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METHODOLOGY FOR IN-STREAM REHABILITATION OF A SILTED STREAM

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ABSTRACT

The purpose of this project was to develop methodology for rehabilitating streams where the fishery and other recreational uses have been destroyed or greatly reduced by heavy silting. Using Emerald Creek in northern Idaho as a study stream two types of in-stream modification structures, flow constrictors and drop structures, were designed, constructed and evaluated. Contour maps and thalweg profiles of the before and after modification conditions are presented for each of the five stream modification sites.

Three drop structures were constructed at two sites on the upper reach of Emerald Creek. These structures concentrate the energy of the flow and scour a hole downstream of the structure. Considerations in the design of drop structures include forces on the structure, backwater caused by the structure, and depth of scour below the structure. In general the log drop structures performed favorably. Scour holes formed downstream of two of the drop structures to a depth of 1.0 ft and 1.5 ft as compared to a predicted scour depth of 2.0 ft and 3.0 ft respectively. The scour holes were lined with rock from six to ten inches in diameter.

The flow constricting structures were designed to reduce the width and increase the depth of flow during low flow periods to increase the sediment transport capacity of the stream thus flushing the silts and fine sands from the modified

reach leaving cobble and small boulders. Gabion deflectors were used at two sites on the lower reach of Emerald Creek to constrict the flow. Log dikes were used at one site to constrict the flow and train the stream to follow a meandering flow path. Design considerations for flow constrictors include forces on the structure, location of structure, meander dimensions, and constriction dimensions. The performance of the gabion constrictors was acceptable at one site. Silts and sands were removed from the reach leaving cobble and boulders. Performance at the other gabion constrictor site was fair. Small amounts of silt and sand were removed from the reach. Cobble and small boulders moved into the constricted flow area from an upstream riffle.

The log dikes used to form a meandering flow pattern during low flow appeared to be functioning as planned but were removed by high flows before analysis of the modification was completed.

A method for measuring the streambed hydraulic conductivity using standpipes is suggested as a means for determining the change in bed characteristics resulting from deposition of silt. Calibration curves for the standpipes were developed in the laboratory for future testing in the field.

INTRODUCTION

Prolonged and increased use of natural resources in America's mountain areas can have a detrimental effect on mountain streams. Some of the more common activities which lead to deterioration of a mountain stream are construction of roads, logging, mining and livestock grazing. These activities can result in the removal of vegetation and exposure of erosive soils. Removal of vegetation decreases the retention time and infiltration rate of water from rainfall and snowmelt which, in turn, increases the overland flow of water and consequently the erosion of soil. The net result is an increase in sediment delivered to the stream.

Excessive sediments or sediments not characteristic of the stream in its pristine condition can result in a direct change in aquatic insect and plant life which serve as food for fish. Discolored water, silted stream beds and slower moving water resulting from deposition of excess sediment lead to a decrease in esthetic values.

In view of increasing emphasis on wildlife and recreational resources in major Idaho watersheds, more information is needed to implement effective fresh water management. Information is needed on the sediment carrying capacity of streams, methods for rehabilitation of silted streams and the effect of the sediments on characteristics and aquatic life of the stream.

Emerald Creek, a tributary to the St. Maries River in northern Idaho, is an example of a silt-polluted stream. The purpose of this project was to develop design criteria for rehabilitation of silt-polluted streams using Emerald Creek as a study stream. The engineering phase of this interdisciplinary study was involved with analyses of stream channel training structures and streambed hydraulic conductivity.

Gabions, log dikes, and drop structures were studied as possible improvement measures for either flushing excess sediment from selected reaches or creating pools. Each structure was designed and its effects on the stream were predicted. The structures were then constructed and their effects on the stream analyzed and compared with the predicted results.

A method for measuring the streambed hydraulic conductivity using standpipes was tested as a means for determining changes in bed character due to silt deposition. Calibration curves for the standpipes were developed in the laboratory. However, bed conditions at improved sites on Emerald Creek were not suitable for field testing standpipes and for this reason field testing was deferred.

HYDRAULIC CONDUCTIVITY AS A MEASUREMENT OF SILTATION

Hydraulic conductivity of a streambed is an indicator of the rate of flow through the interstices of a unit volume of bed material. The rate of flow increases with increased pressure head but is limited by the area of the openings between the particles in a unit cross-sectional area of material. Particles of a uniform size will provide the largest space between the particles and well-graded material will provide minimum space between the particles.

Introduction of sediment sizes smaller than the existing sediment in a reach will result in filling of some of the interstices by the smaller sediment sizes, ultimately causing a reduction in flow in the clogged area. This can be detected by measuring the hydraulic conductivity of the material.

The supply of oxygen to fish eggs within the gravel material of the streambed is dependent on the movement of water through the gravels. A decrease in the hydraulic conductivity of the material resulting from the clogging of pore spaces with sediment means a decrease in the supply of oxygen to the fish eggs and aquatic life within the streambed material. For this reason increased siltation of spawning gravels has a detrimental effect on the fish population.

Deposition of smaller particles near cobble and on the gravel of the streambed fills spaces that serve as places of escape for aquatic insects. Reduction of hiding spaces for insects leads to a reduction in the number of insects in certain species and a reduction in the diversity of species. The reader is referred to

Luedtke (1973) for information concerning aquatic insects and the embeddedness of cobble.

There is a need for a method for measurement of the hydrualic conductivity of streambeds to be used as an indicator of siltation of the bed. By knowing the degree of silatation of a stream and its effect on aquatic life, adverse practices such as logging and dredge mining could be reduced or controlled at a specified level to bring the degree of siltation to an acceptable level.

One method of measurement of the hydraulic conductivity of a streambed is the use of standpipes imbedded in the streambed as dicussed by Terhune (1958). The hydraulic conductivity of the streambed is deduced by measuring the flow rate into a perforated standpipe, driven into a streambed, when a head differential of one inch is created between the water surface in the stream and the water level within the standpipe. The head differential is created by continuously pumping water from the standpipe. The reference cited above describes the methodology in detail. Figure 1 shows the specifications for a standpipe. The purpose of this chapter is to explain the method of using standpipes to measure hydraulic conductivity and to develop a calibration curve for the standpipes.

Calibration of Standpipes

As a part of the Emerald Creek project, a calibration curve was developed for the standpipes. This curve can be compared to the calibration curve developed in a similar manner by Terhune. To develop this calibration curve the hydraulic conductivities of four different sand-gravel mixtures were measured using a falling head permeameter. These conductivities were then plotted against the discharges obtained from standpipes set in respective sand-gravel mixtures with a 1-in head difference between the water surface inside and outside the standpipe. A pump was connected to the intake pipe and continuous pumping kept the water surface at the proper level.

A falling head permeameter was used for measuring the hydraulic conductivity of a saturated sediment sample. The permeameter consisted of a plexiglass tube with wire screening at the bottom to keep the sample in place. A trap door valve was used to start and stop the flow of water. The head, in feet of water, was measured from the bottom of the sample to the water surface at the beginning and end of each test.

A gravel-filled flume was used for simulating field conditions for testing the standpipes. The flume was 25 feet long by 3 feet wide. A 3-foot section in the middle of the flume was used as a test section. This section was screened off so that only the volume of material within this section was changed when different gradations of material were tested.

Procedure

Mixing of sand and gravel. Varying amounts of sand and gravel were mixed to obtain materials with different hydraulic conductivities. Using this system, four mixes ranging from 100 percent gravel to 50 percent gravel were obtained. The same sand and gravel mixtures were used for both the direct measurement of hydraulic conductivity, using the permeameter, and the indirect measurement of hydraulic conductivity, using the standpipes. The two materials were mixed according to percent of the total weight of the mixture. (See Table 1 and Appendix C for size distribution of mixtures.)

Direct measurement of the hydraulic conductivity. Using the permeameter shown in Figure 2, a sand-gravel mixture was placed in the bottom part of the permeameter. The mixture was not packed or compressed. The permeameter was filled with water to a depth H_1 after the sample had been saturated. At time $t = 0$, the flow was started. Flow was stopped at time $t = \Delta t$ and the depth H_2 was measured. Using the standard falling head permeameter equation,

$$K = \frac{L}{\Delta t} \ln \frac{H_1}{H_2} \tag{1}$$

the values for hydraulic conductivity were determined. In equation (1) H_1 is the initial water depth in the permeameter, H_2 is the final water depth, L is the thickness of the sample layer, and Δt is the time interval between depth equal to H_1 and depth equal to H_2.

Data for the direct measurement of the hydraulic conductivity are shown in Table 2.

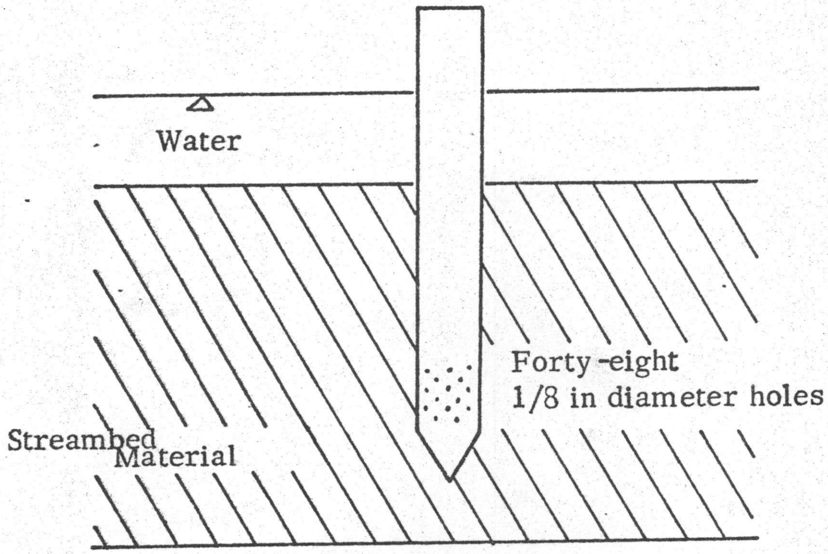


Fig. 2: Groundwater standpipe.

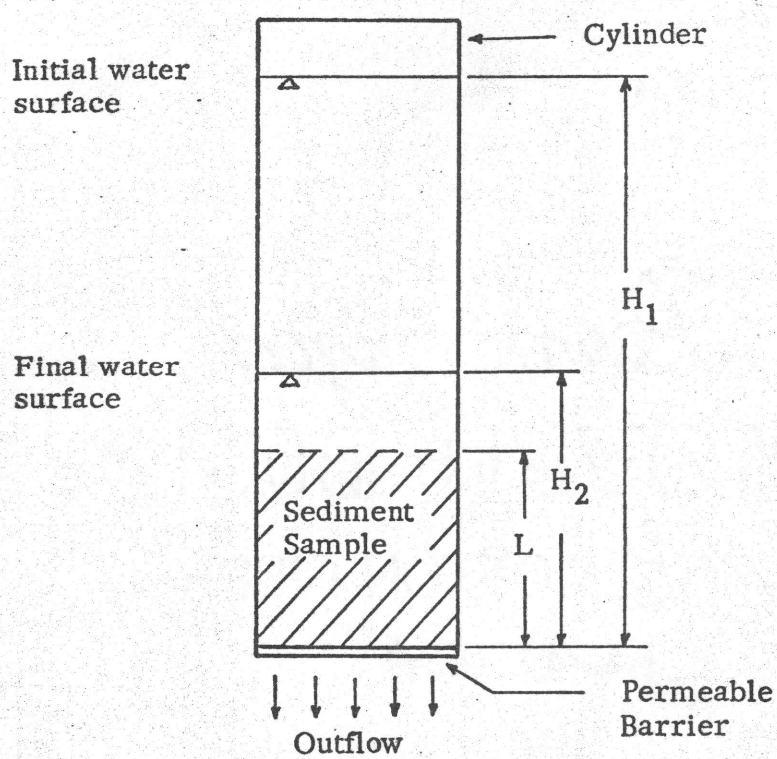


Fig. 2: Falling head permeameter.

Table 1: Size distributions for sand-gravel mixes.

Sieve Size in	Mean Size in	Per Cent by Weight in Sieve Fraction, Run Number 1	2	3	4
0.004					
	0.0055	0	8	9	14
0.007					
	0.0077	0	9	20	29
0.0084					
	0.0117	0	3	5	7
0.015					
	0.0475	18	15	13	10
0.08					
	0.105	21	16	14	12
0.13					
	0.153	19	14	16	16
0.175					
	0.223	20	19	9	7
0.27					
	0.455	22	16	14	5
0.64					

Table 2: Data for hydraulic conductivity.

Run No.	Gravel %	Sand %	H_1 ft	H_2 ft	L ft	T av. sec	K cm/sec	Q av. ml/sec
1	100	0	3.08	1.22	.764	8.69	2.47	94.2
2	79.8	20.2	2.98	1.04	.9	65.8	0.439	52.4
3	66.7	33.3	2.85	1.02	.4	103.6	0.1209	17.1
4	50	50	2.86	1.93	.35	296.4	0.0142	3.37

Indirect measurement of hydraulic conductivity. Three standpipes were imbedded to a depth of 10 in in the sand-gravel mixes in the test section of the flume. Flow through the flume was adjusted to a depth just above the sand-gravel material. The end of the intake tube of the pump was placed inside the standpipe at an elevation one inch below the water surface. When the water within the pipe was evacuated to this level, one-inch head differential existed. This provided the driving force for flow into the standpipe during the pumping operation. The method Terhune suggested for setting the intake tube in proper position was to lower the intake tube while pumping until a "slurping" sound could be heard, which would mean the intake was at the water surface. Then the intake would be lowered one inch. This method is difficult if there is any interfering noise. The method used for this test was to slowly lower the intake tube, while pumping, until water was ejected from the pump. At this point the intake was assumed to be at the water surface. The intake was then lowered one inch.

The small electric pump was started and allowed to run at least 30 seconds to drain the initial one inch of water in the standpipe and to insure that the flow had stabilized. Then the discharge Q was measured by collecting a volume $\forall$ of water over a time interval t. The discharge Q was computed using:

$$Q = \frac{\forall}{t} . \qquad (2)$$

Because discharge through the standpipe is directly proportional to the flow through the surrounding sand-gravel materials, it is an indirect measurement of the hydraulic conductivity of the material. Data for the indirect measurement of the hydraulic conductivity are summarized in Table 2.

Results

Results for the direct and indirect measurements of the hydraulic conductivity are summarized in Table 2. For each sand-gravel mixture the value of K for the direct measurement was plotted against the value of Q for the indirect measurement. This calibration curve and the curve developed by Terhune are shown in Figure 3.

Discussion

The two curves in Figure 3 are not in agreement. Some of the difference can be attributed to slippage of water down the outside of the pipe from above the gravel surface. This flow was reduced by Terhune by using a disk collar around the standpipe at the gravel surface. The remainder of the difference would have to be attributed to differences in equipment and procedures.

The lower end of the solid curve in Figure 3 shows the most pronounced effect of slippage. At a hydraulic conductivity less than 0.1 cm/sec or a discharge less than 15 ml/sec, the amount of water finding its way into the standpipes by slipping down the side becomes a substantial part of the total amount of water entering the standpipes. At high values of Q, greater than 52 ml/sec, discharge

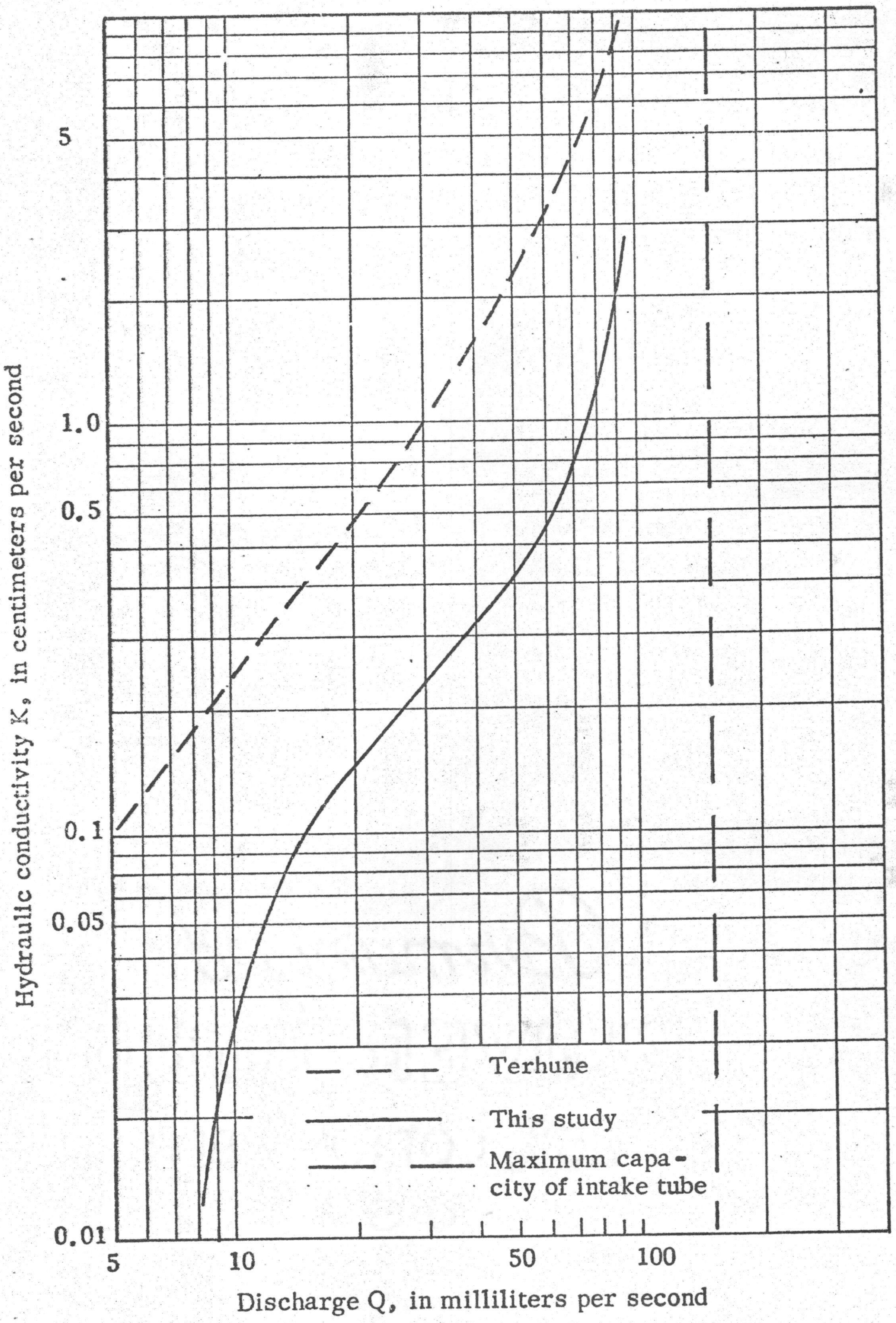


Fig. 3: Calibration curve for standpipe.

is affected by the maximum intake tube capacity which, for the intake tube used, was 156 ml/sec.

A few points are mentioned to help with field application. As many standpipes as possible should be used in an area to obtain a representative value of hydraulic conductivity for the area. The hydraulic conductivity can vary greatly within a small area.

To reduce the slippage of water down the outside of the pipe during testing, the standpipe should be allowed to sit for at least three days after driving before testing. This allows smaller particles to collect near the pipe to help seal against the slippage. Also, it would help to use a disk collar, as described by Terhune, which would fit around the standpipe and sit firmly against the bed material to reduce slippage.

If the standpipes are installed in the streambed for any length of time (a week or more), algae growth in and near the holes in the standpipe could alter the inflow rate into the standpipes. The pipes can be treated with a copper sulfate solution to control algae growth.

It is important that the standpipes be removed before flood flows or ice forms. Either event could remove or damage the standpipes.

STUDY AREA

The field study phase of this project was conducted in the East Fork and the main stem of Emerald Creek, a tributary to the St. Maries River in northern Idaho (Fig. 4). Much of the main stem of Emerald Creek suffers from a high sediment concentration resulting from commercial and private mining of garnets and garnet sand.

The East Fork of Emerald Creek originates at an elevation of 4000 ft in the Hoodoo Mountains in Latah County. It flows northeast until it enters a broad valley at its confluence with the West Fork. The main stem flows into the St. Maries River approximately five miles northwest of Clarkia, Idaho (Luedtke, 1973). The total drainage area is 36 square miles and the mean elevation of this area is 3000 ft.

Emerald Creek, a low-gradient stream, drops approximately 220 feet in the 10-mile section involved in this study. Its width varies from 11-35 ft and the average riffle depth is 2-6 in with pools 2-4 ft deep during midsummer. During the summer months the current velocity ranges from less than 1.0 to 2.3 ft/sec and the average discharge for the main stem is approximately 16 ft^3/sec. Geologically, the East Fork is in the Pre-Cambrian belt series. The stream grades into Columbia River basalt below the confluence of the East and West Forks (Prather, 1971).

Activities taking place within the Emerald Creek drainage include logging, mining, summer livestock grazing, and recreation in the forms of rockhounding,

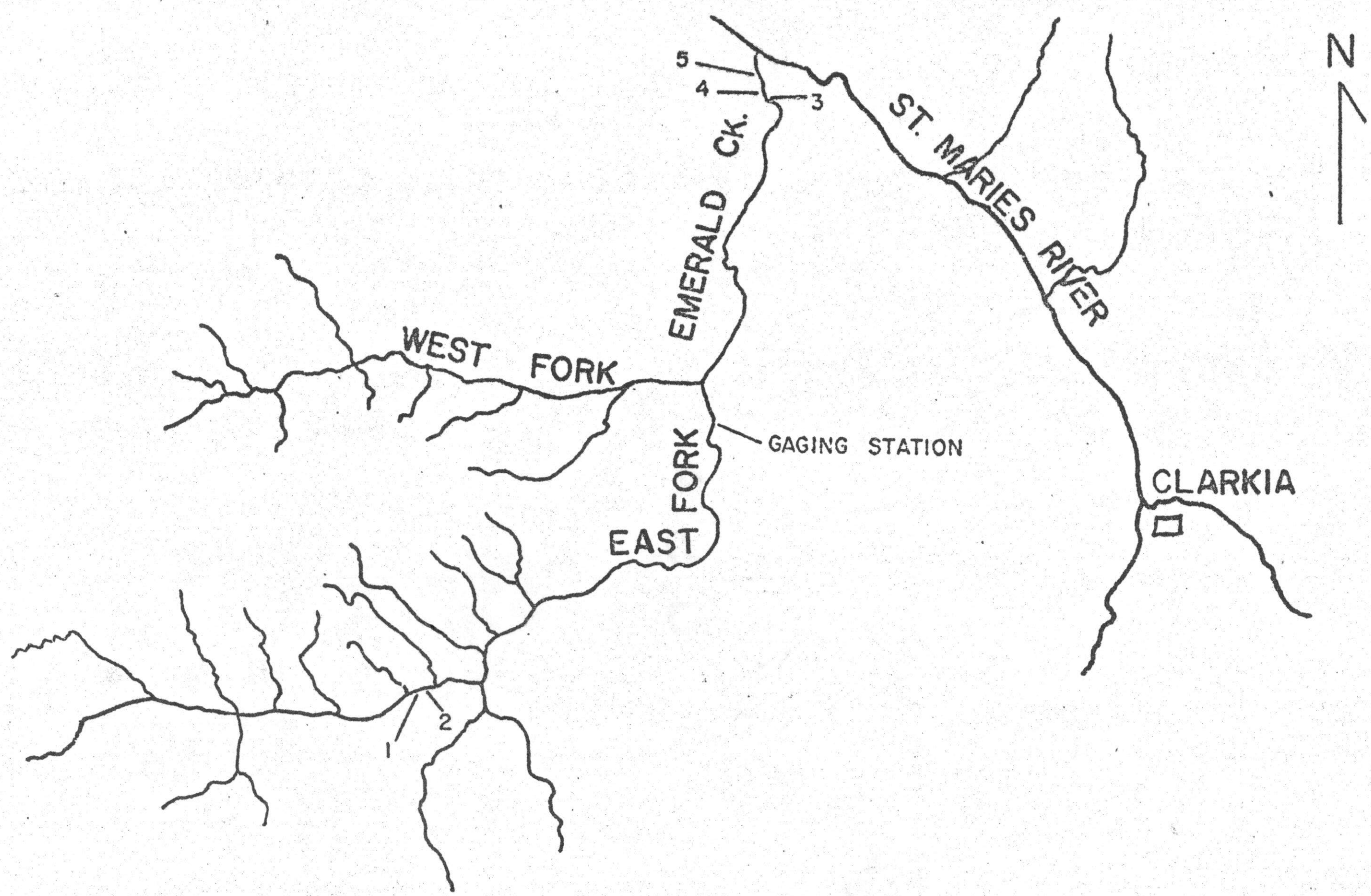


Fig. 4: Emerald Creek and modification sites as identified by number.

fishing and hunting. Until 1969 the East Fork and its tributaries were subjected to heavy use by rockhounds digging for garnets in the bed and banks of the stream with most of the gravel washed and screened in the streams. By trading land with the Sunshine Mining Company, Milwaukee Land Company, and Potlatch Forests, Inc., the Forest Service acquired 930 acres along the East Fork during the winter of 1968-69. The Forest Service then limited digging for garnets to a 40-acre area which is leased to a private concern. Emerald Creek has been diverted around this garnet removal area.

A major source of sediment in the main stem of Emerald Creek can be attributed to the Sunshine Mining Company's dredge site and screening plant located immediately downstream from the confluence of the East and West Forks. Sediment from this 30-acre area is the direct result of soil removal in or near the streambed and runoff from the surrounding nonvegetated soil.

The main stem and East Fork of Emerald Creek were divided into three reaches for this study. The lower reach ran from the St. Maries River to the Sunshine Mining Company's dredging area. This low-gradient reach is characterized by alternating pools and riffles. Bed material in the riffle areas ranges from cobble to small boulders. Some segments of the streambed in the pool areas are covered with one to two feet of sand ($d_{50} = 0.7$ mm) sometimes extending several hundred feet. Low profile gabion and dike constricting structures were constructed at three sites in this reach. Gabion structures were designed to concentrate the low summer flows at two sites to provide sufficient tractive force to keep the sand moving through the constricted reach leaving behind the cobble material. At a third site, log dikes were designed to constrict the flow and form a meandering flow pattern during low summer flows. Constricting the flow increases the local flow velocity and sediment transport capacity. Both the gabions and dikes were designed as low-profile structures to have a minimum effect during high winter and spring flows.

The middle reach of Emerald Creek from the dredging operation upstream to the 40-acre garnet removal area was in better condition than the lower reach. Bed material ranged from coarse sand to small cobble with most of the material in the coarse gravel to cobble range. There was an acceptable pool to riffle ratio in this reach.

The reach of Emerald Creek upstream of the garnet removal area was a typical fast-moving mountain stream with the surface bed material consisting of coarse gravel. The major problem on this reach was the lack of pools for fish. Three log drop structures were constructed at two sites to create scour holes during the high flow events.

In October, 1971, the Forest Service installed a gaging station on the East Fork just upstream of the confluence of the East and West Forks. The stream flow hydrograph for the water year October 1, 1971, to September 30, 1972, is presented in Figure 5. During this period a maximum high flow of

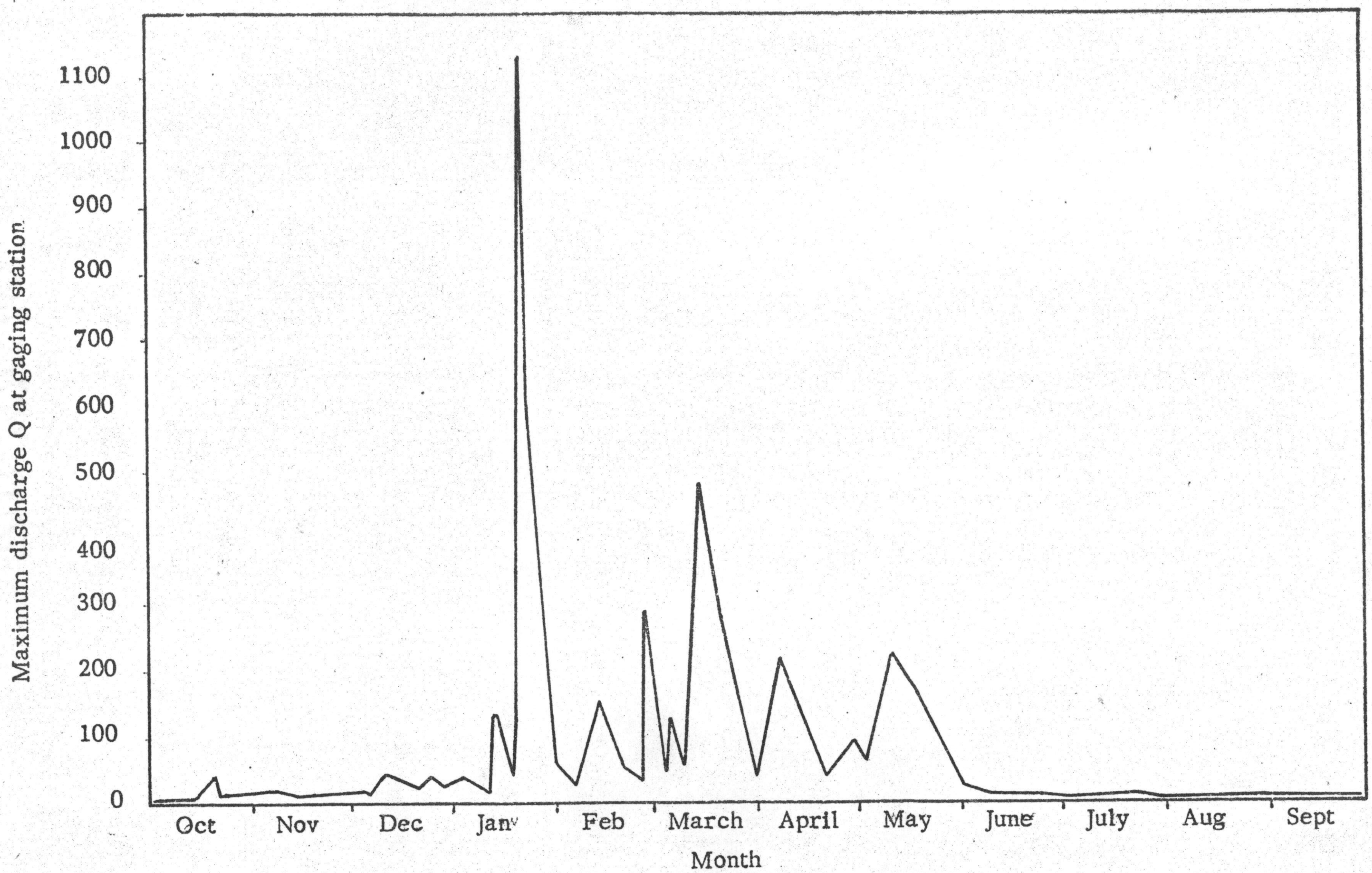


Fig. 5: Emerald Creek maximum stream flow hydrograph for the water year October 1, 1971, to September 30, 1972.

1133 cfs was recorded on January 21, 1972. The high flow for the next two years appeared to occur in January. In January, 1973, the flow was four feet deep over the top of the gabions in the lower reach of Emerald Creek. The flow in this reach was estimated to be in the range of 1500 to 2000 cfs and to correspond to the 50-year flood. The high flow during January, 1974, at the Emerald Creek gaging station was estimated to be in the range of 2500 to 3000 cfs and at the mouth of Emerald Creek to be from 4000 to 5000 cfs. This flow which corresponds to the 200 or 300 year event caused significant damage to some of the stream modification structures.

Observations of the ice conditions on Emerald Creek during the winter of 1972-73 revealed ice thickness ranging from 6 to 12 inches. Sediment particles were noticed in some of the blocks of ice cut from the stream. Also, a wave pattern discussed by Carey (1966) was observed on the underside of the ice. Plate 1 shows a sample of ice cut from Emerald Creek.

It was observed that flood flows associated with the sudden release of water temporarily stored behind ice dams play an important role in the shaping of a stream bed.

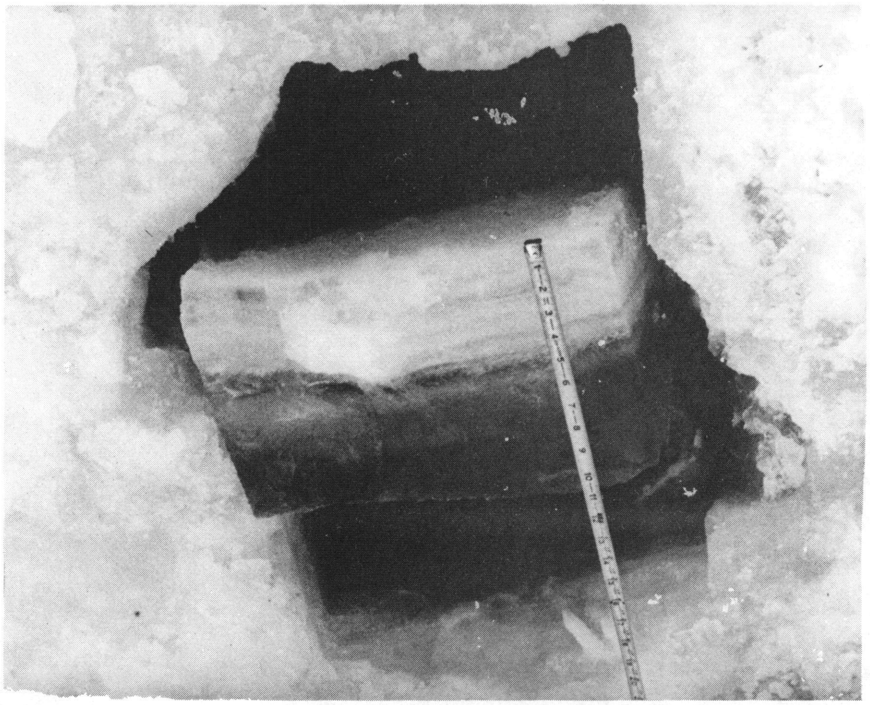


Plate 1. Ice at Emerald Creek during the winter of 1972-73.

STREAM MODIFICATION STRUCTURES

The first step in a stream channel improvement program is an on-site inspection of the stream and the preparation of an inventory of troublesome reaches. For each modification site topographic details and an adequate description of stream bed and bank material must be obtained. A frequency curve for annual floods and for low water discharges must be available. If flow records are not available, this information must be generated by some acceptable hydrologic technique appropriate for the basin and for the anticipated investment in modification structures.

After obtaining the necessary information three basic questions must be answered: (1) what is the problem in the troublesome reach; (2) what type of structure is required to relieve or alleviate the problem; and (3) what size of runoff event should the structure be designed to withstand?

One hundred year flow events for a stream the size of Emerald Creek may be four to six times the average annual flood event. Economics will not permit construction of a structure for the maximum anticipated runoff event. Depending on the type of structure and the time period the structure is supposed to function in, the design flood selected may vary from structure to structure.

When designing small training structures it must be realized that the structure will be rather trivial in the overall hydro-geomorphic evolution of the channel. Selection of a design flood must be based on the premise that the structure is temporary and that it is specifically designed to obtain a particular change in the channel adjacent to the structure.

Structures constructed in Emerald Creek were designed to withstand a 25-year flood which is approximately 300 cfs at the log drop structure sites and 1400 cfs at the channel constrictor sites.

Three basic types of stream modification structures were constructed. Log drop structures and two types of channel constrictors, rock-filled gabions and log dikes were used at different locations according to the stream channel condition and the desired results.

Channel Constriction Structures

Low-profile gabion constrictors were constructed at Site 3 (Fig. 13) and Site 5 (Fig. 16). Site 3 was located downstream of a riffle area in a low-gradient reach.

The bed material consisted of sand ($d_{50} = 0.7$ mm), 1 to 2 feet deep covering cobble and small boulders. Two opposing gabions were designed and built to constrict the channel during low flow periods ($Q < 20$ cfs) and increase the sediment carrying capacity, thus removing the sand cover and exposing the cobble and small boulders.

Site 5 was also located in a low-gradient reach with sand overlying cobble and boulders. At this site two parallel gabions were constructed on the same side of the channel to constrict the flow between them and the far bank during low flow periods.

Site 4 (Fig. 20) was located just downstream of Site 3 in the same sandy reach. At Site 4 log dikes were installed by anchoring logs to the bottom of the channel. The dikes were placed to constrict the flow and produce a meander flow pattern during low discharge periods ($Q < 20$ cfs). The intention at the site was not to scour down to the cobble but to form a channel through the sand that would become armored with 2 to 4-inch material.

The structures at Sites 3, 4, and 5 were designed and constructed as low-profile structures to constrict the flow during the low summer discharges but have minimal effect during high discharge periods.

Design of Channel Constricting Structures

Certain factors must be considered when designing stream modification structures. These factors include the force of the flow and ice on the structures, minimization of backwater, and location and dimensions of the structures.

<u>Resistance to flow and ice forces</u>. The flow of water and the formation of ice create forces which must be resisted by the log or gabion and its supporting elements. The force of the moving water on a structure per unit length can be determined using the concept of momentum. As shown in Figure 6, the summation of forces in the direction of flow equals the change in momentum due to the resisting forces of the log or gabion.

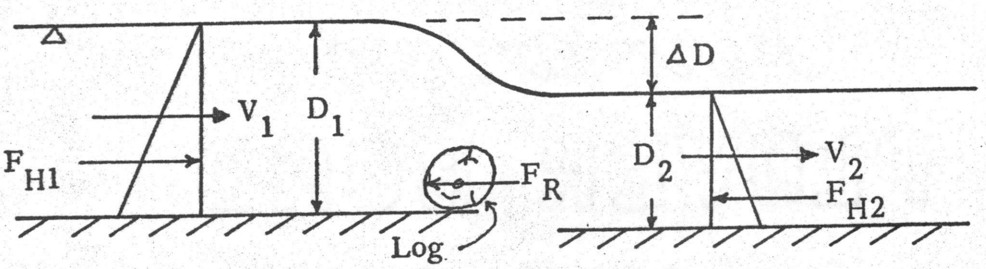


Fig. 6: Force diagram for log or gabion in-stream structure. The hydrostatic force F_{Hi} at each section (i) is equal to $1/2\,\gamma D_i^2$.

The original equation is:

$$F_{H1} - F_{H2} - F_R = \rho q (V_2 - V_1) \tag{3}$$

which, when solved for F_R is:

$$F_R = 1/2\,\gamma (2D_1 \Delta D - \Delta D^2) - \rho q^2 \left[\frac{1}{D_1 - \Delta D} - \frac{1}{D_1} \right] \tag{4}$$

F_R is the force in pounds per unit length the log or gabion and its supports must resist if the structure is to remain stable. The forces F_{H1} and F_{H2} are the forces due to hydrostatic pressure at Sections 1 and 2, D_1 is the depth at Section 1, ΔD is the difference between the depths at Sections 1 and 2, and V is the average velocity. For design an appropriate factor of safety should be applied to F_R. For the Emerald Creek structures the 25-year flood was used for design. A maximum water surface difference (ΔD) of 1 ft was assumed. For the 25-year flood at Sites 3, 4, and 5 the resulting force per unit width (F_R) was 91.6 lb/ft.

The log dike structures were supported at the bank by burying the log and in the stream by driving 1/2 in diameter steel pins alongside of the log into the sand and lower substrate.

Gabion constrictors were also buried in the bank but depended on their own weight to hold them in place in the stream. See Appendix A for the design of the gabions.

Constrictors were designed with a low-profile to avoid damage by moving ice. It was noticed that the ice had a tendency to freeze through and around the upper part of the gabions. The rising water level associated with a thaw would exert an uplift force on the gabion due to the bouyancy of the ice attached to the gabion. Water expands by approximately 10 percent when it freezes. Thus, the uplift of a piece of ice, 6 ft long and 1 ft thick, attached to the gabion would be 34 pounds per foot of gabion. The ice attached to the gabion not only adds to the surface area to be acted on by the force of the flow but it also decreases the resisting force of the gabion (friction) by reducing the effective weight of the structure.

Minimization of backwater. If the Froude number of the flow in a mountain stream is in the subcritical range, any structure placed in the stream will cause water to back up behind the structure. The backwater formed behind the structure will increase the depth of flow above that of the normal depth. The velocity of flow will decrease as a result of this depth increase and the sediment carrying capacity of the stream is reduced in the reach affected by the backwater. Therefore, when designing a stream modification structure, it is necessary to minimize the depth of backwater caused by the structure.

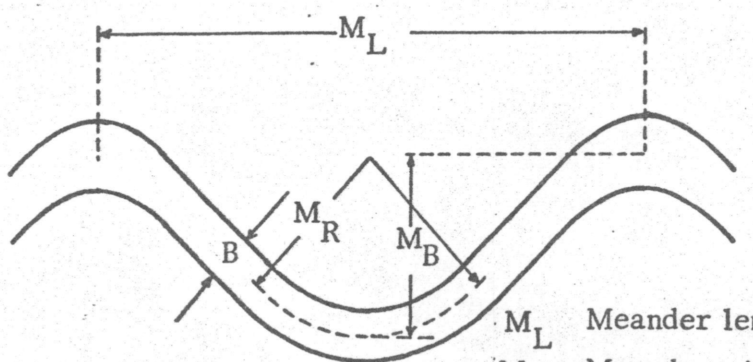
Backwater resulting from channel constrictions, such as gabion deflectors or log dikes, is not a problem during low flow. The effect on high flow depths upstream of the constriction can be estimated using the momentum and continuity equations. If the gabion has a low profile this effect will be minimal at high flow.

Placement of dikes to form meanders. Dikes constructed of logs or rocks can be used to change a relatively straight, shallow, slow velocity reach of a stream to a meandering reach with deeper and faster-moving water. By constricting the channel at low flow an increase in tractive force is achieved and the transport rate is higher. Meander-forming structures are designed so that silt will be transported through the reach during low flow periods and be deposited behind the structures when water levels are receding from peak flows. The stream power associated with high discharges is adequate to move the unwanted sediment without the help of the modification structures. The structures are designed to be overtopped during high flow and have a minimum impact on stream depth.

Literature on dikes and meanders presents some general rules on dike placement and geometry. After observing existing dike systems, Winkley (1971) stated that dikes should be spaced from 1 to 2 times the length of the next upstream dike. Franco (1967), after doing laboratory studies of dike systems, concluded that the most efficient system of dikes was one with the dikes perpendicular to the main flow direction, the crest of the dike sloping up from the water end to the bank end, and the crest of each dike lower than the dike upstream.

Meander dimensions. Meander patterns of a natural river or stream are the result of numerous factors such as geological conditions, slope of the streambed, amount of sediment in the water, and erodibility of the bed and banks of the stream. Virtually all unbraided streams have a meandering pattern and, according to Leopold and Wolman (1960), it is unusual for a reach of a natural stream to be straight for a distance exceeding 10 channel widths.

Modification of a natural stream channel should be accomplished in a manner which will not alter it greatly from the natural meander pattern. Natural meander patterns have been observed by Inglis (1947), Leopold and Wolman (1960), and Zeller (1967). These observers developed expressions that relate the meander length M_L (Fig. 7) to the water surface width B, the meander width M_B to the water surface width, and the meander length to the meander radius of cruvature M_R. The expressions are shown in Table 3.



M_L Meander length
M_B Meander width
M_R Meander curvature
B Channel width

Fig. 7: Definition sketch for meanders (reproduced from Graf, 1971).

The modifications to a natural channel should result in meander dimensions within the limits given by the expressions of Table 3. Modifications which do not result in meander dimensions within the limits of a natural stream may result in excessive scour or failure of the modification structures to control the flow as designed.

Table 3: Dimensions of natural channel meanders.

Meander Length M_L	Meander Width M_B	Radius of Curvature M_R	Reporter
$10.9\ B^{1.01}$	$2.75\ B^{1.1}$	$0.26 M_L^{1.02}$	Leopold & Wolman (1960)
$6.6\ B^{0.09}$	$18.6\ B^{0.99}$		Inglis (1947)
$10.0\ B^{1.025}$	$4.5\ B^{1.0}$		Zeller (1967)

Dimensions of Channel Constriction Openings

A stream's sediment transport capacity is proportional to slope and discharge conditions. Under a graded condition the sediment supplied to a reach equals the transport capacity of the stream thus no scour or deposition occurs. However, if sediment is supplied to the reach in an amount greater than its transport capacity, then the excess sediment will be deposited within the reach. Similarly, if sediment is supplied to the reach in an amount less than its transport capacity, sediment will be picked up or eroded and carried out of the reach. For natural conditions, such as for a mountain stream, the processes of erosion and deposition depend on the availability of sediment sizes which can satisfy the sediment transport capacity of the stream.

For a stream that has become heavily silted it may be necessary to increase the local sediment transport capacity of the stream to erode or flush the smaller-sized sediment out of the reach. The sediment transport capacity of a stream may be increased by placing structures within the channel which will reduce the flow width and increase the depth and velocity of the flow.

The purpose of this section is to present a method for designing the width and height of the opening of a structure to be placed in the stream channel which will flush the smaller sediment sizes out of the reach and create a more desirable particle size distribution for the streambed.

The Lane Diagram (Chow, 1959) has been used effectively in the past for the design of stable channels in noncohesive materials. The curves in the Lane Diagram have a factor of safety of 1.25. By dividing the values of particle diameter for a given value of tractive force by the factor of safety, a modified Lane Diagram can be developed which shows permissible tractive force values for given particle sizes with no factor of safety (Fig. 8). These tractive force values would be those for which the particles are just stable and a tractive force greater than the permissible tractive force would start some of the particles in motion.

Because the values of tractive force shown in Figure 8 are values at which some of the smaller particles of the bed are at incipient motion, the curves can be used for predicting the minimum tractive force required to insure erosion and transportation of a specified size fraction of sediment out of the reach. The net result of transporting the smaller particles out of a reach is to increase the d_{50} size of the streambed particles within that reach. Removal of some of the finer material will increase the size of the surface material. This will benefit the fish and aquatic insects by allowing more oxygen to be supplied to fish eggs and providing better escape regions and habitat for aquatic insects.

By converting the tractive force ($\tau = \gamma DS$) into its components of specific weight of water (γ), depth (D), and slope (S), the curves shown in Figure 8 can be converted to the curves shown in Figures 9, 10, 11, and 12 which have the variables of depth, slope and representative sediment size.

The curves can be used for designing the minimum depth of flow that would result in a tractive force which would flush some of the particles out of a reach and tend to develop the desired d_{50} or d_{75} for the streambed within the reach. The procedure for design is:

1. Select the desired sediment size distribution in terms of the d_{50} or d_{75}.
2. Determine the local slope of the reach.
3. Select the proper design curves among Figures 9, 10, 11, and 12 on the basis of particle size and content of fine sediment in the water.
4. On the proper curve, the design depth is determined by the intersection of the slope curve and the d_{50} or d_{75} size depending on the curve used.

It is important that the local slope of the reach near the design section be used instead of the slope of a longer reach as the two may differ greatly.

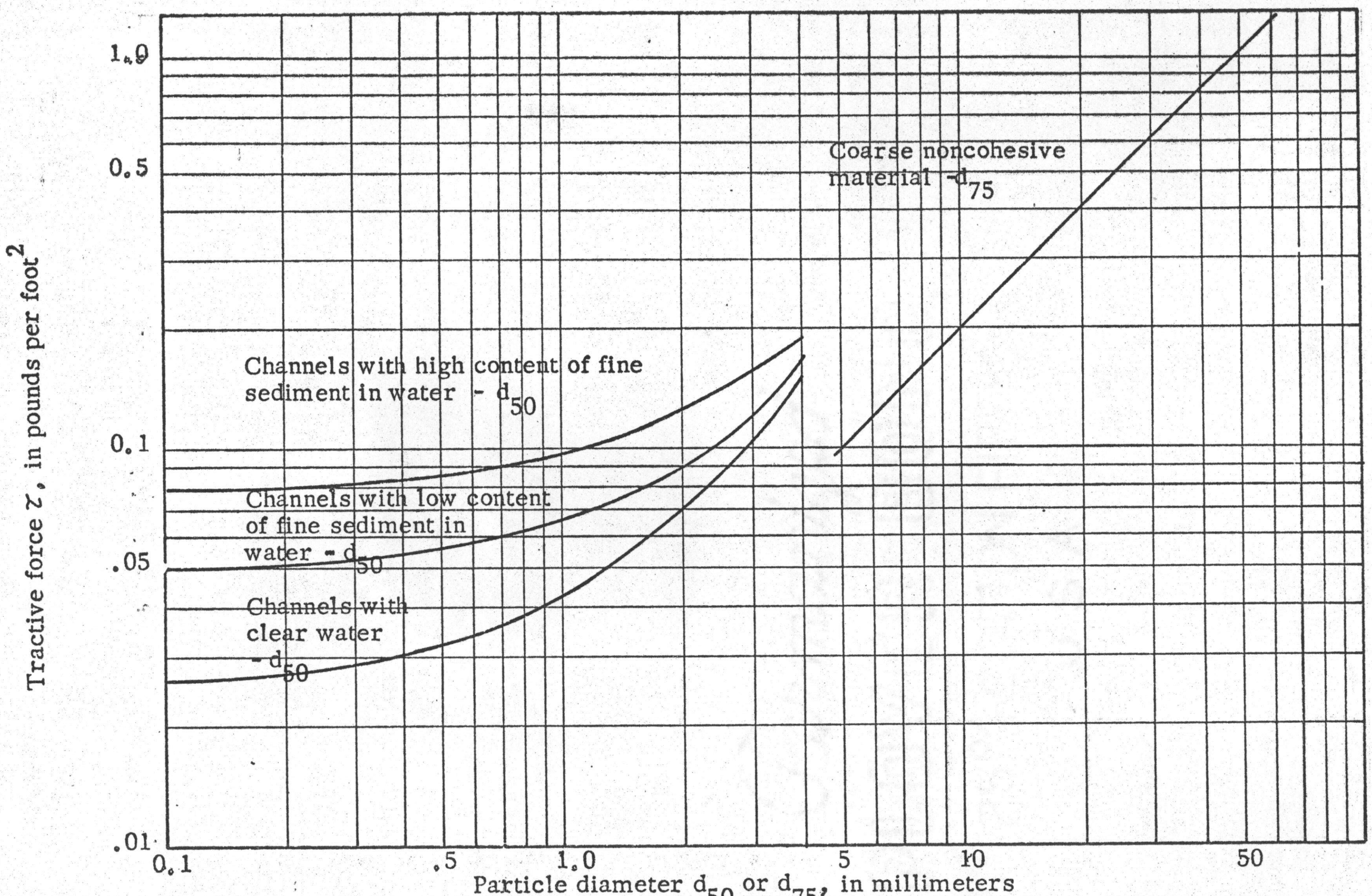


Fig. 8: Modified Lane tractive force diagram

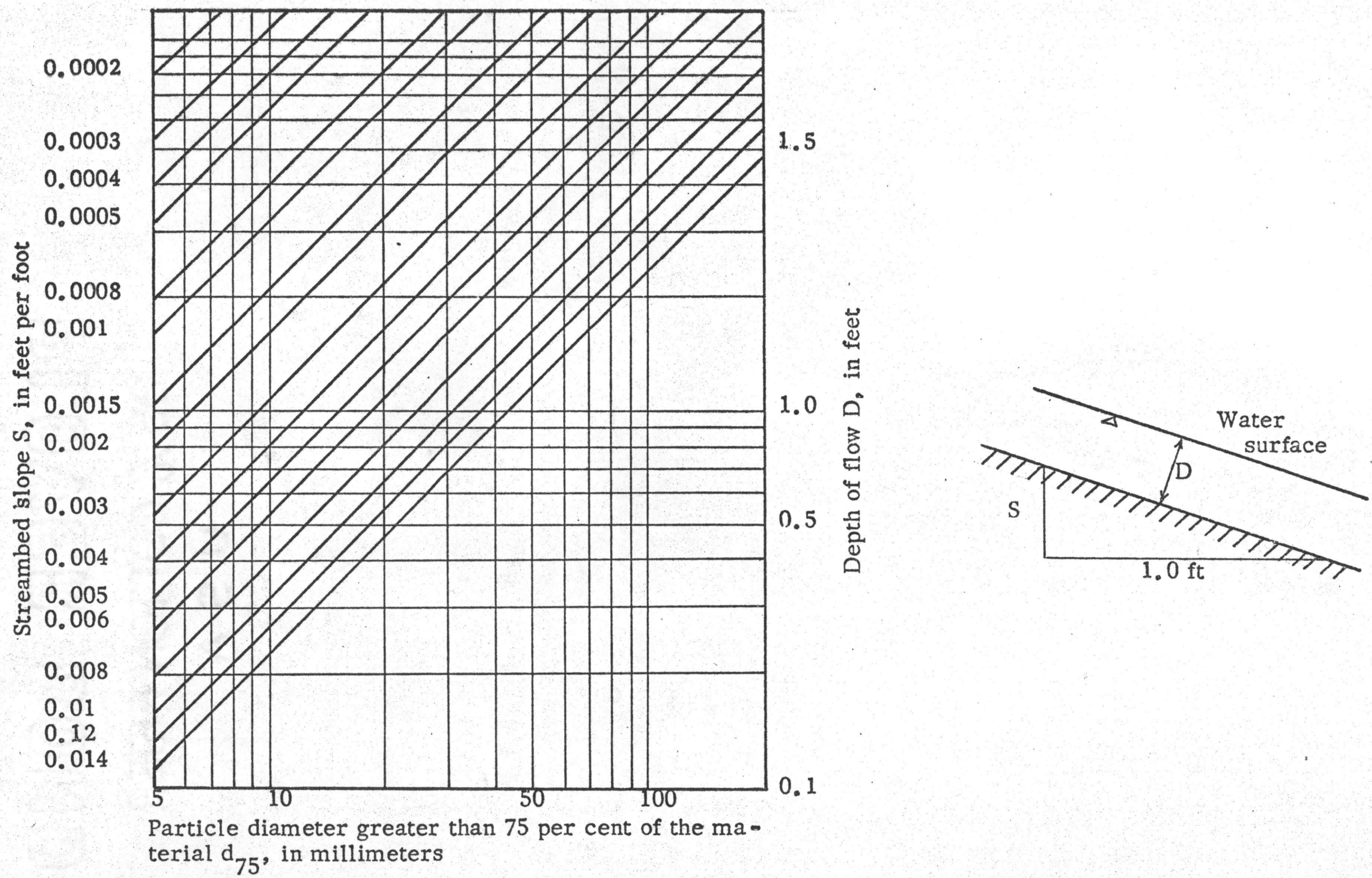
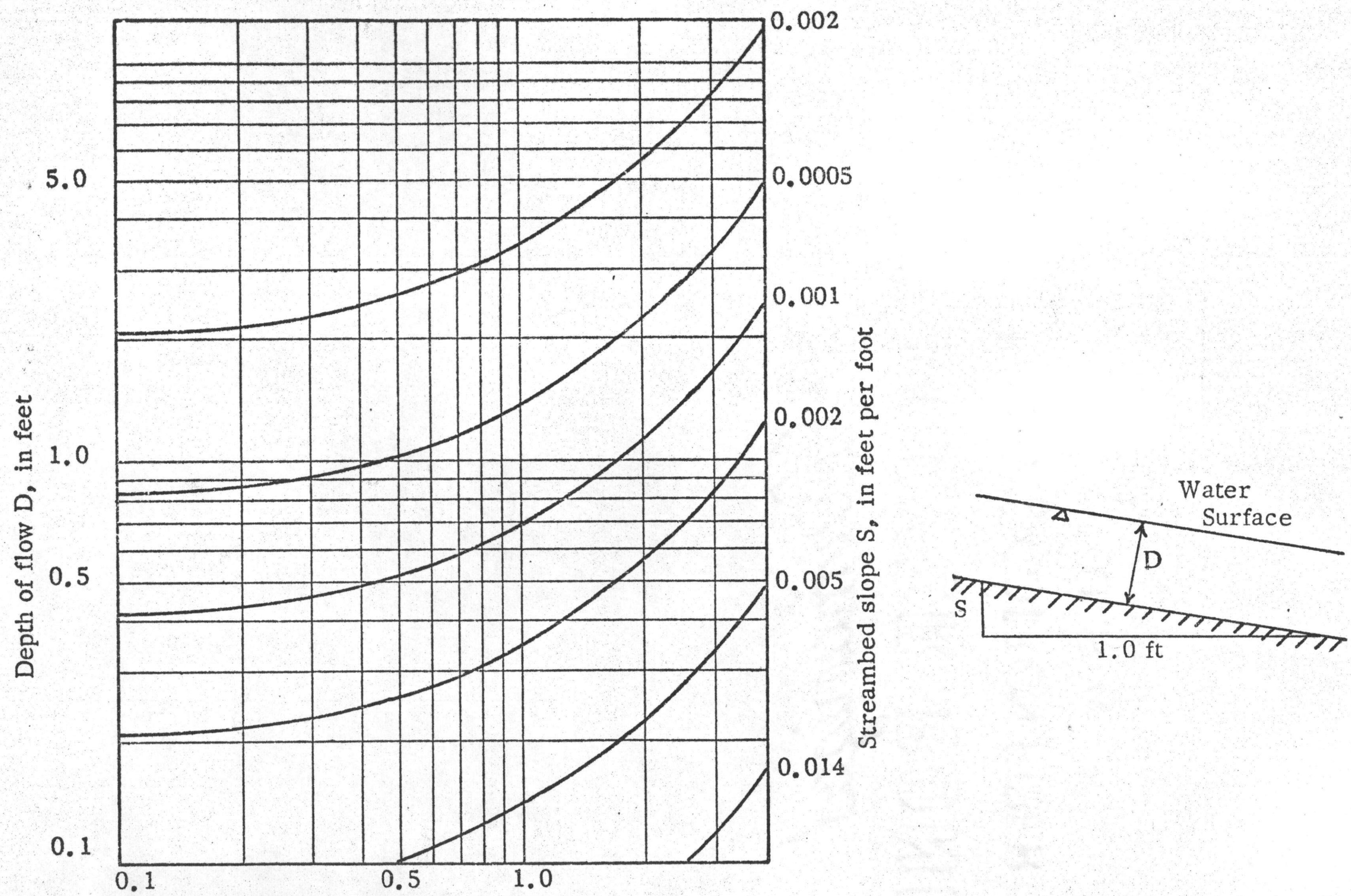


Fig. 9: Expected d_{75} of bed material for a given streambed slope S and depth of flow D for coarse noncohesive material.



Particle diameter greater than 50 per cent of the material d_{50}, in millimeters.

Fig. 10: Expected d_{50} of bed material for a given streambed slope S and depth of flow D for channels with clear water

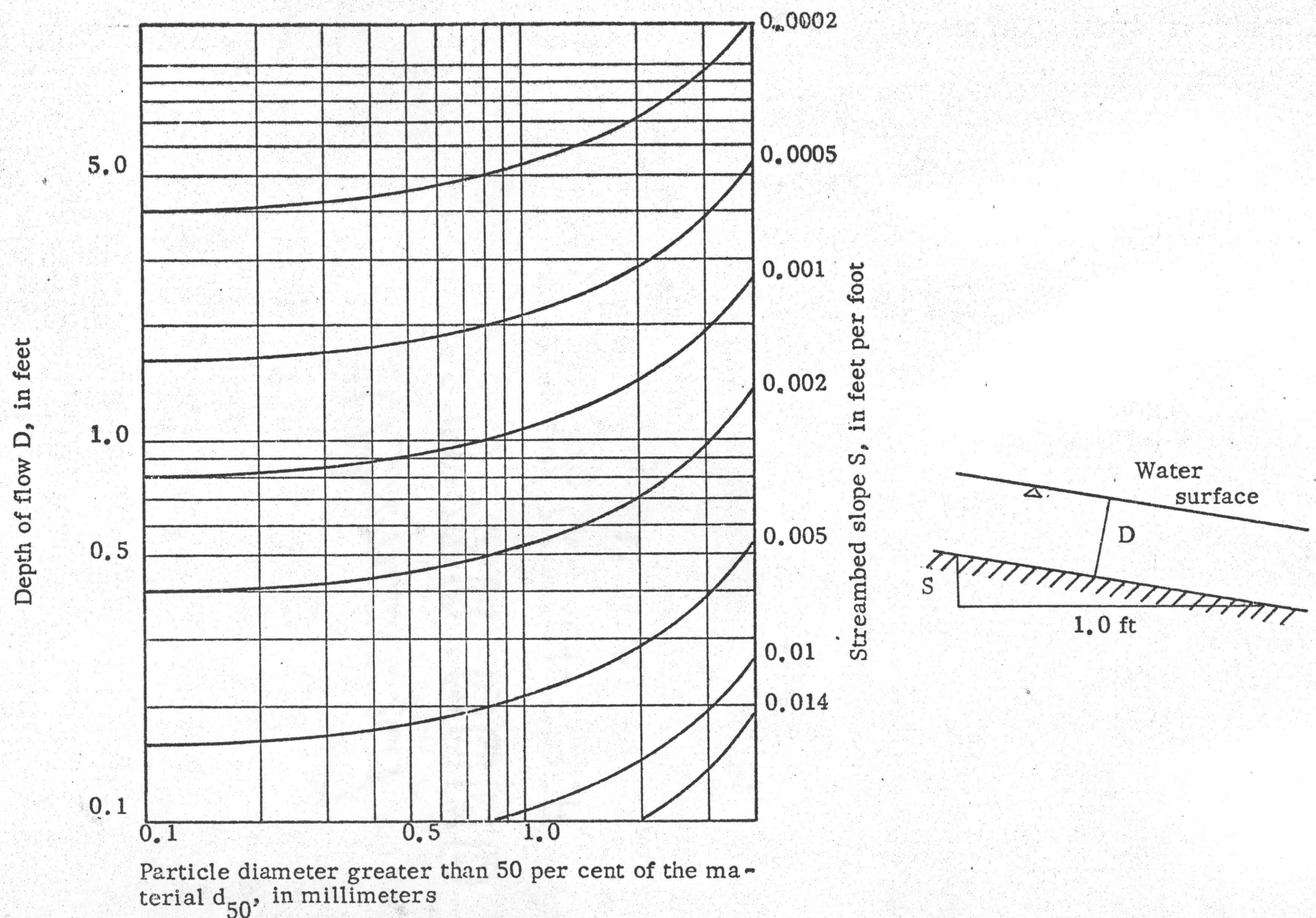
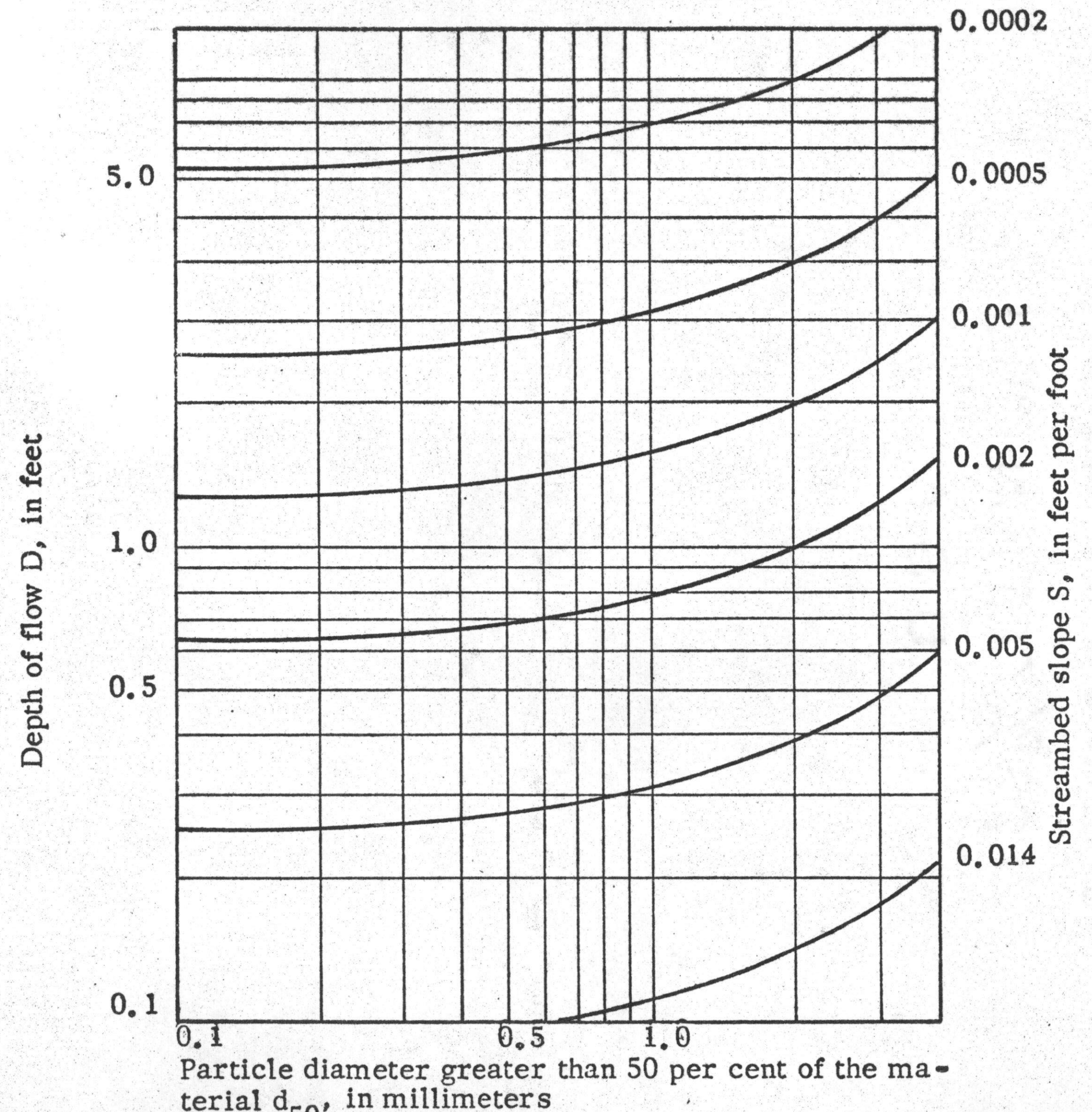


Fig. 11: Expected d_{50} of bed material for a given streambed slope S and depth of flow D for channels with a low content of fine sediment in the water.



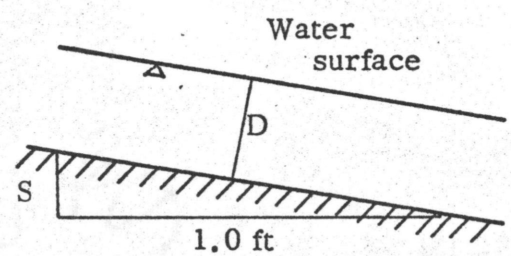


Fig. 12: Expected d_{50} of bed material for a given channel slope S and depth of flow D for channels with a high content of fine sediment in the water.

Depth of flow that is determined by the design procedure is the minimum depth that must be maintained year around to remove the unwanted silt and prevent silt from being deposited in the reach.

Knowing the depth of flow D, the width of flow W can be determined by using the Manning equation and considering the stream section with (condition 1) and without (condition 2) a control structure. The discharge Q is the same for both sections, therefore:

$$Q_1 = Q_2$$

and

$$\frac{1.49}{n} R_1^{2/3} S^{1/2} A_1 = \frac{1.49}{n} R_2^{2/3} R^{1/2} A_2. \qquad (5)$$

In equation (5) n is the Manning friction factor, R is the hydraulic radius, S is the slope of the streambed, and A is the area of the flow section.

The slope will initially be the same for both conditions and will change after the structure is installed. For the initial conditions S_1 equals S_2. Also for wide channels the depth (D) can be substituted for the hydraulic radius (R) without introducing an appreciable error. For the case of a rectangular shaped streambed, equation (5) reduces to:

$$D_1^{2/3}(D_1W_1) = D_2^{2/3}(D_2W_2)$$

or

$$W_2 = \left[\frac{D_1}{D_2}\right]^{5/3} W_1 \qquad (6)$$

where W is the width of flow.

The design procedure is:

1. Measure the width and depth of flow at the channel section.
2. Determine the depth of flow (D) using the design curves.
3. Use equation (6) to determine the width of flow (W_2) which corresponds to the width of opening of the control structure.

<u>Construction of Constriction Structures</u>

Gabion deflectors at Site 3 and Site 5 were constructed by placing rocks ranging from 1 to 24 inches in diameter within a wrapping of 8 ft wire chain link fencing. The rock was obtained from the stream channel and surrounding area. The 9 ft gabion sections were wired together to add length to the structures and placed perpendicular to the stream flow (Site 5) or sloped downstream (Site 3) to constrict the flow width during low flow. The gabions were constructed to a height of just greater than one foot to allow the higher flows to move freely over the top of the structures. Construction time for the gabion deflectors was one man hour per foot of gabion.

After construction the gabions were very porous with only 1 to 2 inches of head loss through the structures. It was hoped that they would eventually "seal" with finer material and that vegetation would grow on the surfaces exposed above water.

Log dikes used to form the meander at Site 4 were constructed by placing logs on the stream bed perpendicular to the flow and holding them in place by burying the end in the bank and driving steel pins into the substrate on the downstream side of the logs. Construction time for the log dikes was three man hours per pair of dikes. Four pairs of dikes plus 2 single dikes were used to form the meander.

Evaluation of Channel Constrictions

The gabion constriction at Site 3 had a satisfactory effect on the stream section. Sand was moved out of the constricted channel exposing the underlying cobble and boulders. Some boulders appeared to have been moved from the upstream riffle into the modified section. Two years after the gabions were constructed, the bed material just upstream, downstream and between the two gabions consisted of cobble and boulders ranging from 6 to 12 inches in diameter.

Contour maps of Site 3 shown in Figures 13 and 14 show the change in the streambed's topography due to the gabion structures. The thalweg profiles (Fig. 15) better depict the actual changes of streambed elevation resulting from scour induced by the gabion structures. Plates 2 and 3 show the gabions at Site 3. The thalweg profile is a line connecting the lowest surface elevation points of a streambed. The thalweg distance is the distance along this line.

Two adverse scour conditions did develop. The gabion constrictors forced the flow into the north bank approximately 30 ft downstream of the structures which scoured the bank. The second scour condition occurred on the upstream side of the gabions. The skew of the gabion caused excessive flow velocity along the upstream face of the gabion. If the bed material under the gabion had been smaller, undermining could have occurred. One advantage of the gabion structure is its ability to deform and adapt to changing foundation elevations.

As predicted some sediment did collect behind the structures. This sediment was removed during high flows when water cascaded over the gabion. Sediment did not collect in the expected amount as the gabions failed to seal with sediment. The porous gabions allowed sufficient flow through them to move some of the sediment.

High flow in Jaunary, 1973, deflected the end of the long gabion at Site 3 downstream as shown in Figure 14. This did not seem to affect the performance of the gabion. During an even higher flow in January, 1974, which was estimated to be in the range of a 300-year flood, a middle section of the long gabion at Site 3 was completely removed and moved approximately 20 ft downstream. The failure occurred at the connection between the gabion sections. This failure in no way reflects faulty design as the gabions were designed to withstand up to a 25-year flood. However, stronger connection between the gabion sections may have prevented the failure.

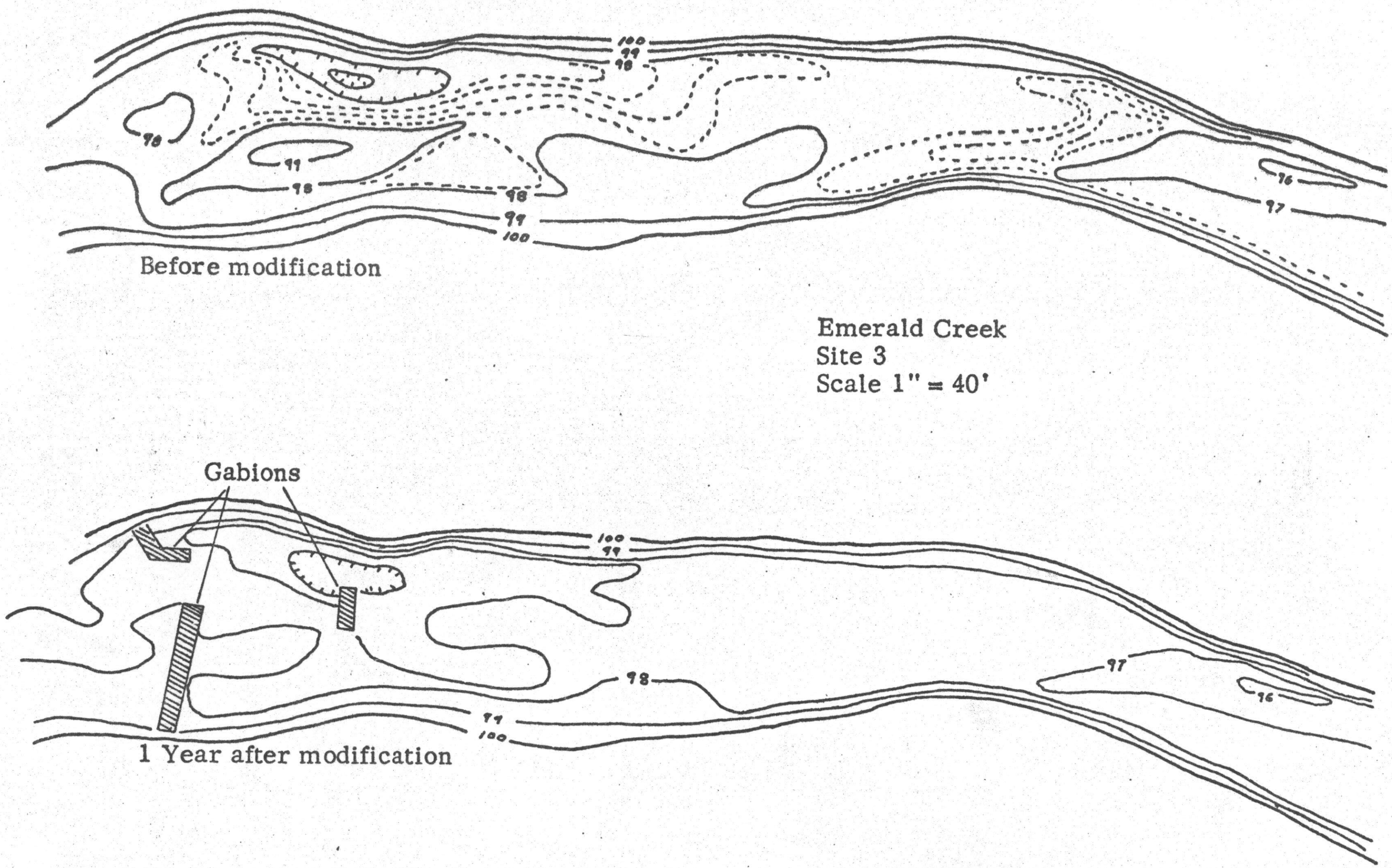


Fig. 13: Emerald Creek, Site 3, before and one year after after construction of gabion constrictors.

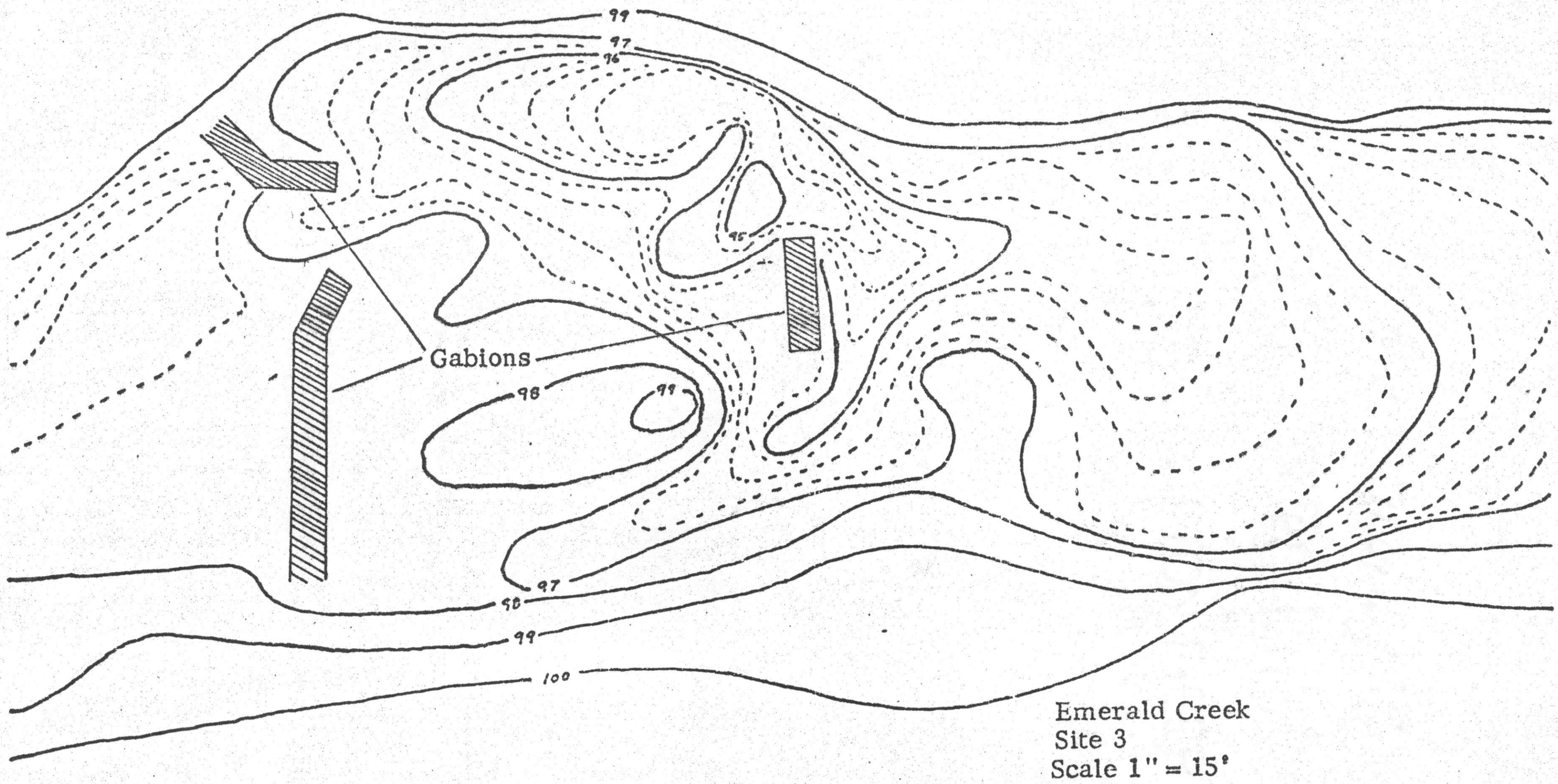


Fig. 14: Emerald Creek, Site 3, two years after construction of gabion constrictors.

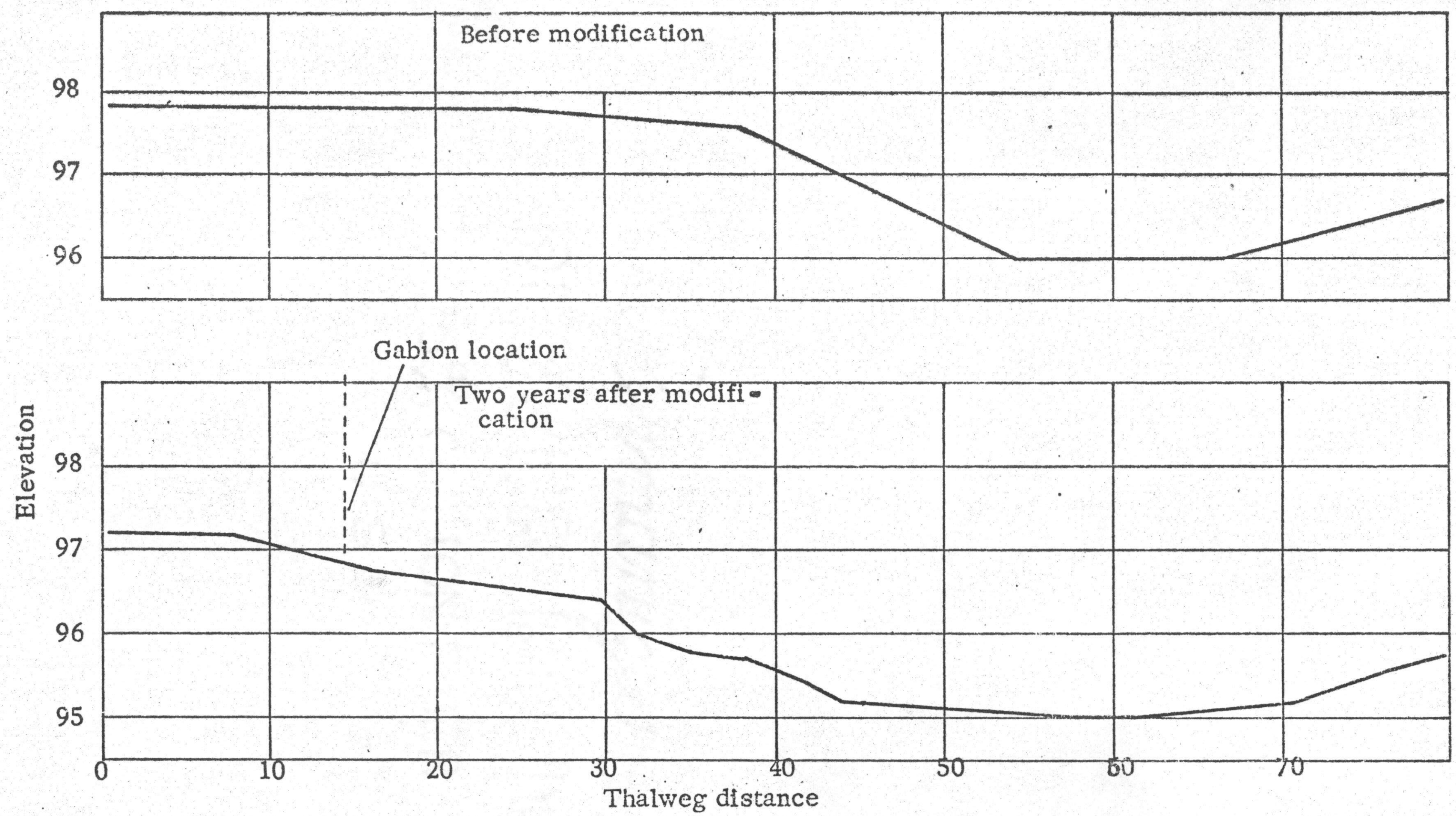
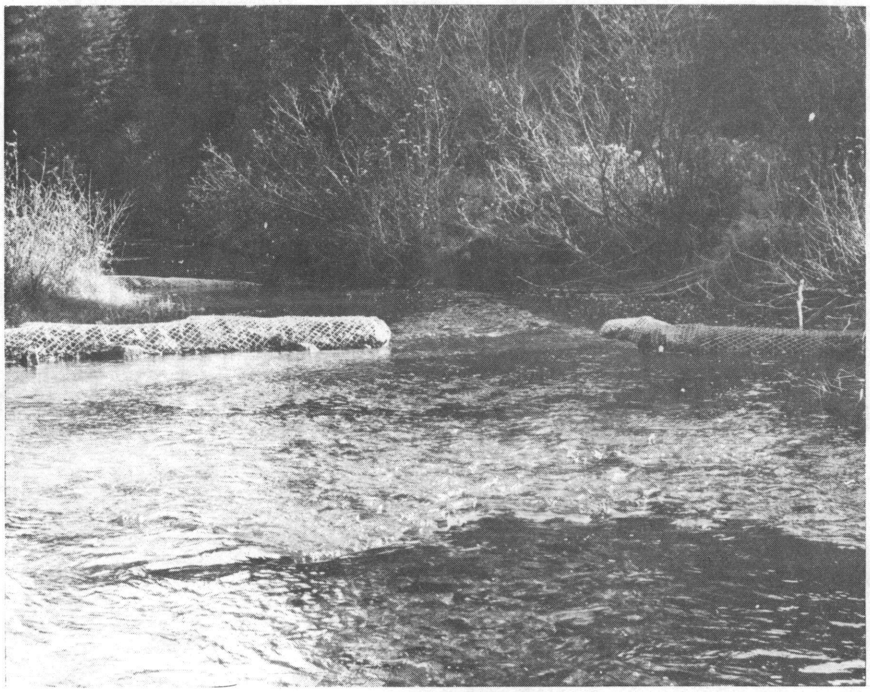


Fig. 15: Emerald Creek, Site 3, thalweg profiles.



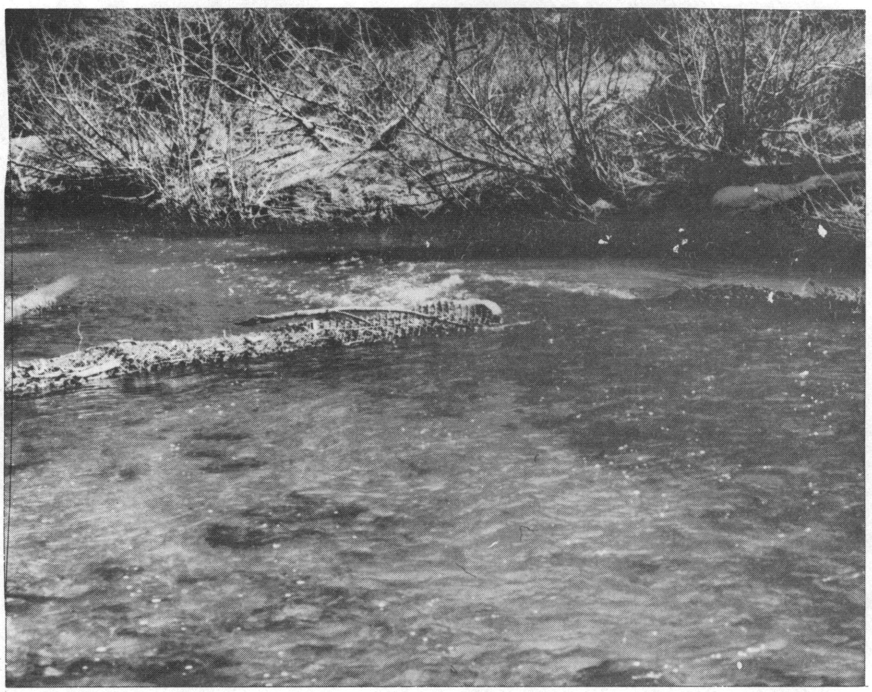
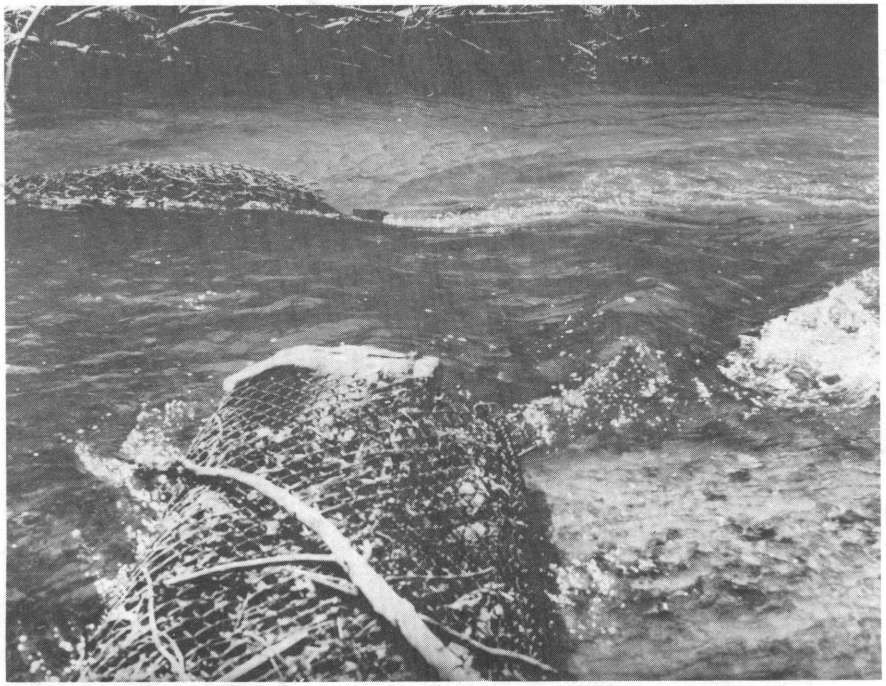


Plate 2. Gabion constrictor at site 3 showing two different flow stages.



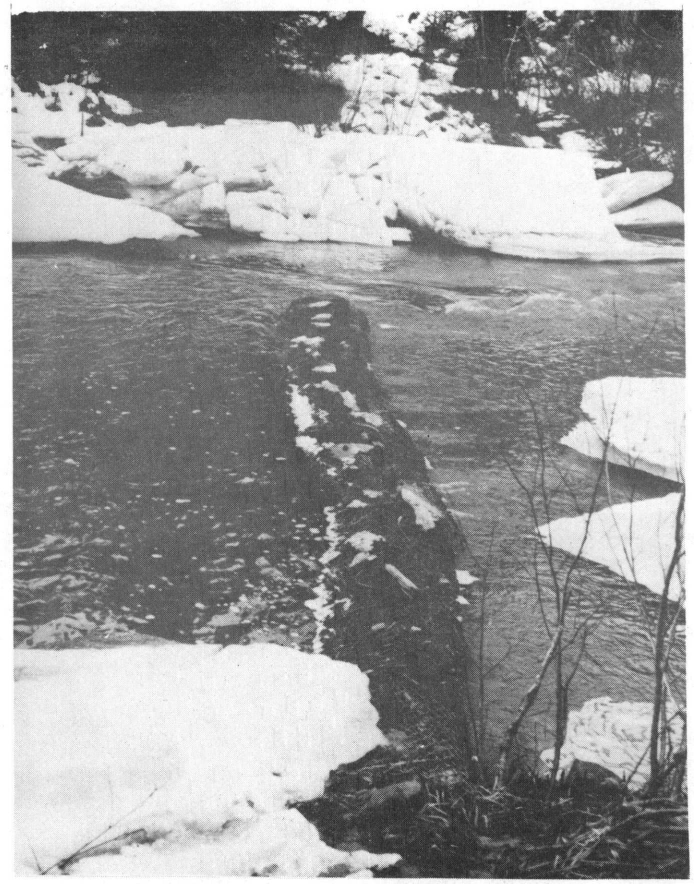


Plate 3. Gabion constrictor at site 3 showing flow through opening and gabions after the spring thaw in 1972 (R).

Performance of the gabion constrictors at Site 5 was only fair. Contour maps in Figures 16, 17, and 18 and thalweg profiles in Figure 19 show that the gabions caused very little scour in the modified reach. The lack of sediment transport capacity in this reach can be attributed to the low streambed slope. Cobble material from upstream did move into the constricted section which appears as an increase in the bed elevation on the contour maps and thalweg profiles at the upstream gabion. Plate 4 shows the gabions at Site 5.

These gabions did seal with sand and gravel and were much less permeable then the gabions at Site 3.

High flow in January, 1973, deflected gabion ends. Excessively high flow in January, 1974, did not further damage the gabions at Site 5, but it did cause some scour between the upstream gabion and its bank connection.

At both Sites 3 and 5, the gabions were too high. They should have been constructed to an elevation just above the water surface during low flow. To keep the gabions low profile and still maintain the weight needed to hold them in place the gabions could have been trenched into the streambed. Low profile structures would have less effect on the high flows and would be less likely to become encased in ice. The low profile could also be accomplished by reducing the size of the gabion. The gabions at Sites 3 and 5 could have been smaller and still withstood a 25-year flood without damage. If needed, the gabion could be anchored to the streambed at the free end.

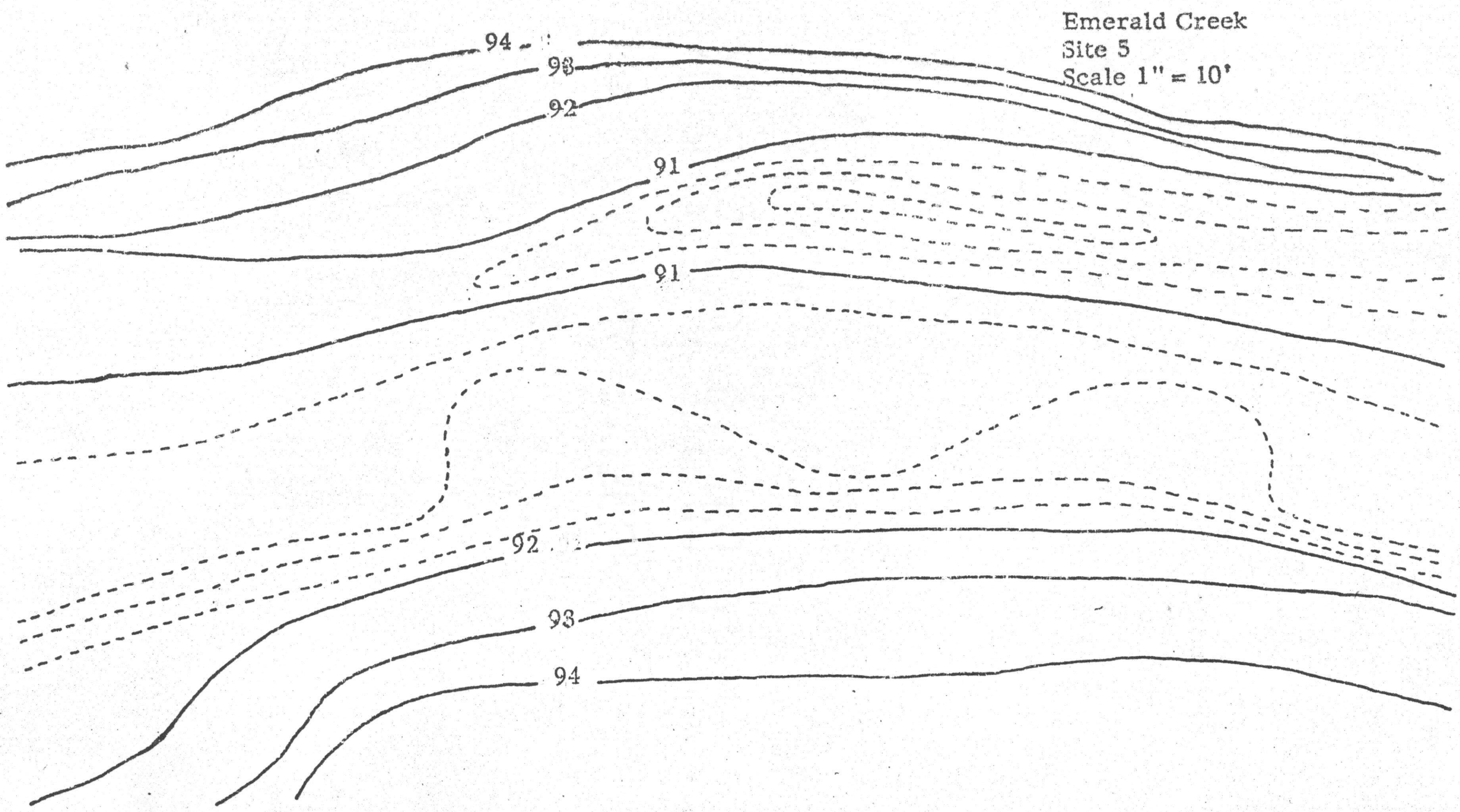


Fig. 16: Emerald Creek, Site 5, before modification.

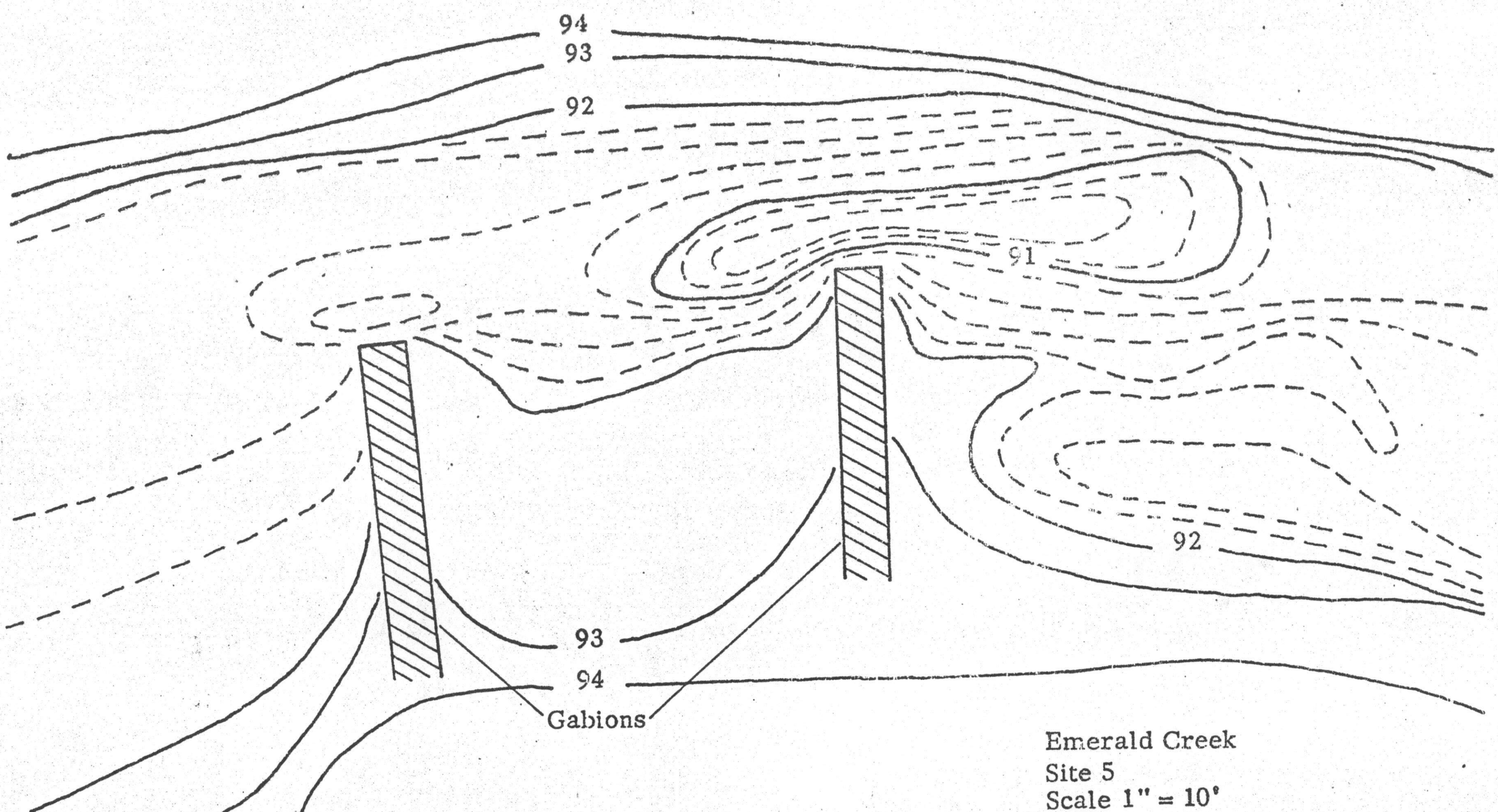


Fig. 17: Emerald Creek, Site 5, one year after modification with gabion deflectors.

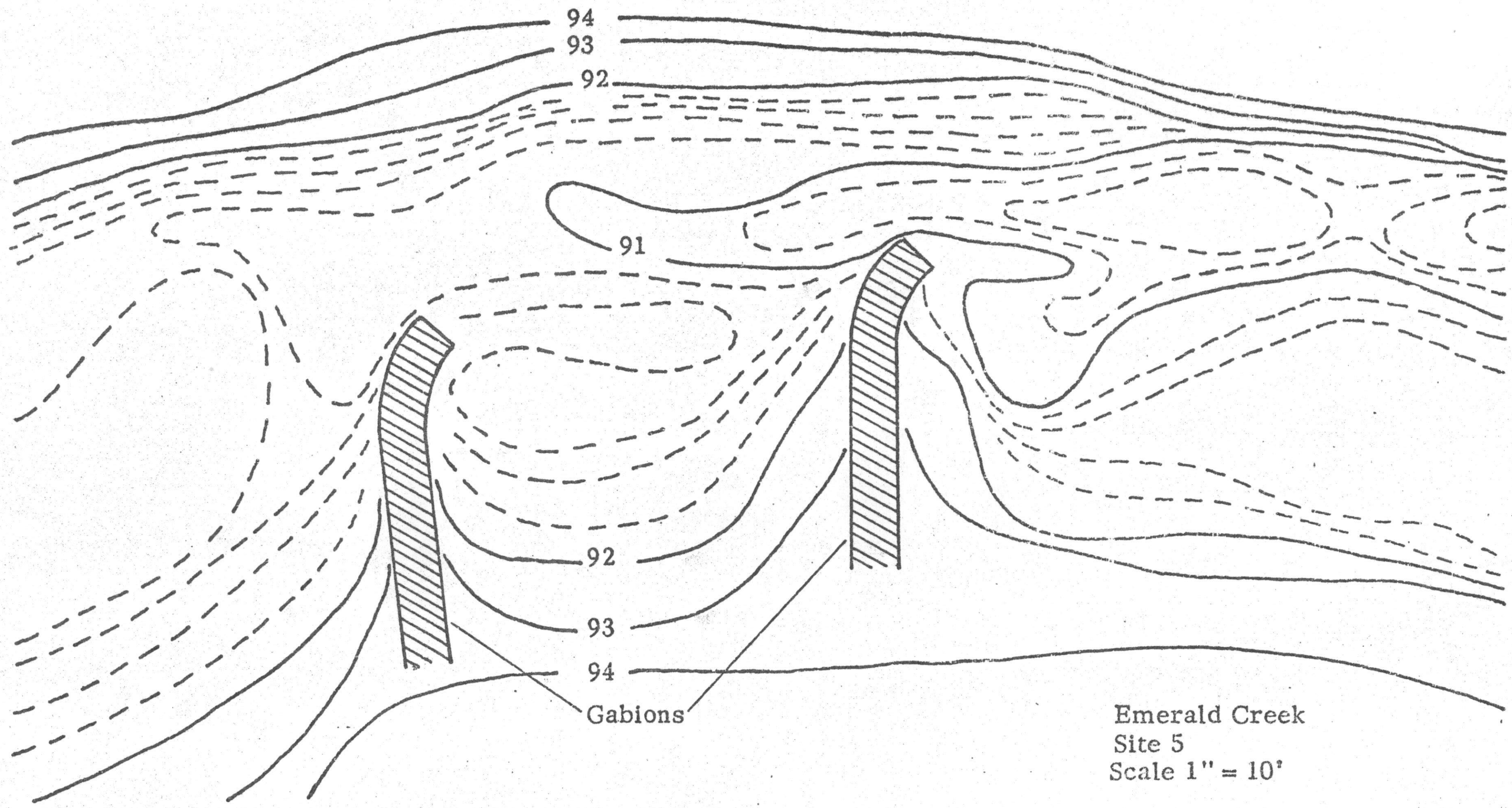


Fig. 18: Emerald Creek, Site 5, two years after modification with gabion deflectors.

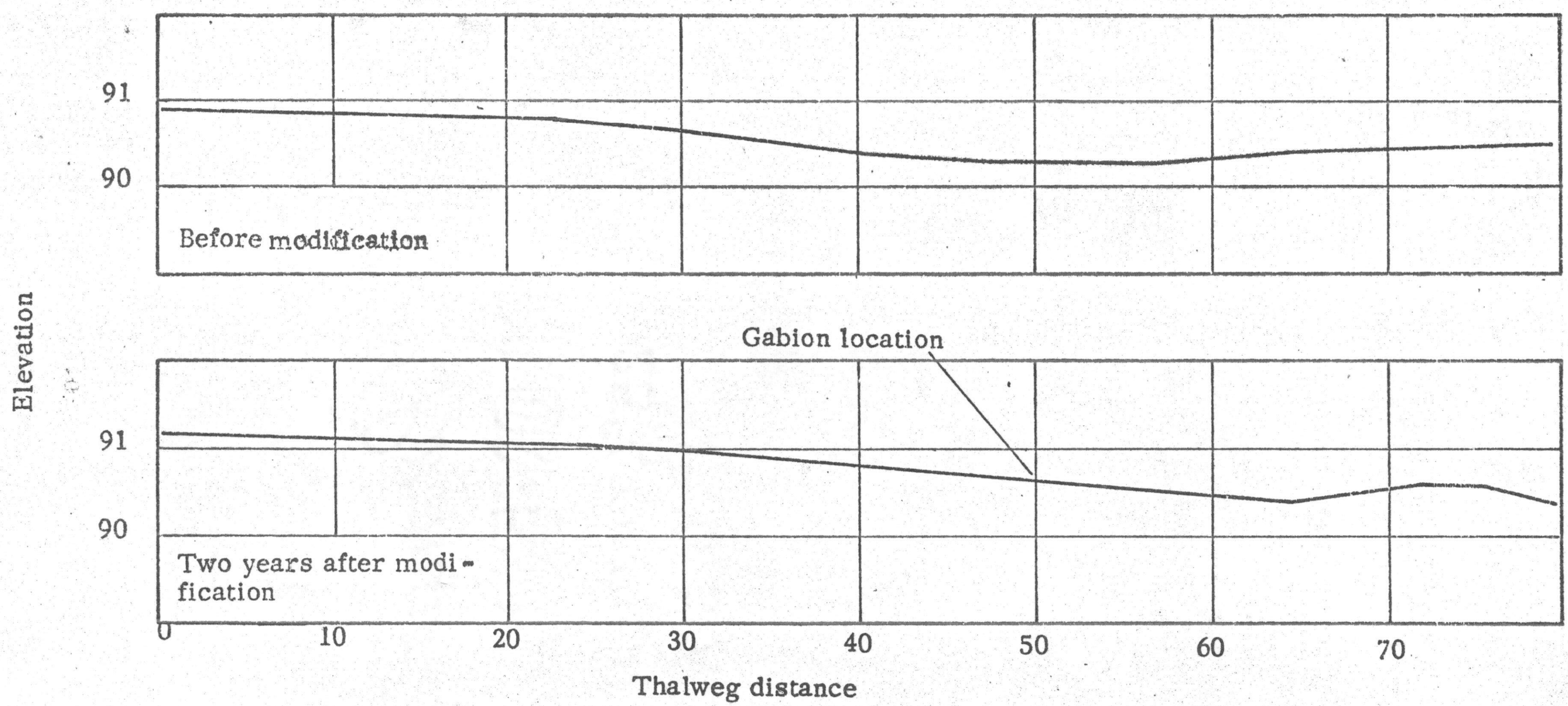
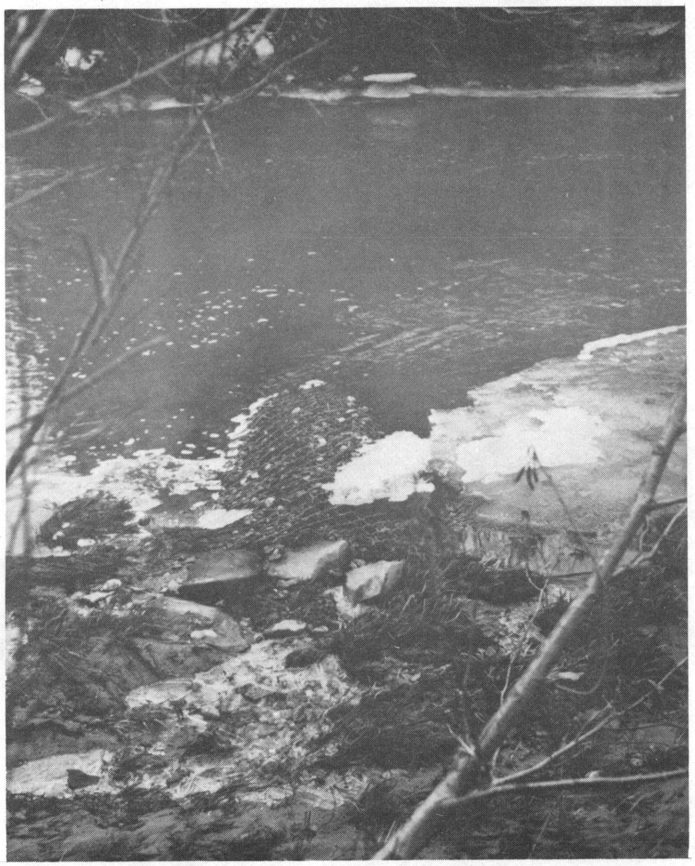


Fig. 19: Emerald Creek, Site 5, thalweg profiles before and after modification.



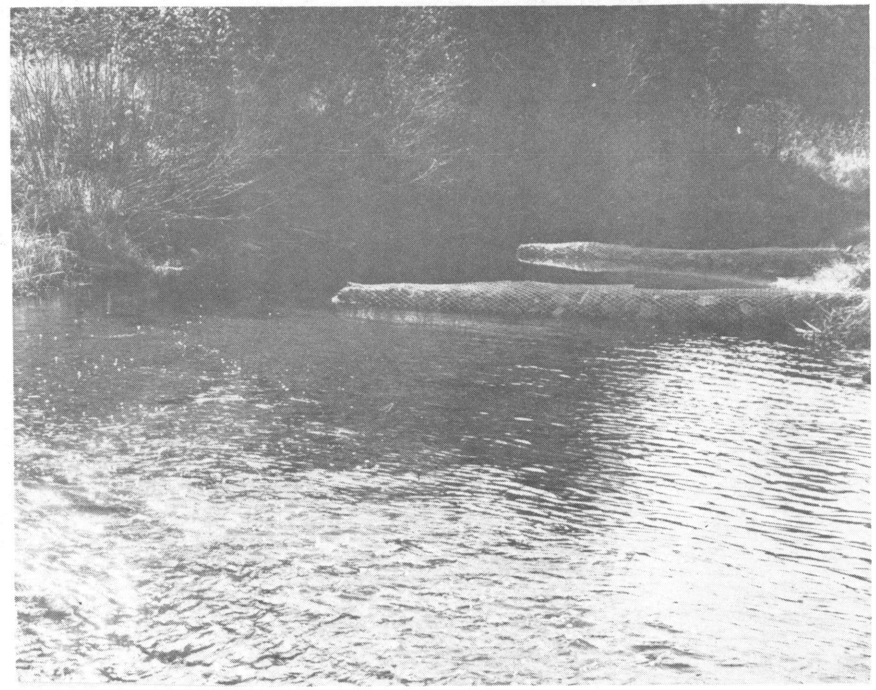


Plate 4. Gabion constrictors at site 5 showing one gabion being overtopped by high flow (L) and both gabions at low flow (R).

Observations at Site 4 just after the log dikes were installed indicated the meander was forming as expected. High spots in the meander path were being scoured out and the bed surface material was in the range of 1 to 3 inches at scoured locations. Figure 20 shows Site 4 before modification. Figure 21 shows Site 4 including log dikes, flow path, and projected contours.

The log dikes were constructed in September, 1973. High flow in January, 1974, removed the middle two sets of log dikes which precluded additional observations at the site as far as the meander was concerned. The remaining sets of dikes were causing negligible disturbances during the highflow and may function well as single channel constricting units during low flow periods.

Drop Structures

Three log drop structures were constructed at two sites on the upper reach of Emerald Creek. This reach is characterized by an incised channel and a relatively steep slope which is a necessary requirement for drop structure sites. The incised channel is needed to contain the increased depth of flow caused by the structures and the steep slope minimizes the backwater behind the structures. The purpose of a drop structure is to concentrate the energy of flood flow and consequently scour a hole downstream of the structure. The height of the drop is limited to the height that migrant fish can jump.

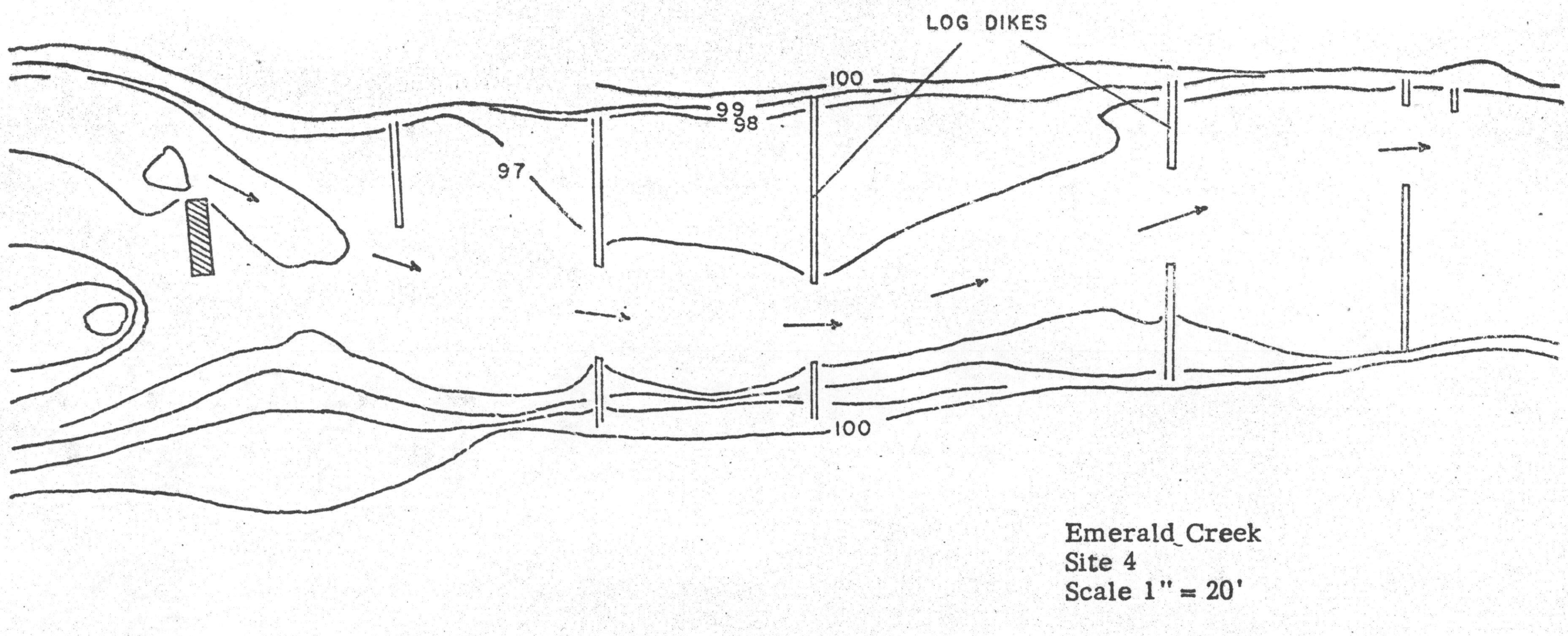
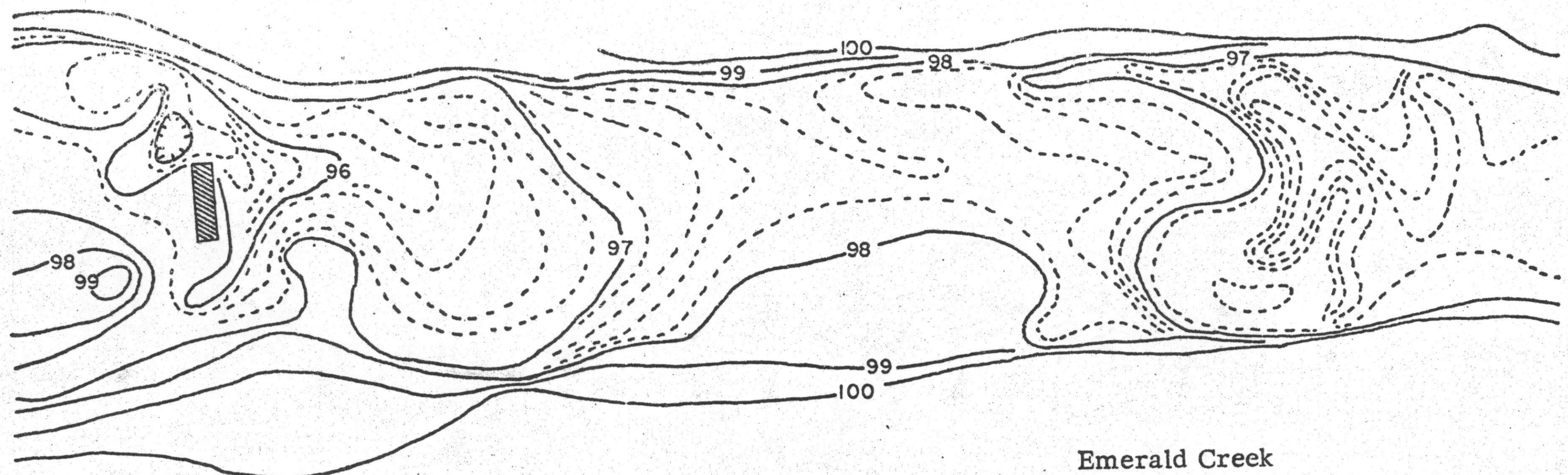


Fig. 21: Emerald Creek, Site 4, showing log dikes, flow path (arrows) and projected contours.



Emerald Creek
Site 4
Scale 1" = 20'

Fig. 20: Emerald Creek, Site 4, before modification.

Two log drop structures were constructed at Site 1. These drop structures (Fig. 22 and Plate 5) were based on a design in the Forest Service's Wildlife Habitat Improvement Handbook.

As shown in Figure 22 and Plate 5, two cedar logs were placed perpendicular to the flow with the ends buried in the bank. Two - inch thick wood planks were driven into the streambed on the upstream side of the logs. Construction time was about 20 man hours per structure.

The drop structure constructed at Site 2 was a variation of the structures at Site 1. As Figure 23 and Plate 5 show, the structure at Site 2 consisted of 3 logs with a gap between the top and lower logs. The structure was designed to allow the flow and fish to pass over the lower log during low discharge periods. During high discharge periods fish could still pass over the lower log but most of the flow would cascade over the top log and fall with a greater impact and thus increase the scour depth downstream of the structure.

The bed material at Sites 1 and 2 is predominantly coarse gravel with the mean sediment size (d_m) equal to 0.7 inches.

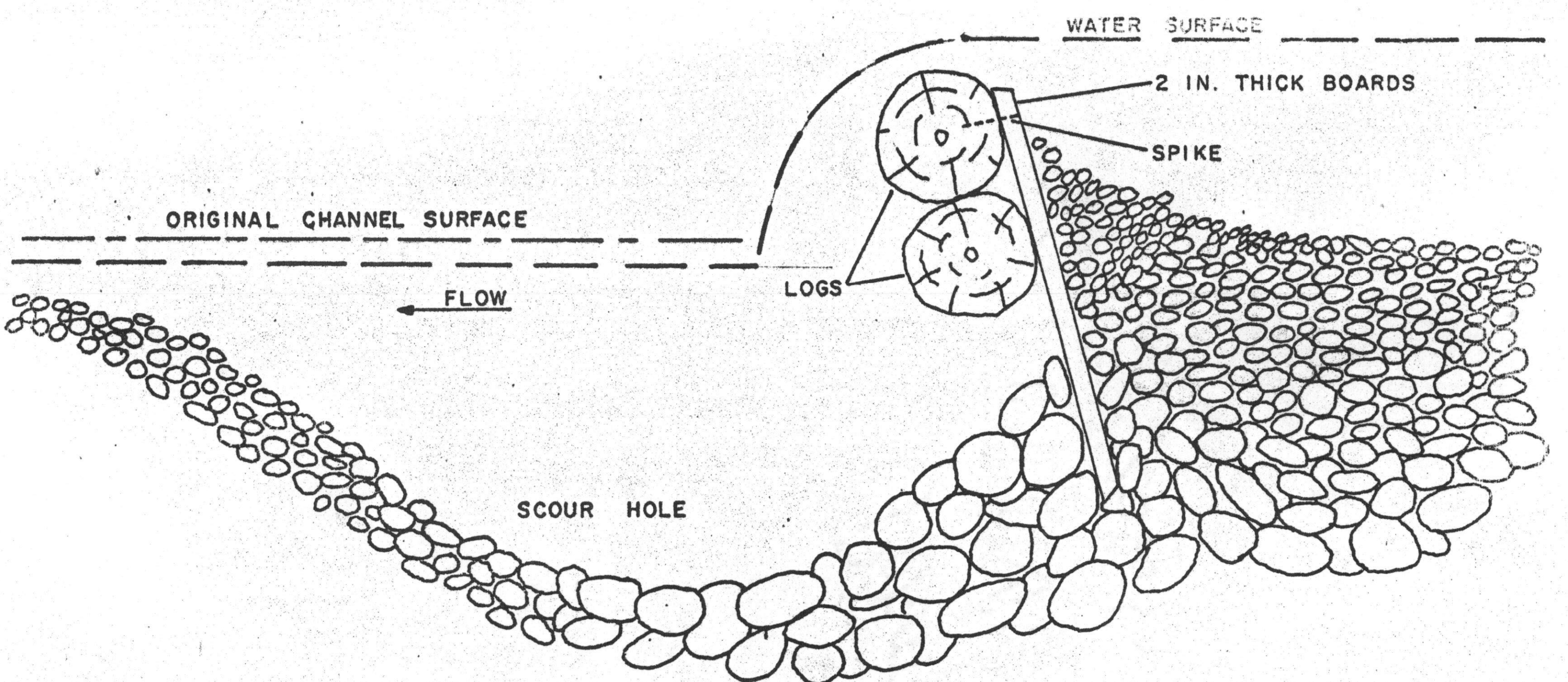


Fig. 22: Log drop structure used at Site 1.

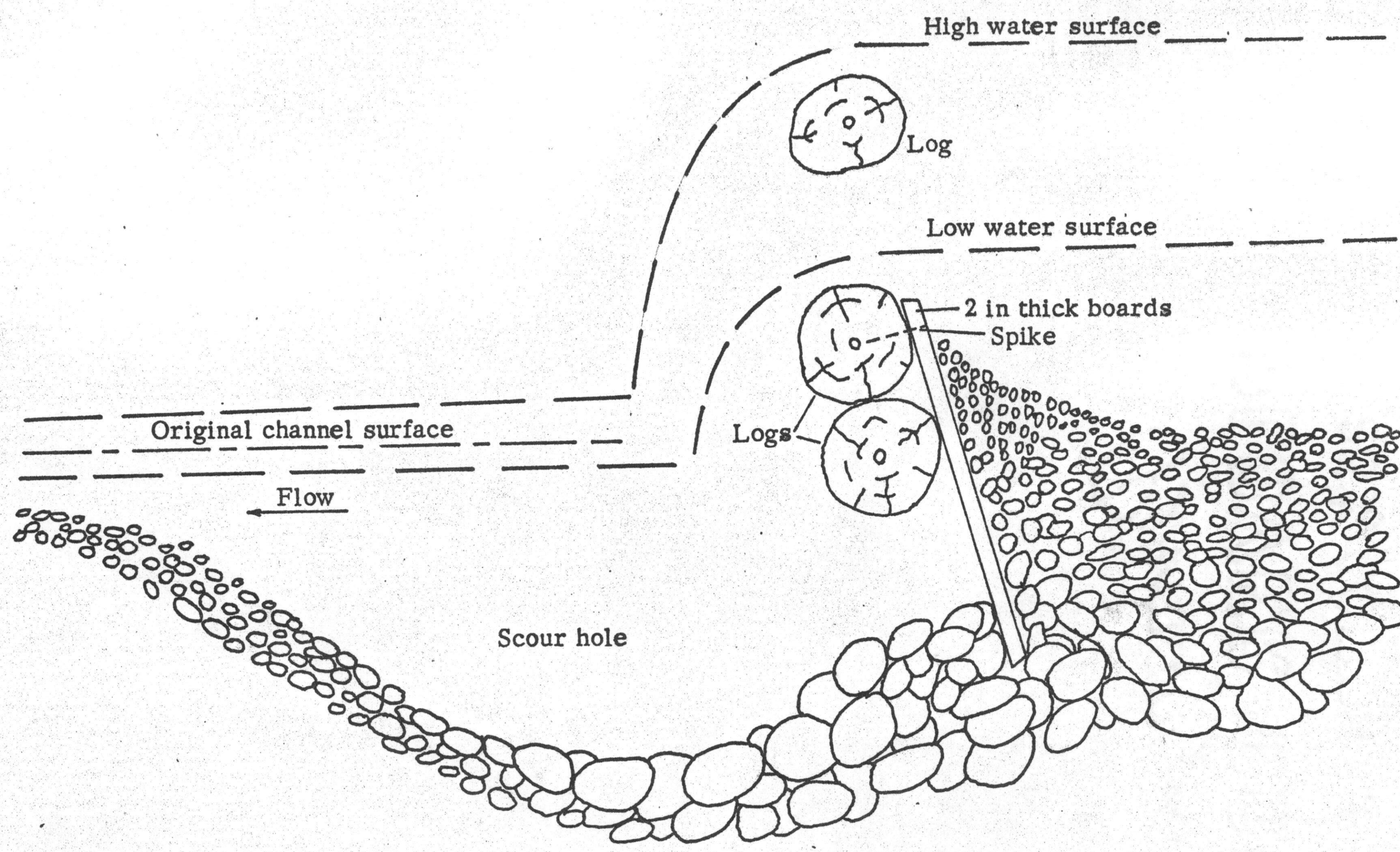
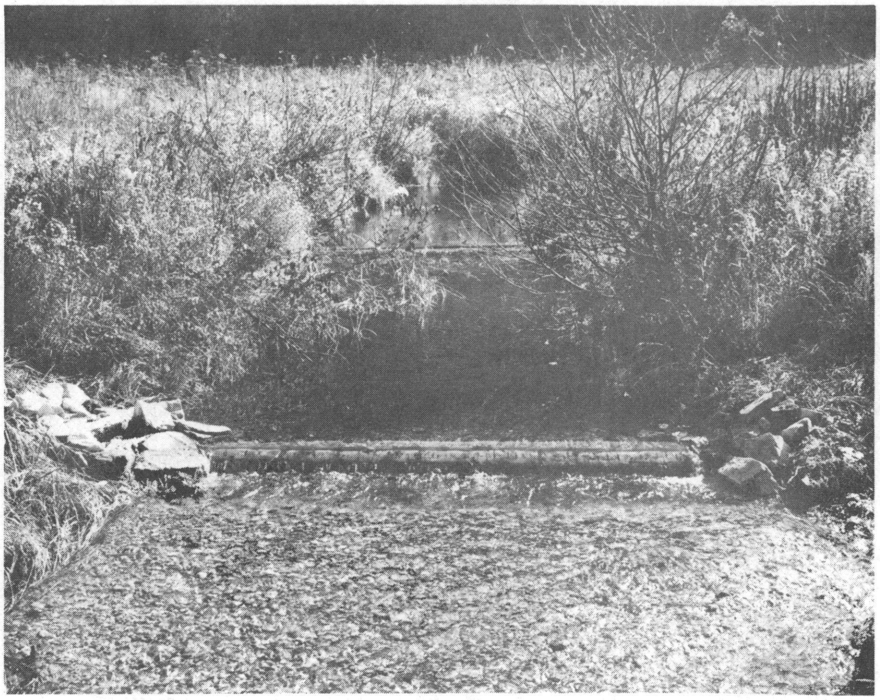


Fig. 23: Log drop structure at Site 2.



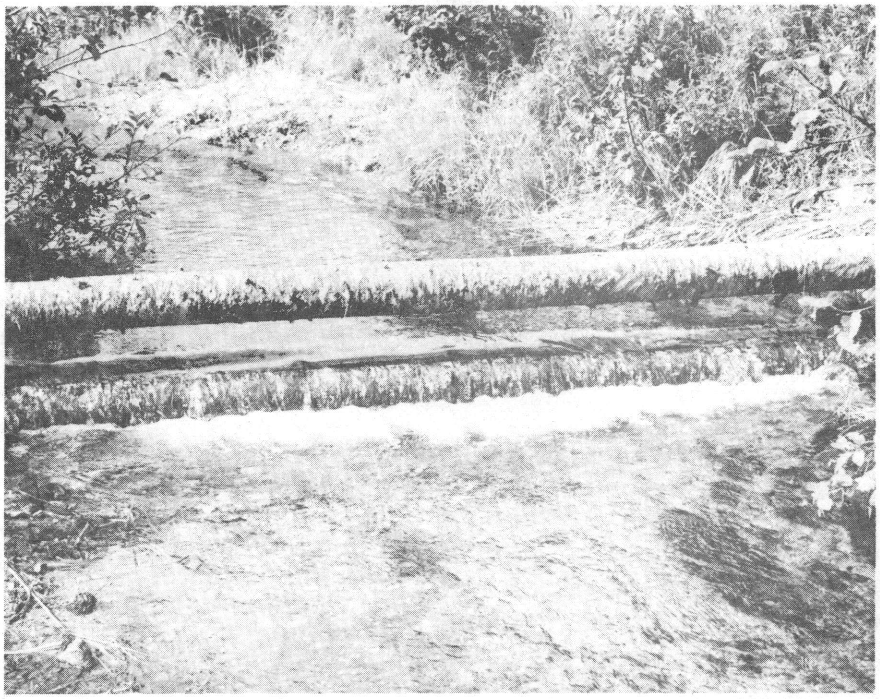


Plate 5. Two drop structures at site 1 (top) and the double log drop structure at site 2 (bottom).

Design of Drop Structures

Factors that should be considered in the design of a drop structure include forces on the structure, minimization of backwater, and scour below the drop structure.

Forces on the drop structure. The forces acting on the log drop structure include hydrostatic pressure, soil pressure, and weight of soil and water on the boards connected to the logs. The total force on the drop structure F_R, as calculated in Appendix B, is 435 lb per foot of log. Equation (7) can be used to determine the embedment length L_2 of the ends of the log in the bank.

$$L_2 = \frac{F.S.\ F_R L_1}{2\,R}, \qquad L_2 \geq 3\text{ ft} \tag{7}$$

In equation (7), F.S. is the factor of safety, L_1 is the length of the logs between the banks, and R is the resistance force of soil per foot of log. See Appendix B for the determination of R.

Minimization of backwater. Increased depths behind a drop structure can be evaluated by using the standard step method, as described by Chow. This method utilizes the channel geometry, discharge, and the Manning equation in a trial and error procedure. Computations are started at a known point in the system and depths are calculated at predetermined intervals in an upstream direction.

Scour below drop structures. Depth of a scour hole below a drop structure depends on depth of flow H over the structure, depth of the tailwater D_2, height of the drop structure P, and size of material d below the plunging water. For a given value P and d, deeper flow over the structure tends to increase scour depth D_s while an increase in tailwater depth D_2 results in a decreased scour depth. There is an optimum set of values for H and D_2 at which D_s is a maximum. This optimum set of values may or may not occur at the maximum discharge.

Smith (1967) presented a detailed description of scour below a drop structure and presents dimensionless curves for solution of the scour depth. As shown in Figure 24, the flow drops over the structure into the tailwater. The

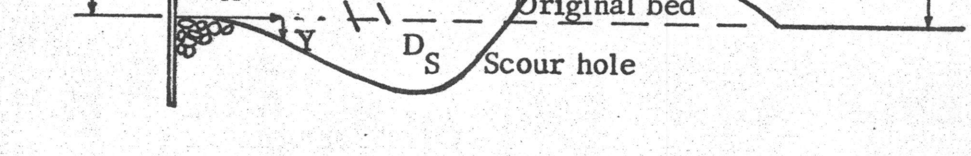


Fig. 24: Scour hole formation in a stone bed, reproduced from Smith (1967).

force of the plunging water is resisted by the stone below the drop structure. Initially drag forces on the stones exceed resisting force and the stones are moved downstream leaving a scour hole. As scour-hole depth increases, the velocity of the falling water, at the point of contact with the stones, is reduced. Eventually the scour hole reaches a depth at which resisting forces on the stone exceed the drag force of the falling water and the scour process ceases.

Smith developed the dimensionless curves shown in Figure 25 that can be used to estimate the depth of scour that can be expected below a drop structure. The range of variables in Smith's curves did not contain the values encountered on the Emerald Creek project. Curves used for Emerald Creek were developed by extending and extrapolating Smith's curves to get the curves shown in Figure 26. At Emerald Creek it was important to keep the P value low enough to allow the migration of fish past the structure.

Stone size d used by Smith is the equivalent spherical diameter of the average stone size by weight. The tests conducted by Smith were limited to rounded stones of narrow size range with a specific gravity of 2.7. At the Emerald Creek test sites the stones were irregularly shaped with a specific gravity of 2.65-2.7. The stone size used for scour depth estimates was the mean stone size (d_m).

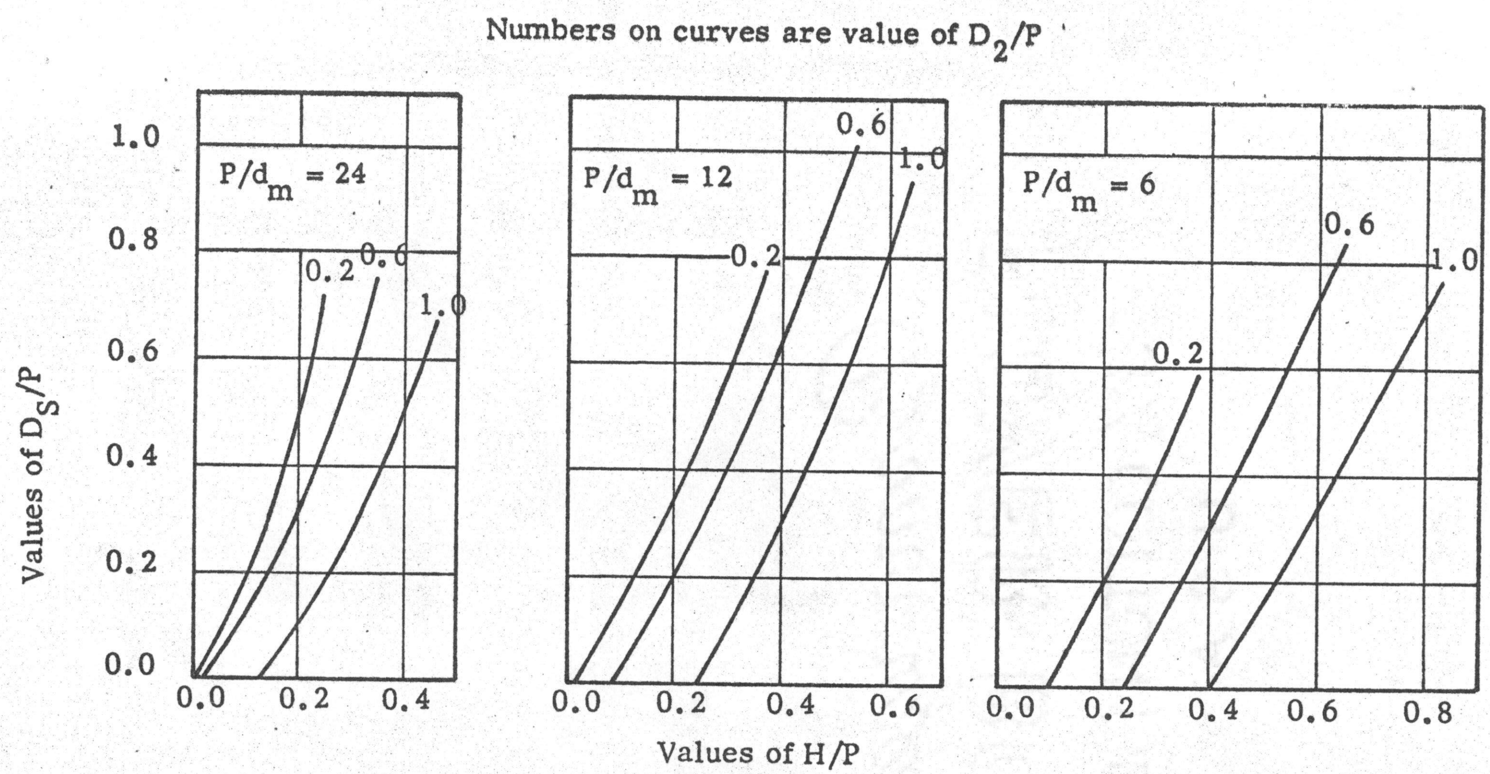


Fig. 25: Dimensionless curves for scour depth [reproduced from Smith (1967)].

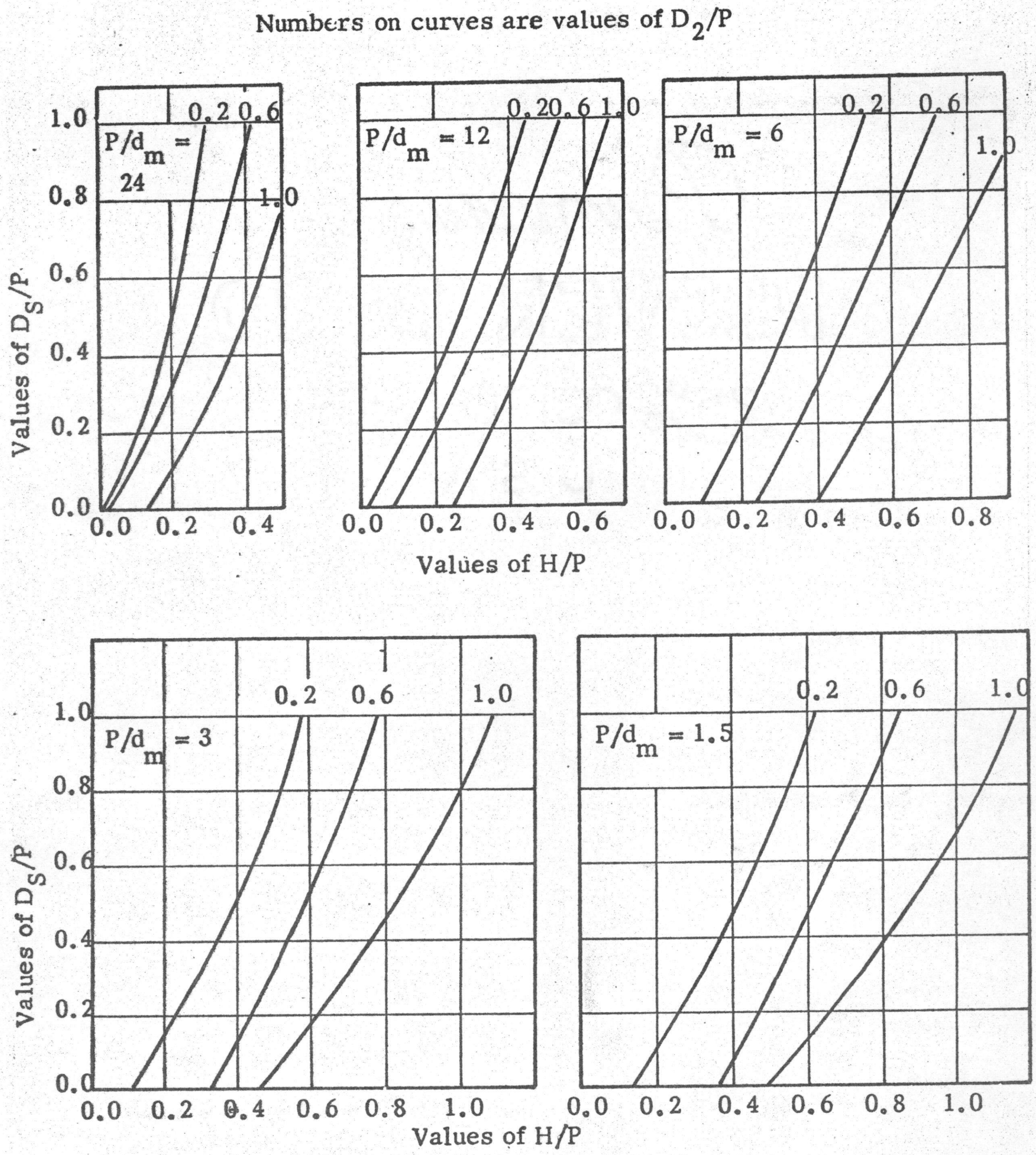


Fig. 26: Extended dimensionless curves for scour depth.

Evaluation of Log Drop Structures

As shown in the contour maps in Figures 27, 28, and 29, the downstream drop structure at Site 1 performed acceptably. A one foot deep hole was scoured below the drop structure and was lined with rock from 6 to 10 inches in diameter. Using the dimensionless curves in Figure 26, the predicted depth of scour D_s was 2 feet. Thalweg profiles in Figure 30 show the profiles before, one year after and two years after modification of the stream with a drop structure. The profile downstream of the second drop structure shows the scour hole and the rise in elevation just downstream of the scour hole where much of the scour material was deposited.

Scour below the upstream structure at Site 1 was limited by high tailwater caused by the downstream structure. The limited scour below the upstream structure is a good example of decreased scour due to the energy dissipation of the high tailwater.

The drop structure at Site 2 scoured a hole 1.5 feet deep as shown in the contour maps of Figures 31 and the thalweg profiles in Figure 32. The scour hole was lined with stones in the range of 6 to 10 inches in diameter. Predicted depth of scour using the curves in Figure 26 was 3 feet. This additional depth of scour at Site 2 is a result of the large drop height at this site.

At low flow the water plunges over the lower log, but the high flow plunges over the top log resulting in a significant increase of kinetic energy working on the bed. The stones underneath the boards have been scoured away leaving the boards almost completely exposed on their underside.

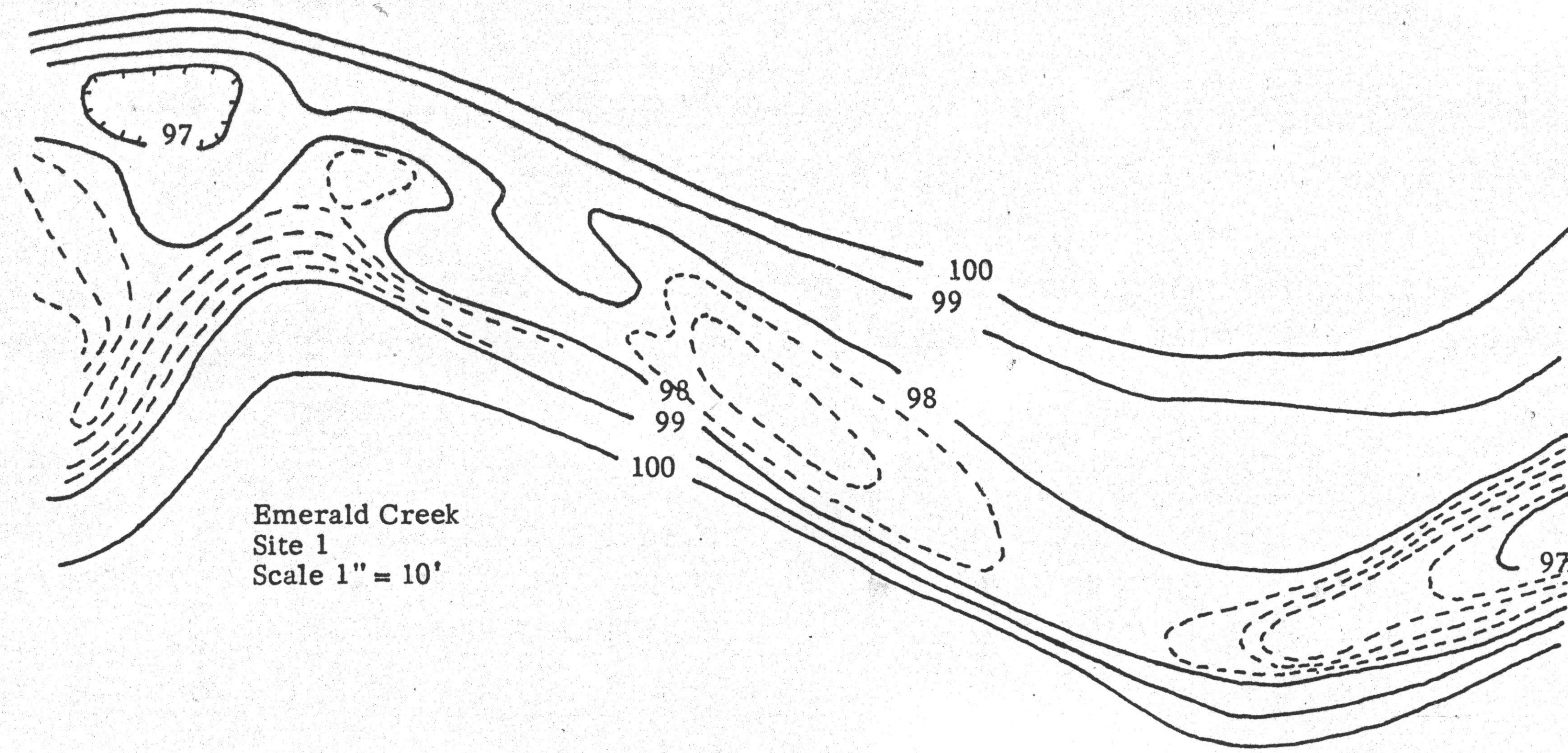


Fig. 27: Emerald Creek, Site 1, before modification.

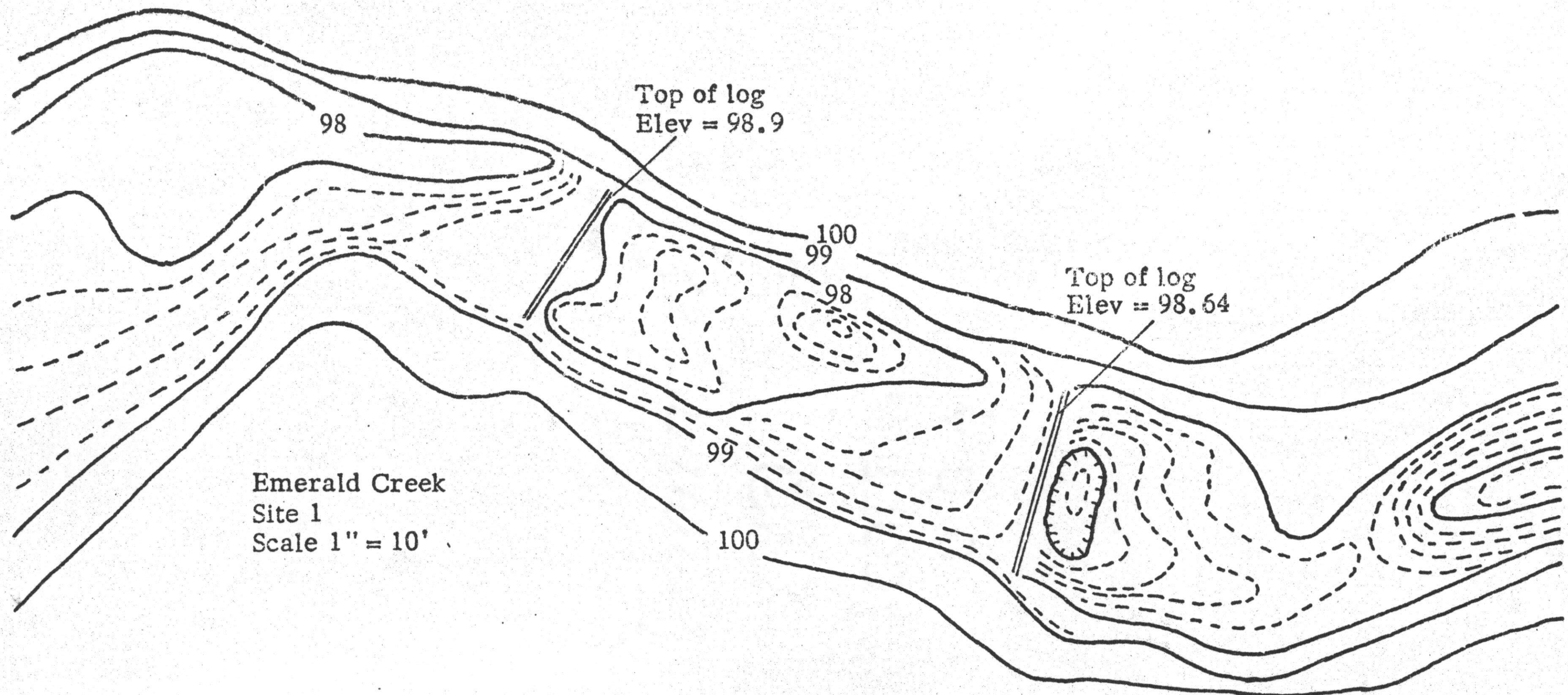


Fig. 28: Emerald Creek, Site 1, one year after modification with two log drop structures.

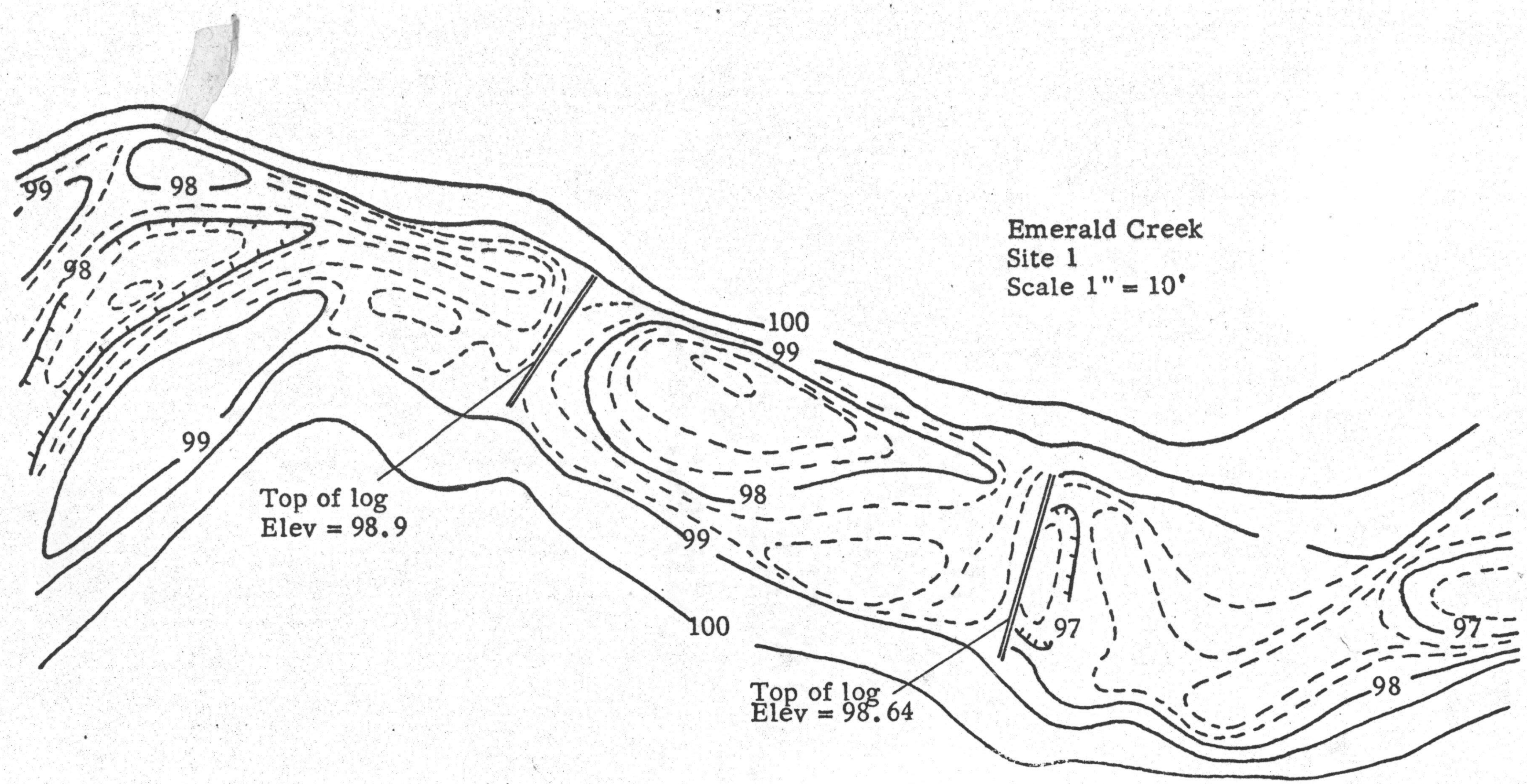


Fig. 29: Emerald Creek, Site 1, two years after modification with two log drop structures.

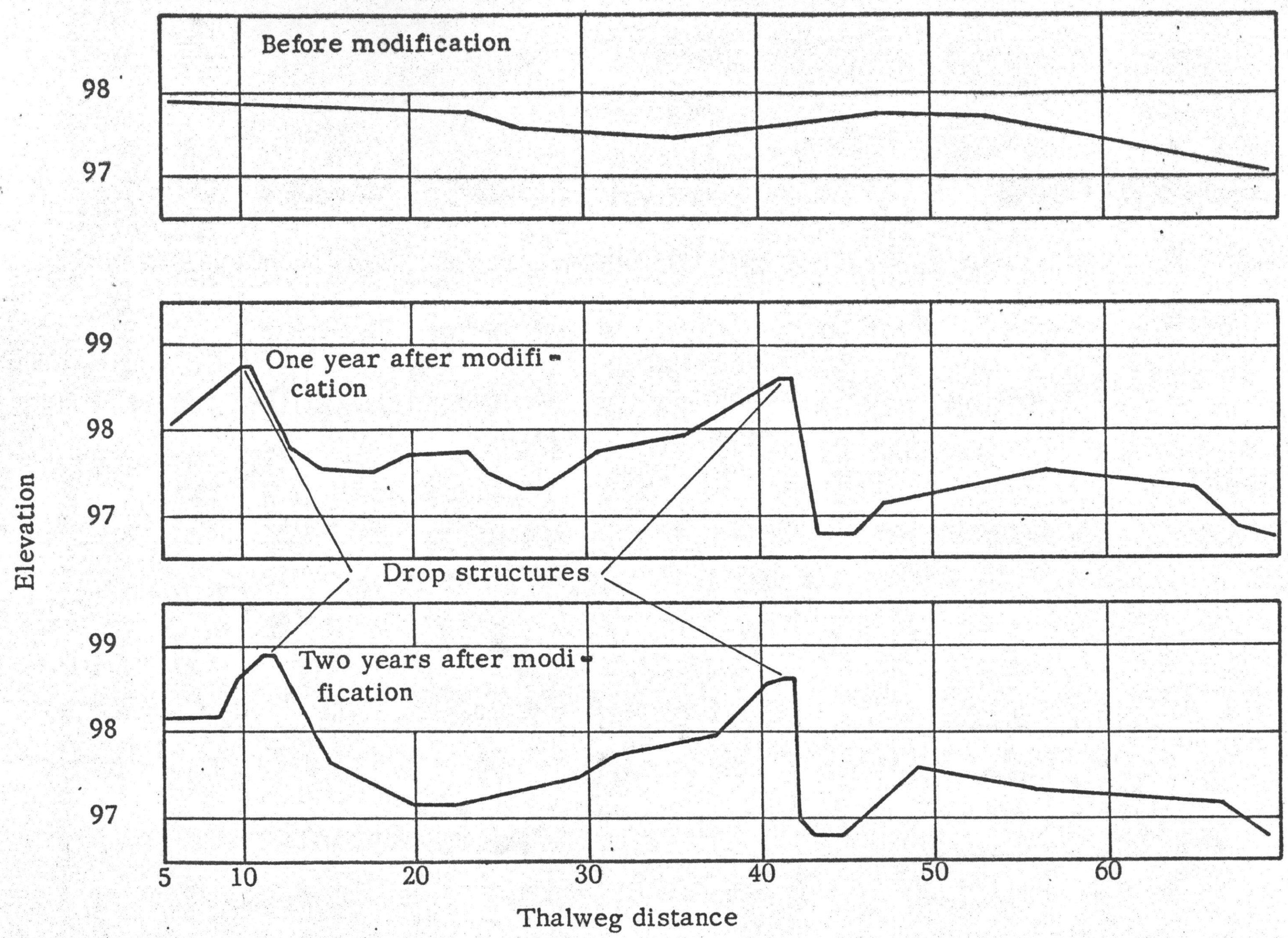


Fig. 30: Emerald Creek, Site 1, thalweg profiles.

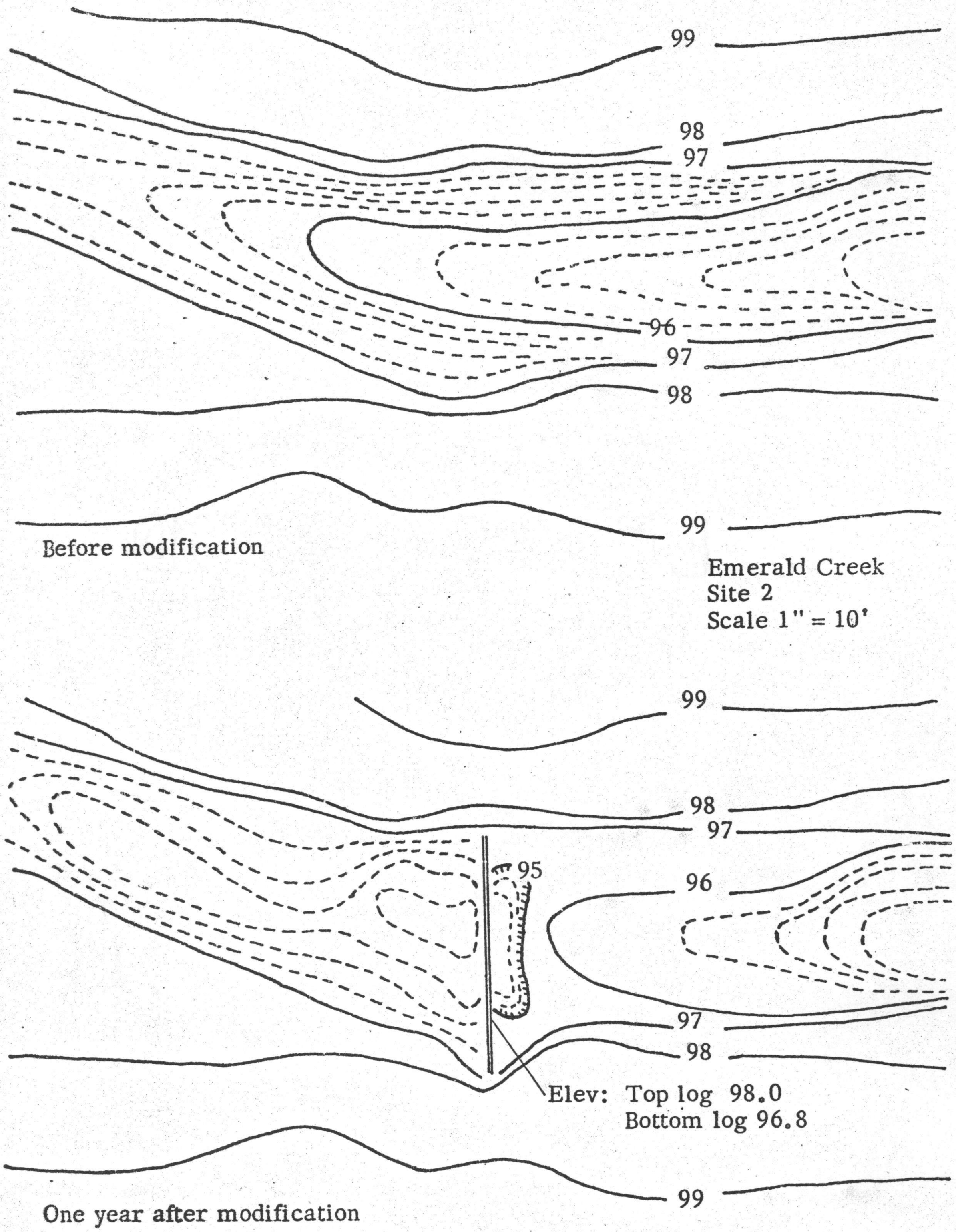


Fig. 31: Emerald Creek, Site 2, before and after modification with drop structure.

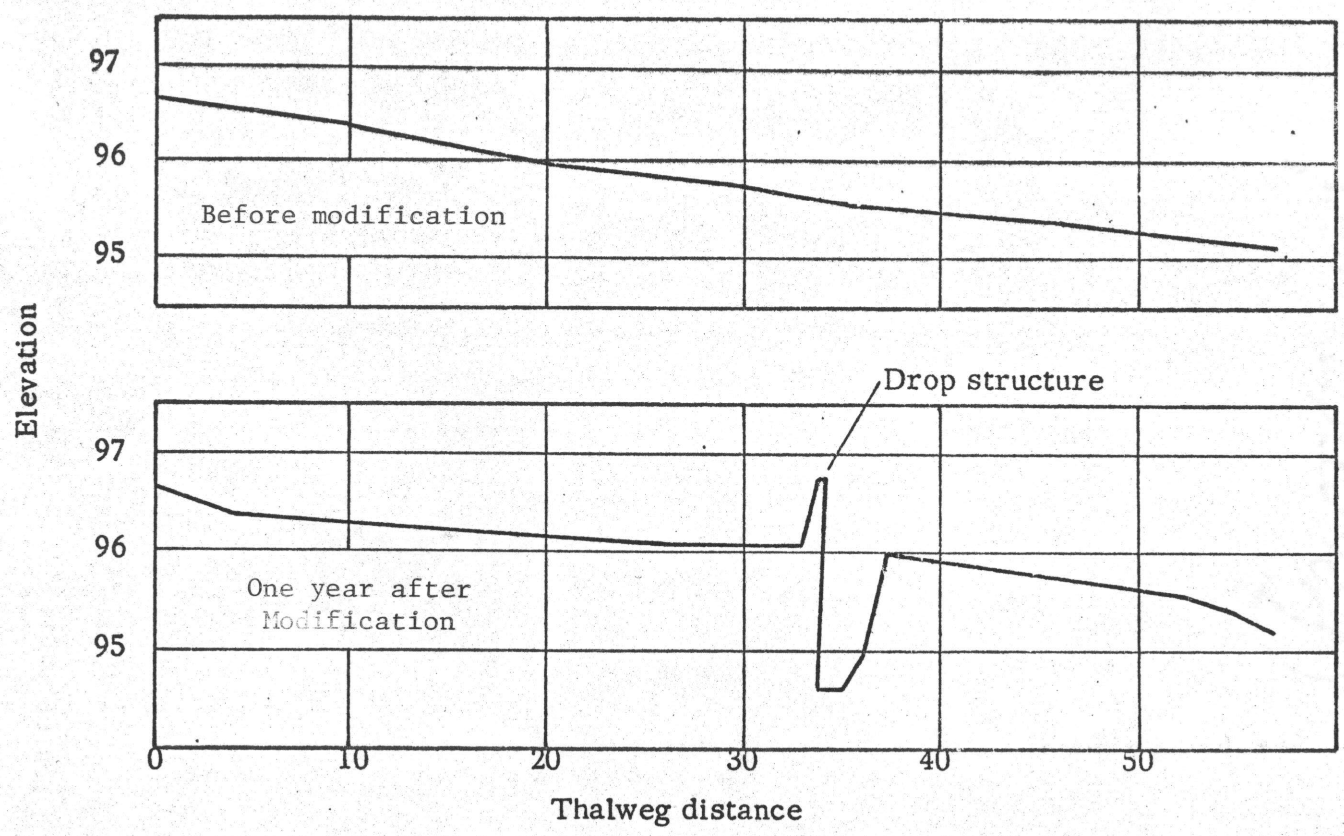


Fig. 32: Emerald Creek, Site 2, thalweg profiles.

RECOMMENDATIONS

While the use of stream modification structures to flush sediment from a reach is no substitute for reducing the sediment load through proper watershed management, this study shows that the modification structures are effective in the removal of sediment from a localized reach. To use natural resources it is at times difficult, if not impossible, to keep from adding to the sediment load of the stream. Therefore, it is recommended that additional research be undertaken to further improve on the use of stream modification structures to control siltation in mountain streams. Further research is recommened in three areas: 1) field application of groundwater standpipes for measuring siltation of a stream; 2) estimation of scour depth for a low drop structure (low P) and high depth of flow (high H); and 3) the use and design of low profile channel constrictors for removal of sediment from a reach.

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APPENDICES

APPENDIX A

Design computations for gabion constrictors at Site 3.

Part I: Force on gabions

Analysis:

The force on the gabion can be determined by using equation (4) which was derived from the momentum equation (3).

$$F_R = 1/2\, \gamma (2D_1 \Delta D - \Delta D^2) - \rho q^2 \left(\frac{1}{D_1 - \Delta D} - \frac{1}{D_1}\right) \qquad (4)$$

F_R, D_1, and ΔD are defined in Figure 6. ρ is the mass of water and q is the discharge per foot of width of stream.

Force calculations:

The variables in equation (6) are:

$\gamma = 62.4$ lb/ft^3 $\qquad \rho = 1.94$ slugs/ft^3

$D_1 = 4$ $\qquad q = \frac{Q}{W} = \frac{1400}{50} =$

$\Delta D = 1$ ft $\qquad$ 28 cfs/ft

Using equation (4):

$$F_R = 1/2\,(62.4)\,[2(4)(1)-1] - 1.94\,(28)^2 \left[\frac{1}{4-1} - \frac{1}{4}\right]$$

$$F_R = 91.6 \text{ lb/ft.}$$

Part II: Stability requirement

Analysis:

The force of the water acting on the gabion is resisted by the weight of the gabion acting through friction with the streambed. The rock in the gabion has a specific gravity of 2.65 and its submerged weight W_R is 100 lb/ft^3.

Referring to Figure A-1 and summing forces in the horizontal direction:

$$F_R = \frac{W_R \, W \, H \, C_F}{F.S.} \qquad \text{(A-1)}$$

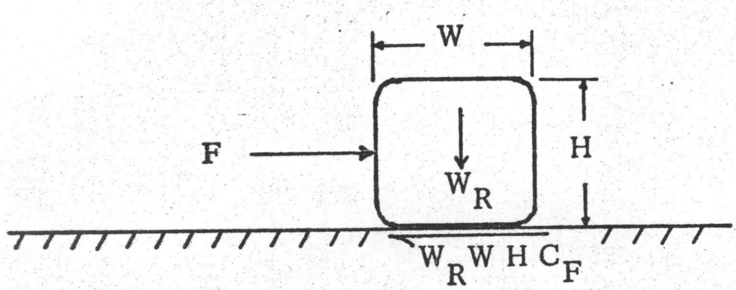

Fig. A-1: Forces on gabion.

Where in equation (A-1) F_R is the force of the water and was calculated to be 91.6 lb per foot of gabion, W is the cross sectional width of the gabion, H is the height of the gabion, C_F is the coefficient of friction between the gabion and the streambed, and F.S. is the factor of safety. The C_F used for design computations is 0.55.

Design H for stability:

Using equation (A-1):

$$F_R = 91.6 = \frac{W_R\ W\ H\ C_F}{F.S.} = \frac{(100)(3)(H)(0.5)}{2}$$

And

$$H = \frac{91.6(2)}{100(3)(0.55)} = 1.11 \text{ ft.}$$

Therefore, the necessary height of gabion to provide the needed resistance to the flow is 1.11 ft.

Part III: Constriction opening dimensions.

Analysis:

After determining to local channel slopes and specifying the designed d_{50} or d_{75} of the bed material, the design curves in Figures 9, 10, 11 or 12 are used to determine the flow depth necessary to provide sufficient tractive force to transport the smaller material from the reach.

The width of the opening W_2 of the constriction is calculated using equation (6).

$$W_2 = \left[\frac{D_1}{D_2}\right]^{5/3} W_1 \qquad (6)$$

In equation (6), W_1 and D_1 are the width and depth, respectively, of the low flow. W_2 and D_2 are the width and depth of flow through the constricted section during low flow.

<u>Determination of opening dimensions</u>:

1. The slope at Site 3 is 0.004 ft/ft.
2. The desired d_{75} of the bed material is 10 mm.
3. From Figure 12 the design depth D is 0.67 ft.
4. H>D, therefore use $H = D_2 = 1.11$ ft.
5. At low flow (Q = 20 cfs) $D_1 = 0.5$ ft and $W_1 = 50$ ft.
6. Using equation (6):

$$W_2 = \left[\frac{D_1}{D_2}\right]^{5/3} W_1 = \left[\frac{.5}{1.11}\right]^{5/3} (50) = 13.2 \text{ ft.}$$

7. The constriction opening design dimensions, as shown in Figure A-2, are:

$$H = 1.11 \text{ ft}$$

$$W_2 = 13.2 \text{ ft}$$

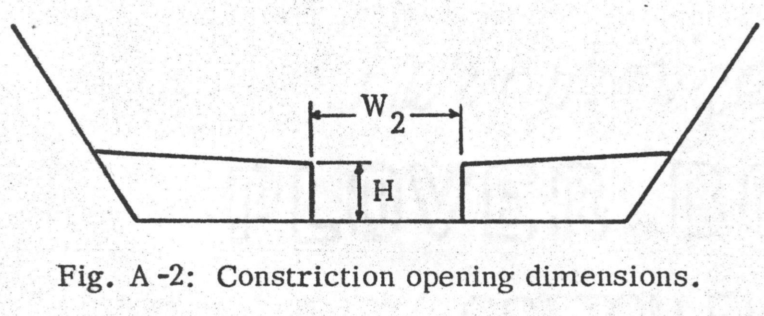

Fig. A-2: Constriction opening dimensions.

APPENDIX B

Design Computations for Log Drop Structures at Site 1

Part I: Embedment length of logs.

Analysis:

As shown in Figure B-1, the forces on the boards and logs of the drop structure include hydrostatic and soil pressures. Summing the moments about point B and equating to F_R results in:

$$F_R = 1/2.0\ [-0.667H_1 + 0.25W_1 + 0.167W_2 + 1.5H_2 - 2.97H_3 + 0.667S_1] \quad \text{(B-1)}$$

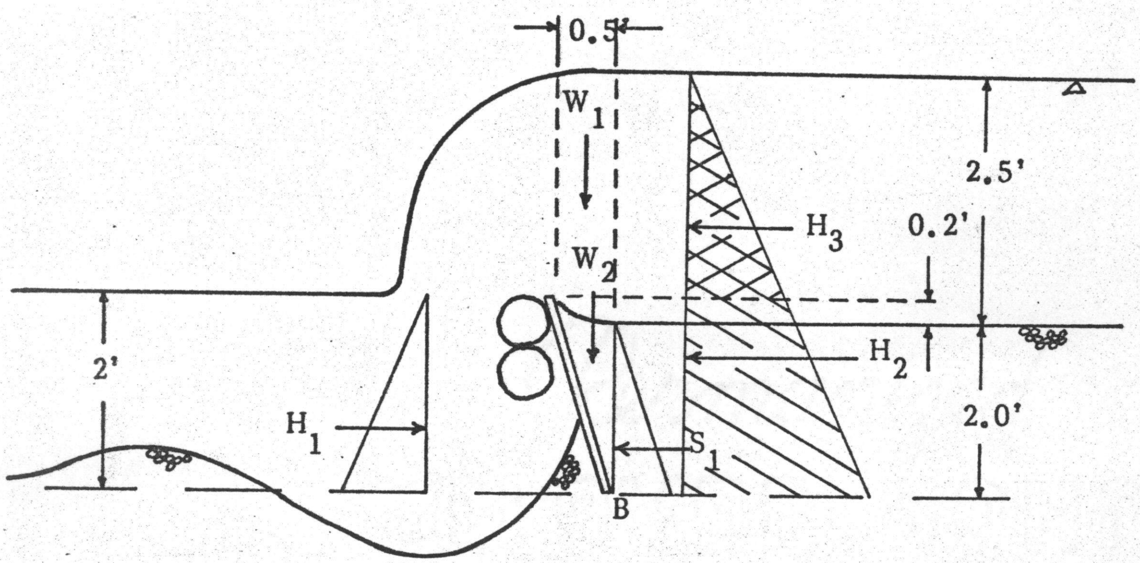

Fig. B-1: Forces on log drop structure.

In equation (B-1), F_R is the force in pounds on the drop structure per foot of width, H is the hydrostatic force, W is the force due to the weight of soil and water on the boards, and S_1 is the force resulting from the active soil pressure. S_1 is computed from equation (B-2):

$$S_1 = \frac{\gamma_{SB} h^2}{2} K_a \qquad \text{(B-2)}$$

The active pressure coefficient K_a is 0.333 for the sand and gravel streambed. The submerged specific weight γ_{SB} of the bed material is 75 lb/ft^3, and h is the depth from the soil surface to the bottom of the logs.

The resistance R in the logs per foot of embedment is computed using equation (B-3):

$$R = D \gamma_S K_p. \qquad \text{(B-3)}$$

In equation (B-3), D is the depth the logs are buried, γ_S is the specific weight of the streambank material, and K_p is the passive pressure coefficient for the soil.

The embedment length L_2 of the logs spanning a channel L_1 feet wide is determined using equation (B-4):

$$L_2 = \frac{F.S.\ F_R L_1}{2R} \qquad \text{(B-4)}$$

A factor of safety F.S. of 2.0 is used.

Embedment length computations:

1. With reference to Figure B-1, the forces on the logs are:

$$H_1 = 1/2 \; h_1^{\;2} = 1/2(62.4)(2^2) = 124.8 \text{ lb/ft}$$

$$H_2 = 1/2 \; h_2^{\;2} = 1/2(62.4)(4.5^2) = 632 \text{ lb/ft}$$

$$H_3 = 1/2 \; h_3^{\;2} = 1/2(62.4)(2.3^2) = 165 \text{ lb/ft}$$

$$W_1 = V \; \gamma_w = (.5)(2.5)(62.4) = 78 \text{ lb/ft}$$

$$W_2 = V(P\gamma_w + \gamma_{SB}) = 1/2(.5)(2)\,[.3(62.4) + 75] = 46.9 \text{ lb/ft}$$

$$S_1 = \frac{{}_{SB} \; h^2{}_1}{2} \quad K_a = \frac{75\,(2^2)}{2} \; (.333) = 50 \text{ lb/ft.}$$

In the above equations h is the height, γ_w is the specific weight of water, γ_{SB} is the specific weight of the submerged bed material, V is a volume, and P is the porosity of the soil.

2. The force F_R in pounds per foot of log is:

$$F_R = 1/2\,[-0.667\,H_1 + 0.25W_1 + 0.167W_2 + 1.5H_2 - 2.97H_3 + 0.667S_1] \qquad \text{(B-1)}$$

$$F_R = 435 \text{ lb/ft}$$

3. The resistance R on the logs per foot at embedment, for D = 2.5 ft, S = 125 lb/ft^3, and K_p = 3, is:

$$R = D\,\gamma_S\,K_p \qquad \text{(b-3)}$$

$$R = 2.5(125)(3) = 938 \text{ lb/ft.}$$

4. The embedment length L_2 of the logs, for F.S. = 2, and L_1 = 10, is:

$$L_2 = \frac{F.S.\ F_R\ L_1}{2R} \tag{B-4}$$

$$L_2 = \frac{2(435)(10)}{2(938)} = 4.6 \text{ ft.}$$

Therefore the embedment length L_2 is 4.6 ft.

Part II: Estimated depth of scour at downstream drop structure of Site 1.

Analysis:

The dimensionless curves of Figure 34 are used to estimate the depth of scour D_S below the drop structure. The variables D_S, D_2, H, P, and d_m, respectively, are depth of scour, depth of tailwater, hydrostatic head above the drop structure, height of the drop structure, and expected mean diameter of the stones after the fine material has been washed away. The variables are shown in Figure 32.

Estimation of scour depth:

1. The variables are:

D_2 = 2 ft

H = 2.3 ft

P = 2 ft

d_m = 0.67 ft

2. The dimensionless variables are:

$$P/d_{50} = 3$$

$$D_2/P = 1$$

$$H/P = 1.2.$$

3. The corresponding value for D_S/P comes from Figure 34:

$$D_S/P = 1.2.$$

Solve for the scour depth D_S:

$$D_S = 1.2P = 2.2 \text{ ft}.$$

The estimate depth of scour is 2.2 ft.

APPENDIX C

Size distributions for sand-gravel mixes

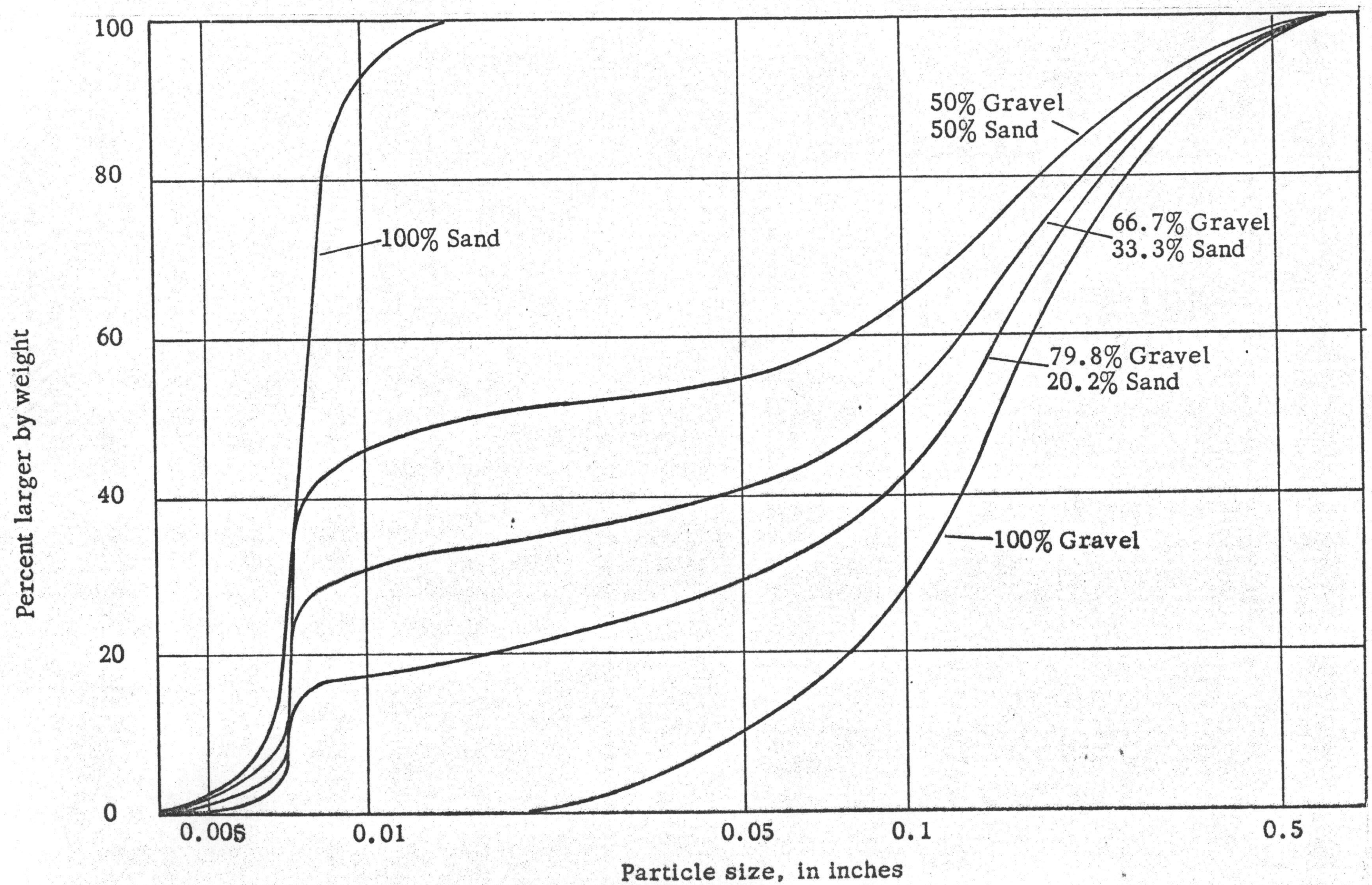

Fig. C-1: Size distributions for sand-gravel mixes.