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Streambank Erosion Protection and Channel Scour Manipulation Using Rockfill Dikes and Gabions

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ABSTRACT

The uses of spur dikes made of rockfill and stone riprap and of gabion groins and gabion weirs for streambank erosion protection and streambed scour control are examined through literature review, laboratory model studies and field investigations.

The results of the literature review are summarized, including general design features, recommended spur dike and groin orientation angles, spacing/length ratios and local scour prediction equations.

Model studies are used to evaluate several spur dike and groin design parameters. The streamflow patterns and bed scour patterns associated with various arrangements of spur dikes and groins are used to determine which orientations for single structures and arrangements of multiple structures are best for protecting eroding banks and to manipulate scour patterns. A model study is also used to evaluate the flow and scour patterns caused by low V-shaped gabion weirs and to determine the relation between weir apex angle and the size and shape of the resulting scour hole. A model study is also used to examine a prototype spur dike arrangement, predict scour patterns, and evaluate several alternative arrangements of dikes for that same prototype river reach.

The principal conclusions from the model studies include: (1) the degree of bank protection provided by spur dikes and groins is a function of the structure length, orientation angle and spacing; (2) as structure length increases, the protected distance downstream increases, but not proportionately with the increasing structure length; the model dikes could protect a bank from two to five times their own length; (3) upstream-oriented structures are more effective than downstream-oriented structures,

with structures perpendicular to the flow intermediate in effectiveness, in deflecting the river current away from the bank and thus providing bank protection farther downstream from the structure tip; (4) upstream-oriented structures and normally-oriented structures cause more extensive scour holes than do downstream-oriented structures and may thus provide larger low-flow scour holes; (5) the V-shaped weir with its apex pointing upstream causes a large scour hole at the center of the channel bed and does not threaten the channel banks, a weir apex angle within the range of 90 to 120 degrees resulting in the maximum scour depth and scour volume; (6) the straight weir produces only a limited scour hole; and (7) the V-shaped weir with its apex pointing downstream causes two scour holes, one near each bank, the holes being smaller than for a weir with the apex pointing upstream but potentially threatening the channel banks.

Field studies are made for comparison with the laboratory studies and with the results of other researchers. In particular, local scour and streambed and streambank adjustments to a groin on a small creek and to a new spur dike field on the Willamette River, Oregon are documented. Flow patterns, current velocities and water depths in the dike field are reported.

FOREWORD

The Water Resources Research Institute, located on the Oregon State University campus, serves the State of Oregon. The Institute fosters, encourages and facilitates water resources research and education involving all aspects of the quality and quantity of water available for beneficial use. The Institute administers and coordinates statewide and regional programs of multidisciplinary research in water and related land resources. The Institute provides a necessary communications and coordination link between the agencies of local, state and federal government, as well as the private sector, and the broad research community at universities in the state on matters of water-related research. The Institute also coordinates the interdisciplinary program of graduate education in water resources at Oregon State University.

It is Institute policy to make available the results of significant water-related research conducted in Oregon's universities and colleges. The Institute neither endorses nor rejects the findings of the authors of such research. It does recommend careful consideration of the accumulated facts by those concerned with the solution of water-related problems.

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TABLE OF CONTENTS

Page

LIST OF FIGURES

Page

LIST OF TABLES

I. INTRODUCTION

Problem Addressed

Scour (localized erosion) in rivers and streams is a contributing factor to streambank erosion throughout the country. Streambank erosion causes annual losses of valuable land along thousands of miles of rivers. A study conducted at the direction of the 1971 Oregon Legislative Assembly disclosed that a minimum of 3,800 miles of streambank in the state were experiencing erosion, creating more than 14 million square yards of visibly eroding banks (Soil and Water Conservation Commission, 1973). These problems occur in all parts of the United States. In many instances, only low-cost techniques, rather than costly riprap protection, can be afforded by local landowners.

Scour in channels is an effective natural means for providing variable flow conditions and habitat for fish. Particularly in seasons of low streamflows, scoured zones provide resting and hiding opportunities for fish. Many simple scour-causing structures and gravel-trapping structures have been placed in streams by trial-and-error methods to enhance fishery habitat. Many more will be installed through ongoing programs by agencies and sports groups.

In both situations (bank erosion control and fish habitat improvement), there is need for the hydraulic evaluation of a variety of low-cost, simple channel devices that can be used to control scour, protect streambanks, and provide fishery enhancement. In each situation, the hydraulics of local flow often are not well-understood nor adequately considered when such bank protection or stream enhancement is undertaken. Users of such channel structures need to know in advance the impact on bank protection and fishery enhancement. A better hydraulic basis is needed for activities that cumulatively cost many thousands of dollars each year.

Purpose, Scope and Objectives

The broad purpose of this research has been to determine the effects of engineered channel structures on local sediment scour and deposition and the potential application of these structures for concurrent streambank protection and fishery habitat enhancement.

The structures investigated include spur dikes, groins, and weirs. Spur dikes and groins are structures extending outward from the streambank into the channel. The terms "rock jetty" and "deflector" are commonly used among biologists to refer to such structures. The terms are used interchangeably, although spur dikes are often considered to be larger (higher and longer) than groins, rock jetties, and deflectors. Spur dikes may be "spurs" extending outward from continuous dikes or revetments along the bank. Sometimes the word spur is dropped. Weirs are low sills that extend from bank to bank across the channel. Spur dikes and groins are partially exposed at most water levels. Weirs, in contrast, are submerged at most water levels.

Two structural types of spur dikes and groins were investigated: riprapped rockfill and rockfilled gabions. One structural type of weir was considered: rockfill gabions. This emphasis on rockfill structures reflects the general ready availability of rock material for construction in much of western North America, the less-complex construction involved, compared with concrete structures, and the greater likely durability compared with timber structures.

The specific objectives of the research have been:

1. to investigate the sediment scour and deposition characteristics for single spur dikes and groins;
2. to investigate the sediment scour and deposition characteristics for multiple spur dikes and groins;
3. to determine the desirable orientation angles and spacing of multiple spur dikes and groins to provide streambank protection;
4. to identify the opportunities for concurrent fishery habitat enhancement when spur dikes and groins are used for bank protection;
5. to investigate the scour and deposition characteristics for various orientations of single gabion weirs; and
6. to identify the opportunities for fishery habitat enhancement by use of gabion weirs, as well as the concurrent needs for streambank protection.

Research Approach

The research was organized into two roughly parallel studies, one involving riprapped rockfill structures and the other involving gabions. Each study emphasized laboratory experimentation, based on preparatory literature reviews and evaluations. Each study also involved field observations and measurements. Scott Kehe was responsible for the study of riprapped rockfill structures and Yaw Owusu was responsible for the study of rockfilled gabion structures.

This report integrates the results of the two respective studies. The studies are also separately reported in greater detail as technical reports in partial fulfillment of the requirements for the M.S. Degree in Civil Engineering (see Kehe, 1984 and Owusu, 1984). Additional field information has been added to extend some of the field observations at a group of new spur dikes.

II. GENERAL CONCEPTS

Erosion and Scour

Erosion is the removal of soil particles by flowing water. It embraces the beginning of motion of soil particles initially at rest and their displacement from the area under consideration (Vanoni, 1975).

Erosion may be divided into two main catagories on the basis of areal extent and erosional intensity: (a) general erosion and (b) local scour. General erosion involves the removal of exposed particles from extensive areas of the land, streambank, or streambed surface. Local scour describes erosion involving a single unified flow pattern, as in the case of local scour at the base of a river structure. Surface erosion can be considered to be the combination of effects of many local scours of varying intensities and patterns covering a wide area of land or streambed.

Soil materials may be classified as cohesive or non-cohesive from the point of view of their ability to erode. Non-cohesive sediment consists of discrete particles. The movement of such particles, for given erosive forces caused by moving water, is affected by particle properties such as shape, size, density and the relative position of the particle with respect to other nearby particles. For cohesive sediment, erosion depends on these discrete particle properties and on the breaking of cohesive bonds between groups of bonded particles. Thus, for the same flow, the resistance to erosion is greater for cohesive particles than for individual non-cohesive particles because of the strong bonds.

Streambank Erosion

The United States has nearly 3.5 million miles of streams and rivers. The U.S. Army Corps of Engineers reports that streambank erosion is occurring on approximately 575,000 miles of these streams (USACE, 1981). Severe erosion is reported on two percent of the seven million bank-miles; these are in need of erosion protection. The total damage resulting from this erosion amounts to about $250 million annually, based on 1981, values in losses of private and public lands, bridges, etc. The annual cost of conventional bank protection required to prevent the damage from occurring is estimated to be $1.1 billion.

The removal of streambank soil particles by flowing water is one of the major mechanisms causing streambank erosion. Bank seepage is a second important mechanism for erosion. The mechanics of streambank erosion and the erosion rate are related to the geometry and hydraulic characteristics of the stream and to the type of soil material present.

The bends of meandering rivers are generally the locations for the severest form of bank erosion, the erosion taking place mainly at the concave banks of the bends. Figure 1 shows the flow distribution in a meander, with isometric views of the longitudinal and lateral components of velocity at various positions in the bend. Figure 2 shows the definition of terms used with regard to the geometry of meanders.

Figure 3 shows that the largest water velocities and deepest parts of the channel (i.e., the thalweg) in a bend shift close to the concave bank (the bank at the outside of the bend). Measured velocity distributions show that the maximum point velocity in a bend occurs somewhat below the water surface. Maximum velocities along the concave banks of bends in several

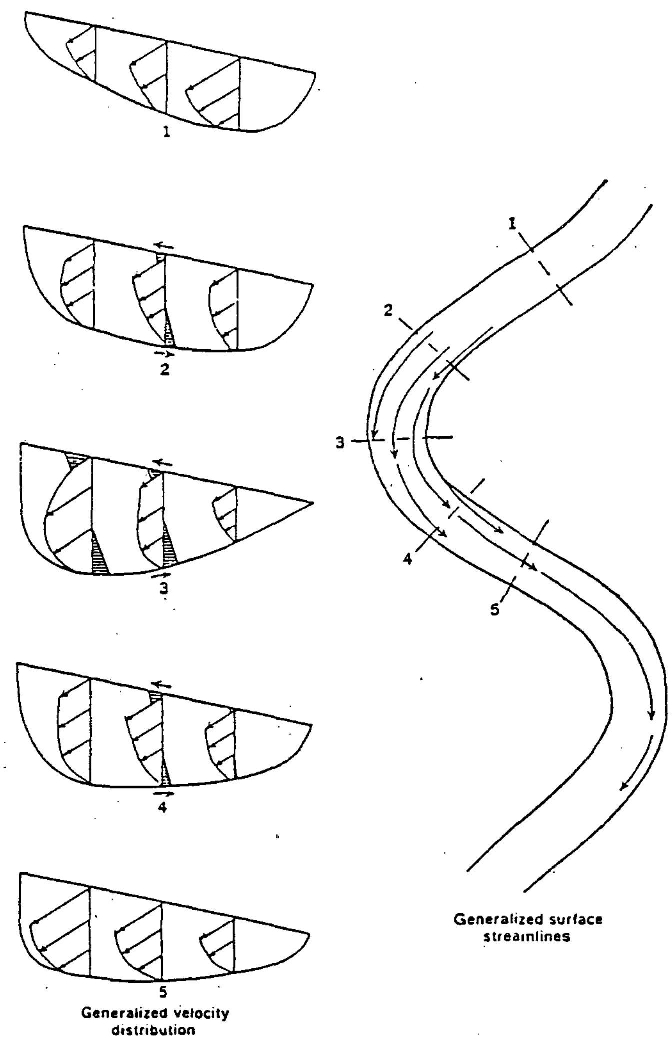


Figure 1. Isometric Views of Flow Distribution in a Meander (Source: Adapted from Leopold, et al., 1964)

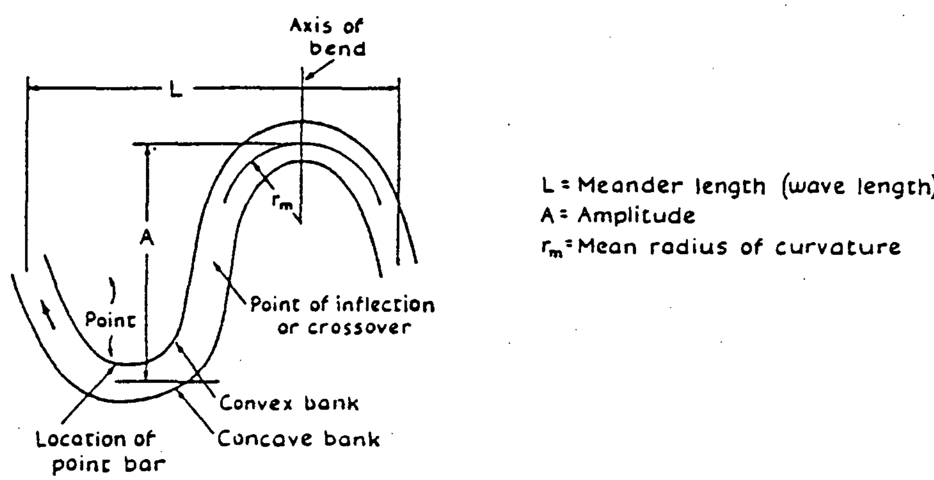


Figure 2. Definition Sketch for Meanders (Source: Leopold, et al., 1964)

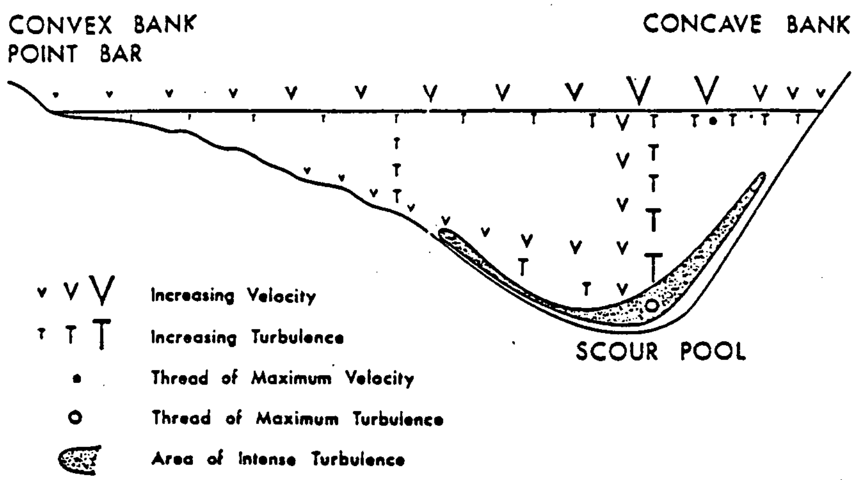


Figure 3. Velocity and Turbulence in a River Bend
(Source: USACE, 1981)

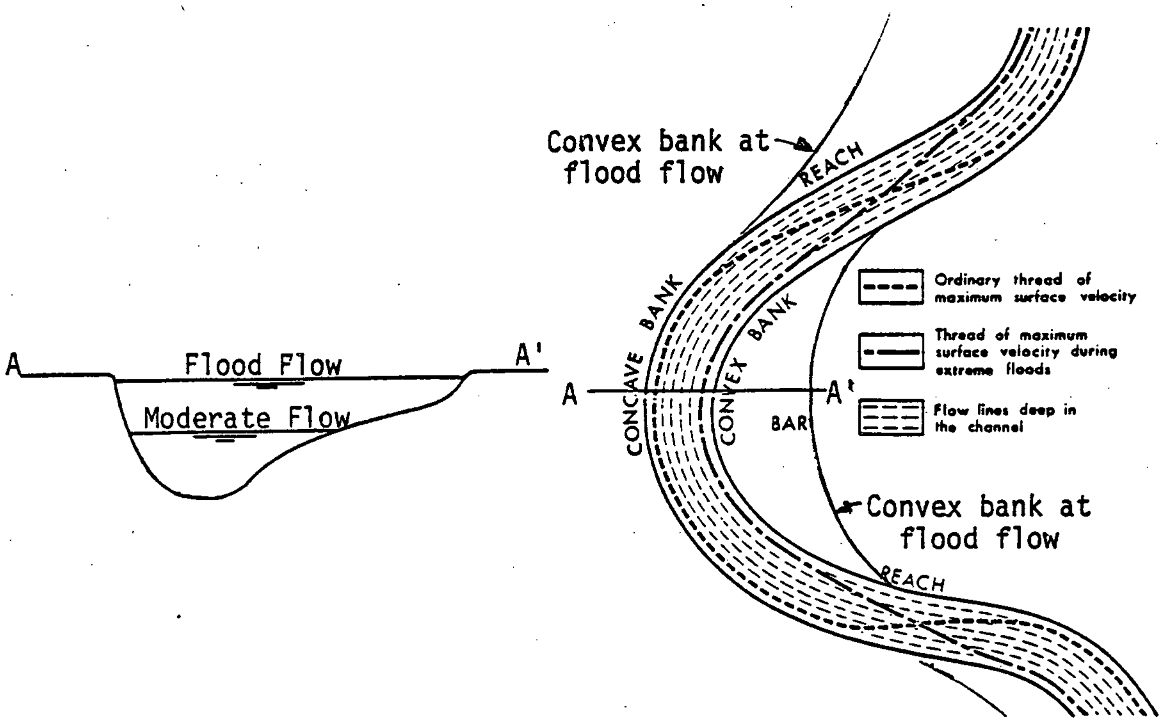


Figure 4. Paths of Maximum Surface Velocity During Moderate and Flood Flows
(Source: Adapted from USACE, 1981)

California rivers are reported to be as high as 1.8 times the average channel velocity (USACE, 1981).

The lateral components of velocity result from the centrifugal force of flow in a bend. The high-velocity masses of water near the surface readily move toward the outside of the bend, tending to cause a "piling up" of water there. This superelevation effect creates a counterflow near the bed, where centrifugal forces are weaker due to smaller water velocities. Hence, an apparent "secondary current" occurs in the plane normal to the longitudinal flow direction.

The combined effect of longitudinal and secondary flow components in a river bend is to give rise to a spiraling flow. This is a major factor in streambank erosion. As the flow erodes the outside of a bed, bank particles fall toward the bed and become entrained in the counterflow across the bed toward the inside of the bend, moving downstream during the process. Depending upon the specific features of the bend, the coarse eroded material may deposit on the point bar near the convex bank within the same bend (see Figure 2) or may be carried farther downstream to deposit. If the bend leads to a straight reach, deposition may occur at a riffle, diagonal bar, or alternate bar near the transition zone. If the bend leads to another bend (as shown in Figures 1 and 2), deposition may occur at a riffle or shoal area at the crossover between bends or at the upstream edge of the next point bar.

During periods of very high water, such as during floods, the bar at the inside of the bend is more deeply covered. Hence, the largest water velocities shift closer to the convex bank. This is shown in Figure 4.

The locations where bank scour may be particularly severe along a bend or straight reach depend upon the local detailed flow pattern and the local soil characteristics. In general, the place where bank erosion is most frequent and where protective revetments most commonly fail is just downstream from the axis of the bend (Parsons, 1960). If this erosion is severe, the vigorous cross-currents near the bed can result in large quantities of bed material being transported to the convex bank. New point-bar deposition forces the flow more strongly against the concave bank, thus sustaining the erosive force there.

Meanders in alluvial rivers increase in size due to progressive erosion of concave banks of river bends. Non-uniform velocity distributions, secondary currents, sediment scour, and sediment redeposition also allow meanders to migrate downstream. Where general bank erosion occurs, the velocities may be fairly well distributed. However, where the riverbank resistance to erosion increases or is variable, the flow tends to concentrate and develop locally greater velocities and depths.

Streambank Erosion Control

The types of methods and structures used to stabilize streams vary widely. Some of the streambank stabilization techniques developed include:

1. Stone riprap revetments;
2. Stone spur dikes;
3. Concrete pavement;
4. Articulated concrete mattresses;
5. Asphalt-mix pavements;
6. Walls and bulkheads;
7. Timber jetties;
8. Rail jacks;
9. Gabions and rock sausages;
10. Vegetation;
11. Automobile frames and bodies;
12. Car tires;
13. Synthetic revetments and matting.

The choice of a particular technique depends to a large extent on the experience and judgement of the engineer. Hydraulic conditions and streambank erosion vary widely from one location to another. This may be due to differences in the various stream characteristics, including flow conditions, bed and bank material, and channel geometry. Even under similar erosive and hydraulic conditions, there is no single universally applicable method. For instance, differing economic and logistic constraints such as the availability of construction material and equipment can also affect decisions. Hence, it has been the engineering practice to solve each bank erosion problem independently.

According to the U.S. Army Corps of Engineers, the state-of-the-art of streambank protection has not advanced significantly since 1950 (USACE, 1981). What has developed is the use of a group of favored methods, the most widely used being stone riprap, rockfill spur dikes, and gabions. The engineer uses basic hydraulic principles to design streambank protection structures. But because of the interrelated complex factors involved, many methods have evolved through a process of "trial and error" experience. Thus, theoretical and empirical techniques are available to determine the necessary particle size and weight to resist erosion caused by the shear or drag forces of flowing water. However, less is known about how to position various structures in the stream to achieve the most effective interaction with the flow to produce desired results. Here, past experience is an important determinant of design methodology.

Fish Habitat Modification

Fish tend to congregate in areas of a stream where food, shelter, temperature range, oxygen content, and other factors combine to create a favorable habitat (Bell, 1973; Hall and Baker, 1982). A varied stream, such as one with a succession of riffles and pools, is usually more condusive to an abundance of game fish than is a monotonous stream, such as one limited to only runs or only wide flat water.

Various structural devices can be used for fish habitat enhancement (see, for example, Bradt and Wieland, 1978; Federal Highway Administration, 1979; Hall and Baker, 1982; Maughan, et al., 1978; Reeves and Roelofs, 1982; and U.S. Fish and Wildlife Service, 1978). Dikes, jetties, deflectors, and groins placed at strategic positions along a streambank can be used to cause scour holes and pools or to deepen the local channel. Weirs across the stream can be used to create pools and plunging flow. The various channel structures can also be used to aerate the water, reduce the water temperature, preserve existing pools, cause sediment deposition, and provide gravel beds suitable for fish spawning. Most importantly, the structures can be designed to serve the habitat function while simultaneously providing bank protection.

Manipulation of Local Scour

Several general principles have been advanced on the nature of local scour in river channels (Laursen, 1952; Vanoni, 1975). These principles can be stated as follows:

1. the rate of local scour equals the difference between the capacity for bed material transport out of the scoured area and the rate of supply of bed material to that area;

2. the rate of local scour decreases as the flow section is enlarged due to erosion;

3. for given initial conditions, there is a limiting extent of scour; and

4. this limit is approached asymptotically with respect to time.

The principles apply for all types of structures or natural obstacles in a channel, whether attached to a bank or located in mid-channel. The general principles are usually applied for the purpose of estimating scour conditions in order to protect a structure. They can also be used to evaluate structural possibilities for manipulating local scour. Such manipulations may be undertaken for streambank protection and for habitat enhancement.

Scour at Spur Dikes and Groins

Spur dikes and groins directly influence flow velocities and patterns in a river. This has a significant effect upon sediment transport, general and local scour, and sediment deposition near the structure. If the structure is built at the concave bank of an eroding river bend or along a straight bank where flow velocities are high, the main current is shifted away from the bank toward the center of the channel. Channel depths adjust to the new velocity and shear stress conditions; this happens by means of local sediment scour and deposition. The effects sometimes carry downstream for some distance because of the new flow alignment caused by the structure.

The obstruction caused by a spur dike or groin generates an intense and complicated system of vortices. The primary vortex impinges on the bed immediately in front of the spur dike, erodes bed material, entrains the eroded material in the flow, and allows it to be carried away downstream by the main flow (Ahmad, 1953). Intermittent vortices of lesser strength occur along both the upstream and downstream faces of the dike, as shown in Figures 5 and 6.

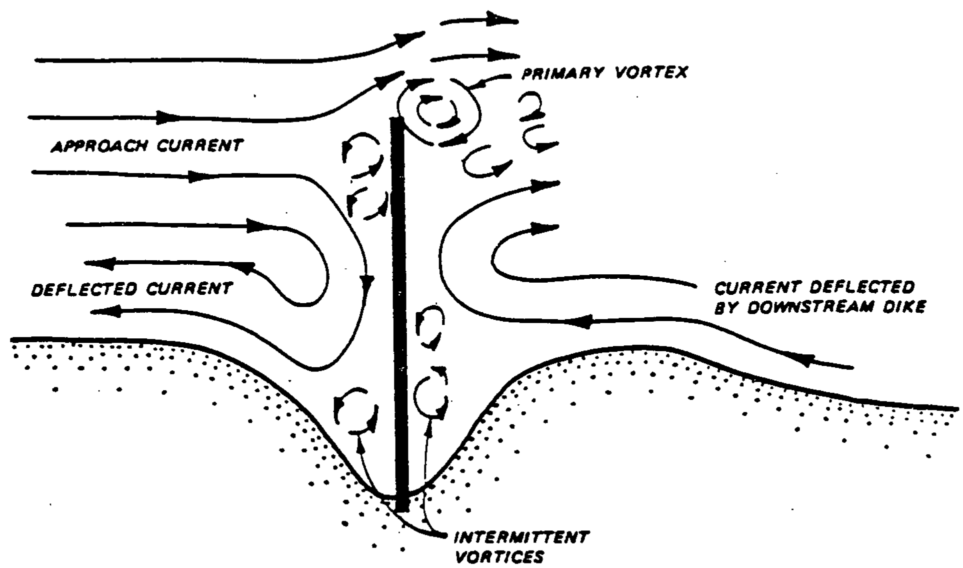


Figure 5. Plan View of Flow Patterns at a Spur Dike or Exposed Groin (Source: Copeland, 1983)

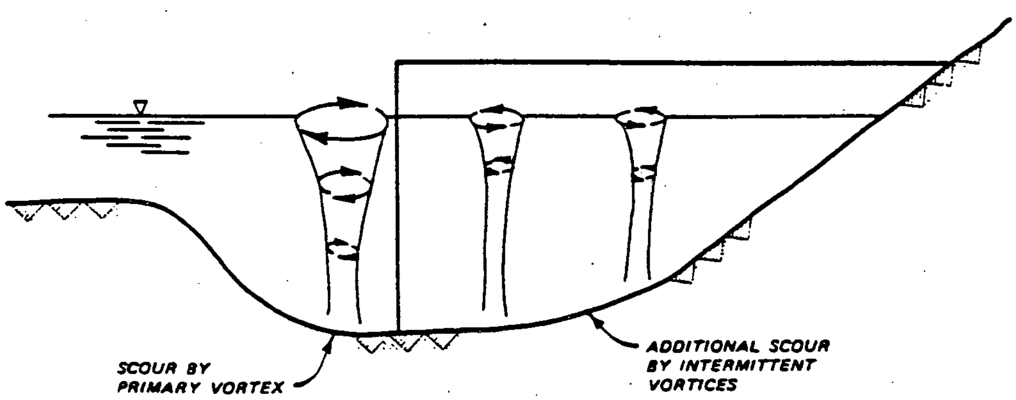


Figure 6. Front Profile of Scour Hole Along a Spur Dike or Exposed Groin (Source: Copeland, 1983)

The deepest point of the main scour hole is located close to the tip of the structure, where the local flow acceleration is most pronounced. If the structure is oriented downstream, the primary vortex is deflected downstream and the main scour hole may be positioned some distance downstream of the tip (Samide and Beckstead, 1975). An upstream-oriented structure may cause greater scour than a downstream-oriented structure (Ahmad, 1951; Garde, et al., 1961; Mukhamedov, et al., 1971; Tison, 1962).

The anticipated scour depth adjacent to the structure is of concern for design, so that the structure's base elevation is set below that of the scour hole. The size, depth, and extent of the scour hole generated by the structure and the angle of repose for material forming the sides of the scour hole are also of concern with respect to possible nearby bank erosion.

Much research has been done on scour depth at a dike. This is also applicable to exposed (unsubmerged) groins. Several parameters have been identified that must be considered in order to determine the depth of scour (e.g., Samide and Beckstead, 1975). These include water parameters, channel flow parameters, structure parameters, and sediment parameters. These can be given in the following equation:

$$d_s = f(\rho_w, \mu, g, h, V, T, B, L, \theta, \beta, D_{50}, \sigma_D, C, \rho_s) \quad . \; . \; . \; . \; . \; . \quad (1)$$

in which d_s = limiting depth of scour below original bed level; ρ_w = density of water; μ = absolute viscosity of water; g = acceleration due to gravity; h = average depth of flow in approach channel; V = average flow velocity in approach channel; T = time of scour after initiation of flow; B = average width of approach channel; L = length of the structure; θ = orientation angle of structure with the downstream bank; β = side-slope angle of the structure with the vertical plane; D_{50} = median grain size of bed sediment;

σ_D = term describing the size gradation of bed sediment; C = sediment concentration by weight; and ρ_s = density of bed sediment. (All symbols used are listed in the Appendix).

Since river flows are highly turbulent when scour occurs, the effects of fluid viscosity can be neglected compared to inertial forces. If the flow is sustained for a long time, the depth of scour will approach a maximum, allowing time to be dropped from further consideration. Assuming h, V and ρ_w as the repeating variables, a dimensional analysis of the remaining variables yields, after some rearragement:

$$\frac{d_s}{h} = f(\frac{V}{\sqrt{gh}}, \frac{B}{h}, \frac{L}{h}, \theta , \beta , \frac{D_{50}}{h}, \sigma_D, C, \frac{\rho_s}{\rho_w},) \qquad (2)$$

The first term in parenthesis is the Froude Number. The second and third terms can be combined to form a flow contraction ratio.

The general concepts and principles have been applied by several researchers to develop mathematical relationships for the prediction of scour. Several of the resulting equations proposed for predicting scour depths at spur dikes or groins are presented in Table 1. Some of the originally-given symbols have been changed here to facilitate comparison.

Investigators disagree as to which parameters are most important in determining scour depths at spur dikes. Early investigators considered the stream velocity and waterway contraction ratio to be the most significant factors. Laursen (1960) maintained that the scour depth is primarily a function of the dike length and the upstream depth and is independent of the contraction ratio if sediment movement occurs upstream of the dike. Liu, et al. (1961) and Cunha (1973) also determined that the contraction ratio was not important once sediment motion was established. Garde, et al. (1961) and Gill (1972) determined that the contraction ratio was an important

Table 1. Summary of Published Scour Depth Prediction Equations Applicable to Spur Dikes and Groins

1.	$y_s = k(\frac{Q}{f})^{0.33}$ k varies between 0.8 and 1.8	Inglis (Copeland, 1983)
2.	$y_s = k(\frac{q^2}{F_{bo}})^{0.33}$ k varies between 2.0 and 2.75	Blench (1969; Samide and Beckstead, 1975)
3.	$\frac{y_s}{q^{2/3}} = 1.616 - 0.908(\frac{2}{3})^{\frac{\theta}{15}}$	Ahmad (1951)
4.	$y_s = yK(\frac{B_1}{B_2})\ (F)^n$	Garde, et al. (1961)
5.	$y_s = 0.3y + 2.15y\ (\frac{B_1 - B_2}{y})^{0.4}\ (F)^{0.33}$	Liu, et al. (1961)
6.	$y_s = 8.375y\ (\frac{D_{50}}{y})^{0.25}\ (\frac{B_1}{B_2})^{0.83}$	Gill (1972)
7.	$Le = 2.75\ (\frac{d_s}{y})\ ([\frac{1}{r}\ (\frac{d_s}{y}) + 1]^{1.70} - 1)$	Laursen (1960)
8.	$y_s = \frac{10.4\ (\sin\theta)^{\frac{1}{4}}\ (\cos\beta)^{\frac{1}{2}}\ Vm(hm)^{\frac{1}{2}}}{(1-M)(\varepsilon_{85\%})^{1/6}(1+0.09C)\ q^{\frac{1}{2}}(1+135F)^{3/2}}$	Mukhamedov, et al. (1971)
9.	$\frac{d_s}{h} = 0.30 + 1.60\ \text{Log}_{10}\ (\frac{\tau_{ns}/N_{ns}}{\tau_{ns*}/N_{ns*}})$ where $\frac{\tau_{ns*}}{N_{ns*}} = \frac{82.6\tau_c}{(3.69M + 0.84)^2}$	Awazu (1967)

See Appendix for Definitions of Symbols

Note: $y_s = h + d_s$ in all cases

parameter. Liu, et al. considered velocity to be an important factor with or without sediment movement. Garde, et al. also stated that it was. Gill reported that velocity was not an important factor. There is also controversy regarding the importance of bed material size. Garde, et al. and Gill found grain size to be important while Laursen, Liu, et al. and Ahmad did not believe it to be a major determinant of scour depth.

The equations developed are primarily based on laboratory testing of a single structure in a straight flume, with limited prototype verification. More prototype data are needed to resolve disagreements as to the main prediction parameters and regarding the conflicting predictions given by the equations. Furthermore, more information is needed to determine the potential applicability of these equations for predicting scour at multiple structures.

Scour at Weirs

Weirs influence the local flow patterns and velocities in a stream. The primary effect upstream of the structure is to cause a backwater zone where water depth is greater and velocity is smaller than in the absence of the weir. As the flow passes over the weir, it accelerates and plunges toward the streambed just downstream. Hence, the primary effect downstream of the structure is to cause local scour and the development of a scour hole near the base of the weir.

The process involved in scour downstream of a weir is roughly analogous to the scour below an outfall pipe due to a free jet or to the scour at the base of a free overfall. The overfall can be considered to be a two-dimensional version of the circular jet from the outfall pipe. Some weir configurations fit the two-dimensional flow concept whereas other weirs are lower near the center and cause the flow to be like a flattened oval jet. An added complication is that often the tailwater level is high enough to partly submerge the jet or overfall.

Numerous studies have been done on the subject of jet scour. These include the work of Rouse (1939), Schoklitsch (1935), Doddiah (1950), Thomas (1953), and more recently, Rajaratnam and Beltaos (1977). The several factors affecting the streambed scour from a circular jet include water parameters, jet parameters, and sediment parameters. These can be expressed in the following relationship (Doddiah, et al., 1953):

$$d_T = f(y_i, V_j, A_j, T, \rho_w, \rho_s, \omega, \sigma_\omega) \quad \text{. . . . . (3)}$$

where d_T = depth of scour below the original bed level at a particular time, T; y_i = tailwater depth at pool over scour hole, measured from original bed level; V_j = the velocity of efflux of the jet; A_j = cross-sectional area of the jet; T = time; ρ_w = density of water; ρ_s = density of sediment; ω = settling velocity of the sediment being scoured; and σ_ω = standard deviation of the sediment settling velocity.

A dimensional analysis of the variables can be made and the resulting expression can be simplified by assuming that the density ratio and the standard deviation of the sediment settling velocity are constant. This gives:

$$\frac{d_T}{y_i} = F\left(\frac{y_i}{\sqrt{A_j}}, \frac{V_j}{\omega}, \frac{\omega T}{y_i}\right) \quad \text{. . . . . (4)}$$

To evaluate the jet scour in a systematic fashion, the following equations were developed by Doddiah, et al. (1953), using a simple process of curve fitting for their experimental data:

a) for a solid jet

$$\frac{d_T}{y_i} = \frac{0.023\sqrt{A_j}}{y_i} \log[\frac{\omega T}{y_i}](\frac{V_j}{\omega} - 1) - 0.0022 \frac{y_i}{\sqrt{A_j}} + 0.4 \quad (5)$$

b) for a hollow jet

$$\frac{d_T}{y_i} = \frac{0.023\sqrt{A_j}}{y_i} \log[\frac{\omega T}{y_i}](\frac{V_j}{\omega} - 1) - 0.032 \frac{y_i}{\sqrt{A_j}} + 0.5 \quad (6)$$

These equations show considerable similarity for scour from a solid jet and scour from a hollow jet. Of particular significance, the scour is directly proportional to a geometric progression of time; that is, a state of equilibrium in the scour process is not reached, even for constant discharge. Moreover, the magnitude of scour decreases with a decrease in the ratio of jet velocity to settling velocity, approaching zero as this ratio approaches unity. Thus, for example, jet flow over a low weir on a coarse streambed is not likely to cause much scour.

For the analysis of scour at the base of a free overfall, Doddiah, et al. (1953) assumed the existence of a relationship of the following type:

$$d_s = f(H, y_i, q_w, T, \omega, \sigma_\omega) \quad (7)$$

where H = height of drop of bed level from upstream to downsteam; and q_w = discharge per unit of crest of the weir on drop structure.

Dimensional analysis of the variables gives the following expression:

$$\frac{d_s}{y_i} = f(\frac{H}{y_i}, \frac{q_w}{H\omega}, \frac{q_w T}{H^2}, \sigma_\omega) \quad \ldots\ldots\ldots (8)$$

Two empirical equations were developed by Doddiah, et al. to represent this expression:

a) for sediment with a narrow size-range

$$\frac{d_s}{y_i} = [0.29 + 0.070 \log (\frac{q_w T}{H^2}] (\frac{q_w}{H\omega})^{\frac{1}{2}} (\frac{H}{y_i})^3 (\frac{q_w}{H\omega})^{1/3} \quad \ldots\ldots\ldots (9)$$

b) for sediment with a wider size-range

$$\frac{d_s}{y_i} = [0.49 + 0.040 \log (\frac{q_w T}{H^2})] (\frac{q_w}{H\omega})^{2/3} (\frac{H}{y_i})^2 (\frac{q_w}{H\omega})^{1/3} \quad \ldots\ldots\ldots (10)$$

These equations show the continuing scour over time and the reduced scour if the sediment is large.

Schoklitsch (1935) developed a more simplified equation for predicting the scour at a drop structure. This can be given as:

$$y_s = \frac{3.15}{(D_{90})^{0.32}} H'^{0.2} q_w^{0.57} \quad \ldots\ldots\ldots (11)$$

where y_s = depth of scour in feet from the water surface over the scour hole to the bottom of the scour hole ($y_s = y_i + d_s$); D_{90} = the diameter of the bed material in millimeters such that 90 percent is smaller; and H' = height of drop in feet of water surface from upstream to downstream.

Schoklitsch's equation does not consider the time variable. The influence of structure height and bed material size are evident.

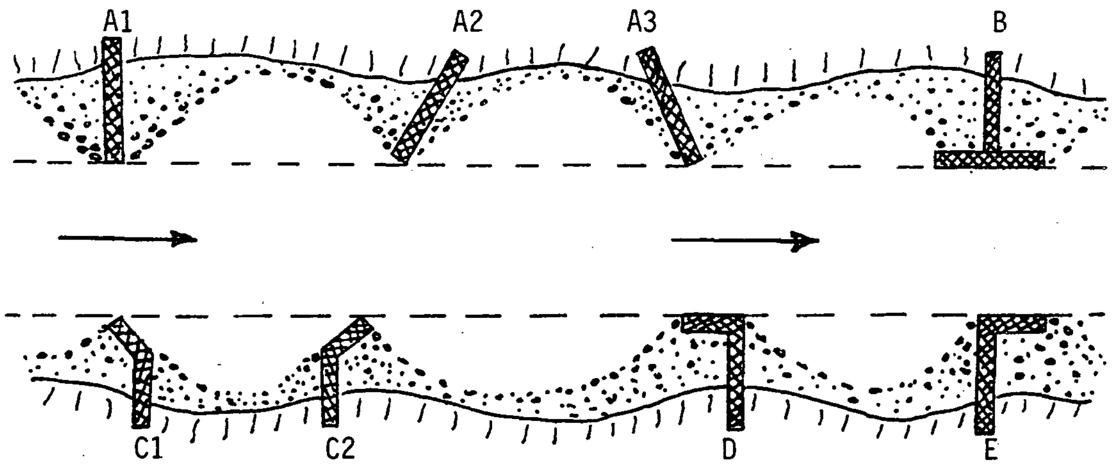
In research conducted by Doddiah, et al. (1953), the work of Schoklitsch was compared with that of Doddiah (1950) and Thomas (1953), and the time variable was demonstrated to be significant. For small scour depths, Schoklitsch's equation compared favorably with data for which time was considered as a variable. However, for big scour depths and more active scouring conditions, the equation of Schoklitsch predicted a scour depth only half as great as that which actually occurred.

The time dependency of scour remains well supported in the literature (e.g., Blaisdell, et al., 1981; Raudkivi and Ettema, 1983). A more complex aspect is the time-variability of river discharge. Whereas the design flood can be used to predict maximum scour depth for structural protection, this approach is not as useful regarding habitat. Bed load transport during the rising and falling limbs of hydrographs causes scour holes to enlarge and then to be partially refilled. This results in variable amounts of habitat space available during different low-flow periods.

Some Illustrations

Spur dikes, groins, rock jetties, and deflectors may be classified according to their structural appearance as seen in plan view. The most common types are illustrated in Figure 7. These types include straight, hammer-head or T-head, bayonet or hockey stick, J-head, and L-head structures.

The use of various spur dikes and groins for bank protection and channel realinement is illustrated in Figure 8. The dash lines show the definition



A) Straight Type
1. At Right Angles to Stream
2. Slanted Upstream
3. Slanted Downstream

B) Hammer-Head or T-Head Type

C) Bayonet or Hockey Stick Type
1. Slanted Upstream
2. Slanted Downstream

D) J-Head

E) L-Head

Figure 7. Conventional Types of Spur Dikes and Groins

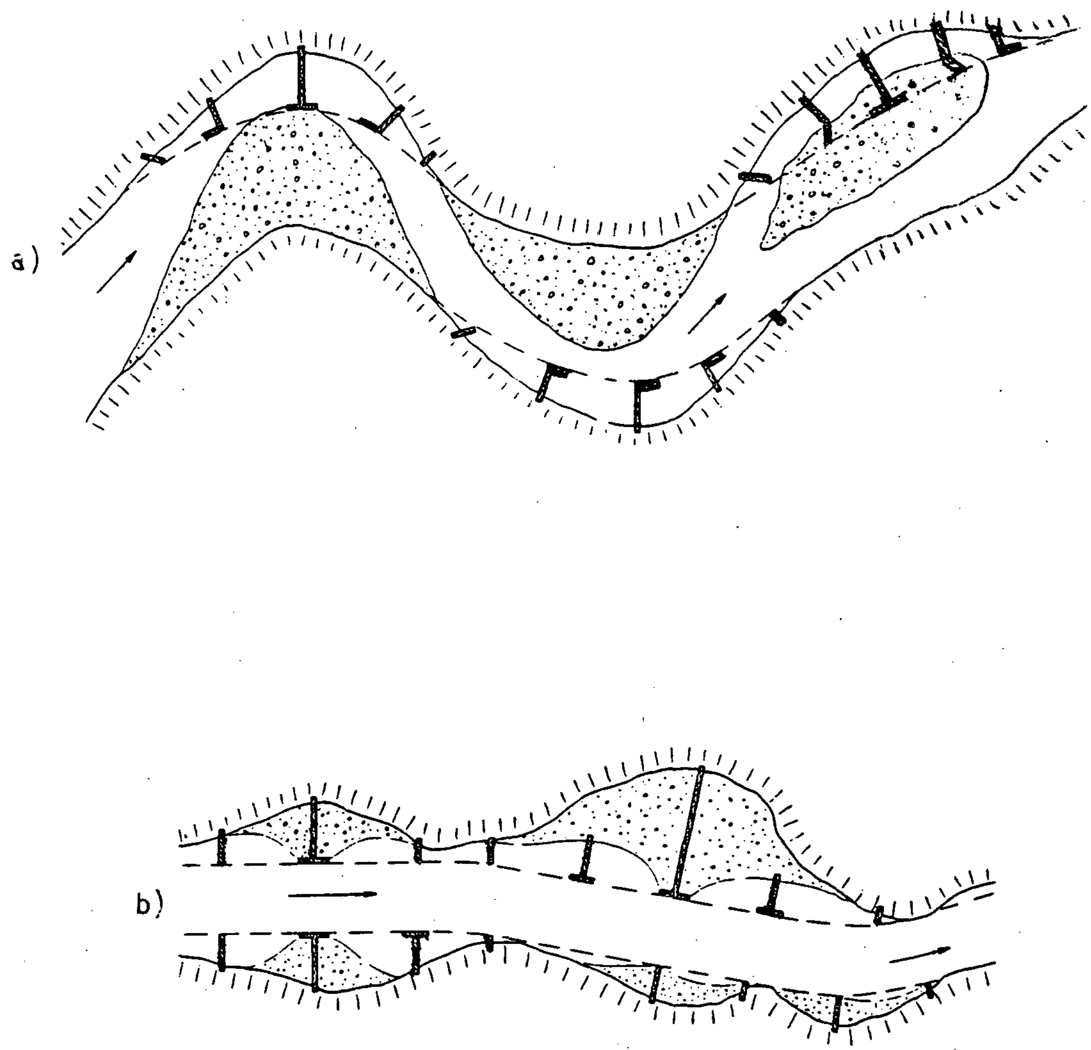


Figure 8. Uses of Various Types of Spur Dikes and Groins for Bank Protection and Channel Realinement. Dash Lines Define Intended Future Channel Banks (Source: Maccaferri Gabions of America; undated-b)

new channel banks that will develop over time as scour occurs in the new main channels and sediment deposition occurs in slackwater areas, followed by vegetation growth.

Figure 9 shows further applications of spur dikes and groins, in this case with emphasis on habitat enhancement. Two arrangements are shown for V deflectors for narrowing and accelerating the current to create a scour hole. Gravel deposits may occur on the downstream side of deflectors and be suitable for spawning. Also shown is a Y deflector arrangement which enhances current acceleration and extends the length of the scoured area.

Figure 10 shows some applications of weirs for habitat enhancement. Flow over the weir causes downstream scour. The backwater effect upstream of the weir increases the stream surface area and water depth there, thus increasing the available fish habitat. Placing a sill structure downstream of the weir gives a means of deepening and controlling the limits of the scour hole below the weir. Gravel trapping usually occurs upstream of a weir and may improve spawning opportunities.

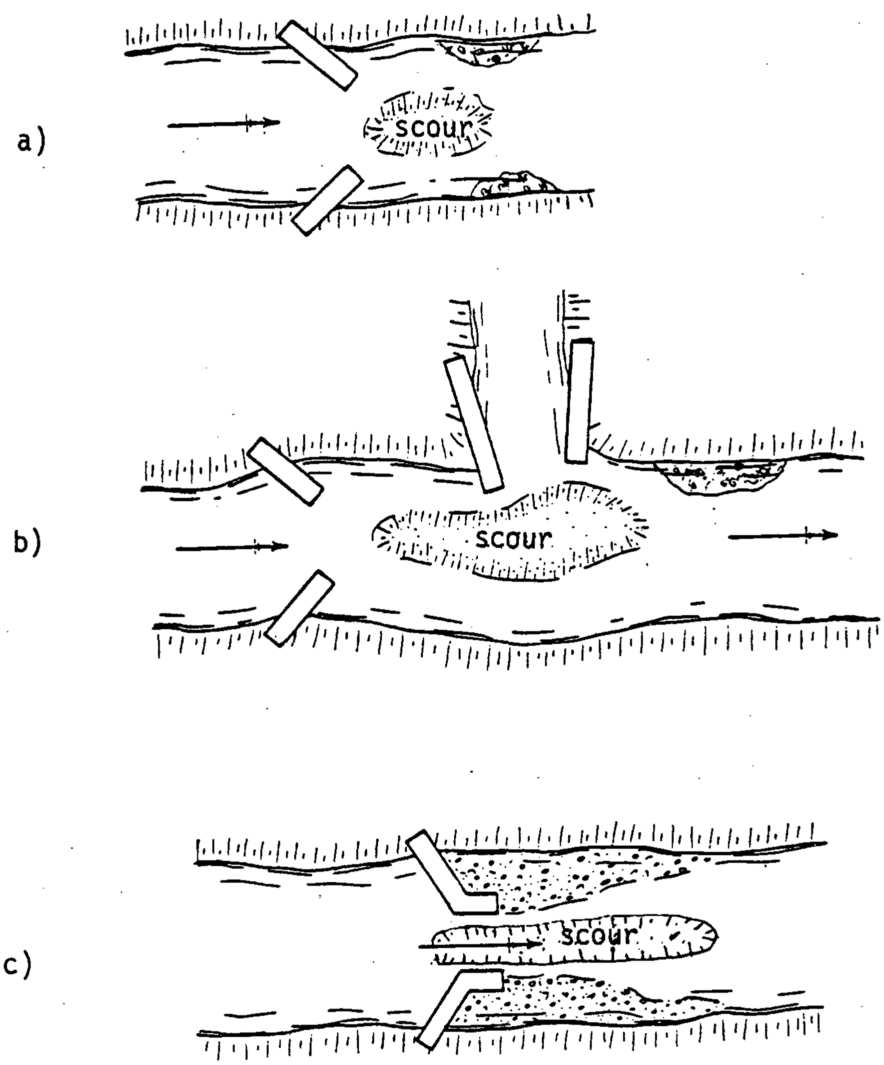


Figure 9. Use of V and Y Deflectors to Concentrate Currents, Scour, and Deposition. 9a and 9b show V-Deflectors and 9c shows Y-Deflectors (Source: Adapted from Maccaferri Gabions of Canada, undated)

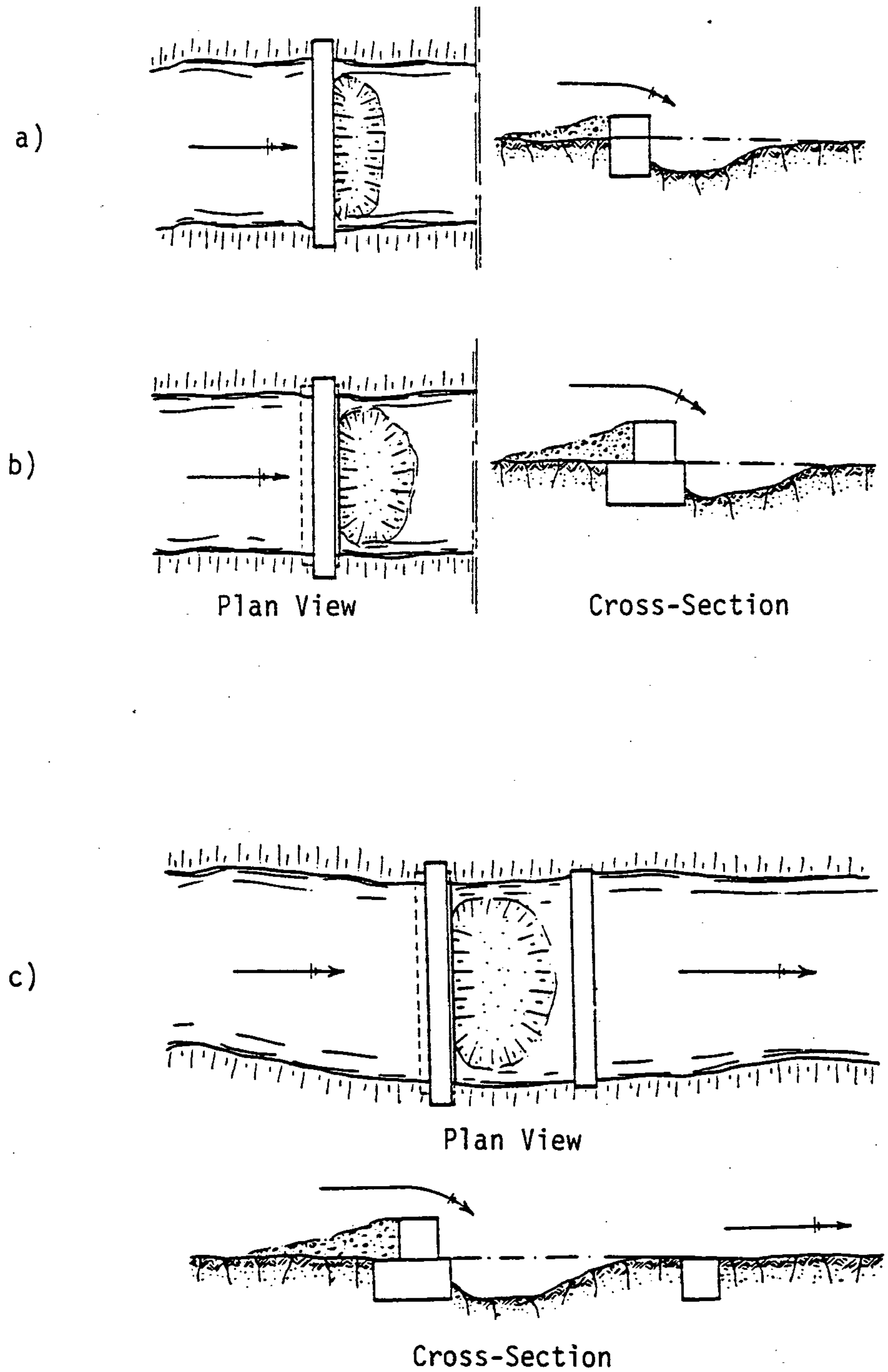


Figure 10. Weir Applications to Enhance Fish Habitat
(Source: Maccaferri Gabions of America, undated-a)

III. USE OF RIPRAPPED ROCKFILL SPUR DIKES AND GROINS

Overview

This part of the report describes the use of semi-impermeable riprapped rockfill structures as a bank protection technique. These structures are called spur dikes or groins. They extend outward from the bank into the river in order to prevent bank erosion and to manipulate river currents. The purpose of this part of the research is to characterize the sediment scour and deposition characteristics based on a comparison of available literature on spur dike design, a model study, and a prototype investigation.

First, the general features and effects of spur dikes are described. Second, the principles of spur dike use and design are presented, including the effectiveness of spur dikes, based upon a review of available literature. Third, the procedures and results are discussed for model studies conducted to evaluate several parameters relating to spur dike design and layout, including length and orientation angle. Fourth, the methods and results are discussed for a field investigation of prototype spur dikes. This was carried out during and after completion of the dike field construction to determine the hydraulic effects of the spur dikes on river flow patterns and bed topography and for comparison with model studies. Finally, some general conclusions are made.

General Features of Riprapped Rockfill Spur Dikes and Groins

All forms of streambank structures extending out from the bank and used for bank protection or channel current manipulation purposes, including dikes, groins and jetties, are commonly called spur dikes and are referred to as spur dikes in this part of the report. The term "spur dike field" refers to the use of more than one dike, intermittently-spaced, at a site.

Spur dikes influence flow velocities and current patterns in a river. Spur dikes are an indirect method of bank protection, by means of which potentially eroding currents are deflected away from the bank or are reduced in velocity. In contrast, direct protection methods physically isolate the bank from the eroding currents, such as by the use of a riprap revetment to blanket the bank with rock.

Spur dikes extend outward from the bank into the channel at an angle which need not be normal to the flow (see Figure 7). Some dikes are straight (as seen in a plan view), whereas others are bent, such as "L" heads for which the outer tip turns downstream parallel to the streamflow or "J" heads for which the outer tip turns upstream or "T" heads for which outer tips turn both upstream and downstream.

Spur dikes may be constructed of various materials, such as masonry, concrete, timber, earth or stone. As a result, spur dikes may be either permeable or impermeable. Impermeable dikes block and deflect the current away from a bank. Permeable dikes also deflect the flow; but in addition they slow the current passing through the dike, thereby inducing deposition of sediment in the lee of the dike near the bank. The accumulation of sediment behind a dike or between successive dikes and the retardation of flow both cause the main channel to carry a larger proportion of the total discharge, with increased current strength and sediment transport capacity. As a result, a greater depth is maintained in the main channel. The permeable dike is most effective in a swift-flowing river carrying a substantial load of coarse sediment that can settle upon reaching a zone of reduced velocity. Timber piles are the basic component of most permeable dikes. Such dikes may also be rock-filled below some predetermined water level (e.g., low-flow level).

Because riprapped rockfill spur dikes have a central zone of heterogeneous rock sizes and a coarse outer covering, they tend to be semi-permeable. Thus, there may be flow through the dike but it is relatively insignificant with respect to influencing sediment deposition.

Design of Riprapped Rockfill Spur Dikes and Groins

General Considerations

Although spur dikes are used extensively, there are no definitive hydraulic design criteria to follow. Design is based primarily on experience and judgement, due to the wide range of variables affecting the performance of the spur dikes. Parameters affecting spur dike design include channel width, water depth, water velocity, channel sinuosity, bed material size, sediment transport rate, bank cohesiveness, spur dike length, dike width, dike profile, dike orientation angle, and dike spacing if more than one dike is present (Lindner, 1969).

Spur dikes must redirect the flow away from an erosion-prone bank. This affects flow patterns and sediment movement. Permeable dikes induce sediment deposition which helps redirect the flow. Impermeable dikes do not depend on sediment deposition to redirect the flows; they rely upon the reduced width of the river to alter flow conditions.

Where the river is contracted by a new dike, the water slope and energy gradient initially become steeper and the velocity becomes greater, increasing the scour potential of the flow. The river may attempt to regain its original cross-sectional area through bank and bed erosion. But, if the dike and the opposite bank are stable, the main flow may only be able to scour out the river bed in order to increase the cross-sectional area and reduce the velocity and scour potential. The size and stability of the bed material will determine the extent to which this can occur. For flow contraction to continue along the entire length of a dike field, either the dikes must be closely spaced or sediment deposition must occur between the dikes. The possible depth of main-channel scour caused by dikes and dike fields must be considered in spur dike design.

Spur dikes affect sediment deposition patterns (Lindner, 1969). While most deposition occurs in the lee of permeable dikes, deposition in the vicinity of impermeable dikes and dike fields can also occur upstream of the dike because of greater flow retardation and decreased velocity. When impermeable dikes in a spur dike field are built to an elevation above the high water level, deposition between dikes can only occur if sediment is brought in by eddy action of currents from the main channel. When impermeable dikes are overtopped by water carrying coarse sediment, deposition can occur on and between the spur dikes, especially with L-head dikes. Unless the stream carries a large amount of coarse material in suspension when the water overtops impermeable dikes, the rate of such deposition will be slow. To increase this rate, it may be necessary to divert the bed load into the dike field. One way to accomplish this is by stepping-down the top elevation of successive dikes in a dike field, from upstream to downstream dike (Lindner, 1969).

As an alternative to inducing deposition, it may be desired to prevent the area between dikes from accumulating sediment. For example, this might be desired so as to maintain a fish habitat there. In such cases, the spur dike elevations and tip features may be designed to prevent overtopping and to allow eddy currents to keep the area scoured out.

Angle of Spur Dike to Bank

The orientation of a spur dike describes the direction the dike points into the flow from the bank where it is rooted. The orientation angle is defined as the angle between the downstream bank and the axis of the dike. Table 2 summarizes some of the spur dike orientations that have been used in different geographic areas or have been recommended in different references.

There is considerable controversy as to whether spur dikes should be oriented upstream, perpendicular to the bank, or downstream (Ahmad, 1953; Copeland, 1983; Das, 1972; Garde, et al., 1961; Haas and Weller, 1953; Lindner, 1969; Mukhamedov, et al., 1971; Tison, 1962; United Nations, 1953). Proponents of upstream orientation claim that flow is repelled from dikes oriented upsteam while flow is attracted to the bank by dikes oriented downstream. They also claim that sedimentation is more likely to occur behind spur dikes oriented upstream, so that less protection is required on the banks and on the upstream face of the dike. Proponents of downstream-oriented spur dikes claim that turbulence and scour depths are less at the end of a spur dike oriented downstream and that the smaller the

Table 2. Recommended Orientation Angles for Spur Dikes and Groins

Recommended or Generally Used Angle of Dike to Bank,* in degrees	Reference
100-120	United Nations, 1953
100-120	Central Board of Irrigation and Power, 1971
100-110	Mamak, 1964
100-110 (convex bank)	Samide and Beckstead, 1975
100 or less (concave bank)	Samide and Beckstead, 1975
Upstream	Neill, 1973 (in Copeland, 1983)
90	U.S. Army Corps of Engineers, 1983 (in Copeland, 1983)
90	Richardson and Simons, 1973 (in Copeland, 1983)
90	U.S. Army Corps of Engineers, Memphis and Vicksburg Districts (in Copeland, 1983)
90 or downstream	U.S. Corps of Engineers, 1970 (in Copeland, 1983)
90 or downstream	Missouri River (Lindner, 1969)
75-90	Red River, Arkansas River (Lindner, 1969)
70-90 (30 for sharp curves)	Alvarez, Mexico (in Copeland, 1983)
75	U.S. Army Corps of Engineers, Los Angeles District, 1980 (in Copeland, 1983)
Downstream	Franco, 1967
Downstream	Lindner, 1969

*Measured from downstream bank line to major axis of spur dike.

orientation angle, the more the scour hole is angled away from the dike. They also claim that an upstream alignment promotes flow towards the base of the dike which endangers the integrity of the dike root and may cause a channel to form along the bank in the dike field. They state that debris and ice are less likely to accumulate on downstream-oriented dikes.

Franco (1967) tested dikes angled normal to the flow, 30 degrees upstream of normal, and 30 degrees downstream of normal. He rated the 30-degree downstream alignment best in performance (based on scour, deposition, channel depth and alignment). The upstream-angled dikes produced the least amount of scour but the scour area was greater, extending along the upstream face of the dike. Upsteam dikes produced more disturbance to flow.

Copeland (1983) recently determined that larger eddies are present on the upstream side of upstream-oriented spur dikes than for downstream-oriented spur dikes. This may afford some protection by displacing the currents away from the spur dike root. However, since scour depths are also greater for upstream-oriented spur dikes, the potential benefits of the upstream eddy may be cancelled out by the increased size of the scour hole. Copeland claims that the effective length of a dike (its projected length perpendicular to the bank) is a more significant factor than the spur dike angle, and dikes should therefore be oriented perpendicular to the bank. Spur dikes placed at an orientation angle other than 90 degrees would cost more than dikes placed normal to the flow because of the greater required length, but they would also produce less disturbance.

It is often recommended to aline spur dikes perpendicular to the flow direction rather than at any other angle because test results have been inconclusive to settle the dispute between upstream and downstream orientations.

The United Nations (1953) several years ago recommended an orientation angle of between 100 and 120 degrees. More recently, the U.S. Army Corps of Engineers has generally oriented its spur dikes perpendicular to the bank or slightly downstream (Lindner, 1969). Another practice has been to angle the first dike downstream and the remaining dikes normal to the flow. The trend among designers in selecting dike orientation appears to be shifting from upstream-oriented to downstream-oriented spur dikes.

Length and Spacing of Spur Dikes

The length of a spur dike is selected so that it is sufficient to shift the eroding current away from the bank. However, the dike length must not unduly restrict the channel and must not cause unacceptably large velocities.

The spacing of spur dikes in a spur dike field has generally been based on the length of the spur dike. As the spacing/length ratio increases, the effectiveness of the dike field to prevent bank erosion decreases. If the dikes are spaced too far apart, the current may return to the bank before reaching the zone of influence of the next dike; as a result, bank erosion may occur between the dikes and, if unchecked, may cause the loss of the downstream dike. Conversely, if the dikes are too close, the dike field will be less efficient and more expensive than a correctly designed system in preventing bank erosion (Samide and Beckstead, 1975).

The spacing/length ratios recommended by several different sources are presented in Table 3. The type of bank mentioned is indicative of the severity of flow, which would be greatest for concave banks. The

Table 3. Recommended Spacing/Length Ratios for Spur Dikes and Groins

Spacing/Length Ratio*	Type of Bank	Reference	Comment
1	Concave	United Nations, 1953	General practice
2 to 2.5	Convex	United Nations, 1953	General practice
1	Concave	Bendegom (Samide and Beckstead, 1975)	
2 to 2.5	Convex	Bendegom (Samide and Beckstead, 1975)	
1.5		Mathes, 1956	
1.5	Concave	Los Angeles, District, 1980**	Levee protection with riprap
2.0	Straight	Los Angeles, District, 1980**	
2.5	Convex	Los Angeles, District, 1980**	
2		U.S. Army (Samide and Beckstead, 1975)	Typical for Mississippi River
2 to 2.5		Central Bd. of Irrig. & Power, 1971	
2		Neill, 1973**	
4		Neill, 1973**	If two or more dikes
2.5 to 4	Curves	Alvarez**	
5.1 to 6	Straight	Alvarez**	
3	Concave	Grant, 1948**	
3 to 4		Acheson, 1968	Variation depends on curvature and river gradient
3 to 5		Strom, 1962	
4.29	Straight	Ahmad, 1951	
5	Curved	Ahmad, 1951	
4 to 6	Concave	Richardson and Simons, 1973**	Bank may need riprap

*Ratio of spacing distance between adjacent dikes to groin length component perpendicular to bank.
**In Copeland, 1983

spacing/length ratio of a spur dike field is also a function of the river's discharge and approach velocity.

In the following discussion, the dike length is taken to mean the effective length (component of true dike length perpendicular to the bank).

Spacing/length ratios have been developed largely from experience. The United Nations (1953) states that it is general practice for spur dikes at convex banks to be spaced at 2 to 2.5 times the length and for spur dikes at concave banks to be spaced at a distance equal to the length of the dike. The United Nations also states that a larger ratio is used for a wide river than for a narrow one if both have similar discharges. According to Tiefenbrum (1963), dikes on the middle Mississippi River were originally spaced at two times the dike length and are now designed to be about 1.5 times the length. Ahmad (1951), based on model studies, gives spacing/length ratios of 4.29 for straight reaches and 5 for curved channels. A design manual used by the Central Board of Irrigation and Power in India (1971) recommends a spacing of 2 to 2.5 times the dike length. Mathes (1956) states that a spacing ratio of 1.5 should be used and that values of 0.75 to 2 are generally used on European rivers. For rivers in New Zealand and Australia, Strom (1962) gives spacing ratios ranging from 3 to 5. Acheson (1968) gives ratios ranging from 3 to 4, depending on the degree of curvature. Some authors recommend that the spacing should not exceed the width of the open channel remaining between the dike tip and the opposite bank. Van Ornum (1914) states the older European practice of fixing the spacing between half the width of the contracted channel and the full width; within this range, typical spacing is about half the channel width at concave sections, seven-tenths of the width in straight sections, and approximately equal to the width at convex sections.

Copeland (1983) describes six current and eddy patterns that develop between spur dikes as the spacing/length ratio between them increases. These are presented in Figure 11, where for graphical convenience the dike length is varied and the dike root spacing is kept constant. The type 1 and type 2 dikes have a small ratio (i.e., close spacing), types 3 and 4 have an intermediate ratio, and types 5 and 6 have the largest ratio (the greatest spacing).

In the type 1 field, the main current is deflected outside the spur dike field and maintains a continuous deep channel there. In the type 2 field a second eddy appears but the main current is still deflected outside the spur dike field. In the type 3 field the main current is directed at the spur dike itself, creating a stronger eddy behind the dike and greater turbulence along the upstream face and lower tip. In the type 4 field, a single strong reverse current develops and the stability afforded to the upstream dike is washed out. In the type 5 field the flow diverted by the upstream spur dike is directed to the bank between the dikes and eddies form on both sides of the flow, providing some protection to the bank. In the type 6 field, the current attacks the bank directly, as the downstream eddy no longer provides protection to the bank.

Spur Dike Configuration

Spur dikes often include segments built at different alinements than is the main portion of the dike. Such configurations include L-head dikes, J-head dikes, hammer-head and T-head dikes, and bayonet dikes, as illustrated in Figure 7.

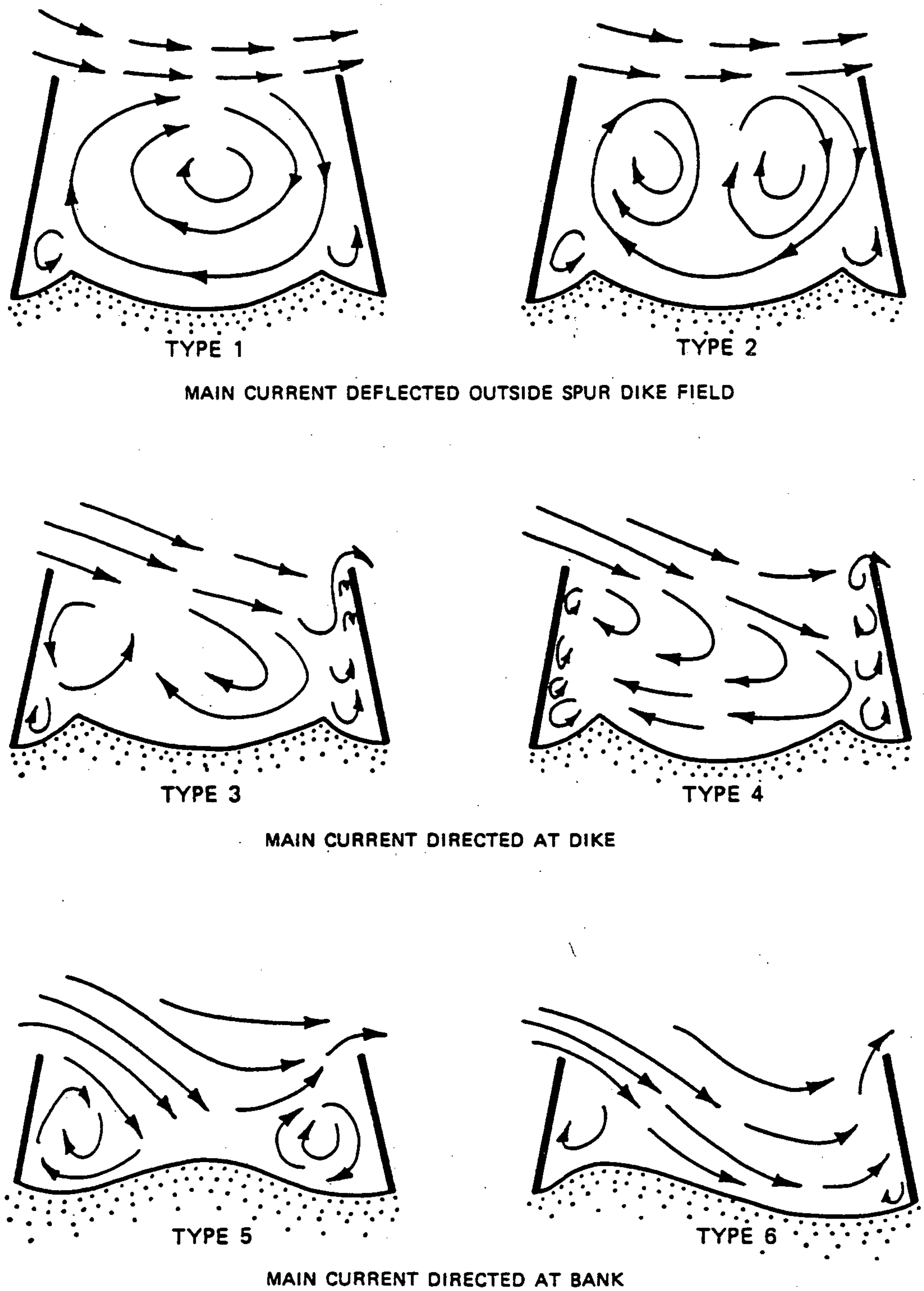


Figure 11. Effect of Spur Dike Spacing/Length Ratio on Current and Eddy Patterns (Source: Copeland, 1983)

The L-head structure is particularly popular. It was developed on the Missouri River to improve protection of the concave banks of curves over that provided by straight spur dikes (Lindner, 1969). The L-head has a downstream-angled segment added to the end of a straight spur dike. This segment is usually parallel to the channel.

Franco (1967) performed tests with the length of the L-head equal to half the distance between the ends of adjacent dikes. He found that the L-head tended to prevent sediment-carrying bottom currents from moving into the areas between the dikes. It was also found that flow over the top of an L-head segment built lower than the main spur dike tended to produce scour along the landward face of that section of the dike. Maximum scour at the ends of the dikes was reduced appreciably, as was the elevation of deposition between the spur dikes. L-heads were reported to reduce scour at the end of the dike, reduce eddy disturbances and cause the flow contraction to persist continuously along the dike system, thus producing a more uniform bed configuration and consistent depths.

In a series of tests by Lindner (1969) it was determined that the L-head should close 45 to 65 percent of the gap between dikes in a spur dike field. He also showed that little benefit was gained from building the L-head above the water surface. His results indicate that the L-heads provided protection to the bank, increased deposition between the dikes, and decreased the scour around the ends of the spur dike. Variations in the river curvature and spacing of the spur dikes would call for corresponding variations of the percentage of closure of the gaps for optimum results. Any degree of closure was found to give added protection to the concave bank, when compared with no closure at all.

The L-head dike thus appears to possess advantages over straight dikes when installed to protect a bank that is caving as a result of the impingement of the current. At such locations, it has been recommended that spur dikes should either be angled downstream or be built with L-heads.

Dikes having the head segment pointing upstream are called J-head dikes. T-head dikes have segments pointing both upstream and downstream. J-head dikes and the upstream leg of T-head dikes are reported to have the same disadvantages as a dike angled upstream (Lindner, 1969). Shapes such as bayonet and hockey-stick shapes are simply variations of the L-head or J-head. There has not been sufficient investigation of these various shapes to ascertain whether they offer any advantages over the L-head. The J-head and T-head apparently possess disadvantages over the L-head such that their use is not recommended; but if used, the upstream leg should not be as high as the straight section of the spur dikes.

Elevation of Spur Dike Crest

The general practice in design of spur dikes in a dike field has been to place all dike crests at about the same height with respect to low water level. The height of the spur dike crest with respect to the water surface depends upon what effect of dike upon flow is sought. The crest or crown of a dike need not be horizontal. There are often situations where a variable-height crown is advantageous. Furthermore, the angle of the dikes is related to the elevation of the dikes.

The sloping-crown or stepped-down crown, in which the dike crown slopes downward or is stepped downward from the bank toward mid-channel, appears to have an advantage where mid-channel shoal erosion is needed over a wide range of stages but where a gradually diminishing channel contraction with increasing stage will suffice. Such a crown design may be required where a spur dike with a level crown would produce objectionable velocities as the stage rises. Even if high velocities are not a concern, if the sloping or stepped down spur dike can produce the shoal erosion desired, it often will be less costly to build than a level crown dike (Lindner, 1969). The flow pattern associated with stepped-down dikes is shown in Figure 12.

Spur dikes with stepped-down crowns are used on the middle Mississippi River and on portions of the lower Mississippi to control meander patterns and to provide the varying degrees of contraction required. The dikes are designed to control and contract stages at mid-bank discharge. They are stepped down for an additional length to confine the low-water channel.

Where deposition of sediment in a dike field is required, stepping-down the crowns progressively from one dike to the next may be advantageous to cause a continuous and comparatively uniform contractional effect along the entire dike field (Lindner, 1969). By the stepped-down arrangement, bed load material moving in the channel beyond the spur dikes is diverted into the spur dike field during stages which progressively overtop each of the dikes from the downstream to upstream spur dike. Flow from the channel moves around the end of the high dike into the area behind it and towards the next lower dike. The faster-moving surface currents continue in a relatively straight line while the slower sediment-carrying bottom currents move into the dike field. For this arrangement to be the most effective, the downstream dike of any two successive dikes should be overtopped for a sufficient length of time before the next upstream dike is overtopped so that there will be enough time for bed load to be diverted to the area between the two dikes.

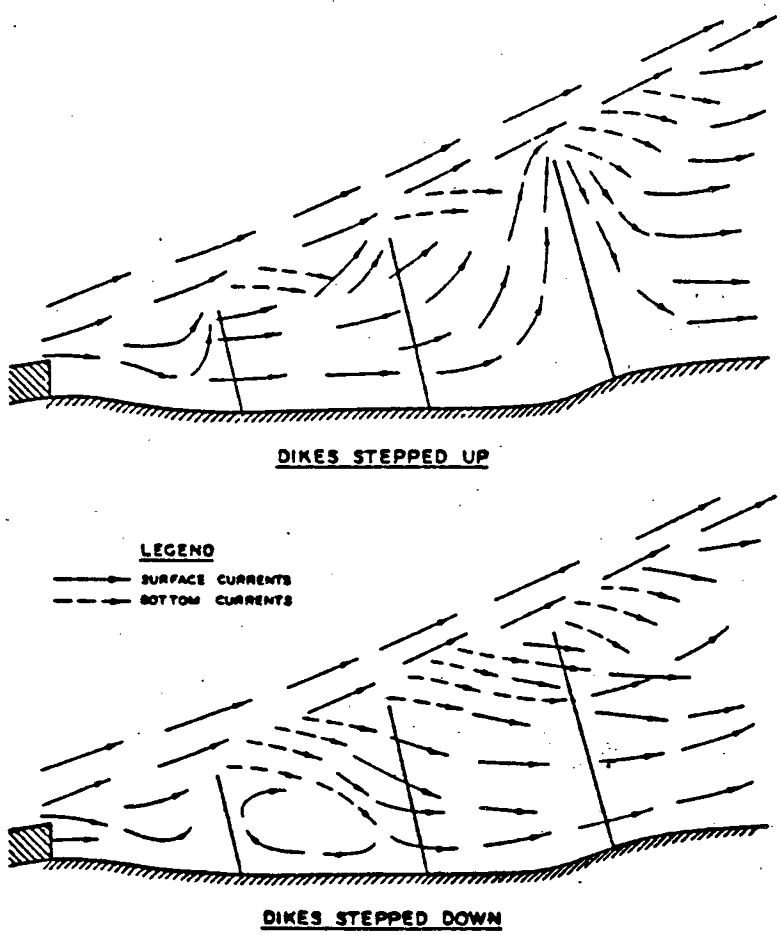


Figure 12. Currents Through Dike Field Having Variable Crest Heights (Source: Franco, 1967)

In a stepped-up spur dike field, where each successive downstream dike is higher, at least some of the flow over the top of the lower dike must move towards the channel, producing disturbances because of its direction. The flow also tends to prevent sediment-carrying bottom currents from moving into the area between the dikes.

Franco (1967) concluded that stepped-down spur dike fields are more effective than fields with all dikes level and that level dike fields are more effective than stepped-up fields (see Figure 12). He also noted that level-crested dikes should be placed normal to the flow or oriented downstream and sloping crested dikes should be normal to the flow or oriented upstream. The reduction in shoaling is almost directly proportional to the elevation of the dikes. The area downstream of the dikes covered by deposition generally increases in size with a decrease in dike elevation. Franco found that dikes placed normal to the flow were the most effective in reducing the amount of shoaling.

Spur Dike Side Slopes and Root

The side slope of the spur dike at its head end affects the nearby scour pattern. With a flatter head, the base of the dike tip extends farther away from the exposed crown. Hence, the scour hole will be more distant from the head and will be longer and shallower (Samide and Beckstead, 1975). Tison (1962) tested trapezoidal-shaped dikes and found that a sloped head reduced the diving motion of the water near the upstream face and reduced the scour depth. Mamak (1964) suggests using a head slope of 3:1 or flatter, perhaps up to 5:1. Mukhamedov, et al., (1971), in calculating scour, use a factor $K_\beta = (\cos\beta)^{1/2}$ to take into account the effects of varying dike head slope, where β is the angle between the sloping side of the dike and the vertical plane.

For the main body of the dike, it has been recommended that the upstream face be inclined at a slope of 1.5:1 to 3:1, and that the downstream face have a slope of 2:1 to 4:1 (Samide and Beckstead, 1975).

The root of a spur dike must be protected against the risk of flood waters cutting into the bank around the main body of the dike. Mamak (1964) recommends that the root be embedded into the bank 4 to 10 meters. He also recommends that short bank revetments be constructed on each side of the root.

Spur Dike Location in River Reach

The locations within a river reach at which spur dikes should be placed is ultimately determined by the location of the erosion area and by appropriate dike spacing ratios. Water velocity and shear stress distributions within the stream should also be considered when placing dikes (Samide and Beckstead, 1975). For the positioning of dikes along the outside of a meander loop, Varshney (1972) recommends that single dikes be placed at 0.55 of the loop length, that if two dikes are used they be placed at 0.5 and 0.6 of the loop length, and that the 0.4, 0.5 and 0.65 positions be used for a field of three dikes.

When a dike field is to be placed upstream of a bridge crossing, Blench (1969) recommends that the first dike upstream of the bridge be placed at 0.4 of the loop length.

Model Studies

Purpose

Model studies were conducted to give qualitative information on scour patterns and the degree of bank protection resulting from various spur dike configurations and arrangements. Several design parameters were tested and evaluated, such as spur dike length, shape, orientation angle and spacing between dikes. The model tests included study of a prototype spur dike field.

Experimental Apparatus

The model tests were conducted in a sand-filled tank with a test section 7 feet long and 4 feet wide. A Willamette River reach having a new spur dike field was molded in the sand. The Froude number formed the basis for open channel modeling and for scaling various parameters between the prototype and model. A horizontal scaling ratio of 600:1 was selected, based upon the space available. A vertical scaling ratio of 200:1 was used. This vertical distortion allowed prototype turbulence to be approximately simulated in the model. The molded sand was covered with a layer of cement approximately 1/4" thick and sprinkled with a fine layer of plaster of Paris. A variety of spur dike models were formed from modeling clay, using the same scaling ratios as for the river model. Water was supplied from a recirculating pump and was passed through an entrance box and a baffle to distribute the flow uniformly over the width of the model river bed.

Experimental Procedures

To conduct each experimental test, spur dikes were first placed in the model in the desired arrangement and at the desired locations. Dry sand finer than 0.59 mm was then sprinkled over the model bed and banks until a uniform depth of approximately 1/8" was obtained. This sand was used to detect scour patterns due to the flow. Water was then allowed to flow in the channel for about five minutes. This was sufficient time for bank erosion and scour to occur and scour patterns around the spur dikes to become relatively stable. A discharge of about 0.03 cfs was used for each test, equivalent to a prototype discharge of 50,000 cfs. At the end of each experimental run, the scour and bank erosion patterns were recorded. During several runs, red dye was introduced at the extrance box so that eddy currents around the spur dikes could be recorded. Scour patterns and bank protection patterns were obtained and recorded for each of the runs.

Table 4 summarizes the test conditions used for each experimental run. The effective dike length is used. This is the component of total dike length measured perpendicular to the bank from the base to the tip of the dike and is equal to the true dike length along its axis times the sine of the dike orientation angle between the downstream bank line and the axis of the dike. For L-head, J-head, and T-head dikes, the length of the main body of the dike, from base to point of dike axis alignment change, is used in this calculation. Scour patterns and bank protection patterns were obtained and recorded for each of the runs.

Experimental runs 1 through 12 were conducted in the straight section of the river reach upstream from the prototype spur dike field. The tests were made to determine the relative ability of single dikes of varying length and orientation angle to deflect the main river current away from the bank and to protect it from erosion. After each run, the distances downstream from the spur dike to the points where the main current returned to the bank and where bank erosion began were measured and recorded.

Experimental runs 13 through 33 were conducted in the concave section of the river reach. Various combinations of spur dike shapes, lengths, orientation angles and configuations were tested. The resulting scour and bank erosion patterns were recorded.

Experimental runs 34 through 37 were conducted using the entire river reach. The prototype spur dike field arrangement was tested in run 34 in order to obtain scour patterns for comparison with those obtained from the

Table 4. Summary of Spur Dike Model Test Conditions

A. Experiments Using Straight Section of River Reach:

Run	Le/W	θ	Run	Le/W	θ	Run	Le/W	θ
1	1/6	90	5	1/6	45	9	1/6	135
2	1/4	90	6	1/4	45	10	1/4	135
3	1/3	90	7	1/3	45	11	1/3	135
4	1/2	90	8	1/2	45	12	1/2	135

B. Experiments Using Concave Section of River Reach:

Run	Le/W	θ	Number of Dikes	Dike Spacing	Dike Shape
13	1/2	90	1		Straight
14	1/2	120	1		Straight
15	1/2	60	1		Straight
16	1/4	30	1		Straight
17	1/4	60	1		Straight
18	1/4	90	1		Straight
19	1/4	120	1		Straight
20	1/4	150	1		Straight
21	1/6	90	1		Straight
22	1/4	90	2	Le	Straight
23	1/4	90	2	2Le	Straight
24	1/4	90	3	2Le	Straight
25	1/4	90	1		L-head
26	1/4	90	1		J-head
27	1/4	90	1		T-head
28	1/2	90	1		L-head
29	1/2	90	1		J-head
30	1/4	90	2	2Le	L-head
31	1/4	90	1		Straight (submerged)
32	1/4	90	1		Straight (sloping)
33	1/3	$\theta_1 = 45$ $\theta_2 = 90$	2	2Le	Straight

C. Experiments Using Entire River Reach:

Run	Description
34	Prototype arrangement (8 dikes)
35	Prototype arrangement with dikes 2, 3, & 6 removed (5 dikes remaining)
36	Prototype arrangement with dikes 2, 3, 4, 6, 7 removed (3 dikes remaining)
37	Control test - no dikes

field study. Several modifications of the prototype arrangement were also tested. Run 37 was a control run conducted with no dikes.

Experimental Results

Figure 13 presents model test results from runs 1-12 showing the distance downstream that the main current is deflected by a dike before again impinging against the bank, based on various dike lengths and orientation angles. This distance is X_2 when measured from the dike base and is X_4 when measured from the dike tip. For dikes with 90-degree orientation angles, $X_2 = X_4$. Figure 13 also shows the distance downstream that the bank is protected from erosion, being X_1 as measured from the dike base and X_3 as measured from the dike tip. (For 90-degree dikes, $X_1 = X_3$.) The effective dike length Le (perpendicular distance from bank to dike tip) is shown as a fraction of the uncontracted channel width W. Table 5 summarizes the observed distances.

Both in model studies and in field work it was observed that the deflected flow, upon approaching the bank, would divide into a main flow continuing downstream and an eddy flow moving upstream. Hence, erosion would occur for some distance upstream from the point of flow trajectory impingement on the bank.

Figure 14 presents model test results from runs 13-21, showing the scour patterns associated with single dikes at various Le/W ratios and orientation angles. The dimensions of the scour area are shown lengthwise and crosswise at prototype scale and the scour area is given in units of square inches as measured in the model. To convert scour area to prototype square feet, the model measurements should be multiplied by 2500. The distance X, shown as a multiple of Le, repesents the X_4 distance defined above. Table 6 summarizes the observed scour areas and distances.

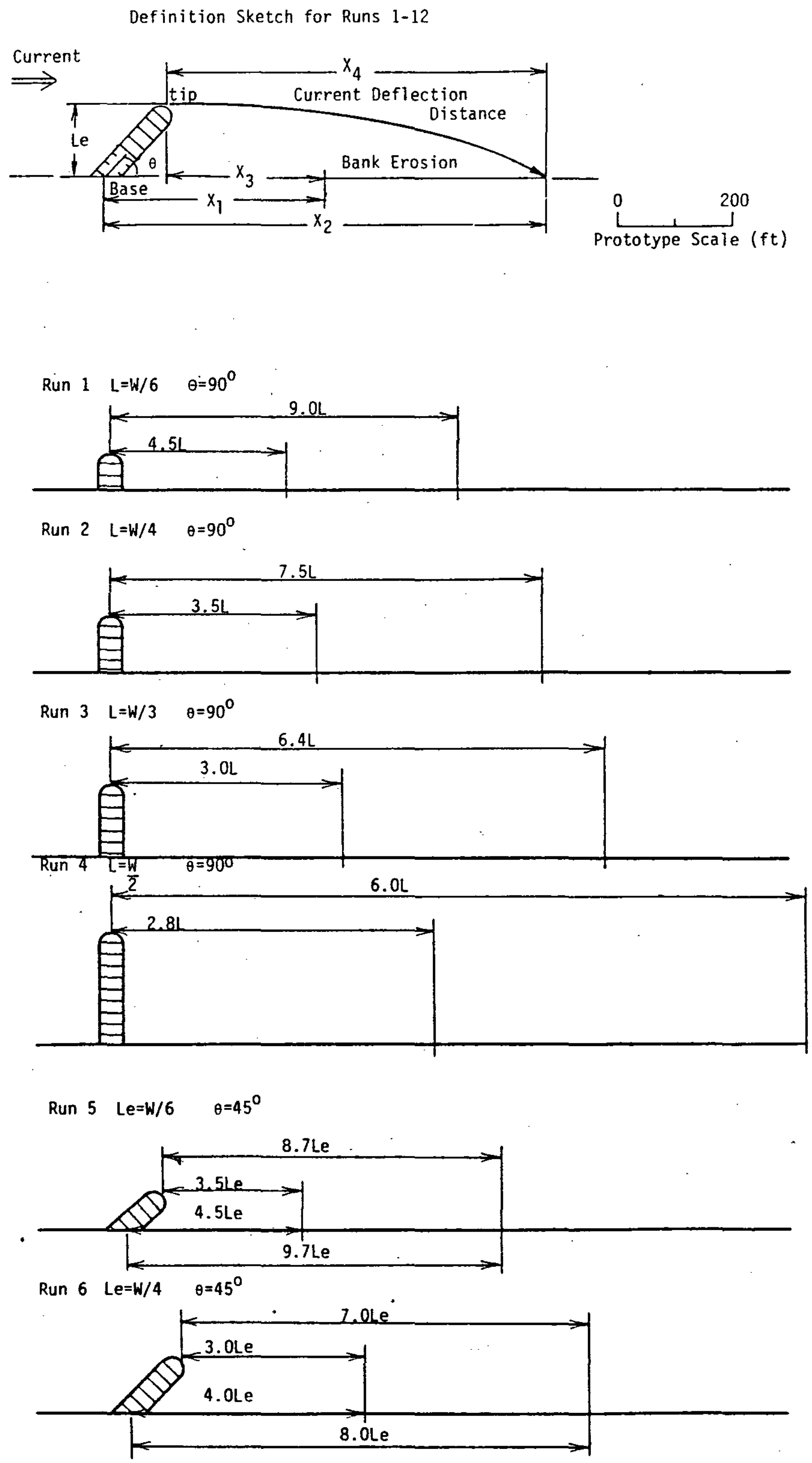


Figure 13. Model Test Results Showing Current Deflection and Bank Protection Distances for Single Spur Dikes (Runs 1-12)

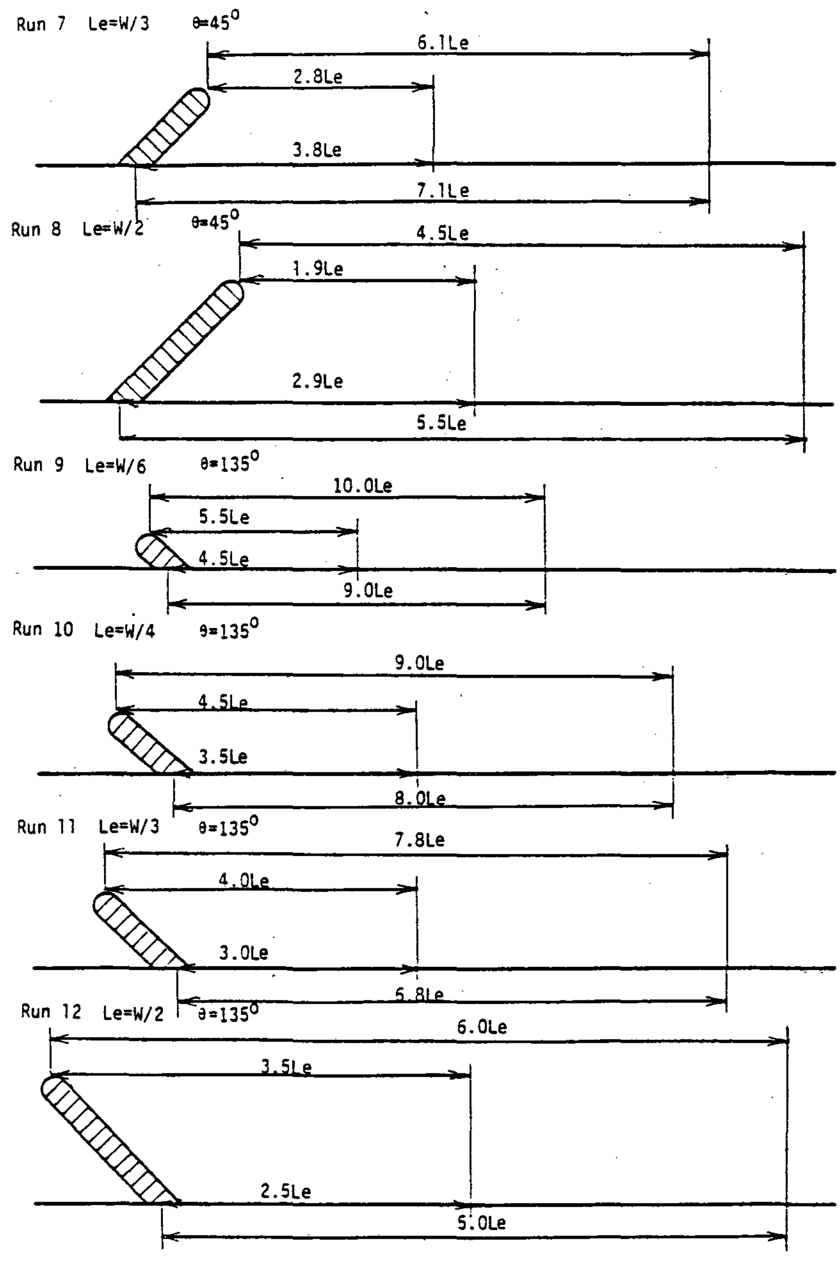


Figure 13. Continued

Table 5. Effect of Spur Dike Length and Orientation Angle on Bank Protection and Current Deflection Distances (From Runs 1-12)

Le/W	X_1/Le			X_2/Le			X_3/Le			X_4/Le		
	45	90	135	45	90	135	45	90	135	45	90	135
1/6	4.5	4.5	4.5	9.7	9.0	9.0	3.5	4.5	5.5	8.7	9.0	10.0
1/4	4.0	3.5	3.5	8.0	7.5	8.0	3.0	3.5	4.5	7.0	7.5	9.0
1/3	3.8	3.0	3.0	7.1	6.4	6.8	2.8	3.0	4.0	6.1	6.4	7.8
1/2	2.9	2.8	2.5	5.5	6.0	5.0	1.9	2.8	3.5	4.5	6.0	6.0

Le/W	X_1/W			X_2/W			X_3/W			X_4/W		
	45	90	135	45	90	135	45	90	135	45	90	135
1/6	0.75	0.75	0.75	1.62	1.50	1.50	0.58	0.75	0.92	1.45	1.50	1.67
1/4	1.00	0.88	0.88	2.00	1.88	2.00	0.75	0.88	1.13	1.75	1.88	2.25
1/3	1.27	1.00	1.00	2.37	2.13	2.27	0.93	1.00	1.33	2.03	2.13	2.60
1/2	1.45	1.40	1.25	2.75	3.00	2.50	0.95	1.40	1.75	2.25	3.00	3.00

Table 6. Effect of Spur Dike Length and Orientation Angle on Scour Area and Current Deflection Distance (Runs 13-21)

Le/W	Scour Area*					X_4/Le				
	30°	60°	90°	120°	150°	30°	60°	90°	120°	150°
1/4	2.41	2.70	2.80	2.95	3.62	2.3	3.4	5.5	6.6	7.7
1/2	--	16.0	17.21	18.4	--	--	3.7	4.5	5.3	--
1/6	--	--	0.43	--	--	--	--	8.25	--	--

*Scour area measured in square inches in the model. For prototype scour area in square feet, multiply by 2500.

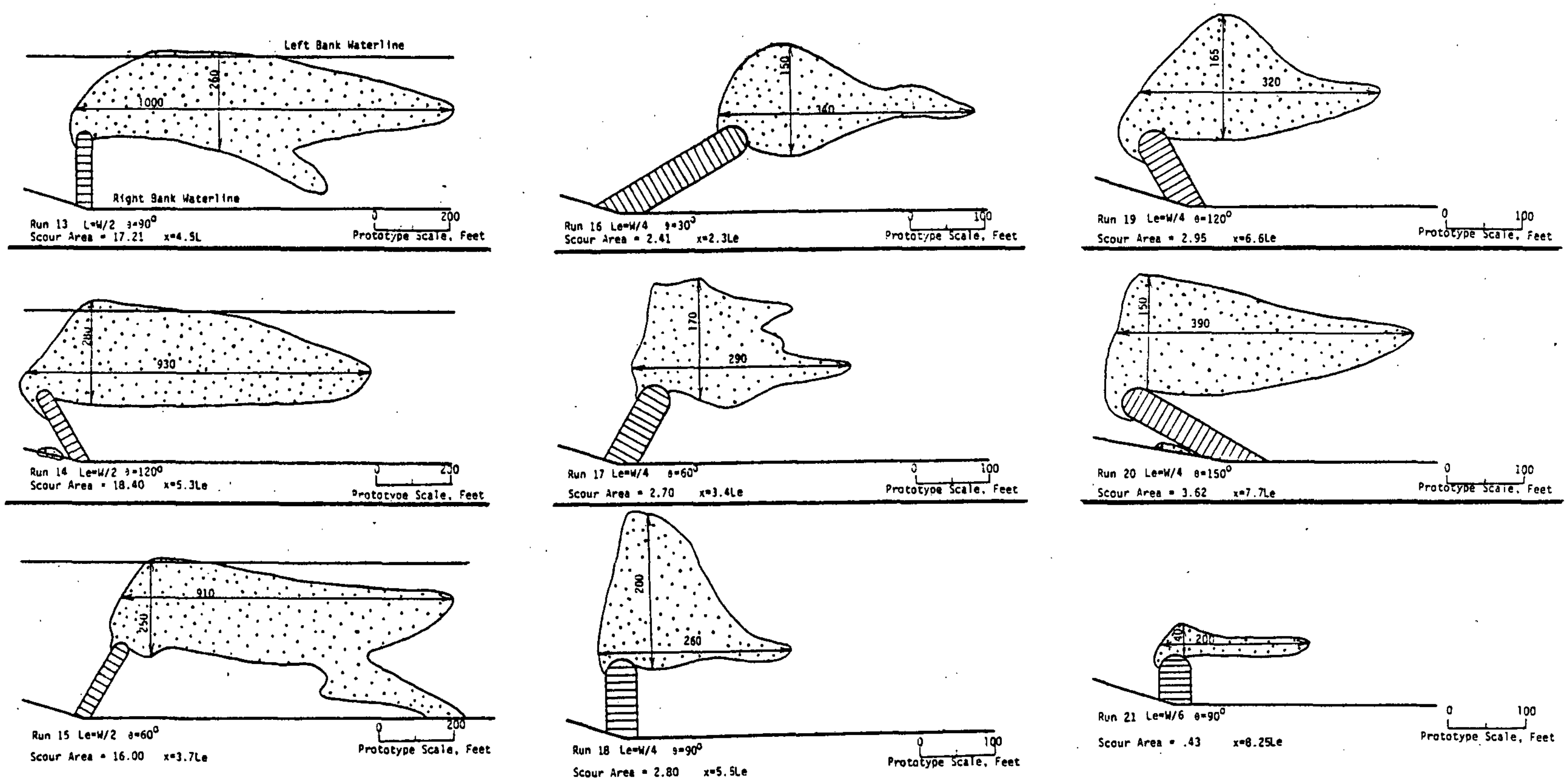


Figure 14. Model Test Results Showing Scour Patterns Associated with Single Dikes (Runs 13-21)

Figure 15 presents model test results showing scour patterns associated with two or three dikes in a dike field, based on runs 22-24 and 33. Dike spacing and orientation angle are varied.

Figure 16 presents test results from runs 25-30 showing the scour patterns associated with various dike shapes, including L-head, J-head and T-head. Results for a pair of L-head dikes are also shown.

Figure 17 shows the scour pattern results of model tests with a fully submerged dike (run 31) and a sloping, partly submerged dike (run 32).

Figure 18 shows the bank and bed scour that occurred in the model of the prototype reach with no structures present to give bank protection. In Figure 18b the dike field is superimposed on the scour results to give a reference for the locations at which dikes were installed. It should be pointed out that the model scour closely identified the actual prototype scour zone observed in the field prior to construction of the dike field.

Figure 19 presents model test results from runs 34-36 showing the scour patterns at the prototype dikes. In run 34, all eight dikes were used. For run 35 the three dikes thought to be least essential were removed. For run 36, two additional dikes were removed.

<u>Effect of Spur Dike Length on Bank Protection and Flow Deflection</u>

Figure 13 and Table 5 show the effect of spur dike length on length of bank protection and on current deflection distances. The length of bank protection (X_1) increases as the effective length of the spur dike increases. However, it does not increase a linear manner. For example, a dike with a 90-degree orientation angle and an effective length of 1/6 the uncontracted channel width will protect a bank 4.5 times its own length (see X_1/Le), whereas a dike three times larger (1/2 the channel width) will protect a bank 2.8 times its own length. The ratio decreases but the absolute length

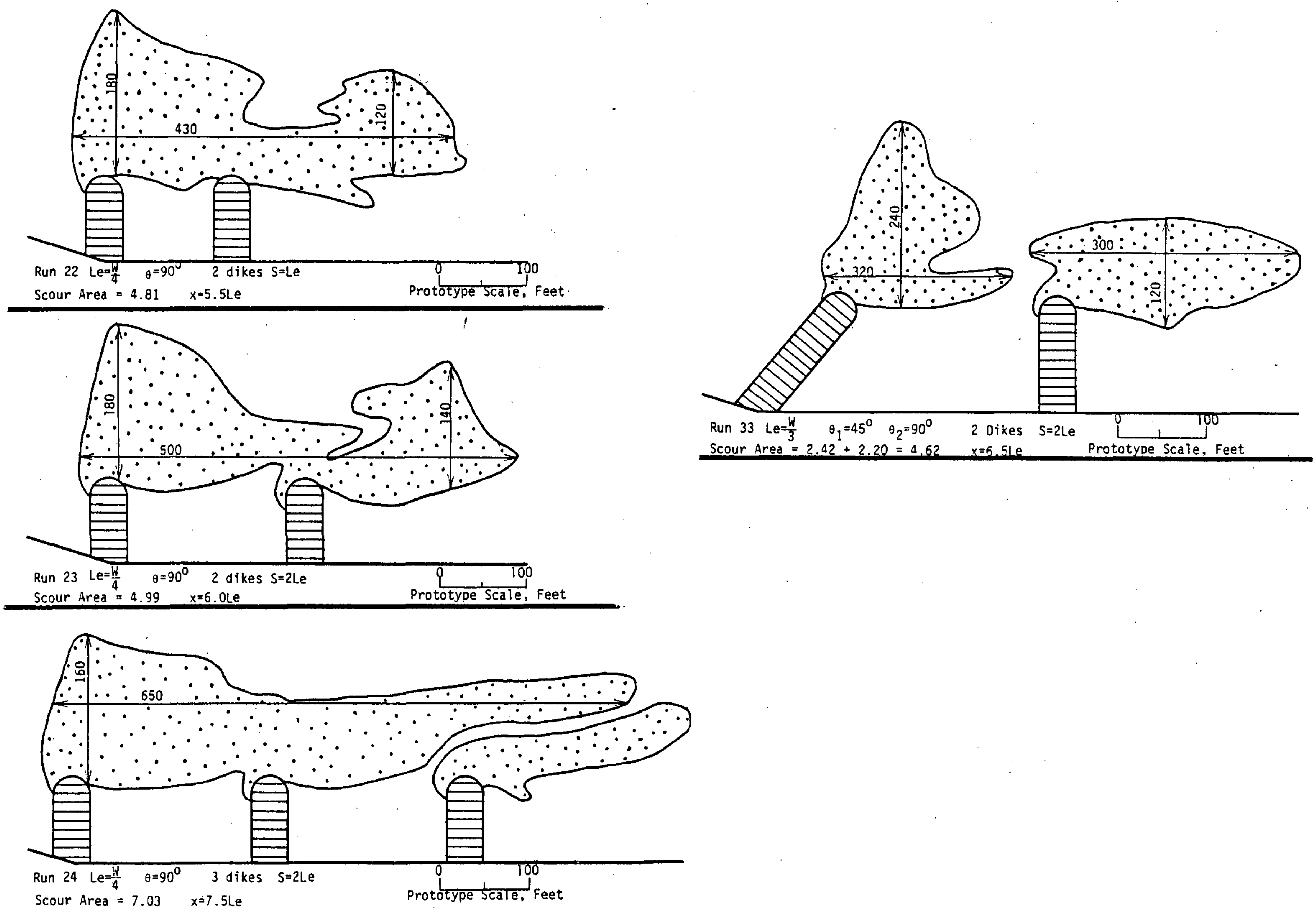


Figure 15. Model Test Results Showing Scour Patterns Associated with Multiple Dikes (Runs 22-24, 33)

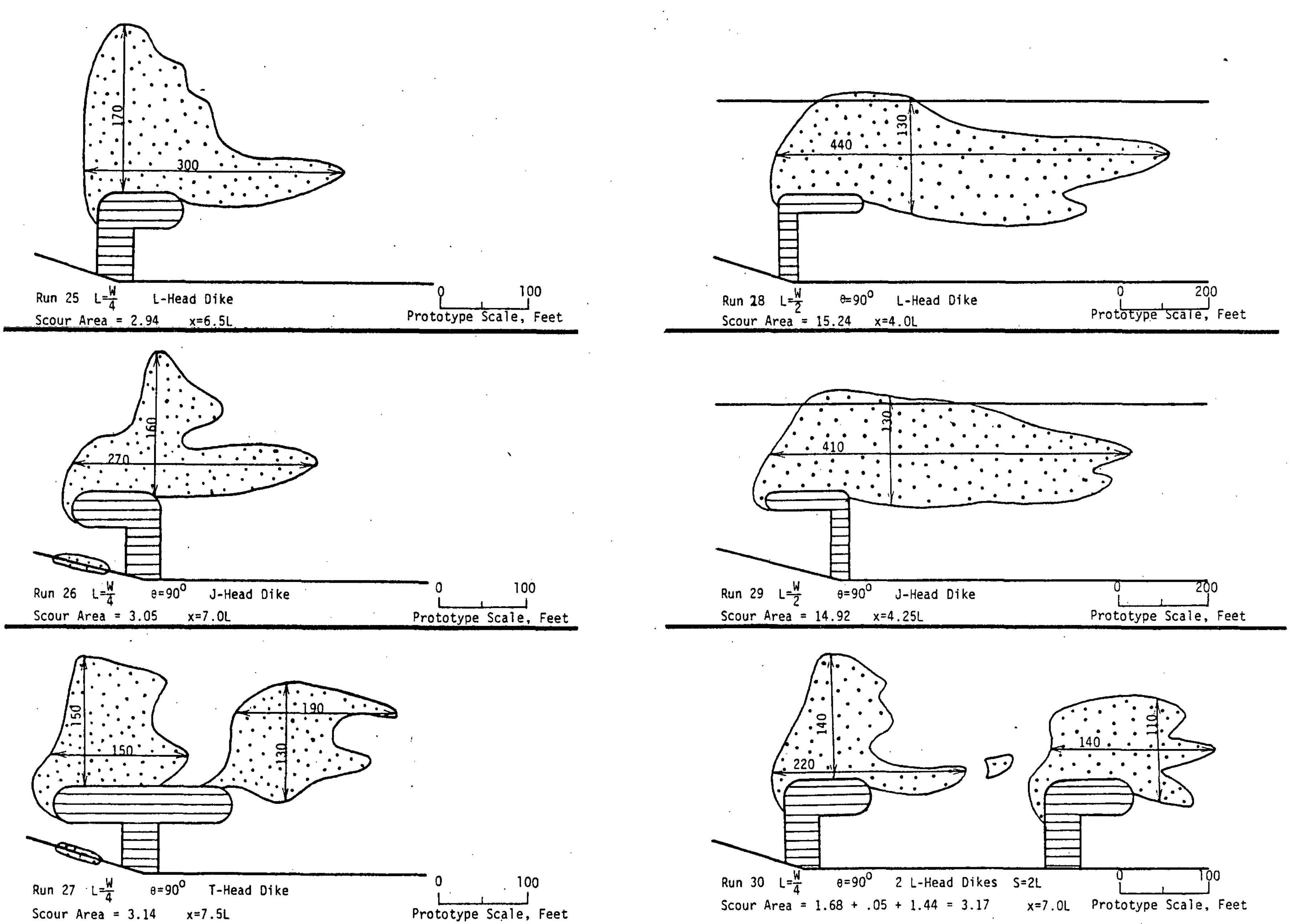


Figure 16. Model Test Results Showing Scour Patterns Associated with Various Dike Shapes (Runs 25-30)

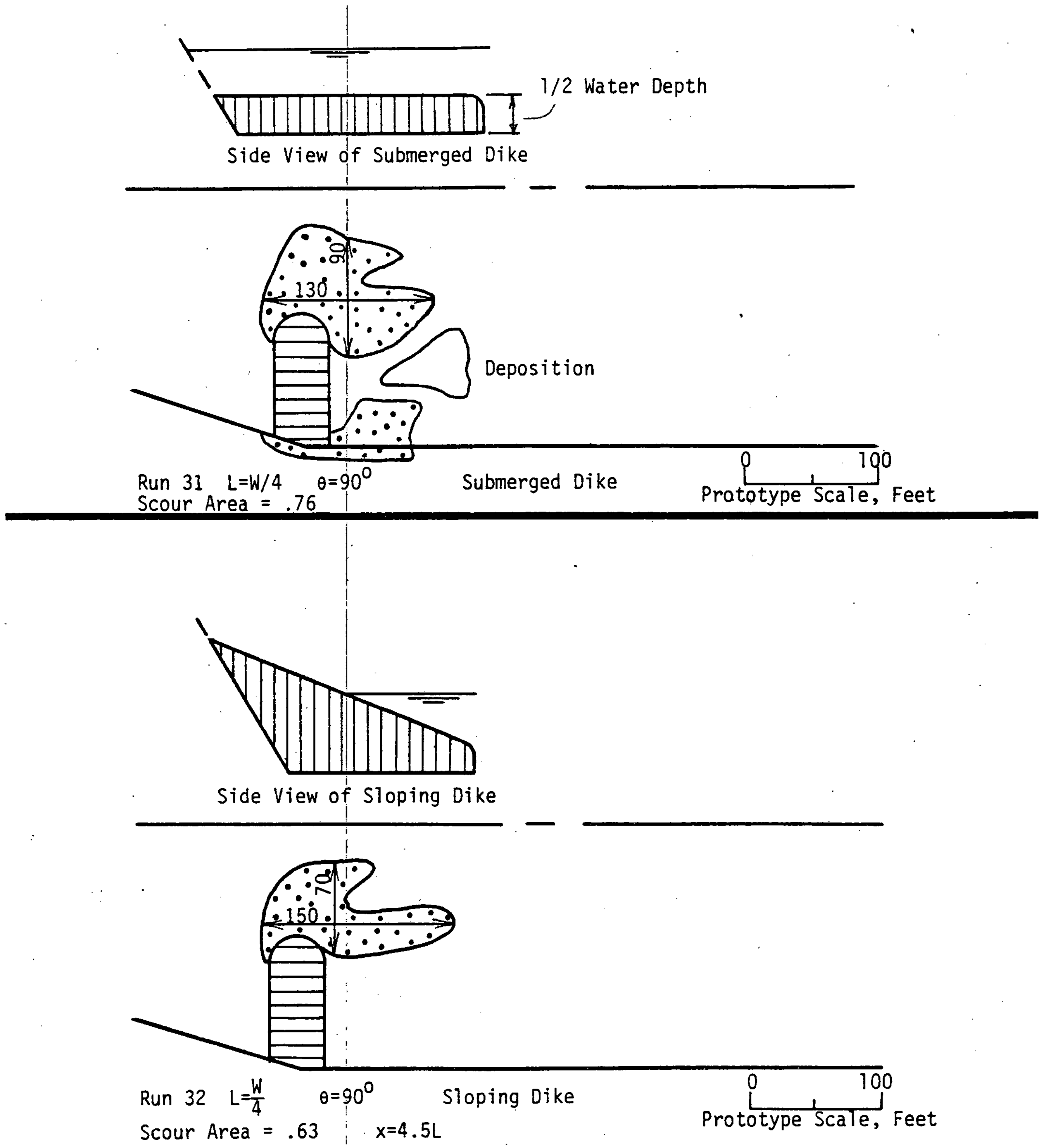


Figure 17. Model Test Results Showing Scour for Partly and Totally Submerged Dikes (Runs 31-32)

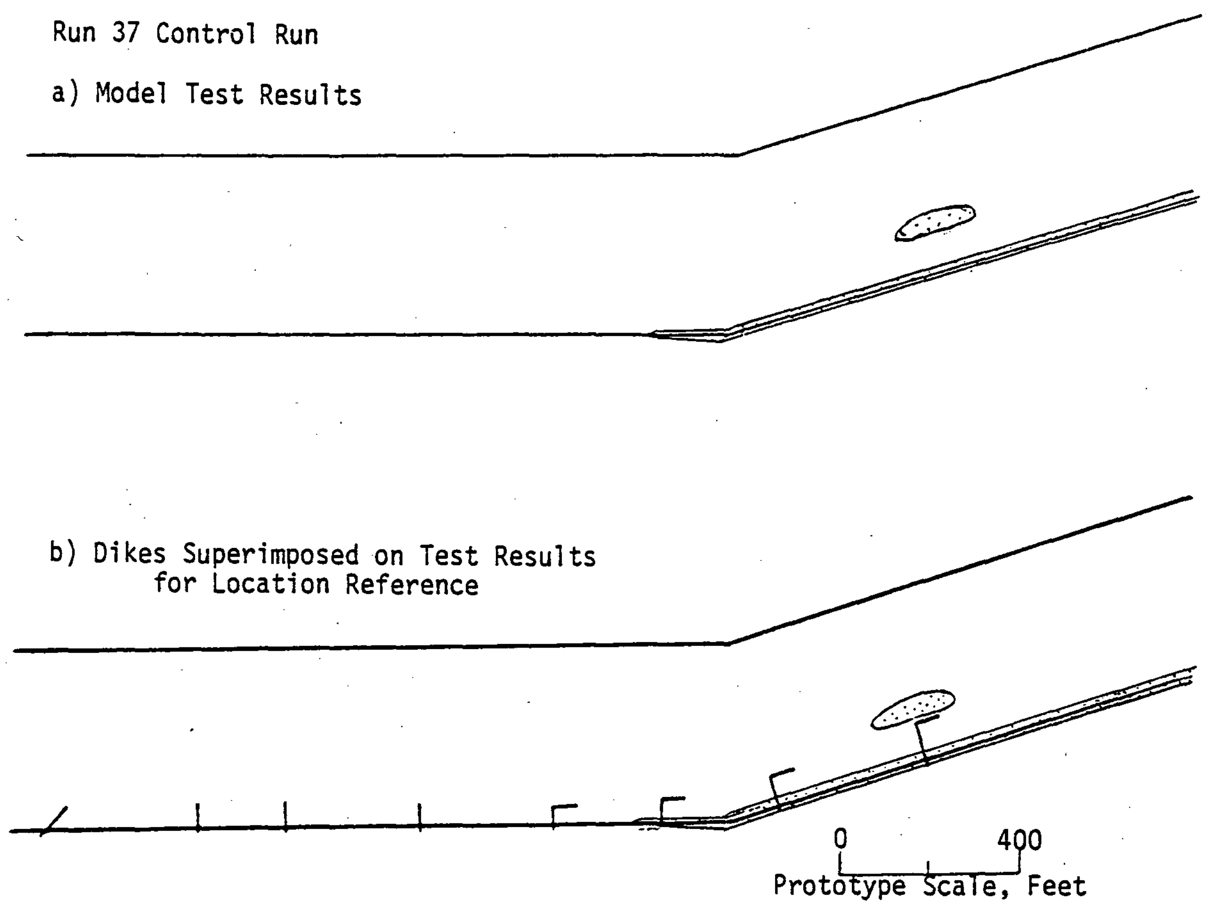


Figure 18. Model Test Results for Prototype River Reach with No Dikes Installed

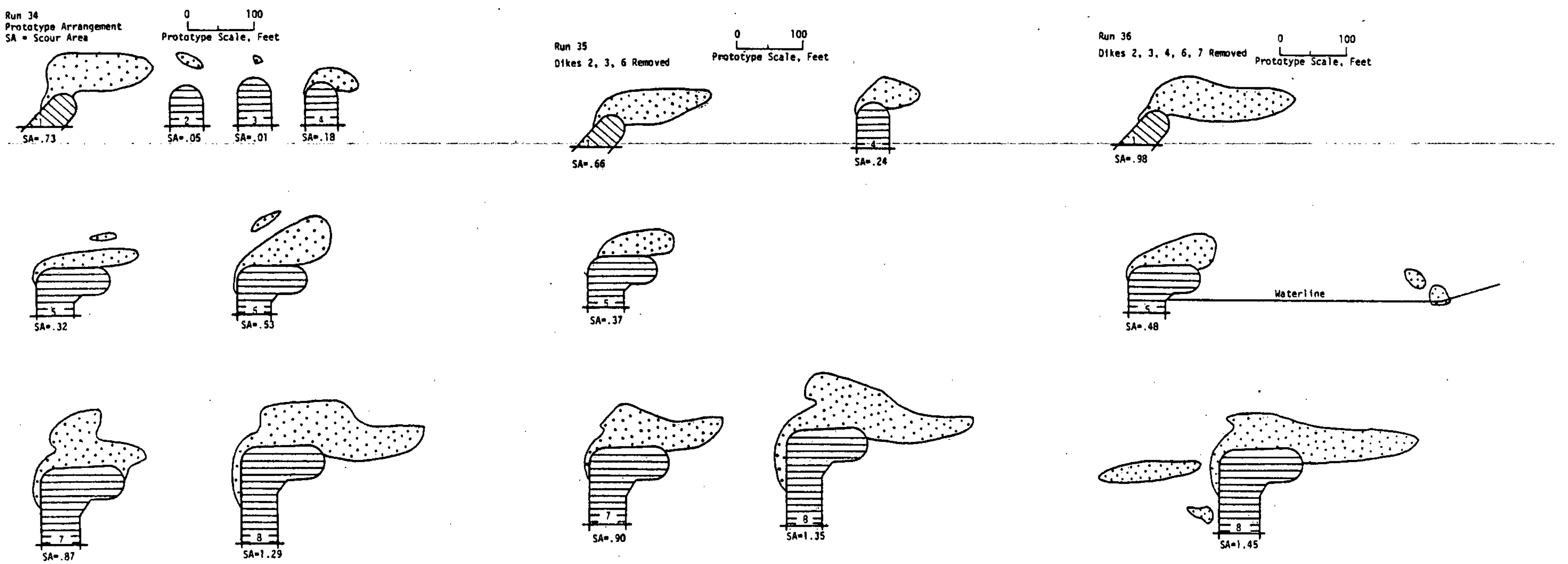


Figure 19. Model Test Results Showing Scour Patterns Associated with the Prototype Dike Field and with Modifications of the Dike Field (Runs 34-36)

protected increases. Thus, using a prototype channel width W of 400 feet and an effective dike length of W/6, the distance downstream from the dike which will be protected would be 4.5 x (400/6) = 300 ft. A dike three times longer will protect the bank for 2.8 x (400/2) = 560 ft.

The distance that the main current is deflected (X_2) behaves in a similar non-linear manner. Thus, for a dike with an effective length of W/6 in a 400 foot wide channel, the deflection distance is 9.0 x (400/6) = 600 ft.; for a dike three times larger (L = W/2), the deflection distance is 6.0 x (400/2) = 1,200 ft.

Figure 20 summarizes the relationships of spur dike length with length of bank protection and with distance of flow deflection. Although there is some scatter of data points, the relationships of relative change are nearly linear and parallel to each other. The trend of diminishing increase of protection distance with increasing dike length occurs for all constant orientation angles.

Effect of Orientation Angle on Bank Protection and Flow Deflection

Figure 13 and Table 5 also show the effect of orientation angle on length of bank protection and on flow deflection distances. If the distance X_1 from the spur dike base to the point of bank erosion is used, the effect of orientation angle on this distance is not entirely clear. However, if the distance X_3 from the spur dike tip to the point of bank erosion is used, it is apparent that increasing the orientation angle increases the degree of bank protection. Figure 20 summarizes these relations.

The upstream-oriented dike is more effective in deflecting the current away from the bank than the downstream-oriented dike. The river current is deflected at nearly a 90-degree angle to the major axis of the spur dike and is directed toward the opposite bank. Therefore, a longer distance downstream

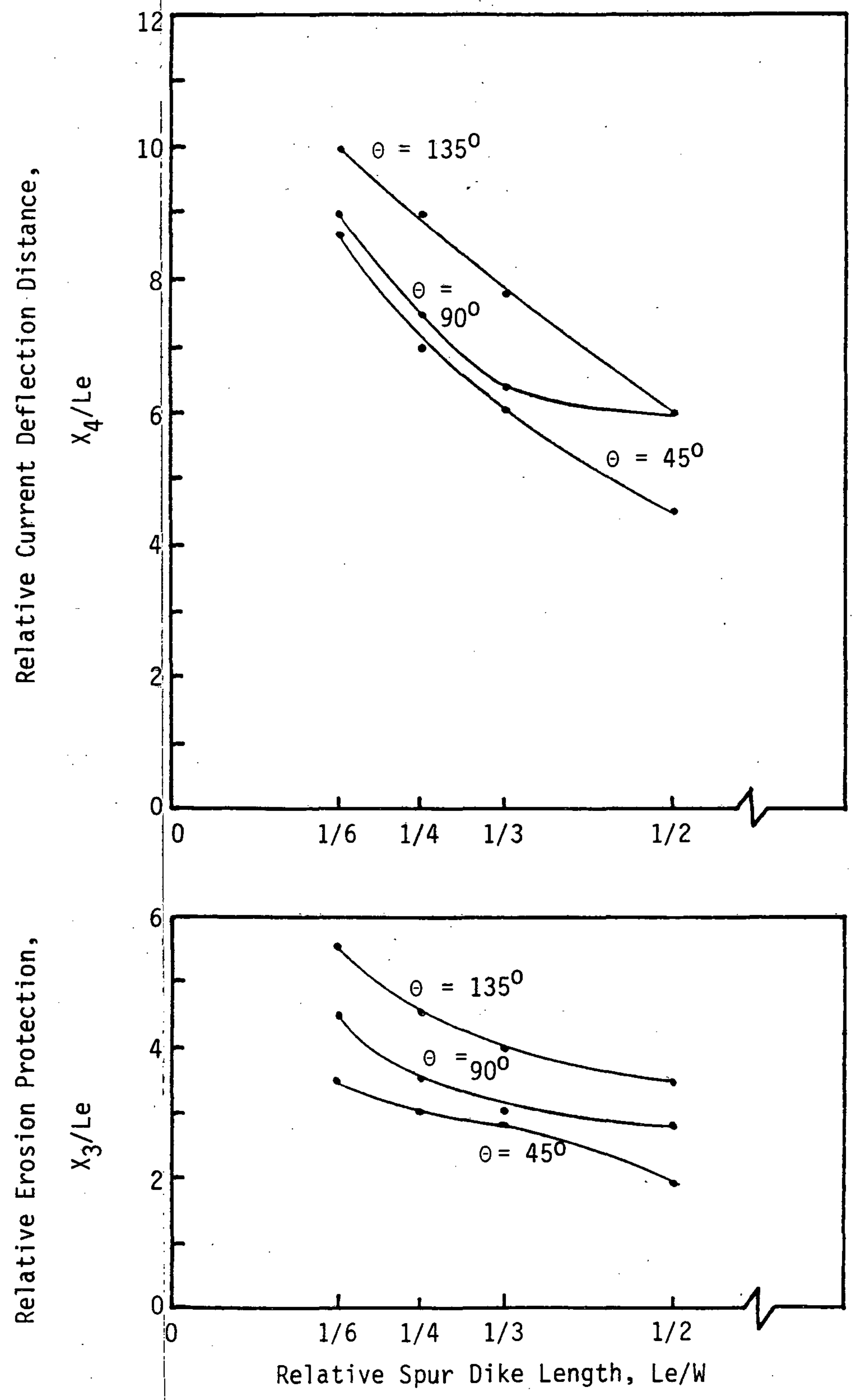


Figure 20. Effect of Spur Dike Length and Orientation Angle on Bank Protection and Current Deflection Distances

is required before the current deflected from a bank returns to that bank. For downstream-oriented dikes, the deflected current may be somewhat attracted towards the bank, resulting in bank erosion at a shorter distance than for the upstream-oriented dike. From Table 5 and Figure 20, the X_3/Le data show that a dike with Le/W of 1/6 and an orientation of 45 degrees will protect a bank 3.5 times its length but if the dike is oriented at 135 degrees it will protect a bank 5.5 times its length; for Le/W of 1/2, the X_3/Le ratios are 1.9 and 3.5, respectively.

For upstream-oriented dikes, bank erosion may occur upstream of the dike (see Figure 14, runs 14 and 20). Part of the impinging flow moves along the upstream side of the dike towards the bank. For long dikes (runs 13-15), an upstream orientation may cause more erosion at the opposite bank than would a downstream orientation.

Effect of Spur Dike Length and Orientation Angle on Scour

The length and orientation of the spur dike apparently have two effects on the scour pattern and size, as can be seen in Figure 14. First, as the dike length increases, the flow section contracts. Because of this, general bed erosion can occur in the contracted section and at the opposite bank. Second, varying vortices develop, depending on the angle and length of the spur dike. These cause local scour around the spur dike.

Table 6 shows the effect of dike length and orientation angle on scour area and flow deflection distance. As the effective spur dike length increases, the scour area also increases. This is shown in Figure 21. With θ = 90 degrees and Le/W of 1/6, the scour area is 0.43 in^2; for Le/W of 1/4, the scour area is 2.80 in^2; and for a ratio of 1/2, the area is 17.21 in^2.

As the orientation angle increases, the size of the scour hole also increases. Figure 21 shows that for Le/W of 1/2, the scour area increases

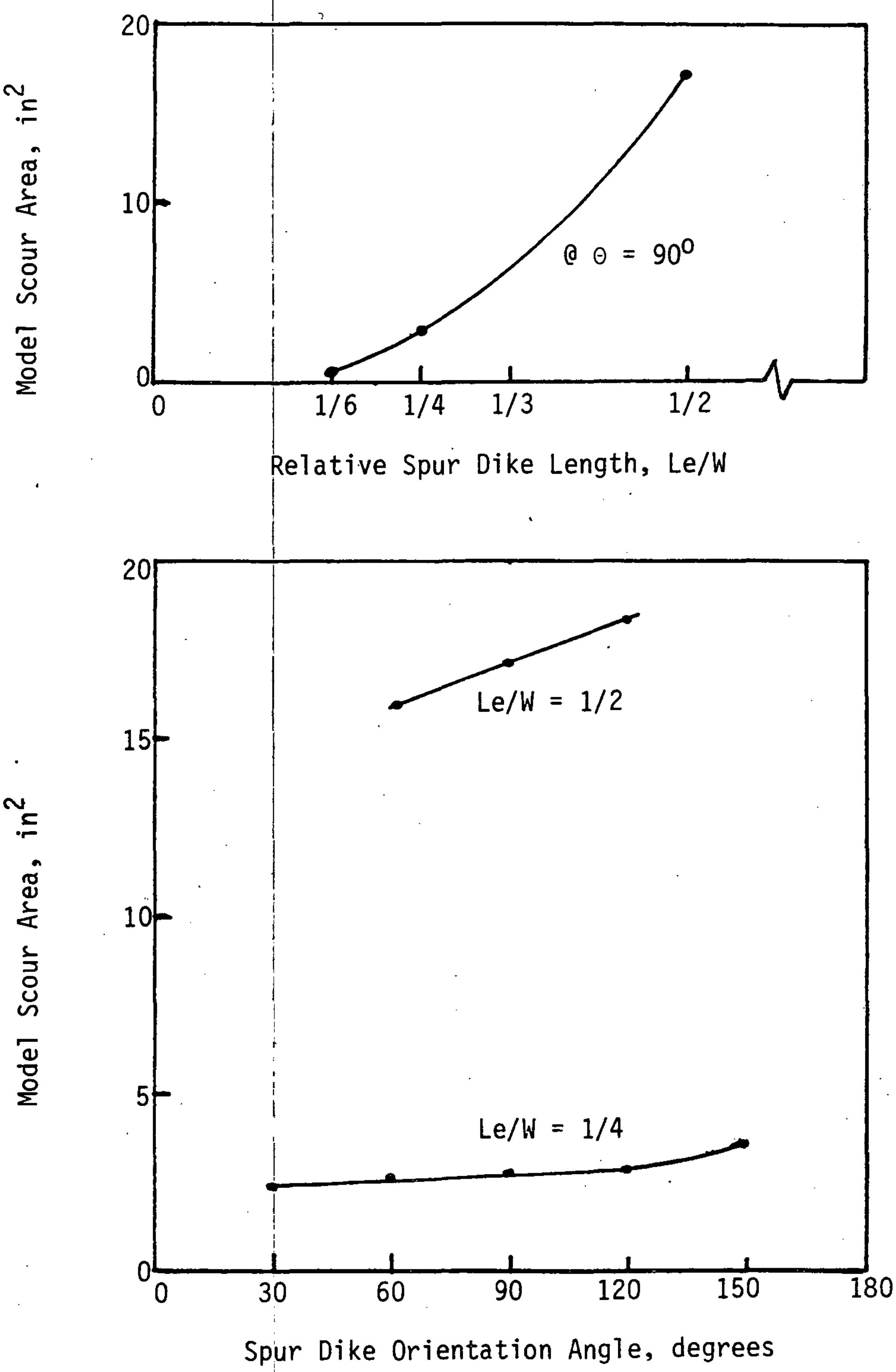


Figure 21. Effect of Spur Dike Length and Orientation Angle on Scour Area

linearly as the angle increases from 60 to 120 degrees. For Le/W of 1/4, the scour area increases linearly from 30 to 120 degrees but more rapidly from 120 to 150 degrees. The scour dimension perpendicular to the bank is greatest for a dike oriented at 90 degrees (see Figure 14). The scour dimension parallel to the bank is greatest for a dike oriented at 30 degrees or 150 degrees (runs 16-20 in Figure 14).

The amount of scour upstream of the spur dike tip increases as the spur dike becomes more upstream-oriented. This trend is evident in Figure 14.

Effect of Spur Dike Shape on Scour Area and Bank Protection

Figure 16 shows the scour patterns that are caused by spur dikes of various shapes. Two degrees of channel contraction were tested: Le/W of 1/4 and 1/2. Table 7 summarizes the effect of spur dike shape on scour area. Data on current deflection are also given. Figure 16 and Table 7 show that the T-head dike causes a slightly larger scour area and deflection distance than the other shapes for a given Le/W ratio. The L-head dike produced the smallest scour area but was also the least effective in deflecting the current. The J-head and T-head dikes caused bank erosion to occur upstream of the dike. The T-head caused a double scour area to develop (see Figure 16, run 17).

Effect of Spur Dike Submergence

Figure 17 shows that a totally submerged spur dike experiences bank erosion near its root. Some of this eroded bank material was deposited just downstream of the dike.

For a sloping dike, the scour area and current deflection distance were similar to those to be expected from an unsubmerged dike having a length equal to the exposed portion of the sloping dike.

Table 7. Effect of Spur Dike Shape on Scour Area and Current Deflection Distance (Runs 25-29)

Shape	Scour Area*	X_4/Le
<u>L/W = 1/4</u>		
L-Head	2.94	6.5
J-Head	3.05	7.0
T-Head	3.14	7.5
<u>L/W = 1/2</u>		
L-Head	15.24	4.0
J-Head	14.92	4.25

*Scour area measured in square inches in the model. For prototype scour area in square feet, multiply by 2500.

Table 8. Scour Areas for Model Tests of Prototype Dike Arrangement and Effect of Removing Various Dikes (Runs 34-36)

Dike and Scour Area, square inches*								
1	2	3	4	5	6	7	8	Total
.73	.05	.01	.18	.32	.53	.87	1.29	3.98
.66	--	--	.24	.37	--	.90	1.35	3.52
.98	--	--	--	.48	--	--	1.45	2.91

*For prototype scour area in square feet, multiply by 2500.

Effect of Multiple Dikes on Scour Area and Bank Protection

Figures 15, 16, and 19 show the scour patterns that result from multiple dikes in a dike field. The individual scour patterns tend to merge when the dikes are closely spaced. Bank protection between adjacent dikes is very good. Multiple spur dikes appear to afford some mutual protection from scour-producing currents.

A comparison run 18 in Figure 14 (single dike at 90 degree orientation) and runs 23 and 24 in Figure 15 shows that as the number of dikes increases (from one to three dikes), the total scour area increases less rapidly. The current deflection distance beyond the downstream dike also increases (from 5.5L to 7.5L).

Modeled Prototype Dike Arrangement and Comparison With Field Study

Figure 19 and Table 8 show the scour areas that were determined in model tests of the prototype dike arrangement. The effects on local scour at each remaining dike and on total scour at the dike field due to removing some dikes from the dike field are also shown.

The scour patterns that developed from the model test of the Willamette River Reach without dikes (Figure 18, run 37) and with the prototype arrangement of dikes (Figure 19, run 34) compare reasonably with the actual patterns observed before dike construction and after dike construction, respectively. The amount of scour measured near dikes 2, 3 and 4 in run 34 was very small (see Figure 19 and Table 8). During run 36, in which dikes 2, 3, 4, 6, and 7 were removed, bank erosion occurred between dikes 5 and 8 but little bank erosion was observed between dikes 1 and 5. Dikes 2, 3, and 4 apparently contributed little protection to the bank in that part of the reach. During the field investigation it was observed that dike 1 deflected the river current sufficiently that dikes 2, 3, and 4 provided little additional benefit. Based on the model study, at least one of those dikes could have been omitted from the dike field with little effect on bank protection. The model test also showed that bank erosion occurred downstream of dike 8. The field investigation also revealed that bank erosion was occurring downstream of dike 8 and that perhaps an additional dike was required there.

Data summarized in Table 8 show that the total scour area for the dike field diminished when some dikes were removed from the dike field but scour at the individual remaining dikes increased (see also Figure 19). However, more than three dikes appear to be required to adequately protect the riverbank in that reach.

Field Study

Background

During the summer of 1983, a spur dike field (called a groin field by the designers) was constructed along 1800 feet of bankline of the Willamette River near River Mile 136 approximately two miles southeast of Corvallis, Oregon. Streambank protection was mandated because erosion at the location, estimated at 10 to 30 feet per year, was affecting cultivated farmland and because of the potential formation of a new channel away from the city's principal water intake. A spur dike system was chosen over conventional riprap bank revetment for environmental reasons, to diversify fish habitat through the creation of deepwater zones at scour holes and slackwater areas between the dikes. Figure 22 shows an aerial view of the dikes, from an infrared color photograph taken on October 1, 1983. The river discharge is approximately 7,700 cfs.

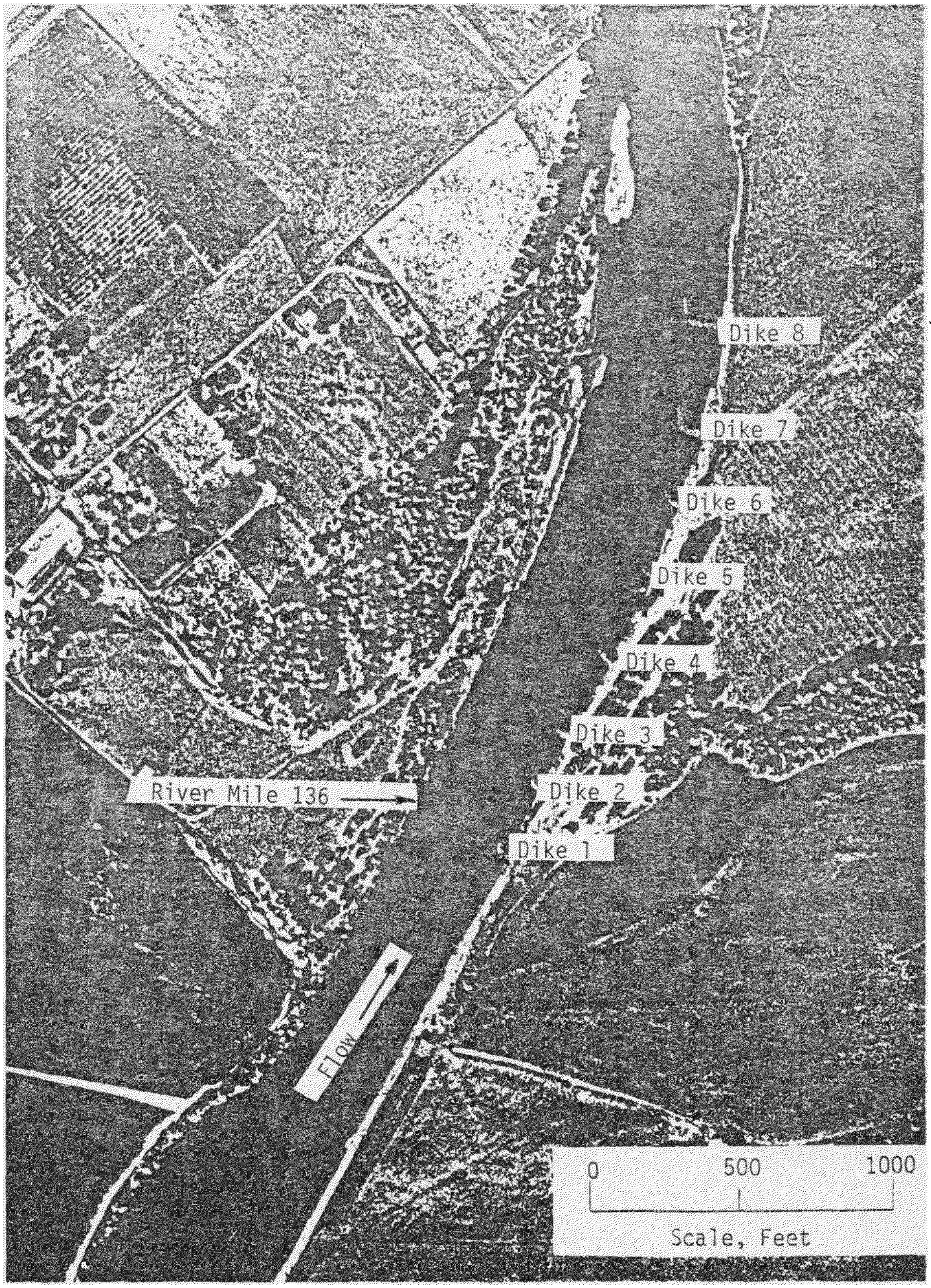


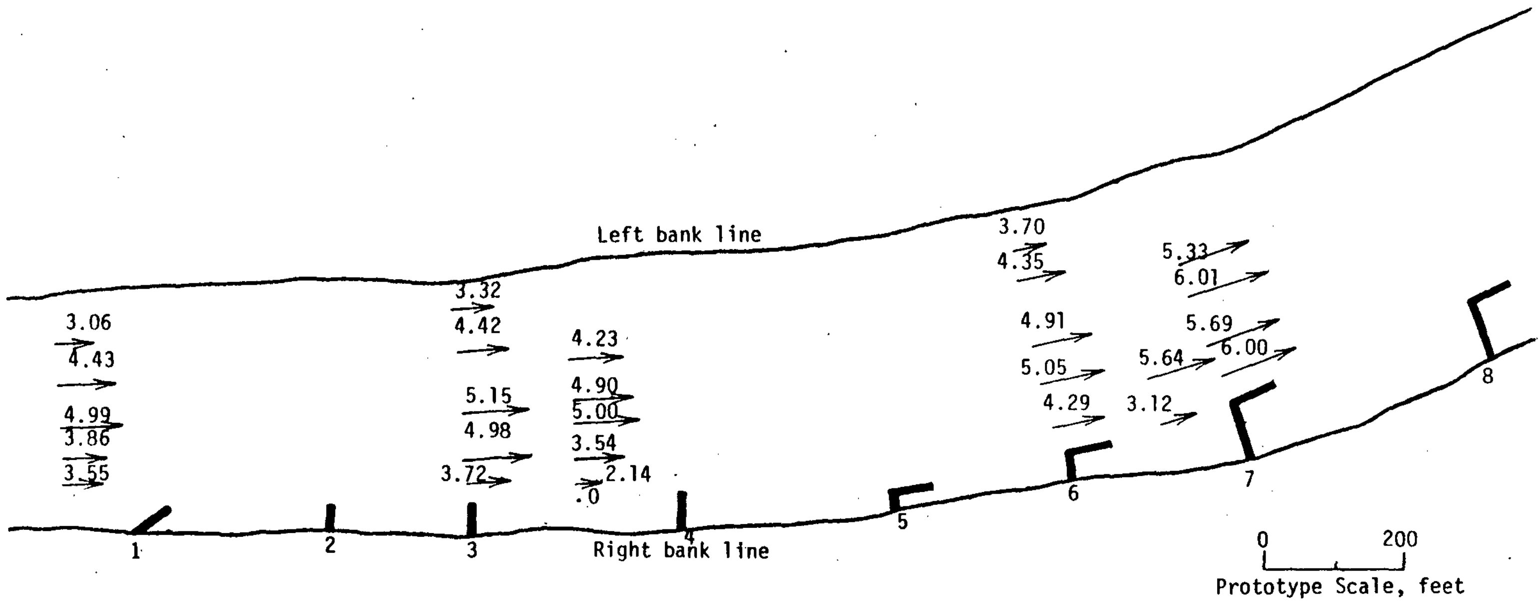
Figure 22. Willamette River Spur Dike Field Upstream of Corvallis

A model study was conducted for design purposes by the U.S. Army Corps of Engineers at the U.S. Army Engineer Waterways Experiment Station in Vicksburg, Mississippi. The design solution consists of eight spur dikes spaced 250 to 350 feet apart, extending 50 to 115 feet into the river (at crest elevation) and consisting of rockfill and riprap. Dike 1, the extreme upstream dike, is oriented 40 degrees from the bank in a downstream direction. Dikes 2-4 are oriented normal to the bank. The four downstream dikes (dikes 5-8) are L-shaped with extensions approximately 60 feet long and parallel to the bank. A 3-foot layer of class V riprap was placed on the upstream side of each dike and a 2-foot layer of class III riprap was placed on the downstream side to protect the dikes from scour and debris.

Research Procedures, Equipment and Data

One purpose of our field investigation was to gather the necessary data to determine the hydraulic effects of spur dikes on river flow and bed topography. Another purpose was to compare observations with our laboratory findings.

In mid-September 1983, soon after dike construction was completed, a detailed site survey was conducted. This included current velocity measurements, surface current patterns, river cross-sections, and streambed bathmetry. Current velocities were measured with a Price current meter at depths equal to 20 and 80 percent of the total depth. From these, the depth-averaged velocity was calculated. The depth-averaged velocities are shown in Figure 23. Surface current patterns around the spur dikes were sketched at the time of velocity measurements. These are shown in Figure 24. A fathometer with strip-chart output was used to record water depths. Cross-sections were taken at stations upstream of the spur dike field, at and between each dike, and downstream of the dike field. Cross-sections were



Velocities shown are in feet per second and represent average values for vertical water column at each point

Figure 23. Average Current Velocities at New Spur Dike Field, 14 September 1983

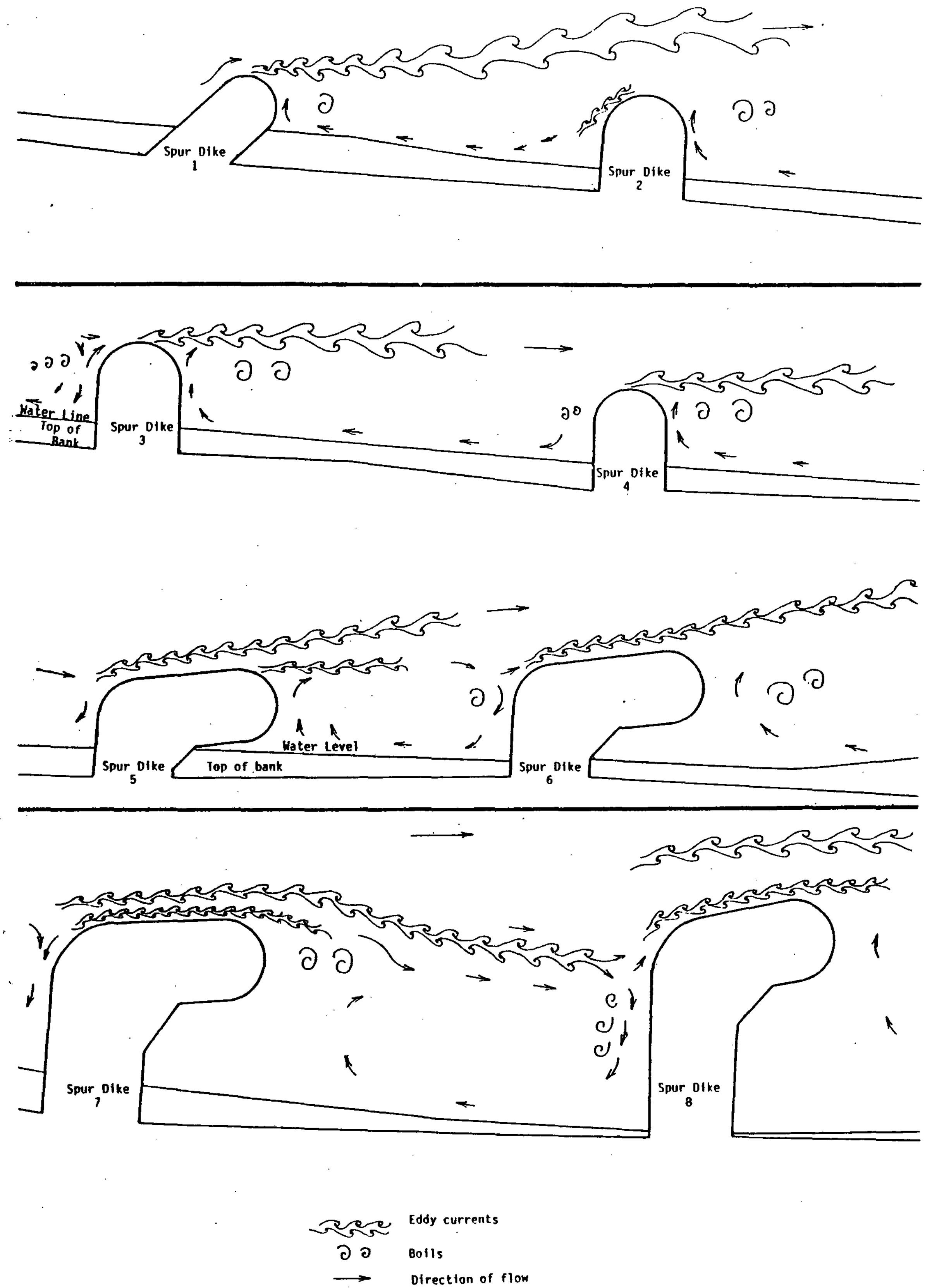


Figure 24. Surface Current Patterns at New Spur Dike Field, 14 September 1983

also taken parallel to the current flow along the river center line, 20 feet from the dike tips, 10 feet from the dike tips, and 20 feet from the bank line in between the dikes. For dikes 1, 3 and 7, cross-sections were also taken radially around the dikes approximately 10 feet and 20 feet from the dike edge. A contour map of the river bed was constructed using the data obtained from the fathometer recordings. This is shown in Figure 25.

To determine the evolution of scour patterns around the spur dikes, subsequent site surveys were conducted in mid-winter 1983-84, after a few months of high water allowed scour to rearrange the river bed and flow patterns. Surveys were repeated in summer 1984 to observe the effects of a full season of high-water conditions. Figure 26 shows the effects on scour and deposition after a year of dike performance.

Discussion of Field Investigation

The initial field investigation was conducted during late-summer low-flow conditions. The river discharge was approximately 7,000 cfs. High-flow winter conditions are much greater, with a two-year flood hydrograph discharge of about 50,000 cfs. At the time of the initial field investigation, local scour around the spur dikes and general streambed erosion had not yet had an opportunity to adjust to initial high water discharge. The scour was therefore expected to increase during the following winter season.

Table 9 contains the prototype spur dike lengths in terms of the river width, spacing ratios in terms of both the spur dike length and river width, and initial scour hole depths. The spacing ratios are greater than the typically recommended values of 2L to 4L given in Table 3. However, there was no observed bank erosion between the dikes.

The current velocities were greatly accelerated as they passed the spur dikes, due to the converging flow. The trailing eddy currents from one dike

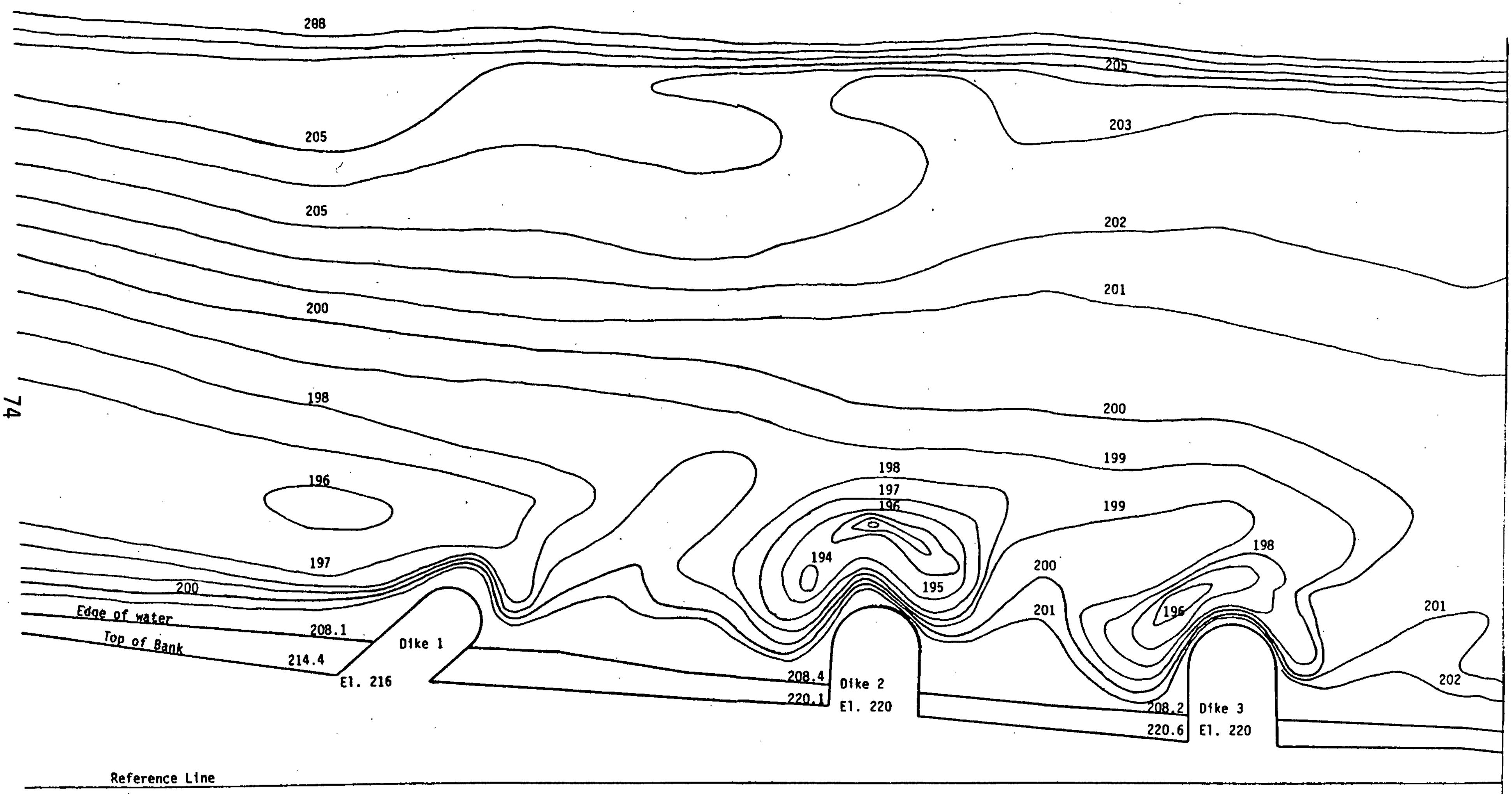


Figure 25. Contour Map of River Bed at New Spur Dike Field, 14 September 1983

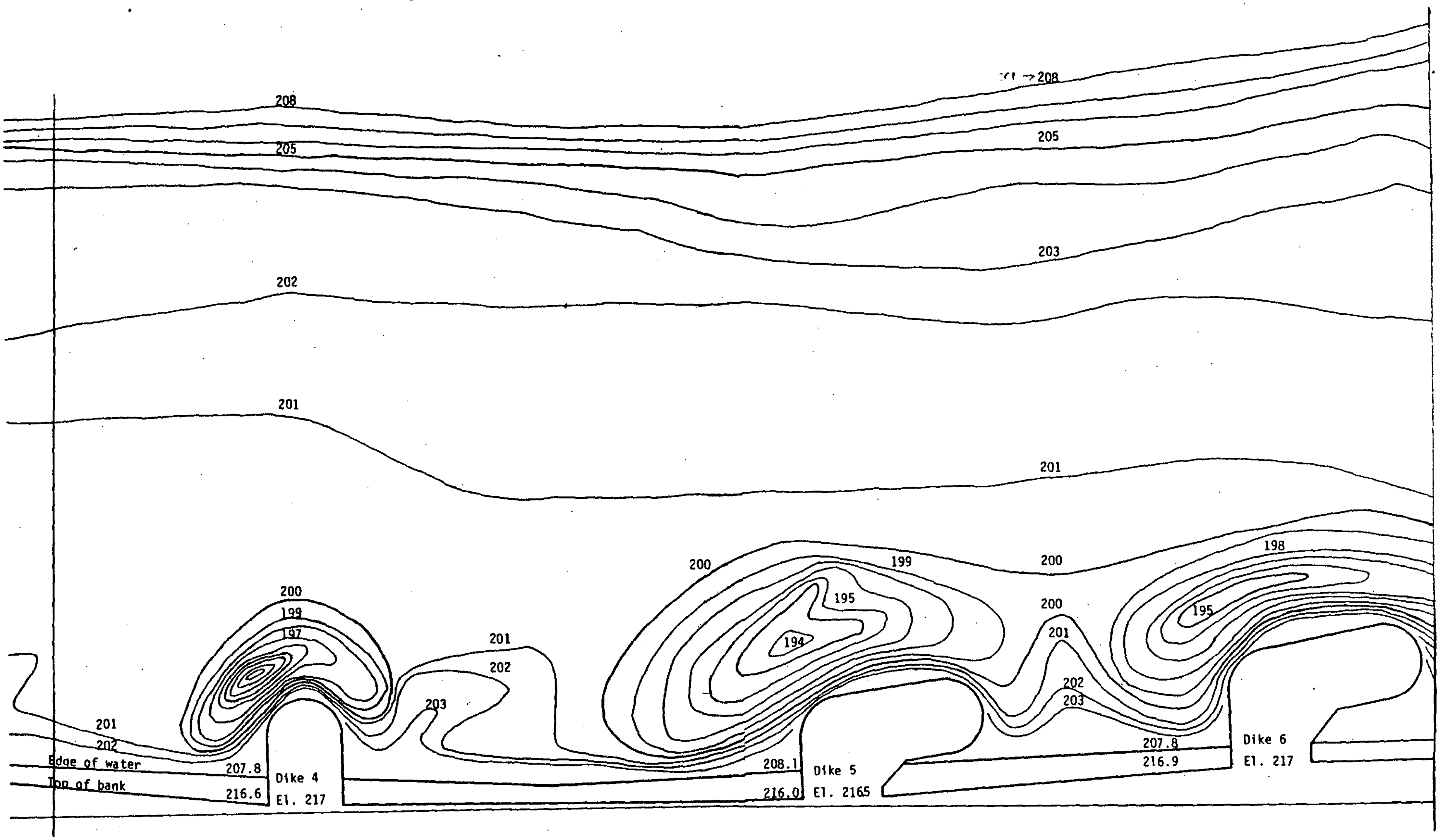


Figure 25. Continued

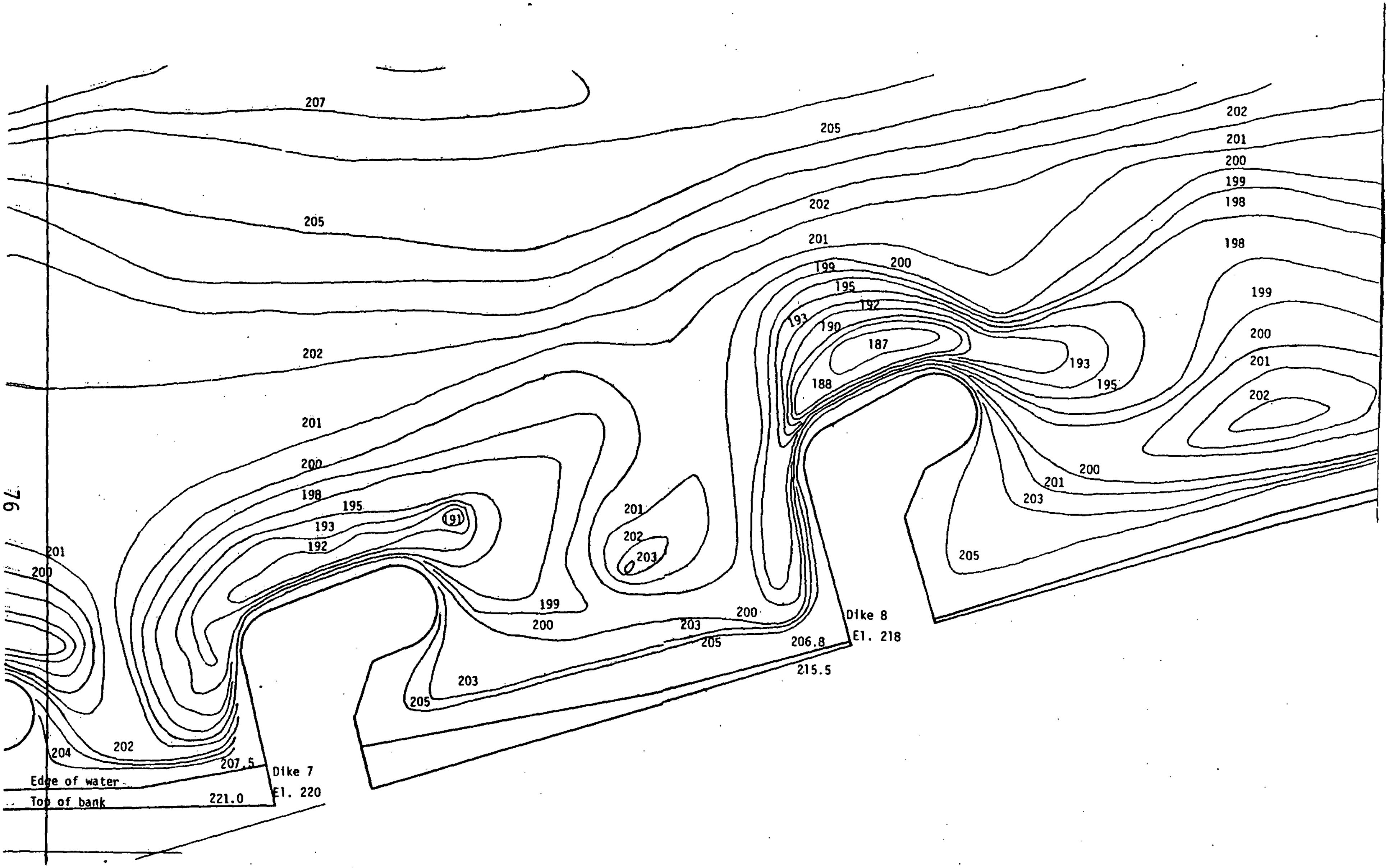


Figure 25. Continued

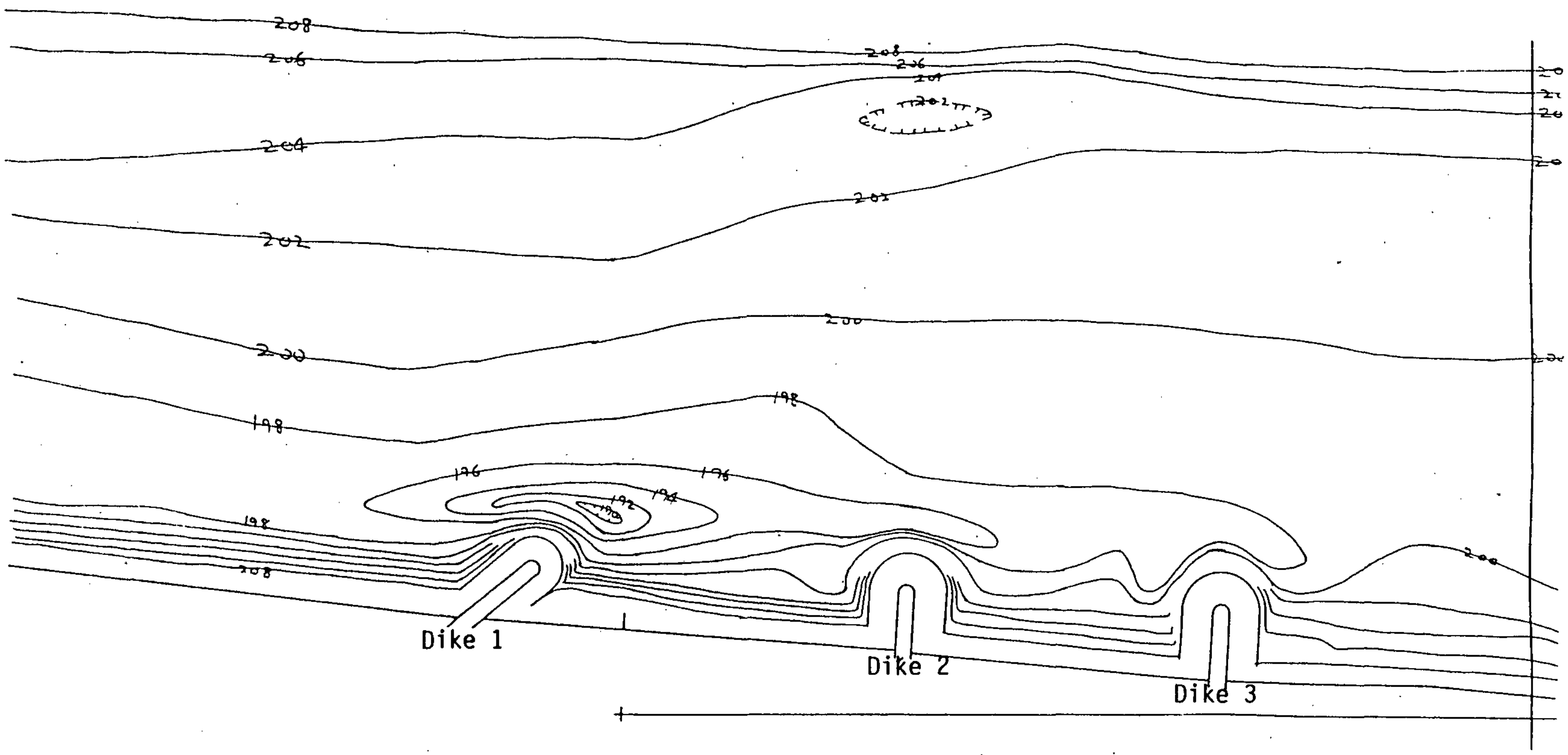


Figure 26. Contour Map of River Bed at Spur Dike Field After One Year of Use

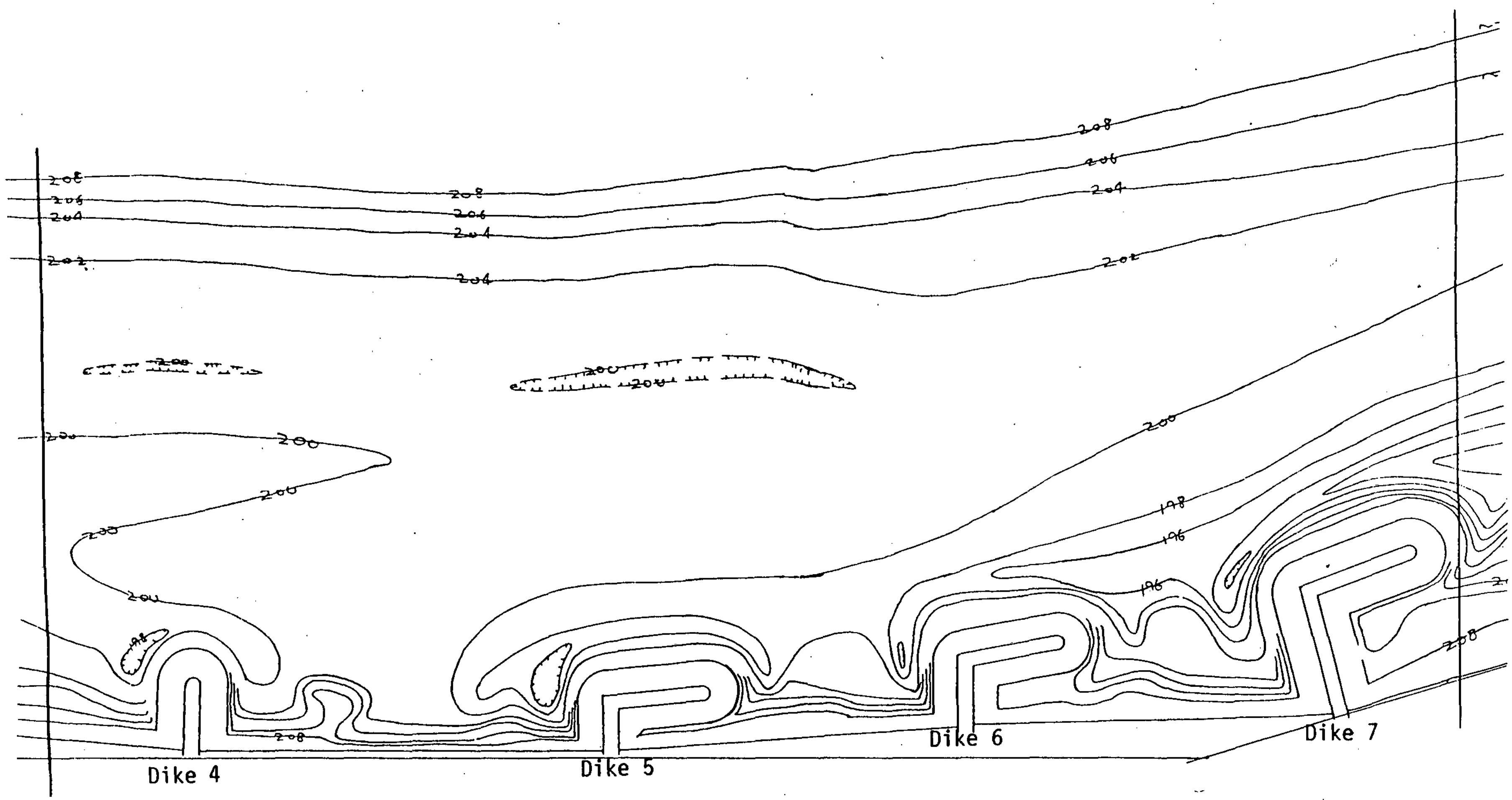


Figure 26. Continued

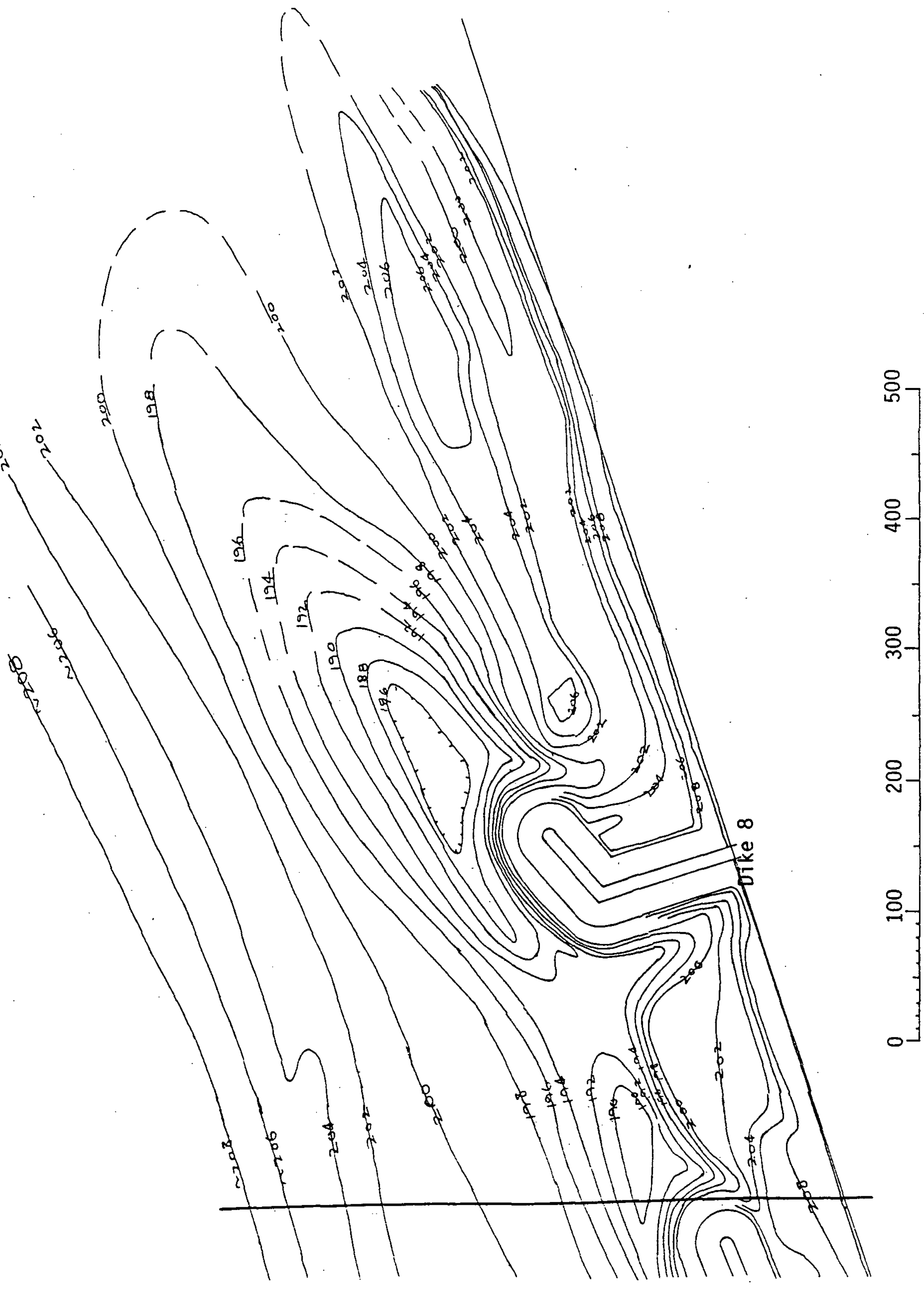


Figure 26. Continued

Table 9. Spur Dike Lengths, Spacing Ratios and Initial Scour Hole Depths in Prototype Spur Dike Field

Spur Dike Number	Length/Width L/W	Spacing/Length X/L	Spacing/Width X/W	Initial Scour Hole Depth (ft)
1	1/9.1			4
		6.0	0.65	
2	1/9.1			5
		4.4	0.52	
3	1/7.7			4
		5.6	0.72	
4	1/7.7			8
		8.0	0.81	
5	1/12.0			6
		6.3	0.64	
6	1/7.7			5
		3.7	0.60	
7	1/4.7			10
		4.0	0.88	
8	1/4.3			13

tended to imprinqe near the tip of the next downstream dike, causing divided flow, with strong currents going around the dike and moderate currents flowing toward the base of the dike long its upstream side. In the compartments between the dikes, the velocities were greatly reduced. An upstream current formed due to eddy effects from the downstream dike and from spreading of the strong current leaving the upstream dike.

Large discharges during winter and spring increased the ability of the flow to scour the bed near the dikes along flow trajectories past the dike tips. The dikes thus became more effective in altering flow patterns. The flow trajectory past dike 8 extended toward mid-channel, where a shallow bar had previously been, and then spread out so that a weaker current impinged on the eroding bank downstream of the dike than had been the case before the dike was built. Some deposition occurred just downstream at dike 8 along the edge of the wake zone. All banks between dikes were well protected and much debris (trees, logs and branches) was carried into the zones between dikes, where it became stranded. Figure 26 shows the streambed contours near the dikes after one year of interaction with the river.

Summary and Conclusions

Local scour around spur dikes and similar structures and the degree of bank protection provided are affected by many factors, including structure characteristics and streamflow characteristics. While the general qualitative effects of these factors have been researched and documented, few quantitative relationships are available for use as design aids. Recommended spur dike orientation angles and spacing ratios vary greatly, depending on the researcher and source. Equations for predicting scour hole depths around spur dikes are questionable, as the results deviate greatly. Model testing may be the most important and effective means to predict results and aid in the design process.

In this part of the research, various spur dike shapes, orientation angles and arrangements were investigated, both experimentally and with a prototype field study. Under the limitations imposed by the model, the following main conclusions can be made:

(1) The degree of bank protection provided by spur dikes is a function of the spur dike length, orientation angle and spacing.

(2) As the length of the spur dike increases, the protected distance downstream of the dike to where bank erosion begins to occur increases, but not proportionately with the increasing spur dike length. In the model tests, a spur dike could protect a bank from 2.5 to 4.5 times its own length, depending upon the spur dike length.

(3) Upstream-oriented spur dikes are more efficient than other orientations in deflecting the river current away from the bank. Therefore, upstream-oriented spur dikes provide bank protection farther downstream from the dike tip.

(4) Upstream-oriented spur dikes cause more extensive scour holes than do downstream-oriented spur dikes. This is because of the increased flow disruption resulting from the upstream orientation. From our small-scale tests it is not known whether the scour hole depth also increases as the area increases, due to upstream-oriented dikes.

These conclusions are consistent with the past studies and literature cited earlier.

IV. USE OF GABIONS

General Features of Gabions

Gabions are wire baskets filled with rocks. The baskets are usually rectangular in shape. They are made of steel wire that is machine-woven in a uniform hexagonal triple-twist pattern. The steel wire may be galvanized with a zinc coating as a rust control measure (Maccaferri Gabions of America, undated-b; Bekaert, 1977).

Gabions are available in different sizes to suit conditions of terrain and application. Typical gabion lengths are 2, 3 and 4 meters. Typical widths are 1 meter. Typical heights are 1 foot, one-half meter, and one meter. Gabions are supplied flat, packed in bundles. Assemblying the gabion involves folding it up to form a rectangular box and wiring it at the edges and at all connections except for the lid. The gabion is then filled. The filling material usually consists of hard, durable stones larger than the wire mesh opening of 3 in. x 4 in. Once filled, the gabion lid is wired closed.

Gabions may be filled by hand or mechanically. A wide variety of earth-handling equipment may be used, such as payloader, grade-all, crane, conveyor, or modifed concrete bucket. Some manual adjustments of the stones are required during the mechanical filling operation in order to eliminate undue voids.

History of Gabion Use

The history of gabions dates back to antiquity. The Egyptians used gabion-like structures to build dikes along the Nile about 5000 B.C.

(Bekaert, 1977). The Chinese are said to have used similar structures along the Yellow River about 1000 B.C. In his ten books of Architecture written in about 20 B.C., the Roman Architectus Vitruvius described the use of gabions as cofferdams. The early gabions were woven from plant fiber; as such they were not very durable.

In their modern form, gabions have been used in Europe quite extensively since the late 1800's. In American construction, gabions are relatively new. However, today they are used more and more frequently to control erosion and to line channels.

Gabions have been used in many situations. These include: river training and flood control; channel linings; retaining walls; bridge abutments and wingwalls; marinas and boat ramps; culvert headwalls and outlet aprons; and shore or beach protection.

Advantages and Disadvantages of Gabions

Gabion structures are considered to be useful structures due to their low cost, ease of installation, flexibility, durability, permeability, and natural appearance. One of the main advantages often cited for gabions over other types of engineering structures relates to their use for installations on unstable foundations (Maccaferri Gabions of Canada, undated). Burroughs (1979) discusses the increasing use of gabions in the U.S. and their economical and environmental advantages. The following is a summary of the reported advantages of using gabions.

Flexibility. The gabion structure is flexible. Its triple-twist hexagonal mesh allows it to tolerate differential settlement without being damaged. This feature is essential when the installation is on unstable ground or in areas where scour from waves or stream currents can undermine the structure.

Strength. The strength and flexibility of the steel wire hexagonal mesh enables the gabion to withstand forces generated by water and earth masses. The pervious nature of the gabion allows it to absorb and dissipate much of the energy developed. This is particularly so on coastal protection installations where gabions are known to have remained effective long after massive rigid structures have failed.

Durability. Plant growth over the gabions, after the voids between the individual stones are filled with soil, becomes a living coating for the wire mesh and stones. The soil, silt, and plant roots become bonding agents for the stones. Moreover, the triple-twisted hexagonal mesh will not unravel if cut. All this enhances the durability of the gabion structure.

Permeability. The gabion wall allows water to drain and stabilizes a slope by the combined action of draining and retaining. Drainage is achieved by gravity and evaporation, as the porous structure allows air circulation through it. Furthermore, as vegetation grows over the structure, transpiration further assists in removing moisture from the backfill. Thus, hydrostatic heads are unlikely to develop behind a gabion wall. This system is more efficient than weep holes in standard masonry walls.

Landscaping. By permitting the growth of natural vegetation and maintaining the natural environment of an area, gabions provide attractive and natural building blocks for decorative landscaping. They can be used effectively and economically in parks, along highways, to beautify the banks of lakes, ponds, and streams.

Economy. Compared to rigid or semi-rigid structures, gabions are more economical. The reasons are as follows: construction is simple and does not require skilled labor; stone fill is usually available on site or from nearby quarries; preliminary foundation preparation is not needed beyond having the surface reasonably level and smooth; no costly drainage provision is required, as gabions are porous; and little maintenance is needed.

There are also reported disadvantages in the use of gabions. A major criticism is that if underdesigned, they will ravel up due to scour and be carried away or become a potential hazard. Their use is sometimes discouraged for aesthetic reasons; the appearance of wire baskets filled with rocks may be considered undesirable. The use of gabions may be discouraged for fear that the wire basket may endanger fish through abrasion. If coarse bed load is transported in a stream, abrasion may cause the wires to break and the gabion to fail.

V. USE OF GABION GROINS

Overview

This part of the report describes the use of gabions for groins. The general concepts involved in their use are similar to those already discussed for riprapped rockfill spur dikes and groins. The gabion groin structures tend to be smaller than riprapped rockfill groins and to differ in their applications.

With regard to gabion groins, the objective of work discussed here is to determine what arrangement (in terms of groin length, spacing between groins, and groin orientation to the flow) will provide optimum streambank protection while improving fish habitat at the same time.

General Features of Gabion Groins

A groin may be defined as an elongated structure protruding into a flowing stream or river from the bank. The root of this structure is embedded into the bank while the head projects into the stream. Several types of groins are illustrated in Figure 7, presented earlier.

A primary function of groins is to manipulate the stream current or flow direction. By diverting erosive flow away from sensitive areas along a streambank, groins provide bank protection. Other functions include training the stream along a desired course by changing the flow direction in the channel and inducing scour along defined lines to create a deeper channel, such as for navigation purposes. Scour holes induced at the head of a groin can provide a habitat for fish rearing.

There are two major types of groins, permeable and impermeable. Permeable groins slow down the local current and, in doing so, induce sediment deposition. They are often made from timber piles and are most effective in alluvial channels having appreciable bed load and coarse suspended load. Impermeable groins deflect the current without necessarily slowing it down. Groins made from rock boulders or gabions tend to be semi-permeable, primarily deflecting the current rather than retarding it.

The main interest, in this part of the report, is in the use of gabions as groins. Gabion groins have the capacity for deformation without damage. Once silt has accumulated around and within the stonework, vegetation growth can consolidate the structure into a new permanent bank. These circumstances are beneficial for erosion control and also may be useful for habitat development or modification.

Groins may be placed pointing upstream, normal to the flow, or downstream. Each orientation has a different impact on the stream current, with a consequential effect on the scour and deposition patterns around the groin. Figure 27 illustrates some impacts of groin orientation on sediment deposition. Samide and Beckstead (1975) observed that a groin pointing upstream repels the approaching flow away from itself while one pointing downstream attracts the approaching flow towards itself and does not repel it towards the opposite bank. The groin at right angles to the flow only changes the direction of the flow without repelling it. In each case, however, the flow leaving the groin has been observed to follow a trajectory initially directed toward the opposite bank. A more detailed discussion has already been presented on the inteaction of the flow with bank structures such as spur dikes. The discussion is equally applicable to gabion groins.

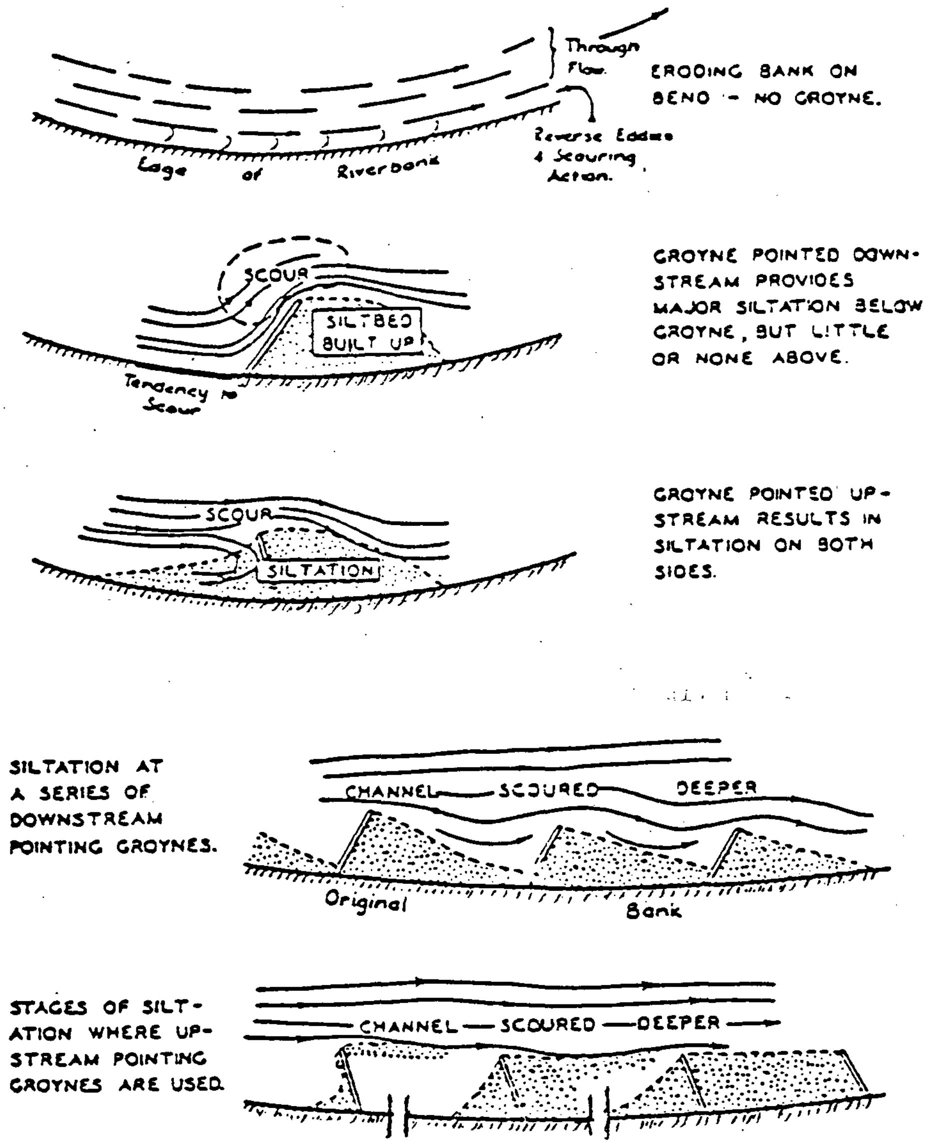


Figure 27. Effect of Groin Orientation on Sediment Deposition (Source: Mamak, 1964)

Design Considerations for Gabion Groins

The major factors that must be taken into consideration for the design of groins include flood depths and discharges, amount of suspended load and bed load, channel slope and width, high and low water depths, and flow velocities. The type and size of bed material (i.e., clay, silt, sand, gravel, cobbles) must also be known. Other factors to be considered include the debris load of the stream during floods, possible damage due to ice, available construction materials, and available funds.

With the above factors in mind, decisions must be made on the following design parameters: (a) groin foundations; (b) height and width of the groin; (c) depth of groin root embedment into bank; (d) structural configuration; (e) number of groins and spacing between them; (f) length of groin projection into the stream; (g) orientation of groin to the flow; and (h) extent and depth of scour to be expected.

Groin Foundation

Gabion groins do not require excavated foundations (Maccaferri Gabions of America, undated-b). It is enough to level off the stream bed at a depth approaching that of the lowest point of the nearby bed. If much local scour is anticipated, some foundation excavation may be helpful to minimize the amount of differential settlement.

The gabion groin itself may be sited either directly on the stream bed or on a gabion mattress. Figure 28 shows a gabion groin placed on a gabion mattress foundation with an apron. Except where the stream bed consists of bedrock and boulders and as such is not erodible, a mattress apron is needed to protect large groin superstructures from being undermined by scour. Figure 29 shows the behavior of a gabion apron if it is undermined. The

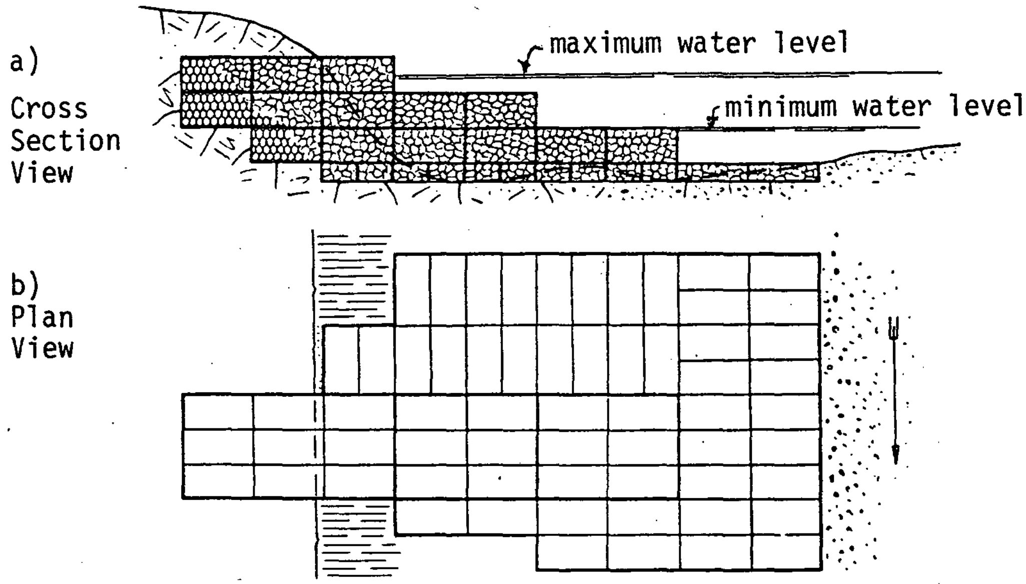


Figure 28. Gabion Groin with Gabion Mattress Foundation and Apron
(Source: Maccaferri Gaboins of America, undated-b)

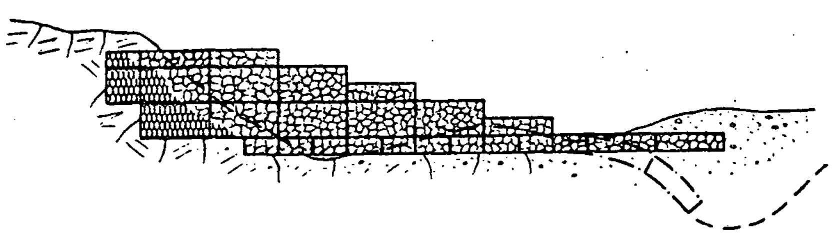


Figure 29. Behavior or Undermined Gabion Apron
(Source: Maccaferri Gabions of America,
undated-b)

mattress apron may be eliminated where the groin is small (i.e., 1 to 2 meters high and up to 5 meters long) (Maccaferri Gabions of America, undated-b). The mattress apron is flexible and consists of gabions laid flat on the streambed and wired together. The flexibility of the apron ensures that the apron settles following scour at the head of the groin. The mattress must be thin (e.g., one-half meter or less), but with sufficient weight to keep it on the bed, resistant to drag by the current and any tendency to lift or curl. The projection of the apron depends on the extent of scour expected. Experience has shown that this should be between 6 and 20 feet (Maccaferri Gabions of America, undated-b).

Groin Height

The height of the groin is generally designed in such a way as to prevent flood water from cutting behind the inshore root of the groin. Therefore, the height is generally set by a design criterion based on providing protection for a specific return frequency of discharge. The maximum height should be equal to the level of the flood plain.

Groin Width

The literature gives some general guidelines for determining groin width. It has been found in practice that a one-meter width is adequate for small streams and where the water velocity is small enough to cause no scouring action (Maccaferri Gabions of America, undated-b). The largest gabion groin structures need not be wider than 3 meters. As a general rule, the width should not be less than the height of the submerged part of the groin.

Groin Root Bank Embedment

Mamak (1964) recommends that the groin root penetrate 4 to 10 meters into the bank. This distance is too long for the small streams where gabions are often used. The root distance must be adequate to give good structural anchorage and to prevent end scour. Where anticipated flow conditions appear to threaten the groin near the bank, short revetments be used along the bank on each side of the root.

Type of Structure (Configuration in Plan View)

The simple straight type of gabion groin is suitable on gradual bends and straight reaches if the groins are short (Maccaferri Gabions of America, undated-b). If groins are long, the bayonet type pointing diagonally against the current is said to be preferable because it favors deposition (see Figure 7). Hammer-head groins have been found to be quite effective on narrow bends. Alternating the bayonet and straight types of groins, the latter being shorter and smaller in section, has been found to work equally as effectively as using the bayonet type throughout the channel reach to be protected and to be less expensive.

Generally, a properly designed system using the straight type of groins should provide adequate bank protection and induce sedimentation between the groins (Samide and Beckstead, 1975).

Number of Groins and Groin Spacing

The number of groins used to alter the flow will primarily depend upon the length of the project zone, the stream width, and the structure length. The number of groins is also dependent upon the spacing used.

It is important that the groins are not spaced too far apart. Otherwise, the stream current may return to the bank being protected before the next groin in the system begins to influence the flow direction. Where the groins are spaced too closely, they work less efficiently and cost more than a system of groins that is properly designed.

Table 3, presented earlier, shows a summary of literature on recommended groin spacing. The tabulated ratios represent the distance between two consecutive groins divided by the effective groin length normal to the bank.

For gabion groins, Maccaferri Gabions of America (undated-b) recommends a groin-spacing-to-groin-length ratio ranging from 4 to 6, depending on the curvature of the stream. The minimum ratio is used for concave banks and the maximum ratio is used for convex banks.

Distance of Groin Extension Into Stream

The projection of groins into a stream should be such that the heads of the groins are alined to define a smooth curve or a straight line representing a new channel bank, as was illustrated in Figure 8 (Samide and Beckstead, 1975). The length must enable the groins to shift the eroding current away from the bank. However, the groins must not create any instability by over-constricting the flow. Therefore, the groins must be positioned so as to provide adequate channel cross-sectional area for flow.

Groin Orientation to Flow

It has already been indicated that groins may be oriented upstream, downstream or normal to the flow. In choosing a particular orientation, the primary interest, as far as bank protection is concerned, is to shift the scouring flow away from the bank and encourage deposition between the groins. Researchers vary in their recommendations for groin orientation. This has already been shown in Table 2.

Samide and Beckstead (1975) observed that groins facing upstream caused more deposition adjacent to the downstream bank than groins inclined at 90 degrees to the flow. The groins placed normal to the flow protected a smaller area, while the groins facing upstream sustained the bulk of the erosive power of the flow and were able to protect bank areas upstream and downstream of the groins. Groins facing downstream attracted flow towards themselves and to the root of the next downstream groin. This threatened the downstream groin and the surrounding area. For this reason, Samide and Beckstead do not recommend downstream-oriented groins for bank protection purposes.

In contrast, Franco (1967) rated the groin facing downstream as best in performance on the basis of scour, deposition, channel depth and alignment. The groin facing upstream produced more disturbance to the flow.

As further contrast, Copeland (1983) indicated that the effective length of the groin is a more significant factor than the angle of orientation. Therefore, he recommended groins perpendicular to the flow.

Extent and Depth of Expected Scour

The scour depth at a gabion groin can be predicted from various formulas, such as those presented in Table 1. The flexibility of gabions allows them to maintain structural integrity if actual scour is somewhat more severe than predicted scour. Riprapped rockfill structures do not have this margin of safety. It is probably because of the flexibility of gabion structures that no major foundation excavation is recommended by the manufacturer (see earlier discussion). However, if bank anchorage is inadequate, the deformed structure may pull away from the bank into the scour hole.

Model Studies

Scope of Studies

The laboratory investigations undertaken with gabion groins involved single and paired gabion groins at various orientations to the flow, at various groin spacings, and for differing lengths. The objective in this part of the work was to observe and compare the performance of the groins, including the resulting flow patterns and scour patterns. The groin arrangement that best served the co-purposes of bank protection and habitat modification was also to be determined.

Laboratory Apparatus

The laboratory studies were conducted in a flume with a test section 16 feet long and 3.5 feet wide. A sand bed 6 inches deep and initially flat for each test was used to study scour and deposition. The median diameter of the sand was 1.5 mm. Bed elevations and scour depths were measured with a point gage.

The flume hydraulic system consisted of a storage sump, supply pump, head tank, stilling basin, flume, tailgate, and volumetric weighing tank. The water discharge was controlled by varying the pump discharge valve and/or the pump speed. The discharge was selected such that the streambed was stable at slightly below the critical conditions for incipient motion.

Gabion baskets were modelled with copper window screen having a mesh opening of 0.04 in x 0.04 in (1 mm x 1 mm) and filled with gravel with a mean size of 0.5 inches (1.7 cm). Straight-type gabion groins were modelled in two different lengths: 21.0 in (53.3 cm) and 10.5 in (26.7 cm). These lengths corresponded to one-half (21.0 in) and one-quarter (10.5 in) of the channel width. The dimension of the groin cross-section was 3.9 in x 3.9 in (10.0 cm x 10.0 cm). This was chosen to represent a realistic size in relation to the channel width. This dimension of the model groin represents a scaling ratio of 1:10 when compared with a commercial gabion basket with a cross-section of 1.0 m x 1.0 m.

Laboratory Procedures

The gabion groin experiments were conducted by means of thirty test runs. Table 10 summarizes the test conditions for each test run. The terms and symbols used in this table are explained by the definition sketch shown in Figure 30.

For each test run, the sequential procedures were as follows:

1. The channel bed was leveled and the initial bed elevation was measured.
2. One or two gabion groins with the predetermined length, spacing, and orientation angle were placed in the flume.
3. The root of each groin was nailed to the channel wall to represent prototype bank anchorage conditions.
4. The elevation of each groin was measured.
5. The pump was turned on with the discharge, Q, set at 0.51 cfs.
6. Flow patterns around each groin were traced by means of small drops of red dye poured into the upstream end of the channel. The observed patterns were sketched.
7. The average upstream water depth, y, was measured after the flow had reached steady-state conditions. The average channel velocity V, was calculated from the measured water depth and discharge and the channel width.
8. Progressive channel changes due to scour and deposition, and the corresponding gabion behavior, were noted.
9. The flow was maintained for 20 hours to allow a definite scour pattern to form.
10. The pump was stopped at the end of the 20 hours and the water was allowed to drain.

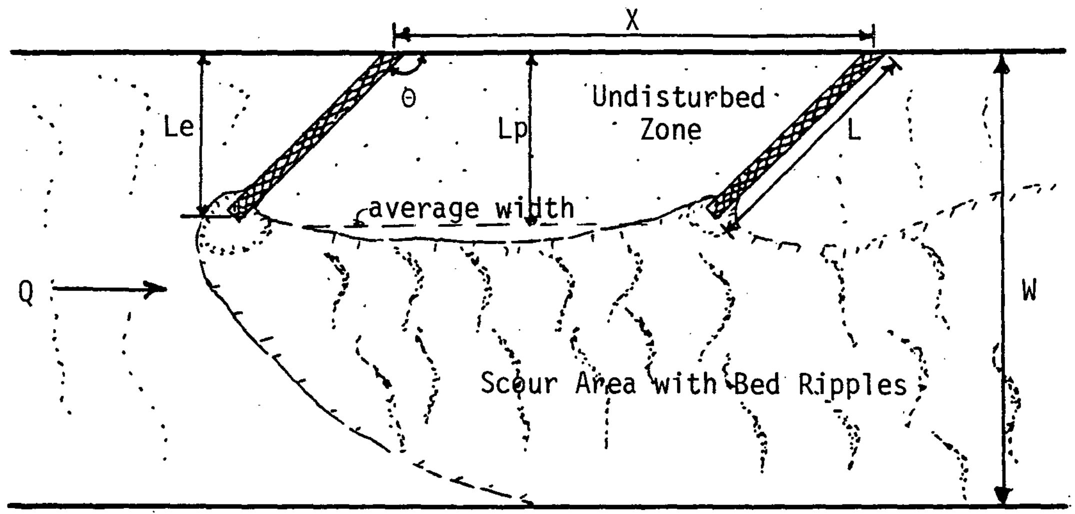
Table 10. Summary of Gabion Groin Laboratory Test Conditions

A. Tests With Single Gabion Groins

Run	L/W	θ
1	1/2	135
2	1/2	90
3	1/2	45

B. Test With Double Gabion Groins

Run	L/W	θ	X	Run	L/W	θ	X
4	1/2	135	L	18	1/4	135	3L
5	1/2	135	2L	19	1/4	135	4L
6	1/2	135	3L	20	1/4	135	5L
7	1/2	135	4L	21	1/4	90	L
8	1/2	90	L	22	1/4	90	2L
9	1/2	90	2L	23	1/4	90	3L
10	1/2	90	3L	24	1/4	90	4L
11	1/2	90	4L	25	1/4	90	5L
12	1/2	45	L	26	1/4	45	L
13	1/2	45	2L	27	1/4	45	2L
14	1/2	45	3L	28	1/4	45	3L
15	1/2	45	4L	29	1/4	45	4L
16	1/4	135	L	30	1/4	45	5L
17	1/4	135	2L				



L = Actual Groin Length

Le = Effective Groin Length

Lp = Average Width of Undisturbed (Protected) Zone Between Groins

X = Spacing Between Groins

Θ = Groin Orientation Angle with Downstream Bank

W = Channel Width

Q = Discharge

Figure 30. Definition Sketch for Terms Used in Gabion Groin Experiments

11. The new groin elevation was measured to determine the amount of groin settlement caused by scour.

12. The maximum scour depth near each groin was measured and its position with respect to the groin was noted.

13. The scour pattern around each gabion was photographed.

Laboratory Results and Observations

The general flow patterns associated with single and double groins are shown in Figures 31 and 32, respectively. In each case, the groins are oriented upstream, normal to flow, and downstream .

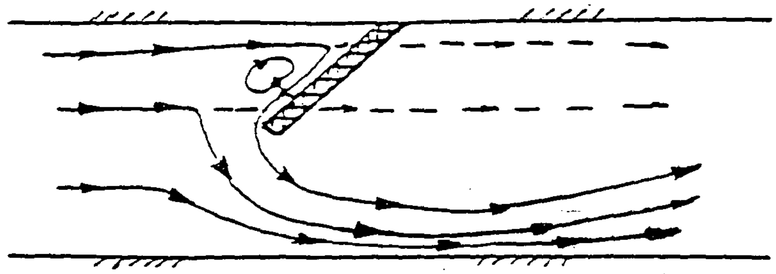
The leading upstream-oriented groin repelled the flow from itself with a still-water pocket (or reverse eddy) forming upstream of the groin. The normally-oriented groin simply changed the direction of the flow away from the bank being protected. The groin pointing downstream directed the flow downstream without repelling it. All the groin orientations resulted in flow being deflected away from part of that bank being protected by groins.

Figures 33 and 34 show the scour patterns for these single and double groins after 20 hours of flow. The test conditions involved a discharge of 0.51 cfs, an upstream approach velocity of 0.48 fps, a boundary shear stress of 0.03 psf and a Froude number of 0.15.

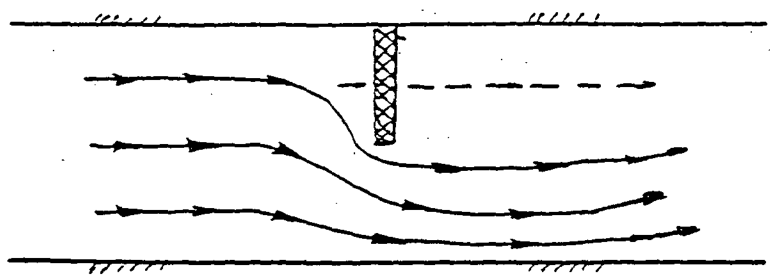
Bed scour caused by groins pointing upstream and downstream tended to extend from the tip of the groin to the opposite bank. For groins pointing normal to the flow, scour at the tip of the groin was more localized and extended more downstream than toward the opposite bank.

The scouring eddies were most pronounced at the upstream sides of the groins. This caused the gabions to twist in most cases, rotating upstream and downward. The upstream groins showed more twisting than the downstream groins. Also, the longer groins (L/W = 1/2) showed more twisting than the

a) Upstream Orientation



b) Oriented Normal to Flow



c) Downstream Orientation

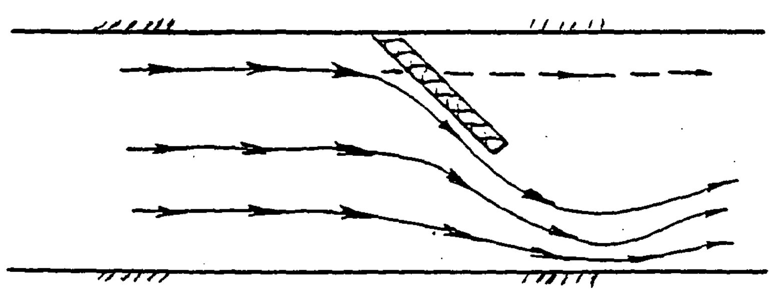


Figure 31. Flow Patterns for a Single Gabion Groin at Three Orientations

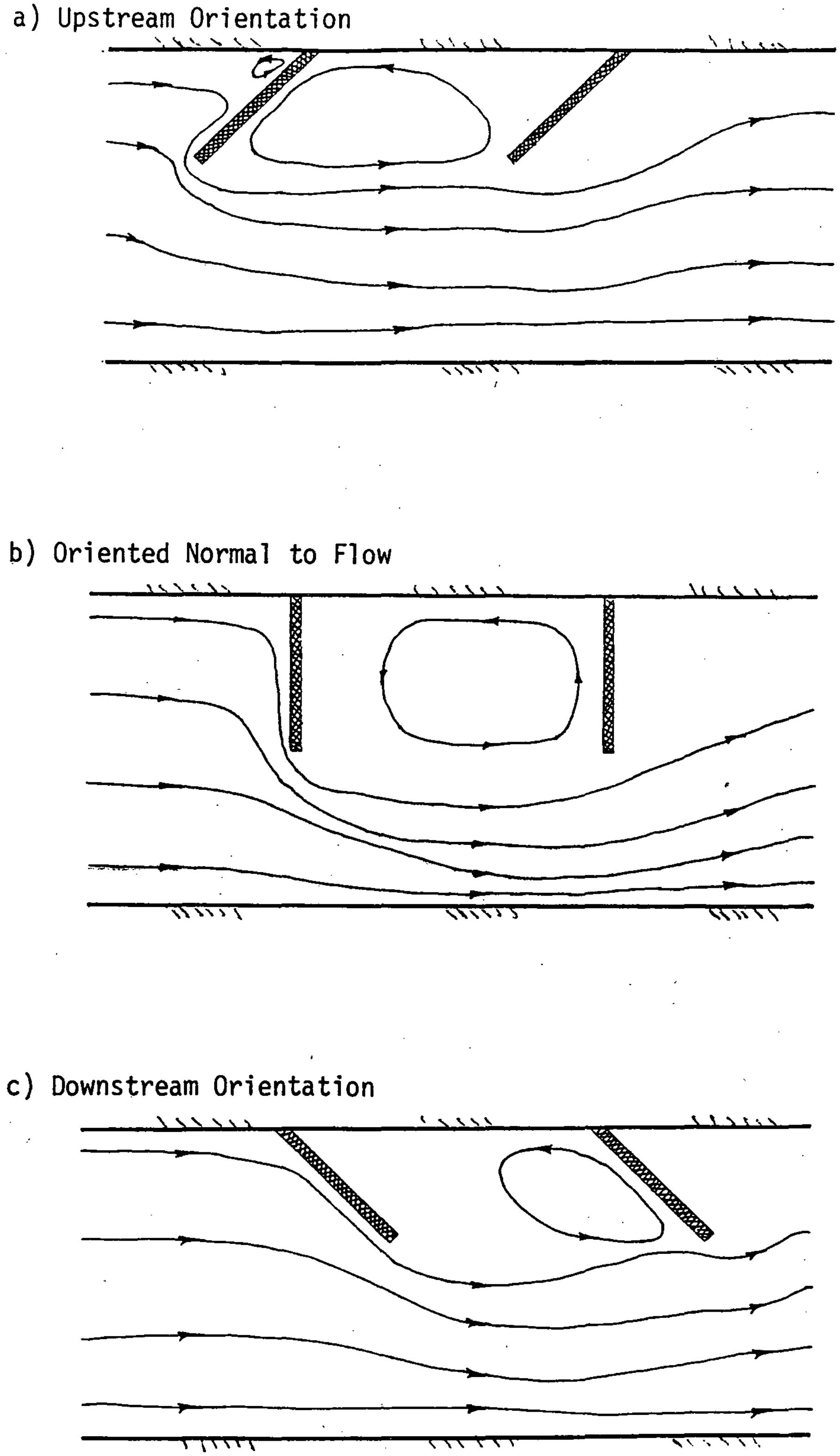
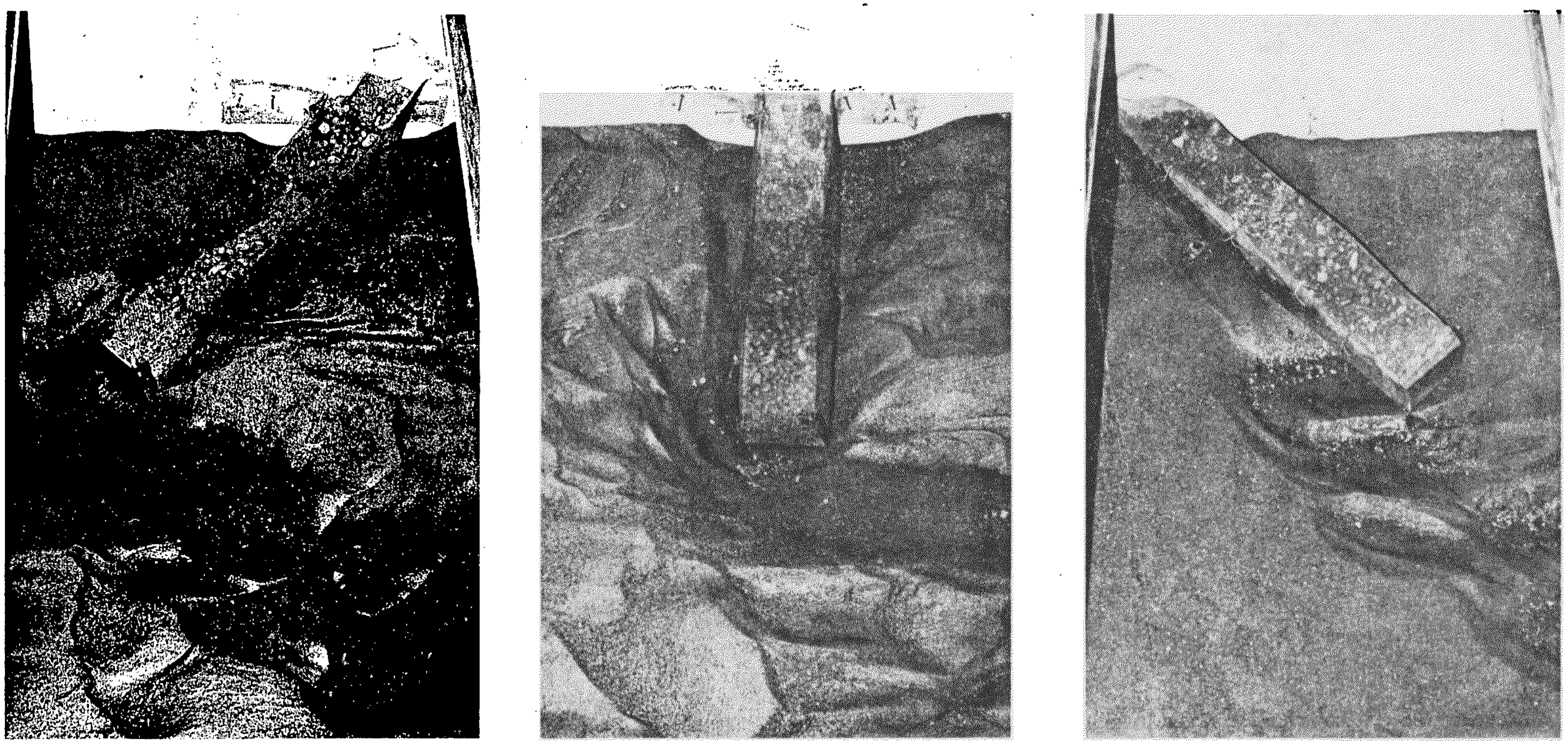


Figure 32. Flow Patterns for Double Gabion Groins at Three Orientations

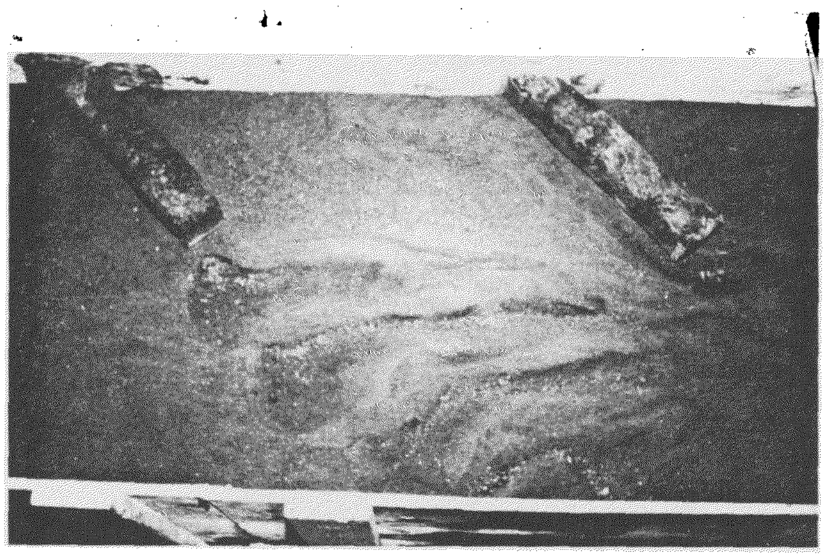


a) Upstream Orientation b) Oriented Normal to Flow c) Downstream Orientation

Figure 33. Scour Patterns Around Single Gabion Groins at Three Orientations

a) Upstream Orientation

b) Oriented Normal to Flow



c) Downstream Orientation

Figure 34. Scour Patterns Around Double Gabion Groins at Three Orientations

shorter groins (L/W = 1/4). During the runs, the flexible gabion groins settled into the developing scour zones. Because the groin root was anchored to the channel bank, the groin sloped; its tip and about one-third of its length were submerged in the flowing water. More scour occurred beneath and around the upstream groins than near the downstream groins, as the upstream groins sustained the bulk of the erosive power of the flow. Maximum scour depth generally occurred at the outer tip of the groin, where local acceleration of the flow was most pronounced.

Groins oriented upstream caused more bed scour than groins oriented downstream. Compared to the other two orientations, the downstream-oriented groins caused the least bed scour.

Tables 11 and 12 show the measured groin settlements and the maximum scour depths for double groins having L/W ratios of 1/2 and 1/4, respectively. The data are plotted in Figures 35 and 36. Except for a few inconsistencies, possibly due to experimental errors, the tabulated data confirm the above general observations regarding the effect of orientation angle and local scour. Figures 35 and 36 show that, for a given orientation angle, the upstream groin experienced essentially the same amount of scour and settlement, regardless of groin spacing. (The variation might be a measure of experimental error.) The downstream groin experienced less scour and settlement than the upstream groin, but the amount experienced depended upon the groin spacing. When the spacing exceeded twice the structure length, the amount of scour and settlement increased. The amount of scour at the downstream groin approached that at the upstream groin for X/L spacings of three or more if the flow constriction was severe (i.e., L/W = 1/2). The amount of settlement was much less when the flow constriction was small (at L/W = 1/4) than at larger flow constrictions (at L/W = 1/2).

Table 11. Gabion Groin Settlement and Maximum Scour Depth at Groin Head for Double Groins with L/W Ratio of 1/2

Groin Spacing	θ = 45° Downstream Orientation				θ = 90° Normal to Flow				θ = 135° Upstream Orientation			
X/L	Z_1^*,ft	Z_2^{**},ft	d_1^{***},ft	d_2^{****},ft	Z_1,ft	Z_2,ft	d_1,ft	d_2,ft	Z_1,ft	Z_2,ft	d_1,ft	d_2,ft
1	0.019	0.019	0.136	0.042	0.240	0.035	0.239	0.079	0.241	0	0.309	0.223
2	0.008	0	0.159	0.015	0.235	0.174	0.289	0.080	0.247	0.017	0.302	0.228
3	0.009	0.003	0.266	0.153	0.234	0.230	0.280	0.249	0.142	0.165	0.288	0.259
4	0.021	-	0.212	0.065	0.174	0.231	0.275	0.224	0.190	0	0.284	0.268

$^*Z_1$ = settlement at tip of upstream groin
$^{**}Z_2$ = settlement at tip of downstream groin
$^{***}d_1$ = maximum scour depth at tip of upstream groin
$^{****}d_2$ = maxmium scour depth at tip of downstream groin

Table 12. Gabion Groin Settlement and Maximum Scour Depth at Groin Head for Double Groins with L/W Ratio of 1/4

Groin Spacing	θ = 45° Downstream Orientation				θ = 90° Normal to Flow				θ = 135° Upstream Orientation			
X/L	Z_1^*,ft	Z_2^{**},ft	d_1^{***},ft	d_2^{****},ft	Z_1,ft	Z_2,ft	d_1,ft	d_2,ft	Z_1,ft	Z_2,ft	d_1,ft	d_2,ft
1	0.183	0.001	0.184	0.003	0.156	0	0.239	0.026	0.175	0.003	0.245	0.082
2	0.028	0.002	0.166	0.028	0.177	0	0.224	0.041	0.192	0.002	0.233	0.125
3	0.001	0	0.124	0.070	0.174	0	0.235	0.074	0.197	0.003	0.245	0.120
4	0.008	0.005	0.145	0.103	0.175	0	0.214	0.118	0.220	-	0.261	0.259
5	0.007	0.047	0.167	0.018	0.127	0.034	0.222	0.073	0.155	0.006	0.231	0.128

$^*Z_1$ = settlement at tip of upstream groin
$^{**}Z_2$ = settlement at tip of downstream groin
$^{***}d_1$ = maximum scour depth at tip of upstream groin
$^{****}d_2$ = maximum scour depth at tip of downstream groin

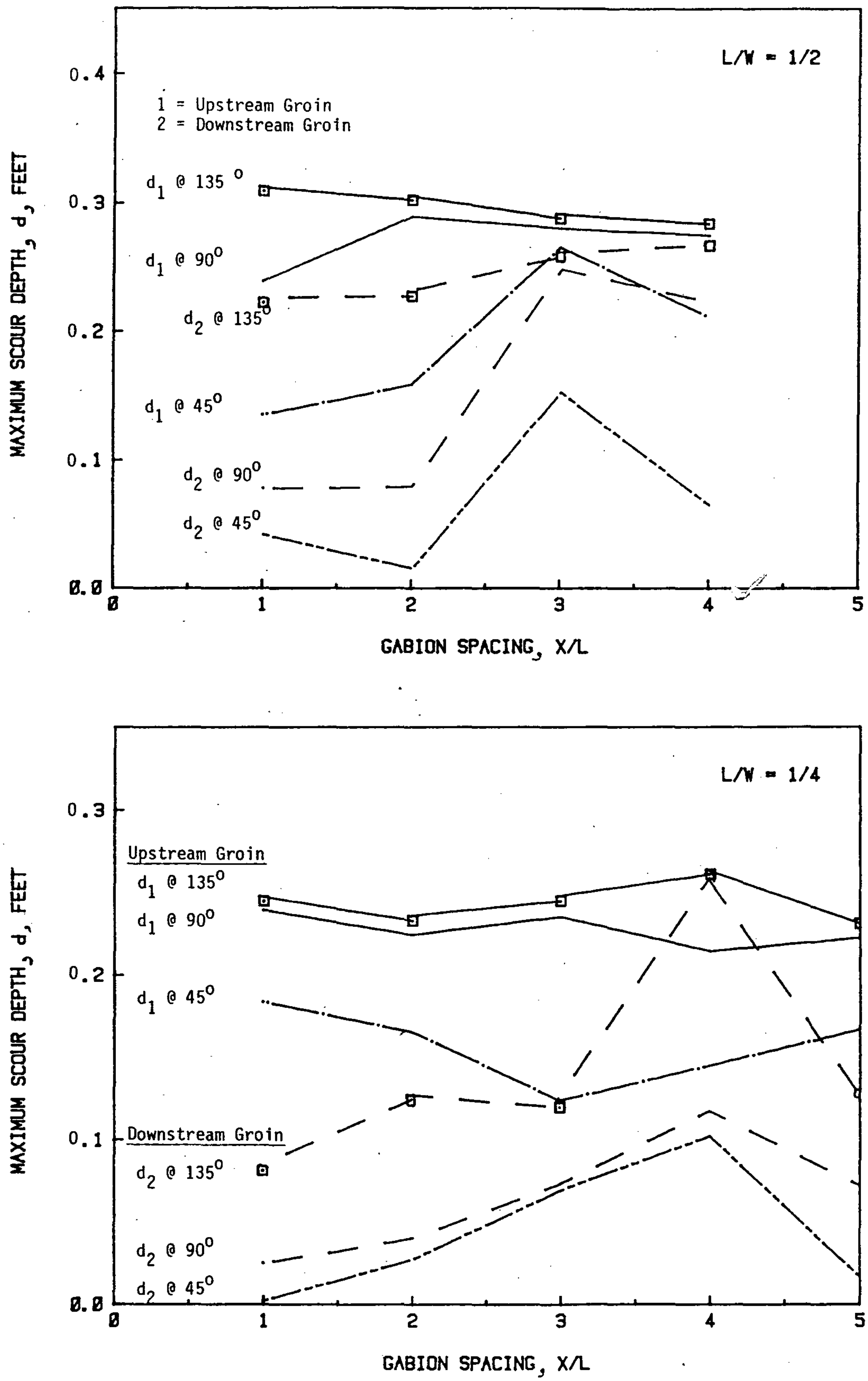


Figure 35. Maximum Scour Depths for Double Gabion Groins at Various Spacings, Orientations and Lengths

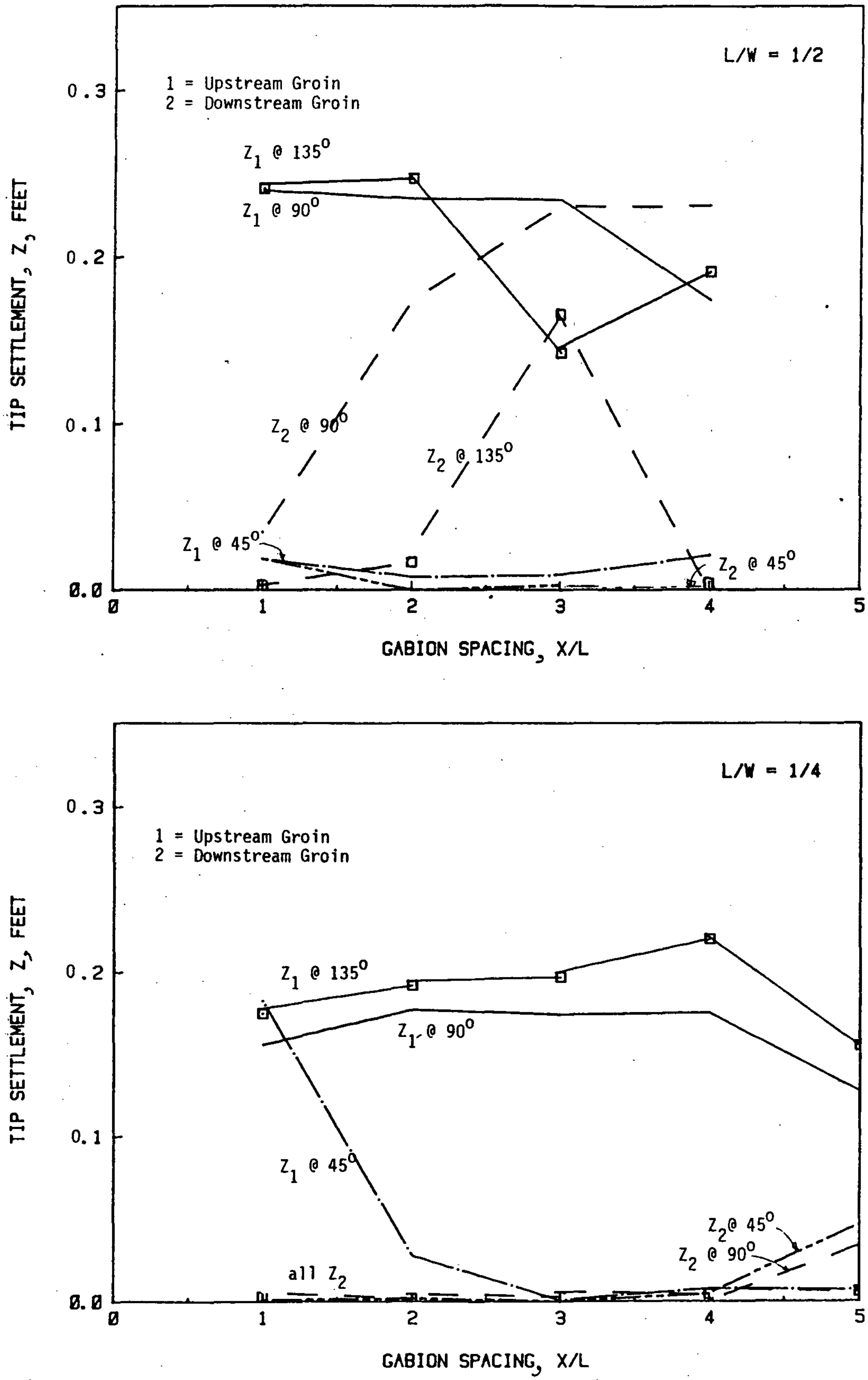


Figure 36. Tip Settlements for Double Gabion Groins at Various Spacings, Orientations and Lengths

The zone between the double groins experienced little or no current and was characterized by an undisturbed and generally smooth bed. The average width of this protected zone, Lp, varied as the groin spacing, X, was changed. The width, Lp, was measured and used as an index for determining bank protection; the larger Lp was, the more protection the bank received. To standardize this index of bank protection, Lp was divided by the effective groin length, Le, which is the projected length of the groin measured from the groin tip perpendicular to the bank along which the groin is placed. Tables 13 and 14 show the variation of the effective bank protection per unit effective groin length, Lp/Le, with the relative groin spacing, X/L. Figure 37 shows the plot of Lp/Le versus X/L for various orientation angles and channel contractions.

Field Study

A limited field study was conducted to observe the performance of a gabion groin. The ability of such a structure to cause scour and deposition in a gravel-cobble stream was of particular interest, to allow comparison with the more easily eroded sand bed in the laboratory.

Field Site and Procedures

The field work involved a 2 m x 1 m x 0.5 m prototype gabion groin installed in Oak Creek along a bank experiencing higher currents and some erosion. Oak Creek drains the western slope of the Oregon Coast Range near Corvallis. Its bed material near the study site is predominantly gravel and cobbles. The average size of armor layer material is about 60 mm; that of the sub-armor material is about 20 mm. The test site chosen was in a straight reach with an average channel width of 14.0 ft (4.3 m) and a

Table 13. Effect of Gabion Groin Spacing and Orientation Angle on Bank Protection for Double Groins with L/W Ratio of 1/2

Groin Spacing	$\theta = 45^0$ Downstream Orientation			$\theta = 90^0$ Normal to Flow			$\theta = 135^0$ Upstream Orientation		
XL	L_p (in)	Le* (in)	L_p/Le	L_p (in)	Le* (in)	L_p/Le	Lp (in)	Le* (in)	L_p/Le
1	15.2	14.85	1.02	22.2	21.0	1.06	16.5	14.85	1.11
2	11.0	14.85	0.74	18.0	21.0	0.86	14.3	14.85	0.96
3	10.0	14.85	0.67	16.1	21.0	0.77	11.95	14.85	0.80
4	7.30	14.85	0.49	13.0	21.0	0.62	9.76	14.85	0.66

*Le = L Sinθ, where L = 21 inches

Table 14. Effect of Gabion Groin Spacing and Orientation Angle on Bank Protection for Double Groins with L/W Ratio of 1/4

Groin Spacing	$\theta = 45^0$ Downstream Orientation			$\theta = 90^0$ Normal to Flow			$\theta = 135^0$ Upstream Orientation		
XL	Lp (in)	Le* (in)	L_p/Le	Lp (in)	Le* (in)	L_p/Le	Lp (in)	Le* (in)	L_p/Le
1	2.25	7.42	0.30	4.25	10.5	0.40	5.65	7.42	0.76
2	5.83	7.42	0.79	8.75	10.5	0.83	7.00	7.42	0.94
3	5.98	7.42	0.81	9.55	10.5	0.91	8.48	7.42	1.14
4	6.53	7.42	0.85	10.9	10.5	1.04	9.33	7.42	1.26
5	5.34	7.42	0.77	10.3	10.5	0.98	9.67	7.42	1.30

*Le = L Sinθ, where L = 10.5 inches

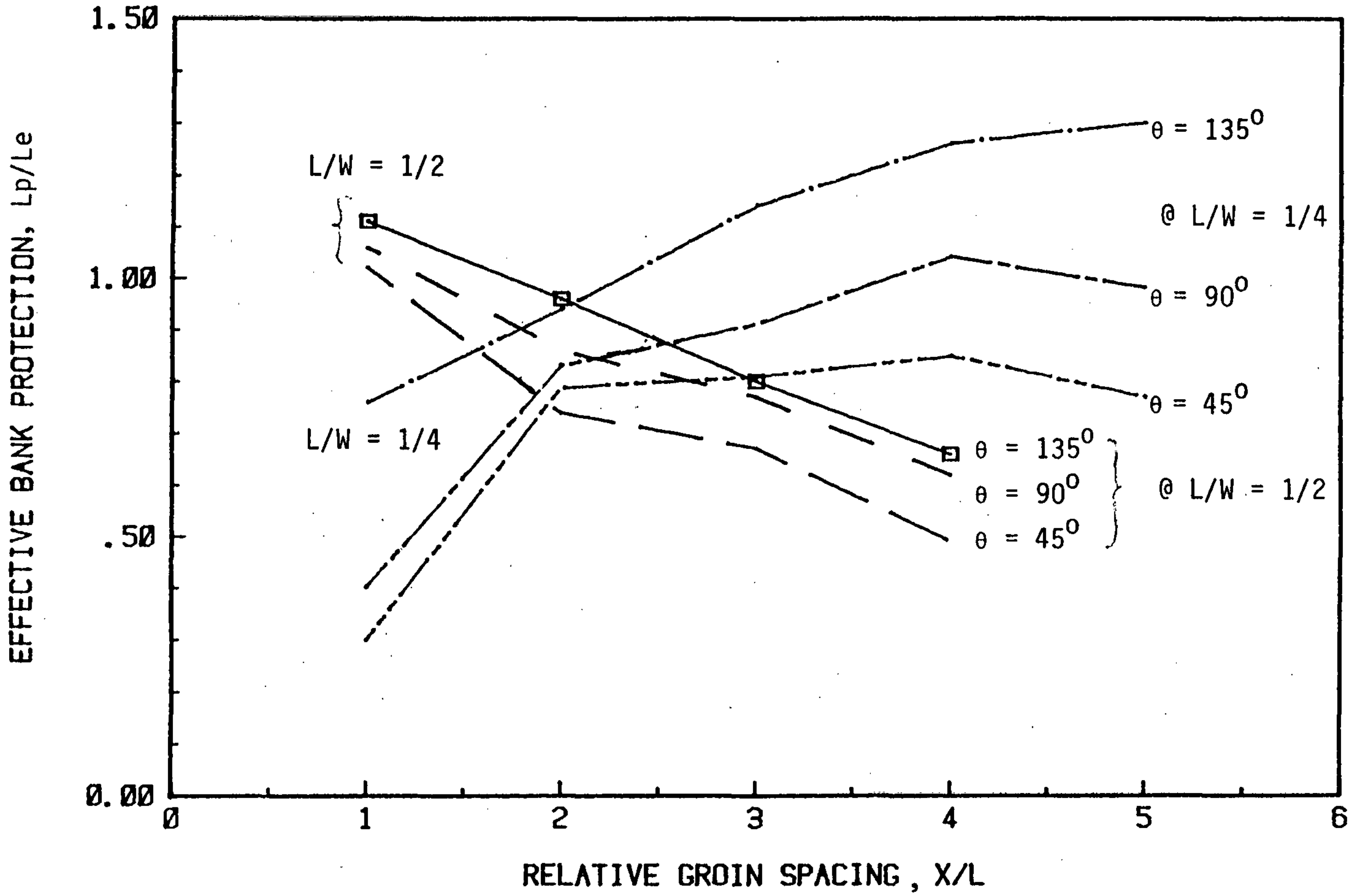


Figure 37. Gabion Groin Spacing and Effective Bank Protection for Various Orientation Angles and L/W

gradient of about 1 percent. The gabion extended out from the right bank to mid channel. A staff gage was installed and a nearby stream gaging station provided a continuous record of the stream hydrograph during the 4.5-month test period.

Cross sections were established and marked at 5-foot intervals for 10 feet upstream and 40 feet downstream of the gabion. The bed slope and cross-sectional shapes were determined on several occasions from the date of installation until winter storms ceased four and one-half months later. The position and settlement of the groin, caused by scour, were checked periodically.

Field Results and Observations

The flow pattern around the gabion was essentially identical to the flow pattern around the model gabion installed normal to the flow in the laboratory. The performance of the prototype gabion and the resulting bed scour and deposition were also comparable to those for the model gabion.

Figure 38 shows the stream cross sections immediately after gabion installation and four and one-half months later. Four major storms occurred during this period, with peaks ranging from 170 cfs to 220 cfs. The smallest discharge during the period was 3 cfs.

Local scour occurred around the tip of the groin. A maximum scour depth of 3.0 feet occurred at the gabion tip. This caused to the gabion to settle. About two-thirds of the gabion length was submerged during the larger discharges, yet the gabion still performed well. The prototype groin did not twist, as was the case for the laboratory model. Reinforcing steel bars installed through the gabion into the stream bed as anchors prevented the twisting from taking place.

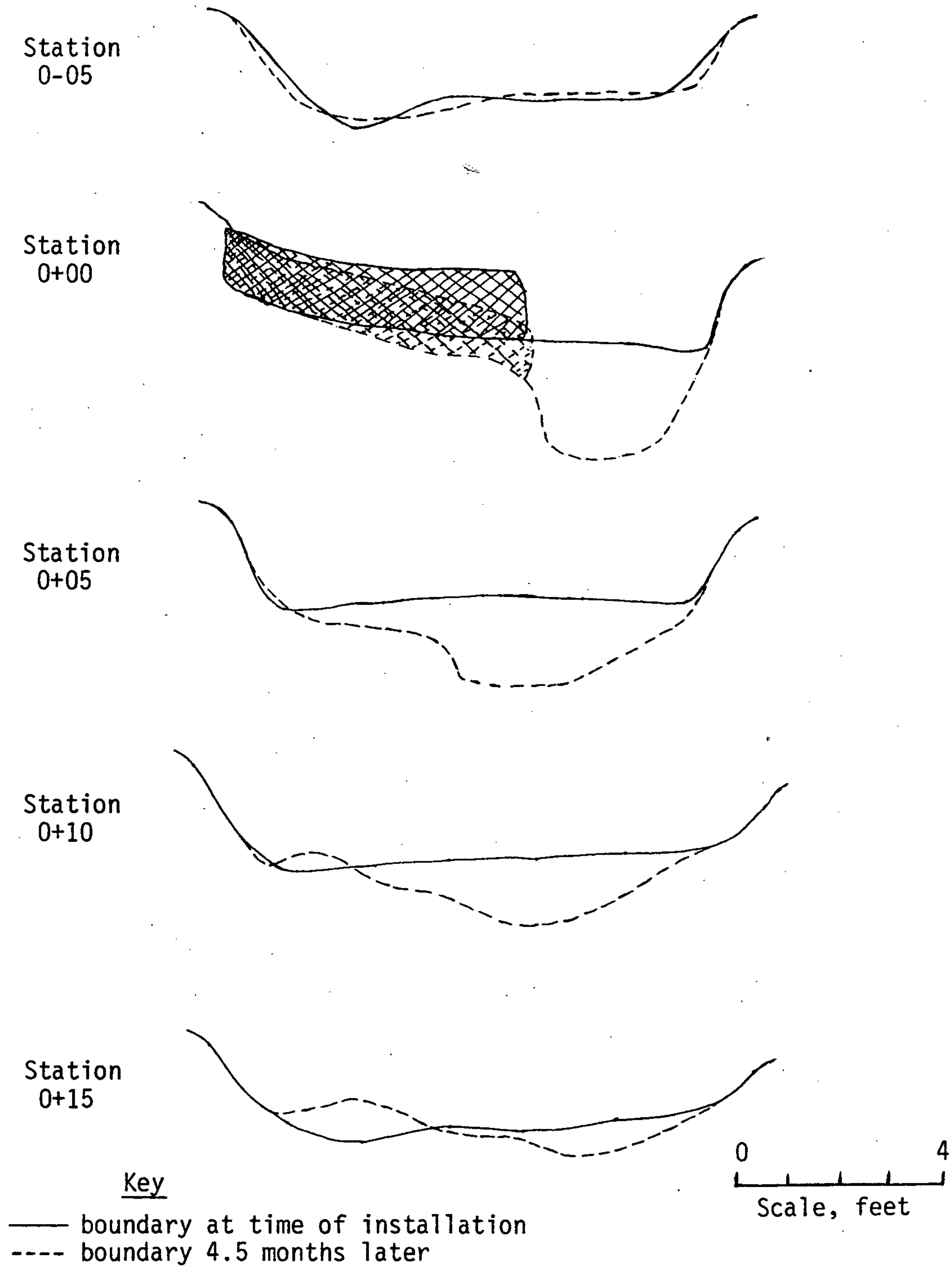


Figure 38. Oak Creek Cross Sections Immediately After Gabion Groin Installation and Four and One-Half Months Later

Sediment deposition downstream of the gabion resulted in a bar 29 feet long and 3 feet wide along the bank being protected. This bar caused the stronger currents to shift from the bank being protected and to scour the bed near the opposite bank.

Discussion of Studies

The flow pattern that developed for each groin orientation was distinctive and showed a definite relationship with the corresponding scour pattern. The influence of the groins on the flow velocities thus significantly affected sediment transport and general and local scour.

The nature of the flow and scour patterns around the groins indicates that the obstruction to flow caused by groins created an intense system of vortices. The primary vortex impinged on the stream bed at the groin tip, eroded the bed material there, entrained the eroded material in the flow, and allowed it to be transported downstream by the main flow. Intermittent vortices of lesser strength occurred along the upstream and downstream faces of the groin and added to the scouring action. Because of the location of the primary vortex at the groin tip, the maximum scour occurred there.

The observation that groins oriented upstream caused more scour than those oriented downstream is in agreement with work done by Samide and Beckstead (1978) and Tison (1962). The general trends observed in this experiment were shown quantitatively by Ahmad (1953) and Garde, et al. (1961).

It is seen from Tables 13 and 14 and from Figure 37 that for groin length to channel ratio of $L/W = 1/2$, the effective bank protection, Lp/Le, decreased as the groin spacing, X, was increased. A different trend is shown for L/W = 1/4. In this case, Lp/Le increased with X up to X = 4L. The shorter groins (L/W = 1/4) showed more interaction between the eddies around the upstream and downstream groins at small spacings. The interaction of the eddies resulted in a narrower width, Lp, for the undisturbed zone between the groins. Scour developing around the upstream groin easily extended to join scour developing around the downstream groin when the groin spacing was small. As X increased, the interaction of the eddies around the upstream and downstream groins diminished, leading to higher Lp values. For L/W = 1/2, the upsteam groin was able to deflect the flow beyond the downstream groin and thus minimized or prevented the kind of eddy interaction experienced by the shorter groins. Beyond X/L = 4, the groins with L/W = 1/4 began to show the same trend as groins with L/W = 1/2; the effective bank protection, Lp/Le, began to decrease with increasing X. It can be inferred from the above discussion that shorter groins should not be spaced too close together, to prevent eddies around the upsteam and downstream groins from interacting.

The higher Lp/Le ratios were shown for groins oriented upstream, followed by groins pointing normal to the flow. Thus, groins pointing upstream gave the most bank protection, followed by groins pointing normal to the flow. Groins pointing downstream gave the least bank protection, based on their Lp/Le ratios. However, the amount of protection offered by the downstream-oriented groins was adequate, for all the groin spacings tested.

The observation that downstream-oriented groins provided adequate bank protection (for groin spacings up to 4L at L/W = 1/2 and 5L at L/W = 1/4) is supported by much of the reviewed literature and is in contrast to other findings. For example, Samide and Beckstead (1975) observed that for downstream-oriented groins, the current flows toward the root of the next downstream groin. However, it is the finding of this project that this problem can be eliminated by proper spacing of the groins; if the current is flowing to the root of the next downsteam groin, it is generally because the groin spacing is too large.

Figure 37 also shows that for a relative groin spacing of about 2, the groins with length-to-channel-width ratio of 1/2 and 1/4 provided approximately the same effective bank protection per unit effective groin length. Beyond X/L = 3, groins with L/W = 1/4 offered better bank protection per unit effective groin length than did groins with L/W = 1/2.

Summary and Conclusions

Based on the results and discussion presented for the gabion groin experiments, the following conclusions and recommendations can be made with regard to gabion groins:

1. Upstream gabion groins sustain the bulk of the erosive power of the stream flow, compared to downsteam groins. This resulted in deeper local scour and greater settlement of the gabion tip into the scour hole. Therefore, careful design attention must be given to upstream groins in a groin field to assure their stability.
2. Greater scour occurs for upstream-oriented and normally-oriented groins than for downstream-oriented groins. Therefore, special design attention should be given to gabion groin stability for upstream-oriented and perpendicular structures.

3. In using gabion groins oriented upstream or downstream in small streams, ratios of groin length to stream width, L/W, greater than or equal to 1/2 should not be used because of the threat of eroding the opposite bank. Even the 1/2 ratio may pose a serious threat for weak banks.
4. If fish habitat modification is of interest in addition to bank protection, gabion groins oriented upstream or normal to the flow may be preferred over groins oriented downstream because of greater opportunities for bigger scour holes to be created.
5. Groins oriented upstream give the greatest bank protection, followed by groins oriented normal to the flow. Groins oriented downstream offer adequate bank protection for groin spacings up to 4L and 5L, at the tested ratios of L/W = 1/2 and L/W = 1/4, respectively. However, downstream-oriented groins give the least protection, compared to upstream and normally-oriented groins.
6. At a relative groin spacing of about 2, groins with length-to-channel-width ratios of 1/2 and 1/4 offer about the same effective bank protection per unit effective groin length.

VI. USE OF GABION WEIRS

Overview

This part of the report describes the use of gabions for weirs. One emerging use for such structures has been to modify fish habitat by altering water depths and velocities and by inducing local bed scour and sediment deposition.

The objective of work discussed here is to determine the effect of V-shaped gabions on the stream flow and bed scour patterns and the influence of weir apex angle on channel scour and deposition characteristics.

A desirable scour hole for fish habitat modification is considered to be one that is deep and large, provides enough room for fish rearing and maintains favorable temperatures during periods of low flow. Also, its location must not pose a threat to the structure and the streambanks.

General Features of Gabion Weirs

Weirs are built across channels for diverse purposes. These include use for soil erosion control, to reduce flood damage, to trap sediment and to prevent it from going downstream, as flow measuring devices, to recharge ground water from the stream, and as a means of raising the upstream water level. Raising the upstream water level may be important to form small reservoirs, for canal off-takes, for pumping station intakes, and to make a given channel reach suitable for navigation. Weirs flatten the local channel gradient, which can reduce channel scour and cause bed deposition.

This can help protect upstream structures such as bridges against scour and protect the base of eroded banks. Weirs have also been used to trap gravel for fish spawning and to create scour holes downstream for fish rearing purposes.

Gabion weirs can be used in all of the above situations. They are used particularly where loose or fine-grained soils having high permeability are found (Agostini, et al., 1981). They have two distinct advantages over other types of weirs: flexibility and permeability. Their flexibility allows gabion weirs to follow shifts of ground level beneath the structure with little damage. Thus, if material under the weir is scoured away, the weir simply settles. Raising the weir to its original height can be done by adding a new layer of gabions on top of the existing structure. The permeability of a gabion weir allows a portion of the flow to pass through the gabions, if the upstream face of the weir is not sealed. This reduces the volume of water falling over the crest. Therefore, somewhat less downstream toe protection is required against scour.

Gabion weirs are classified into three types, according to the shape of their downstream face at the center of flow (Agostini, et al., 1981). These types are shown in Figure 39 and include: vertical weirs, sloped weirs and stepped weirs. The vertical gabion weir produces a nappe which is separated from the downstream face of the weir. Only the crest mesh is exposed to abrasion and must be protected. A larger scour hole can develop than for the other types of weirs. The sloped gabion weir has been recommended for large weirs, when the height of the structure ranges up to 10 or 15 meters and the weir requires greater stability and improved hydraulic behavior.

Stepped gabion weirs offer better stability and the dissipation of some energy on each step, which may be of advantage if a scour hole is not

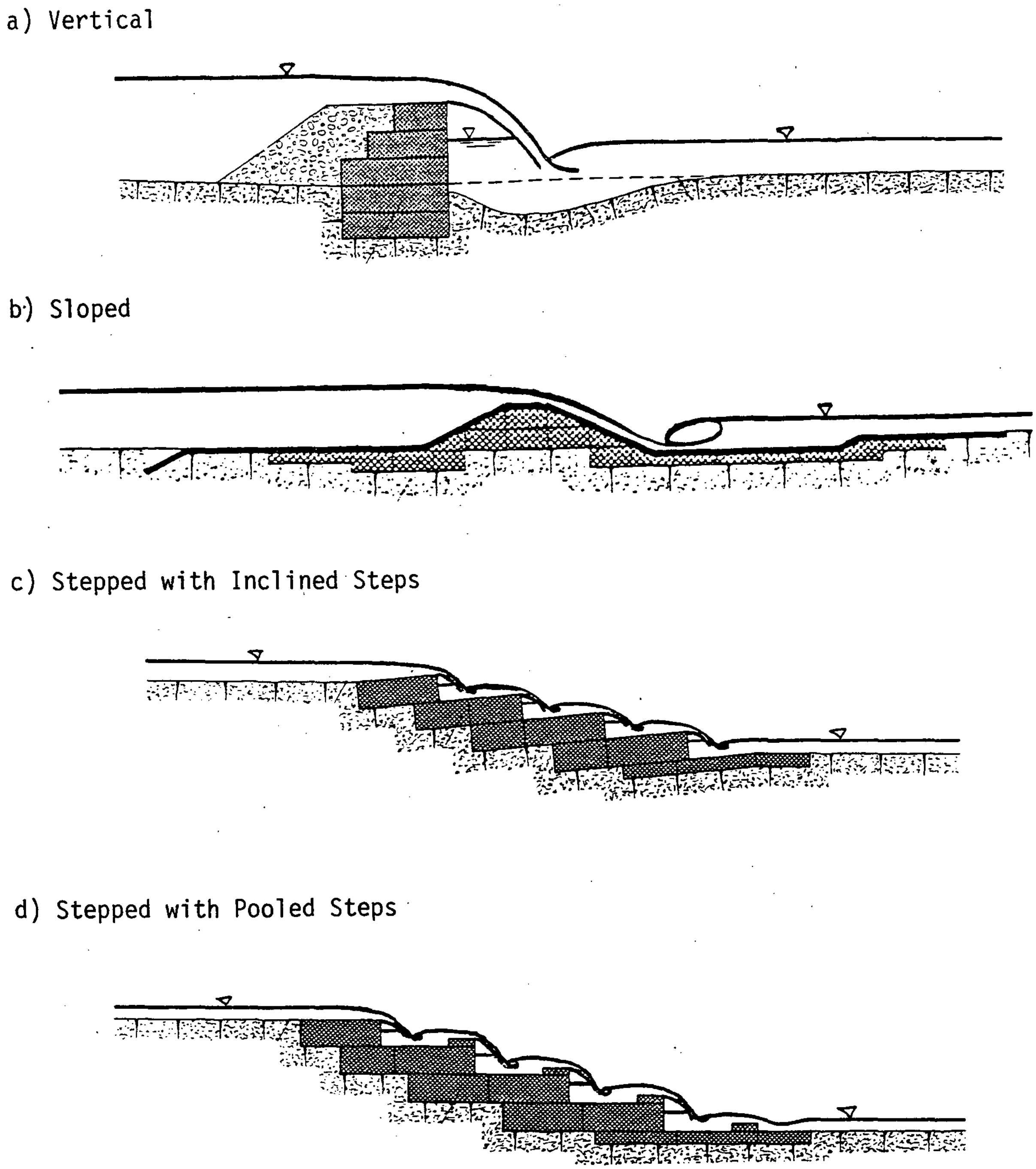


Figure 39. Types of Gabion Weirs
(Source: Agostini, et al., 1981)

sought. The stepped type is not recommended if a heavy bed load is carried, because of potential damage to the mesh on the steps.

Design Considerations for Gabion Weirs

The design considerations for weirs involve hydraulic and structural stability criteria. In this section, the design criteria are discussed in a general manner, based on a summary of the procedures given by Agostini, et al. (1981).

Hydraulic design must include: (1) design of the crest to maintain the maximum discharge at the center of the river; (2) design of the stilling pool for energy dissipation and scour control downstream of the weir structure; and (3) control of seepage under and around the weir to prevent fine soil material from washing away.

Structural design must include considerations of: (1) the stability of the weir against overturning and sliding; (2) the stability of the bed of the stilling pool against uplift; and (3) the bearing pressures on the weir structure and on the foundation soil.

Crest Design

The crest of the gabion weir may have the shape of a rectangle, a trapezoid, or an arc. It is usually designed to maintain the design discharge at the center of the river and to prevent overtopping of the wings and scouring of the adjacent banks. On smaller streams the weir crest may extend almost from bank to bank or be a long arc that is slightly higher at the anchor points on the banks than in mid-stream.

The gabion mesh on the crest must be protected from abrasion and the impact of heavy bed load material transported by the river during severe runoff conditions. This may be done by use of timber, angle iron or concrete. Each will cause a greater amount of structural rigidity. The concrete can be damaged if weir settlement takes place.

Stilling Pool Design

The stilling pool may be allowed to form naturally or it may be designed using a counter weir placed at a suitable distance downstream of the main weir to form a stilling basin. In one case, the river bed may be left unprotected upstream and downstream of the secondary weir, allowing a deep scour hole to form for energy dissipation. A second way is to have the bed of the stilling pool protected against scour by use of gabions and to control the hydraulic jump and form a pool using a broad-crested counter weir. The third way is to have the gabion apron protect the stilling pool below the original bed level. The hydraulic jump is controlled by the abrupt rise at the counter weir.

In most situations where gabion groins are used, the energy head of water to be dissipated is only a few meters and the river bed is made up of coarse or very compacted material that does not scour deeply. When the river bed is made up of loose material, the maximum depth of scour than can be caused by clear-water fall must be evaluated. The foundation of the weir should be deeper than the maximum possible scour depth, in order to avoid undermining of the structure.

Additional recommendations given with regard to the stilling basin include: (1) using a double layer of thin gabions to protect the bed of the stilling pool if severe floods carry heavy bed load that could cause damage; (2) filling the gabions in the apron with large stones (20 to 30 cm), preferably rounded; and (3) protecting the side slopes adjacent to the weir from scour with either sloping revetments or side walls, possibly extending upstream and downstream of the weir and not connected with the downstream apron, as the apron must be left free to deflect downward.

Seepage Control And the Prevention of Undermining

Seepage through the foundation soil must be minimized to prevent the weir structure from being undermined or outflanked. The seepage velocity should be such that the smallest particles of the foundation soil are not washed away. Undermining of the weir structure can be prevented by constructing an impermeable cut-off under and at the sides of the structure. When technical or economic reasons make the construction of the cut-off impossible or inconvenient, other methods for controlling seepage may be needed, such as placing gravel or synthetic filter cloth underneath the structure. Laying the synthetic filter cloth is usually easier and faster than placing the stone filter.

Structural Stability

The factors affecting structural stability are given in detail by Stephenson (1978). They include consideration of the unit weights of water and of the filling material for the gabion baskets and the soil. The density of water can double when suspended sediment loads are large; this must be considered in stability analysis. For the gabion basket filled with quarry stones, the mass of the wire mesh is negligible when compared with the mass of the filling material. The horizontal thrust on the structure involves the hydrostatic and soil pressures, so these too must be considered. Hydraulic uplift forces are exerted on the weir, the steps of the weir, and the stilling pool apron and must be included in analysis.

Model Studies

Scope of Studies

The laboratory investigations undertake with gabion weirs involved individual V-shaped weirs at several apex angles, ranging from 30 degrees (i.e., the V pointing upstream) to 300 degrees (i.e., the V pointing downstream). The objective of this part of the work was to determine the effect of weir apex angle on flow patterns and streambed scour and deposition patterns just downstream of the weir. The weir apex angle that provided the largest scour hole was also to be determined. One purpose of the model tests was to learn which weir shapes might be useful for fish habitat modification.

Laboratory Apparatus

The laboratory studies were conducted in the same flume as used for gabion groin experiments. The model weir cross sections had dimensions of 5.3 in x 3.9 in (13.5 cm x 10.0 cm). This corresponded to a 1:10 scaling ratio compared to a prototype gabion, assuming the weir to be built with a partially buried 1.0 m x 1.0 m gabion stacked with a 0.3 m x 1.0 m gabion.

Laboratory Procedures

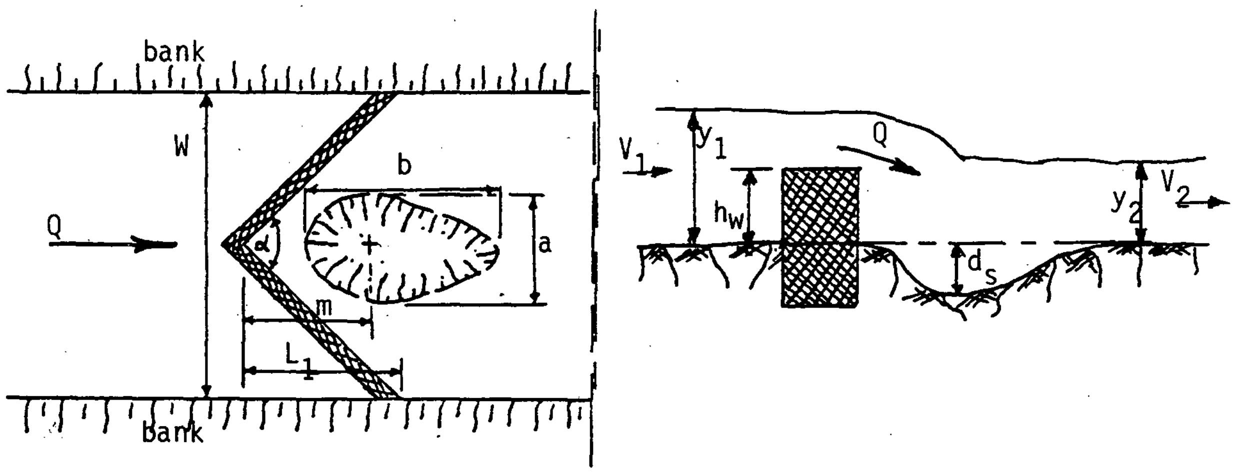
The gabion weir experiments were conducted by means of fifteen test runs. The test conditions for each run are summarized in Table 15. The terms and symbols used in this section of the report are explained by the definition sketch shown in Figure 40.

For each test run, the sequential procedures were as follows:

1. A V-shaped model gabion weir basket was constructed with the desired apex angle.

Table 15. Summary of Gabion Weir Laboratory Test Conditions

Run	Discharge Q, cfs	Apex Angle, α, degrees	Run	Discharge Q, cfs	Apex Angle, α, degrees	Run	Discharge Q, cfs	Apex Angle, α, degrees
1	0.51	30	6	0.87	60	11	0.51	150
2	0.87	30	7	0.51	90	12	0.87	150
3	0.51	45	8	0.87	90	13	0.51	180
4	0.87	45	9	0.51	120	14	0.87	180
5	0.51	60	10	0.87	120	15	0.87	300



Plan View

Profile View

Q = discharge
W = channel width
α = weir apex angle
a = scour width
b = scour length
m = location of point of maximum scour depth from downstream side of weir apex
d_s = maximum scour depth
L_1 = distance from downstream side of weir apex to V-weir base
y_1 = water depth upstream of weir
y_2 = water depth downstream of weir
V_1 = water velocity upstream of weir
V_2 = water velocity downstream of weir
h_w = height of weir

Figure 40. Definition Sketch for Terms Used in Gabion Weir Experiments

2. The empty V-shaped weir basket was installed over a stable foundation made of gabion blocks. The joints of adjacent foundation blocks were covered with thin plastic sheets to prevent concentrated flows there that might undermine the bed scour pattern.
3. The weir basket was filled with gravel and wired closed.
4. The channel bed was levelled and the initial average bed elevation was measured.
5. The height of the weir, hw, above the channel bed was measured.
6. The distance, L_1, from the downstream apex of the weir to the base of the V-weir was measured.
7. The pump was turned on with the discharge, Q, set at either 0.51 cfs or 0.87 cfs.
8. The flow was timed, beginning at the time water reached the downstream end of the channel.
9. Flow patterns near the weir were traced by means of red dye. The observed patterns were sketched.
10. The water depths upstream, y_1 , and downstream, y_2, of the weir were measured when flow reached steady-state conditions. The corresponding channel velocities V_1 and V_2, and Froude numbers, F_1 and F_2, were calculated from the measured water depths and discharge and the known channel width.
11. Progressive channel changes due to scour and deposition, and the corresponding gabion weir behavior, were noted.
12. The pump was stopped after a flow time of twelve minutes.
13. The scour pattern around the weir was photographed.

14. The maximum scour depth, d_s, and its distance downstream from inside the weir apex, m, were measured. The length, b, and width, a, of the scour hole was measured.

15. The experiment was repeated from step 4 and the new measured values were averaged with those obtained the first time to improve the accuracy of the measurements.

The experiments were conducted with weir apex angles of 30, 45, 60, 90, 120, and 150 degrees, as well as for the special case of 180 degrees (a straight weir across the channel). A less detailed experiment was performed for the case of the weir apex pointing downstream, using an apex angle of 300 degrees and the larger test discharge. The primary interest here was to contrast the difference in flow and bed scour patterns for weirs pointing upstream and downstream.

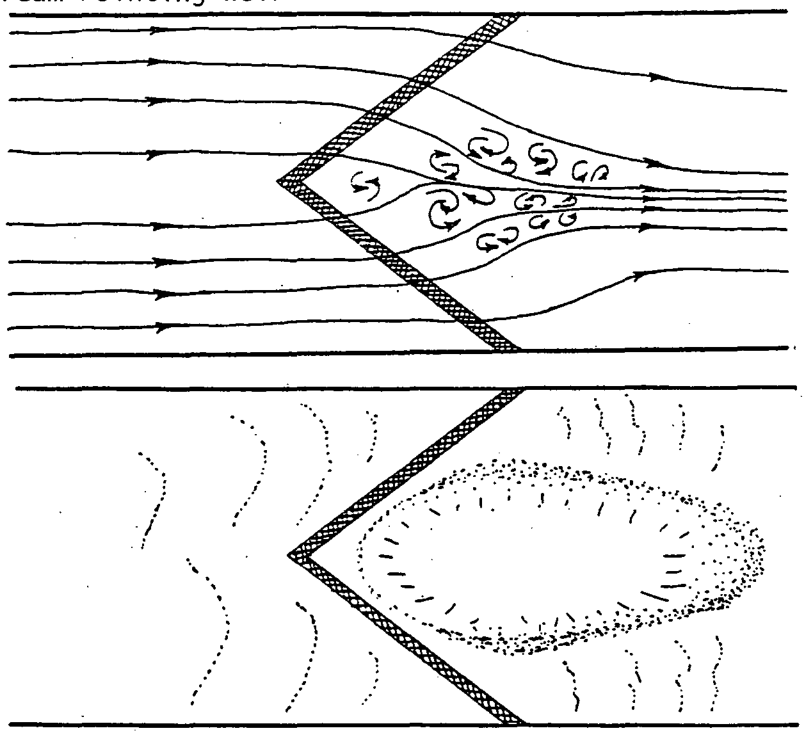
Laboratory Results and Observations

The general flow and scour/deposition patterns associated with V-shaped weirs are schematically illustrated in Figure 41. Figure 42 shows actual test results. The patterns shown are typical of those observed for various apex angles with the V-weir pointing upstream and downstream.

With the weir apex pointed upstream (apex angle of less than 180 degrees), the flow past the weir was focused toward mid-channel. The resulting converging flow formed eddies and vortices that scoured the channel bed to create an oval-shaped scour hole at the center of the channel. Sediment eroded from the scoured area was deposited in weakening currents at the edges of the scour hole or was transported downstream.

Different flow and scour patterns occurred when the weir apex pointed downstream (apex angle greater than 180 degrees). In this case, the flow past the weir was spread away from mid-channel. The deflected flow tended

a) Upstream-Pointing Weir



b) Downstream-Pointing Weir

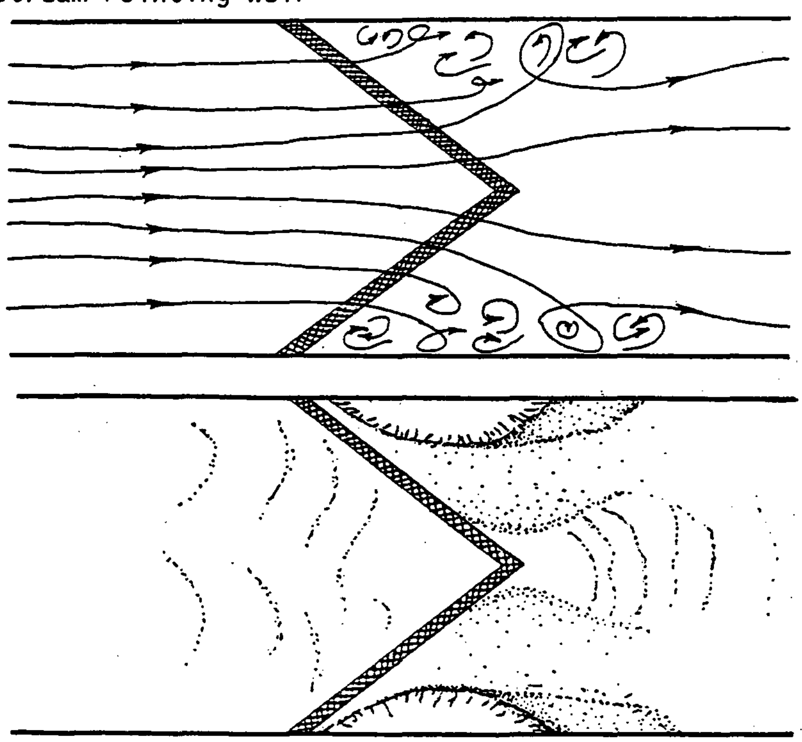


Figure 41. Flow Patterns and Corresponding Scour Patterns for V-Shaped Gabion Weirs Pointing Upstream and Downstream (Dots represent deposition)

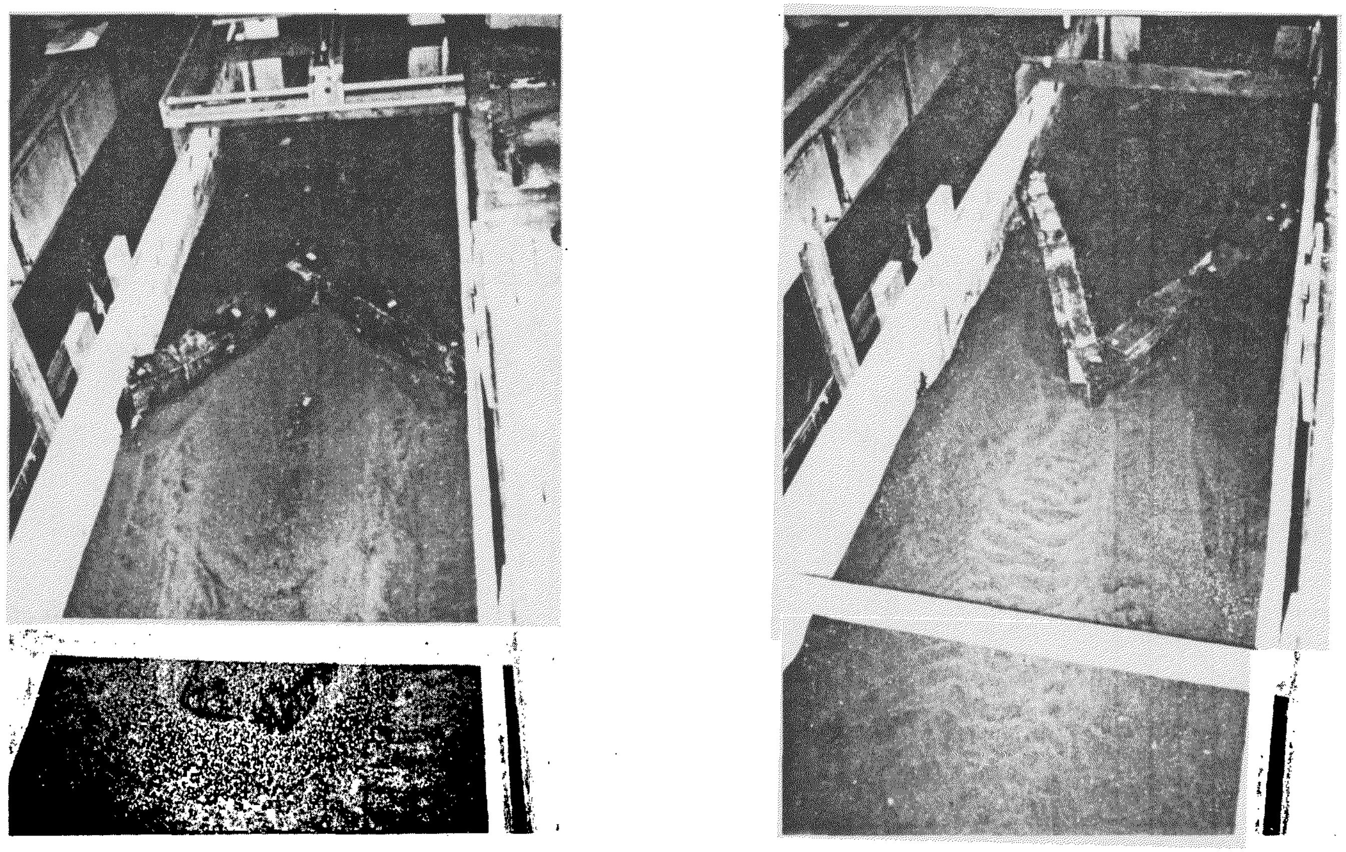


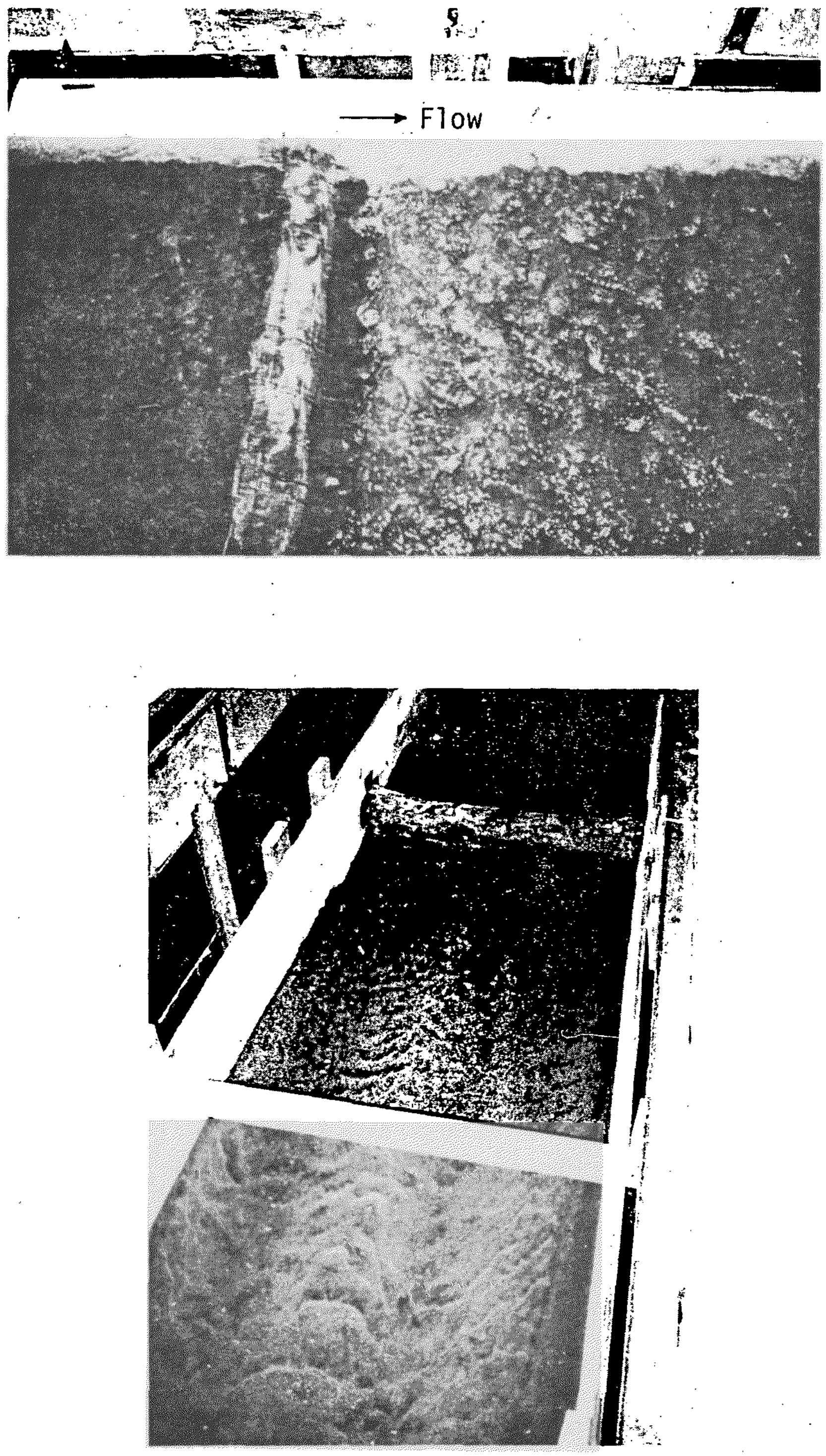
Figure 42. Typical Bed Scour Patterns for V-Shaped Gabion Weirs
Pointing Upstream and Downstream

to concentrate at the sides of the channel downstream of the weir. The resulting eddies and vortices scoured the channel bed to create two scour holes, one near each bank. (These could be thought of as symmetrical halves of the single scour hole created when the weir apex pointed upstream.) Sediment eroded from the scoured holes deposited just downstream and at the middle of the channel bed. Some sediment was also transported farther downstream.

With the weir straight across the channel (weir apex angle equal to 180 degrees), the turbulence and eddies downstream of the weir were quite uniformly spread across the channel, as shown in Figure 43. This flow pattern differed strongly from the flow patterns associated with the V-shaped weir, where flow either concentrated at the center or at the sides of the channel. The bed scour pattern associated with the straight weir is also shown in Figure 43. The whole cross-section of the channel was scoured, without any one point on the bed subject to a distinctly greater scour depth. Eroded sediment was transported downstream, forming bed ripples along the way.

The measurements for the several parameters are shown in Table 16. The computed hydraulic values associated with the test runs are shown in Table 17. The critical velocity was found to be 1.0 fps, using the flume bed grain size of 1.5 mm (0.06 in) and Hjulstrom's curve for incipient motion (Vanoni, 1975).

The graphical relationships between the weir apex angle and the maximum scour depth, the location of the maximum scour depth from the weir apex, the scour hole length, the scour hole width, and the scour volume index, SVI, are shown in Figures 44 to 48, respectively. SVI is a contrived term to indicate the relative scour volume associated with the various weir apex



Looking Upstream

Figure 43. Flow Pattern and Corresponding Scour Pattern for Straigh Gabion Weir

Table 16. Gabion Weir Scour for Various Apex Angles and Discharges. Flow Time of 12 Minutes and Weir Height of 0.365 Feet.

α (degrees)	L_1 (ft)	ds (ft)	b (ft)	a (ft)	SVI =axbxds (ft^3)	m (ft)
Q = 0.51 cfs						
30	6.50	0.000	----	----	----	----
45	4.00	0.050	4.50	0.54	0.122	1.69
60	3.25	0.240	1.83	0.71	0.312	0.85
90	1.92	0.266	1.67	0.92	0.408	0.88
120	1.17	0.256	1.96	1.04	0.522	2.00
150	0.58	0.208	2.67	0.88	0.489	1.44
180	0	0.053	1.02	3.50	0.203	----
Q = 0.87 cfs						
30	6.50	0.118	7.50	0.83	0.735	2.04
45	4.00	0.176	5.42	1.08	1.030	1.88
60	3.25	0.420	3.71	1.33	2.072	1.17
90	1.92	0.465	4.04	1.54	2.893	1.21
120	1.17	0.407	4.25	1.71	2.958	2.13
150	0.58	0.359	4.42	1.42	2.253	1.71
180	0	0.089	1.74	3.50	0.542	----

Table 17. Computed Hydraulic Values for Gabion Weir Laboratory Test Runs

Q (cfs)	y_1 (ft)	y_2 (ft)	V_1 (fps)	V_2 (fps)	τ (psf)	F_1	F_2
0.51	0.410	0.325	0.355	0.448	0.199	0.10	0.14
0.87	0.476	0.389	0.533	0.639	0.204	0.14	0.18

Note: Subscripts 1 and 2 represents the values upstream and downstream of the gabion structure, respectively.

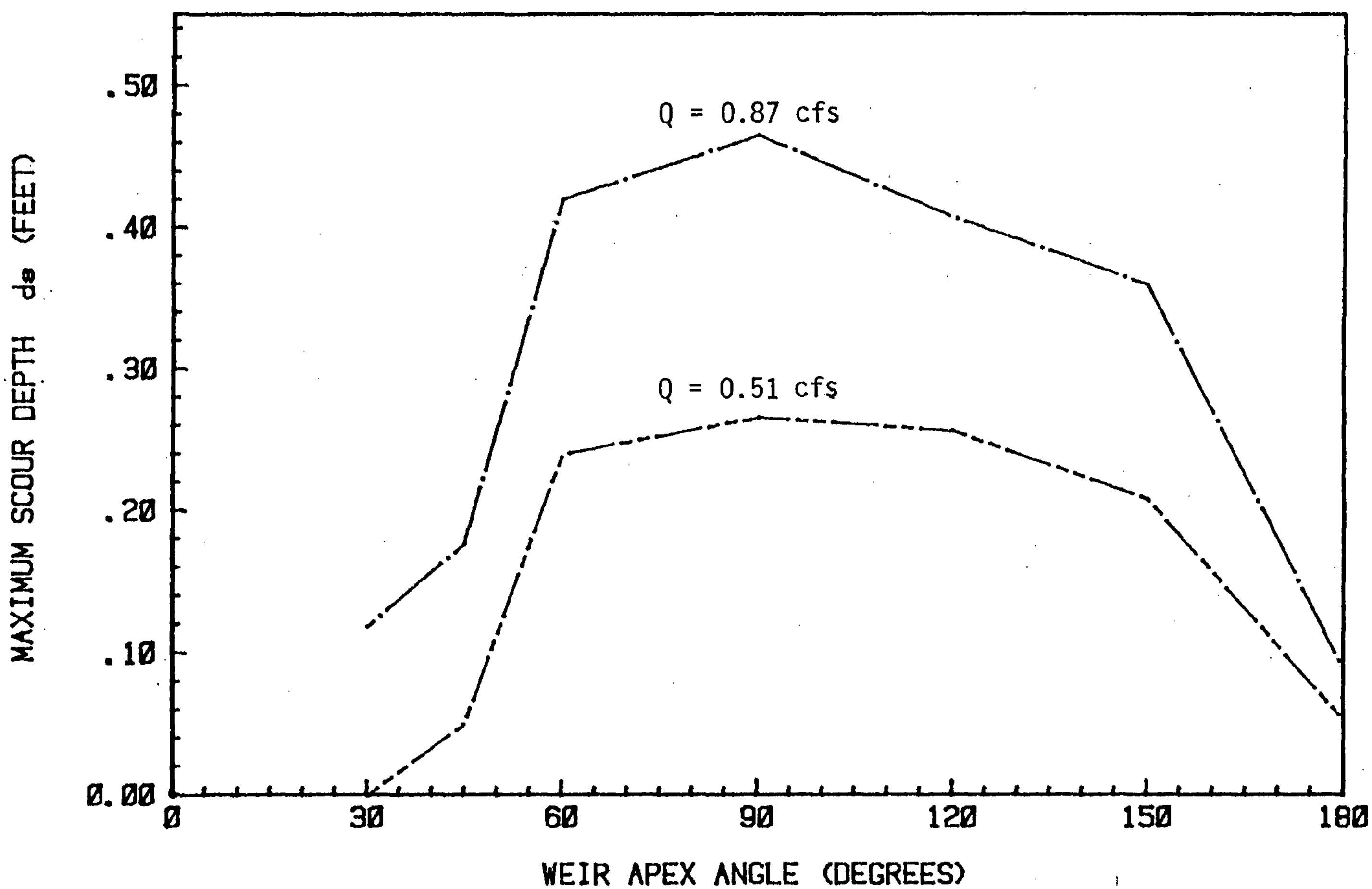


Figure 44. Maximum Scour Depth as Function of Gabion Weir Apex Angle

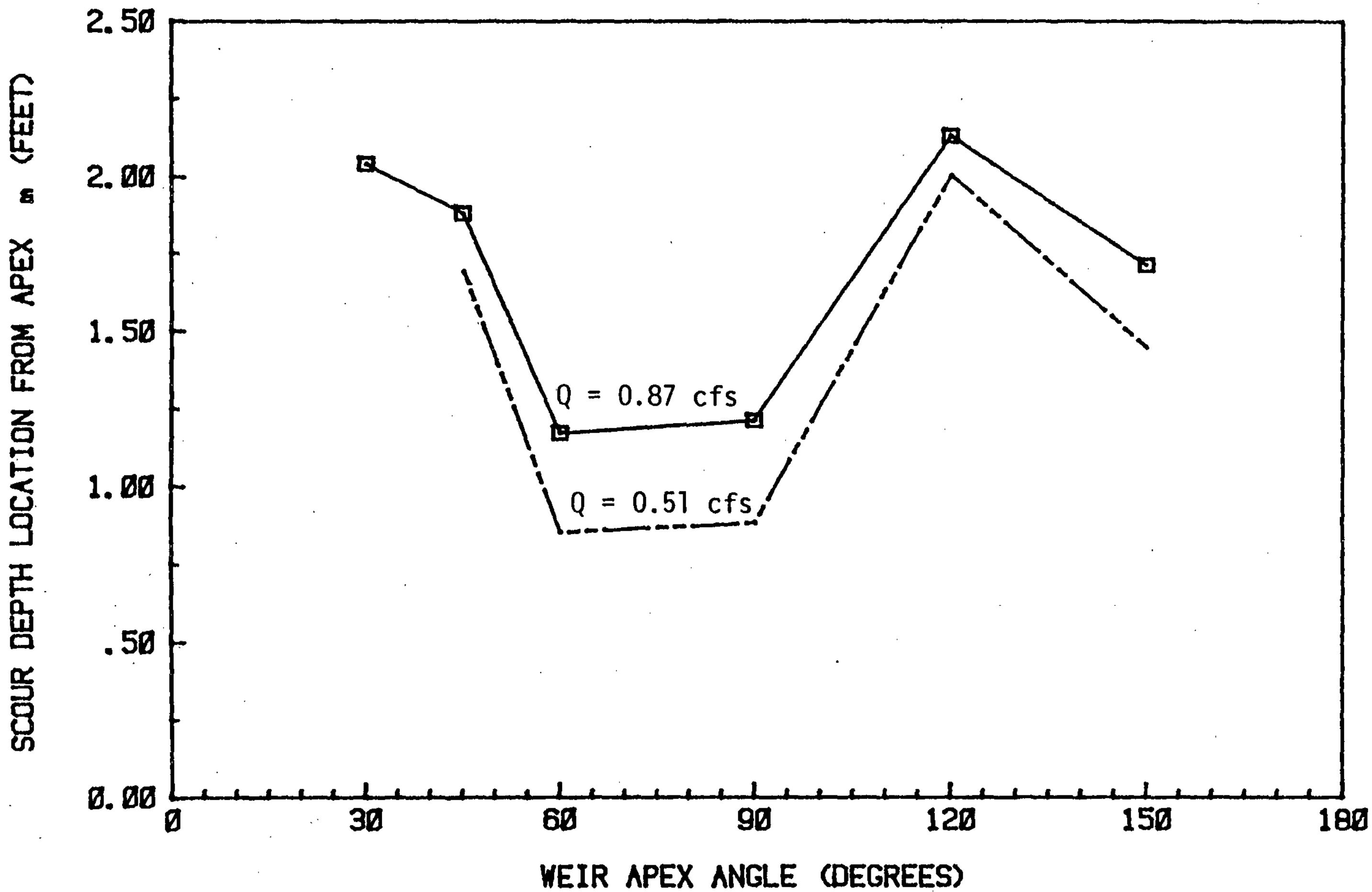


Figure 45. Location of Maximum Scour Depth as Function of Gabion Weir Apex Angle

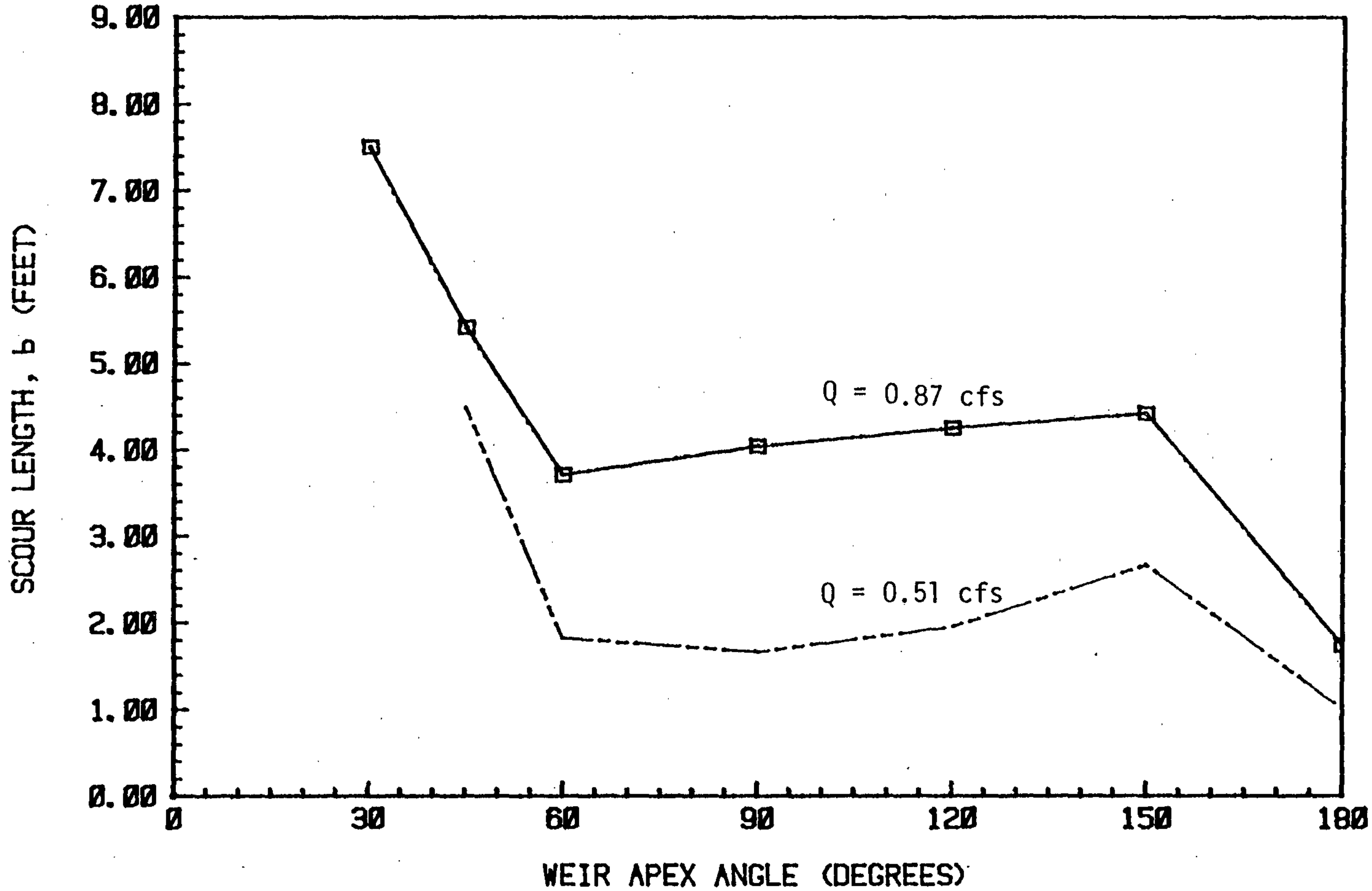


Figure 46. Length of Scour Hole as Function of Gabion Weir Apex Angle

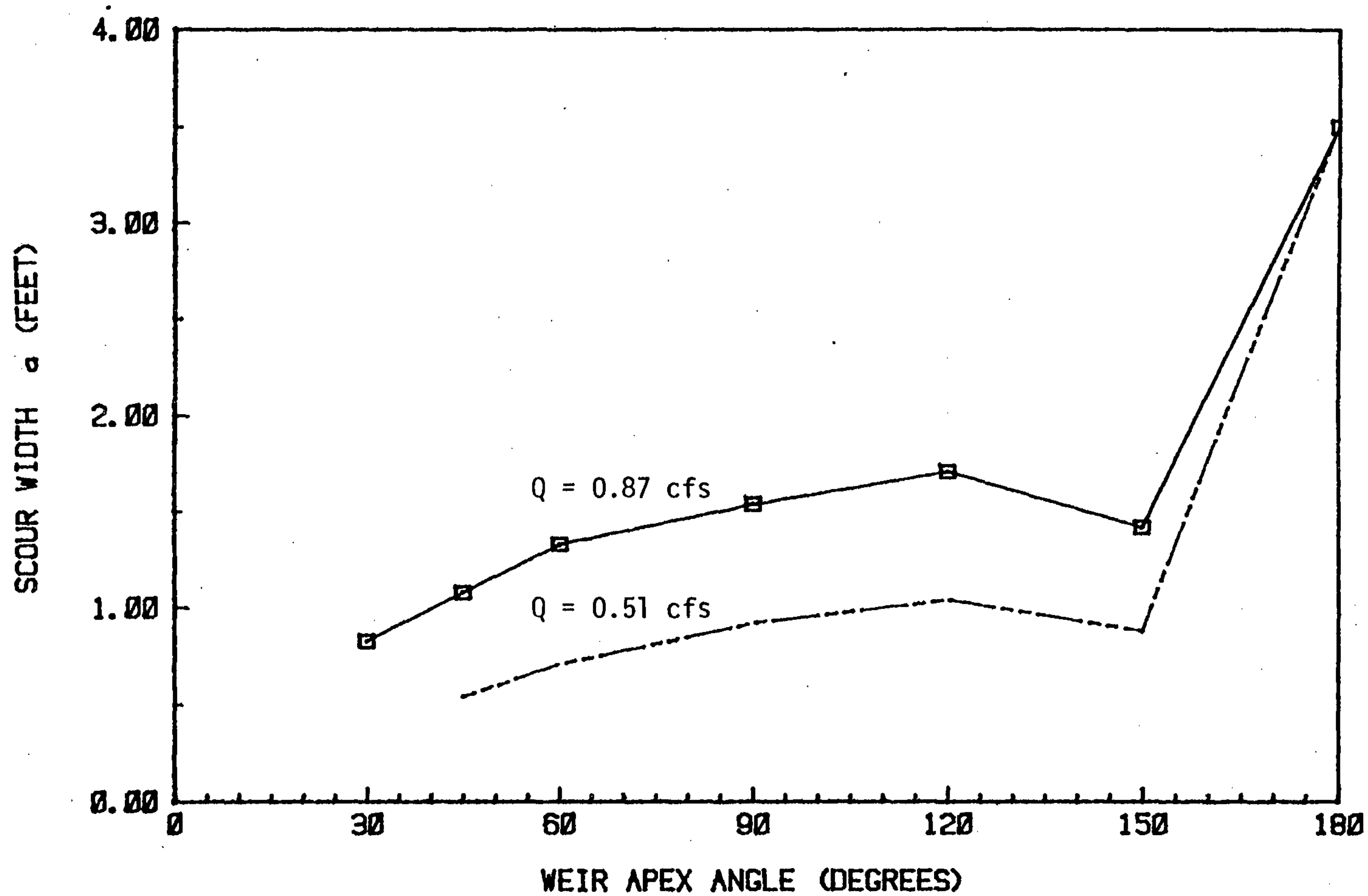


Figure 47. Width of Scour Hole as Function of Gabion Weir Apex Angle

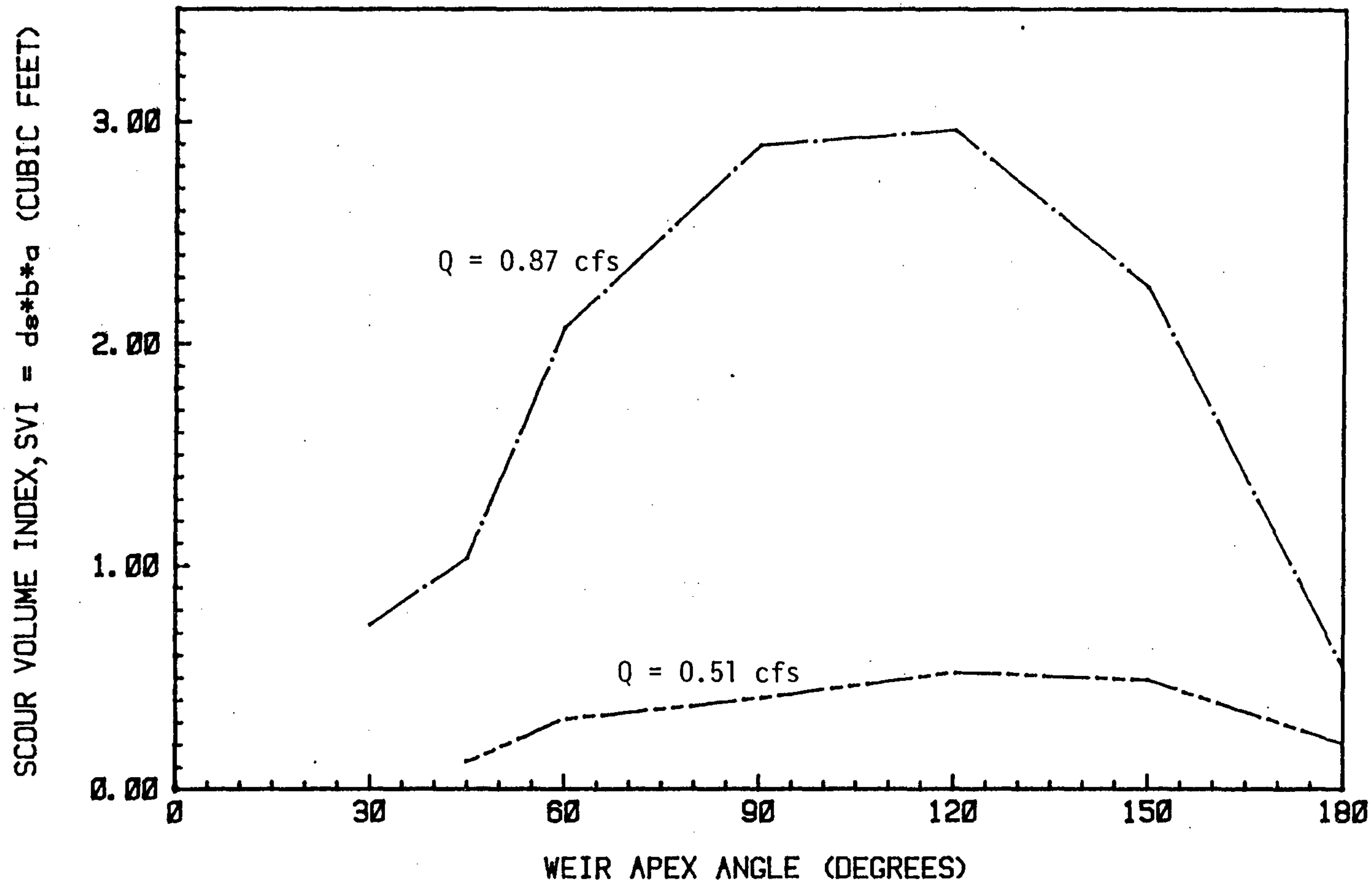


Figure 48. Scour Volume Index as Function of Gabion Weir Apex Angle

angles tested. It is defined as the product of the width and length of the scour hole and the maximum scour depth (i.e., $SVI = a \times b \times d_s$). It should be remembered that all test runs were for short times only. These gave good indications of the relative effects due to different apex angles but did not give ultimate magnitudes for each parameter.

<u>Discussion</u>

Figure 44 shows that for the various weir apex angle values tested, a 90-degree weir apex resulted in the greatest depth of scour at the tested discharges. More generally, apex angles from 60 degrees to 120 degrees gave relatively deep scour. The point of maximum scour was closer to the structure for 60- and 90-degree apex angles than for the 120-degree angle, as shown in Figure 45.

The location of the point of maximum scour was also the location where the width of the scour hole, a, was measured, since the two parameters generally coincided at this location. Thus, the location of maximum scour provides information about the critical width of scour and the distance downstream from the weir at which streambank protection may be needed.

The width of the scour hole created by the weir is of interest because of the possibility that it may extend to erode the streambanks. Figure 47 shows that there was not much variation in scour width for differing weir apex angles until the straight weir condition was approached. Figures 43 and 47 show that the whole width of the bed was scoured when a straight weir was used. Bank protection measures may be necessary in the vicinity of the straight weir, such as revetments on both sides of the channel.

The length of the scour hole provides information on how far downstream the scour hole could extend. Figure 46 provides a comparison for the expected scour lengths associated with the various weir apex angles. The straight weir (apex angle of 180 degrees) gave the minimum scour length. The maximum scour length was obtained at the smallest apex angle tested (30 degrees).

From Figure 48, it is seen that a 120-degree weir apex angle gave the largest scour volume index, although the index for a 90-degree apex was not much smaller. If pools are desired for fish rearing habitat, one might want a scour hole with a large scour volume and scour depth. A weir apex angle within the range of 90 to 120 degrees would appear to provide these needs better than would other angles.

Supporting Field Observations

Field visits were made to several sites where gabion weirs have been installed for fishery enhancement purposes and have been in place for lengths of time ranging from a few months to a few years. Field observations at these sites generally confirmed the scour patterns associated with the weir shapes tested in the laboratory. Field scour was limited in depth and extent because of the coarse cobble streambeds at most sites.

Trapping gravel for fish spawning has been done in the field by combining two or more V-weirs with their apex pointing downstream. The deposition of sediment in the middle of the channel, which was observed in the laboratory when the apex of the weir was pointed downstream, also was observed in the field. It appeared that when two or more such weirs are combined, gravel was easily trapped between the weirs. Diagonal weirs also appear to be effective in trapping gravel, causing scour, and inducing bar deposits downstream of the weir.

More complex gabion structure configurations are also being used in Oregon's streams for fish habitat enhancement purposes, such as herringbone layouts and arrow layouts in mid-channel, W-shaped weirs, and F-shaped groins. More research needs to be done to determine the performance of these complex structures.

Summary and Conclusions

The results and discussion already presented for the gabion weir experiments lead to the following conclusions and recommendations with regard to gabion weirs:

1. The V-shaped gabion weir with its apex pointing upstream (weir apex angle less than 180 degrees) creates a scour hole at the center of the channel bed downstream of the weir.
2. When the apex of the weir is pointed downstream (weir apex angle greater than 180 degrees), two scour holes are created, one at each side of the channel. In this case, bank protection measures at the sides of the channel are necessary to prevent erosion. Sediment deposition tends to occur in mid-channel downstream of the weir apex.
3. The spread-out nature of the flow and bed scour patterns created by the straight weir suggests that bank protection measures may be necessary near the weir at both sides of the channel.
4. The biggest scour hole development (e.g., for fish rearing habitat) is expected to occur for a weir apex angle within the range of 90 to 120 degrees, as these angles result in the maximum scour depth and scour volume.

5. The V-shaped weir with its apex pointing upstream provides a bigger scour hole than does the straight weir. The V-weir creates a deep scour hole at the center of the channel bed, while the straight weir creates a shallow scour hole that is spread across the width of the channel.

VII. SUMMARY AND CONCLUSIONS

Scope and Limitations

The research reported here emphasizes the hydraulic evaluation of spur dikes, groins and low weirs used in streams to protect banks against erosion and to manipulate the location and depth of bed scour. Of particular interest is the potential joint application of these structures for streambank protection and fishery habitat modification.

Two structural types of dikes and groins were investigated: riprapped rockfill and rockfilled gabions. One structural type of weir was considered: rockfilled gabions. Rockfill structures were investigated because of their widespread use, the general ready availability of rock material in most locations, the relatively non-complex design and construction involved, and the expected long life of well-designed structures.

The research focused on the geometric characteristics of structure design. These included dike/groin orientation with respect to the bank, dike/groin extension into the flow, dike/groin spacing when more than one structure is used, and weir apex angle for V-shaped weirs. The sediment scour and deposition characteristics were also evaluated in the geometric sense of location, depth and extent.

The research was based on a combination of literature review, laboratory experimentation with physical models, and field investigations. The laboratory work was more extensive than the field work, although the time span for field work was up to one year in the case of a model-prototype comparison of spur dikes on the Willamette River.

The laboratory work involved physical models in artificial channels. The structures varied in size, ranging from 1:600 for study of Willamette River spur dikes to 1:10 for study of gabion performance in small streams. The laboratory flow conditions in the approach channels upstream of the structures were such that near-critical conditions for incipient motion of bed material prevailed. Many researchers consider this case to produce the most severe scour at a structure, as larger flows cause general bed load transport to bring replacement material into the scour hole. However, for small bed material, general transport can produce deeper scour, particularly if the structures greatly constrict the flow. The model tests were conducted for time intervals ranging from several minutes to 20 hours. These did not give maximum scour, which is approached asymptotically with time. The short tests were used to determine relative scour characteristics for various structure placements in the channel. The longer tests were used to verify that the shorter-term observations were consistent with longer-term trends and thus properly indicative of scour differences due to structure placement differences. The longer tests were also used to estimate impacts on structure stability. The movable bed material used for model studies was fine-to-coarse sand, which gave qualitative information on scour. This size was chosen arbitrarily, rather than modeling any particular prototype sediment.

Hydraulic Behavior of Spur Dikes and Groins

General Features

Spur dikes and groins directly affect flow velocities and patterns. The flow impinging on the structure produces strong vortices. Eddy currents trail downstream from the structure. The vortices and eddies concentrate the flow strength and erosive capability. This has a direct effect on the location and amount of sediment scour and deposition. The structure also deflects the flow, which may then impinge against the opposite bank or curve back to the original bank. In either case, the structure has a direct effect on the location of bank erosion and bank protection.

The deepest scour occurs near the tip of the structure. The actual magnitude of this local scour depends upon how the structure interacts with the flow. Important factors investigated in this study that affect the depth and size of scour include the orientation of the structure, the amount of flow constriction caused due to the length of the structure, and the structure configuration. Other important factors that must be considered for design but that were not specifically investigated in this study include the sediment size characteristics and cohesiveness and the effect of variations of discharge that produce short periods of general bed load tansport. Regarding these uninvestigated factors, a few words of comment must be added. If the bed is relatively coarse (e.g., coarse gravel and cobbles), the depth and extent of scour are expected to be smaller than for a relatively fine bed (e.g., sand and fine gravel). A cohesive bed is also expected to be less deeply scoured than a non-cohesive bed. A typical river experiences variable large discharges rather than sustained large discharges. Consequently, the ultimate maximum depth of scour over time is never attained. Furthermore, when the river discharges are most capable of producing deep scour they are also capable of transporting bed load into the scour hole from upstream. It is not yet clearly agreed in recent literature whether the upstream clearwater-flow case or the general-bed-load-transport case produces the deepest scour. But the recession flows for a runoff hydrograph are likely to transport bed material into the scour hole to deposit while the streambed armor layer in redeveloping upstream and clearwater flow conditions are being reestablished. Hence, the residual scour hole is likely to be smaller than the maximum high-water scour hole. This aspect is of great importance when the structure is being used to create scour holes and is different from the structural design aspect involving determinating the base elevation from the predicted maximum scour depth.

The length of bank downstream of a dike that is protected by that dike against erosion is somewhat less than the distance to the point on the bank where the dike-deflected current impinges against the bank. This is because an eddy current moves along the bank upstream of the point of impingement and can cause some erosion.

Table 18 summarizes the hydraulic behavior observations made for spur dikes and groins during this investigation. The effects are noted of structure orientation and relative length, as well as any differing effects due to use of single or multiple structures. Two categories of applications of the structures are considered: bank erosion control and channel scour control. The several conditions mentioned in this table are summarized in the following paragraphs.

Dike/Groin Orientation to Bank

The literature indicates considerable controversy as to whether structures placed perpendicular to the flow, oriented upstream, or oriented downstream give the greatest amount of bank protection against erosion.

Our model tests showed that dike orientation did make a difference in the flow deflection and length of bank protection provided downstream of the dike (see Figure 20). The upstream-oriented (135-degree) spur dike

Table 18. Summary of Hydraulic Behavior Observations for Spur Dikes and Groins

Application of Structure	Relative Channel Constriction Le/W	Single or Multiple Structures	Structure Orientation to Flow, Measured from Downstream Bank: Upstream Oriented (135 degrees)	Normal to Flow (90 degrees)	Downstream Oriented (45 degrees)
For bank protection	1/6 - 1/2	Single	Most Effective	Very Effective	Effective
			Strong flow deflection	Limited flow deflection	Good flow deflection
			Longest flow deflection and bank protection ◄	(intermediate)	► Shortest flow deflection and bank protection
			Some bank erosion upstream of dike	No bank erosion upstream of dike	
	1/6		Downstream erosion protection extends for a distance of 3.5 - 5.5 Le		
			5.5 Le ◄		► 3.5 Le
			Downstream flow deflection extends for a distance of 8.7 - 10.0 Le		
			10.0 Le ◄		► 8.7 Le
	1/2		Downstream erosion protection extends for a distance of 1.9 - 3.5 Le		
			3.5 Le ◄		► 1.9 Le
			Downstream flow deflection extends for a distance of 1.9 - 3.5 Le		
			6.0 Le ◄		► 4.5 Le
	<1/4	Multiple Prototype	90 degrees Excellent bank protection for X/L range of 3.7 - 8.0 where bend curvature was moderate		
	1/4 - 1/2	Multiple (2)	Part of the streambed between structures is also protected. Width of protection depends on structure spacing and flow constriction.		
			◄ (increasing) protection		(decreasing) protection ►
To cause or avoid scour in channel	1/6 - 1/2	Single	Largest depth and surface area of scour	Large depth and surface area of scour	Large depth and surace area of scour
			◄ (increasing) depth and area		(decreasing) depth and area ►
			Scour area includes zone upstream of tip	Scour area localized near tip	Scour tends to occur in deflection path downstream of tip
			Scour pronounced at upstream side and tip of groin, causing gabions to twist, rotating upstream and downward into the scour hole, with greater scour and structure twist for longer groins.		
	1/4 - 1/2		Scour extends across channel to opposite bank	Scour tends to extend downstream rather than across channel to opposite bank	Scour extends across channel to opposite bank

Table 18. Continued

Application of Structure	Relative Channel Constriction Le/W	Single or Multiple Structures	Structure Orientation to Flow, Measured from Downstream Bank: Upstream Oriented (135 degrees)	Normal to Flow (90 degrees)	Downstream Oriented (45 degrees)
	1/6 ↑ ↕ 1/2 ↓			Scour Area: ↑ Decreasing ↓ Increasing	
	1/4	Multiple	90 degrees Individual scour patterns tend to overlap and merge		
	1/4 - 1/2	Multiple (2)	Upstream structure protects downstream structure, experiences greater maximum scour and greater structure settlement ← (increasing)		(decreasing) →
	1/4	1 to 2 to 3 X/L = 2	90 degrees As number of structures increases, so does total scour area, but at a lesser rate As number of structures increases, so does the deflection distance past the last structure		
	1/4 - 1/2	Single	90 degrees T-head, L-Head, J-Head and straight structures cause similar scour areas		

protected almost 50 percent more streambank against erosion than did the downstream-oriented (45-degree) spur dike at all four conditions of channel contraction tested (1/6, 1/4, 1/3, 1/2). The perpendicular dike gave slightly more protection than the downstream-oriented dike. The upstream-orientation caused some bank erosion upstream of the dike. The flow deflection findings followed a pattern similar to that for erosion pattern, except that the increase in deflection distance for 135-degree dikes over 45-degree dikes was only about 20 percent.

The surface area of scour was found to be affected by the structure orientation. As orientation angle increased, the scour area also increased. The rate of increase and the absolute area of scour were greater as the amount of channel contraction increased (see Figure 21). The amount of scour upstream of the dike tip also increased as the dike becomes more upstream oriented.

The scale of model testing with spur dikes did not allow realistic measurements of scour depth to be made. Therefore, it is not known if scour depth also increased with scour area. For gabion groins, model-tested in larger sizes, scour depths and structure settlement were determined. Both increased with orientation angle (see Tables 11 and 12 and Figures 35 and 36).

Dike/Groin Length and Spacing

The literature generally treats structure length in conjunction with the spacing of multiple structures. (The assumption appears to be made that individual structures are unlikely to be used for bank erosion control. However, single or isolated structures are likely to be used for habitat modification.) The effectiveness of bank protection diminishes as the structure spacing/length ratio increases, as would be expected.

Conservative recommendations in the literature are that the structure spacing should not exceed about twice the structure length; however, some recommendations are for ratios as large as 4 to 6 along concave banks, with a supplemental recommendation that the bank may need riprap.

Dike/Groin Length.

Our model tests showed that dike effective length normal to the bank did make a difference in the length of bank protected and in the distance of flow deflection, regardless of orientation (see Figure 20). Relatively short dikes (Le/W = 1/5) gave downstream erosion protection for 3.5-to-5.5 times the effective length, whereas long dikes (Le/W = 1/2) gave protection for 1.9-to-3.5 times the effective length. Even though the latter ratios are smaller than those for short dikes, the absolute distances are greater due to the greater magnitude of the effective length. The corresponding deflection distances were 8.7-to-10.0 for short dikes and 4.5-to-6.0 for long dikes. Again, even though the ratios decreased, the absolute distances increased.

The surface area of scour was affected by the structure length relative to the channel width. As the degree of channel contraction increased, at a fixed orientation (e.g., 90 degrees), the scour area increased (see Figure 21).

Multiple Dike/Groin Spacing

Our model tests showed that when more than one structure was used to protect a bank, the individual scour patterns tended to overlap and merge unless the dikes were far apart. For conditions of L/W = 1/4 and X/L = 2, it was observed that as the number of structures increased from one to three, the total scour area also increased, but less rapidly. The current deflection and bank erosion protection distances also increased downstream of the last structure.

Our model tests also showed that for paired structures, the upstream structure protected the downstream structure from experiencing as much scour and settlement as the upstream structure for spacings of up to about three times the structure length (see Figures 35 and 36). At spacings of three or more lengths, the downstream structure experienced almost as large a maximum scour depth as the upstream structure, particularly if the flow constriction was severe (an L/W ratio of 1/2) or the structure was oriented upstream for flow constrictions for L/W = 1/4 or more. However, this scour may not have been over as extensive an area, because the settlement of the downstream structure tended to remain less than that for the upstream structure. The width of the protected streambed between structures, measured away from the bank, varied with structure spacing. This width may represent a margin zone for buffering eddy currents that leave the upstream structure. For structures that severely constricted the channel flow (i.e., L/W = 1/2), the width of protected bed decreased for increasing spacing beyond an X/L ratio of 1, the closest spacing tested (see Figure 37). If there was less channel flow constriction (i.e., L/W = 1/4), the protected zone was narrowest at close structure spacings and actually increased until the spacing became X/L = 4, after which the protected zone again narrowed. This trend indicated variable flow interaction between adjacent structures.

Our field observations showed that structures with variable X/L spacings of 3.7 to 8.0, as part of an 8-structure dike field, gave good bank protection. The approach flow to the dike field was fairly straight and the bend curvature at the dike field was moderate. A common condition between adjacent dikes was an eddy current similar to the type 1 pattern shown in Figure 11. The flow deflection past the last structure did not extend as far downstream as expected from our model tests. The difference is attributable to the time-lag before prototype adjustments occur when the bed material is very coarse and the length of time for large discharges is short, even when the structure has been in place for one year. Hence, residual streambed features can persist and influence flow patterns and deflection trajectories.

Dike/Groin Configuration

The literature indicates that the downstream-angled L-head structure is preferred over other non-straight configurations. Apparently, scour is not too severe nor too localized. Also bank protection is reported to be better when such structures are closely spaced than for straight structures having the same spacing.

Our limited model tests showed that the T-head structure caused a slightly larger scour area than the J-head and L-head. However, all three were similar in scour area produced and flow deflection trajectory (see Table 7). Furthermore, their performance was similar to that of a straight structure oriented at 90 degrees to the bank (see Tables 5 and 6). The scour areas were greater and the relative deflection distances were less at flow contractions of L/W = 1/2 than at L/W = 1/4. Sloping dikes that were partially submerged were found to behave like shorter dikes, in terms of resulting scour area and flow deflection. The effective lengths of such structures was related to their unsubmerged lengths.

Hydraulic Behavior of Weirs

General Features

Like spur dikes and groins, weirs directly affect the local flow velocities and patterns. The weir causes a backwater effect that extends upstream for some distance, flattening the local stream gradient, compared to the general stream gradient, with corresponding local decrease of flow velocity and increase of flow depth. This can cause sediment deposition. If the entire space behind the weir becomes filled with deposited sediment, the weir instead acts like a sill across the channel.

As the flow reaches the weir or still, it accelerates and plunges toward the streambed just downstream. This accelerating, plunging flow causes local scour; a scour hole forms near the base of the weir. The scoured material redeposits in the channel a short distance downstream, possibly helping to "pool" the water over the scour hole. The amount of scour depends upon the weir height relative to the upstream and downsteam flow depths.

V-Weirs: Influence of Apex Angle

The literature generally deals with straight weirs placed at right angles to the flow (i.e., a V-weir with an apex angle of 180 degrees). Scour evaluation is typically based on concepts of jet scour and free-overfall scour. Such evaluations usually emphasize the maximum depth of scour, rather than scour location, shape, and volume or resulting sediment redeposition.

Our model tests showed that the low straight weir had quite different effects on flow patterns, bed scour, and sediment redeposition than did low V-shaped weirs. The straight weir represents a transition case between

V-shaped weirs that point upstream and those that point downstream. For the straight weir, the approaching flow tended to continue straight downstream across the weir, plunging as it passed the weir and causing some scour at the toe of the weir and for a short distance downstream. A shallow scour pool formed across the full width of the channel and extended for a short distance downstream. The scoured sediment redeposited downstream of the scour hole but was spread out over an extensive surface area of the bed.

V-weirs with their apex pointing upstream had the effect of focusing the approach flow so that it moved toward mid-channel as it passed over the weir. This caused intense local scour of considerable depth. The extent of the scour area was limited in part by the space available between the two arms of the weir extending from apex to bank. The scoured sediment was pushed toward the channel banks as well as downstream before it redeposited.

V-weirs with their apex pointing downstream had the effect of spreading the approach flow so that it divided over the apex and moved toward both banks as it passed over the weir. Approaching the banks, the flow was then turned strongly downstream. This situation caused intense local scour near both banks. The scoured sediment was transported out of each scour hole, part of it moving to mid-channel, where it redeposited a short distance downstream of the weir apex and part of it redepositing near the banks a short distance downsteam from the scour holes.

Use of Rockfill Structures to Manipulate Scour

Favorable Situations Exist

The general literature and the specific work conducted in this study make it clear that rockfill structures can be used to manipulate sediment scour and deposition. In most existing applications, such manipulation has been undertaken for "defensive" or preventative reasons of protecting river banks or river structures. Scour manipulation in the "offensive" or positive sense of encouraging scour to occur has been an uncommon application. Yet there are many situations where the intentional encouragement of scour may be desirable. For example, it may be advantageous to encourage bed scour in the vicinity of water supply intakes so that clogging will not be a problem and so that pumps can operate at maximum capacity with adequate submergence of the inlet. Many other examples exist that can be cited. An example of particular interest in the Pacific Northwest (one which illustrates how the findings of this study can be used--if the study limitations are recognized), involves physical modification of stream habitat.

<u>Example: Fish Habitat Modification</u>

The typical situations in the Pacific Northwest where structures have been used for physical habitat modification involve coarse-bedded streams of small-to-moderate size that are subjected to strong seasonal variations of streamflow. During the summer low-flow season, warm temperatures combine with limited flow to greatly stress anadromous fish habitat. The coarse streambed often has extensive riffles and runs but relatively few pools to provide deeper water that may remain cool due to intragravel seepage. Fishery management for such stream reaches often includes efforts to modify habitat to increase the pool-to-riffle ratio.

One general concern regarding such management is that stream habitat modification may be undertaken as a single-purpose activity that ignores streambank stability and may accidently aggravate bank erosion. An understanding of the ways in which channel structures can modify scour and deposition should allow avoiding this side-effect. It might even allow the undertaking of dual-purpose projects to protect existing eroding banks and simultaneously provide habitat modifications.

Another concern regarding efforts to modify physical habitat in small streams involves the potential risk of inadequate design. Because many structures may be placed on small streams, some of the design guidelines may not apply that are applicable to large rivers. For example, our literature review did not reveal specific statements expressing concern over erosion of the bank opposite to that at which the structure was placed. Yet our model studies showed this to be a problem at flow constrictions of Le/W = 1/2 and a potential problem at Le/W = 1/4. Our field work in a small stream showed that the local channel and the opposite bank were severely affected by a groin causing a flow constriction of 1/2. Our field work in a major river showed that downstream effects could alter conditions at the opposite bank when structures caused a constriction of less than 1/4. This indicates that design for large rivers is not 100 percent risk-free and that structures in small streams may dominate the hydraulic conditions and lead to unexpected or undesired effects. Hence, large-river design methods must be used with considerable added caution in small streams.

The hydrology of small streams is often not known and must be estimated. Even for larger streams, information may be sketchy. While many hydrologic techniques are available to estimate missing streamflow characteristics, the net effect is that some risk and uncertainty will exist that will enter the design process. Fortunately, some rockfill structures are tolerant of moderately exceeded design conditions and can adust. For example, a gabion structure or rockfill with adequate riprap can settle into a scour hole that somewhat exceeds the expected design depth. The deformed structure can then continue to serve a useful function. However, if design conditions are severely exceeded, or if little design was used to install a structure, failure is as likely with rockfill structures as with other structural types.

In summary, with regard to this illustration, rockfill structures can be used to significantly increase the amount of bed scour and the pool-riffle ratio in a stream without causing bank erosion, as long as proper attention is given to design concerns. Such modifications usually require a large number of structures along the length of reaches where such changes are sought. (Obviously, this can become quite costly.) The structures must be positioned based upon their effects upon flow patterns and the resulting locales for sediment scour and deposition. This study has examined some of those effects; the findings add to the usable knowledge available because of the types of structures and structural materials considered and because of the specific interest in creating scour.

XIII. REFERENCES

Acheson, A.R.; River Control and Drainage in New Zealand; Ministry of Works; Wellington, New Zealand; 1968; 296 pp.

Agostini, R., Bizzarri, A., and Masetti, M.; Flexible Structures in River and Stream Training Works: Section 1, Weirs for River Training and Water Supply. S.P.A. Officine Maccaferri-Bolgona; 1981.

Ahmad, Mushtaq; "Spacing and Projection of Spurs for Bank Protection"; Civil Engineering and Public Works Review; London; March, 1951; pp. 172-174; and April, 1951; pp. 256-258.

Ahmad, Mushtaq; "Experiments on Design and Behavior of Spur Dikes"; Proceedings I.A.H.R. 5th Congress; U. of Minnesota, Minneapolis; 1953; pp. 145-159.

Awazu, Seizo; "On Scour Around Spur-Dike"; Proceedings I.A.H.R. 12th Congress; Vol. 3; Fort Collins, Colorado; 1967; pp. 97-104.

Bekaert Steel Wire Corp.; Bakaert Gabions; Form No. 6m; January, 1977; pp. 1-12.

Bell, Milo C.; Fisheries Handbook of Engineering Requirements and Biological Criteria; U.S. Army Corps of Engineers, North Pacific Division; Portland, Oregon; 1973.

Blaisdell, F.W., et al.; "Ultimate Dimensions of Local Scour"; Journal of the Hydraulics Division, ASCE ; Vol. 107, No. HY3; March, 1981; pp. 327-337.

Blench, Thomas; Mobile-Bed Fluviology; University of Alberta Press; Edmonton, Alberta, Canada; 1969; 168 pp.

Bradt, P.T. and Wieland, G.E.; The Impact of Stream Reconstruction and a Gabion Installation on the Biology and Chemistry of a Trout Stream; Department of Biology, Lehigh University; Pennsylvania; 1978.

Burroughs, M.A.; "Gabions: Economical, Environmentally Compatible Bank Control," Civil Engineering Magazine, ASCE; January, 1979; pp. 58-61.

Central Board of Irrigation and Power; Manual on River Behavior Control and Training; D.V. Joglekar, ed.; Publication No. 60; New Delhi, India; 1971; pp. 182-206.

Copeland, Ronald R.; Bank Protection Techniques Using Spur Dikes; Paper No. HL-83-1; Hydraulics Laboratory, U.S. Army Engineers Waterways Experiment Station; Vicksburg, Mississippi; 1983.

Cunha, L. Veiga; Discussion of "Erosion of Sand Beds Around Spur Dikes"; Journal of the Hydraulics Division, ASCE; Vol. 99, No. HY 9; September, 1973; pp. 1637-1639.

Das, B.P.; Hydraulics of End-Dump Closure of Alluvial Channels; University of Alberta, Department of Civil Engineering, Edmonton, Alberta, Canada; 1972; 229 pp.

Doddiah, D.; Comparison of Scour Caused by Hollow and Solid Jets of Water; Master's Thesis; Colorado A & M College; 1950; 156 pp.

Doddiah, D., Albertson, M.L., and Thomas R.; "Scour From Jets"; Proceedings, 5th Congress IAHR; 1953; pp. 161-169.

Federal Highway Administration; Restoration of Fish Habitat in Relocated Streams; FHWA-IP-79-3; U.S. Department of Transportation; Washington, D.C.; 1979.

Franco, J.J.; "Research for River Regulation Dike Design"; Journal of the Waterways, Harbors and Coastal Engineering Division, ASCE; Vol. 93, No. WW3; August 1967; pp. 71-87.

Garde, R.J., Subramanya, K. and Nambudripad, K.D.; "Study of Scour Around Spur Dikes"; Journal of the Hydraulics Division, ASCE; Vol. 87, No. HY6; November 1961; pp. 23-37.

Gill, Mohammad A.; "Erosion of Sand Beds Around Spur Dikes"; Journal of the Hydraulics Division, ASCE; Vol. 98, No. HY9; September 1972; pp. 1587-1602.

Haas, Raymon H. and Weller, Harvell E.; "Bank Stabilization by Revetments and Dikes"; Transactions, ASCE; Vol. 118, 1953; pp. 849-870.

Hall, James D. and Baker, Calvin O.; Rehabilitating and Enhancing Stream Habitat: 1. Review and Evaluation; General Technical Report PNW-138; USDA Forest Service; Pacific Northwest Forest and Range Experiment Station; Portland, Oregon; 1982; 29 pp.

Kehe, Scott M.; Streambank Protection by Use of Spur Dikes; M.S. Project Report; Civil Engineering Dept., Oregon State University, Corvallis; 1984; 78 pp.

Laursen, E.M.; "Observations on the Nature of Scour"; Proceedings of the Fifth Hydraulic Conference; Bulletin 34; University of Iowa; Iowa City, Iowa; June 9-11, 1952; pp. 179-197.

Laursen, Emmett M., "Scour at Bridge Crossings," Journal of the Hydraulics Division, ASCE; Vol. 86, No. HY2; February 1960; pp. 39-54.

Leopold, L.B., Wolman, M.G. and Miller, J.P.; Fluvial Processes in Geomorphology; Freeman and Co.; San Fancisco; 1964; pp. 295-308.

Lindner, C.P.; "Channel Improvement and Stabilization Measures"; State of Knowledge of Channel Stabilization in Major Alluvial Rivers; Technical Report No. 7; G.B. Fenwick, ed.; U.S. Army Corps of Engineers, Committee on Channel Stabilization; 1969.

Liu, H.K.; Chang, F.M. and Skinner, M.M.; Effect of Bridge Constrictions on Scour and Backwater; Section Report CER 60HKL22; Colorado State University, Fort Collins; February 1961.

Maccaferri Gabions of America; The Use of Gabions in the Construction of Weirs; Technical Booklet No. 9; undated-a; pp. 1-25.

Maccaferri Gabions of America; Gabions for Flexible River Groynes; Technical Booklet No. 10; undated-b; pp 3-14.

Maccaferri Gabions of Canada; Maccaferri Gabions Stream Improvement Handbook; Pub. No. GT-204; undated; pp. 1-11.

Mamak, Witkor; River Regulation; Arkady, Poland; 1964; 415 pp.

Mathes, Gerald H.; "River Engineering", Section 15 in 3-volume set, American Civil Engineering Practice; R.W. Abbett, ed.; J. Wiley and Sons; 1956; 181 pp..

Maughan, O.E., et al.; Evaluation of Stream Improvement Practices in Southeastern Trout Streams; Virginia Water Resources Research Center Bulletin 115; Virginia Polytechnic Institute and State University, Blacksburg; 1978.

Mukhamedov, A.M., Abduraupov, R.R., Irmukhamedov, Kh.A., and Urbinbaev, R.; "Study of Local Scour and Kinematic Structure of Flow Around Solid and Through Spur Dikes"; Proceedings I.A.H.R. 14th Congress; Vol. 3; Paris; 1971; pp. 389-396.

Owusu, Yaw A.; Gabion Structures for Streambank Erosion Control, Bed Scour and Fish Habitat Modification; M.S. Project Report; Civil Engineering Dept., Oregon State University, Corvallis; 1984; 101 pp.

Parsons, D.A.; "Effects of Flood Flow on Channel Boundaries"; Journal of the Hydraulics Division, ASCE; Vol. 86, No. HY4; April, 1960; pp. 21-34.

Rajaratnam, N. and Beltaos, S.; "Erosion by Impinging Circular Turbulent Jets"; Journal of the Hydraulics Division, ASCE; Vol. 103, No. HY 10; October 1977; pp. 1191-1205.

Rajaratnam, N. and Nwachukwu, B.A.; "Flow Near Groin-Like Structures;" Journal of Hydraulic Engineering, ASCE; Vol. 109, No. 3; March, 1983; pp. 463-480.

Raudkivi, A.J. and Ettema, R.; "Clear Water Scour at Cylindrical Piers;" Journal of Hydraulic Engineering, ASCE; Vol. 109, No. 3; March, 1983; pp. 338-350.

Reeves, Gordon H. and Roelofs, Terry D.; Rehabilitating and Enhancing Stream Habitat: 2. Field Applications; USDA Forest Service; Pacific Northwest Forest and Range Experiment Station; Portland, Oregon; 1982; 38 pp.

Rouse, Hunter; "Criteria for Similarity in the Transportation of Sediment", Hydraulics Conference Proceedings; Studies in Engineering, Bulletin 20; University of Iowa; 1939; pp. 33-49.

Samide, G.W. and Beckstead, G.; Design Considerations for Stream Groynes; Alberta Department of the Environment, Environmental Engineering Support Services, Technical Services Division; October 1975; 65 pp.

Schoklitsch, A.; Prevention of Scour and Energy Dissipation; Translated by Edward F. Wilsey, Bureau of Reclamation; Denver; 1937; 86 pages. (Schoklitsch,A.; Kolkabwehr und Stauraumberlandung; Berlin: Julius Springer; 1935; pp. 17-183.)

Soil and Water Conservation Commission; Streambank Erosion in Oregon; State of Oregon; Salem; 1973.

Stephenson D.;"Hydraulics of Gabions and Rockfill"; Atti del 16 Congresso di Idraulica e Costruzioni, Idrauliche; Torino; 1978.

Strom, H.G.; River Improvement and Drainage in Australia and New Zealand; State Rivers and Water Supply Commission; Melbourne; 1962; 378 pp.

Thomas, R.K.; Scour at the Base of Free Overfall; Master's Thesis; Colorado A & M College; 1953.

Tiefenbrum, A.J.; "Bank Stabilization of the Mississippi River Between the Ohio and Missouri Rivers"; Proceedings; Federal Inter-Agency Sedimentation Conference; U.S. Department of Agriculture; Agr. Research Service, Misc. Pub. No. 970; Paper No. 45; January 1963; pp. 387-399.

Tison, G. Jr.; "Discussion of Scour Around Spur Dikes"; Journal of the Hydraulics Division, ASCE; Vol. 88, No. HY4; July 1962; pp. 301-306.

United Nations; River Training and Bank Protection; Publication No. 1953 II F-6, Flood Control Series No. 4; Bangkok; 1953.

U.S. Army Corps of Engineers; Final Report to Congress, The Streambank Erosion Control Evaluation and Demonstration Act of 1974, Section 32, Public Law 93-251; Main Report and Appendix A; December 1981.

U.S. Fish and Wildlife Service; Western Reservoir and Stream Habitat Improvements Handbook; FSH/OBS-78/56; U.S. Department of the Interior; Washington, D.C.; 1978.

Van Ornum, J.L.; The Regulation of Rivers; McGraw Hill; 1914; pp. 214-219.

Vanoni, V.A.; ed., Sedimentation Engineering; ASCE Manual on Engineering Practice, No. 54; New York; 1975; pp. 44-65.

Varshney, R.S.; "An Analysis of Location and Orientation of Repelling Spurs in Rivers"; Irrigation and Power; Vol. 29, No. 1; January 1972; pp. 59-76.

APPENDIX. LIST OF SYMBOLS

a = width of scour hole created by V-shaped weir

A_j = cross-sectional area of jet flow

b = length of scour hole created by V-shaped weir

B = average width of approach channel

B_1 = original channel width (= B)

B_2 = constricted channel width or average width of contracted channel

C = sediment concentration by weight

C_D = drag coefficient

d_1 = measured scour depth at tip of upstream groin

d_2 = measured scour depth at tip of downstream groin

d_s = limiting depth of scour below original bed level

d_T = depth of scour below the original bed level at any particular time, T

D_{50} = median grain size of bed sediment

D_{90} = sediment size such that 90 percent is smaller

f = Lacey silt factor

F = Froude number = $\frac{V}{\sqrt{gh}}$

F_1 = Froude number upstream of weir

F_2 = Froude number downstream of weir

F_{bo} = Blench's "zero bed factor" = function of grain size

g = acceleration due to gravity

h = average depth of flow in unconstricted approach channel

hm = maximum depth of approach flow

hw = height of weir above the original bed level

H = height of drop of bed level from upstream to downstream

H' = height of drop of water surface from upstream to downstream

k = function of approach conditions; k varies with investigator

K = function of C_D and varies between 2.75 and 5.0

K_β = factor accounting for effects on scour of varying dike head slope

L = actual length of spur dike or groin

Le = effective length of spur dike or groin measured normal to the bank from the base to the tip of the structure

Lp = width of undisturbed bed zone between two groins

L_1 = distance from downstream side of V-weir apex to V-weir base

m = location of point of maximum scour depth, measured from downstream side of weir apex

M = contraction ratio = $(B_1 - B_2)/B_1$

n = function of C_D and varies between 0.65 and 0.9

N = dimensionless term for bed roughness

N_{ns} = term N applied to approach channel or dike site

$N_{ns}*$ = term N applied to approach channel or dike site at beginning of scouring motion

q = stream discharge per unit width at constricted section (use flood conditions to find maximum scour depth)

q_w = discharge per unit width of crest of weir or drop structure

Q = total stream discharge

r = assumed multiple for scour at dike compared with scour in a long contraction (taken to be 11.5 by Laursen)

SVI = scour volume index = $a \cdot b \cdot d_s$

T = time

V = average velocity in unconstricted approach channel

Vm = maximum velocity of approach flow

V_1 = water velocity upstream of weir

V_2 = water velocity downstream of weir

V_j = velocity of efflux of jet flow

W = width of uncontracted channel (= B)

X = distance between two groins of spur dikes

X_1 = distance from dike base to downstream bank point where erosion begins

X_2 = distance from dike base to downstream bank point where main current impinges

X_3 = like X_1 but measured from dike tip

X_4 = like X_2 but measured from dike tip

y = average depth of flow in unconstricted section

y_i = tailwater depth at pool over scour hole, measured from original bed level

y_s = equilibrium scour depth measured from the water surface to the bottom of scour hole

y_1 = water depth upstream of weir

y_2 = water depth downstream of weir

Z_1 = settlement of tip of upstream gabion groin

Z_2 = settlement at tip of downstream gabion groin

α = apex angle of V-shaped weir

β = angle between side slope of dike and vertical plane

θ = dike or groin orientation angle between axis of structure and downstream bank (or channel thalweg)

ρ_s = density of bed sediment (mass per unit volume)

ρ_w = density of water (mass per unit volume)

σ_D = term describing the size gradation of the bed material

σ_ω = standard deviation of the sediment settling velocity

μ = absolute viscosity of water

ω = settling velocity of sediment

$\xi_{85\%}$ = ratio of D_{85} to D_{50} for bed sediment

τ_c = critical bed shear stress

τ_{ns} = bed shear stress in approach channel or dike site

τ_{ns*} = bed shear stress in approach channel or dike site at beginning of scouring motion