8.8.5. 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 and considerable armoring takes place during degradation. The USGS has recorded a sediment load 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 Fig. 8.8.5 shows outlines of the waters 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 1964 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.

8.8.2 Design example 1

In this example, a bridge crossing on the Mainstream River at the alignment shown in Fig. 8.8.1 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 Fig. 8.8.6. 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 - Design flow for bridge safety is stated to be the 100-year flood and the 100-year flood is 110,000 cfs. 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 essentially the design flood. This actual measured value of stage is much superior to any that could be calculated by backwater methods.

Fig. 8.8.5 Recent alignment changes of Mainstream River

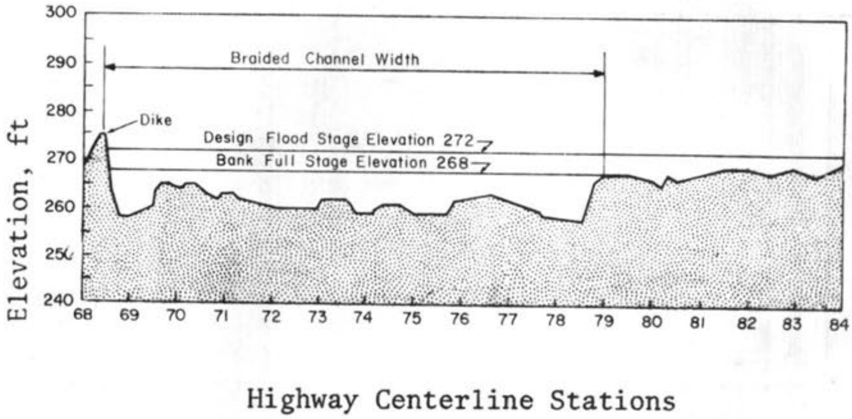


Fig. 8.8.6 Cross section of the crossing on Mainstream River

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. Fig. 8.8.7 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 anomoly 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 (Eq. 2.3.20) can then be written in terms of unit discharge and solved for depth. That is,

$$y_o \simeq \left\{\frac{qn}{1.486 S_f^{1/2}}\right\}^{3/5}$$

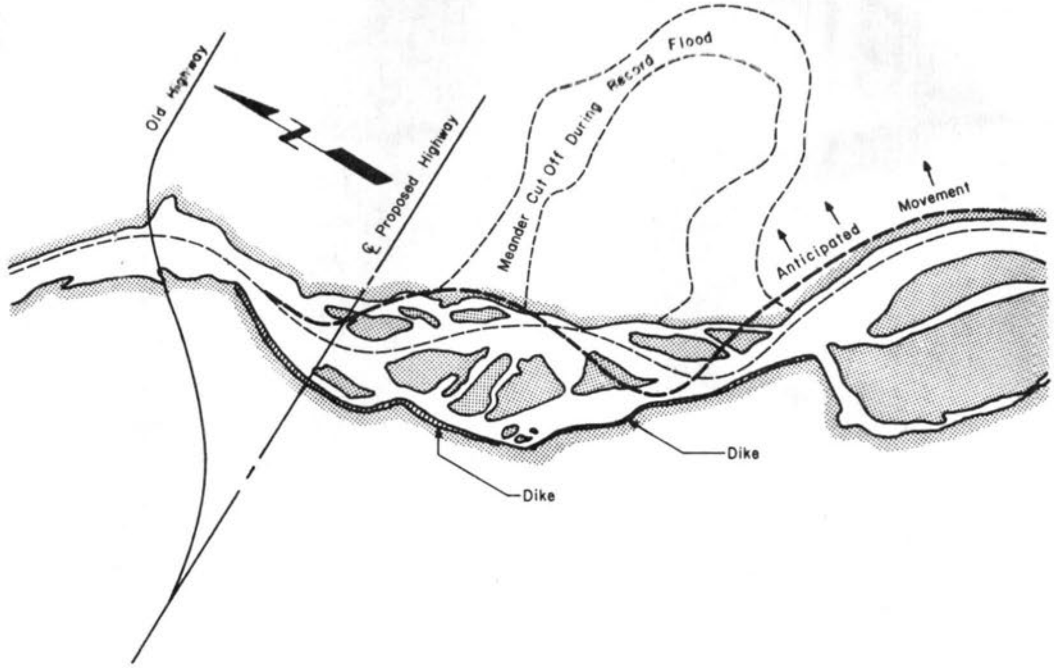


Fig. 8.8.7 Existing and anticipated channel alignments for Mainstream River

In Table 8.8.1 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 of 0.025 (Table 2.3.1, alluvial channel bed in transition). Also, from Eq. 3.10.46 and the D_{90} of the bed material equal to 15 mm Manning's n is 0.024 and Fig. 8.8.1 indicates large bars in the channel. Then

$$y_o = \left\{\frac{(132)(0.025)}{(1.486)(0.00138)^{1/2}}\right\}^{3/5} = 11.6 \text{ ft}$$

and

$$V = \frac{q}{y_o} = \frac{132}{11.6} = 11.4 \text{ fps}$$

Then, the average shear stress on the bed is (from Eq. 2.2.31)

$$\tau_o = \gamma y_o S_f,$$

$$= (62.4)(11.6)(0.00138) = 1.0 \text{ psf}$$

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, Fig. 3.4.1 indicates that the flow should be in the upper flow regime. That bed form does not check our assumption that flow is in transition. However, because of the gravel bars and large D_{90} 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 (from Eq. 2.4.8)

$$Fr = \frac{V}{\sqrt{gy_o}} = \frac{11.4}{\sqrt{(32.2)(11.6)}} = 0.59$$

The width-to-depth ratio for the channel at the design flow is

$$\frac{W}{y_o} = \frac{500}{11.6} \simeq 43$$

A summary of the calculations are given in Table 8.8.1.

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. The geometry of pools in bendways has been discussed in Section 5.6.2, Chapter V. For a given river system, the maximum depth in a meander bend y_{max} is primarily dependent on the river width W and usually to a lesser extent on the radius of curvature r_c. (See Fig. 5.6.1). Looking back on Fig. 5.3.2 it is noted that the radii of curvature vary in a river. A study of the topograph maps will reveal the information required to determine the frequency of occurrence of bends.

For the Mainstream River, the most common value of r_c is 2500 ft. Therefore

$$\frac{r_c}{W} = \frac{2500}{500} = 5$$

In the crossings, the average depth is 11.6 ft at design discharge so the depth-to-width ratio is

Table 8.8.1 Data and computations for design example 1

Data:

Valley slope, S_v = 0.0019 (10.0 ft/mile)
River slope, S_o = .00138
Average discharge = 2,900 cfs
Average five-month winter discharge = 7,000 cfs
Bankfull discharge = 42,000 cfs

Record discharge:	Channel =	64,000 cfs
	Overbank =	41,000 cfs
		105,000 cfs
Design discharge:	Channel =	66,000 cfs
(100-year)	Overbank =	44,000 cfs
		110,000 cfs

Design flood stage = 272 ft
Minimum channel width, W = 500 ft
Mean bed-material size, D_{50} = 1 mm

Conditions at design discharge:

Unit discharge in channel, q = 132 cfs/ft
Manning's n = 0.025
Average flow depth, y_o = 11.6 ft
Average velocity, V = 11.4 fps
Channel Froude number, Fr = 0.59
Average bed shear, τ_o = 1.0 psf
Average stream power, $\tau_o V$ = 11.4 ft lb/sec/sq ft

Width-to-depth ratio, W/y_o = 43
Flood stage elevation = 272 ft

Conditions in meander pool:

Maximum depth in pool, y_{max} = 29 ft
Bed elevation in deep part of pool = 243 ft
Average velocity in the pool = 14.2 fps
Froude number in the deep pool, Fr = 0.46

Embankment riprap for present conditions:

Angle of repose, ϕ = 37°
Side slope angle, θ = 18.4°
Shear stress on side slope, τ_o = 1.0 psf
Specific weight of riprap, S_s = 2.50
Safety factor, S.F. = 1.5
Stability number, η = 0.356
Effective grain size, D = 7.5 in
Recommended median size, D_{50} = 6 in
Recommended maximum size, D_{100} = 12 in
Minimum thickness or riprap = 12 in

Pier scour for present conditions:

Width of round nose pier, a = 5 ft
Length of pier, ℓ = 20 ft
Skew angle = 30 deg
Approach Froude number, Fr_1 = 0.59

Table 8.8.1 Data and computations for design example 1 (continued)

Approach flow depth, y_1 = 11.6 ft

maximum depth of scour, y_{smax} = 28 ft

Pier scour for future conditions:

Skew angle = 30 deg
Approach flow depth, y_1 = 29 ft
Approach Froude number, Fr_1 = 0.46
Maximum depth of scour, y_s = 34 ft

$$\frac{y_o}{W} = \frac{11.6}{500} = 0.023$$

It is possible that Fig. 5.6.3 can be used to determine the maximum depth in the pool of a meander bend. The stream order of Mainstream River can be found from topography maps, the r_c/W ratio is known and the stability factor is (from Eq. 5.6.8)

$$\chi = \frac{D_{50}W}{y_o^2 S}$$

$$= \frac{(1)(500)}{(304.8)(11.6)^2(0.00138)} = 8.8$$

The information in Fig. 5.6.3 is for rivers with $\chi \leq 1.5$ so Fig. 5.6.3 is not applicable in this design problem.

To determine the maximum depth in the meander bend the limiting value of y_{max}/W from Fig. 5.6.1 for r_c/W equal to 5 could be used. The value of the ratio is approximately 0.06. Therefore the maximum depth would be 30 ft. The limiting value is for relative stable streams so one could assume that y_{max} would be less than 30 for unstable rivers. A depth of 30 is probably conservative (deeper) for scour determinations but is not conservative for design of riprap (gives too low a velocity).

At this point, a few field measurements would be appropriate. The bed elevation and bank elevation at the deepest part of the bends with the largest, the smallest and the median radii of curvature should be measured along with the river width at these locations. The data establish the relation between y_{max}, W and r_c for the Mainstream River equivalent to that shown in Fig. 5.6.1.

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

$$272-29 = 243 \text{ ft}$$

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 Fig. 5.3.5. For a long bend in a parabolic channel, the maximum velocity in the bend is only about 10 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

$$V = (1.25)(11.4) = 14.2 \text{ fps}$$

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. Example 2 examines the case where the total flow goes through the bridge.

Abutment protection - The projected river alignment changes indicate the need for protection on the right embankment of the crossing as shown in Fig. 8.8.8. The left abutment already has a protective dike that has withstood flows in excess of 100,000 cfs.

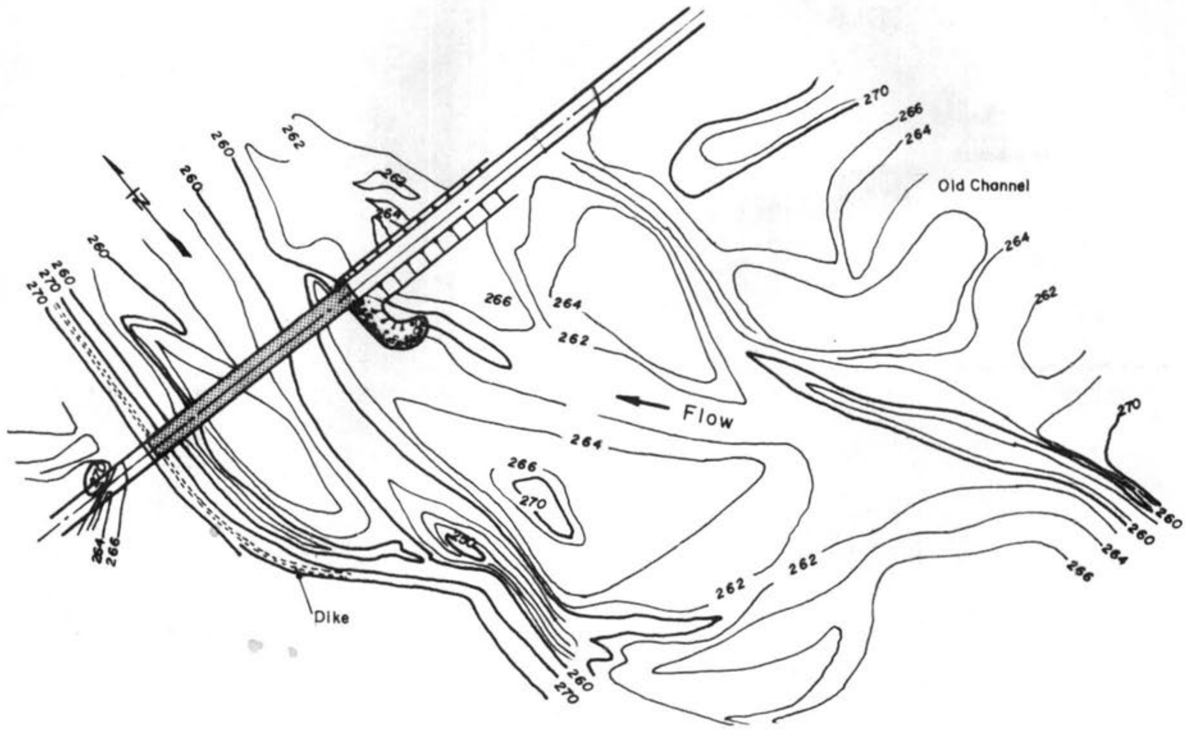


Fig. 8.8.8 Right embankment and spur dike

A spur dike is ideally suited for this embankment end. While this reach of river is in the braided form, the spur will help direct the flow from the right side of the channel through the bridge opening. After the river has narrowed, the spur will protect the embankment by holding the meander loops away from the embankment. In addition the spur will move the local scour away from the embankment end under the bridge to the nose of the spur.

The recommended spur dike configuration is shown in Fig. 6.3.6. The U.S. practice is to limit the length of the spur dike to 150 or 200 ft. Here, a length of 200 ft is chosen because the spur dike will have to direct a substantial amount of channel flow while the approach river channel is braided. The spur bends outwards on a 1/4 ellipse and is terminated at (0.4)(200) = 80 ft back from the embankment end. The plan view of the spur is shown in Fig. 8.8.9.

The top of the spur 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. Fig, 8.8.8 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, rock filled trench of riprap has been provided (see Fig. 8.8.9) that will stop any local pockets of scour.

The size of riprap required for the spur can be determined by the method in section 6.4.0 of Chapter VI. The computations are summarized in Table 8.8.1. As the embankments for this bridge do not constrict the river flow after the river narrows (relief bridges are provided on the floodplain), the flow through the bridge will not accelerate. Therefore, this is a case of horizontal flow on a side slope.

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 spur. Therefore

$$\theta = 18.4°$$

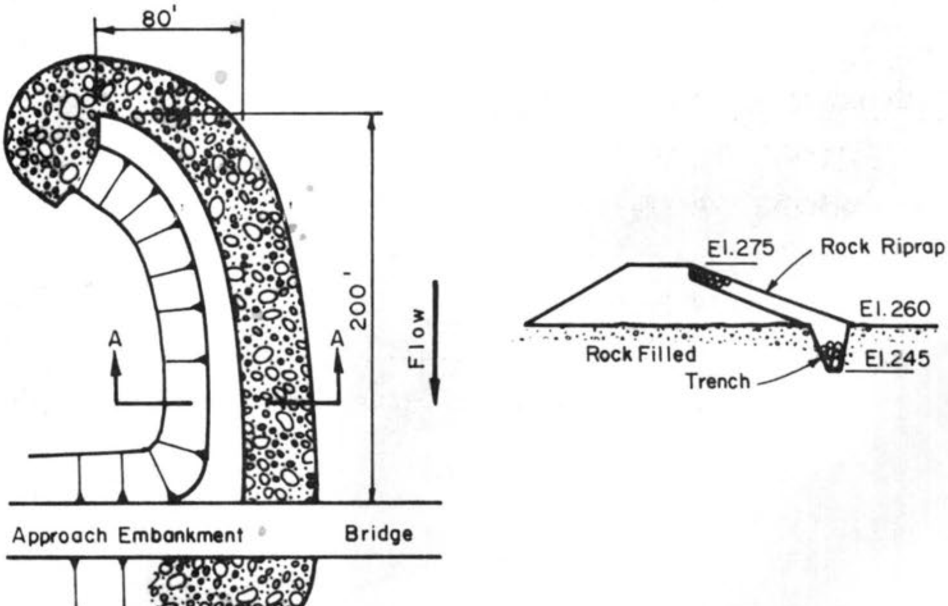


Fig. 8.8.9 Spur dike with a rock trench

The anticipated size of riprap is in the range between 1.0 and 10 in. Then, from Fig. 3.7.3 the angle of repose for the dumped riprap will be approximately 37° or

$$\phi = 37°$$

For flow alignment as shown in Fig. 8.8.8, the average shear stress on the bed, as expressed by Eq. 3.9.12 is

$$\tau_o = \gamma\, y_o S_f$$

$$= (62.5)(11.6)(0.00138) = 1.0 \text{ psf}$$

where y_o is equal to R for wide floodplain crossing. The bed shear stress is equal to the side slope shear stress at the toe of the side slope. Therefore, a shear stress of 1.0 psf is used to design the side slope riprap.

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 Eq. 6.4.9, or

$$\eta = \{\frac{S_m^2 - (S.F.)^2}{(S.F.)S_m^2}\} \cos\theta$$

Here $$S_m = \frac{\tan\phi}{\tan\theta}$$

$$S_m = \frac{\tan 37°}{\tan 18.4°} = 2.27$$

The recommended safety factor for design is (Secton 6.A5.0)

$$S.F. = 1.5$$

so

$$\eta = \{\frac{(2.27)^2 - (1.5)^2}{(1.5)(2.27)^2}\} \cos 18.4° = 0.356$$

The rock size is related to stability number by Eq. 6.4.5 or

$$D = \frac{21\tau_o}{(S_s-1)\gamma\eta}$$

$$= \frac{(21)(1.0)}{(2.50-1)(62.4)(0.356)} = 0.63 \text{ ft}$$

or

$$D \simeq 7\ 1/2 \text{ in}$$

The effective rock size of the required riprap is then 7 1/2 in.

The recommended gradation is given in Fig. 6.4.3. This gradation is such that

$$D = 1.25\ D_{50}$$

or

$$D_{50} = \frac{7.5}{1.25} = 6 \text{ in.}$$

and

$$D_{100} = 2D_{50}$$

or

$$D_{100} = (2)(6) = 12 \text{ in.}$$

The recommended minimum thickness of the riprap blanket is $2D_{50}$ or D_{100} which ever is greater. Here, use a minimum thickness of 12 in.

When 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. Assuming that y_o/D is 23, Eq. 6.A3.7 indicates the shear stress on the side slope will be approximately twice as much as in the crossing. That is

$$\tau_o \simeq (2)(1.0) = 2.0 \text{ psf.}$$

The stability number for the 6 in. diameter riprap (D = 7.5 in.) on a 3:1 side slope with a shear stress of 2.0 psf is given by Eq. 6.4.5 or

$$\eta = \frac{(21)(2)}{(2.50-1)(62.4)(0.63)} = 0.712$$

and the safety factor becomes (from Eq. 6.4.11)

$$S.F. = \frac{S_m}{2}\{(S_m^2\,\eta^2\sec^2\theta + 4)^{1/2} - S_m\,\eta\sec\theta\}$$

or

$$S.F. = \frac{S_m}{2}\{(\xi^2 + 4)^{1/2} - \xi\}$$

where $\xi = S_m\,\eta\,\sec\,\theta$

$$= (2.27)(0.712)(1.054) = 1.704$$

Then $$S.F. = \frac{2.27}{2}\ \{[(1.704)^2 + 4]^{1/2} - 1.704\}$$

$$= 1.048$$

As the safety factor is only slightly greater than unity, the riprap could move when flow in the meander bend attacks the spur protection.

The size of riprap required to give a 1.5 safety factor for the riprap under a shear stress of 2.0 psf is found by employing Eq. 6.4.9 and 6.4.5 in order. Accordingly, the stability factor is still

$$\eta = 0.356$$

but

$$D = \frac{(21)(2.0)}{(2.50-1)(62.4)(0.356)} = 1.26 \text{ ft}$$

or

$$D = 15 \text{ in.}$$

which corresponds to

$$D_{50} = 12 \text{ in.}$$

and

$$D_{100} = 24 \text{ in.}$$

Future river alignment conditions dictate that the riprap on the abutment have a D 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 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 4 in. (D_{50}) in diameter.

Scour at the piers - The scour to be expected around the piers can be computed with the equations in Section 6.5.3, Chapter VI. For the present-day conditions at the bridge crossing the approach flow depth is

$$y_1 = y_o = 11.6 \text{ ft}$$

and the corresponding Froude number is (from Table 8.8.1)

$$Fr_1 = 0.59$$

The pier width a is five ft and the length ℓ is 20 ft. If the pier skew angle is zero deg, then the equilibrium depth of scour is given by Eq. 6.5.11 (round nose pier) or

$$y_s = 2.0 y_1 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\,0.43}$$

$$= (2.0)(11.6)\left(\frac{5.0}{11.6}\right)^{0.65}(0.59)^{0.43} = 10.7 \text{ ft}$$

According to the plan view on Mainstream River shown in Fig. 8.8.7 a pier skew angle (to the approach flow) of 30 deg can be anticipated. From Table 6.5.2, 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

$$y_s = (10.7)(2.0) = 21.4 \text{ ft}$$

The maximum depth of scour can be 30 percent greater (Section 6.5.3) so the maximum depth of scour is

$$y_s = (21.4)(1.3) \simeq 28 \text{ ft}$$

The greatest contributor 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

$$272-11.6-28 \approx 232 \text{ ft}$$

This pier scour would be obtained if the bed material were sand only. However, the scour hole armorplates if there is any gravel or cobbles in the underlying bed deposits. The depth of scour would be decreased if there is underlying hardpan. 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 gravel and a few cobbles in the underlying bed material. The scour hole will armor plate so that the scour hole bed elevation will be greater than El. 232 ft.

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 as El. 243 ft. (Table 8.8.1). 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 8.8.1)

$$y_1 = 29 \text{ ft}$$

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 Eq. 6.5.11

$$\begin{aligned} y_s &= (2.0)(29)\left(\frac{5}{29}\right)^{0.65}(0.46)^{0.43} \\ &= 13.2 \text{ ft} \end{aligned}$$

Again, the skew angle could be as great as 30° so the multiplying factor is 2.0 (Table 6.5.2) and the depth of scour is

$$y_s = (2.0)(13.2) = 26.4 \text{ ft}$$

and the maximum depth of scour is

$$y_s = (26.4)(1.3) \approx 34 \text{ ft}$$

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* the pier foundations be set at an elevation of 200 ft.

8.8.3 Design example 2

In example 1, a far reaching assumption was made that the overbank flood flow passed under the highway through relief bridges so that, of 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 example 1 will affect the river behavior. In design example 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. The increased flow also causes a local backwater effect that increases the stage under the bridge and decreases the clearance. Fortunately, the design flood is not long lasting as shown in the design hydrograph of Fig. 8.8.2. This hydrograph was adapted from the flood of record measured at the USGS gage simply by multiplying those flows by a constant to get a peak of 110,000 cfs. 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. Fig. 8.8.10 shows a schematic drawing

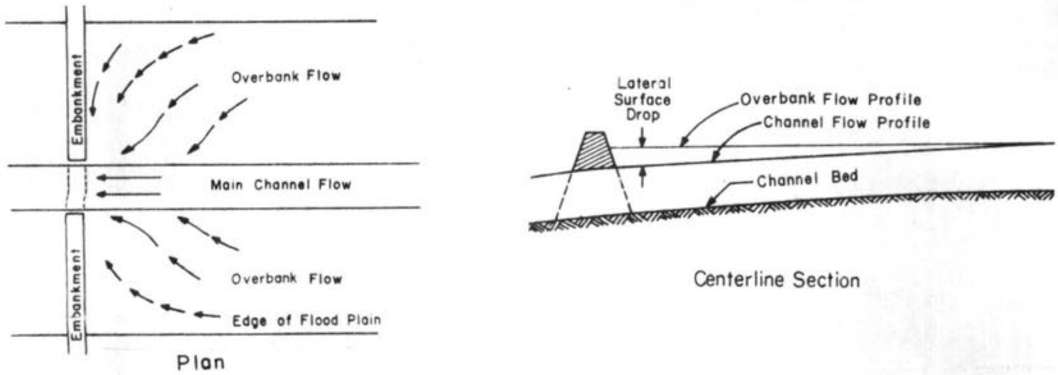


Fig. 8.8.10 Schematic of overbank and main channel flow

of the overbank flow being impounded by a highway embankment and returning to the river. At the embankment, there is not overbank flow in the downstream direction and the slope of the longitudinal profile is necessarily level there. The profile gradually picks up slope until it parallels the main channel slope. The difference in the two profiles 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.

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 110,000 cfs?

The unit discharge is

$$q = \frac{Q}{W} = \frac{110{,}000}{500} = 220 \text{ cfs/ft}$$

Assume the friction slope is

$$S_f = 0.00138$$

With the higher discharge, the bed should be plane (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 Eq. 2.3.20

$$y_o = \left\{\frac{(220)(0.015)}{(1.486)(0.00138)^{1/2}}\right\}^{3/5} = 11.6 \text{ ft.}$$

and the average velocity is

$$V = \frac{q}{y_o} = \frac{220}{11.6} = 19.0 \text{ fps}$$

The average shear stress on the bed is (from Eq. 2.3.31)

$$\tau_o = \gamma y_o S_f$$
$$= (62.4)(11.6)(0.00138) = 1.0 \text{ psf}$$

and the stream power is

$$\tau_o V = (1.0)(19.0) = 19.0 \text{ ft lb/sec/sq ft.}$$

For 1 mm sand and this stream power, Fig. 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 general type (see Section 6.5.2) and Laursen's equation (Eq. 6.5.4) is applicable. The equation is

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{\frac{6(2+f)}{7(3+f)}} \left(\frac{n_2}{n_1}\right)^{\frac{6f}{7(3+f)}}$$

In this case, Q_t is the total design flood discharge or

$$Q_t = 110{,}000 \text{ cfs}$$

Q_c is the flow in the main channel upstream of the bridge or

$$Q_c = 66{,}000 \text{ cfs}$$

W_1 and W_2 are the approach channel and contracted channel reaches. Here

$$W_1 = W_2 = 500 \text{ ft}$$

The term f is dependent on the approach channel shear velocity and the fall velocity of the bed material. The approach channel shear velocity is

$$V_{*c} = \sqrt{\frac{\tau_1}{\rho}}$$

Here τ_1 is the average bed shear in the upstream channel and is computed from Eq. 2.2.31 or

$$\tau_1 = \gamma y_1 S_f$$

In the upstream approach channel the average depth is (from design example 1)

$$y_1 = 11.6 \text{ ft}$$

and

$$S_f \simeq 0.00138$$

Then $\tau_1 = (62.4)(11.6)(0.00138) = 1.0 \text{ psf}$

and

$$V_{*c} = \left(\frac{1.0}{1.94}\right)^{1/2} = 0.72 \text{ fps}$$

The median diameter of the bed material (sieve analysis) is

$$D_{50} = 1.0 \text{ mm}$$

Assume a shape factor of 0.7 (normal for sands) so that the fall velocity is (from Fig. 3.7.2)

$$\omega \simeq 12 \text{ cm/sec} = 0.39 \text{ fps}$$

so that

$$\frac{V_{*c}}{\omega} = \frac{0.72}{0.39} = 1.85$$

With this value of v_{*c}/ω, the value of f can be found in the table accompanying Eq. 6.5.4. Use

$$f \simeq 2.0$$

Upstream of the bridge, Manning's n for the main channel is 0.025 (from design example 1) 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 general scour scould 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

$$Fr = \frac{14.7}{\sqrt{(32.2)(15)}} = 0.67$$

In reality, flow conditions in the downstream reach have a Froude number somewhere between the Froude number 0.67 for the general scour assumption and the Froude number 1.02 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 general scour occurs. If the rise time is very short, the scour at the peak of the flood is much less than the general 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 Fig. 8.8.2, 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 (see Section 2.4.0) 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 designer 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 design example 1 or 2 are complete. At this stage the designer 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 designer would need to continue the analysis outlined in the two examples and in addition would need to make a backwater analysis on the most feasible designs before going to final design.

8.8.4 Design example 3

Design example 3 is concerned with degradation of the channel. The same crossing and the same 500-ft bridge as in design example 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 100-year design flood is now only 40,000 cfs. The natural flow of sediment in the river has also been checked 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 10,000 cfs, ignoring the periods when the flows are very low. According to Fig. 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 and this effect may be estimated by Eq. 4.4.6 which is

$$W \sim Q^{0.26}$$

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 at a flow of 40,000 cfs is computed from Manning's equation (Eq. 2.3.20) or

$$y_o = \left\{\frac{Vn}{1.486 S_f^{1/2}}\right\}^{3/2}$$

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

$$q = \frac{Q}{W} = \frac{40{,}000}{550} = 72.7 \text{ cfs/ft}$$

The large D_{90}, 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 1. Therefore, from our experience in working design examples 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 is (from Eq. 2.3.31)

$$\tau_o = \gamma\, y_o S_f$$

$$= (62.4)(8.7)(.00138) = 0.75 \text{ psf}$$

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

$$268-9 = 259 \text{ ft}$$

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 shear 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 (Fig. 3.2.3). Assume the flow is fully turbulent at the bed. Then

$$\frac{V_* D}{\eta} \geq 400$$

$$\frac{\tau_c}{(\gamma_s - \gamma) D_{90}} = \frac{\tau_c}{(S_s - 1)\gamma D_{90}} = 0.047$$

and

$$\tau_c = (0.047)(2.65-1)(62.4)\left(\frac{15}{304.8}\right)$$

$$= 0.24 \text{ psf.}$$

It will 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 Eq. 3.10.46

$$n = 0.04\ D_{90}^{1/6}$$

$$= (0.04)\left(\frac{15}{304.8}\right)^{1/6} = 0.024$$

Because there will be some form roughness from the gravel bars use

$$n = 0.028$$

Manning's equation (Eq. 2.3.20)

$$q = \frac{1.486}{n}\ y^{5/3} S_f^{1/2}$$

Here we know q, and n. Because

$$\tau_c = \gamma y S_f = 0.24$$

Then

$$S_f = \frac{0.24}{62.4y} = \frac{0.00385}{y}$$

Put this expression for S_f in Manning's equation so that

$$18.2 = \frac{1.486}{0.028}\ (0.062) y^{7/6}$$

or $\quad y = 4.3$ ft

It follows that

$$S_f = \frac{0.00385}{4.3} = 0.00090$$

$$\approx 4.8 \text{ ft/mile}$$

The slope before degradation is 7.3 ft/mile.

The existing profile and the ultimate profile are shown in Fig. 8.8.11 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

$$y_s = (9)(7.3\text{-}4.8) \approx 22 \text{ ft}$$

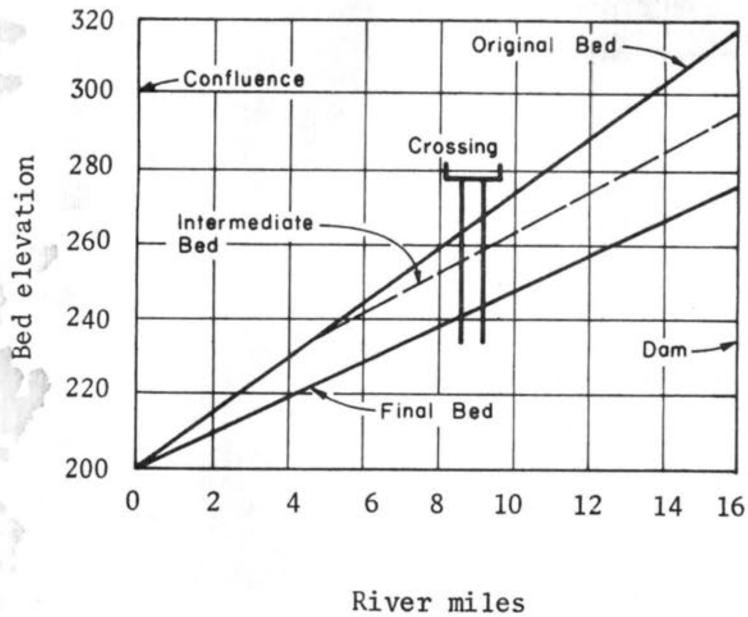


Fig. 8.8.11 Degradation due to dam upstream of the crossing

This represents the removal of a substantial amount of material and in the meantime and 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 Fig. 3.10.10 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 fps. Using Fig. 3.10.10 for 1 mm sand gives a transport rate of approximately 35 tons per day of per foot of width, but since the release is for 6 hours only, the actual transport in Mainstream river is (6/24)(130) = 9 tons per day per ft of width. If the bed material has a dry weight of 100 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 0.004 ft per day or about 1.6 ft per year. When degradation has progressed to the confluence 9 miles downstream, the rate will be about one-half this or 0.002 ft per day Thus degradation proceeds at an ever decreasing rate.

Table 8.8.2 Data and computations, Design example 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 = 15 mm
	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

8.8.5 Design example 4

In this example the problem of aggradation will be considered. A dam is located eight miles downstream of the proposed crossing. The present width of the river at the proposed crossing is 1000 feet as shown in Fig. 8.8.1. The reservoir level is at Elev. 250 during the winter with summer drawdowns as low as Elev. 240. The design flood level of the reservoir is Elev. 260. The effects of the reservoir are to increase the design flood stage and to cause general aggradation of the bed at the proposed crossing. The aggradation is considered because the design flood stage is superimposed on the aggraded profile.

Aggradation occurs as the river flow slows down upon entering the reservoir and deposits its sediment load. The method of deposition is usually complex in rivers with fine sediment load, but in coarse sandbed rivers such as the Mainstream River the sediment is largely deposited at the entrance of the reservoir in a delta formation. The top slope of the deposit formation is generally concave as shown in Fig. 8.8.12. At the upstream end of the deposition the river retains its normal bed slope and sediment transport rate. The sediment carrying capacity of the river decreases downstream until it is almost zero at the end of the delta. Point a in Fig. 8.8.12 must remain at the same elevation as the delta moves into the reservoir. The aggraded reach lengthens until the reservoir is filled with sediment and sediment starts to pass

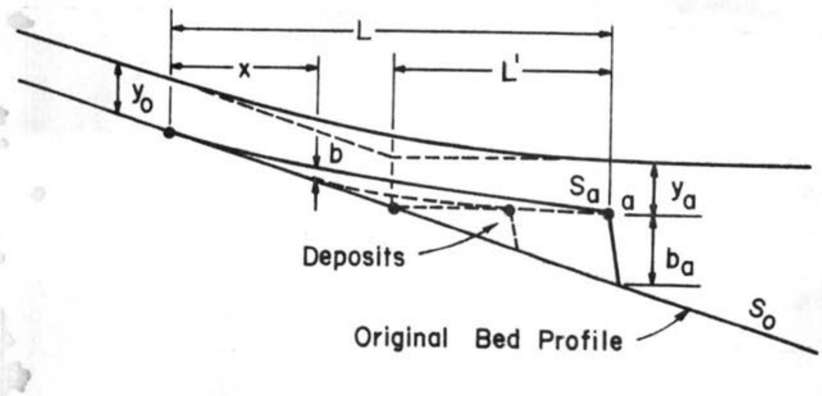


Fig. 8.8.12 Definition diagram for aggradation from deposition in a reservoir

out with the release water. As with degradation, the increase in the bed elevation proceeds at a decreasing rate as the length of the aggraded reach grows.

A simplified evaluation of the amount of aggradation at any point can be made by assuming (1) that the slope at the end of the delta is the maximum slope for zero sediment transport, (2) that the slope increases linearly until it reaches the natural slope of the river, and (3) that the width of the delta formation is constant. The last assumption is a gross oversimplification but it results in a conservative design.

If the slope S at a distance x from the upstream end of the aggrading reach varies linearly from S_o to S_a then

$$S = S_o - \frac{x}{L}(S_o - S_a) \qquad 8.8.1$$

where the terms are defined in Fig. 8.8.12. Then the deposit thickness b changes according to

$$db = (S_o - S)\,dx = \{S_o - [S_o - \frac{x}{L}(S_o - S_a)]\}dx$$

$$= \frac{x}{L}(S_o - S_a)\,dx \qquad 8.8.2$$

Integrating over length x yields

$$\int_o^b db = \int_o^x \frac{x}{L}(S_o - S_a)\,dx$$

or $$b = \frac{x^2}{2L}(S_o - S_a) \qquad 8.8.3$$

where the slopes are considered positive and L is the length of the aggrading reach. Then the volume of sediment per unit width in the deposit is given by

$$\int_o^L b\,dx = \int_o^L \frac{x^2}{2L}(S_o - S_a)\,dx$$

$$= \frac{L^2}{6}(S_o - S_a) \qquad 8.8.4$$

If sediment per unit width is supplied by the river at a rate q_s, and deposits with a specific weight γ_s, including voids, then the time to develop the volume of aggradation in Eq. 8.8.4 is

$$t = \frac{L^2}{6}(S_o - S_a)\frac{\gamma_s}{q_s} \qquad 8.8.5$$

The downstream face of the delta moves downstream as the depth of deposit increases to keep point a of Fig. 8.8.12 at a constant level. Thus

$$\frac{b_a}{L'} = S_o \qquad 8.8.6$$

where b_a can be found from Eq. 8.8.3 with $x = L$,

$$b_a = \frac{L}{2}(S_o - S_a) \qquad 8.8.7$$

Therefore during the time in which length L develops, the distance that the face of the delta moves downstream L' is given by

$$L' = \frac{b_a}{S_o} = \frac{L}{2}\,\frac{S_o - S_a}{S_o} \qquad 8.8.8$$

In this example, the mean five-month winter flow of 7000 cfs is considered to be the dominant discharge in the aggradation process. The river channel is 500 ft wide except at the crossing, the natural slope is 1.38 ft/1000 ft and the median material size is 1 mm. The data for design example 4 is given in Table 8.8.3. *The channel width in the flow and sediment transport problems is 500 ft as opposed to the 1000 ft at the crossing.* This is because the 1000 ft wide reach is a short reach which does not affect the large scale properties of the river.

The unit discharge in the channel is

$$q = \frac{Q}{W} = \frac{7000}{500} = 14 \text{ cfs/ft}$$

Assuming that Manning's n = .025, the depth upstream of the aggrading reach can be calculated from Manning's equation (Eq. 2.3.20)

$$y_o = \left\{\frac{qn}{1.486 S_f^{1/2}}\right\}^{3/5}$$

$$= \left\{\frac{(14)(.025)}{(1.486)(0.00138)^{1/2}}\right\}^{3/5}$$

$$= 3.0 \text{ ft}$$

Thus, the velocity becomes

$$V = \frac{q}{y_o} = \frac{14}{3.0} = 4.7 \text{ fps}$$

The shear stress on the boundary is given by Eq. 2.3.30

$$\tau_o = \gamma R S_o$$

$$\simeq \gamma\, y_o\, S_o = (62.4)(3.0)(.00138) = .26 \text{ psf}$$

The stream power becomes

$$\tau_o V = (.26)(4.7) = 1.21 \text{ ft lb/sec/sq ft}$$

With a median sieve diameter of 1.0 mm and the computed stream power Fig. 3.4.1 can be used to determine the probable bed form. The bed would be in transition and from Table 2.3.1 (Alluvial sandbed channels), the assumed Manning's n of .025 is acceptable.

The slope for zero transport must be computed assuming the shear stress remains constant throughout the aggrading reach. The shear velocity can be written

$$V_* = (\frac{\tau_o}{\rho})^{1/2} = (\frac{.26}{1.94})^{1/2} = .37 \text{ fps}$$

by definition. Assuming $\nu = 1.3 \times 10^{-5}$ ft^2/sec, the grain size Reynolds number can be computed by

$$\frac{V_* D}{\nu} = \frac{(.37)(1)(.00328)}{(1.3)(10^{-5})} = 93$$

Referring to Shields' diagram (Fig. 3.2.3), for a grain size Reynolds number of 93, the dimensionless shear stress for incipient motion is .04. That is,

$$.04 = \frac{\tau_c}{(\gamma_s - \gamma)D}$$

or $$\tau_c = .04\ (\gamma_s - \gamma)D$$

$$= .04\ (165.4 - 62.4)(.00328) = .014 \text{ psf}$$

This value of shear stress should now be used to get the shear velocity, Reynold's number for the grain and the Shields parameter as follows

$$V_* = (\frac{\tau_o}{\rho})^{1/2} = (\frac{.014}{1.94})^{1/2} = 0.85 \text{ fps}$$

$$\frac{V_* D}{\nu} = \frac{(.085)(.00328)}{(1.3)(10^{-5})} = 21$$

and $$.031 = \frac{\tau_c}{(\gamma_s - \gamma)D}$$

Thus

$$\tau_c = .031\ (165.4 - 62.4)(.00328) = .010 \text{ psf}$$

Using Eq. 2.2.30 for the critical shear stress on the bed and Manning's equation (Eq. 2.3.20) for the critical transport conditions prevailing, the depth and slope at the end of the delta can be obtained as follows:

$$\tau_c = \gamma y_o S_c$$

and
$$q = \frac{1.486}{n} y_o^{5/3} S_c^{1/2}$$

or
$$q = \frac{1.486}{n} y_o^{5/3} \left(\frac{\tau_c}{\gamma y_o}\right)^{1/2}$$

$$= \frac{1.486}{n} y_o^{7/6} \left(\frac{\tau_c}{\gamma}\right)^{1/2}$$

In solving the above equation for the depth at the end of the delta, it is assumed that the delta is wider than the width of the river. Inspection of maps and aerial photos shows that the average width of the delta is approximately 1000 ft. Thus the unit discharge becomes

$$q = \frac{Q}{W} = \frac{7000}{1000} = 7 \text{ cfs/ft}$$

It will also be assumed that Manning's n is about 0.015 since the bed forms at the end of the delta should be small or nonexistent. Thus, the depth at the end of the delta becomes

$$y_o = \left\{\frac{n\, q\, \gamma^{1/2}}{1.486\, \tau_c^{1/2}}\right\}^{6/7}$$

$$= \left\{\frac{(.015)(7)(62.4)^{1/2}}{(1.486)(.010)^{1/2}}\right\}^{6/7} = 4.37 \text{ ft}$$

The velocity becomes

$$V = \frac{q}{y_o} = \frac{7}{4.37} = 1.6 \text{ fps}$$

Checking the assumption of n, the stream power is

$$\tau V = (.010)(1.6) = .016 \text{ ft lb/sec/sq ft}$$

According to Fig. 3.4.1, with a median fall diameter of approximately 1.0 mm, the bed form is flat with no transport. Table 2.3.1 (Alluvial sandbed channels) shows that .015 for Manning's n is a good selection.

The slope for zero sediment transport at the downstream end of the delta is determined from Eq. 2.2.30

$$\tau_c = \gamma y_o Sc$$

or $$S_c = \frac{\tau_c}{\gamma y_o} = \frac{.010}{(62.4)(4.37)} = .000037$$

Thus, the slope at the end of the delta is .037 ft/1000 ft (.19 ft/mile).

Table 8.8.3 Data for design example 4

Hydraulics for natural river:

Slope, S_o = .00138(7.29 ft/mile)
Width, W = 500 ft
Width (at crossing), W_c = 1000 ft
Mean river flow, Q = 7000 cfs
Unit discharge, q = 14 cfs/ft
Manning's n, n = .025
Depth of flow, y_o = 3.0 ft
Velocity, V = 4.7 fps
Boundary shear stress, τ_o = .26 psf

Hydraulics for no transport:

Critical shear, τ_c = .010 psf
Unit discharge, q = 7 cfs/ft
Manning's n = 0.015
Depth of flow, y_o = 4.37 ft
Velocity, V = 1.6 fps
Slope, $S_c = S_a$ = .000037 (.19 ft/mile)

Sediment transport:

Rate of transport, q_T = 50 tons/day/ft
= 3,750,000 tons/yr.
Transport deposited on delta, q_s = 3.750 tons/yr/ft
Specific weight of deposit, γ_s = 100 pcf

Profile dimensions when the delta reaches the dam:

Distance from crossing to the dam = 42,000 ft
Bed elevation at start of delta = El. 246 ft
Distance from crossing to start of delta = 10,000 ft
Movement required of delta = 32,000 ft
Equivalent length of aggraded reach, L = 66,000 ft
Time required to deposit material, t = 13 yrs
Distance from beginning of deposit to the crossing = 24,000 ft
Depth of deposit at the crossing, b = 5.8 ft

Tabulated results of bed profile:

Distance x mi	Distance x ft	Original Bed El., ft	Deposition ft	Aggraded Bed El., ft	Aggraded Bed Slope
0	0	292	0	292	.00138
2.85	15,000	272	2	274	.00108
5.70	30,000	251	9	260	.00077
8.50	45,000	230	20.5	250.5	.00047
11.40	60,000	209	36.5	245.5	.00016
12.50	66,000	201	45	246	.00004

It is assumed that the delta initiates at that point at which the bed is 4.37 ft below the normal operating level of Elev. 250. The basis of this assumption is that the reservoir fills rapidly to the operating level and that 4.37 ft is the depth required for deposition. The depth for deposition is less than this due to the slope being the average bed slope at the initiation of the delta. Thus, use 4.0 ft, resulting in a bed Elev. 246 at the initiation of the delta. With a bed elevation of 260 ft at the crossing, the point at which the delta begins is

$$\Delta x = \frac{\Delta y}{S_o} = \frac{260 - 246}{.00138} \approx 10,000 \text{ ft}$$

downstream of the crossing. *The dam is 8 miles or 42,000 ft downstream of the crossing.* Thus the delta must move a distance

$$L' = 42,000 - 10,000 = 32,000 \text{ ft}$$

before filling the reservoir. The length of the aggrading reach when the reservoir is filled is determined from Eq. 8.8.8

$$L = 2L' \frac{S_o}{(S_o - S_a)} = (2)(32,000) \frac{(.00138)}{(.00138 - .000037)} = 66,000 \text{ ft}$$

The time required to fill the reservoir can be obtained from Eq. 8.8.5 if the sediment discharge is known.

The sediment transport can be determined using Colby's method (see Section 3.10.13). The Mainstream River carries very little fine sediment and thus $k_2 = 1.0$. Assuming a temperature of 60°F, $k_1 = 1.0$. Thus Eq. 3.10.43 reduces to

$$q_T = [1 + (k_1 k_2 - 1)k_3]\, q_n$$

$$= q_n$$

Thus, even though k_3 is different from 1.0, it does not enter into the calculations. The rate of transport can be obtained from Fig. 3.10.10. For the 1.0 foot depth and $V = 4.7$ fps, $q_n = 42$ tons/day/ft of width. For a 10.0 foot depth, $q_n = 60$ tons/day/ft of width. Using logarithmic interpolation, $q_n = 50$ tons/day/ft of width for a depth of $y_o = 3$ ft. This is equivalent to 7500 tons/ft of width for the 5 months of high flow per year or 3,750,000 tons/year. This amount of material is deposited on the delta, which was assumed to average 1000 ft wide. Thus, the unit sediment discharge depositing on the delta is 3,750 tons/year/ft of width (7,500,000 lbs/year/ft).

If it is assumed that the material deposits with a specific weight of $\gamma_s = 100$ pcf (a good assumption for coarse sediment) Eq. 8.8.5 gives

$$t = \frac{L^2}{6} (S_o - S_a) \frac{\gamma_s}{q_s}$$

$$= \left(\frac{(66{,}000)^2}{6}\right)(.00138 - .000037)\left(\frac{100}{7{,}500{,}000}\right)$$

$$= 13 \text{ years}$$

as the time required to fill the reservoir.

The aggrading bed will affect the design discharge stage at the proposed bridge site. The depth of aggradation at the proposed crossing can be obtained from Eq. 8.8.3. In this equation, x is the distance from the upstream end of the aggrading section (66,000 ft from the dam) to the crossing (42,000 ft upstream from the dam). Thus,

$$x = 66{,}000 - 42{,}000 = 24{,}000 \text{ ft}$$

and
$$b = \frac{x^2}{2L}(S_o - S_a)$$

$$= \frac{(24{,}000)^2}{(2)(66{,}000)}(.00138 - .000037) = 5.8 \text{ ft}$$

The depth of aggradation at the proposed crossing is 5.8 feet, raising the bed elevation to approximately 266 feet. The effect of this on the flood stage must now be considered.

The flood stage can be computed by a backwater calculation for the 110,000 cfs design flood over the aggraded profile. Several problems arise because of the fact that aggradation has filled in most of the channel and much of the design flood will pass over the floodplain. In this example we will consider the floodplain to be 4000 ft in width, virtually flat normal to the river and at a level 15 ft above the original bed. A reasonable assumption for this example is that the design flood is restricted to a 1000 ft wide channel at the crossing but begins immediately to spread out into the floodplain downstream of the crossing. Also it can be assumed that the spreading is complete and the flow is uniformly distributed across the floodplain at the point c where delta formation rises above the floodplain. This occurs when the depth of aggradation equals 15 feet, the assumed level of the floodplain above the original bed level. The location of point c can be obtained from Eq. 8.8.3

$$b = \frac{x^2}{2L}(S_o - S_a)$$

or
$$x = \left\{\frac{2\,bL}{S_o - S_a}\right\}^{1/2}$$

$$= \left\{\frac{(2)(15)(66{,}000)}{(.00138 - .000037)}\right\}^{1/2} = 38{,}500 \text{ ft}$$

Thus, point c is 66,000 - 38,500 = 27,500 ft upstream from the dam. In this section, *where the flow is spreading, the total design flood should be divided into channelized flow and floodplain flow and separate backwater curves computed for each.* However, provision must be made, to see that there is flow from the channel to the floodplain in just the correct amounts to produce backwater profiles curves that are identical except for the small lateral gradients needed to accomplish this lateral flow. Thus the problem becomes complex.

A simplification would be to arbitrarily establish the rate of lateral movement and then compute the backwater curve for the resulting channel flow. Different patterns of lateral flow could be assumed to see how sensitive the stage at the crossing is to this variable, a reasonable approach for computer work. *Herein, the assumption is that the rate of lateral flow is constant along the reach from the crossing to point* c. Then the unit discharge would vary linearly from 110 cfs per foot width at the crossing to 27.5 cfs per foot width at point c and remain at that value to the dam.

The backwater problem to be solved is one in which both unit discharge and bed slope are changing with distance x, complications not existing in the theoretical development of the backwater equation given in Section 2.7.3. Variables are functions of both x and y and the direct method of specifying Δy and computing Δx is not available. It is assumed that *the discharge over a given step can be considered uniform and equal to the average discharge in the reach,* Eq. 2.7.7 can be applied in a trial and error solution. This solution is shown in Table 8.8.5. It is further assumed in this table that the energy gradient in a given step is the same as occurring for the average flow conditions in the reach taken as steady uniform flow. So, in Eq. 2.7.7 $n = n_o$. Table 8.8.5 shows a step method in which Δx are specified rather than Δy. The dam face is at $x = 66,000$ ft, the point c is at $x = 38,500$ ft and the crossing is at $x = 24,000$ ft. The reservoir pool is at El. 260 and the aggraded bed is at El. 246 at the dam so that the depth y at the dam is 14 ft.

The computational procedure in Table 8.8.4 is as follows:

(1) Station 1 is assumed to be the location of the dam at which $x = 66{,}000$ ft and $y = 14.00$ ft are given.

(2) Choose an increment of length, Δx (the smaller the increment, the more accurate is the solution) and determine the upstream position x_{us} by

$$x_{us} = x_{ds} + \Delta x$$
$$= 66{,}000 - 5000 = 61{,}000 \text{ ft}$$

where x_{us} = the position of the upstream end of the reach
x_{ds} = the position of the downstream end of the reach
Δx = length of the reach chosen.

(3) Compute the average x over the reach:

$$x_{ave} = \frac{1}{2}(x_{us} + x_{ds}) = x_{ds} + \frac{1}{2}\Delta x$$
$$= \frac{1}{2}(61{,}000 + 66{,}000)$$
$$= 63{,}500 \text{ ft}$$

(4) Compute the average slope over the reach using Eq. 8.8.1:

$$S_{ave} = S_o - \frac{x_{ave}}{L}(S_o - S_a)$$
$$= .00138 - \frac{63{,}500}{66{,}000}(.00138 - .000037)$$
$$= .000091$$

where S_o = natural bed slope,
S_a = bed slope at the end of the delta
L = length of the aggradation.

(5) Compute the average discharge over the reach q_{ave}. The average discharge is 27.5 cfs/ft until the point is reached at which the flow is spreading onto the floodplain (x = 38,500 ft) after which q_{ave} can be computed using

$$q_{ave} = 27.5 \text{ cfs/ft} + \left(\frac{38{,}500 - x_{ave}}{38{,}500 - 24{,}000}\right)(110.0 - 27.5)$$
$$= 27.5 \text{ cfs/ft} + \left(\frac{38{,}500 - x_{ave}}{14{,}500}\right)(82.5 \text{ cfs/ft})$$

in which q = 27.5 cfs/ft denotes the unit discharge downstream of complete spreading,

Table 8.8.4 Backwater computations for example 4

Sta.	x ft	y ft	Δx ft	x_{ave} ft	S_{ave} ft/1000 ft	q_{ave} cfs/ft	$y_{o_{ave}}$ ft	$y_{c_{ave}}$ ft
1	66,000	14.00						
			-5,000	63,500	.0878	27.5	10.36	2.84
2	61,000	13.73						
			-5,000	58,500	.1896	27.5	8.22	2.84
3	56,000	12.98						
			-3,000	54,500	.2710	27.5	7.39	2.84
4	53,000	12.31						
			-3,000	51,500	.3321	27.5	6.95	2.84
5	50,000	11.49						
			-3,000	48,500	.3931	27.5	6.61	2.84
6	47,000	10.54						
			-3,000	45,500	.4541	27.5	6.33	2.84
7	44,000	9.51						
			-2,000	43,000	.5050	27.5	6.13	2.84
8	42,000	8.48						
			-2,000	41,000	.5457	27.5	5.99	2.84
9	40,000	7.56						
			-1,500	39,250	.5813	27.5	5.88	2.84
10	38,500	7.12						
			-1,500	37,750	.6118	31.77	6.31	3.12
11	37,000	6.87						
			-2,000	36,000	.6475	41.72	7.31	3.74
12	35,000	7.10						
			-2,000	34,000	.6882	53.10	8.29	4.38
13	33,000	7.81						
			-3,000	31,500	.7390	67.33	9.36	5.13
14	30,000	9.01						
			-3,000	28,500	.8000	84.40	10.46	5.95
15	27,000	10.10						
			-3,000	25,500	.8611	101.47	11.43	6.72
16	24,000	11.10						
			-3,000	22,500	.9221	110.00	11.76	7.09
17	21,000	11.57						
			-3,000	19,500	.9832	110.00	11.53	7.09
18	18,000	11.55						
			-3,000	16,500	1.044	110.00	11.33	7.09
19	15,000	11.39						
			-3,000	13,500	1.105	110.00	11.13	7.09
20	12,000	11.20						
			-3,000	10,500	1.166	110.00	10.96	7.09
21	9,000	11.01						
			-3,000	7,500	1.227	110.00	10.79	7.09
22	6,000	10.82						
			-3,000	4,500	1.244	110.00	10.75	7.09
23	3,000	10.76						
			-3,000	1,500	1.345	110.00	10.49	7.09
24	0	10.51						

q = 110.0 cfs/ft denotes the unit discharge upstream of the initiation of spreading which occurs at the bridge site,

x = 38,500 ft indicates the location of the completion of the spreading of the flow, and

x = 24,000 ft marks the location of the crossing which is where the spreading of the flow starts.

(6) Compute the normal depth for the reach $y_{o_{ave}}$ using Manning's equation (Eq. 2.3.20):

$$y_{o_{ave}} = \left\{\frac{n}{1.486} \frac{q_{ave}}{S_{ave}^{1/2}}\right\}^{3/5}$$

$$= \left\{\frac{.025}{1.486} \frac{2.75}{(.000091)^{1/2}}\right\}^{3/5}$$

$$= 10.27 \text{ ft}$$

(7) Compute the critical depth for the reach $y_{c_{ave}}$:

$$y_{c_{ave}} = \sqrt[3]{q_{ave}^2/g}$$

$$= \sqrt[3]{(27.5)^2/32.2}$$

$$= 2.86 \text{ ft}$$

(8) Assume a depth of flow at the upstream end of the reach y^*_{us} and compute the resulting average depth over the reach y^*_{avg}:

$$y^*_{avg} = \frac{1}{2}(y^*_{us} + y_{ds})$$

$$= \frac{1}{2}(14.00 + 14.00)$$

$$= 14.00 \text{ ft}$$

where y^*_{ave} = the average depth in the reach based on an assumed upstream depth,

y^*_{us} = the assumed upstream depth, and

y_{ds} = the known downstream depth

(9) Compute the quantities

$$\{1 - (\frac{y_{o_{ave}}}{y^*_{ave}})^{10/3}\} = \{1 - (\frac{10.27}{14.00})^{10/3}\} = 0.644$$

and

$$\{1 - (\frac{y_{c_{ave}}}{y^*_{ave}})^{3}\} = \{1 - (\frac{2.86}{14.00})^{3}\} = 0.992$$

(10) Compute the change in depth over the reach Δy using Eq. 2.7.7.

$$\Delta y = S_{ave}\Delta x \left\{\frac{1 - (\frac{y_{o_{ave}}}{y^*_{ave}})^{10/3}}{1 - (\frac{y_{c_{ave}}}{y^*_{ave}})^{3}}\right\}$$

$$= (.000091)(-5000)\ \{\frac{.644}{.992}\}$$

$$= -0.30 \text{ ft}$$

(11) Compute the upstream depth of flow y_{us} :

$$y_{us} = y_{ds} + \Delta y$$
$$= 14.00 + (-.30)$$
$$= 13.70 \text{ ft}$$

and check against the assumed value, y^*_{us}

$$y_{us} = 13.70 \neq y^*_{us} = 14.00$$

If the assumed depth and computed depth are approximately equal ($y^*_{us} \simeq y_{us}$), the computations for that reach are complete and y_{us} becomes the known downstream depth for the next reach.

Succeeding steps are performed in the same manner. In each case, the appropriate unit discharge and bed slope is determined first. Then the upstream depth is computed.

As shown in Fig. 8.8.13 *the backwater curve is complex*. From the dam (x = 66,000 ft) to x = 37,000 ft, the water surface profile is type M1 (see Table 2.7.1 and Fig. 2.7.2). Then from x = 37,000 ft to x = 20,000 ft, the water surface profile is type M2. The proposed crossing is in this reach. Upstream of station x = 20,000 ft, the backwater curve is again type M1. Normal depth is reached at x = 0 ft.

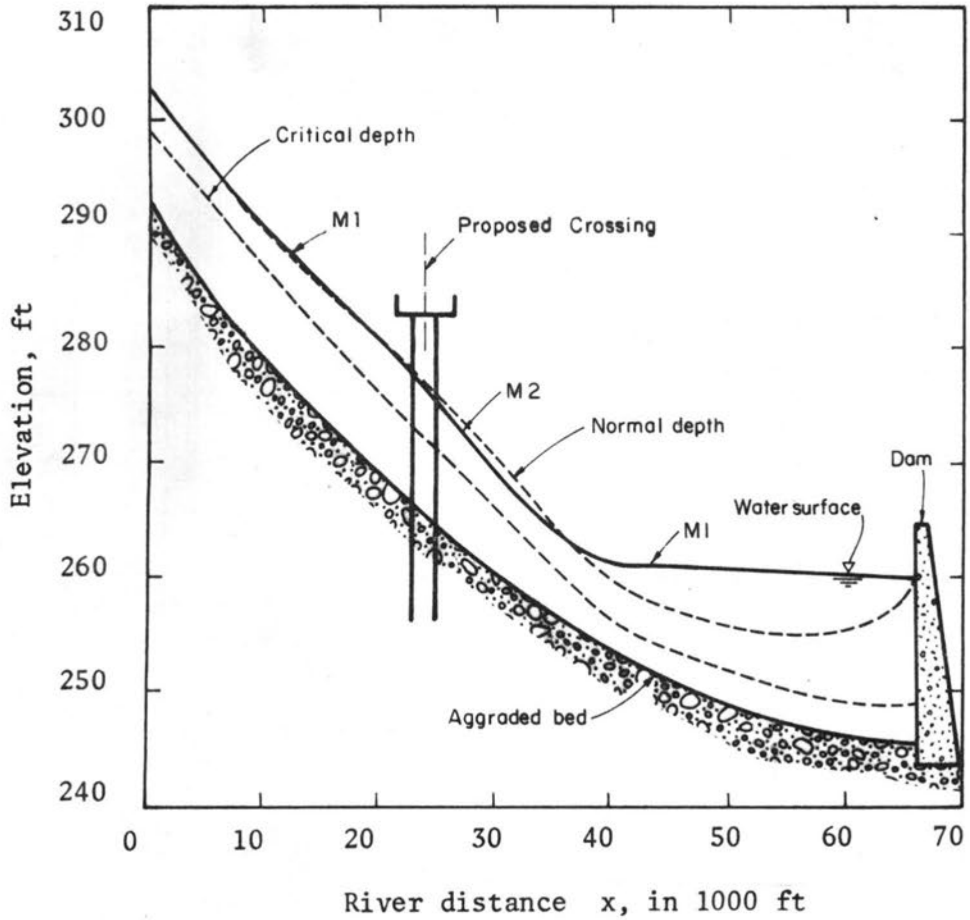


Fig. 8.8.13 Backwater with aggradation

The depth of flow at the crossing becomes 11 ft. Considering the original bed at the crossing was at El. 260, the aggradation at the crossing was computed to be 5.8 ft and the depth of flow is 11 ft, *the stage for the design flood becomes El. 277.* The resulting velocity at the crossing is 10 fps.

Considering the increased stage due to the backwater superimposed on the aggraded bed, resulting from the downstream dam, *the proposed bridge site is a poor one.* The river can become perched and very unstable in its location. If it is possible to route the highway 4 or more miles upstream, the problem of the increased stage can be alleviated.

ACKNOWLEDGMENTS

Many of the case histories of highways crossing rivers presented in this chapter have been taken from a report prepared by Joe W. Keeley titled "Bank Protection and River Control in Oklahoma;" Federal Highway Administration, Oklahoma Division, 1971. The other case histories were supplied to us by Mainard Wacker of the Wyoming State Highway Department.

The type of information in Keeley's report and Wacker's files is of great benefit to highway and river engineers. The information shows the many interactions between rivers and highway structures. It is proof that 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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