March 1965

U. S. Forest Service
Research Paper RM-12

MULTIPURPOSE PREFABRICATED CONCRETE CHECK DAM

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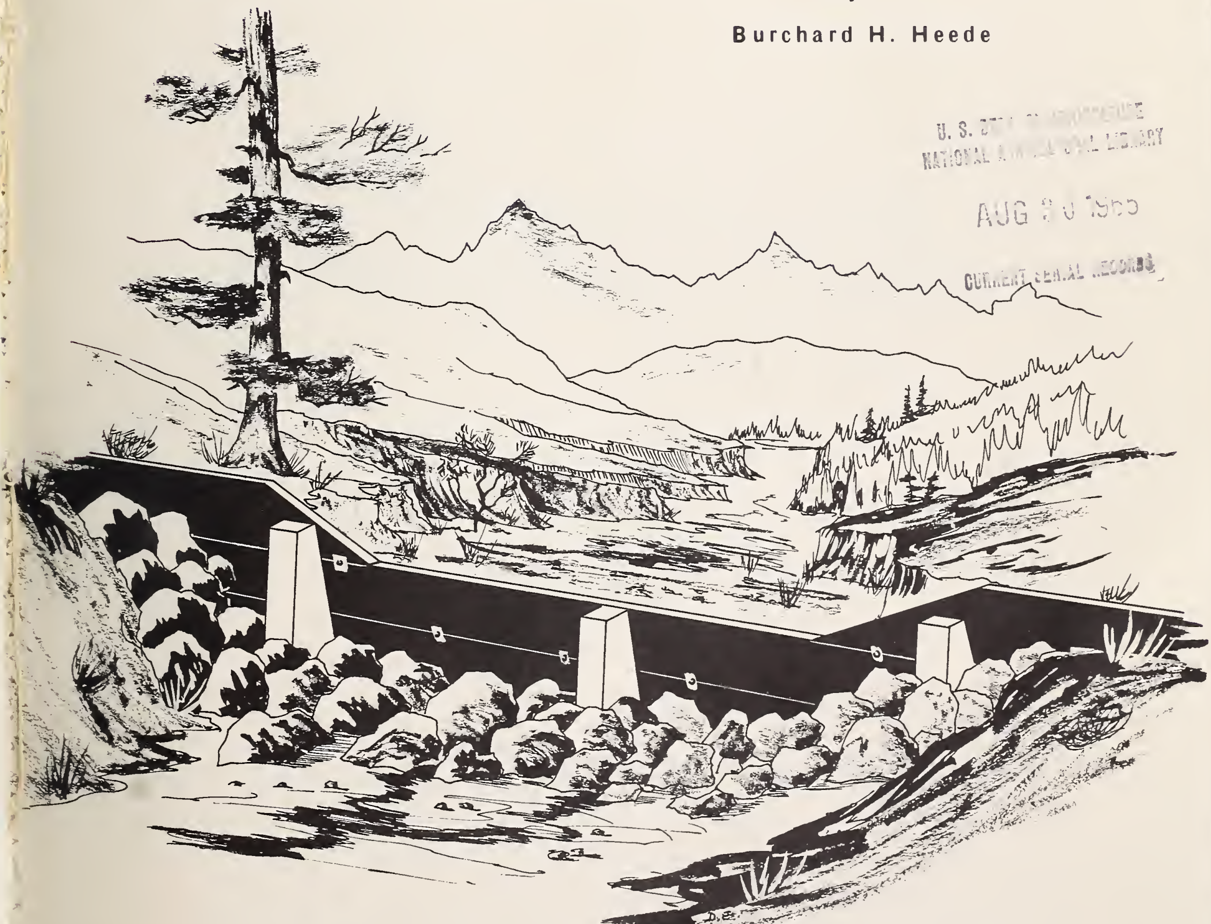

ROCKY MOUNTAIN FOREST AND RANGE EXPERIMENT STATION
RAYMOND PRICE, DIRECTOR FORT COLLINS, COLORADO
FOREST SERVICE U. S. DEPARTMENT OF AGRICULTURE

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NEED FOR A NEW DESIGN

A new design was needed to simplify the construction of check dams in fairly inaccessible mountain forest areas. Often suitable materials are not available locally, or check dams built from these local materials require various types of equipment, large amounts of labor, and intensive supervision.

A prefabricated, partially prestressed concrete check dam[2] has been designed, built, and installed in a gully in western Colorado (fig. 1). This type of structure should prove useful for future gully control projects, stream gage cutoff walls, and other applications.

[2]*Prestressed Concrete of Colorado, Inc., Denver, helped in the design of the concrete elements. Mention of commercial enterprises is solely for necessary information; no endorsement by the U. S. Department of Agriculture is implied.*

DESIGN OBJECTIVES

The objectives for the design were to develop a structure (1) composed of units of predetermined size, (2) suited to mass production, (3) requiring a minimum of labor, machinery, and supervision, and (4) adaptable to a wide variety of sites and situations.

The feasibility of gully control is to a great extent a cost problem. This is especially true for the remote mountain areas of the National Forests in Colorado, where present land values are low and where flood flows do not directly endanger human life and property. Therefore, cost aspects also guided the design of the prefabricated check dam.

Naturally, costs are always involved in or influenced by a design. Yet, some cost-design problems are easy to deal with while others offer great obstacles, especially if viewed in terms of years. Thus, it was relatively easy to cope with cost factors such as transportability of a prefabricated structure, lifting and

Figure 1.-- Looking upstream on prefabricated check dam. Height of rod leaning against dam is 5.5 feet.

placement of structural units, and number of individual units per check dam. It was decided that the prefabricated dam should be assembled in the field. that the individual unit should not weigh more than 2 tons, that one machine (backhoe) should be able to handle the units, and that the number of concrete units should be small.

But solving other cost-design problems, such as those connected with stability aspects and factors of safety, offers greater difficulties. Where life and property would be endangered by the failure of a structure, extensive effort to evaluate all site characteristics pertinent to the safe design of the structure are justified, and large safety margins in the design itself are conventional and proper. In general, structures designed for water storage entail some jeopardy to life and high-cost property. It is also true, in general, that their structural stability remains static, or deteriorates with age, so that the initial design must be adequate to withstand the full stresses of anticipated events indefinitely or over some specified life of the structure.

On the other hand, possible failure of gully control structures in wildlands does not usually jeopardize life or valuable property. Also, and very significantly, their design stability is augmented in time by the very nature of their function--that of trapping sediment to the point where the structure crest becomes a new level of the upstream gully floor. Thus an effectively designed and placed check dam should be exposed to the full magnitude of design stresses only during an initial period of time, and one so limited that there should be very low probability of an event with destructive potential.

These considerations have been incorporated into the design of the prototype check dam, and into the general recommendations for such structures. It is believed that they form a sound basis for a realistic design for check dams from the viewpoints of reliability and economics.

THE PROTOTYPE

A slab-buttress structure offers great advantages in the design and installation of a multipurpose check dam. Such a structure was designed of nine major parts: six 3-inch thick prestressed wall slabs, manufactured from conventional concrete, and three buttresses with footing, formed into one unit each from lightweight reinforced concrete (fig. 2). The use of lightweight aggregates saves one-third of the weight as compared with conventional concrete. The heaviest part of the structure (outside buttress) weighs 1.7 tons. Total weight of the dam is 9.8 tons. The overall length of the check dam is 45 feet; its effective height is 4 feet. Depth of freeboard is 2.5 feet.

Location

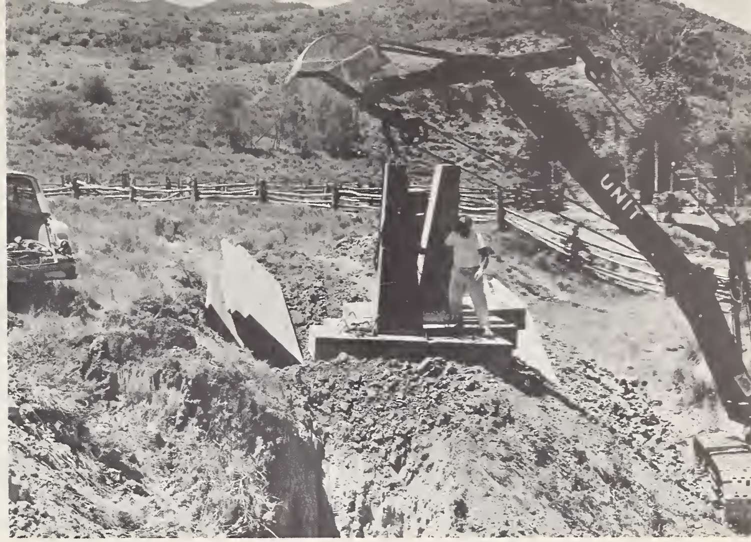
The prototype check dam was installed in July 1963 in the main gully of a watershed on the western slope of the Rocky Mountains about 40 miles southwest of Glenwood Springs, Colorado (fig. 3). The watershed area above the dam is approximately 1 square mile; elevation ranges from 7,500 to 8,500 feet. Soils are derived from shales and sandstones of the Wasatch formation. A tendency for subterranean erosion or "soil piping" is an outstanding characteristic of the alluvial soils of the main valley bottom. In selecting a site for the dam, a location was chosen where the soils did not "pipe." The vegetation consists mainly of Gambel oak, big sagebrush, Kentucky bluegrass, and western wheatgrass. Sagebrush occurs predominantly on the bottom lands and the south slopes. Gambel oak occupies ridges and the north slopes.

Records from 1961 to the present show that the area received an average total precipitation per water year (October 1 to September 30) of 16.5 inches. Compared with longer records from nearby stations, this amount appears to be close to normal. Gully flow is ephemeral, and occurs during spring snowmelt and exceptionally intense summer storms. The load of these flows consists mainly of fines and sand; gravel and boulders are present only occasionally. Flow was not measured before the dam was installed.

Placement

The check dam can be placed within 3 hours, not counting time for excavation and

Figure 2.--
The units were placed on the gully bank to facilitate handling by the backhoe.



backfill. A backhoe (or a similar type of equipment) and only two laborers are required. The machine (1) excavates the foundation, (2) excavates the keys in the channel bottom and side slopes, (3) installs the units, and (4) backfills. Steel loops, attached to all concrete units during manufacture, facilitate their handling. No hole-to-hole fitting is required for the placement of the units. Buttresses can be alined horizontally by eye, but a hand or engineer's level should be used for the vertical alinement.

The upstream face of the buttresses is inclined to an angle of about 10 degrees with the vertical, and causes the wall to lean downstream. This inclination of the wall facilitates installation of the individual slabs, since gravity holds the slabs against the buttresses. The sloping wall also increases the stability of the dam by adding the weight of the water to the structure (see W_{W_2}, fig. 7). To prevent slippage at the base of the wall, the footing adjacent to the buttress is grooved (fig. 4). Steel plates and angle irons hold the upper wall and the freeboard slabs in place during the backfill operations, and add some stability to the dam wall.

INSTALLATION PROCEDURES

Construction plans are shown in figure 5, pp. 8-9. Step-by-step procedures are:

Figure 3.--
Note that the channel side slope was disturbed as little as possible during excavation for the key and buttresses.

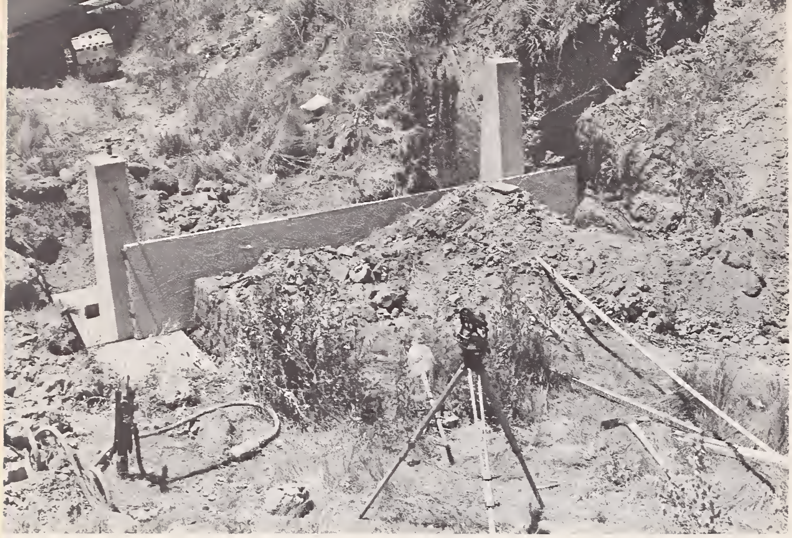


Figure 4.--
The first wall slab is set into a groove in the footing on the upstream side of the buttresses.

1. Excavate for keys and foundation slabs. Place all structural parts on the gully bank.
2. Install one outside buttress and the center buttress. Set a bottom wall slab against these buttresses so that one end of the slab touches the bolts protruding on the upstream face of the center buttress.
3. Place the second outside buttress (use the first bottom wall slab to align this buttress). Install the second bottom wall slab in the same manner as the first. With hammer and chisel, provide grooves on the upper side of the bottom wall slabs to accommodate a 3/8-inch bolt (spacing indicated in fig. 5). Attach two steel plates each to all bolts and tighten nuts loosely.
4. Place the next two wall slabs.
5. Fasten steel plate on the bolts of the center buttress and tighten nuts on all other bolts. Make grooves on the upper side of the dam wall for the accommodation of 3/8- and 5/8-inch bolts, respectively.
6. Loosely fasten angle irons and steel plates to the bolts.
7. Place freeboard slabs on the dam wall between the angle irons and steel plates (fig. 6).
8. Tighten all loose nuts.
9. Drive at least two steel posts into the ground behind the downstream side of the foundation slabs if on bedrock foundation.
10. Backfill structure in layers. Compact each layer before the next is applied with machine operated compactor if possible.
11. Excavate for the installation of apron and bank protection device below the dam. Deposit excess material upstream from the dam.
12. Install apron and bank protection work below the dam with a length not shorter than 1.5 times the effective dam height.

Figure 6.--
Looking at the upstream face of the completed dam before it is backfilled.

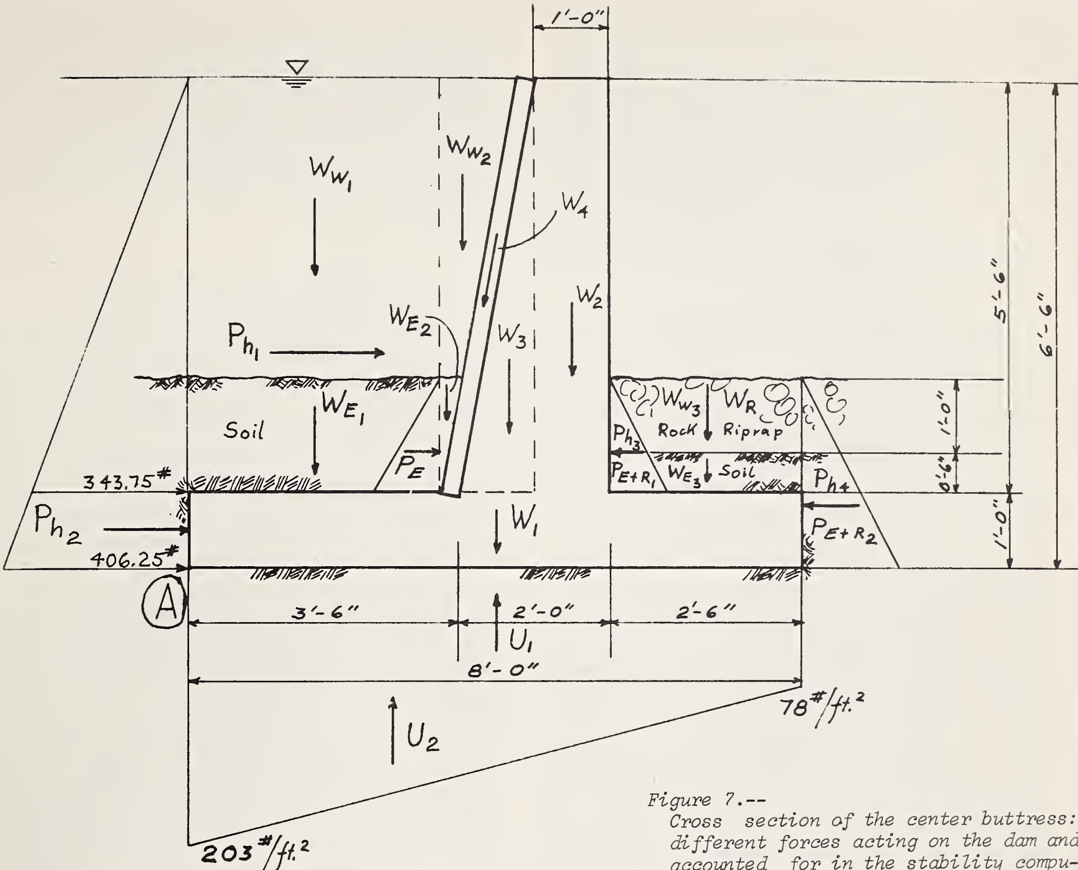


Figure 7.--
Cross section of the center buttress: different forces acting on the dam and accounted for in the stability computations are illustrated. The symbols in the drawing are explained below.

W_1, W_2, W_3	= *Weight of components of buttress.*
W_4	= *Weight of wall.*
W_W	= *Weight of water.*
W_E	= *Buoyant weight of earth.*
W_R	= *Buoyant weight of rock riprap.*
P_{h_1}, P_{h_3}	= *Hydrostatic force against wall.*
P_{h_2}	= *Hydrostatic force against footing of buttress.*
P_{h_4}	= *Hydrostatic force against buttress.*
P_E	= *Active soil force on upstream face of wall.*
$P_{E\,+\,R_1}$	= *Passive force of soil and riprap on downstream face of wall.*
$P_{E\,+\,R_2}$	= *Passive force of soil and riprap on downstream face of buttress.*
U_1, U_2	= *Components of uplift.*

DISCUSSION AND APPLICATION

Stability Computations

Stability computations on the prototype (see appendix, p.14) considered overturning, foundation pressure, and sliding. Since the outside buttresses of the structure are partially embedded in the channel side slopes, these computations are based on the conditions at the center buttress. The design of the individual components of the structure is beyond the scope of this paper.

Overturning and Foundation Pressure

Figure 7 presents schematically the different forces acting on the center buttress and dam wall that were used in the stability calculations.

The following vertical forces and their moments about A were computed: dry weight of the components of the check dam; weight of the water acting on the dam; buoyant weight of the soil placed on the upstream and downstream buttress footing; buoyant weight of the rock riprap located on the downstream footing of the buttress; and uplift.

Since uplift was accounted for by assuming 50 percent saturation of the foundation by water, the dry weight of the structural components was used. The applied uplift factor of 0.50 is believed to be rather high for the particular location of the check dam, but if evaluated in the light of a complete gully treatment, the factor appears to be realistic.

It was assumed that the soils on both sides of the dam and the rock riprap would be fully saturated by water. Buoyant weights were therefore used. The buoyant weights were calculated by subtracting the weight of water from the estimated weight of the materials.

The following horizontal forces and their moments about A were considered: (1) the hydrostatic head of the water upstream and downstream from the dam; (2) the active soil pressure on the upstream face of the dam wall; and (3) the passive pressure of the soil and rock riprap against the downstream side of the dam.

The hydrostatic head upstream from the dam was taken at 6.5 feet. This represents a safety margin in the stability computations, since the actual hydrostatic head decreases considerably at the lower 2.5 feet where open channel flow does not exist.

The center buttress was estimated to take half of the load acting on the wall between the outside buttresses. In the computation of this load, the bottom wall slab was used. This slab is cantilevered beyond the outside buttress for 5.5 feet (see appendix, p.15). The calculations of the hydrostatic force, acting against the footing of the center buttress (Ph_2, fig. 7), were simplified by assuming a rectangular force diagram.

Based on the expected magnitude of flow and the steep channel gradient (6 percent), it was assumed that tail water will not occur at the structure, and the hydrostatic head on the downstream side of the dam is given by the saturation water. The resultant force was calculated by the Rankine formula based on active and passive pressures of soil and riprap:

$$P_E = \frac{h^2 \times \gamma b}{2} \tan^2 \left(45^\circ \pm \frac{\theta}{2}\right)$$

where P_E is the active or passive soil force on the upstream or downstream face of the dam, respectively (the "plus" sign denotes the equation for the passive and the "minus" sign the equation for the active pressure), h the depth of the material, γb the buoyant weight of the material, and θ the angle of repose. To simplify the calculations, one angle of repose for both soil and riprap was assumed.

The location of the resultant of the external forces was determined by dividing the sum of the moments about A of all vertical and horizontal forces by the sum of the vertical forces. The result shows that the resultant is located 6.09 feet to the right of A or 0.76 foot outside of the middle third section of the structure. Because stability conditions usually improve with time at a check dam, the resultant will shift toward the center of the dam.

When the maximum foundation pressure is considered in the light of the allowable pressure, the present location of the resultant appears to be acceptable. The pressure at the base of the structure was computed as follows:

$$q = \frac{\sum V}{BL}\left(1 \pm \frac{6e}{L}\right)$$

where q is the pressure at base, $\sum V$ the sum of all vertical forces acting on the dam, BL the area of the base of the structure, e the eccentricity denoting the distance between the midpoint of the structural base and the location of the resultant, L the length of the base, and the "plus" and "minus" sign the maximum or minimum pressure, respectively. The maximum foundation pressure was found to be 1,184 pounds per square foot--a fraction of the estimated allowable pressure of 20,000 pounds per square foot. The foundation material was a sandstone member of the Wasatch formation. This rock is horizontally jointed, has some vertical seams, and is very hard where not exposed to long weathering processes.

Sliding

The shear available at the base of the check dam was obtained by correcting the sum of all vertical forces with an estimated coefficient of friction. The value of this coefficient depends largely on the type of the foundation. A sliding factor of 1.2 was obtained when the available shear was divided by the sum of all horizontal forces. Ideally, the dam would not slide if the factor were 1. Yet, a safety margin is desirable. Conventionally, a sliding factor of 1.5 or larger is considered adequate for safety. In the light of general gully control aspects, the given sliding factor of the check dam is regarded as adequate.

Ways to Increase Safety Factors

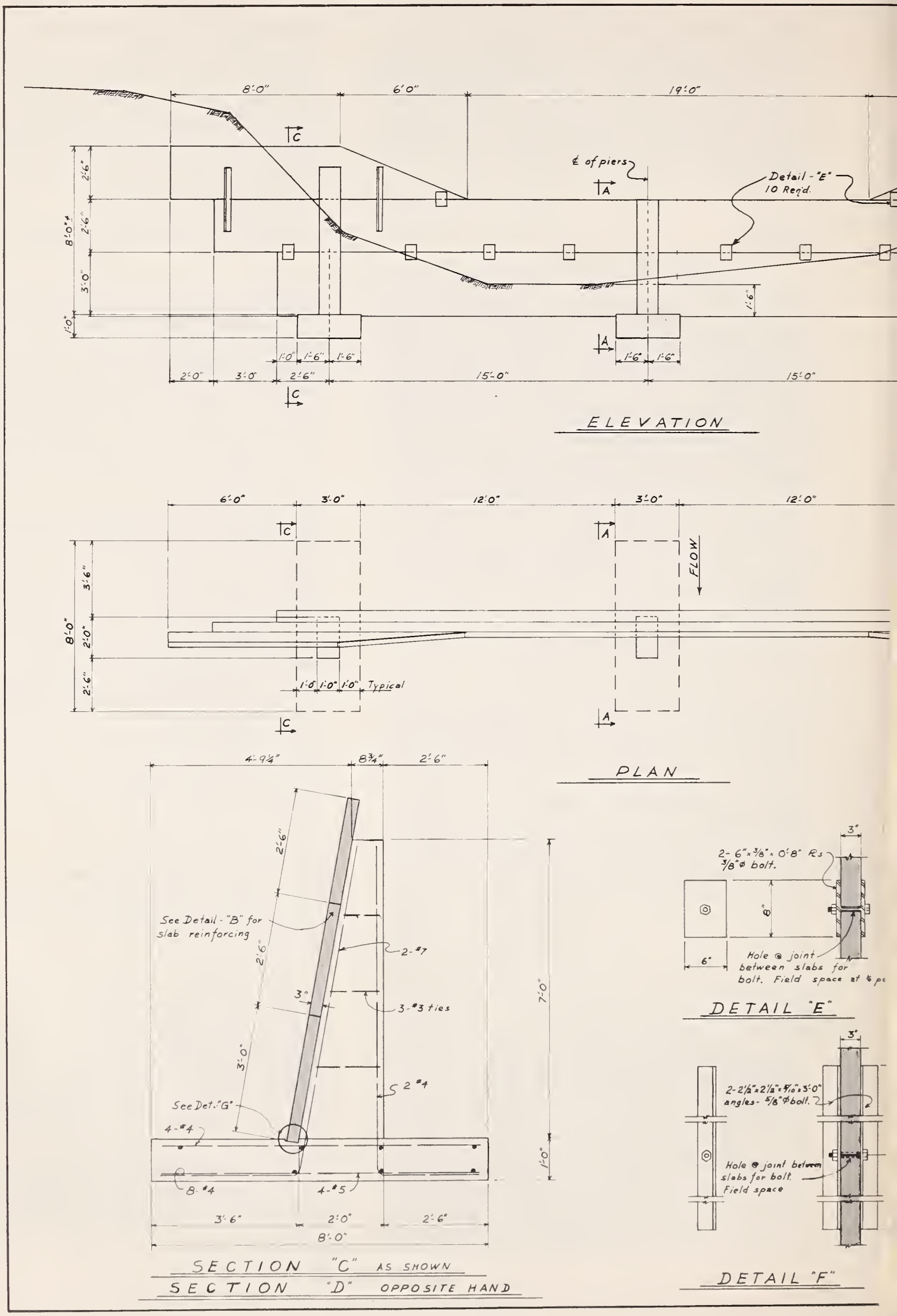
When doubt exists about the bearing capacity of the foundation, the resultant should be shifted into the middle third section by a change in the design. A very undesirable foundation would be presented by swelling clays, for instance. Just a few proposals shall be made here; a multitude of measures would lead to increased stability. For example, an additional foot of riprap could be added to the apron of the dam to provide a total rock depth of 2 feet. This would raise the sliding factor to 1.83, and would place the resultant of the external forces 4.66 feet to the right of point A or well into the middle third of the structure. Similar effects could be obtained by shortening the distance between the buttresses. Safety factors on stability could also be increased by changing part of the structure such as the connection of a key to the footing of the buttresses. The key would increase the vertical forces (acting against sliding) and it would decrease the uplift forces (again benefiting the forces against overturning and sliding). The use of conventional concrete in the buttresses, the placement of weep holes into the buttress footings, or enlargement of the footings would be other possible measures. One may also consider placing the structure on interlocking steel sheet piling or any other type of cutoff wall. To obtain greatest benefit, the piling should be watertight.

Keying the Structure to Gully Side Slopes

The main critical locations on the prefabricated check dam exist where the ends of the wall are buried in the banks, usually called the keys of the structure in the channel side slopes. Movement of the water around the ends of the wall, determined primarily by the head of water, the permeability of the soils, and the length of the keys, may lead to piping if the fines are washed out of the bank material. Investigations on older check dams have shown, however, that, in ephemeral streams, the stability of the keys is mainly endangered by the scouring action of the water on the downstream side.[3] Here, the direct impact from the waterfall or the development of eddies may cut the bank, with resultant loss of key and structure, if it is not adequately protected. Failures of check dams by piping were not recorded. These findings are not surprising. Depositions that occur with time on the gully bottom and banks above the check dam have a sealing effect, lengthen the route of the seepage water, and thus counteract the seepage forces. In contrast, on the downstream side of the dam, the water flow overtopping the structure gnaws on the channel side slopes. Therefore, an efficient protection of the side slopes below a check dam is more important for structural stability than the length of the keys.

Rock riprap, if used correctly, effectively protects gully side slopes. It provides extreme surface roughness to dissipate the energy of the falling water, and has a tendency to close larger voids that may occur. Along with other factors, gradation and angularity of the rock are very important for efficient use of riprap.

[3] *Heede, Burchard H. A study of early gully-control structures in the Colorado Front Range. U. S. Forest Serv. Rocky Mountain Forest and Range Expt. Sta., Sta. Paper 55, 42 pp., illus. 1960.*



8'-0"
6'-0"
19'-0"
C
¢ of piers
A
Detail - "E"
10 Req'd.
2'-6"
2'-6"
8'-0"±
3'-0"
1'-0"
1'-6"
1'-0"
1'-6"
1'-6"
1'-6"
1'-6"
2'-0"
3'-0"
2'-6"
15'-0"
15'-0"
ELEVATION
6'-0"
3'-0"
12'-0"
3'-0"
12'-0"
FLOW
3'-6"
8'-0"
2'-0"
2'-6"
1'-0 1'-0" 1'-0" Typical
PLAN
4'-9¼"
8¾"
2'-6"
2'-6"
See Detail - "B" for slab reinforcing
2'-6"
2-#7
3"
3-#3 ties
3'-0"
7'-0"
2 #4
See Det. "G"
4-#4
1'-0"
8-#4
4-#5
3'-6"
2'-0"
2'-6"
8'-0"
SECTION "C" AS SHOWN
SECTION "D" OPPOSITE HAND
3"
2- 6"x3/8"x 0'-8" Rs
3/8"ø bolt.
8"
6"
Hole @ joint between slabs for bolt. Field space at ½ pc
DETAIL "E"
3"
2-2½"x2½"x5/16"x3'-0" angles- 5/8"ø bolt.
Hole @ joint between slabs for bolt. Field space
DETAIL "F"

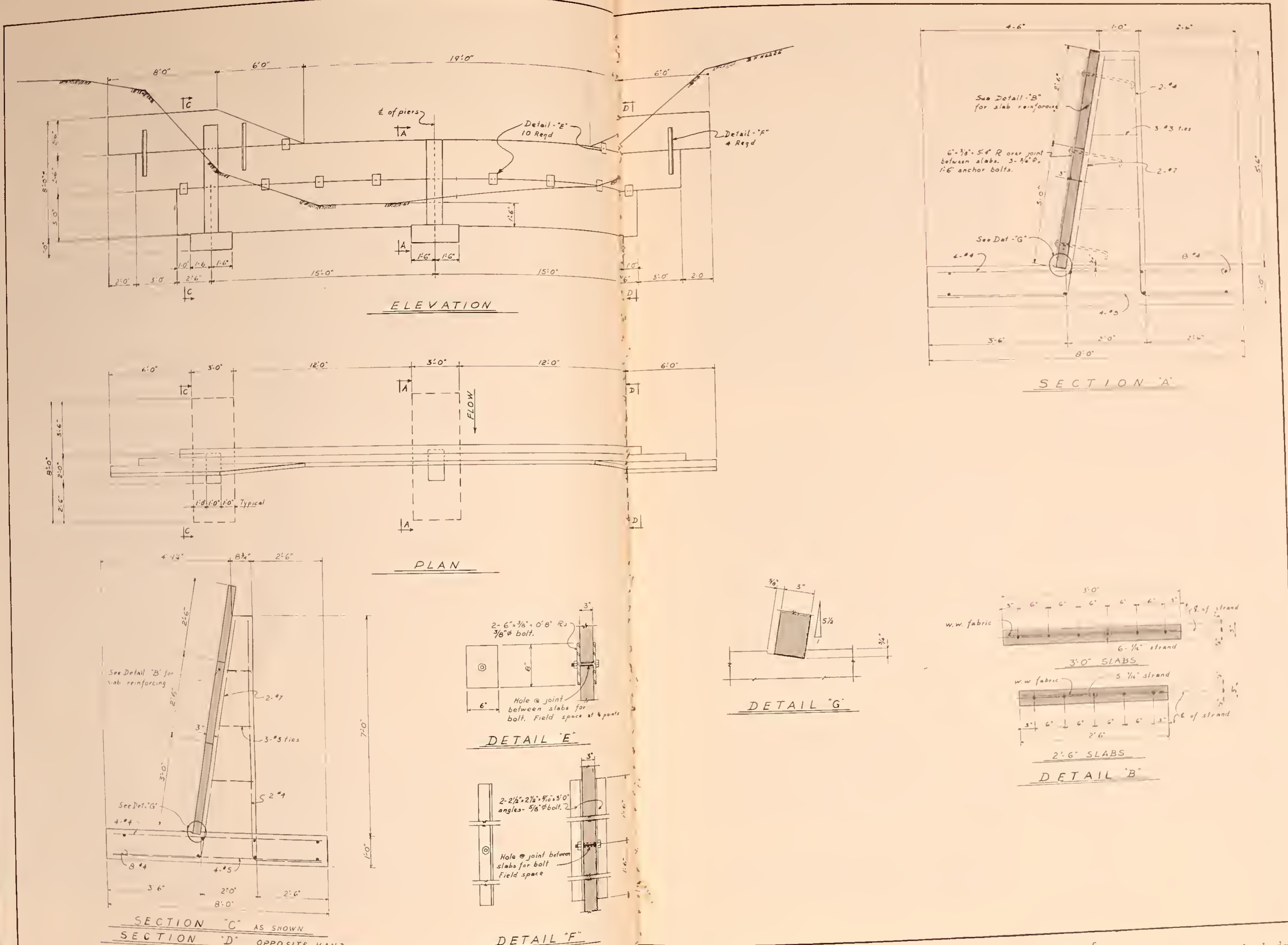


Figure 5.--
Detailed plan for concrete check dam.

Well-graded rock prevents the occurrence of larger openings in the riprap during placement, and angularity of the material provides an anchorage between the individual rocks. Large openings in the riprap should be eliminated under all circumstances, otherwise wave actions may attack the underlying bank material and remove it.

Seepage pressure and the safety of the keys against piping can be analyzed by the flow net method. To yield meaningful results, this method should be applied by designers with considerable experience in the field of soil mechanics. In general gully control work, such computations do not appear to be justified. Characteristics of soils vary greatly over short distances along the banks of alluvial streams. To obtain the data needed for the computation, extensive field sampling and office work would be required that are out of proportion to the size of the structures involved.

Our experience indicates that a key length of 4 feet in clayey soils and 6 feet in sandy soils is adequate (see fig. 3), but a final decision on these values should be made after site inspection.

If the dam will be installed in a sand bed, selected material should be used for backfilling on the upstream side of the structure. Clayey soils would best fulfill the purpose to lengthen the seepage route of the water around the structure.

Textbooks on soil mechanics give values for the coefficient of permeability, k, of broad soil classification groups. These table values can be used as a general guide to the order of magnitude of the permeability of a given soil. If, for instance, a porous soil is indicated, perhaps when k exceeds 100 feet per year, supplemental measures may be required. An effective device to prevent fines from washing out from around the keys is the reverse filter. Such a filter is established by placing materials, graded from sand to fine gravel, underneath the rock riprap on the channel side slopes.

Other measures exist to counteract seepage where it may create serious problems. The application of swelling clays or cement grout to the channel side slopes upstream from the dam has the same effect as the beneficial natural silting processes expected to take place with time.

While installing the keys of the check dam, there should be minimum disturbance of adjacent channel side slopes because previous sorting of surface materials has clogged many of the voids in these slopes. Destruction of this natural arrangement will increase the vulnerability of the banks to erosion until a certain quasi-equilibrium between a given flow and the side slopes is established again.

On our study area, certain soils seem to exhibit a pronounced tendency for "soil piping." Since little is known about the mechanics and the origin of this type of subterranean erosion, no remedy to this phenomenon can be given. It is advisable to avoid locations where soils appear to be susceptible to this erosion.

Suitability for Mass Application

The given design of the check dam is applicable to gullies of different widths without any change, but the maximum spacing between buttresses should not exceed 15 feet. The length of the individual wall slabs can easily be adjusted without increasing costs, since prestressed slabs are usually formed in beds several hundred feet long. Length of slab will be restricted only by aspects of transportability and weight. The application of several different widths of slabs will increase costs, however, since each width requires a separate bed. In the design of the prototype, only two widths, 2.5 and 3 feet, were used.

Any increase in the effective height of the check dam necessitates redesign. The stability computations on the prototype indicate that an increase in the height of the center buttress without other changes in the design may place the resultant overturning force into a position that would make the dam unsafe. The design may be altered as shown before.

From the discussion, it follows that a mass fabrication on the basis of the prototype design is feasible if the conditions of the individual treatment locations in the gully are considered. Mass fabrication is highly desirable. It is estimated that the manufacture of 15 or more of the prototype dams would cost $1,000 each.

The geometry of gullies usually varies greatly over short distances. These variations in gully dimensions may allow the selection of locations that not only satisfy the requirements for the placement of the dams as dictated by structural spacing or foundation conditions, but may also be suitable to a dam with a given design.

The design of the prototype has been satisfactorily tested for mass application in a treatment plan for a gully more than three-fourths of a mile long. At the proposed structural sites, the depth of the gully ranges from approximately 12 to 28 feet and the width from about 25 to 40 feet.

Applicability to Weirs

The design of the check dam is also applicable to different types of weirs. In fact, it appears to be advantageous to use prefabricated concrete units in weir construction because of the cost savings. Forming cutoff walls and pouring concrete in the field are time consuming and expensive. If the design is used for stream gages, however, the requirements for the factors of safety against overturning and sliding should be increased. Alterations should be made, as discussed previously, to shift the resultant force against overturning into the middle third section of the dam, and to raise the sliding factor to 1.5. It will always be desirable to place the structure on some type of a cutoff wall such as steel piling, concrete blocks or slabs if a bedrock foundation does not exist.

The prototype represents a trapezoidal broad-crested weir. The spillway could have other shapes or be fitted with a sharp-crested weir blade. If a center buttress is required, the height of this buttress should be lowered and the top slanted to assure free flow over the weir.

Seepage is undesirable for stream gaging. Products such as swelling clays and gunite may be used to close voids between the structure and the bedrock. To eliminate or decrease the magnitude of the cracks between the individual wall slabs, it may be advisable to manufacture the wall in one or two units only.

The weight of a stream gage wall, consisting of one or two units, can be illustrated by the prototype: its wall is 45 feet long at its maximum extension, 5.5 feet high at spillway elevation, and weighs 4.8 tons. If lightweight aggregates were used, this wall would weigh 3.2 tons, or each half 1.6 tons.

RESULTS

The ideas on the design and construction of the check dam, expressed earlier, were tested during the flows from spring snowmelt and found to work.

In April 1964, the peak flow of the melt season occurred with a head of 0.42 foot over the spillway of the dam. This head was estimated roughly to correspond to a discharge of 20 c.f.s. Depth-integrated samples of suspended sediment, taken from the stream near dam site, showed a concentration of 21,000 p.p.m. Gravel and boulders were present only occasionally in the total sediment load of the flow. The load consisted mainly of fines and sands.

Shortly after the peak flow had passed, the catchment basin of the dam was filled with sediment deposits to the crest of the spillway (fig. 8). It was estimated that this sediment amounts to 4,000 cubic feet or 140 tons. Runoff continued for 5 weeks, keeping the sediment above the dam saturated.

Thus, the hydrostatic forces of the water and the saturated soil pressure of the deposits acted simultaneously on the dam, and the structure passed through the most severe test it will probably ever experience. Yet, no displacement of the structural units or erosion at the dam site took place. The dam, the gully side slope protection installed below the dam, and the apron withstood all forces. Maintenance will not be required during the coming season.

Figure 8.--
Looking upstream on the prototype during the recession flow of the spring snowmelt. The head of the flow with reference to the crest of the spillway is 0.20 foot, as measured at the crest gage near the left margin on the figure. Sediment has filled the catchment basin of the dam to the crest of the spillway.

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APPENDIX

GENERAL DESIGN DATA AND ASSUMPTIONS

Weight of water	= 62.5 lbs./cu. ft.
Dry weight of soil	= 110 lbs./cu. ft.
Buoyant weight of soil	= 47.5 lbs./cu. ft.
Dry weight of rock riprap	= 125 lbs./cu. ft.
Buoyant weight of rock riprap	= 62.5 lbs./cu. ft.
Weight of reinforced lightweight concrete	= 100 lbs./cu. ft.
Weight of reinforced conventional concrete	= 150 lbs./cu. ft.
Angle of repose of rock riprap	= 35°
Angle of repose of soil (angle assumed at high value for convenience; true angle ranges between 27° and 30°.)	= 35°
Coefficient of friction between dam and bedrock foundation	= 0.80
Uplift factor (bedrock foundation seamy)	= 0.50
Allowable foundation pressure (bedrock with seams)	= 20 K/sq. ft.

NOMENCLATURE USED IN THE CALCULATIONS

B = Width of dam foundation.

$\sum H$ = Sum of all horizontal forces.

L = Length of dam foundation.

$\sum M_A$ = Moment of all external forces about point A.

$\sum M_a$ = Moment of all external forces about the outside buttress a.

P = Horizontal forces of water, soil, or soil and rock riprap.

R = Shear available along base (sum of all vertical forces corrected by coefficient of friction).

U = Uplift forces.

$\sum V$ = Sum of all vertical forces.

V_b = Vertical reaction from one side of the dam wall on the center buttress b.

W = Weight of water, soil, or rock riprap.

e = Eccentricity (horizontal distance between midpoint of structure and location of resultant of the vertical and horizontal forces).

f = Coefficient of friction between the dam and the foundation.

h = Head or depth of water upstream or downstream of the dam or depth of soil.

q_{max} = Maximum pressure of structure exerted on foundation.

γb = Unit buoyant weight of soil or soil and rock riprap.

θ = Angle of repose of soil and rock riprap.

STABILITY COMPUTATIONS

Overturning

Vertical Forces and Moments about A

Force × Arm = Moment

		Force (lb.)	Arm (ft.)	Moment (ft.-lb.)
W_1	$= 3 \times 8 \times 100$ =	2,400	4.00	9,600
W_2	$= 1 \times 5.5 \times 100$ =	550	5.00	2,750
W_3	$= 1 \times \frac{5.5}{2} \times 100$ =	275	4.17	1.147
W_4	$= .25 \times 5.5 \times 15 \times 150$ =	3,094	3.875	11,989
W_{W_1}	$= 3.25 \times 3 \times 5.5 \times 62.5$ =	3,352	1.625	5,447
W_{W_2}	$= 1 \times \frac{5.5}{2} \times 15 \times 62.5$ =	2,578	3.58	9,229
W_{W_3}	$= 1.5 \times 2.5 \times 3 \times 62.5$ =	703	6.75	4,745
W_{E_1}	$= 3.25 \times 3 \times 1.5 \times 47.5$ =	695	1.625	1,129
W_{E_2}	$= .27 \times \frac{1.5}{2} \times 15 \times 47.5$ =	144	3.34	481
W_{E_3}	$= .5 \times 2.5 \times 3 \times 47.5$ =	178	6.75	1,201
W_R	$= 1 \times 2.5 \times 3 \times 62.5$ =	469	6.75	3,166
U_1	$= -1\ (.50 \times 62.5 \times 2.5 \times 24)$ =	-1,875	4.00	-7,500
U_2	$= -1 \left[\frac{(.50 \times 62.5 \times 6.5) - (.50 \times 62.5 \times 2.5)}{2} \times 24\right]$ =	-1,500	2.67	-4,005
		$\sum V = 11,063$		$\sum M_A = 39,379$

Horizontal Forces and Moments about *A*

Cantilever Reaction of Wall

1 lb./ft.
(Assumed load for ease of calculations)

← 5'-6" → | ← 15'-0" →

a outside buttress — *b* center buttress

$$\sum M_a = 1 \times \frac{15^2}{2} - 1 \times \frac{5.5^2}{2} - V_b \times 15 = 0$$

$$V_b = \frac{1 \times 15^2}{15 \times 2} - \frac{1 \times 5.5^2}{15 \times 2} = 6.49 \text{ lb./ft.}$$

Active Soil Pressure on Wall

$$\frac{h^2 \times \gamma b}{2} \tan^2 (45° - \frac{\theta}{2}) = \frac{1.5^2 \times 47.5}{2} \tan^2 (45° - 17.5°) = 14 \text{ lb./ft.}$$

Force × Arm = Moment

		Force (lb.)	Arm (ft.)	Moment (ft.-lb.)
P_{h_1}	$= 6.49 \times 2 \times 62.5 \times 5.5 \times \frac{5.5}{2}$	= 12,270	2.83	34,724
P_E	$= 6.49 \times 2 \times 14 \times 1.5 \times \frac{1.5}{2}$	= 204	1.50	306
P_{h_2}	$= (\frac{62.5 \times 5.5 + 62.5 \times 6.5}{2}) \times 3$	= 1,125	0.50	562
P_{h_3}	$= -1\ (62.5 \times 1.5 \times \frac{1.5}{2} \times 12)$	= - 844	1.50	-1,266
P_{h_4}	$= -1\ (62.5 \times 2.5 \times \frac{2.5}{2} \times 3)$	= - 586	.83	- 486
$P_{E + R_1}$	$= -1 \left[(\frac{47.5 \times .5 + 62.5 \times 1}{1.5}) \times \frac{1.5^2}{2} \times \tan^2 (45° + \frac{35°}{2}) \times 12 \right]$	= -2,864	1.50	-4,296
$P_{E + R_2}$	$= -1 \left[(\frac{47.5 \times 1.5 + 62.5 \times 1}{2.5}) \times \frac{2.5^2}{2} \times \tan^2 (45° + \frac{35°}{2}) \times 3 \right]$	= -1,851	.83	-1,536
		$\sum H$ = 7,454		$\sum M_A$ = 28,008

Sum of moments about A of vertical and horizontal forces:

39,379 ft.-lb. + 28,008 ft.-lb. = 67,387 ft.-lb.

Location of Resultant

$\frac{67,387}{11,063}$ = 6.09 feet to the right of point A, then $e = 6.09 - \frac{8}{2} = 2.09 > \frac{8}{6}$

Pressure at Base

$$q = \frac{\sum V}{BL}\left(1 \pm \frac{6e}{L}\right)$$

$$= \frac{11,063}{8 \times 3}\left(1 \pm \frac{6 \times 2.09}{8}\right)$$

$$= 461 \pm 723$$

$$q_{max} = 1,184 \text{ lb./ft.}^2$$

Sliding

$R = \sum V \times f$ = 11.063 lb. × 0.80 = 8,850 lb.

$\sum H$ = 7,454 lb.

$$\frac{R}{\sum H} = \frac{8,850}{7,454} = 1.19$$

Heede, Burchard H.
1965. Multipurpose prefabricated concrete check dam. U. S. Forest Serv. Res. Paper RM-12, 16 pp., illus. Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colorado.

The dam consists of nine major parts: six prestressed wall slabs of conventional concrete, and three buttress-footing units of lightweight reinforced concrete. It can be placed in 3 hours (except for excavation and backfill) with two laborers and a backhoe.

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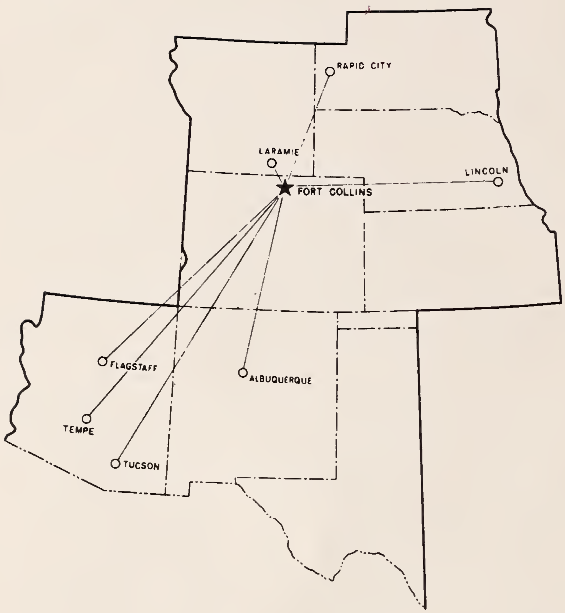
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