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DONALD H. GRAY

Professor of Civil Engineering
University of Michigan
Ann Arbor, Michigan

Dr. Gray received his B.S. and M.S. degrees from the University of California at Berkeley. After graduation in 1961 he was employed as a Research Engineer by Chevron Research Company. He subsequently returned to the University of California and completed requirements for a Ph.D. in Civil Engineering in 1966.

After graduation Dr. Gray joined the Department of Civil Engineering at the University of Michigan. He is currently a Professor of Civil Engineering at Michigan with responsibility for research and teaching in the field of geotechnical engineering.

Dr. Gray is a member and past Chairman of the Environmental Concern's Committee of the Geotechnical Engineering Division of ASCE. He is also a member of the Division Publications Committee. He was the principal organizer and chairman of a Symposium on Soil Erosion for the Highway Research Board in 1973. He has also organized and instructed in a summer short course on erosion control at construction sites sponsored by the Michigan College of Engineering. In addition, Dr. Gray has been a consultant to the State of Michigan, Department of Natural Resources in preparation of a manual on soil erosion and sediment control, and more recntly to the National Park Service adivsing them on slope protection measures for rehabilitation of disturbed sites in Redwood National Park.

Dr. Gray has conducted research for a decade on the role of vegetation in reinforcing soils and stabilizing slopes. Lately he has been involved in biotechnical earth support and slope protection systems which entail the combined or integrated use of plants and structures. He has published several articles on these topics and is author of a book on biotechnical slope protection and erosion control.

ROBBIN B. SOTIR

President,
Soil Bioengineering Corporation
Marietta, GA

Robbin Sotir received her Bachelor of Landscape Architecture from the University of Guelph in 1972. Her education includes Soil Bioengineer/Biotechnical Consultant--Soil Bioengineer Apprenticeship, Europe and Canada, 1976-1979. She is a member of the Canadian Land Reclamation Association and Soil Conservation Society of America. Her technical specialties related to soil bioengineering services are Project Management/Construction Management; Environmental Site Analysis, Design and Planning; Biotechnical Design; Site and Recreation Planning; Reclamation/Rehabilitation; Erosion and Sedimentation Control; and Specification Writing/Cost Estimating.

Robbin Sotir, the president of Soil Bioengineering Corporation, has successfully completed over 140 projects in the past 10 years in North and Central America, Europe and S.E. Asia. Miss Sotir has worked as an international soil bioengineer/landscape architectural consultant in: site analysis, design, construction project management, research, specification writing, cost estimating and general contracting, in projects in excess of 1 million dollars. Robbin Sotir has worked with world-reknown soil bioengineers to study these specialized erosion control techniques. In this way she apprenticed for this work and developed her expertise for the techniques. To our knowledge she is the only European trained soil bioengineer practicing in the S.E. United States. Soil Bioengineering Corporation. She has developed soil bioengineering construction specifications for the North American continent.

Robbin Sotir's work has been receiving more attention from the U.S. Army Corps of Engineers in the past year. She has worked with them on several Phase 1 & 2 reconnaissance, cost estimating, design and specification writing projects and site inspection project management. The U.S. Army Corps of Engineers, Mobile, has installed Soil Bioengineering systems on the Tennessee Tombigbee Waterway under the direction of Soil Bioengineering Corporation. It appears that this agency is serious in considering and developing this type of technologyin the United States, to solve certain instability problems that cannot be solved by using conventional means alone. They have recognized the immediate cost effectiveness as well as the low long term maintenanance cost savings and beautiful products.

WILLIAM E. WEAVER
Geologist
Redwood National Park
Arcata, CA

William Weaver received his B.S. in geology, University of Washington; and his Ph.D. in geology (geomorphology), Colorado State University. Since 1976 he has specialized in applied geomorphic research related to the causes and effects of accelerated erosion on cutover lands in northern California including 1) technical reviews of timber harvest proposals, 2) evaluation of forest practice regulations (as they influence erosion processes), 3) erosion control techniques used on steep, highly erosive terrain, and 4) gully erosion on logged lands. He is also serving his second term as an appointed member of the Coast District Technical Advisory Committee to the State Board of Forestry. The Committee is responsible for changes and additions to the/State's Forst Practice rules. His most recent publications deal with 1) the causes and rates of sediment production from logged land in the Redwood Creek basin, 2) the effectiveness and cost-effectiveness of erosion control techniques used in Redwood National Park, and 3) the approaches and techniques to watershed rehabilitation.

C. ALLEN WORTLEY

Professor-Engineering and Applied Science
University of Wisconsin-Extension
Engineering and Applied Science
Madison, Wisconsin

Allen Wortley received the usual protracted engineering education and after exhaustive study of esoteric subjects of no practical value, received a degree from Caltech.

He is a licensed professional engineer and land surveyor, but ever since he learned from Webster's that the initials P.E. are an abbreviation for physical education, probable error, price-earnings ratio, Protestant Episcopal, and lastly, professional engineer, he does not use them after his name. For similar reasons he does not use L.S., as it stands for U.S. Navy landing ship or place of the seal (locus sigilla). However, he uses C. before his name as most people understand it is an abbreviation for a first name, Charles, rather than capacitance, carat, Centigrade, cent, century, chapter, circa, city, college, cycle or hundredweight.

He left California and moved east to begin working for a living. As a consulting engineer he inspected old sewer manholes in Pittsburgh. This assignment proved to be very deep but by no means dry.

Later he moved up, north to Wisconsin, where he began to climb the technical professional ladder working on zero-bouyancy floating boat docks, French garbage grinders, and lawn watering systems for penitentiaries. He outraced scientific obsolescence by achieving appointment as chief engineer of his consulting firm.

Then, instead of working for a living, he became a university professor. He has pursued the Peter Principle earning a Professional Development degree and becoming Department Chairman.

He teaches soil mechanics and foundation engineering and has unearthed the fact that although many people think soil is simply dirt, geotechnical engineers have found it is really bread and butter.

His research is beautification of water tanks in combination with restaurants, discotheques, and hockey rinks; and ice engineering where he has found that lake water is very cold in the winter, and in fact freezes at all temperatures below thirty-two degrees Fahrenheit.

The most notable accomplishment of Allen Wortley, holder of eight certificates of appreciation and of-the-year awards, is receipt of two Who's Who without purchase of their limited edition directories.

Professor Wortley boasts the usual affiliations that carry arcane Greek letter and acronym designations in return for the payment of annual dues, as well as membership in societies that present awards and offer cheap life insurance.

His hobby is bimonthly golf outings on rainy days, and although not a scratch golfer, he has been scratched while mastering nearly all the Madison areas nineteenth holes.

When not administering, teaching, researching, studying or golfing, he either cuts grass, shovels snow or conducts Extension institutes. As course director his main task is to keep speakers on schedule and avoid running over into social hours.

BIOTECHNICAL SLOPE PROTECTION AND EROSION CONTROL

September 27-29, 1984
San Francisco (Burlingame), California

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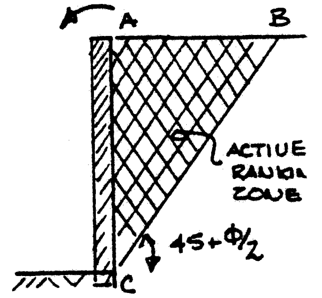
1. LATERAL EARTH PRESSURE AND RETAINING STRUCTURES

SUMMARY OF LATERAL EARTH PRESSURES

I. FREE STANDING WALLS

A. COHESIONLESS BACKFILLS - ACTIVE PRESSURE $\qquad s = \sigma \tan \phi$

1. Rankine Theory



Sufficient deformation occurs in backfill (i.e., by rotation or translation of a FRICTIONLESS wall) to produce a state of active, plastic equilib in a wedge of soil ABC behind the retaining wall.

Lateral earth pressure is reduced to a MINIMUM value consistent with equilib throughout the zone ABC (local equilib must be satisfied everywhere)

Case i) Horizontal Fill, Vertical Wall - P_A is HORIZONTAL

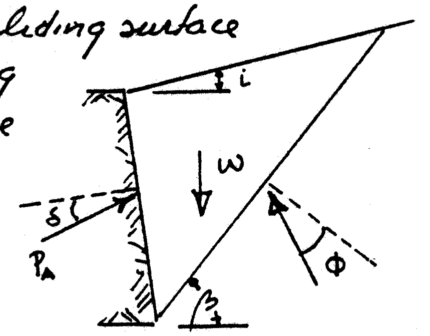
$$K_A = \tan^2(45 - \phi/2)$$
$$P_A = \tfrac{1}{2} K_A \gamma H^2$$

Case ii) Inclined Backfill - P_A inclined same angle (i) as backfill

$$\sigma_H = k \gamma z$$

$$\text{where } k = \cos^2 i \left[\frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi}} \right]$$

2. Coulomb Theory

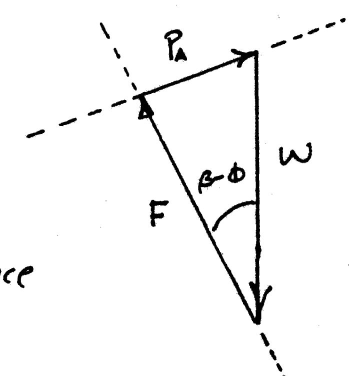


Assumes failure occurs along a PLANAR sliding surface rising from the heel of the wall and intersecting the backfill surface. Trial & error approach ... analyze forces acting on sliding wedge. Critical wedge is one that yields maximum thrust against wall.

a) Graphical solutions

I. Force polygons

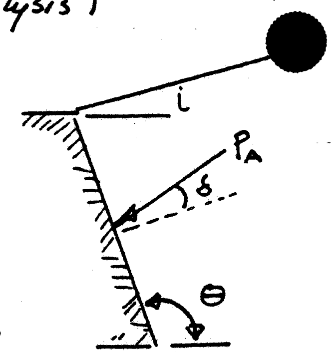
II. Engessor & Culman methods



No restrictions on angle of wall friction nor shape & inclination of backfill ($i \leq \phi$). Culman method especially useful for examining influence of line load on backfill.

b) Analytical solutions (based on "sliding wedge" analysis)

Case i) Sloping, planar backfill w/ battered wall & wall friction



$$P_A = \tfrac{1}{2}\gamma H^2 \left\{ \frac{\csc\theta \,\sin(\theta-\phi)}{\left[\sin(\theta+\delta)\right]^{1/2} + \left[\dfrac{\sin(\phi+\delta)\sin(\phi-i)}{\sin(\theta-i)}\right]^{1/2}} \right\}^2$$

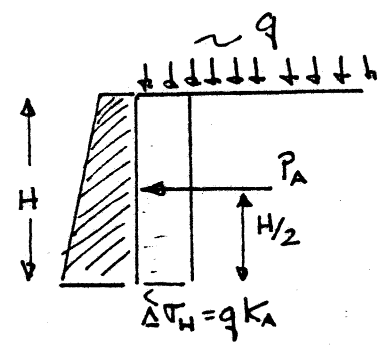
See also Fig. 13.22 in L & W for solns to above equation for special case where $\delta = 0$

Case ii) Vertical walls, horizontal backfill ($i=0$, $\beta=90°$)

.... see charts in L & W (Fig. 13-18) also T & P for $K_A = f(\delta, \phi)$

NOTE: Rankine & Coulomb theories give approx. same earth pressures in ACTIVE case regardless of δ (see Fig. 13-18); therefore, can generally use simpler Rankine theory ($\delta=0$) for active earth pressure calcs. Main effect of δ is to change direction of P_A

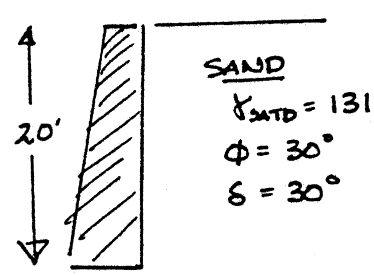
3. Influence of Surcharge and Water



- Surcharge (uniform) produces an additional stress distrib. against retaining wall equal to $\Delta\sigma_H = qK_A$. Resultant acts @ midpoint.

- Water in the back fill can have a profound effect on resultant force against wall. A perched GWT may inc. resultant 5X over dry backfill case. Essential to provide adequate drainage.

Condition	P_{AH} (lbs/ft)
submerged	3500
dry	5600
satd, slope drain	6700
satd, vert. drain	8800
satd, perched GWT	16,000



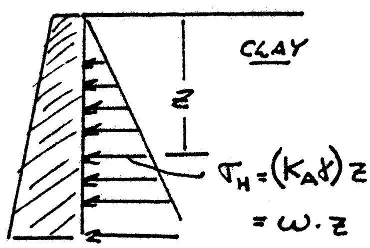
B. COHESIVE SOIL - ACTIVE PRESSURE $\quad s = c + \sigma \tan\phi$

1. General Precautions

a) Avoid use of cohesive backfills whenever possible becaus actual behavior often completely at odds with theoretical predictions, e.g.,

- underestimate P_A & overestimate P_P as result of CREEP
- soil properties change w/ time & climate
- volume instability (swell & shrink) w/ $\Delta w/c$

b) If creep likely then use "EQUIVALENT FLUID PRESSURE" concept to est. lateral pressures, i.e. design wall to resist thrust of "heavy liquid"

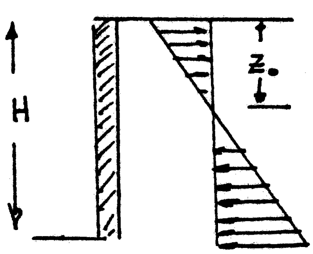


ω = equiv. fluid density

Backfill Matl.	ω (pcf)
dry sand	30
silty sand	35
clayey sand	45
sandy clay	55
silty clay	70
clay	85

c) Only use CLASSICAL theories if sure no changes in gross properties of soil w/ time & creep potential low.

2. Rankine Theory



Existence of cohesion implies a depth (region) where there are no compressive stresses against wall from the backfill, i.e.

$$\sigma_H = \sigma_A = \gamma z \tan^2(45 - \phi/2) - 2c \tan(45 - \tfrac{\phi}{2})$$

Depth (z_o) to which tensile forces act ($\sigma_H = 0$, $z = z_o$)

$$z_o = \frac{2c}{\gamma} \tan(45 + \phi/2)$$

Total thrust against wall

$$P_A = \tfrac{1}{2} \gamma H^2 \tan^2(45 - \phi/2) - 2cH \tan(45 - \tfrac{\phi}{2})$$

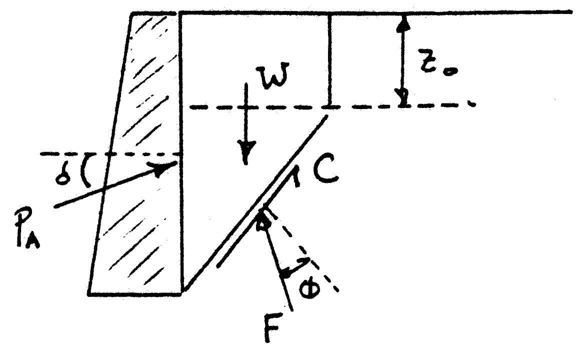
.... purely cohesive backfill ($\phi = 0$)

$$P_A \Big|_{\phi=0} = \tfrac{1}{2} \gamma H^2 - 2cH$$

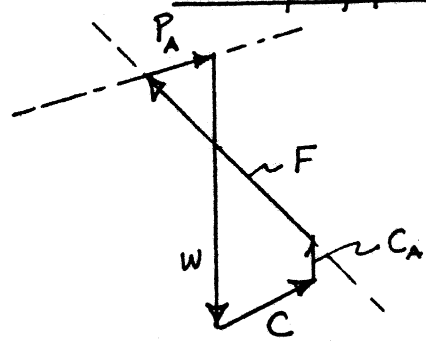
3. <u>Coulomb Theory</u>

Use graphical method or analysis of force polygon with following modifications

- Take into <u>account cohesion</u> along slip surface
- <u>Neglect adhesion</u> along the wall
- Assume discontinuity in sliding surface at depth z_o due to <u>tension crack</u>.



<u>Force polygon</u>:



4. <u>Critical Height of Vertical Unsupported Cuts</u>

Presence of cohesion in soil implies ability of soil to stand unsupported in a vertical cut or bank up to some critical height H_c. This critical height can be estimated by various theories... each of which makes certain assumptions about the stress boundary conditions or shape of the failure surface

OSSHA rules preclude reliance on any of these estimates although in many cases the predictions are accurate (e.g., height of loessial banks in hwy cuts)

	Theory	N_s
	Rankine	4.00
Kinematically Admissable	Upper Bound - Planar	4.00
	" " - Log Spiral	3.83
	Circular Arc	3.85
	Planar (w/tension crack)	2.67
	Cycloid Arc	2.00
	Lower Bound (Static Equil)	2.00

$$H_c = \frac{N_s \cdot C}{\gamma} \tan(45 + \frac{\phi}{2})$$

<u>N_s = Stability No.</u>

C. COHESIONLESS SOILS - PASSIVE PRESSURE

1. Rankine Theory

Some general comments under active stress apply... strains required to reach PASSIVE Rankine equilib are much higher. Lateral stress becomes MAJOR princ. stress

$$K_p = \tan^2(45 + \phi/2)$$ vert walls & horiz fill $\delta = 0$

2. Coulomb Theory

Planar slip surface only valid for $\delta < 1/3\,\phi$

If this condition is met, may analyze failure wedge by
- a) Force polygons - trial & error
- b) Engesser & Culman methods

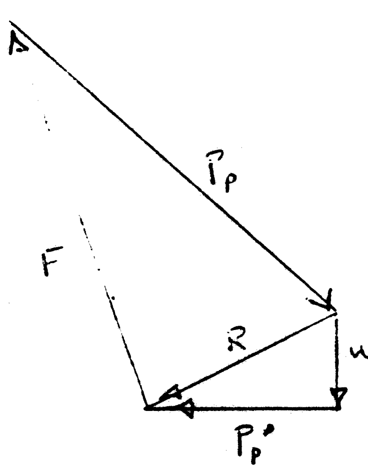
3. Curved Failure Surface Analyses ($\delta \geq 1/3\,\phi$)

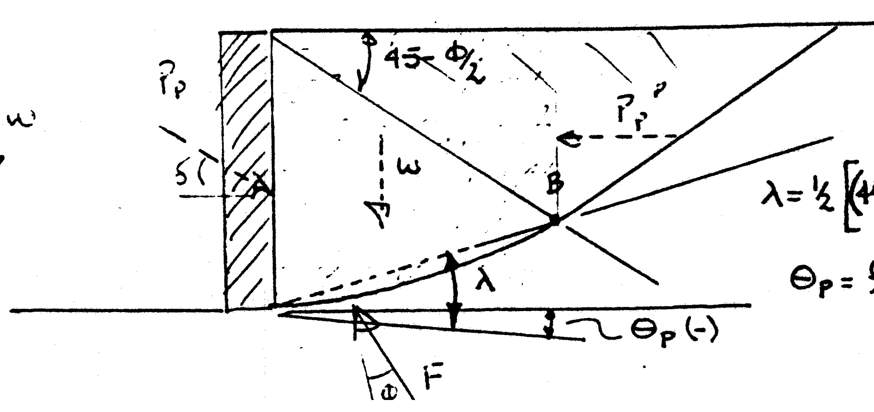
Case i) Horiz. Backfill, Vert. Wall

Use Fig. 13-20 in L&W to find $K_p = f(\delta, \phi)$
Note considerable influence of δ on K_p

Case ii) Irregular or Sloping Backfill,

- Use "trial & error" graphical method with circular arc or log spiral failure surface in backfill adjacent the wall.
- Method of Abdul-Baki may be used to locate critical failure surface (see Fig 7.07c in P&M).





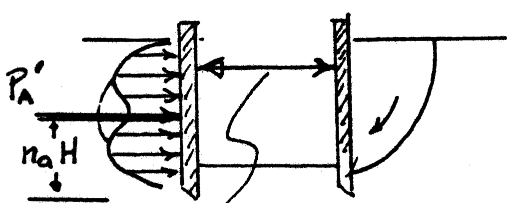
$$\lambda = \tfrac{1}{2}\left[\left(45 - \frac{\phi}{2}\right) - \theta_p\right]$$

$$\theta_p = f(\phi, \delta)$$

II. BRACED WALLS & EXCAVATIONS

A. Stress & Deformation Conditions

- Classical earth pressure theories do not apply directly because deformations and stress distrib quite different from those assumed in case of free standing walls



- deformation will be along curved failure surface (log spiral)
- stresses will tend to be higher towards top (more parabolic or rectangular) than free standing wall.

constraint due strut or brace radically affects both mode deformation & resulting stress distribution

B. Stress Distribution and Resultant Thrust - COHESIONLESS SOILS

① Terzaghi Peck Approx.

$$P_A' \approx 1.1 \to 1.2\ P_{A\,COULOMB}$$

Based on experience & observation (rule-of-thumb)

Resultant acts at dist. $n_a H$

sands: $0.45 \leq n_a \leq 0.55$

clays: $0.30 \leq n_a \leq 0.50$

② Kim & Preber Analysis

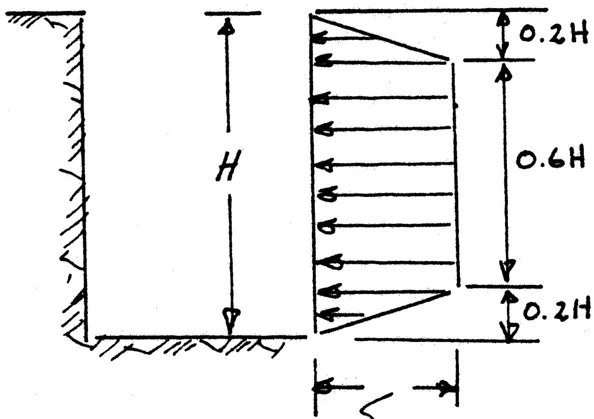
Based on equilib of failure mass bounded by log spiral failure surf. Also applies to soils w/ cohesion

$$\frac{P_A'}{\frac{1}{2}\gamma H^2} = f\left(\phi, \delta, n_a, \frac{c}{\gamma H}\right)$$

See tabulated solns. in J. of SMFD Vol. 95 No. SM 6

③ Terzaghi Peck Trapezoidal Stress Distribution

Based on experience & field instrumentation of bracings in dense sand during const. of Berlin subway



$\sigma_A' = \gamma H K$

$K = f(\delta, \phi)$

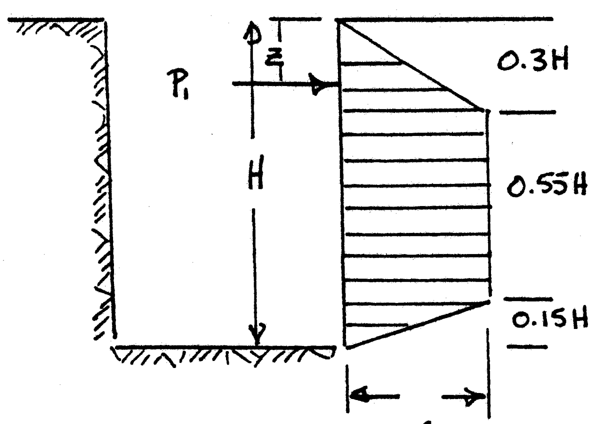
See Fig 7.14 in P&M.

$P_A' = 0.8 H^2 K \gamma$

$$\sigma_A' = \gamma H K = \gamma H \cdot \left\{ 0.8 \cos\delta \left[\frac{\cos\phi}{\sqrt{\cos\delta} + \sqrt{\sin(\phi+\delta)\sin\phi}} \right]^2 \right\}$$

C. Stress Distribution & Resultant Thrust. – COHESIVE SOILS

Based on experience & field instrumentation of bracings in medium to soft clays during const. of Chicago subway



$\sigma_A' = \gamma H K$

$K = 1.2\left(1 - \frac{2 q_u}{H\gamma}\right)$

q_u = unconfined compress strength of clay.

$$\sigma_A' = \gamma H K = 1.2(\gamma H - 4c)$$

$z \leq \frac{q_u}{\gamma}$ (to prevent excessive yield in soil)

Stratified Cohesive Soils:

Must compute an "average" or weighted density & unconf. comp. str.

$$q_u' = \frac{1}{H_t}\left(H_1 q_1 + H_2 q_2 + \ldots . H_n q_n\right)$$

REINFORCED EARTH® WALLS - METAL STRIPS

A. Elements of Reinforced Earth Structure

1. Fill

To insure good frictional stress transfer to ties use only cohesionless, free draining mat'l.

#200 mesh fraction ≤ 15%

$\phi \geq 25°$

2. Skin (Facing)

May be relatively thin & flexible... main function is to hold in backfill at face of wall. Little lateral stress transfer to skin

3. Reinforcement (Ties)

Shear stresses which develop in backfill are transferred by friction to imbedded ties which react in tension. Important design criteria for ties are:

a) Strong enough (large enough X section) to resist yielding or breaking in tension.

b) "Frictional" enough (long and wide enough) to resist pullout.

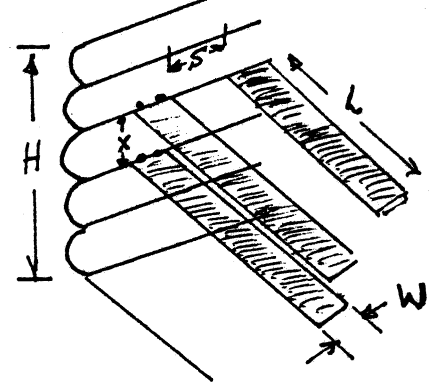
B. Tie Design - Resistance to Tensile Breaking

1. Tie Back Wall Hypothesis

Ⓡ Method - Consider local equilib of wall area suppt. by one tie @ depth z

- Assume lateral stress incr. linearly w/depth
- Assume sufficient deformation in backfill to develop Active pressure conditions

$\sigma_A = \gamma z K_A$



$$(FS)_y = \frac{\sigma_y W t}{K_A \gamma H S X}$$

Ⓒ Ⓕ & Ⓒ Ⓜ methods give same FS provided

$n > 10$ $\qquad n =$ no. ties

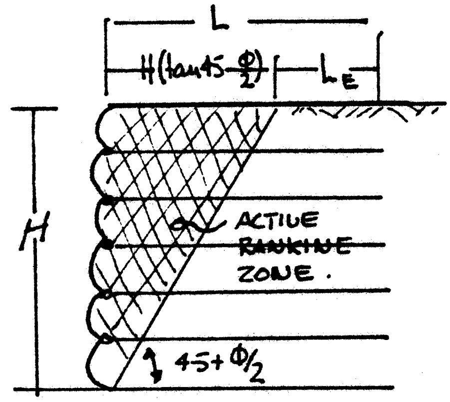
2. <u>Coherent Gravity Wall Hypothesis</u>

Reinforced earth backfill behaves as a coherent mass. Lateral earth stresses will be greater than those computed by Rankine tie back approach.

$$\sigma_A = \gamma z K (1 + K_A)$$

C. Tie Design - Resistance to Pullout

1. <u>Tie Back Wall Hypothesis</u>



Ⓡ <u>Method</u> (MOST CONSERVATIVE)

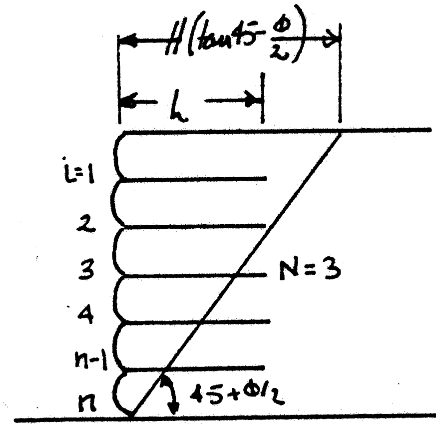
Postulates that <u>ALL</u> ties must extend a minimum length (L_E) beyond Rankine active zone

$$(FS)_\phi = \frac{2 L_E W \tan \phi_u}{S X K_A}$$

L_E = "effective" length

ϕ_u = angle of sliding or frictional resistance between tie & soil

$$L \geq L_E + H \tan(45 - \phi/2)$$



Ⓒⓕ <u>Method</u> (LEAST CONSERVATIVE)

Considers overall stability of wall... not <u>necessary</u> that all ties extend beyond assumed sliding surface.

$$(FS)_\phi = \frac{4 X W \tan \phi_u}{K_A H^2 S} \sum_{i=N}^{n} i \left[L - (n-i) X \tan\left(45 - \frac{\phi}{2}\right) \right]$$

i = summation index for counting no. ties

N = value of i for first tie from top to extend past failure plane

n = no ties (in vertical tier).

CF Method (cont)

<u>Iterative procedure</u>:

#1. Set FS_ϕ on LHS

#2. Estimate L ($L \approx 0.8H$)

#3. Calc $N(L)$

$$N = 17 - \frac{L}{X \tan(45 - \phi/2)}$$

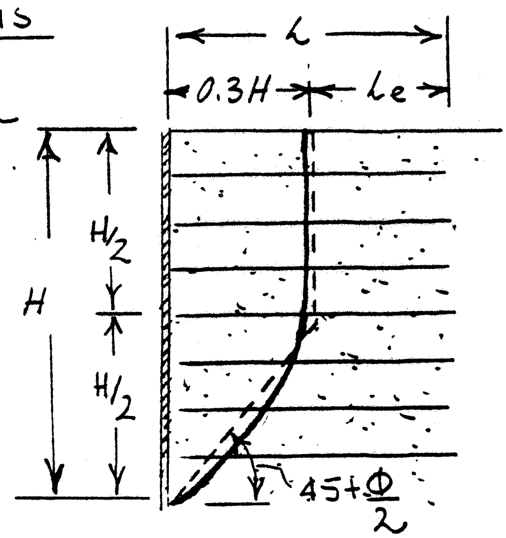
#4. Compute FS_ϕ on RHS

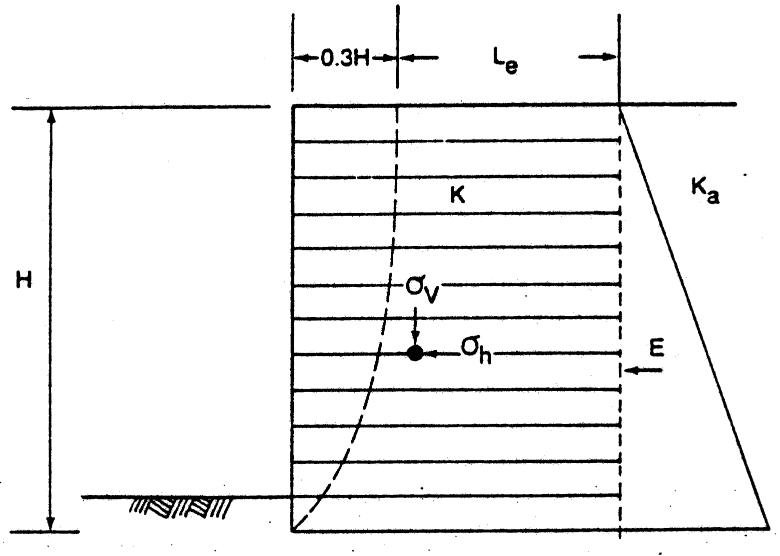
#5. Iterate... vary L until RHS = LHS

2. <u>Coherent Gravity Wall Hypothesis</u>

Failure surface is distorted (i.e. bent up) by presence of reinforce. The total length of tie is therefore reduced accordingly

$$L \geq L_e + 0.3H$$





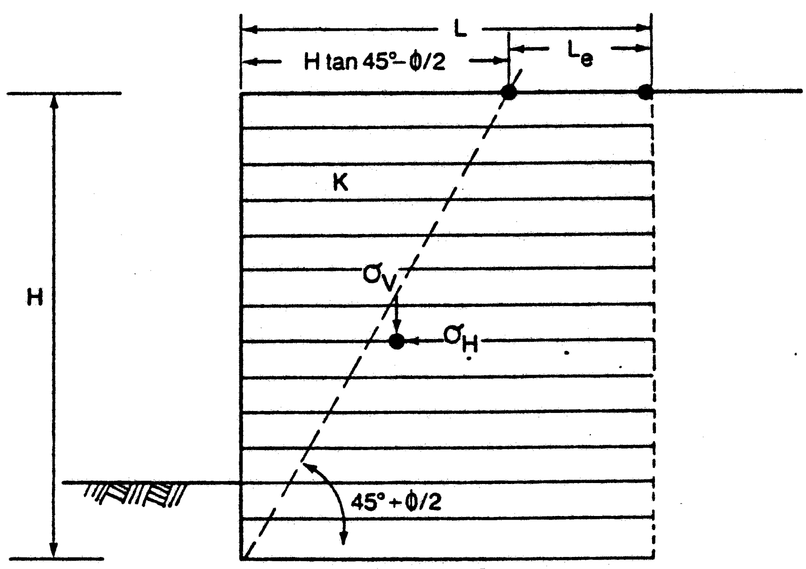
(a) Coherent Gravity Structure
Hypothesis

$\sigma_h = K(1 + K_a)\,\gamma H$

$L_e < L - 0.3H$ $L \geq L_E + 0.3H$

for $\phi = 30°$:

$\sigma_h = 1.33K \cdot \gamma H$



(b) Tie-Back Structure
Hypothesis

$\sigma_h = K \cdot \gamma H$

$L_e < L - H\tan(45° - \phi/2)$ $L \geq L_E + H\tan(45 - \phi/2)$

for $\phi = 30°$:

$L_e < L - 0.58H$

Design Hypotheses for Reinforced Earth Walls

III. DESIGN REQUIREMENTS - GRAVITY RETAINING STRUCTURES

A. Types of Gravity Structures

1. Masonry & concrete walls
2. Crib & bin walls
3. Gabions
4. Cantilever & counterfort walls
5. Reinforced earth.

B. External Stability

1. Overturning: $\frac{R.M.}{\text{Net } O.M.} > 1.5$

2. Sliding: $\frac{N \tan \phi}{P_{AH}} > 1.5$ neglect P_p @ toe.

3. Bearing Force location: $\frac{\bar{x}}{(B/3)} > 1.0$

4. Bearing Capacity: $\frac{q_{ULT}}{\sigma_{AV.\,BASE}} > 2.5$

C. Internal Stability

Structural members of wall must safely resist stresses placed upon them.

1. Cantilever wall - stem resist bending w/o cracking

2. Crib walls - "stretchers" & "headers" resist bending, torsional & compressive stresses from crib fill.

4. Reinforced Earth.
   a) Tie breaking
   b) Tie pullout.

See Scharzoff (1975). Retng. wall practice for low-vol. forest roads. TRB Spec. Rept. #160, pp. 128-140

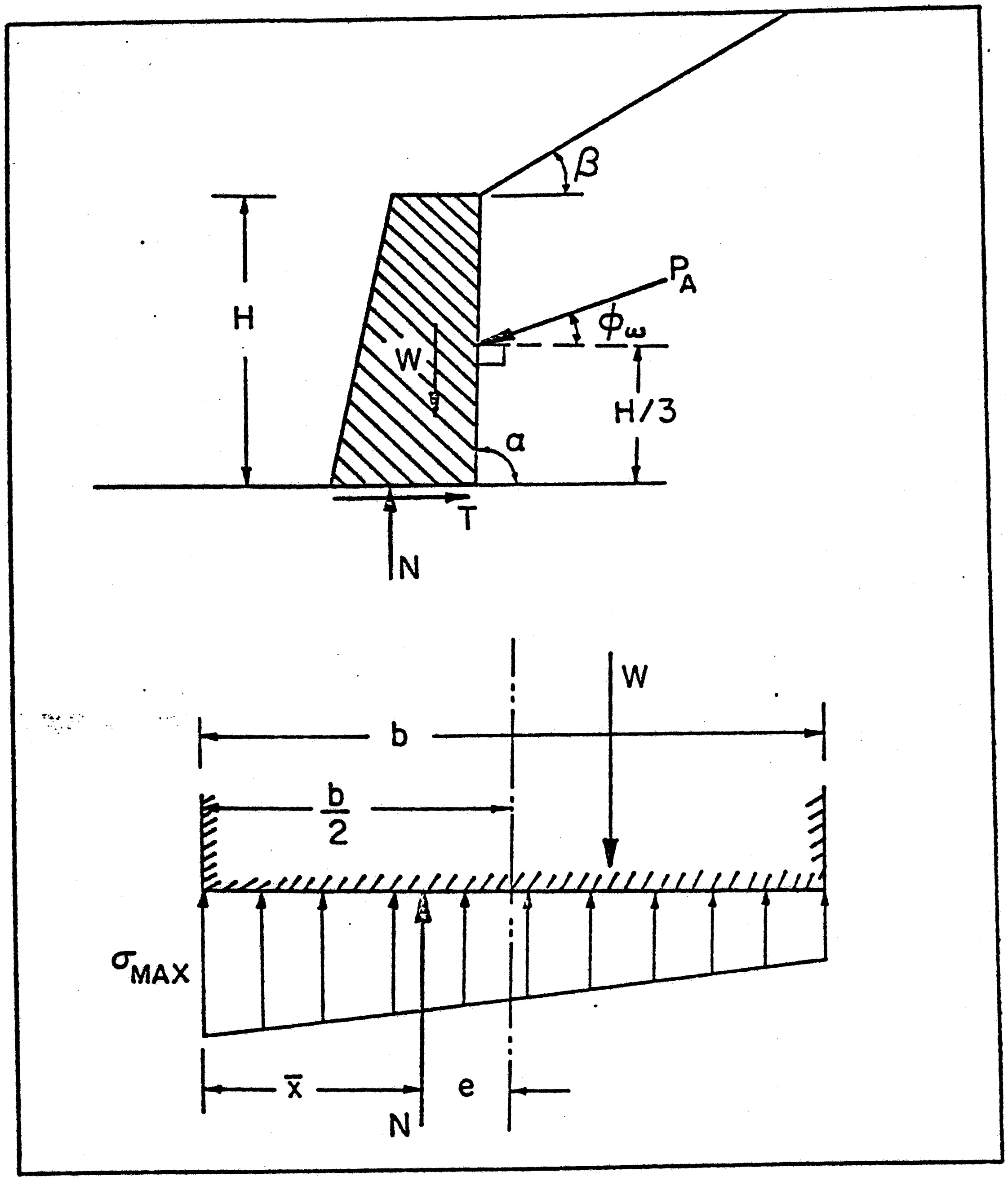


Figure 5.4 Schematic diagram of forces and stresses acting on a gravity retaining wall.

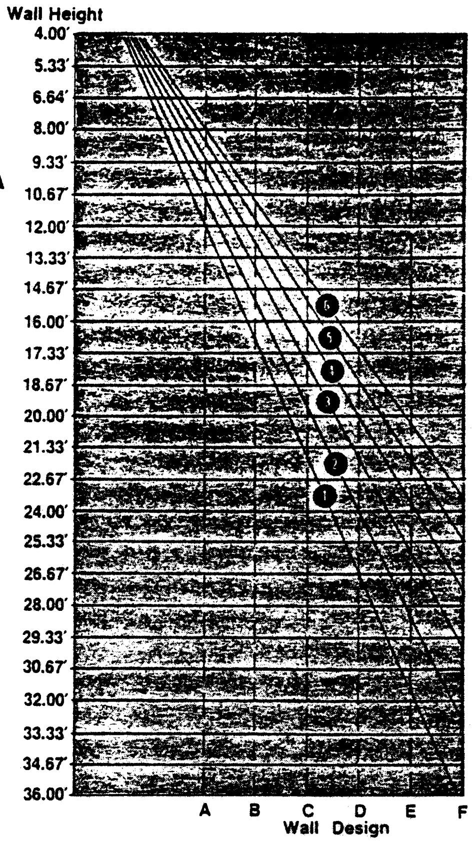
DESIGN OF GRAVITY TYPE RETAINING WALLS

Bin-Type Retaining Wall is a gravity retaining wall in which an earth mass inside bins acts as the gravity wall and the steel members hold the earth mass intact. These two components combine to resist overturning and sliding forces imposed by the retained soil and other superimposed loads. Because of this design, support for the wall is needed under the earth mass. On rigid foundations, provision must be made to allow slight settlement of the vertical corner members. Normal practice is to provide a compressible cushion under the base plates with approximately 8 inches of loose fill.

Individual walls should be designed for stability in accordance with established criteria for gravity walls. Recognized texts are available which thoroughly cover the design of gravity retaining walls, and these should be consulted by the engineer responsible for the design of the wall. Design Chart A, while no substitute for individual site design, presents long-used gravity wall criteria for width-to-height ratio under the typical loading conditions listed in Table I. However, they are presented here only as suggested guidelines.

A critical factor in wall design is the adequacy of the foundation. The resistance of the foundation to the overturning and sliding forces acting on the wall is a sophisticated engineering evaluation. *Proper site investigations and analyses should be carried out for any retaining wall.*

CHART A

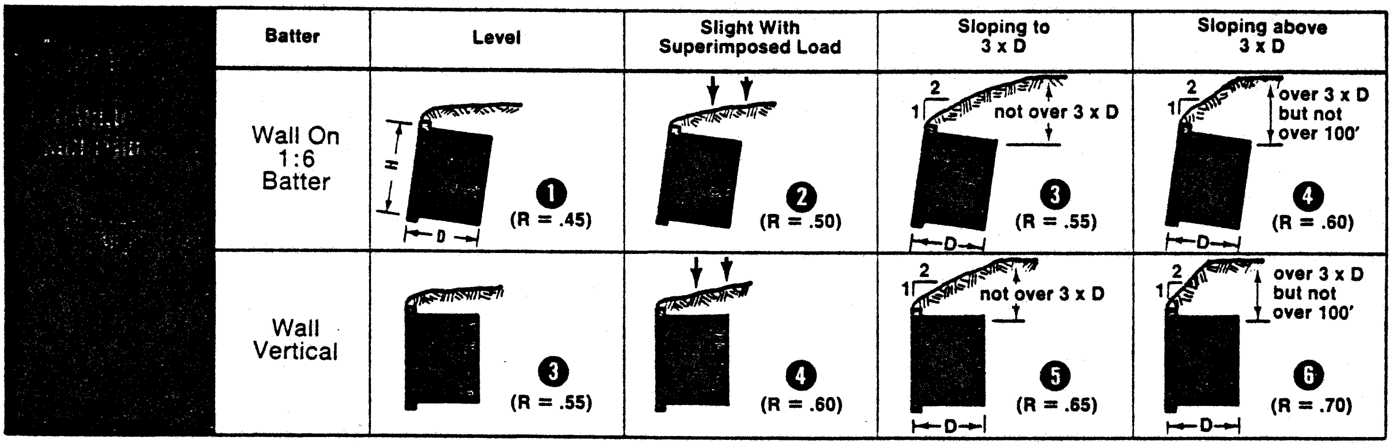


BATTER VS. VERTICAL. While batter walls should always be considered first, the advantages of vertical Bin-Walls, *where other considerations permit their use*, should not be overlooked. Careful analysis of a given situation will sometimes show a vertical wall of the same thickness as a batter wall will be structurally adequate. Even a thicker vertical wall will sometimes prove economical, land values considered.

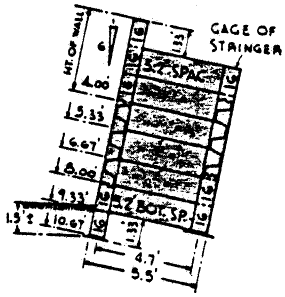
Invariably, it is easier to construct a vertical Bin-Wall on a curve. Short stringers can be used in adjacent bins, for example, without restriction. If sharp bends are required, the special plates required are much simplified and more economical.

Under some circumstances, the obvious gain in usable space, by use of a vertical wall, will assume importance. For example, a vertical 24-foot-high wall will provide 4 square feet of valuable land for every foot of wall, as compared to a 1 to 6 batter wall with its toe in the same location.

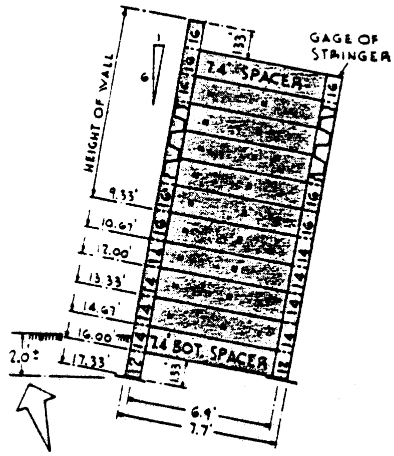
It must be remembered that Armco Bin-Walls are flexible structures that will adjust to minor ground movements. To allow for this, as well as normal construction tolerances, vertical walls are frequently installed on a slight batter.



DESIGN A

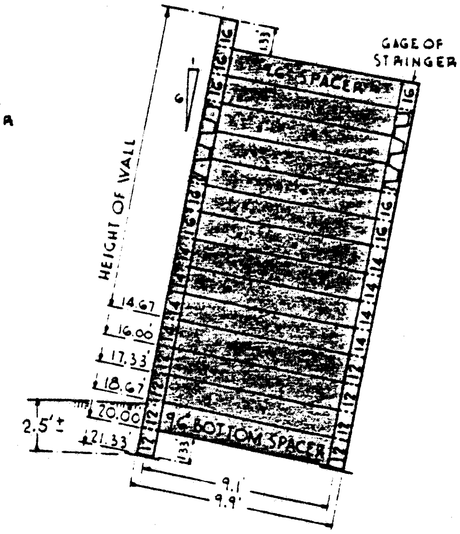


DESIGN B

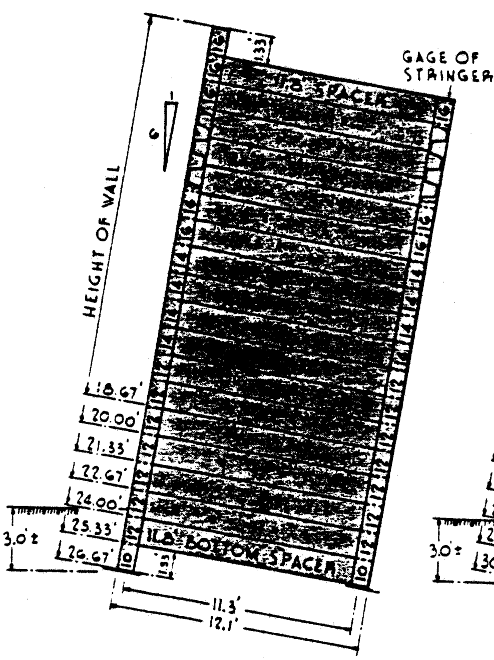


Note: These depths may vary to suit conditions

DESIGN C

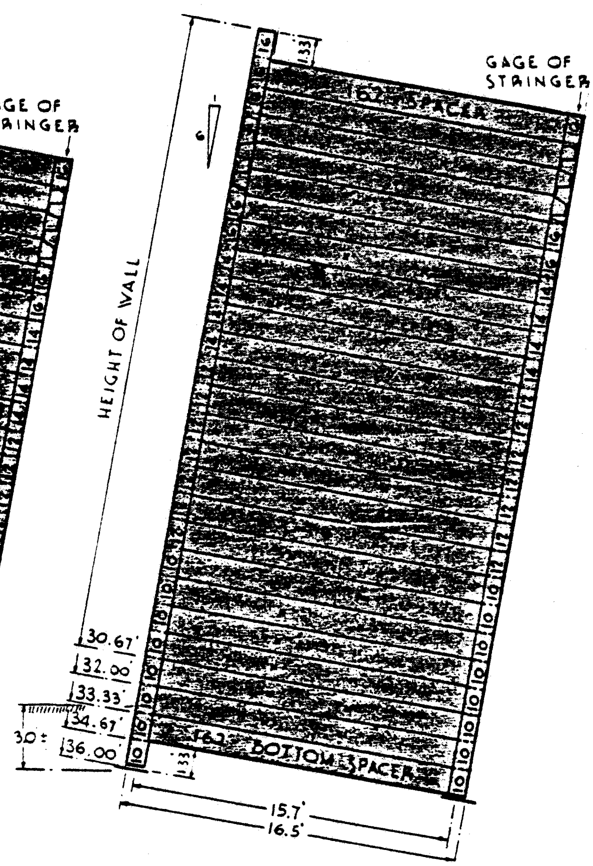


DESIGN D



DESIGN E

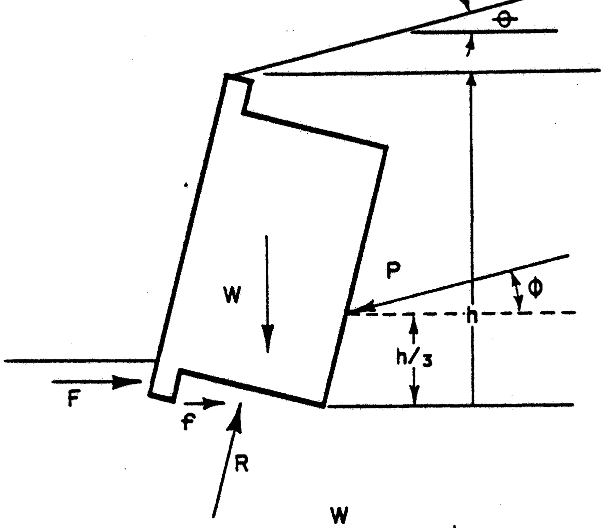
DESIGN F

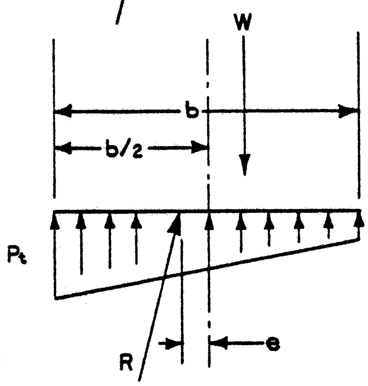


DESIGN OF ARMCO BIN-TYPE RETAINING WALLS

GRAVITY WALL DESIGN. Bin-type gravity wall design should include computations to determine the stability of the wall for (1) overturning; (2) soil bearing resistance under the base; (3) sliding; and (4) bin internal pressures.

The formulae which follow are typical of those found in most engineering texts on retaining wall design. Substitution factors can be found for the necessary design computations in these same texts.





> **Note:** Assumptions used in computing loads are based on soil construction in which no hydrostatic conditions exist. The installation of subdrains does help eliminate potentially dangerous hydrostatic situations.

A. Check for overturning

$\Sigma F_v = \Sigma F_h = 0$ (FIND R)

$\Sigma M_{heel} = 0$ (FIND e)

$P = \frac{wh^2}{2} K$ WHERE w = SOIL WEIGHT
h = WALL HEIGHT

$$K = \cos\theta \left[\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}} \right]$$

θ = SURCHARGE ANGLE

φ = ANGLE OF INTERNAL SOIL FRICTION

FOR THE CASE OF θ = 0°(NO SOIL SURCHARGE) THEN

$K = \tan^2(45° - \phi/2)$

B. Check for supporting soil pressure

$P_t = W/b + 6We/b^2$

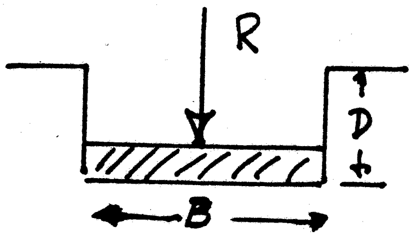
$P_h = W/b - 6We/b^2$

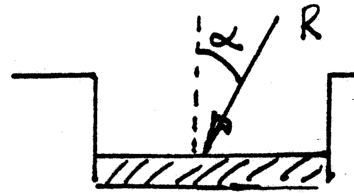
C. Check for sliding resistance

$\Sigma F_h = 0$

$f = W \tan\alpha$

WHERE α = ANGLE OF SLIDING FRICTION FOR THE WALL BASE AND SOIL. FOR ARMCO BIN-WALLS α MAY BE TAKEN TO EQUAL φ, ANGLE OF INTERNAL FRICTION OF THE SOIL.





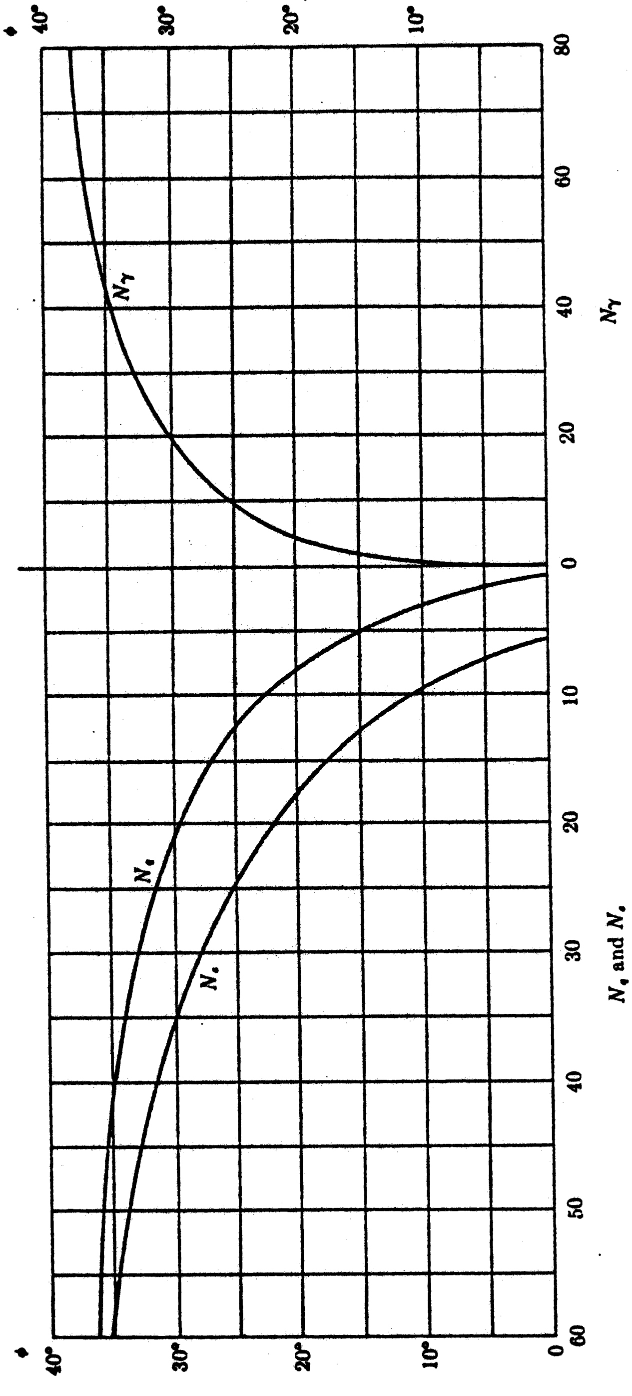


Fig. 8.14. Bearing-capacity factors. [After Terzaghi and Peck (1948).]

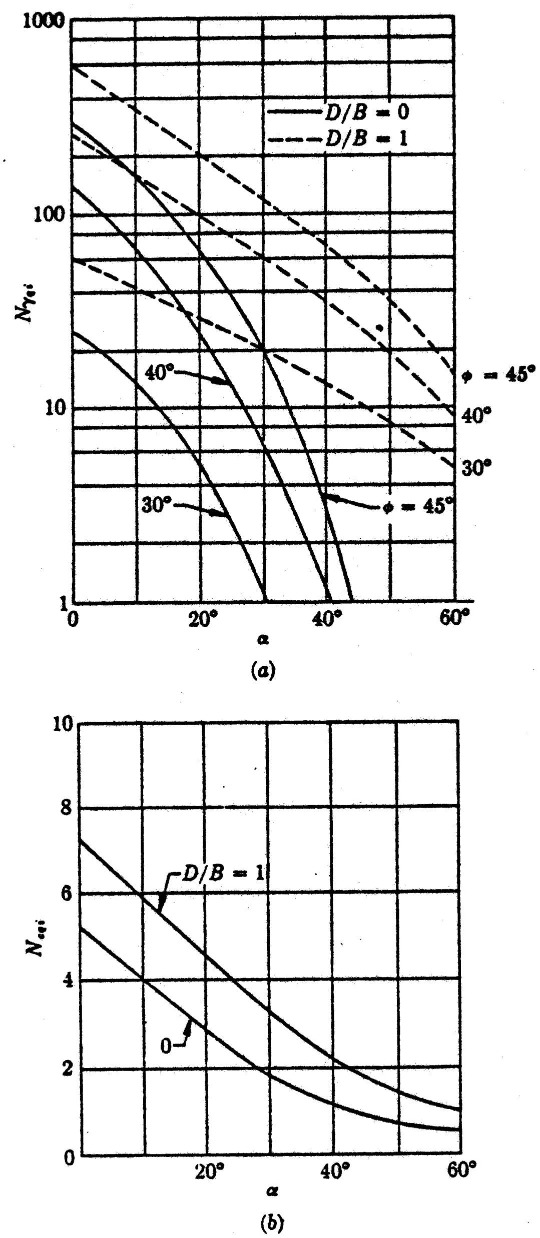


Fig. 8.20. Bearing capacity numbers for foundations with inclined loads. [After Meyerhoff (1953).]

$$q_{ult} = \tfrac{1}{2} B \gamma N_\gamma + c N_c + D \gamma N_q$$

$$q_{ult} = c N_{cq} + \tfrac{1}{2} B' N_{\gamma q}.$$

B' = effective width

TABLE 5.1 COEFFICIENT OF ACTIVE EARTH PRESSURE AS A FUNCTION OF WALL AND BACKFILL INCLINATION, FOR $\alpha' = \alpha - 90$, AND $\phi_w = 0$.

(from Lambe and Whitman, 1969)

	$\beta =$	$-30°$	$-12°$	± 0	$+12°$ 1:4.7	$+30°$ 1.1:7
$\phi = 20°$	$\alpha' = +20°$		0.57	0.65	0.81	
	$\alpha' = +10°$		0.50	0.55	0.68	
	$\alpha' = \pm 0°$		0.44	0.49	0.60	
	$\alpha' = -10°$		0.38	0.42	0.50	
	$\alpha' = -20°$		0.32	0.35	0.40	
$\phi = 30°$	$\alpha' = +20°$	0.34	0.43	0.50	0.59	1.17
	$\alpha' = +10°$	0.30	0.36	0.41	0.48	0.92
	$\alpha' = \pm 0°$	0.26	0.30	0.33	0.38	0.75
	$\alpha' = -10°$	0.22	0.25	0.27	0.31	0.61
	$= -20°$	0.18	0.20	0.21	0.24	0.50
$\phi = 40°$	$\alpha' = +20°$	0.27	0.33	0.38	0.43	0.59
	$\alpha' = +10°$	0.22	0.26	0.29	0.32	0.43
	$\alpha' = \pm 0°$	0.18	0.20	0.22	0.24	0.32
	$\alpha' = -10°$	0.13	0.15	0.16	0.17	0.24
	$\alpha' = -20°$	0.10	0.10	0.11	0.12	0.16

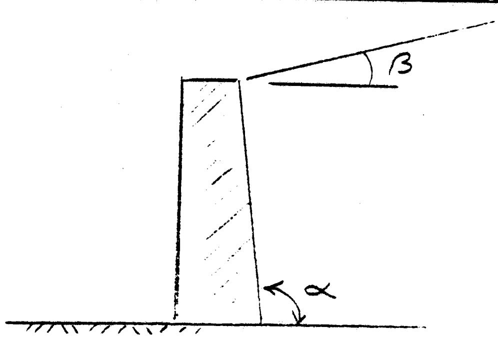


Table 7.03a
Proposed Coefficients of Skin Friction between Soils and Construction Materials—after Potyondy

$$[f\phi = \delta/\phi,\ fc = \frac{c_a}{c},\ fc\ \text{max} = \frac{c_a\ \text{max}}{c\ \text{max}};\ \text{without factor of safety}]$$

Construction material			Sand		Cohesionless silt			Cohesive granular soil		Clay		
			0.06< D< 2.0 mm		0.002< D<0.06			50% Clay-50% Sand		D≤0.06 mm		
Surface finish of construction material			Dry	Sat.	Dry	Sat.		Consist. I = 1.0–0.5		Consist. Index: 1.0–0.73		
			Dense		Dense	Loose	Dense					
			$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	$f\phi$	fc	$f\phi$	fc	fc_{max}
Steel	Smooth	Polished	0.54	0.64	0.79	0.40	0.68	0.40	-	0.50	0.25	0.50
	Rough	Rusted	0.76	0.80	0.95	0.48	0.75	0.65	0.35	0.50	0.50	0.80
Wood		Parallel to grain	0.76	0.85	0.92	0.55	0.87	0.80	0.20	0.60	0.40	0.85
		At right angles to grain	0.88	0.89	0.98	0.63	0.95	0.90	0.40	0.70	0.50	0.85
Concrete	Smooth	Made in iron form	0.76	0.80	0.92	0.50	0.87	0.84	0.42	0.68	0.40	1.00
	Grained	Made in wood form	0.88	0.88	0.98	0.62	0.96	0.90	0.58	0.80	0.50	1.00
	Rough	Made on adjusted ground	0.98	0.90	1.00	0.79	1.00	0.95	0.80	0.95	0.60	1.00

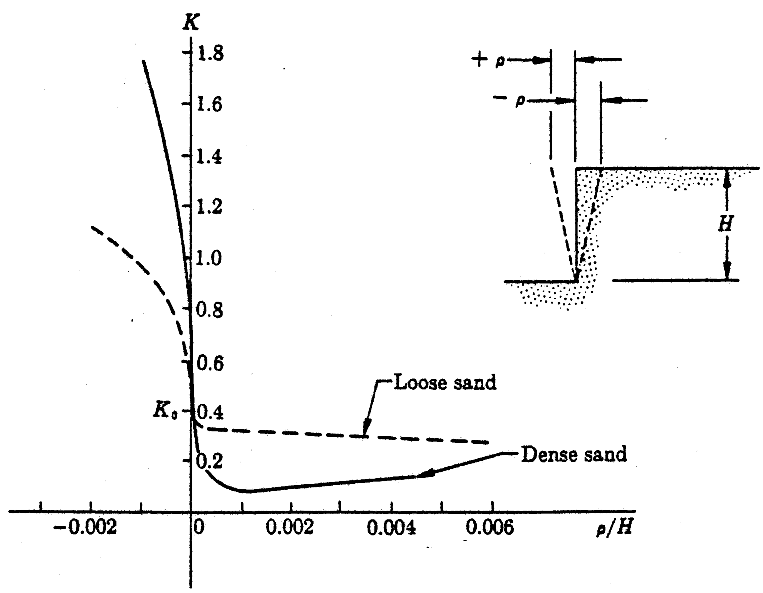


Fig. 9.3. Relationship between earth pressure and wall deflection. [After Terzaghi (1954).]

2. CLASSIFICATION AND CAUSES OF SLOPE FAILURES

DHG 4/8/82

CLASSIFICATION & CAUSES OF SLOPE FAILURES

I. CLASSIFICATION

A. Materials

1. Ice
2. Rock (jointed, weathered, bedded, folded)
3. Soil (dry vs. saturated, sandy vs. clayey)

B. Velocity

1. Rapid (seconds → minutes)
   rockfalls, avalanches, earthflows, "air cushion" slides
2. Intermediate (minutes → hours)
   debris slides, block glides, slumps
3. Slow (days → years)
   creep, solifluction, lateral spreading

C. Displacement

Dist. mat'l. moves = f(max veloc, topography, size, entrainment of air & H_2O)

D. Failure Mechanism

1. Slides - Movement along well defined sliding or shearing surface of largely intact blocks or masses of earth and rock.

   a) Planar: Occurs in slopes where there is some geologic control, e.g., bedding planes, joints, colluvium mantle. Also shallow slides (sloughing) in homogeneous, sandy slopes (See Fig. 2.11)

   b) Rotational: Occurs in slopes composed of homogeneous cohesive soils in which resistance to sliding is independent of depth. Crit. sliding surf. tends to be an arc passing deeply under slope where shear resistance is lowest and shear stresses high. (See Fig. 2.12)

2. Flows - Quasi viscous flow in which difficult to detect a distinct sliding surface. Motion dies out with depth. Tends to occur in saturated soils (sands, silts, clays) with a high water content.

3. Falls - falling mass of material loses coherent contact with stable, unmoving base. Tends to occur in jointed, brittle rock forming steep slopes.

E. Classification System

See classification system (Fig. 2.10) by Varnes which classifies according to:

Type of Movement - falls, topples, slides (rotational vs translational), spreads, flows

Type of Material - rock, soil (coarse vs. fine)

II. FACTORS CONTRIBUTING TO INSTABILITY OF EARTH SLOPES.

See subdivision of factors (Table 2.10) according to those that contribute to:

High Shear Stress and/or Low Shear Strength

Slopes fail when shear stress > shear strength along a critical sliding surface.

III. IDENTIFICATION OF UNSTABLE SLOPES

Identification of unstable slopes or slopes with a high landslide potential can be made using certain topographic, vegetative, hydrologic, and geologic indicators (see Table 2.11).

<table>
<tr><th colspan="3" rowspan="3">TYPE OF MOVEMENT</th><th colspan="3">TYPE OF MATERIAL</th></tr>
<tr><th rowspan="2">BED-ROCK</th><th colspan="2">ENGINEERING SOILS</th></tr>
<tr><th>Predominantly coarse</th><th>Predominantly fine</th></tr>
<tr><td colspan="3">FALLS</td><td>Rock fall</td><td>Debris fall</td><td>Earth fall</td></tr>
<tr><td colspan="3">TOPPLES</td><td>Rock topple</td><td>Debris topple</td><td>Earth topple</td></tr>
<tr><td rowspan="3">SLIDES</td><td>ROTA-TIONAL</td><td rowspan="2">FEW UNITS</td><td>Rock slump</td><td>Debris slump</td><td>Earth slump</td></tr>
<tr><td rowspan="2">TRANS-LATIONAL</td><td>Rock block slide</td><td>Debris block slide</td><td>Earth block slide</td></tr>
<tr><td>MANY UNITS</td><td>Rock slide</td><td>Debris slide</td><td>Earth slide</td></tr>
<tr><td colspan="3">LATERAL SPREADS</td><td>Rock spread</td><td>Debris spread</td><td>Earth spread</td></tr>
<tr><td colspan="3" rowspan="2">FLOWS</td><td rowspan="2">Rock flow</td><td>Debris flow</td><td>Earth flow</td></tr>
<tr><td colspan="2">(soil creep)</td></tr>
<tr><td colspan="3">COMPLEX</td><td colspan="3">Combination of two or more principal types of movement</td></tr>
</table>

Figure 2.10 Abbreviated classification of slope movements (from Varnes, 1978)

TABLE 2.10

FACTORS CONTRIBUTING TO INSTABILITY OF EARTH SLOPES

(After Varnes, 1958)

Factors that Contribute to <u>High Shear Stress</u>	Factors that Contribute to <u>Low Shear Strength</u>
A. Removal of Lateral Support 1. Erosion - bank cutting by streams and rivers 2. Human agencies - cuts, canals, pits, etc.	A. Initial State 1. Composition - inherently weak materials 2. Texture - loose soils, metastable grain structures 3. Gross structure - faults, jointing, bedding, planes, varving, etc.
B. Surcharge 1. Natural agencies - wt of snow, ice and rainwater 2. Human agencies, fills, buildings, etc.	B. Changes Due to Weathering and Other Physico-Chemical Reactions 1. Frost action and thermal expansion 2. Hydration of clay minerals 3. Drying and cracking 4. Leaching
C. Transitory Earth Stresses - earthquakes D. Regional Tilting	
E. Removal of Underlying Support 1. Subaerial weathering - solutioning by ground water 2. Subterranean erosion - piping 3. Human agencies - mining	C. Changes in Intergrannular Forces Due to Pore Water 1. Buoyancy in saturated state 2. Loss in capillary tension upon saturation 3. Seepage pressure of percolating ground water
F. Lateral Pressures 1. Water in vertical cracks 2. Freezing water in cracks 3. Swelling 4. Root wedging	D. Changes in Structure 1. Fissuring of preconsolidated clays due to release of lateral restraint 2. Grain structure collapse upon disturbance

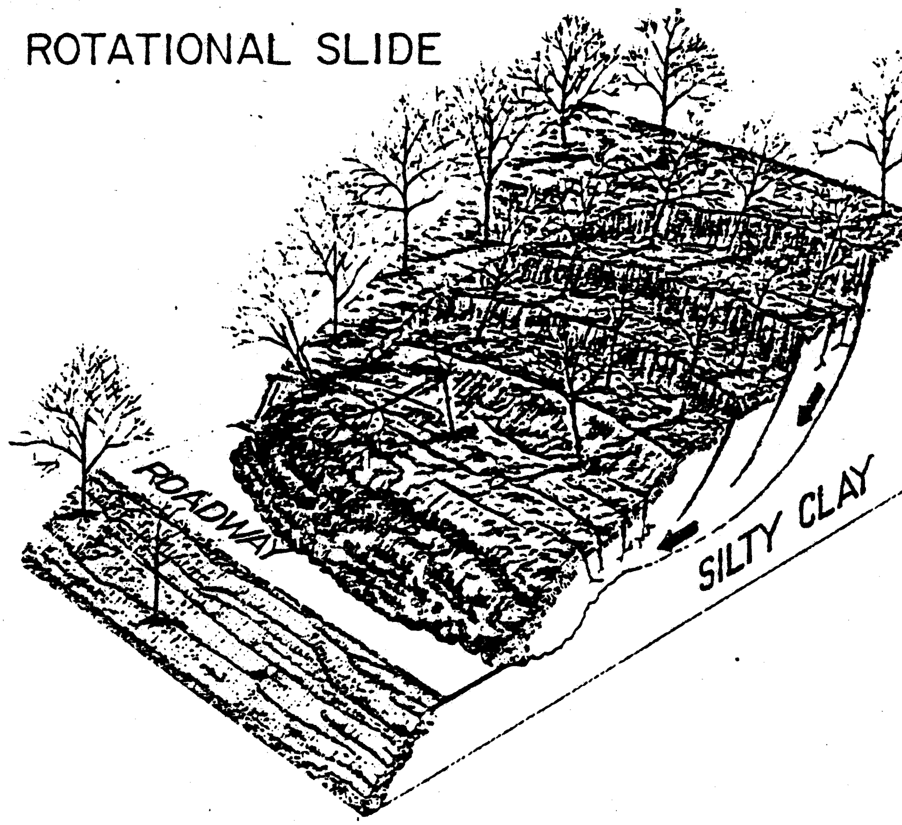


Figure 2.12. Schematic illustration of rotational, earth slump. (from Royster, 1978)

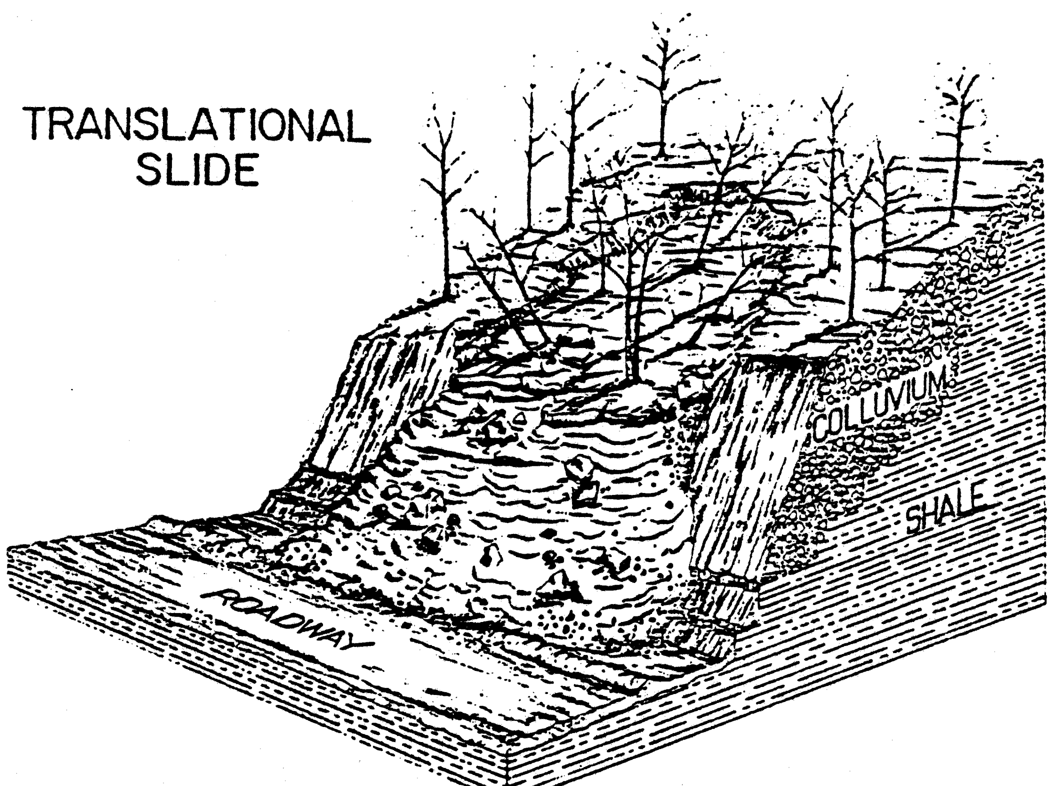


Figure 2.11. Schematic illustration of translational, debris slide(from Royster, 1978).

TABLE 2.11 FEATURES INDICATING LANDSLIDES OR AREAS WITH HIGH LANDSLIDE POTENTIAL

FEATURE	SIGNIFICANCE
1. Hummocky, dissected topography	Common feature in old and active progressive slides (slides with many individual components). Slide mass is prone to gullying.
2. Abrupt change in slope	May indicate either an old landslide area or a change in the erosion characteristics of underlying material. Portion with low slope angle is generally weaker and often has higher water content.
3. Scarps and cracks	Definite indication of an active or recently active landslide. Age of scarp can usually be estimated by the amount of vegetation established upon it. Width of cracks may be monitored to estimate relative rates of movement.
4. Grabens or "stair step" topography	Indication of progressive failure. Complex or nested series of rotational slides can also cause surface of slope to appear stepped or tiered.
5. Lobate slope forms	Indication of former earthflow or solifluction area.
6. Hillside ponds	Local catchments or depressions formed as result of (4) above act as infiltration source which can exarcerbate or accelerate landsliding.
7. Hillside seeps	Common in landslide masses. Area with high landslide potential. Can usually identify by associated presence of denser or phreatophyte vegetation (cattails, equisetum, alder, etc) in vicinity of seep.
8. Incongruent vegetation	Patches or areas of much younger or very different vegetation, e.g., alder thickets; may indicate recent landslides or unstable ground.
9. "Jackstrawed" trees	Leaning or canted trees on a slope are indicators of previous episodes of slope movement or soil creep.
10. Bedding planes and joints dipping downslope	Potential surface of sliding for translational slope movements.

IV. APPROACHES TO PREDICTING OR EVALUATING MASS-STABILITY

A. Statistical Correlations

B. Stability Analyses

1. Limiting Equilib
2. Limit Analysis
3. FEM
4. Probabilistic Methods

C. Rheological Studies

D. Requisites for Stability Analyses

1. Accurate description of slope geometry
2. Reliable soil properties - c, Φ, γ
3. Correct definition external loads - surcharge, line loads, earthquake
4. Correct description of slope hydrology - phreatic surface (GWT) & seepage
5. Correct method of analysis
   - effective vs. total stress
   - infinite vs. finite slope
   - planar vs. rotational

V. FACTOR OF SAFETY

A. Definition

Most commonly accepted definition is based on strength of soil

$$F = s/\tau \qquad (1)$$

Where s = shear strength along crit surf
τ = shear stress " " "

Rotational sliding surface:

If cylindrical surface of sliding is assumed:

$$F = M_R / M_O \qquad (2)$$

where M_R = resisting moment
M_o = overturning moment

Definitions (1) and (2) are equivalent because M_R & M_o per unit width of arc are $M_R = s \times R$ and $M_o = \tau \times R$ respectively, where R = radius of curvature of the critical surface.

C. Uncertainties in Safety Factor

1. Type of failure
   - planar vs rotational
   - shallow vs deep seated

2. Type of analysis
   - total vs effective stress anal.
   - peak vs residual shear strength

Caveat: Results of calculations only indicate safety against failure of type investigated or assumed... even if all input parameters correctly determined.

D. Acceptable Limits

Acceptable safety factors depend upon such factors as:

1. Cost of repair or reconstruction
2. Consequences of a failure
3. Permanency of slope or embankment
4. Uncertainties in shear strength measurements or other slope and soil parameters.

See Table 1 for suggested values.

TABLE 1.-- RECOMMENDED MINIMUM VALUES OF STATIC FACTOR OF SAFETY

Costs and Consequences of Slope Failure	Uncertainty of Strength Measurements	
	Small [1]	Large [2]
Cost of repair comparable to cost of construction. No danger to human life or other property if slope fails.	1.25	1.5
Cost of repair much greater than cost of construction, or danger to human life or other valuable property if slope fails.	1.5	2.0 or greater

[1] The uncertainty of the strength measurements is smallest when the soil conditions are uniform and high quality strength test data provide a consistent, complete and logical picture of the strength characteristics.

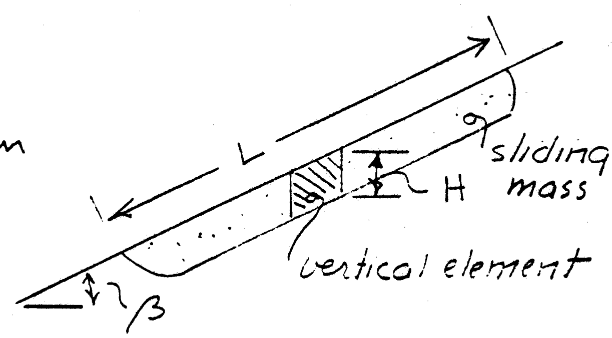
[2] The uncertainty of the strength measurements is greatest when the soil conditions are complex and when the available strength data do not provide a consistent, complete, or logical picture of the strength characteristics.

3. "INFINITE SLOPE" ANALYSIS

"INFINITE" SLOPE ANALYSIS

A. Definition

An "infinite" slope is an idealization of a slope in which the thickness of the sliding mass is small in comparison with its length and breadth, i.e.,



$$H << L$$

The failure surface is essentially planar and parallel to the slope over most of its length

B. Assumptions of Model

1. Slope has constant angle of inclination (β)
2. Conditions at top & bottom of slope mass have little or no effect on stability
3. Stability can be analyzed by considering the equilib of a single vertical element in the slope.
4. Side forces on element are opposite and equal (not considered).

C. Applicable Slopes & Geologic Conditions

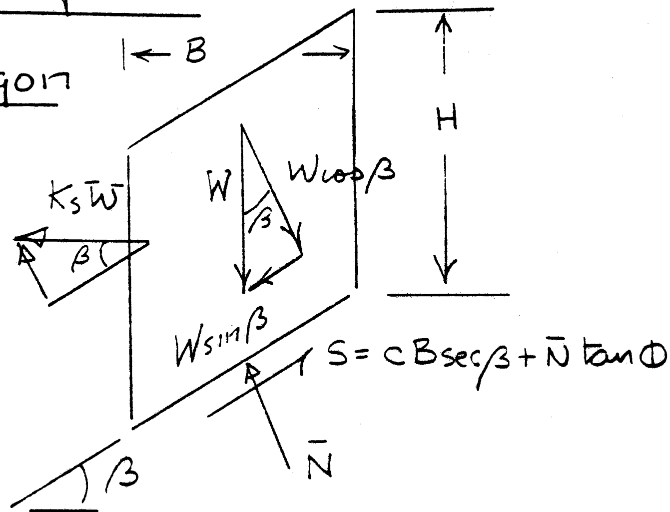
1. Shallow sloughing or face sliding in sandy slopes
2. Slopes with inclined planes of weakness (stratification, joints, etc) dipping downslope
3. Loose products of weathering (residual soils, colluvium, etc) or till mantle on inclined bedrock surface.

D. Factor of Safety Equations

1. Space-Force Polygon

$D.F. = W\sin\beta + K_s\cos\beta$

$R.F. = cB\sec\beta + \bar{N}\tan\Phi$

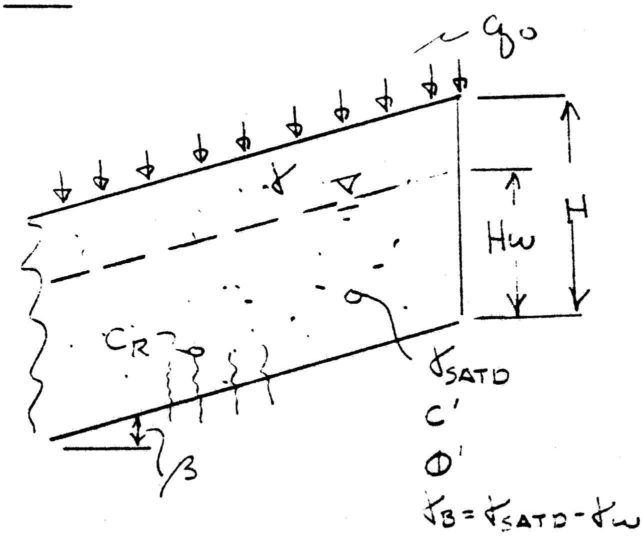


2. General Case

q_o = vertical surcharge

K_s = seismic coef

C_R = cohesion contrib from root fibers



$$F = \frac{\left[\dfrac{(c'+C_R)}{\cos^2\beta\tan\Phi'} + \left\{q_o + \gamma H(1 - K_s\tan\beta)\right\} + (\gamma_B - \gamma)H_w\right]\dfrac{\tan\Phi'}{\tan\beta}}{\left[\left\{q_o + \gamma H\left(K_s/\tan\beta + 1\right) + (\gamma_{SATD} - \gamma)H_w\right]} \qquad (3)$$

alternative notation ($q_o = 0$):

$$F = \frac{\left[\dfrac{(c'+C_R)/\gamma H}{\cos^2\beta\tan\Phi'} + \left\{(1 - r_u) - K_s\tan\beta\right\}\right]\dfrac{\tan\Phi'}{\tan\beta}}{\left[(K_s/\tan\beta) + 1\right]} \qquad (4)$$

where r_u = pore water pressure ratio.

Approx estimate of r_u:

Assume $\gamma_{SATD} \approx \gamma$; $\gamma_B \approx \frac{1}{2}\gamma$

then $$1 - r_u = \frac{H - \frac{1}{2}H_w}{H} \quad ; \quad r_u \approx \frac{H_w}{2H} \qquad (5)$$

3. Special Cases

a) Dry, Cohesionless Slope ($c'=0$, $q_o=0$, $H_w=0$, $K_s=0$)

$$F = \frac{\tan\Phi'}{\tan\beta} \qquad (6)$$

Note: F for submerged slope is SAME?

b) Satd, Cohesionless Slope ($c'=0$, $q_o=0$, $H_w=H$, $K_s=0$)
seepage ∥ to slope

$$F = \frac{\gamma_{SATD} - \gamma_w}{\gamma_{SATD}} \frac{\tan\Phi'}{\tan\beta} \qquad (7)$$

since $\gamma_{SATD} - \gamma_w \approx \frac{1}{2}\gamma_{SATD}$

$$F_{SATD} \approx \frac{1}{2} F_{DRY}$$

c) Moist, Cohesive Slope ($\Phi=0$, $q_o=0$, $H_w=0$)
no GWT

$$F = (c/\gamma H) \times \frac{1}{\cos^2\beta \tan\beta} \qquad (8)$$

where H = depth to firm layer or bedrock

E. Computer Program

"INSLOPE" - Calcs either seismic or static factor of safety for most general case
$F = f(c, \Phi, \gamma_{SATD}, \gamma, H, H_w, K_s, q_o, C_R)$

F. Parameter Sensitivity Analysis

1. Influence of Surcharge

Surcharge is beneficial under the following conditions:

$\gamma_s = 0 \qquad c < H_w \gamma \cos^2\beta \tan\phi \qquad (9)$

2. Influence of Shear Strength Parameters

For thin soil mantles and high GWT ($H_w \to H$) small changes in cohesion (c) have a bigger influence on safety factor than corresponding changes in friction angle (ϕ). See figure below... granitic slope.

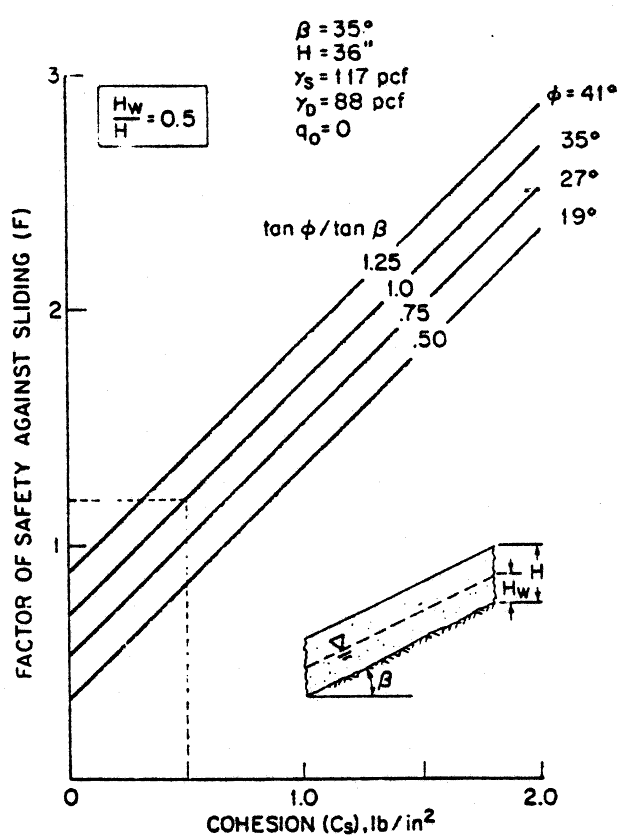


Figure 28. — Influence of cohesion on the stability of a sandy residual soil resting on an inclined bedrock contact. $\frac{H_w}{H} = 0.5$.

3. General Sensitivity Analysis

A general method can be employed that reveals not only the sensitivity but also the direction of change in safety factor corresponding to a change in any input variable in the general infinite slope equation.

The approach is conducted in five steps:

a) Select a realistic range of values $(\Delta X_i)_{max}$ for each input variable (X_i) & compute the mean value $\bar{X}_i$.

b) Compute a base factor of safety $(\bar{F}_o)$ using the median values for all variables

c) Compute safety factor $F(X_i)$ as each parameter is varied across its range

d) Calculate the relative change in safety factor ΔF_i as a percent of the base safety factor

$$\Delta F_i (\%) = \frac{F(X_i) - \bar{F}_o}{\bar{F}_o}$$

e) Display results graphically by plotting ΔF_i for each parameter vs. ΔX_i as shown

$$\Delta X_i (\%) = \frac{\bar{X}_i - X_i}{(\Delta X_i)_{max}} \times 100$$

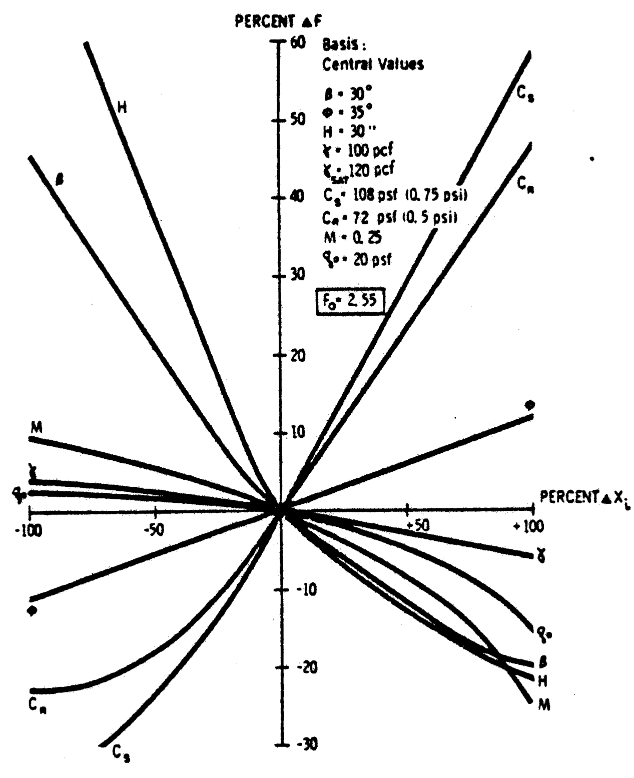


Figure 30. — Percent change in slope safety factor versus percent change in input variables. Base safety factor (F) was computed using the median or central value for all variables (table 7).

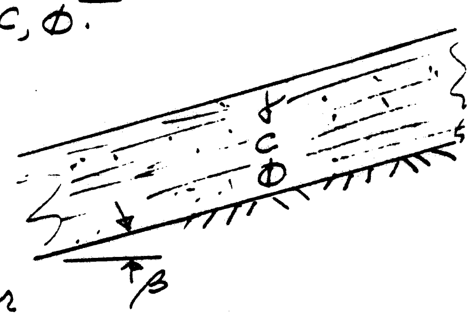
STABILITY CHARTS

I. "INFINITE" SLOPE - ENGR. FIELD MANUAL (UC)

A. Total Stress Analysis

Effect of pore pressure (seepage) not considered explicitly... only via value of c, ϕ.

$$F = \frac{\tan\phi}{\tan\beta} + B\frac{c}{\gamma H}$$



where c, ϕ = total stress, shear str. parameters.

γ = total unit wt.

$$B = \frac{1}{\cos^2\beta\tan\beta} \quad \text{(see Fig. 10)}$$

B. Effective Stress Analysis

Influence of pore pressure considered explicitly. Must ascertain seepage conditions in slope from flow net or piezometer study.

$$F = A\frac{\tan\phi'}{\tan\beta} + B\frac{c'}{\gamma H}$$

where c', ϕ' = effective stress, shear str. parameters

$A = f(\beta, r_u)$
$B = f(\beta)$ 〉 see chart in Fig. 10

r_u = pore pressure parameter $= \dfrac{u}{\gamma H}$

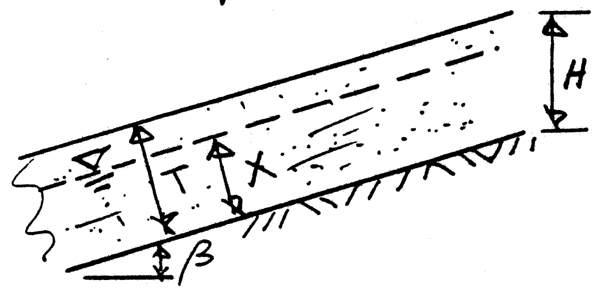
u = pore pressure

H = depth (vert) corresponding to pore pressure.

I. "INFINITE" SLOPE - EFF. STRESS ANAL. (cont.)

Case ① - Seepage Parallel to Slope

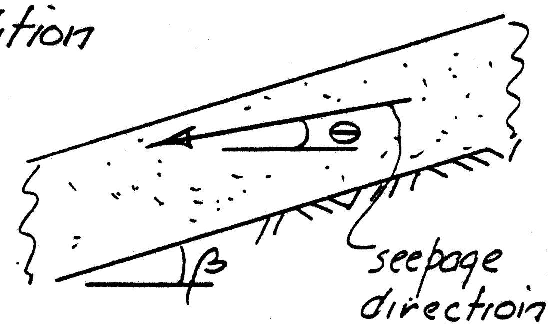
$$r_u = \frac{X}{T} \cdot \frac{\gamma_w}{\gamma} \cos^2\beta$$



where X = dist. from sliding surface to GWT (normal)
T = dist " " " to slope surface "

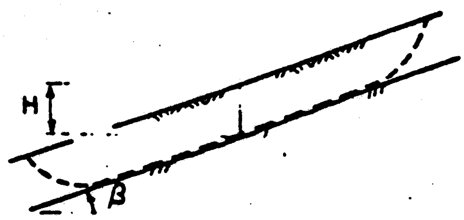
Case ② - Seepage Emerging from Slope

More critical condition



$$r_u = \frac{\gamma_w}{\gamma} \cdot \frac{1}{1 + \tan\beta \tan\phi}$$

where θ = angle of seepage measured from horizontal.



γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept } Effective

ϕ' = friction angle } Stress

r_u = pore pressure ratio = $\frac{u}{\gamma H}$

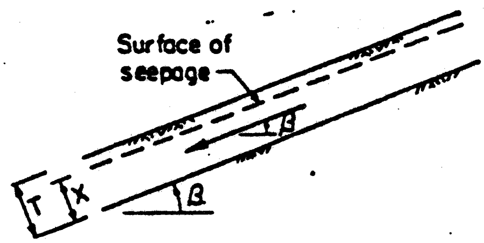
u = pore pressure at depth H

Steps:

① Determine r_u from measured pore pressures or formulas at right

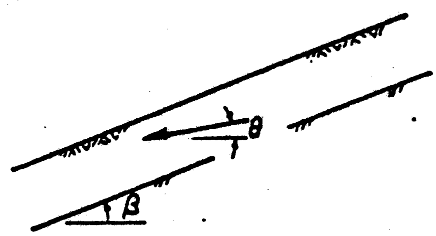
② Determine A and B from charts below

③ Calculate $F = A \frac{\tan\phi'}{\tan\beta} + B \frac{c'}{\gamma H}$



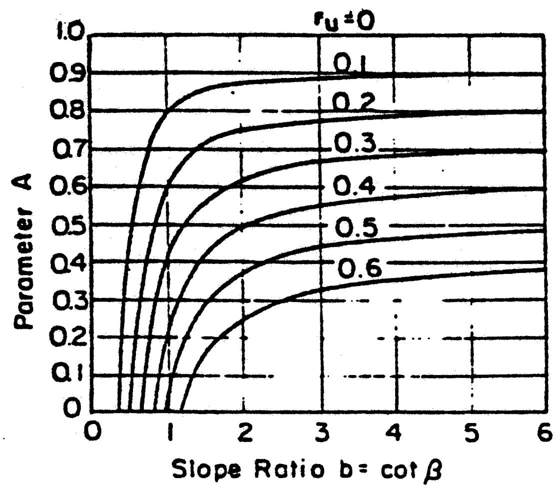
Seepage parallel to slope

$r_u = \frac{X}{T} \frac{\gamma_w}{\gamma} \cos^2\beta$



Seepage emerging from slope

$r_u = \frac{\gamma_w}{\gamma} \frac{1}{1 + \tan\beta \tan\theta}$



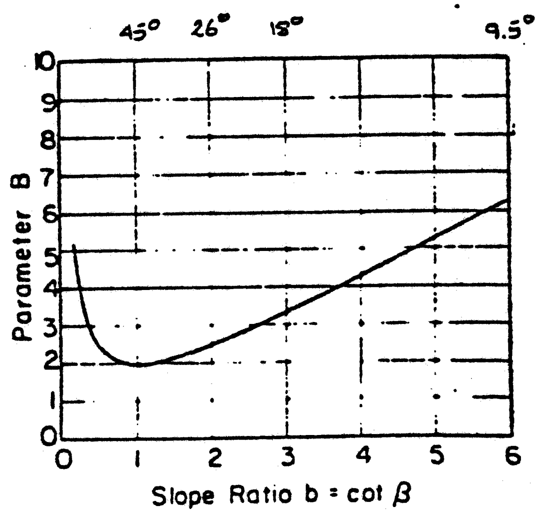
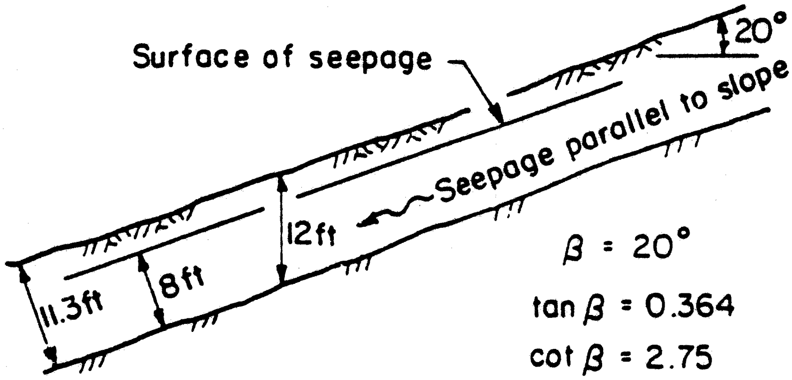


Fig. 10 STABILITY CHARTS FOR INFINITE SLOPES.

FROM "SLOPE STAB. MANUAL" by DUNCAN



For seepage parallel to slope, with X = 8 ft., T = 11.3 ft. } Formula from Fig. 10

$$r_u = \frac{8}{11.3}\ \frac{62.4}{120}\ (0.94)^2 = 0.325$$

From Fig. 10, $A = 0.62$ for $r_u = 0.325$ and $\cot\beta = 2.75$

$B = 3.1$ for $\cot\beta = 2.75$

$$F = 0.62\,\frac{0.577}{0.364} + 3.1\,\frac{300}{(120)(12)} = 0.98 + 0.65 = 1.63$$

For horizontal seepage emerging from slope, $\theta = 0$ } Formula from Fig. 10

$$r_u = \frac{62.4}{120}\ \frac{1}{1+(0.364)(0)} = 0.52$$

From Fig. 10, $A = 0.41$ for $r_u = 0.52$ and $\cot\beta = 2.75$

$B = 3.1$ for $\cot\beta = 2.75$

$$F = 0.41\,\frac{0.577}{0.364} + 3.1\,\frac{300}{(120)(12)} = 0.65 + 0.65 = 1.30$$

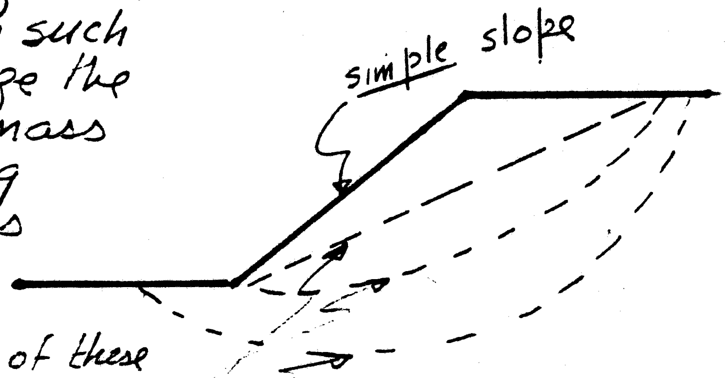
Fig. 14 EXAMPLE OF USE OF INFINITE SLOPE CHARTS.

4. FINITE SLOPE ANALYSIS — AN OVERVIEW

FINITE SLOPE ANALYSES – AN OVERVIEW

Definition

Finite slope consists of sloping grade terminated at its top & bottom by a horizontal surface. The dimensions of the slope are such that more reasonable to analyze the stability of the entire slope mass with respect to sliding along various assumed surfaces of failure.

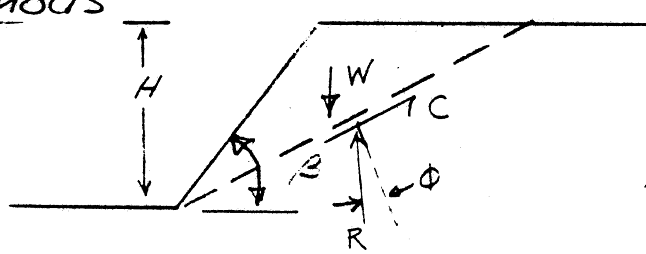


NOTE: Unless otherwise noted, all of these analyses are based on "Limit equilib" approaches.

I "SLIDING WEDGE" APPROACH

A. Classical Methods

Coulomb
Culman

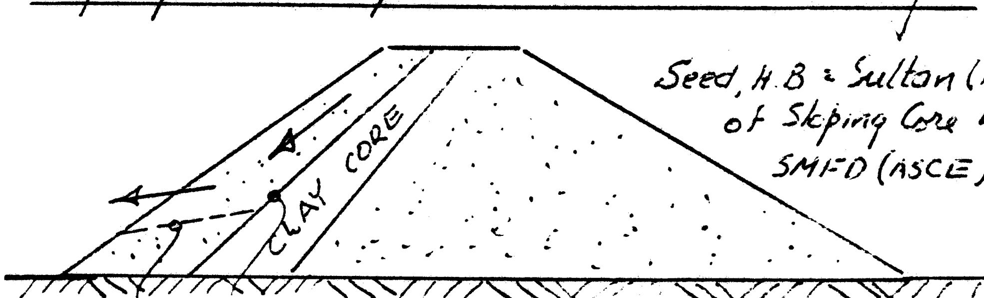


Assume:
1. planar sliding surface
2. Homogeneous soil

Culman, K (1866). Die Graphische Statik, Berlin

B. Sloping Core & Hydraulic Fill Dam Analysis

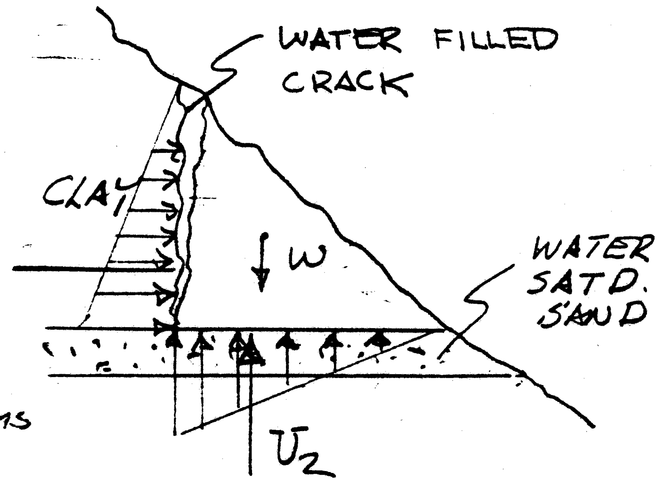
Seed, H.B & Sultan (1967). "Stability of Sloping Core Earth Dams," J. of SMFD (ASCE), Vol 93, No. SM4.



C. Fissured, Stratified Slope

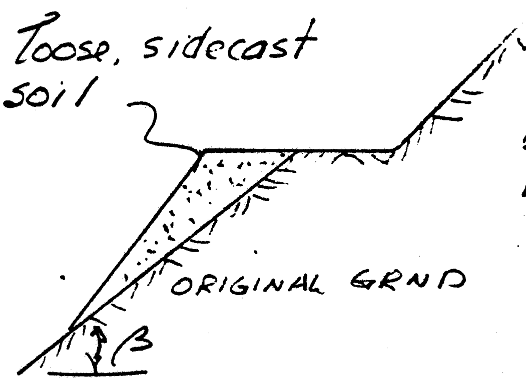
U_1 water force tend to slide wedge out

U_2 water pressure reduces effective stress at base



Porcher & Means (1968)
Soil Mechanics & Foundations
p. 496

D. Debris Slides in Sidecast Road fills

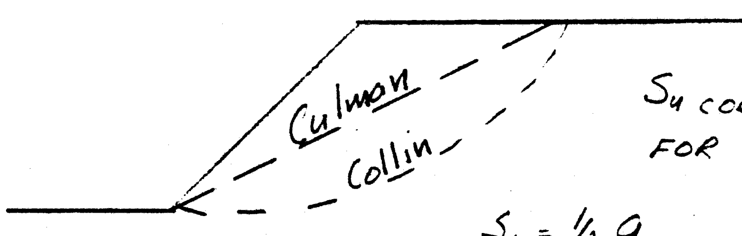


Burroughs et al, "Slope Stability in Road Const.", BLM, U.S. Dept. of the Interior, Oreg State Office, 101 pp.

II CIRCULAR ARC ANALYSES

A. Mass Methods

① Collin - noticed that actual failure surface curved. Shear str. required for equilib greater along curved surfaces than along planar... except very steep slopes



$S_{u\,COLLIN} > S_{u\,CULMAN}$ FOR EQUILIB

$S_u = \frac{1}{2} q_{UNCONFINED}$ $(\phi = 0)$

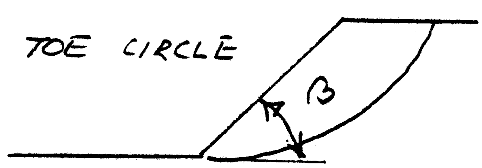
② Hulten & Peterson - developed "friction-circle" method of analysis to take into account presence of internal friction in soil. Homogeneous slopes

③ Fellenius & Taylor - developed stability charts[1] or nomographs for use with above analyses. Total stress type of analysis... pore pressure not taken into account

Stability Number (N_s) obtained from charts

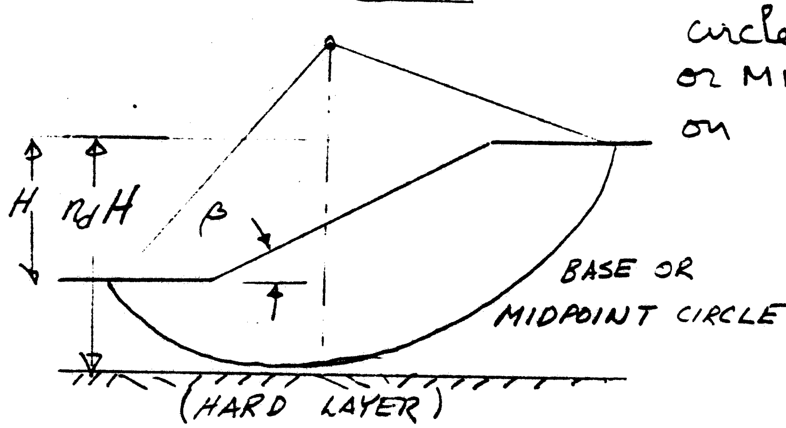
$$N_s = \frac{\gamma H_c}{c} \qquad H_c = \text{crit slope ht.}$$

Case i) $\phi = 0$ soil



$N_s = f(\beta)$ if $\beta > 53°$, all failure circles will be TOE circles

$N_s = f(\beta, n_d)$ if $\beta < 53°$, failure circle can be either TOE, SLOPE, or MIDPOINT circle depending on n_d



n_d = depth factor

Case ii) $\phi \neq 0$ soil

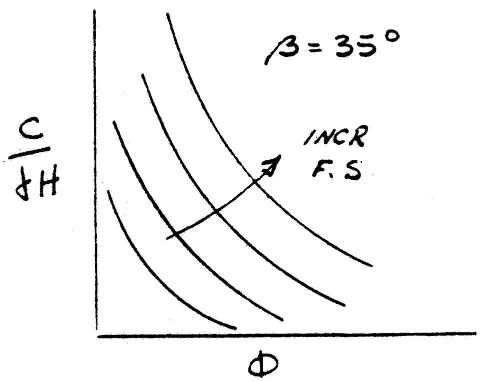
$N_s = f(\beta, \phi)$ all circles will be TOE circles provided $\phi \geq 5°$

(1) Charts may be found in:

Taylor, D.W. (1937). "Stab. of Earth Slopes," Journ Boston Soc Civil Engineers, Vol. 24, pp. 197-224

Terzaghi & Peck (1967). Soil Mechanics in Engr. Practice John Wiley

② **Singh** - developed nomograph solutions for selected slope angles based on Taylor stability charts which yield a balanced factor of safety directly (avoids trial & error proc necessary with charts)

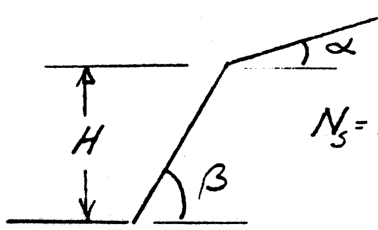


Assumptions
1. Homogeneous slope
2. Total stress anal (no seepage)

J. of SMFD (ASCE) "Shear Strength and Stability of Man Made Slopes" Vol. 96, No. SM6, pp. 1879 (1970)

Also showed that diff in FS between "mass method" (Taylor) and "slices method" (Janbu) insignificant compared to differences introduced by errors in shear strength parameters.

③ **Chen & Giger** - calculated stability nos. and developed stability charts for slope terminated at its upper end by another sloping surface (e.g. cut slope in a natl. slope)



$N_s = f(\beta, \alpha, \Phi)$

Assumptions:
1. Homogeneous slope
2. Total stress anal (no seepage)
3. Upper bound, limit anal based on plasticity theory
4. Log-spiral failure surf

Chen & Giger (1971). "Limit Analysis of Stability of Slopes," J. of SMFD (ASCE), Vol. 97, No. SM1

④ **Prellwitz** - devised a modified stability chart or method for analyzing the stability of a cut slope with seepage

$H_{CRIT} = N_s \times R$

$R = f\left(\frac{H_W}{H}, \frac{H_R}{H}\right)$

H_W, H_R = piezometric surface parameters

"Simplified Slope Design for Low Std. Roads in Mountainous Regions, NRC-TBR SPEC REPT #160 (1975)

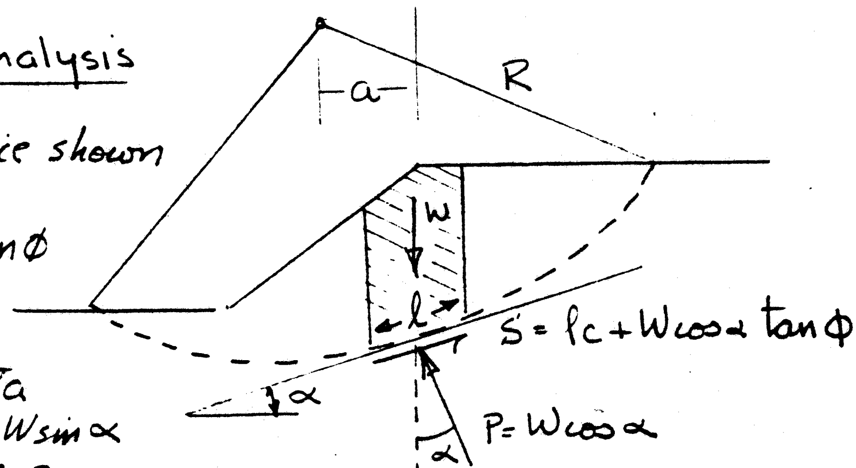
B. Method of Slices

Trial failure mass (bounded by a lower circular arc) is broken up into a suitable no. of vertical slices. The forces acting on each slice are computed. Stability is determined by computing the ratio of resisting to overturning moment for each slice.

① Conventional (Ordinary) Method

Assumptions: Side forces are neglected... force equilib is not satisfied; only overall moment equilib.

a) Total Stress Analysis



...for shaded slice shown

$O.M. = W \cdot a$

$R.M. = R\{cl + P\tan\phi\}$

...for entire slope

$$\text{Total } O.M. = \Sigma Wa = R\,\Sigma W\sin\alpha$$

$$\text{Total } R.M. = \Sigma S \cdot R = R\,\Sigma\{cl + P\tan\phi\}$$

$$F.S. = \frac{\text{Tot. } R.M.}{\text{Tot. } O.M.} = \frac{\Sigma\{cl + W\cos\alpha\tan\phi\}}{\Sigma W\sin\alpha}$$

b) Effective Stress Analysis

Use c' & ϕ' and incl. pore pressure (u) @ base of ea. slice

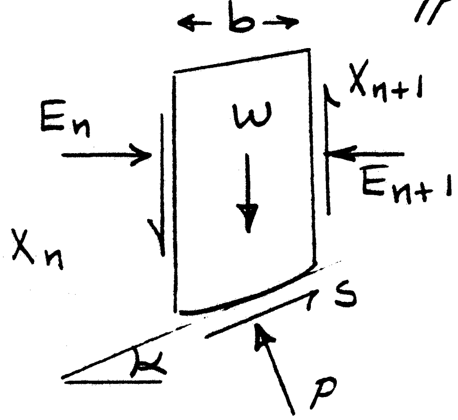
$$F.S. = \frac{\Sigma\{c'l + (W\cos\alpha - ul)\tan\phi'\}}{\Sigma W\sin\alpha}$$

Reference:

Fellenius, W. "Calc of Stability of Earth Dams," Trans. 2nd Congress on Large Dams (Washington), Vol. 4, p. 445 (1936).

② Modified (Bishop) Method

Assumptions: Overall moment equilib and vertical force equilib on each slice are satisfied by assuming opp. & equal vertical side forces, i.e.



$$(X_n - X_{n+1}) = 0$$

$$F.S. = \frac{\sum \left[\{ c'b + (W - ub)\tan\phi' \right] \frac{1}{m_\alpha}}{\sum W \sin\alpha}$$

$$\frac{1}{m_\alpha} = \frac{\sec\alpha}{1 + (1/F.S.)\tan\phi' \tan\alpha}$$

Bishop A.W. (1955). "The Use of the Slip Circle in Slope Stab. Analyses," Geotechnique, Vol 5, No. 1, pp. 7-17

③ Modified Method (Stability Coefficients)

A nomograph solution based on the Bishop method in which pore water pressure is taken into account by computing an "average" or weighted pore water pressure ratio (r_u) for entire slope.

$$F.S. = m - n r_u$$

$$m = f(\phi', \beta, c/\gamma H)$$

$$n = g(\phi', \beta, c/\gamma H)$$

$$r_u = \frac{u}{\gamma_w h}$$

Bishop, A.W. & Morgenstern, N. (1960) "Stability Coefficients in Earth Slopes," Geotechnique Vol 10, No. 4,

Cousins, B.F. (1978). "Stability Charts for Simple Earth Slopes," J. of GET (ASCE). Vol. 104 No. GT2.

$$F.S. = N_F \times \frac{c'}{\gamma H} \qquad N_F = f(\beta, \lambda_{c\phi}, r_u, D)$$

$$\lambda_{c\phi} = \text{dimensionless shear str. parameter} = \frac{\gamma H \tan\phi'}{c'}$$

III. IRREGULAR FAILURE SURFACES

Stability is still calculated by considering stability of vertical slices in the assumed failure mass. All requirements for static equilibrium are satisfied; therefore, shear surfaces of any shape can be analyzed.

Various methods require user to either prescribe (and check) or assume a distributional pattern between side forces, e.g.,

Morgenstern-Price: $X_i = \lambda f(x) E$
Spencer: side forces assumed parallel inclination (θ) $X = E \tan\theta$
Janbu: assume value for X_1 and check with calc. values from force equilib solutions

Morgenstern, N.R. & Price, V.E. (1965). "The Analysis of the Stability of General Slip Surfaces," Geotechnique Vol 15, No. 1, pp. 79-93

Spencer, E (1967). "A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces," Geotechnique Vol. 17, No. 1, pp. 11-26

Janbu, N. (1955). "Application of Composite Slip Surfaces for Stability Analysis," Proc of European Conf. on Stab of Earth Slopes, Stockholm, Vol. 3, 1955, pp 43-49

Wright, S.G (1973). "Accuracy of Equilib. Slope Stability Analysis," Journ. of SMFD (ASCE) Vol. 99, No. SM 10, pp. 783

→ Bailey, W.A. & Christian, J.T. (1969). "ICES-LEASE-I," A Problem Oriented Language for Slope Stability Analyses, Users Manual," MIT Soil Mech Publ. No. 235 MIT, Cambridge, Mass

5. REVETMENT DESIGN FOR COASTAL SLOPES

(from U.S. Army Corps of Engineers Design Manual)

EXAMPLE DESIGN PROBLEM

A low bluff shoreline will be examined (Figure 25). The nine-foot bluff face is steep and there is no fronting beach except at low tide when a gravel-covered shoreline is exposed. At high tide, the water is just above the toe of the bluff. The bluff soils are sandy-silt and the offshore slope is mild. The shoreline has steadily receded for years as is evident by the fallen trees along the shore.

Figure 25 Example Design Problem Site

The owner purchased the land to construct a retirement home (no structures have yet been built). He intends to extend a dock out to deeper water but he has no other plans for the shoreline. After examining the site, he determines that no slope stability or groundwater seepage problems exist and erosion is being caused by wave action undercutting the bluff toe (Figure 26).

Figure 26 Eroded Bluff

Design Parameters

Figure 27 is a profile of the site. As shown on the figure, the water depth was measured, the depths at spring tide and the design stillwater level were calculated (using the Tide Tables) and the design wave height was determined following procedures described earlier in the text. The bluff toe is susceptible to erosion during the design storm when it is submerged under approximately 3.0 feet of water. A protective device must be installed to prevent further erosion.

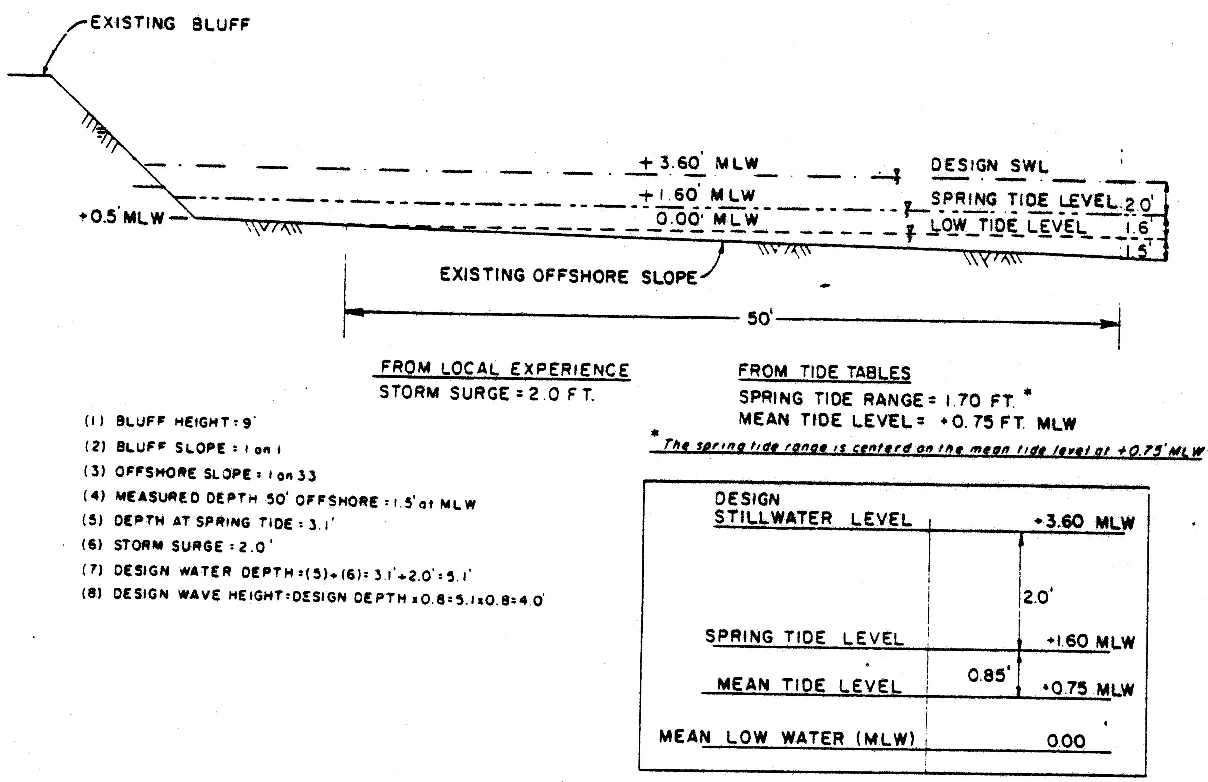


Figure 27 Profile and Physical Conditions at the Site

Device Selection

Bulkheads are applicable to low bluffs if frequent access to the shore is not required. At this site, a bulkhead would not interfere with the planned use of the property and once backfilled to the height of the existing bluff, the amount of useable land would be increased. Any of the sheet pile bulkheads would easily meet the design wave and other criteria. It is likely, however, that their material and installation costs at this site would be significantly greater than other devices that would also meet the design criteria. The post supported bulkhead using treated timber sheeting is well suited and will be illustrated in this design example. The Longard tube was not selected for aesthetic reasons and its short life expectancy when exposed to water-borne debris. The stacked used tire and used concrete pipe bulkheads were rejected because they did not meet the design wave criteria.

Regrading is acceptable, as the land is undeveloped, so revetments are a possibility. Little useable land would be lost in this case because of the low bluff height. A rubble revetment is a likely alternative because stone is available in the area at a reasonable price. A typical design will be shown. Concrete blocks would also be applicable. The steps involved in designing concrete block and stone revetments are similar and therefore will not be repeated, but in a real design, the comparative costs of stone and concrete block revetments should be developed. Stacked bags and mats were eliminated for aesthetic reasons and because of short life expectancy when exposed to water-borne debris and bombardment by stones and cobbles. Gabions were also judged to be too short-lived in this situation. Materials for fuel barrel and concrete slab revetments were unavailable.

A breakwater does not provide positive protection to the bluff toe. To avoid downdrift erosion problems, sand would have to be imported from a borrow area nearby. This would require additional expense and would still not provide positive toe protection. Therefore, all breakwaters were eliminated. Groins were also rejected for the same reasons.

A beach fill and a perched beach were considered as possibilities because the offshore slope is mild. However, they do not positively protect the bluff toe and enhanced recreational use of shoreline was not a high priority of the owner. Because neither would provide the needed protection, they were not selected as possible alternatives. Slope flattening or infiltration and drainage controls were inappropriate. Slope flattening, however, would be a part of the revetment design and proper groundwater drainage would be included in all designs.

Vegetation, if used alone, would be ineffective. Completion of the Vegetation Stabilization Site Evaluation Form (Figure 28) yields a score of 32, which places the site just beyond the acceptable range.

One possible combination approach will be developed that employs devices that were rejected when considered alone: a gabion revetment, a perched beach retained by a sand bag sill, and vegetation. The vegetation will provide a buffer zone to inhibit wave action against the bluff toe. The existing gravel beach will not support plantings so the perched sand beach and sill are provided to encourage plant growth while also protecting the new plantings against wave action. A recomputation of the Evaluation Form (Figure 28), with a perched beach of medium sand, yields a score of 28, which is in the acceptable range.

Design Example No. 1 - Treated Timber Bulkhead (Figure 29)

Runup Calculation. From Table 4, with the design wave height, H = 4.0 feet for a vertical face

$$R = 2.0\ H = 2.0 \times 4.0 = 8.0 \text{ feet}$$

The design top of structure is at the crest of the bluff, or +9 feet MLW. The runup above the design stillwater level would be to +11.6 feet MLW (8.0' + 3.6'). The structure, therefore, will be overtopped during design conditions, and a splash apron must be provided at the crest.

Backfill. Only granular backfill material should be used. The fill must be placed and compacted around the deadmen before any is placed behind the wall. Otherwise, load would be applied to the wall without support of the anchoring system and failure could result.

Filter Cloth. A continuous filter cloth is provided behind the planks and under the overtopping apron. It is needed to prevent the backfill and natural bluff material from being washed out. Additional holes in the wall were not included in the design because the small spacing between the planks will provide sufficient drainage.

Toe Protection. Toe protection is provided to insure stability against scour. A filter cloth is used to prevent settlement of the rock. Given a unit weight of stone, w_r = 165 lbs/ft^3 and a design wave height, H = 4.0 feet; from Tables B-1, B-2 and B-3 (*APPENDIX B*), find the required stone weight, W.

From Table B-1, with H = 4.0 feet, W = 390 pounds.

The toe protection is placed on an essentially flat surface so enter Table B-2 with the flattest slope shown (1:6) to correct W.

From Table B-2, for a 1 on 6 slope (1:6), the correction factor, K_1 = 0.3. Therefore,

$$W = 390 \times 0.3 = 120 \text{ pounds}$$

The range of allowable stone weights is 0.75W to 1.25W with 75% greater than W, therefore,

$$W_{min} = 0.75 \times 120 = 90 \text{ pounds}$$
$$W_{max} = 1.25 \times 120 = 150 \text{ pounds}$$

Seventy-five percent should exceed 120 pounds and no stone should be accepted if the longest dimension is more than three times the shortest.

Flanking. The bulkhead will be tied into the existing bluff.

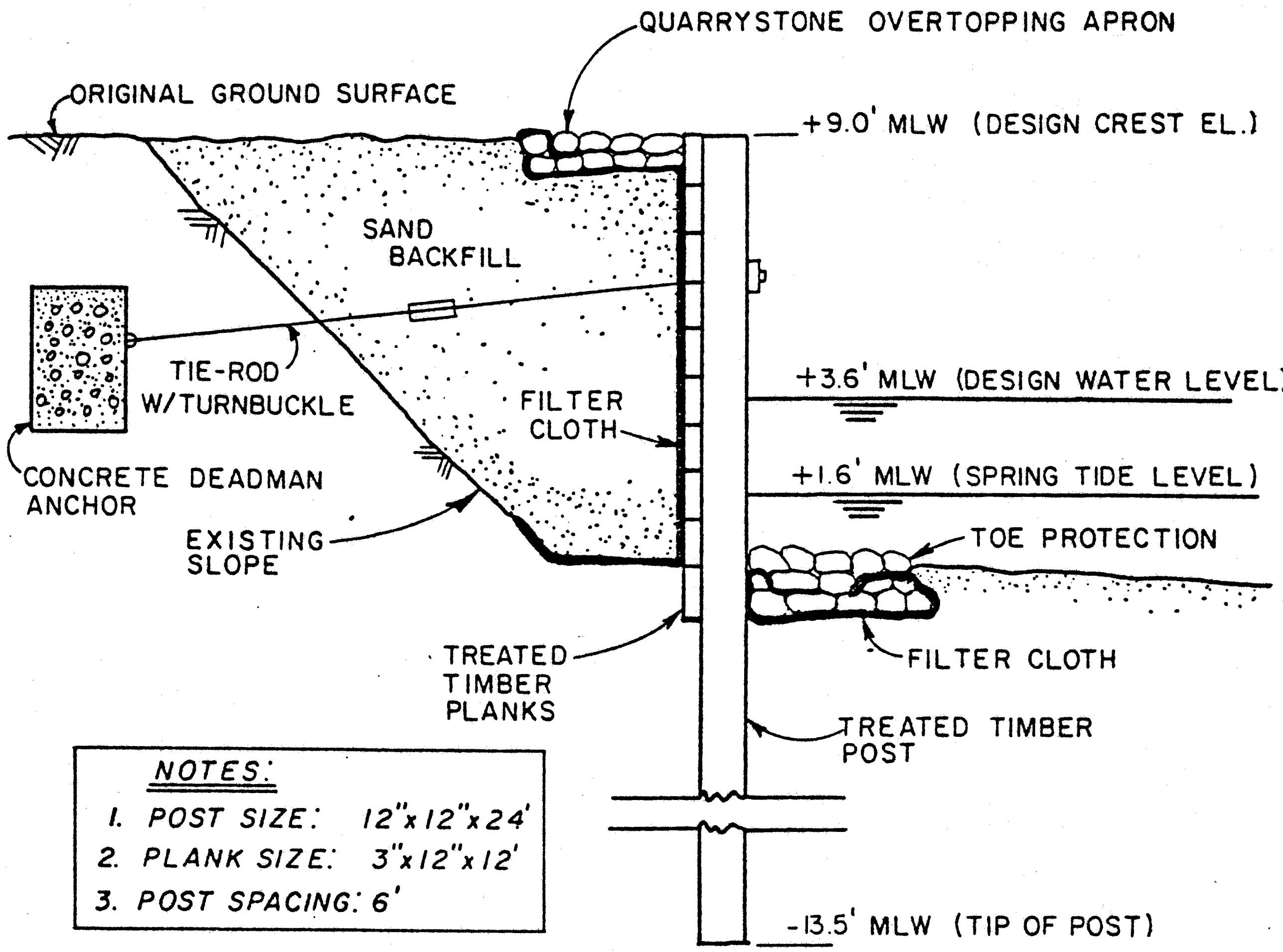


Figure 29 Treated Timber Bulkhead

Design Example No. 2 - Quarrystone Revetment (Figure 30)

Required Slope. The selected slope angle depends on the amount of available land at the top of the bluff, the amount of runup expected and the cost of materials. A milder angle results in less runup, smaller stone sizes, greater slope length, and more loss of land. A 1 on 2.5 slope was selected.

Runup Calculation. From Table 4, for a rough face structure with a 1 on 2.5 slope, the runup, R = 1.0 H or 4.0 feet. The revetment must extend vertically 4.0 feet above the design still-water level. There is enough height available (5.4 feet) and no splash apron is required.

Weight of Armor and Underlayer Stone. Use Tables B-1 and B-2.

Armor Stone. Given: H = 4.0 feet

W = 390 pounds (Table B-1)

K_1 = 0.8 (Table B-2) for a 1 on 2.5 slope

Therefore: Use W = 390 x 0.8 = 310 pounds.

Underlayer Stone. Use W/10 = 30 pounds.

Range of Allowable Stone Weights. The range for both the armor and underlayer is 0.75W to 1.25W with 75% of the stones weighing more than W. All stones should be sized so that no side is greater than 3 times its least dimension.

Armor Layer. W_{min} = 0.75 x 310 = 235 pounds.
W_{max} = 1.25 x 310 = 390 pounds.
75% must exceed 310 pounds.

Underlayer. W_{min} = 0.75 x 30 = 20 pounds.
W_{max} = 1.25 x 30 = 40 pounds.
75% must exceed 30 pounds.

Filter Cloth. A properly sized filter cloth will allow water to pass while retaining the bluff soil. When laying the filter cloth, insure continuous coverage by overlapping the cloth edges at least 18 inches.

Toe Protection. Bury the toe (extend the slope down into shore) at least one design wave height below the existing bottom. Add an additional layer of armor stone to thicken the toe section.

Flank Protection. The revetment will be tied into the existing bank.

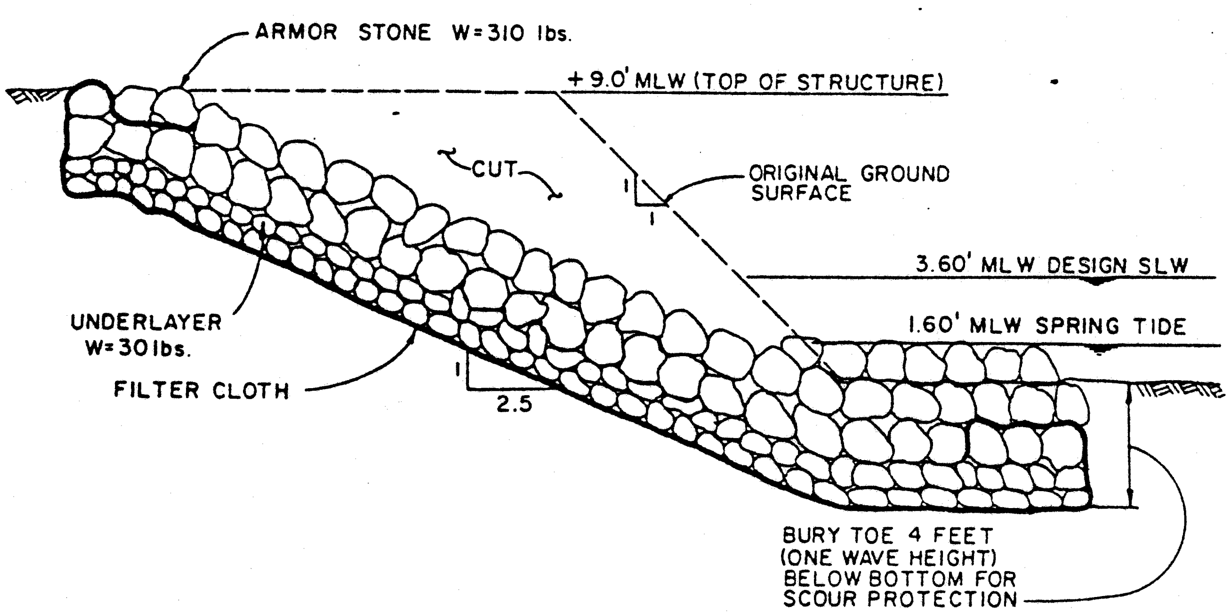


Figure 30 Quarrystone Revetment

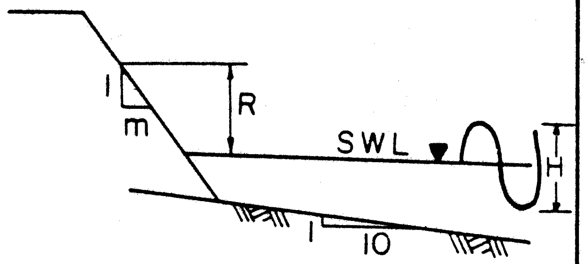
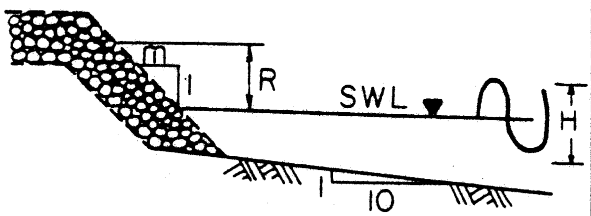
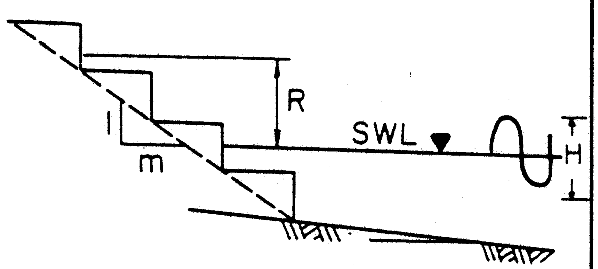
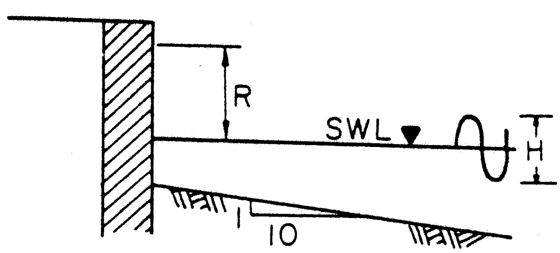
 SMOOTH FACE	m 1.5 2.5 4.0	R 2.25H 1.75H 1.50H
 ROUGH FACE	m 1.5 2.5 4.0	R 1.25H 1.00H 0.75H
 STEPPED FACE	m 1.5	R 2.00H
 VERTICAL FACE	m —	R 2.00H

Table 4 Wave Runup Heights

Quarrystone

Wave Height Range: Above five feet.

Runup Characteristic: Rough slope.

Stone revetments, a proven method of shoreline protection, are durable and can be relatively inexpensive with a local source of suitable armor stone. Such stone should be clean, hard, dense, durable, and free of cracks and cleavages. Figure B-3 shows a typical cross section of a stone rubble revetment. Table B-1, which gives the required weight of armor units for a given design wave height, was developed for a 1:2 bank slope and a stone unit weight of 165 lbs/ft^3. If your bank slope is something other than 1:2, find the value on Table B-2 and multipy the stone weight from Table B-1 by this factor. Flatter slopes require smaller rock sizes. Table B-3 contains a second correction factor to be applied when the unit weight (density) of the rock varies from 165 lbs/ft^3. The tables contain an illustrative example of their use.

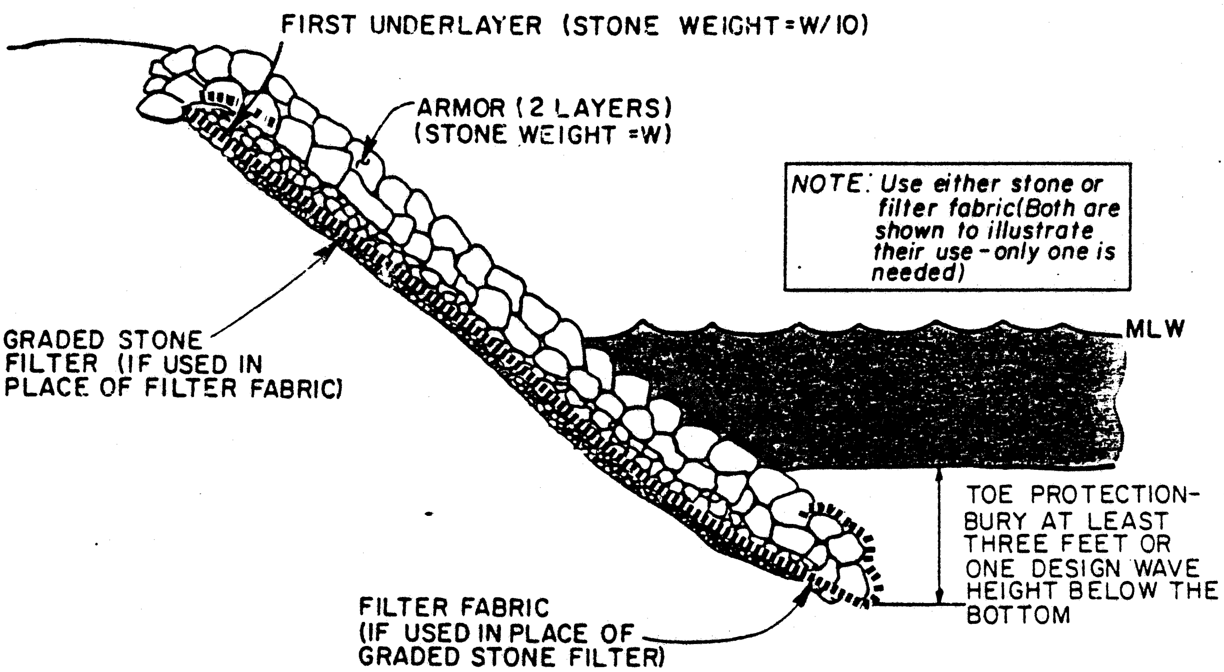


Figure B-3 Typical Quarrystone Revetment

Since it is not possible to obtain quarrystones of exactly the same weight, one must specify a range of permissible sizes. For any given required weight, W, stones ranging from 0.75W to 1.25W can be used, but at least 75% should weigh W or more. For example, if 100-pound stones are required, the armor stones may range from 75 to 125 pounds, as long as 75% weigh at least 100 pounds.

If graded stone filter material is used, it generally will be much finer than the armor stone. An intermediate layer of stone, between the armor and filter, one-tenth as heavy as the armor units (100/10 = 10 pounds in the example), should provide the necessary transition to the filter material.

TABLE B-1 ESTIMATED WEIGHT OF ARMOR STONE		TABLE B-2 CORRECTION FOR SLOPE		TABLE B-3 CORRECTION FOR UNIT WEIGHT	
WAVE HEIGHT H (ft)	ESTIMATED WEIGHT W (lb)	SLOPE (ft/ft)	CORRECTION FACTOR K_1	UNIT WEIGHT w_r (lb/ft^3)	CORRECTION FACTOR K_2
0.5	1	1 : 2	1.0	120	4.3
1.0	10	1 : 2½	0.8	130	2.8
1.5	20	1 : 3	0.7	135	2.4
2.0	50	1 : 3½	0.6	140	2.0
2.5	100	1 : 4	0.5	145	1.7
3.0	160	1 : 4½	0.4	150	1.5
3.5	260	1 : 5	0.4	155	1.3
4.0	390	1 : 5½	0.4	160	1.1
4.5	550	1 : 6	0.3	165	1.0
5.0	750			170	0.9
5.5	1000			175	0.8
6.0	1300			180	0.7
6.5	1650			185	0.6
7.0	2100			190	0.6

EXAMPLE

GIVEN: The wave height (H) is 3.0 feet and the structure slope is 1 on 3 (1 Vertical on 3 Horizontal) and one cubic foot of rock weighs 155 lbs (w_r)

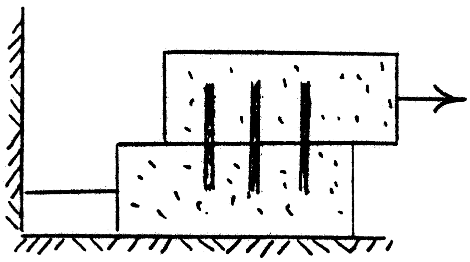
FIND: The required weight of armor stone (W) from the tables (Dashed Line)

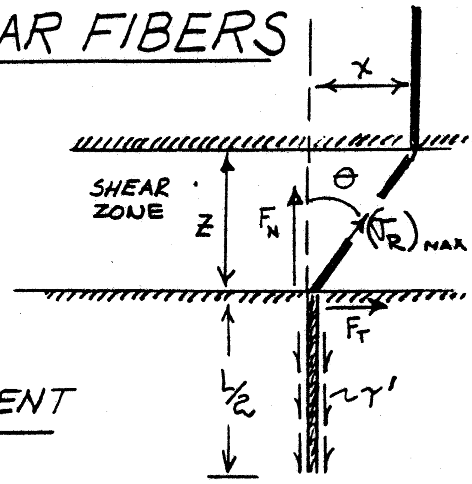
W = 160 lbs x 0.7 x 1.3 = 145 lbs

6. MECHANICS OF FIBER REINFORCEMENT IN SOILS

MECHANICS OF FIBER (FABRIC) REINFORCEMENT OF DRY SANDS

I. DIRECT SHEAR – PERPENDICULAR FIBERS





A. SHEAR STRENGTH INCR. FROM REINFORCEMENT

Linear tensile stress distrib in fiber:

$$\Delta S_R = \frac{A_R}{A} \overbrace{\left(\frac{4E_R\tau'}{D}\right)^{1/2} \left\{z(\sec\theta - 1)\right\}^{1/2}}^{(\sigma_R)_{MAX}} \left[\sin\theta + \cos\theta \tan\phi\right] \qquad (1)$$

where:
- ΔS_R = shear str. increase from fiber reinforce
- E_R = Young's modulus of fiber
- τ' = skin friction resistance to pullout
- D = diam. of fibers
- A_R/A = area ratio of fibers
- z = thickness of shear zone
- θ = angle of shear distortion
- $(\sigma_R)_{MAX}$ = maximum, developed tensile stress in fiber

B. MAX. THEORETICAL SHEAR STR. INCR. FROM REINFORCEMENT

Assume <u>full</u> mobilization of tensile strength of fibers:

$$(\Delta S_R)_{MAX} = \frac{A_R}{A} \cdot T_R \left[\sin\theta + \cos\theta \tan\phi\right] \qquad (2)$$

where: T_R = tensile strength of fibers

Approximation: when $\begin{cases} 20 \le \theta \le 60 \\ 25 \le \phi \le 40 \end{cases}$ then $.8 \le [\quad] \le 1.2$

$$\therefore \boxed{(\Delta S_R)_{MAX} \approx \frac{A_R}{A} \cdot T_R} \qquad (3)$$

B. MAX THEORETICAL SHEAR STR. INCR FROM REINF. (cont)

Minimum fiber length (L_{MIN}) to prevent pullout

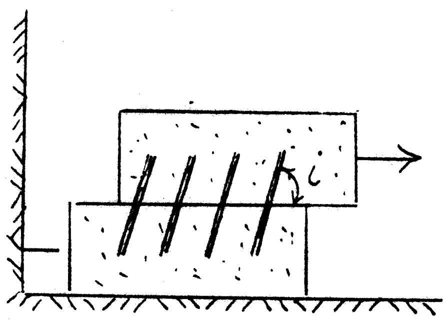
$$L_{MIN} \geq \frac{T_R \cdot D}{2\tau'} \qquad (4)$$

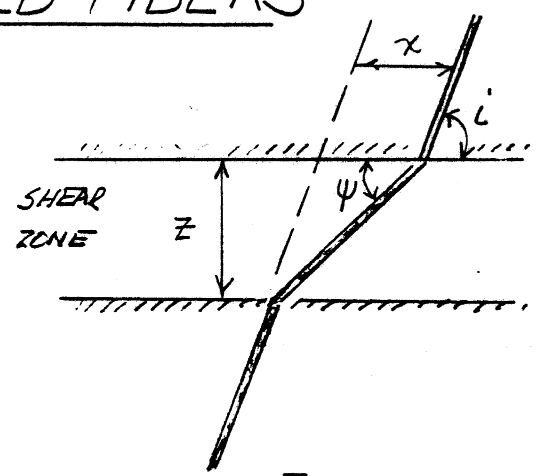
Direct shear test:

$$\boxed{L_{MIN} \geq \frac{T_R \cdot D}{2\,\sigma_z (1-\sin\phi)\tan\delta}} \qquad (5)$$

where: σ_z = vertical confining stress
δ = skin friction coef.

II. DIRECT SHEAR - INCLINED FIBERS





$$\Delta S_R = \frac{A_R}{A} \cdot \sigma_R \left[\sin(90-\psi) + \cos(90-\psi)\tan\phi \right] \qquad (6)$$

where: $\psi = \arctan\left[\dfrac{1}{k + (\tan i)^{-1}}\right]$

$k = \dfrac{x}{z}$

i = fiber orientation w/ respect to shear plane

Note:

Max. value of ΔS_R occurs at or near $i = 60°$ for most values of k & ϕ.

7. SOIL LOSS PREDICTIONS

PROBLEM NO. 6 - SOIL LOSS ESTIMATES

Assume Ann Arbor, Michigan as the locale of a construction or development site. The site is 40 acres in size, however, only 20 acres will be disturbed by grading and construction.

The following soil and topographic conditions are encountered at the site:

	disturbed area	undisturbed area
ave. slope length, ft	400	600
ave. slope gradient, %	6	10
soil type	Miami silt loam	(K = 0.37)
vegetation factor	1.0	0.12

QUESTION #1 - Compute the total, annual soil loss from the site in the absence of any soil erosion control measures. Give answer in both tons and cubic ft. of soil.

Assume the construction period lasts 12 months during which time the disturbed areas lie fallow and exposed for 3 months. Seed, fertilizer, and straw mulch are then applied. The vegetation establishment period lasts another 3 months during which the representative vegetation (or C-factor) value is 0.4. The vegetation factor for the balance of the construction period is 0.2.

In addition to seeding and mulching, a small sediment basin is also constructed at the site with a total capacity of .137 acre-ft. The relationship between trap efficiency and capacity/inflow ratio is shown in Figure 1. The basin receives sediment from only 70% of the construction area.

Storm runoff or inflow to the basin can be estimated from precipitation data for the area. Assume the worst conditions occur during the winter-spring. A typical, hourly rainfall distribution for a 5-year frequency rain is shown in the table below. The tabulated values express the rates of hourly rainfall in terms of percent of 24 hour rainfall. Figure 2 gives total 24-hour precipitation as a function of storm frequency.

Successive Time Units-hrs	Proportion of Total Precip-%
1 (max)	24
2	14.5
3	10.9
4	8
5	6
6	5
7	4

In Southeast Michigan the average value of infiltration capacity is 0.1 in/hr in winter-spring. The retention is approximately 0.10 in. (In summer the values change to 0.4 and 0.15 respectively). Assume the effective drainage area at the construction site is 50 acres.

QUESTION #2 - Estimate the total soil loss from the site with the erosion control system and the percent effectiveness of the control system (for the disturbed area only). Note: If you are unable to calculate the runoff from the precipitation data, use a rule-of-thumb estimate of 0.1 acre ft/acre of drainage area.

QUESTION #3

What will be the ratio of annual soil losses at two construction sites in Ann Arbor which have the following site and soil conditions. Assume both have the same areas.

Site No. 1

soil:
- 65% silt + very fine sand
- 5% sand (0.1 to 2.0 mm)
- 3% organic matter

site:
- completely disturbed
- 600' ave. slope length
- 10% ave. slope gradient

Site No. 2

soil:
- 40% silt + very fine sand
- 40% sand (0.1 to 2.0 mm)
- 4% organic matter

site:
- weeds & wild grass cover
- 1000' ave. slope length
- 8% ave. slope gradient

Use the Wischmeir nomograph to solve this problem.

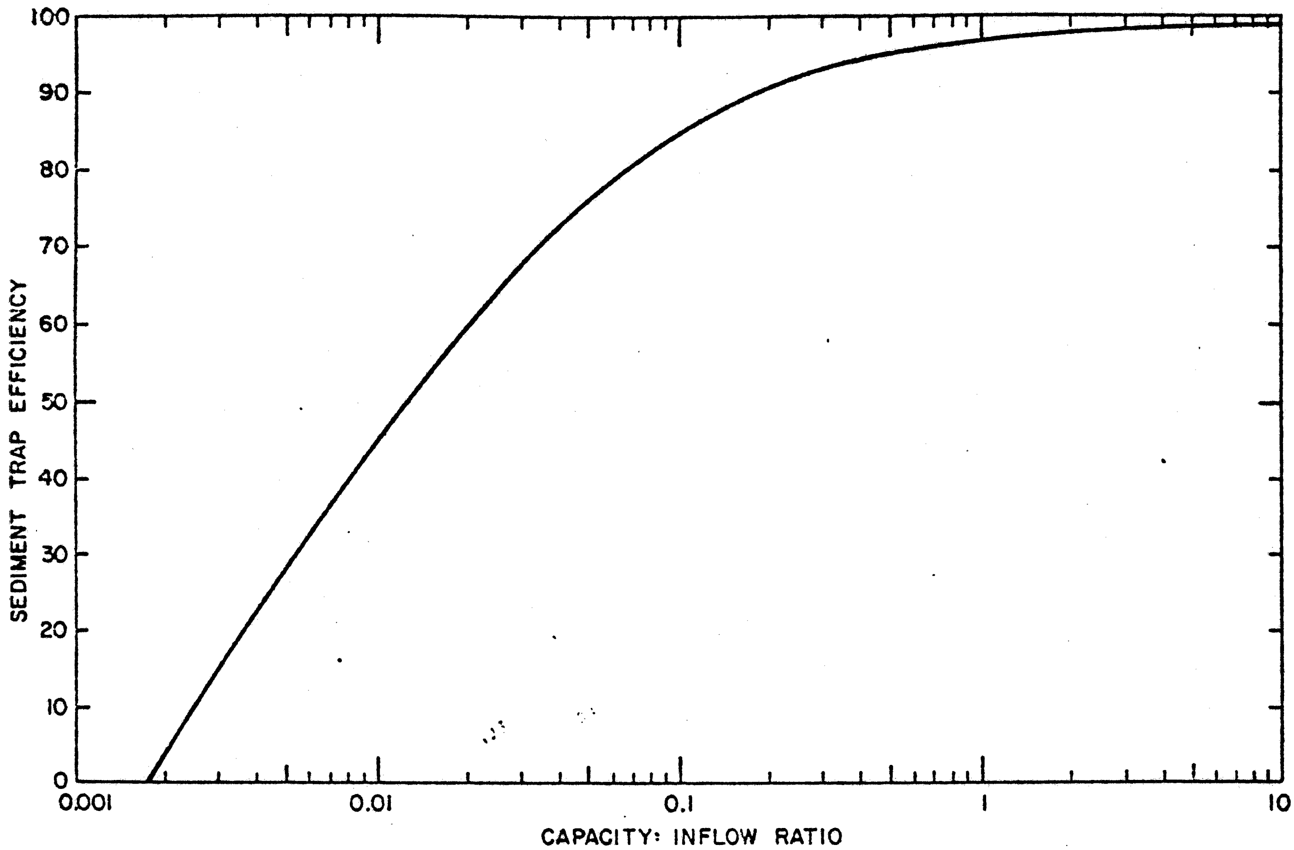


Figure 1. Relationship Between Capacity/Inflow Ratio and Sediment Trap Efficiency of Reservoirs

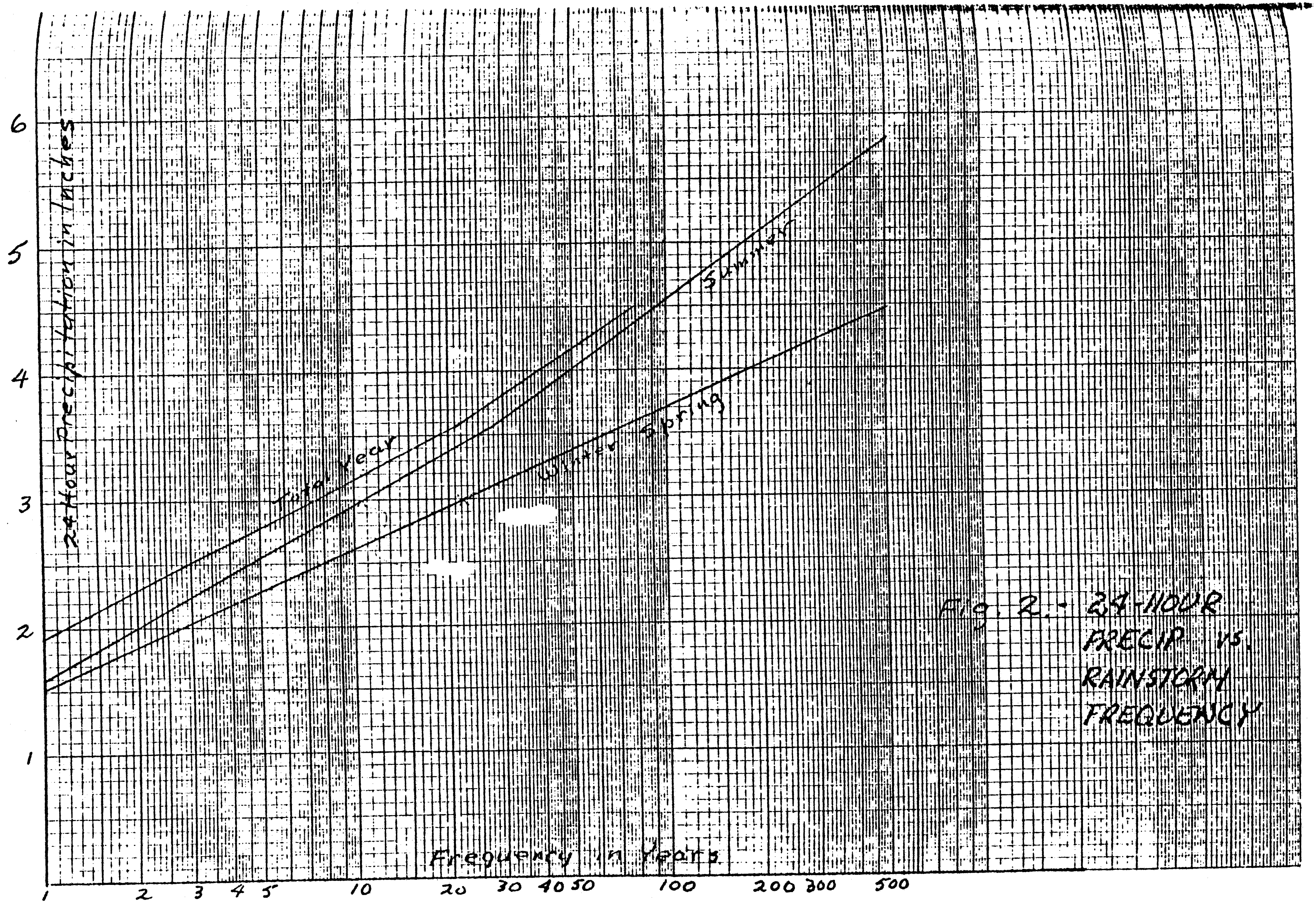


Fig. 2. - 24-HOUR PRECIP. VS. RAINSTORM FREQUENCY

Question #1

Disturbed area (20 acres)

Ave. slope length: 400 ft, -"- -"- gradient: 6% } LS = 1.3

K = 0.37

C = 1.0

R = 100

Undisturbed area 20 (acres)

Ave. slope length: 600 ft, -"- -"- gradient: 10% } LS = 3.3

K = 0.37

C = 0.12

R = 100

Computation of soil loss:

Disturbed area: $A_1 = R.C.K.LS = 100(1.0)(0.37)(1.3)(20\ \text{acres}) = 962$ Tons

Undisturbed -"- : $A_2 = R.C.K.LS = 100(0.12)(0.37)(3.3)(20\ \text{acres}) = 293$ Tons

Total Annual loss: $962 + 293 = 1255.0$ Tons or:

$$1255(0.87) = 1091.85\ \text{Cu. Yds} = 3^3(1091.85)\ \text{cu.ft.} =$$
$$= 29{,}479.95\ \text{cu.ft}$$

Finally: Annual Soil Loss = 1255.0 Tons or 29,480.0 cu. ft.

Question #2

The difference now is, that we will have to compute a weighted value of C-factor, for the disturbed area, and that we will have to compute the value of P-factor (due to the existance of sediment basin).

The inflow due to the drainage area will be:

Inflow = (effective drainage area)(rainfall excess depth)

Inflow = 50 acres (0.865 in) = 43.25 acre-in = 3.6 acre-ft ✓

The capacity/inflow ratio will be: $\frac{0.137}{3.6} = 0.038$

From Fig. 1, the sediment trap efficiency of the sediment basin will be: 70%, for ratio = 0.038.

The sediment basin receives sediment from 70% of the construction area. So the factor P for the basin will be:

$P = 1.0 - 0.7 \times 0.7 = 0.51$ ✓

Computation of the effectiveness of the control system

% Effectiveness = $(1 - CP) \times 100$ Where: $C = 0.45$

$P = 0.51$ ✓

Effectiveness = $(1 - 0.45 \times 0.51) \times 100 = 77\%$ (for disturbed area)

The soil loss from the disturbed area will be:

$A_1 = R \cdot LS \cdot K \cdot C \cdot P \cdot (\text{Area}) = 100(1.3)(0.37)(0.45)(0.51)(20) = 220.8$ Tons

The soil loss from the undisturbed area will remain the same as in question # 1. $A_2 = 293$ Tons.

Total soil loss: $A_1 + A_2 = 220.8 + 293 = 513.8$ Tons or

$513.8(0.87) = 447.0$ cu. yds = 12,069.2 cu. ft ✓

Computation of C-value

3 months : C = 1.0

3 —"— : C = 0.4

6 —"— : C = 0.2

12 months

The weighted average for 12 months period will be:

$$\underline{C} = 1.0\frac{3}{12} + 0.4\frac{3}{12} + 0.2\frac{6}{12} = 0.25 + 0.1 + 0.1 = \underline{0.45}$$

Computation of P-value

Find the inflow from the drainage area:

From Fig. 2 the total (24-hour) precipitation for a rainfall with 5-year frequency during the Winter-Spring, is: 2.3 inches. Now we can lay out the following Table:

Time Units hrs	Proportion of 24-hour prec., %	× Rate of Rainfall iph	− Infiltration rate, f_a, iph	= Depth of rainf. f_e, in
1	24	0.55	0.1	0.45
2	14.5	0.33	0.1	0.23
3	10.9	0.25	0.1	0.15
4	8	0.18	0.1	0.08
5	6	0.14	0.1	0.04
6	5	0.115	0.1	0.015
7	4	0.09	0.1	—
			Total =	0.965 in

The retention is: 0.1 in, so the final depth of raintall will be: 0.965 − 0.1 = <u>0.865 in</u>

ESTIMATING

Sample Calculations

SHEET-RILL EROSION AND SEDIMENT YIELD ON DISTURBED WESTERN FOREST AND WOODLANDS

Display 3.3-1
Transect
Highlead Logging Data

Location and Size

Acres	80
County	Yamhill
Lat. N	45°00'
Long. W	123°30'

Average Buffer Data

Percent Slope to Channel	42
Slope length to channel	200
Future Slope length/channel	220

Average C Factor Elements

	Present	Future
Effective Canopy Cover Percent	60	60
Effective Canopy Height feet	4	4
Root Network Percent	70	91

Average Transect Data

Transect Elements	Present				Future			
	Log paths	Road Land	Fire Trail	Un-dist.	Log paths	Road Land	Fire Trail	Un-dist.
% Bare Ground	5.0	3.0	5.3	-	3.0	2.5	3.5	-
% Protected Ground	3.7	2.0	3.0	78.0	3.7	2.0	3.0	81.3
% Slope	34	5	30	-	34	5	30	-
Slope Length	133	40	55	-	107	30	20	-
% Area	8.7	5.0	8.3	78.0	6.7	5.5	6.5	81.3

2.2-1 Present Conditions

A highlead logging operation was sampled and the data recorded (display 3.3-1)

Recorded elements are totaled and averaged. Slope and length of water travel are properly weighted to reflect the area in various disturbances. The buffer data pertains to the distance and slope between a given disturbance and the

from USDA SCS, West Tech Ctr, Portland

functioning channel. These elements are used to develop a simple sediment delivery ratio. The cover factors are averaged and recorded.

In the example used for this discussion 22 percent of the transect paces fell on log paths, roads, or fire trails. All other hits were on forest duff. Elements are recorded in a simple way. Display 3.3-1.

Percent Slope and Length Feet

1. The percent area based on transect hits for each disturbance class is recorded in display 3.3-1. For log paths this is 5 + 3.7 = 8.7 percent.

2. Bare ground in this example is 5 + 3.0 + 5.3 = 13.3 or 13 percent.

3. Weighted averages of percent slope and lengths are developed for each disturbance class.

   Weighted % Slope = {(34)(8.7)+(5)(5.0)+(30)(8.3)}/22 = 26 percent

   Weighted Slope Length feet = {(133)(8.7)+(40)(5.0)+(55)(8.3)}/22 = 82 ft.

LS Factor - The slope length factor is provided in table format, tables 3.2-4, and 3.2-5, or figures 3.1-6 & 3.1-7.

For the example, display 3.3-1, the present slope and length were calculated to be 26 and 82 respectively. Using figure 3.1-5 the area for the sample is located in the Xeric moisture regime and Mesic and Thermic temperature zone requiring the use of table 3.2-5 for the estimate of LS. The LS factor is computed to be 4.1. Enter the LS into the erosion computation (see pages 8 and 9.

Cover Factor (C)

Type I effect - figure 3.1-1 percent protected ground for the whole operation is 100 less the percent bare ground. Enter bottom of figure with 100-13 = 87 and read 0.08.

Type II effect - figure 3.1-2. The Type II effect is the produce of rainfall energy intercepted by the canopy (REc) and the decimal percent bare ground minus 1.

{Type II = 1-(REc)(% Br. Gr.)}

1/ Percent canopy cover is 60 percent. Enter figure at bottom with 60 and read from 1 meter line 42 percent on right side of figure. Enter 0.42, the gross reduction in energy by the canopy (REc), in equation.

2/ Percent bare ground is thirteen hundredths and entered into the equation.

$$\text{Type II} = \{1-(0.42)(0.13)\} = 0.95$$

Type III effect - figure 3.1-3. Since this is a forest floor the forest duff curve is used. The root network was estimated in 70 percent. Enter with 70 at bottom of figure and read 0.13. Enter 0.13 into the equation.

The C factor is the product of Types I, II, and III effects. (See graphic description display 3.3-4.) C = (0.08(0.95)(0.13) = 0.01.

Additional C factors are available for consideration in table 3.2-1.

K Factor - The erodibility of the soil is taken from soil survey information, developed by the soil scientist on the project, or estimated using table 3.2-2. This table requires the use of a wetting bottle and the procedure outlined on display 3.3-2. For the example, use a K of 0.32.

R Factor - Where no R value charts are available, use the procedures outlined in Technical Note, Conservation Agronomy No. 32, dated September 1974 (Rev. March 1975), to compute the total R factor (R_T). For our example the R factor was taken from a map provided by the Oregon State Conservationist, Soil Conservation Service, figure 3.1-4. The R of 47 was entered into the Erosion Computation, page 14. It should be noted that if snowmelt adds to the water available for erosive activity, a snowmelt R_S must be added to the R to form a total R_T.

For forested lands there are few areas where R_S is a problem. These are isolated on those sites where concrete frost develops or other factors prevent the melt waters from percolating into the soil. Where this does occur it is suggested that the R_T be weighted for the site in proportion to its occurrence, or erosion and sediment calculations be kept separate.

The procedure for developing EI factors for individual storms is presented in the Appendix. A sample of the procedure is shown in display 3.3-3.

Erosion Estimate

Rate Per Acre

$$A = R_T KLSC$$

$$= (47)(0.32)(4.1)(0.01)$$

$$= 0.62 \text{ tons/acre/year}$$

For the 80-Acre Highlead Operation

$$A = (0.62)(80)$$

$$= 49.6 \text{ tons/year}$$

Sediment Yield Estimate

Where:

L = Slope length of buffer strip to channel.

S = percent slope of the buffer strip.

$$\text{Sediment delivery ratio} = 1 - \{L/[50+(4)(\%\ S)]\}$$

$$= 1 - \{200/[50+(4)(42)]\}$$

$$= 0.08$$

$$\text{Estimated Sediment Yield} = (0.62)(0.08)$$

$$= 0.05 \text{ tons/acre/year}$$

Sediment Yield for Highlead Operation

$$= (0.05)(80)$$

$$= 4.0 \text{ tons/year}$$

The sediment yield estimated by this procedure takes the material only as far as a functioning channel. Routing sediment through the stream system is not considered. It should also be noted that permanent road, slide, gully, streambank, and channel sediments must be added where necessary to complete the sediment picture.

2.2-2 Future Conditions

Seven elements within the computation process allow for anticipated change. These are in order of their appearance in the USLE: K, L, S, C. The three remaining are the total acres of disturbance type, percent bare ground within the disturbed area, and the changes expected in the buffer widths.

The following elements in the example are adjusted for the new estimate:

Bareground: reduced from 13 to 9 percent by improved management.

Rootnet: increased from 70 to 91 percent by new plant growth.

Slope Length: all changes are due to improved management.

Logpaths from 133 to 107 feet - a better highlead setting

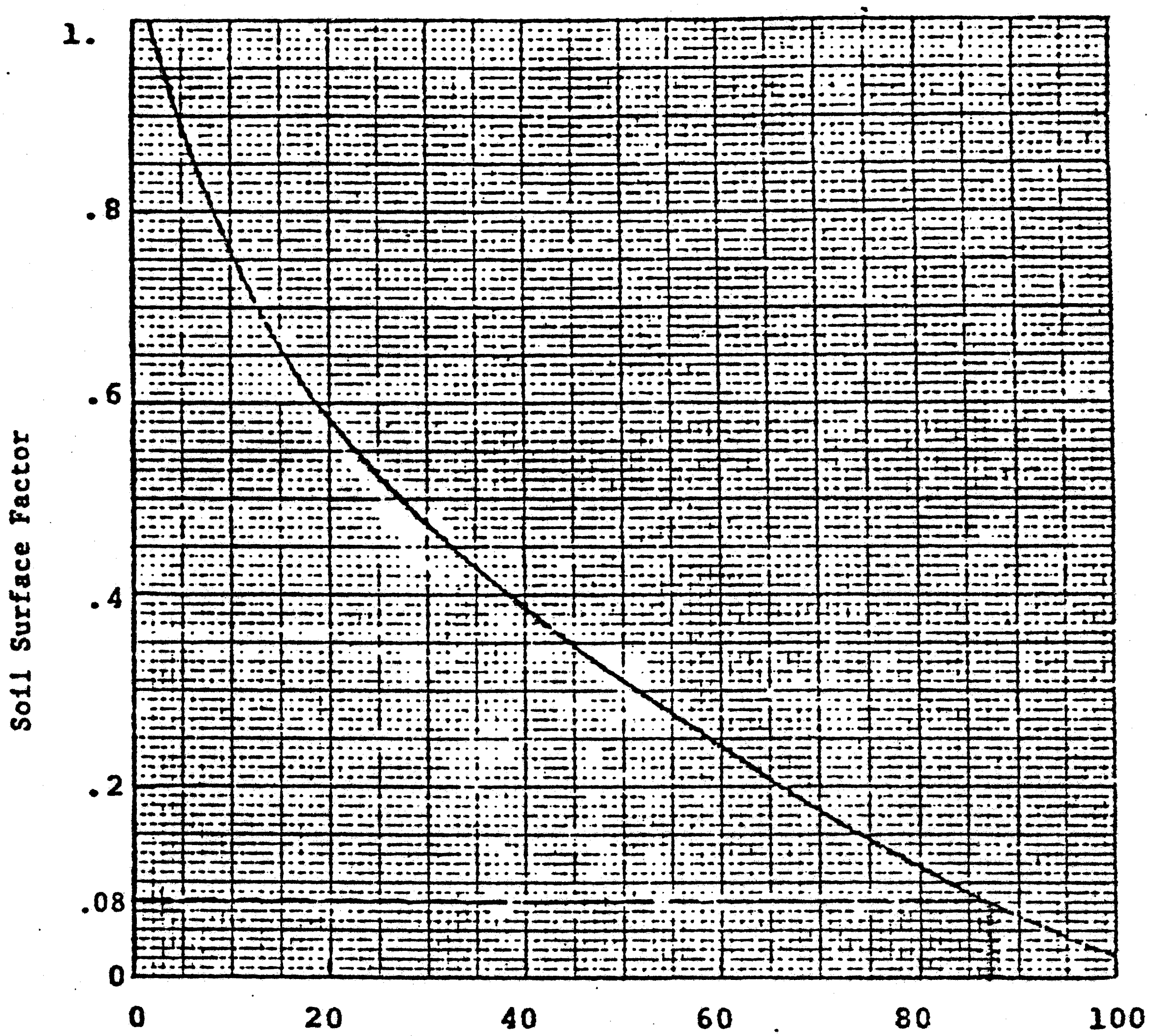
Road and Landings from 40 to 30 feet - increased drainage

Fire trails from 55 to 20 feet - by increasing waterbars

Percent Slope: No change anticipated.

Buffer: a 220-foot wide strip is set to assure there would be little or no sediment yield.

Introducing these changes then will give a new estimate of the erosion and sediment yield for this example as follows:



Protected Soil Percent Humus and Litter

Figure 3.1-1

Cover in Direct Contact with the Soil Surface, Type I Effect

W.H. Wischmeier. Approximating the Erosion Equation's Factor C for Undisturbed Land Areas, USDA, Agricultural Research Service, Unpublished Report

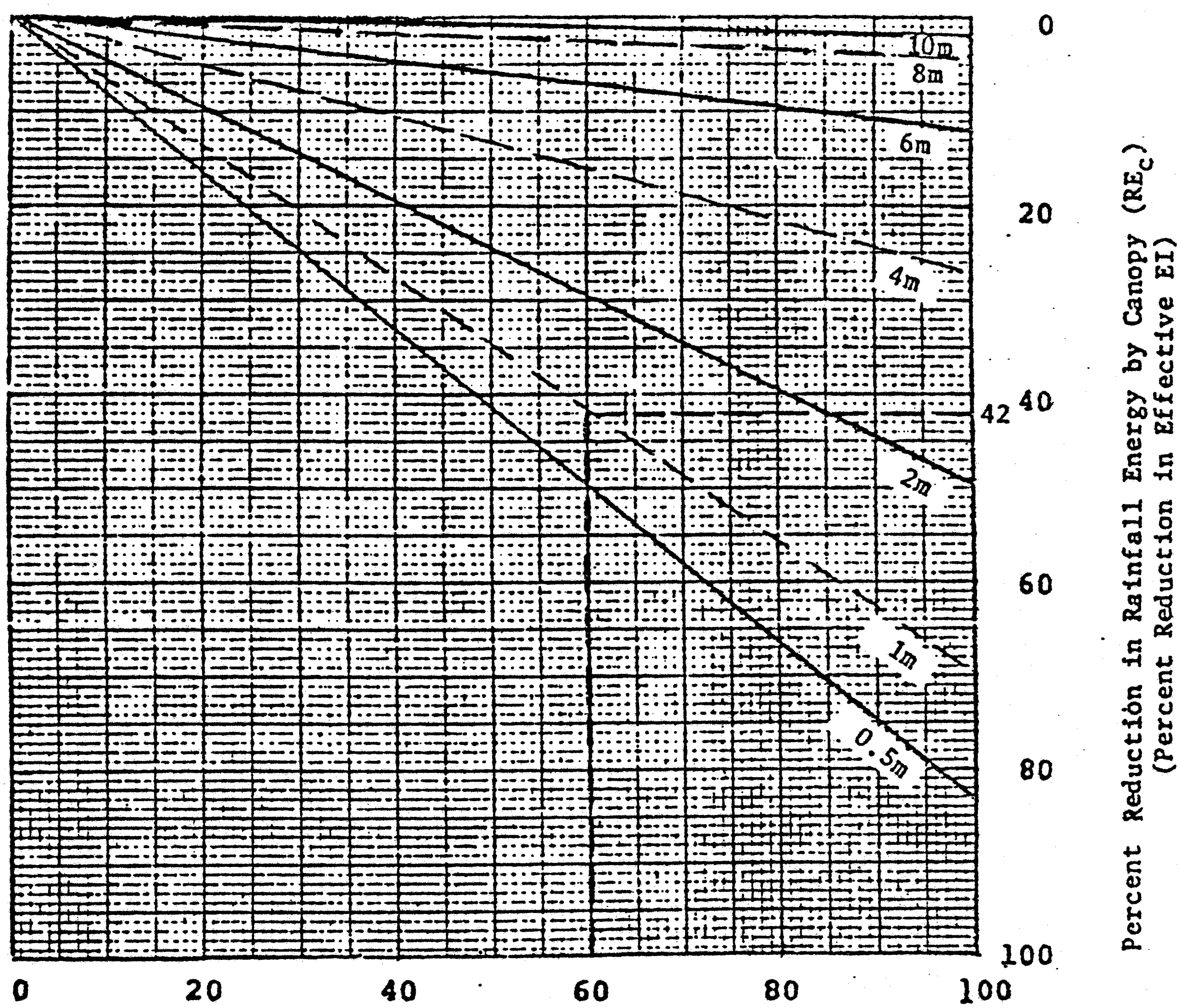


Figure 3.1-2

Reduction in Rainfall Energy (RE_c) by Effective Canopy Cover Above the Soil Surface

Adapted from W. H. Wischmeier, Approximating the Erosion Equation's Factor C for Undisturbed Land Areas, USDA Agricultural Research Service, Unpublished Report

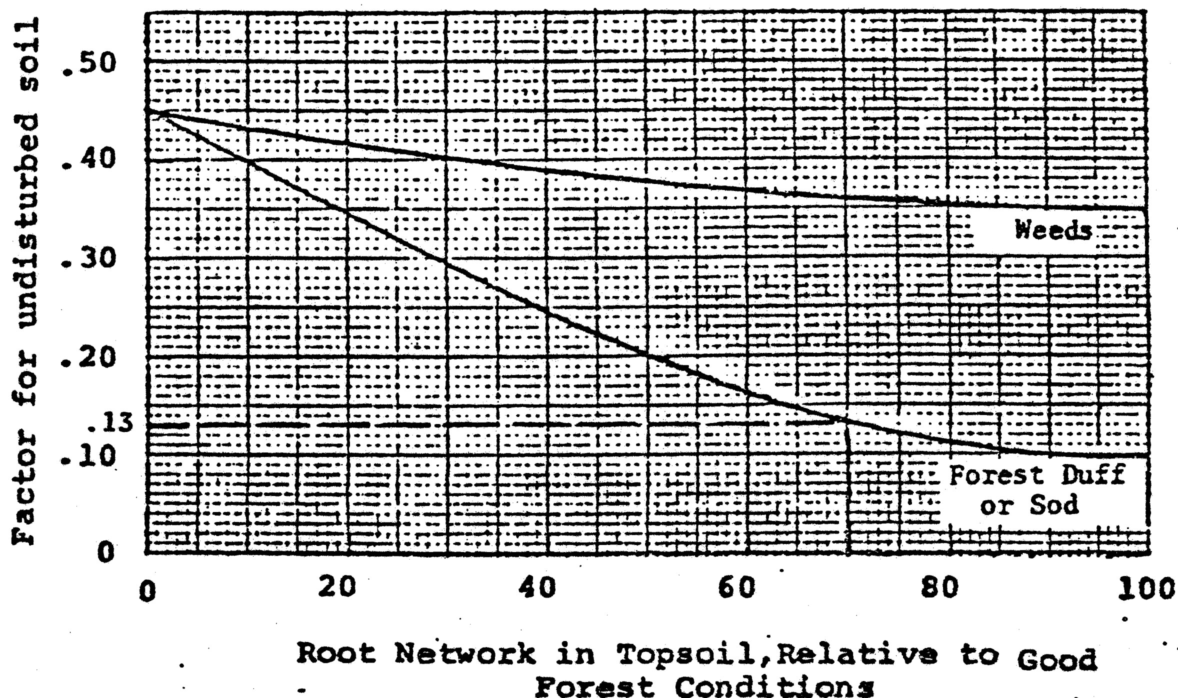


Figure 3.1-3

Effect of No Soil Disturbance for at Least 10 Years, Root Accumulation in the Upper Layer of Soil, and other Related Factors, Type III Effect

W.H. Wischmeier, Approximating the Erosion Equation's Factor C for Undisturbed Land Areas, USDA Agricultural Research Service, Unpublished Report

DISPLAY 3.3-4

EVALUATION OF C

TYPE II = [1 - (RE$_C$)(BR. GRD.)]

TOTAL RAINFALL ENERGY

60% CANOPY
(FROM FIG. 3.1-2 42 REDUCTION)

.13 BR. G. | .87 DUFF

[1-(.42)(.13)] = .95

TYPE I		TYPE III
87% PROT. GR.		70% FOREST ROOT
.08	[1-(.42)(.13)] = .95	.13

C = (.08)(.95)(.13) = .01

NOTE: (.42)(.13) = .05 ENERGY REDUCTION

: ENERGY = 1-(.05) = .95

Table 3.2-2

GUIDE FOR ESTIMATING ERODIBILITY (K) VALUES

Soil Surface Texture 1/	Permeability			
	Very Slow	Slow	Mod. Slow, Moderate	Mod. Rapid, Rapi Very Rapid
c, sic, sc 2/	0.37	0.32	0.28	0.24
scl, sicl, cl	0.43	0.37	0.32	0.28
sil, l, vfsl	0.49	0.43	0.37	0.32
fsl, sl	0.37	0.32	0.24	0.20
ls, s, lcs, cls	0.28	0.24	0.20	0.17 or .15

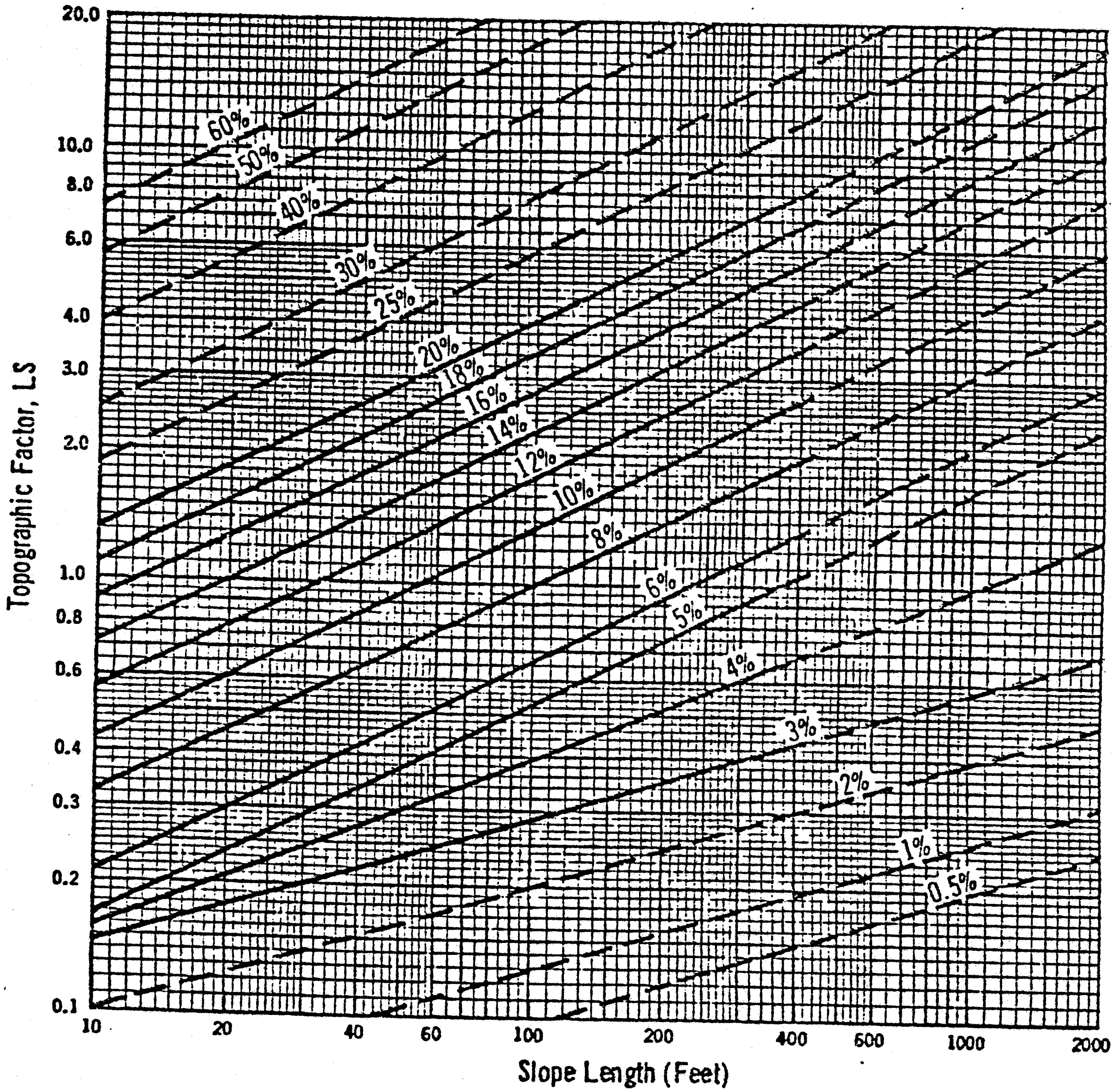
1/ Gravelly, channery, shaly, slaty, cherty, cobbly, or flaggy phases of these textures are normally reduced one or two classes in K value.

2/ C, Clay; Si, Silt; S, Sand; l, loam; vf, very fine; f, fine

REFERENCE: Soil Conservation Service, 1969, Hydrologic Group K and T Factors of Series Having Type Locations in the South Region: South Regional Technical Service Center, Fort Worth, Texas

Figure 3.1-6 - Applicable to all Soil Moisture - Soil Temperature Regimes except A=3, and A-1 in WN, OR, and ID.

SLOPE-EFFECT CHART (Topographic Factor, LS)*



*The dashed lines represent estimates for slope dimensions beyond the range of lengths and steepnesses for which data are available. The curves were derived by the formula:

$$LS = \left(\frac{\lambda}{72.6}\right)^m \left(\frac{430x^2 + 30x + 0.43}{6.57415}\right)$$

where λ = field slope length in feet and m = 0.5 if s = 5% or greater, 0.4 if s = 4%, and 0.3 if s = 3% or less; and x = sin θ. θ is the angle of slope in degrees.

Figure 3.1-7

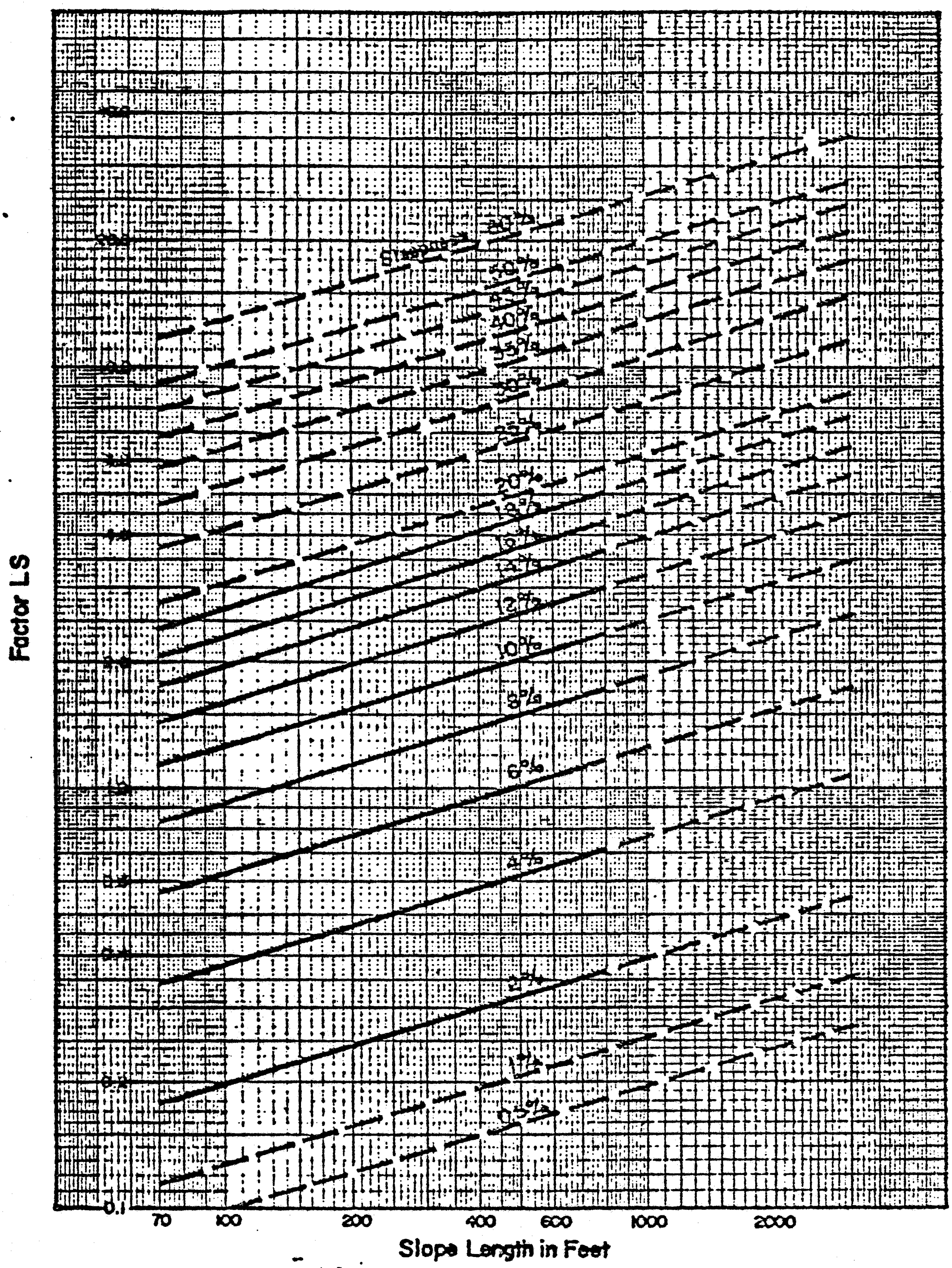


Figure 3.1-7 Applicable to Soil Moisture-Soil Temperature Regimes A-1 in WN, OR, and ID; and in A-3

Note: Dashed lines are extensions of LS Formulae beyond values tested in studies.

For slopes less than 9% $LS = \left(\frac{\ell}{72.6}\right)^{0.3} \left(\frac{0.43 + 0.30 + 0.043s^2}{6.613}\right)$

For slopes greater than 9% $LS = \left(\frac{\ell}{72.6}\right)^{0.3} \left(\frac{s}{9}\right)^{1.3}$

ℓ=length of slope
s=percent slope

OBTAINING PLANTS AND HANDLING OF PLANT MATERIALS

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INTRODUCTION

During the planning step for a revegetation project the choice of the plant species will have been made on the basis of the information obtained in the site analysis and on the various limiting components of the total project: biological, environmental, physical and economic.

The procurement and handling of plant materials is of paramount importance to a successful revegetation project. Plants are living things and must be grown and handled properly if success is to be obtained. Many failures are due to the poor quality of planting material and to improper handling during shipping, storage or holding period and on the planting site.

The procurement of the proper plants, properly grown, of the proper size, condition (e.g. acclimatized or "hardened off"), of good vigor, in sufficient quantity, and at the proper time for the planting project is one of the most difficult aspects of the revegetation project. There are a number of reasons this is true. Many of the plant species desired may not be available from commercial or governmental nurseries. Available plants may not have been propagated from seeds or vegetative materials collected from climatic areas similar to that of the project. This is especially true of those species which grow over wide geographic areas. Of the total plant spectrum grown in nurseries, relatively few species are grown in deep tubes which are often most suitable for plantings under difficult site conditions. Native plants, especially shrubby species, are often impossible to obtain because the commercial nurseries grow primarily for ornamental use - the urban market. Nurserymen have had relatively little experience growing many of the native plants because of lack of demand. The quantities of native and introduced plants needed may be insufficient even if the species are grown. Interstate plant quarantines may restrict shipment from nurseries where the plants are available, _e.g._ oak species may not be moved from the midwest to California.

The procurement process, therefore, often must be started at an early stage in the project. Seed or cuttings may have to be obtained. Contract growing often must be arranged in advance and the soil mix, container size, and plant size and quality must be specified. Such contracts may have to be made as long as 18 to 24 months before planned planting dates.

Some explanations of these problems and suggestions for solving them will be suggested in this paper.

PROCUREMENT AND PLANT SELECTION

Importance of Ecotypes or Provenances

The existence of ecotypes or provenance (source) variation for many species is not known. Most studies of plants with a wide geographical range (altitudinal, latitudinal, climatic) have shown wide differences in response among plants collected throughout the range to a variety of environmental factors: heat, cold, drought, soils, and flooding tolerance.

Cornus stolonifera, red-osier or American dogwood is a riparian species ranging in the West from Alaska to California to Newfoundland south to Virginia, Kentucky and Nebraska in the East and Mid-west. Extensive studies on this species indicate large differences in hardiness. Some of the differences are due to latitudinal distribution. Some are due to climatic differences as in the case of collections from Western Washington and Minnesota which are about the same latitude. In the latter case, absolute hardiness in mid-winter was similar but the Washington collection did not attain this degree of hardiness until much later in the season.

Acer rubrum, red or swamp maple, another riparian species, is found from Newfoundland south to Florida and west to Minnesota to Texas. Collections from many parts of its range were grown in tests conducted simultaneously in several states with a wide range in climate. Large differences were shown in growth rate, fall color and hardiness. Differences in hardiness were sometimes due to plants growing too late in the fall or breaking dormancy too early in the spring.

Acer macrophyllum is a West Coast, sometimes riparian species. Collections from a wide range of California habitats, when grown at Davis in the Sacramento Valley, exhibited large differences in growth rates and tolerances to summer heat and winds. The best selection for Davis conditions were from a dry, Southern California location at about a 5,000 foot elevation.

Fraxinus pennsylvanica is considered by older botanists to consist of two varieties, *F. pennsylvanica*, red ash, on upland sites and *F. pennsylvanica* var. *lanceolata*, green ash. The total species ranges from Cape Breton Island and Nova Scotia west to Alberta and Montana and south to Central Texas and Northern Florida. Provenance studies show pronounced ecotypic differences in moisture and low temperature tolerances. The varietas, *F. pennsylvanica* var. *lanceolata* has glabrous rather than pubescent petioles and seems to inhabit more riparian habitats. This varietas or type appears to have a wider tolerance to wet soils. More studies are warranted of possible ecotypes of this flood-tolerant species.

Studies on *Euculptus cumaldulensis*, river red-gum in Israel and elsewhere have shown large differences in tolerance to soils and available water among ecotypes. This species is flood tolerant but our studies were not of sufficient scope to permit testing of ecotypes for flood tolerance.

Similar studies have not been made as far as I know on *Quercus lobata*, valley oak. This species has a wide geographic range from Northern to Southern California and sites range from dry foothills to riparian where annual flooding occurs in winter and spring. It is very probable that ecotypic variations occur within this species. Many of our losses of the species when urbanization occurs seem to be due to lack or water rather than too much water.

The best rule to follow with native plants is, when information on ecotypes is not known, use seeds of other propagation material from as near site conditions as possible. When studies have shown ecotypic differences, use seed sources shown to most closely fit site conditions. This may require the contractor to produce proof of origin of the propagation materials.

Timing: Timing of plant production and of planting season is critical. For most sites there will be a time or times when plantings will be most successful. Drought, cold (including time of snow melt), season of flooding, annual seasonal variations and the choice of deciduous and bare-root plants vs. containerized plants all affect the choice of planting time, sometimes called the "planting window". This "planting window" is the time or times when conditions are best suited for plant establishment. This timing affects both the growing schedule and the planting contract specifications. Timing should be such that plants are of optimum size and top: root ratio, and condition for planting during the "planting window". The decision must be made on a site-to-site basis.

Advance Planning: A large advanced lead time is required for species not readily available. Seed of some species are available from seed dealers but others may have to be collected. Many species do not set a reliable seed crop every year. This is an important factor in plant selection as well as in obtaining seed from the most desirable ecotypes.

Fall Versus Spring Planting: Fall plantings may be preferred in areas with late growing seasons, winter rains and summer drought. This allows a longer period of establishment before late spring when flooding or drought occur. However, bare-root plants may not be available until late fall or even mid winter. Late fall plantings may not be desirable where late fall droughts occur, or where frost heaving is severe before new root growth occurs.

Spring planting dates are usually required for bare-root stock, where sites are subject to late fall and winter frost heaving problems, or where flooding occurs in late fall to early spring. Spring planting should be scheduled as early as site conditions permit. Summer plantings should be avoided unless adequate rainfall or supplemental irrigation is assured.

Types of Plant Materials

Direct "sticking" of unrooted cuttings of easy-to-root native or introduced species is often successful and is one of the most economical methods of plant establishment.

Direct seeding of woody species may be successful if proper care is used in selection of species, preparing planting "spots", and planting. Direct seeding is more economical than transplanting. (Chan, *et al*).

Bare root transplants are successful for many species. The "planting window" is more restricted and survival may be lower than for transplanting container-grown stock.

"Tublings", plants grown in relatively small and deep containers, have proven very useful on difficult sites. Root-top ratios are favorable when properly grown. Roots are deep to allow maximum use of limited water supplies and root disturbance at planting time is minimal. Tree species used in reforestation and some *Eucaluptus* may be readily available but shrub species are frequently not available unless grown under contract.

Plants in gallon or larger cans are often available for species in regular commercial production but are limited in variety of species best suited for revegetation projects. Plants in larger containers increase the cost for purchase and planting substantially. Survival is frequently reduced because of limited root systems in relation to size of the tops of the plants.

Growing Quality Tubling or Other Container Plants

Roots of container-grown plants should be well developed, adequately filling the soil mass so that it holds together when removed from the container but not so overgrown as to be "pot-bound".

A common fault of container-grown stock is the presence of kinked or girdling roots. These poor root systems result from poor transplanting of seedlings or rooted cuttings and from failure to remove circling roots when shifting plants to larger containers. Such roots have been shown to reduce growth and sometimes ultimate survival because of girdling of the crown or loss due to wind throw., (Harris, et al).

Time does not permit a detailed description of the topic. It is covered quite thoroughly in the references listed for Harris, et al Baker et al. and Tinus. Many designs and sizes of growing tubes are available. Many are designed to minimize or eliminate kinked and girdling roots.

Soils or growing media must be well-drained because, in containers a perched water table exists after irrigation. Plants should be of good vigor and nutrient status.

Hardening-off and Holding Plants

Plants should be adequately "hardened-off" and "acclimatized". This is particularly critical when the environment of the growing nursery is different from the planting site. This can often be done by moving the plants near the site as long as possible before planting time, or, with bare-root materials, holding under refrigeration until planting time when the planting season starts late in the spring at the site than in the nursery.

During the holding period and when moved on site, plants must be carefully watered and refertilized if necessary. Plants must be thoroughly watered immediately prior to planting. On hot sites plants may need to be partially shaded to prevent overheating of root systems and should be removed from such protection only as the work progresses and planted immediately. Lethal temperatures can occur in dark colored containers in a few hours under hot, sunny conditions.

Planting

Plants should be removed from containers at planting time unless the containers are biodegradeable. Biodegradeable containers should be "shouldered off" to prevent drying of the root system through "wicking" action. These should also be removed if roots have not penetrated the container sufficiently to have intimate contact with the site soils.

Circling roots on the outside of the root ball must be removed at planting time.

Plants should be planted promptly as holes are dug to minimize drying of the hold and backfill soil. The backfill should be thoroughly tamped to obtain intimate contact with the plant roots.

Handling Live Brush and Cuttings

Live brush and cuttings have been severed from their root systems. Careful handling to prevent drying is essential. They may be stored in the adjacent lake or stream or thoroughly shaded and kept moist if not stored in water. No more materials should be cut than can be planted within one or two days. They should be exposed during the planting process as short a time as possible. Like container plants they should be moved on-site only as work progresses if hot or dry conditions exist.

SUMMARY

Plants and plant materials are living organisms and must be treated as such. They may need to be fed (fertilized) and must never suffer drought or other undue stress. The success or failure of any revegetation project, no matter how well planned, depends on the proper selection, production, care, and handling of the plant materials at each step of the project.

LITERATURE CITED

Baker, Kenneth F., Ed. 1957. The U. C. system for producing healthy Container grown plants. Division of Agricultural Sciences, University of California, Berkeley, Calif. 23:331 pp.

Chan, F. J., Harris, R. W., and Leiser, A. T. 1971. Direct seeding woody plants in the landscape. AXT-n27, Agricultural Extension Service, University of California, Berkeley, Calif., 12 pp. (Reprinted as Leaflet 2577, 1979.)

Harris, R. W., Davis, W. B., Stice, N. W., and Long, D. 1971. Root pruning improves nursery tree quality. J. Amer. Soc. Hort. Sci. 96(1):105-118.

Tinus, R. W., and McDonald, S. E. 1979. How to grow tree seedlings in containers in greenhouses. Gen. Tech. Rpt. RM-80, Rocky Mountain Forestry and Range Experiment Station, USDA Forest Service, Fort Collins, Colo.

Landslide Analysis Concepts for Management of Forest Lands on Residual and Colluvial Soils

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A forest land management analysis scheme is discussed for dealing with landslides that occur in residual and colluvial soils. No one geotechnical or statistical model can be expected to apply to all levels of land management where an assessment of the potential for landslide is vital to a rational decision-making process. The U.S. Department of Agriculture Forest Service in cooperation with the University of Idaho is developing a scheme for evaluating soil-mantle landslide potential to provide information at three levels of land management activities: (a) resource planning; i.e., relative landslide hazard evaluation for resource allocation; (b) project planning; i.e., evaluation of management impacts for comparing alternate transportation routes and timber harvest techniques; and (c) road design and landslide stabilization; i.e., evaluation of alternate road stabilization techniques at a specific critical site. Both geotechnical and statistical analysis techniques are advocated so that the information can be in geotechnical form (factor of safety against failure or critical height of slope) or in statistical form (probability of landslide occurrence) with landslide inventories used as a link between the two. A hypothetical example of the three-level analysis is given.

Many forest lands in the West, particularly those on residual and colluvial soils, are classified as unstable and have a high potential for mass failure. Timber-harvesting operations, road construction, and other resource-management activities in these areas can accelerate mass erosion and cause significant degradation of water quality unless carefully planned and executed. Successful management of these lands requires development of a specialized body of knowledge to quantify and integrate those site factors that influence slope stability. Site factors that require special attention are slope, soil depth, soil shear strength, seasonal ground water levels, and the strength derived from vegetation (effective root strength). Geotechnical characterization of these site factors can then be the basis for a landslide hazard analysis tailored to a specific management decision level.

MANAGEMENT COMPLEXITY

The management of lands that have a high potential for landslide is inherently complex, not only because of the nature of the interacting natural processes and management activities but also because of the number of persons of varied disciplines who must possess a degree of understanding of the slope failure processes and be able to contribute to the total stabilization effort. Considerable overlap and interaction between members of key disciplines must be coordinated.

Members of different disciplines must deal with problems of slope stability at several levels of intensity. For example, the resource planner must recognize high-hazard areas, but only on a general scale. The road locator needs to recognize potentially unstable areas along proposed routes and to avoid the problem through adjustment in alignment. The engineer must be able to use soil mechanics in the stability analysis of remedial measures before, during, and after construction to prevent or correct specific road cut or fill slope failures.

FAILURE MODE

Consistent with Varnes (1), landslides may be grouped into two broad categories, depending on the type of slide mass material--either soil (debris or earth) or bedrock. This grouping enables orderly selection of stability analysis techniques and the data required. The concept should apply to soil or bedrock landslides with the proper selection of slope analysis techniques and required data. However, this discussion is directed at landslides where the failure is confined to a soil mantle primarily of colluvial or residual origin.

The usual setting for this type of failure is a relatively loose, cohesionless soil mantle that overlies a less permeable bedrock or denser soil mass. An exception to this is an extremely altered bedrock or residual soil near the surface that overlies a less altered bedrock at some depth. Each of these conditions can result in similar failures and can be analyzed in the same manner. The contact with the underlying, less permeable, material forms a drainage barrier for the normal downward migration of ground water that originates from rainfall, snowmelt, or both. Ground water is concentrated at the drainage barrier and, if sufficient quantities are available, the soil mantle develops within it a perched water table with seepage moving along the barrier. The drainage barrier, phreatic surface (water table), and ground surface are often parallel or nearly so. Seepage of this form is usually considered to be of the infinite slope form because of this parallelism.

Failure of the entire soil mantle can occur naturally due to higher-than-normal ground water concentrations that result from unusually high rainfall or snowmelt. Failure also may result from wildfire, which destroys vegetation and thus the beneficial effects of evapotranspiration and root strength. Failure more often occurs through land management activities such as timber harvest and road construction, which in some manner increase ground water concentration, destroy root strength, or affect the natural parallelism of the ground surface or phreatic surface in relation to the drainage barrier.

Failures are often confined to the soil mantle because the underlying material usually has a higher strength and the critical failure surface is usually at the maximum depth of the soil and water table (tangent to the contact with the drainage barrier). The failure surface may be circular arc or translational in shape, depending on local conditions. Translational failures may begin as a small circular arc and progress into a translational shape or a series of circular arc failures as more of the soil mantle is mobilized.

IDEALIZED LANDSLIDE EVALUATION SYSTEM

A complete system of landslide hazard evaluation is needed that begins early in the resource planning phase, follows through into project development, and provides information back to the planning phase to improve future hazard analyses. The system should be structured on a common scheme but branch early into either soil-mantle landslide analyses or bedrock landslide analyses and use the respective analysis techniques and data. In either case, the complete system should be structured on a common basic analysis form that is simplistic in the resource planning phase and requires primarily available resource inventory data and becomes more complex and

Table 1. Idealized analysis system.

Item	Level 1, Resource Allocation	Level 2, Project Planning	Level 3, Critical Site
Base map	Landslide hazard map on resource inventory scale; 1:24,000; 1 in. = 2,000 ft	Project map of larger scale; 1 in. = 500 ft	Critical site map on even larger scale: 1 in. = 20 ft to 1 in. = 100 ft
Stability analysis	Infinite slope equation requires values for geotechnical variables and their inherent variance	Combination of infinite slope analysis from level 1 but used to model effects of tree removal and critical height analysis of anticipated road cut and fill slopes	Critical failure path analysis by computer program with search routine for circular arc, translation failures, or both; anticipated drained phreatic surfaces generated through computer analysis to predict effects of road with and without various stabilization techniques on infinite-slope-recharged phreatic surface (2,3)
Data display	Resource inventory map overlay of factor of safety against failure or probability of landslide occurrence	Same as level 1 but for more localized project area that has potentially unstable locations of road cut and fill slopes shown on proposed route	Cross-sections of critical site conditions with proposed road and alternate stabilization techniques superimposed
Required data	Available forest resource inventory data, values for geotechnical variables and variance through broad characterization of forest land forms, variables and analysis model tested and refined through association with landslide inventory and subsequent evaluation in levels 2 and 3	Level 1 data, data from timber and route reconnaissance to delineate local areas within project where failures are most likely	Surface and subsurface critical site data; subsurface data from geophysical methods and drilling if severity warrants; soils and ground water hydrologic data from soil sampling and testing and ground water monitoring
Prime use	To delineate areas susceptible to landslides on broad scale to alert land manager to land units where hazard intensity is greatest; through statistical correlation to landslide inventory, to predict number and magnitude of landslides as a result of resource development	To assess severity of instability more accurately as local islands of instability are predicted through reconnaissance; to make decisions to limit development or to continue to level 3 analysis based on improved assessment of probable failure magnitude and intensity; to better evaluate transportation planning, timber harvest techniques, and route locations for project so critical sites can be isolated along selected routes where level 3 analysis will have most benefit	To select and design road stabilization measures through relative stability-probability of failure cost analysis of feasible alternatives

requires more exact data only as the intended use demands greater accuracy.

For soil-mantle landslide analysis the ideal system should be structured to

1. Provide landslide hazard evaluation to guide management decisions on unstable lands at three crucial phases: resource allocation, project planning, and road design;
2. Include soil, vegetation, slope, and ground water hydrologic variables together with their inherent natural variance in a geotechnical analysis (factor of safety against failure or critical height of slope), a statistical analysis (probability of landslide occurrence), or both;
3. Begin with a simplified analysis that requires primarily available resource inventory data and progresses into more complex analyses that require more exact data (the selection of technique should be commensurate with the level of management decision; thus, the user at any level is faced only with the complexity and need for data required at that level); and
4. Facilitate the inventory of new landslides as they occur and slope failures as they are corrected and feed back the data gathered into earlier processes to improve the planning of subsequent projects.

Three levels of analysis complexity and data are visualized for the idealized system in Table 1.

RESTRICTIONS ON USE

Existing Stability Analyses

Current restrictions on the use of an idealized evaluation system for soil-mantle landslides are not due to the lack of slope stability analysis technology. The program recently developed by Simons, Li, and Ward (4) for mapping potential landslides is based on an infinite slope analysis and includes both factor of safety against failure and probability of landslide occurrence options for a level 1 analysis. Stability number charts that have seepage correction factors are being developed for infinite slope seepage conditions (5) and converted to computer programs for the critical height analysis of typical road cut and fill slopes in a level 2 analysis. Numerous programs are available and in use by geotechnical specialists in stability analysis for the correction of existing landslides that have either circular arc and translational failure surfaces. The most widely used methods of slices (primarily the Fellenius, the simplified Bishop, and the Janbu methods) can be integrated into one program to cover a variety of failure surface analyses for level 3. Statistical counterparts for the probability of landslide occurrence option used in level 1 are planned for levels 2 and 3 based on methods currently used in geotechnical engineering (4,6).

Existing Data Base

One current restriction on using the system is the small existing data base for most forests. Many forest managers have (or are in the process of developing) resource inventory maps for soils, bedrock, topography, timber type, and other features. These maps could provide the start of a level 1 data base through proper characterization of geotechnical variables for the inventoried conditions. Statistical analyses used by Simons and Ward (4) and DeGraff (7) will prove invaluable for linking inventoried physical factors such as bedrock, aspect, and slope to inventoried landslides. The accuracy of the values assigned for geotechnical variables, analysis models, and the probability of landslide occurrence can be tested through association with corresponding physical factors. Currently, only a few forests have landslide inventory data. Geomorphic landtype maps (8), where available, should be the most useful tool for geotechnical variable characterization because the landtype classification includes the major physical factors on which to assign values for the variables.

Existing Variable Definition Methodology

The main restriction to implementing the system is the current state of the art in defining certain geotechnical variables. Techniques for defining slope, soil depth, and soil shear strength have progressed to a state where the values and their variance can be used with some degree of confidence. This is not true for the two most dynamic variables--ground water concentration and tree root strength.

The part of the soil mantle that can be expected to be below the phreatic surface at any point in time is perhaps the most dynamic of the variables. It can fluctuate constantly in response to precipitation. Practical and inexpensive methods are needed to develop local correlations between rainfall and snowmelt and the resulting rise in ground water. Although general knowledge of the time-related effects of tree root strength on forest slope stability has been advanced through research, currently no cost-effective quantitative methods are available for determining the effective tree root strength to use in analysis. To use the system now, it may be necessary to back-calculate to determine values for these two important variables until the state of the art progresses.

ILLUSTRATIVE PROBLEM

The following hypothetical problem illustrates the concept of the three-level analysis system. Where available, actual analysis results are used to demonstrate current progress. All studies within the project should be completed by mid-1985.

Level 1 Analysis for Developed Area

Step 1

Figures 1-3 show drawings of three inventory map overlays for part of the Clearwater National Forest in northern Idaho--the transportation map, landslide

Figure 1. Transportation inventory map of developed part of Clearwater National Forest, Idaho.

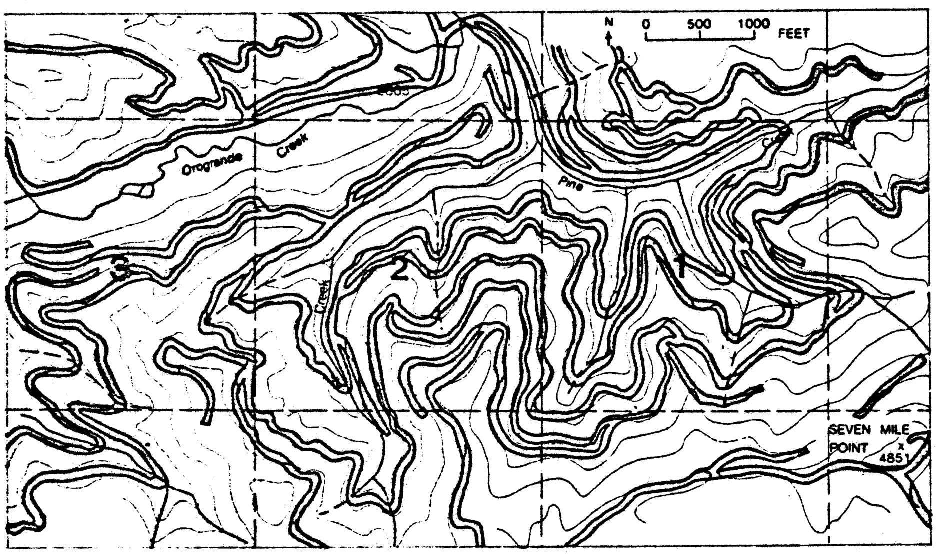


Figure 2. Landslide inventory map of area in Figure 1.

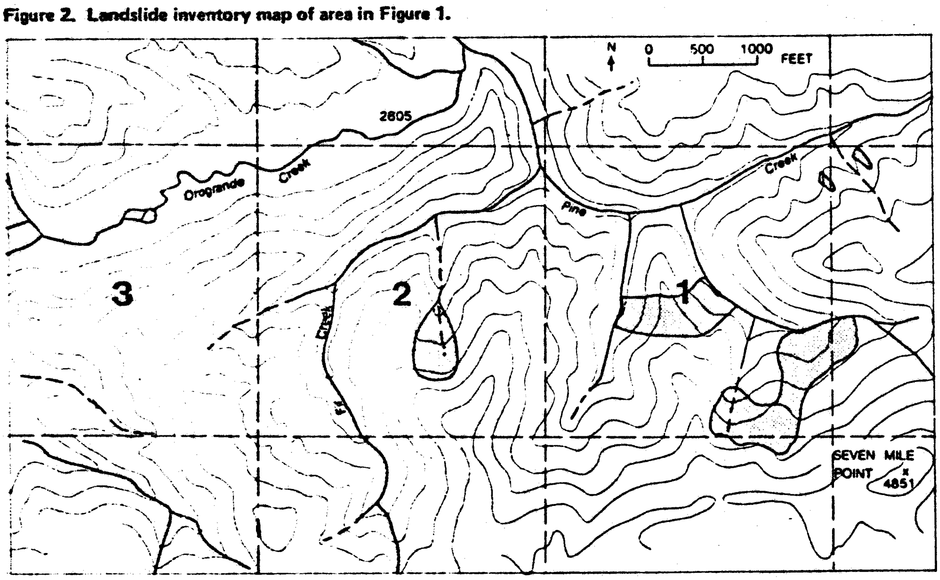


Figure 3. Geomorphic landtype inventory map of area in Figure 1.

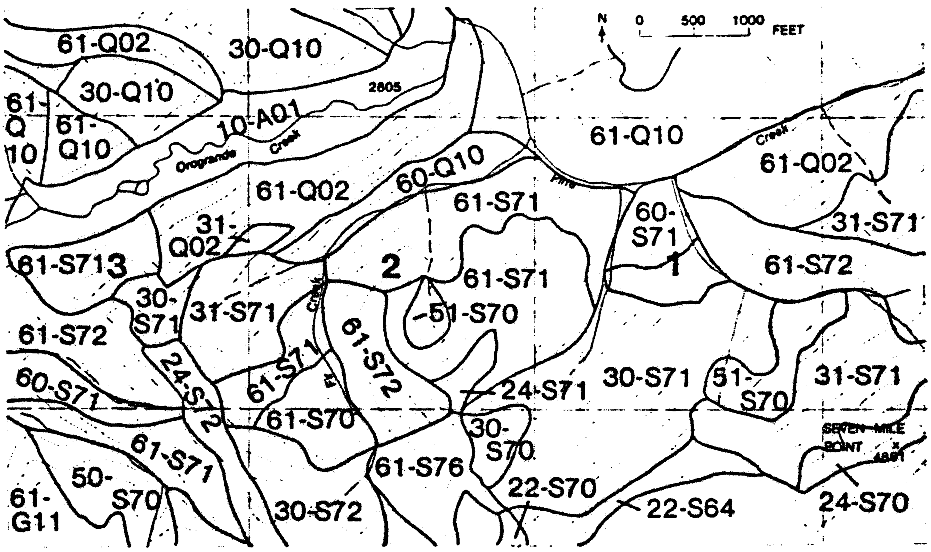
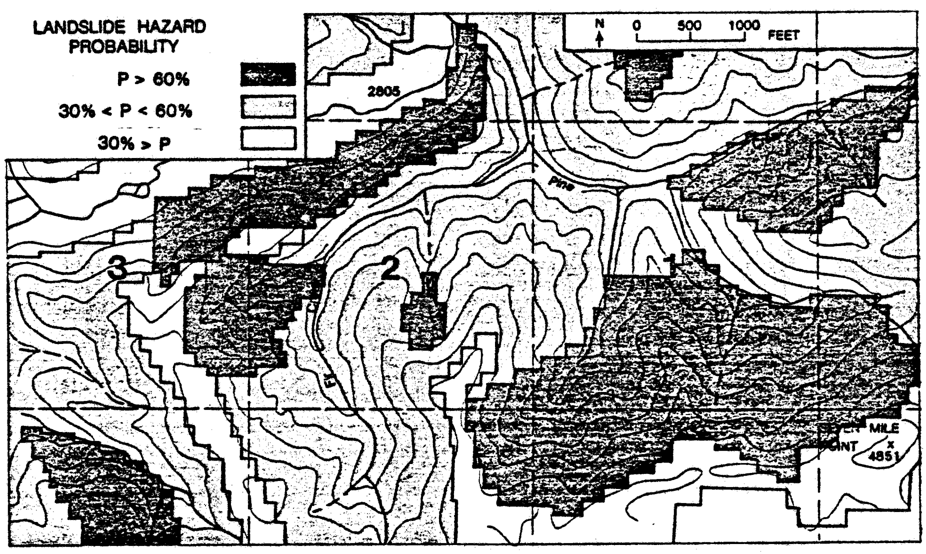


Figure 4. Results of level 1 analysis for Figure 1 area by using probability of landslide occurrence option.



inventory map, and geomorphic landtype map, respectively. The association of these inventories is the most useful for this forest as an initial level 1 data base. The transportation and landslide maps show the location of all existing roads and landslides. The landtype map shows the boundaries of distinctive geomorphic landforms. The coded landtype classification includes the major geomorphic physical factors that make this mapping unit distinctive (8). Physical factors such as parent rock type, aspect, slope, and timber type are used in the classification and corresponding values for geotechnical variables such as soil shear strength, soil depth, slope, and ground water concentration, and are characterized and stored into an initial level 1 data base. This data base will be updated by using new data from levels 2 and 3.

Step 2

The level 1 (infinite slope) analysis is completed. The results will be either in ranges of factor of safety against failure or probability of landslide occurrence. Figure 4 shows the printout of the Simons, Li, and Ward program by using the option of ranges of probability of landslide occurrence for the data from step 1. This analysis for developed areas can be repeated as necessary to test and refine the model and variable values until the planner is satisfied that the results correlate realistically to the landslide inventory. In this procedure, the planner must remember that the values are to be used in conjunction with the infinite slope equation. This model is selected for this level of analysis because of its simplicity but not necessarily because of its accuracy. Accuracy depends largely on how well the model fits the ground water concentration mechanism and whether translational failures develop; even then the model will probably be applicable only to parts of any landtype (where the worst conditions exist).

Level 1 Analysis for Undeveloped Area

Step 3

Step 3 is similar to steps 1 and 2 for adjacent undeveloped areas with similar landtypes. Figure 5 shows the transportation map of the undeveloped area. Figure 6 is the level 1 analysis printout of landslide hazard probability. By beginning the analysis in this manner, the planner can calibrate the analysis by using the developed areas for predictions about the undeveloped areas to aid the land manager in resource planning decisions on whether or not to develop, how intensely to develop, and the landslide risk involved as a result of development. In addition, the following advantages are available through a level 1 analysis:

1. The land manager can be given a comparison of landslide magnitude and consequences by relating to experiences in the developed areas.
2. The accuracy of at least some of the level 1 data base can be improved through the feedback loop from levels 2 and 3, which follows.
3. The intensity and location of the level 2 analysis can be planned commensurate with the anticipated landslide hazard.

Level 2 Analysis

Step 4

Figures 5 and 7 show the area selected for level 2 analysis on levels 1 and 2 scales. In this case, the level 2 analysis is used to evaluate two possible routes to a proposed log landing site. Reconnaissance data are gathered at selected cross-sections along each route for better assessment of the extent of the anticipated problem areas and estimation of values for geotechnical variables.

Figure 5. Transportation resource inventory map of undeveloped part of Clearwater National Forest, Idaho, showing existing road terminal.

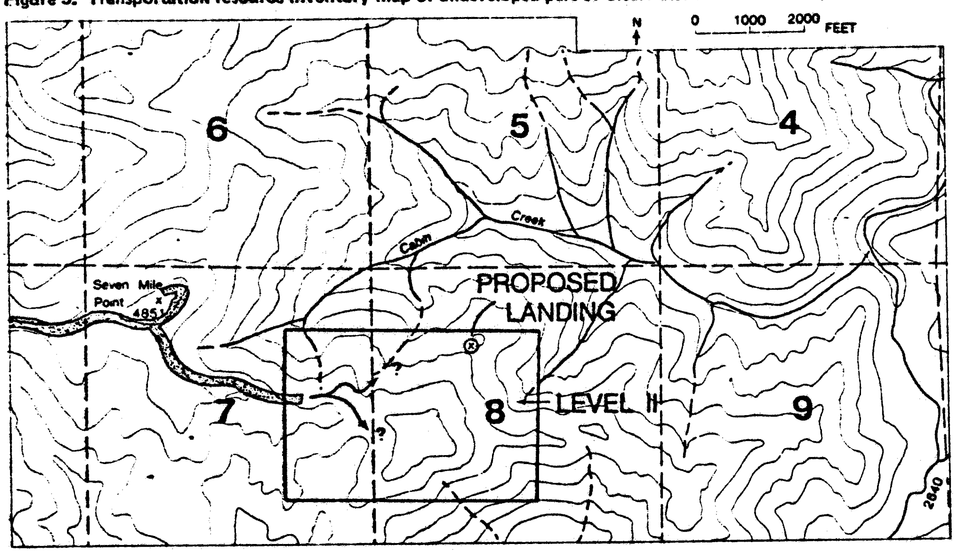
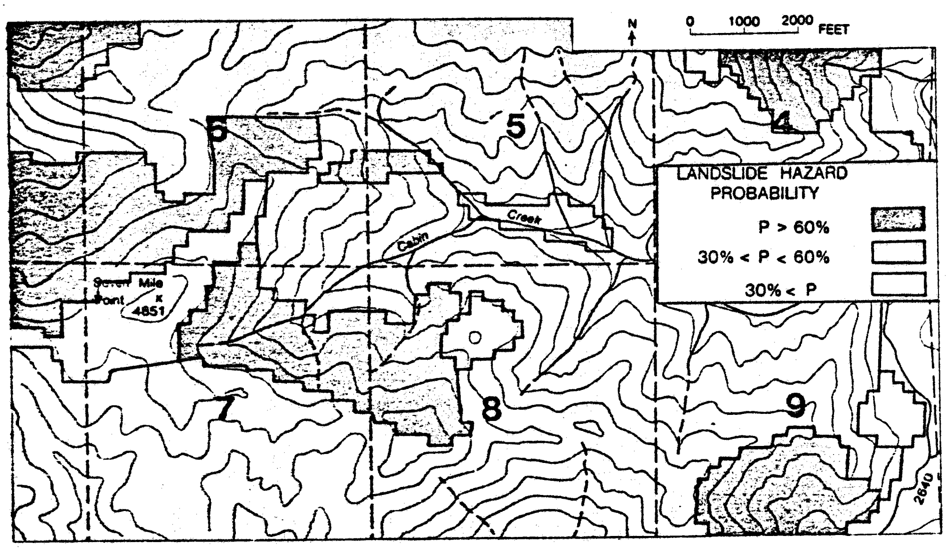


Figure 6. Results of level 1 analysis for undeveloped area in Figure 5.



Step 5

Typical road template sections are superimposed on the selected cross-sections and cut slope height, fill slope height, and the relation of cut and fill to the ground water level, root zone, and drainage barrier contact are determined by computer analysis. Figure 8 shows a self-balance road template commonly used on forest roads (cut volume balances fill volume with appropriate compaction factor). The critical heights of the cut and fill slopes are then determined and compared with the anticipated slope heights. Figure 9 shows the prototype program printout from a programmable calculator for a combined levels 1 and 2 analysis of the cross-section of Figure 8. The compaction factor can also be evaluated by this analysis. A full-bench road template may also be used on steep slopes where a fill slope will not catch or would be too high.

Step 6

A program similar to that used for Figure 9 will be developed as a subroutine for a computer analysis that represents the results as either S for stable or U for unstable on a project map. In addition, a statistical subroutine will be developed similar to that in level 1 for an optional output in terms of probability of slope failure. Figure 10 is a hypothetical drawing of the anticipated display.

Step 7

To assess the impact of timber harvest (tree removal) on the stability of the natural slopes, the level 1 analysis will be repeated at level 2 with changes made in tree-root strength, tree surcharge, and ground water concentration to reflect the impact

Figure 7. Level 2 analysis area showing location of alternate routes to proposed landing and selected cross-section locations on each route.

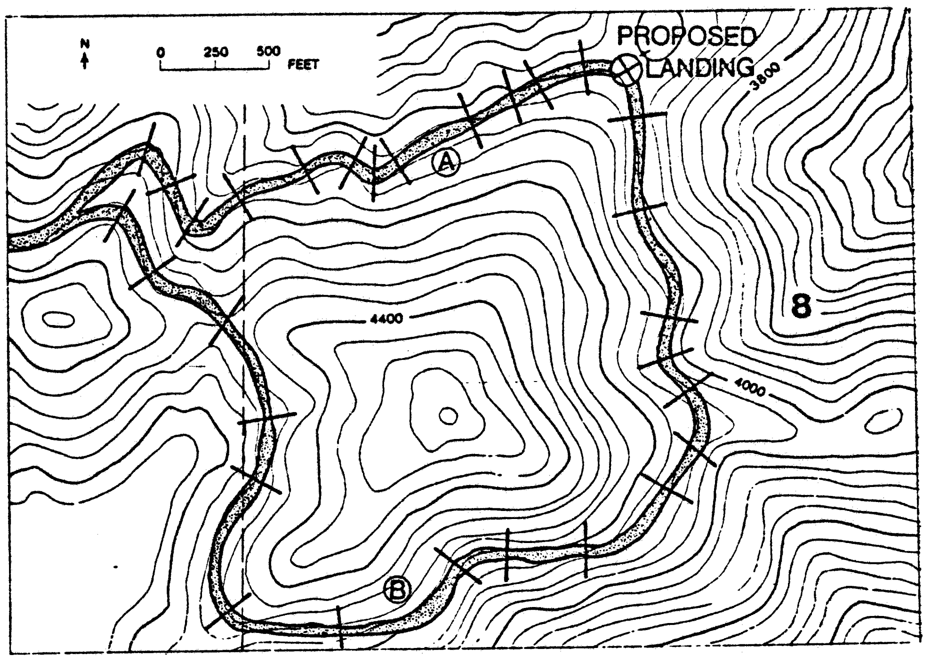


Figure 8. Self-balancing road template cross-section from level 2 analysis summarized on Figure 9.

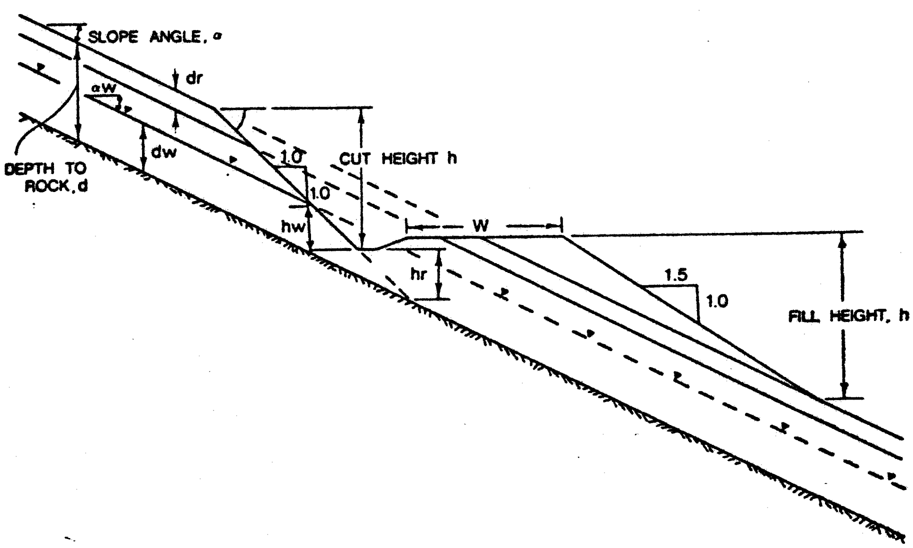
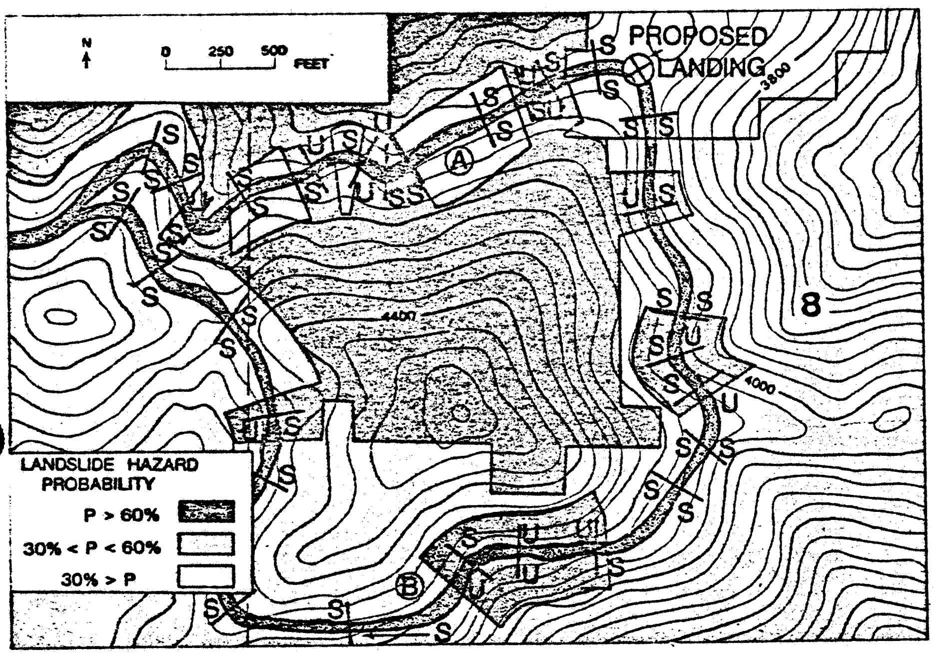


Figure 9. Printout of level 2 analysis of Figure 8 cross-section data.

```
STA.134+50

SOIL DATA
 DEN.1 DEN.2  PHI  COH.
 120.0 130.0 32.0 40.0

ROOT DATA
 5 YRS. AFTER HARVEST
ROOT COH.= 20.0PSF/FT.

SITE DATA
ALPHA= 50.0%= 26.6DEG.
AL.W= 50.0%= 26.6DEG.
  d   dw   dr
 10.0 3.0 2.0
   INF. SLOPE F.S.= 1.02
           STABLE

ROAD DATA
      CUT SLOPE= 1.00:1
DITCH
 SLOPE  DEPTH  BOT.W
 3.0:1   1.0   2.0
    ROAD WIDTH= 16.0FT.
     FILL SLOPE= 1.50:1
      COMP. FACT.= 25.%
FOR 50.% LOSS IN dr,
       COMP. FACT.=22.%

SELF-BALANCE SECTION

CUT
  h    hw    hr
 14.4  4.4 -5.6
  N    S    Hc
 42.8 0.52 14.7
         STABLE

FILL
 DEN.  PHI  COH
 130.0 34.0 40.0
  h   hw   hr
 17.3 -1.4 -17.3
  N    S    Hc
 151.8 0.73 34.0
         STABLE
```

Figure 10. Hypothetical drawing of the probability of landslide occurrence for level 2 analysis.



(9). The uses of the level 2 analysis are then as follows:

1. To facilitate management decisions on development through evaluation of alternate transportation routes and alternate timber harvest techniques and
2. To locate the critical sites where level 3 analyses are necessary on the selected routes.

Level 3 Analysis

Step 8

Figures 11 and 12 show one critical site selected for level 3 analysis on levels 2 and 3 scales. A critical site investigation (both surface and subsurface) is made for each site selected. The extent of this investigation and the subsequent analysis are planned by the geotechnical specialist in the same manner as a landslide correction project is planned.

Step 9

The anticipated road section is superimposed on cross-sections of the critical site and the stability of the anticipated cut and fill slopes are analyzed for circular arc, translational failure, or both. This step differs from step 5 in that the mode of failure is analyzed to determine the failure surface that has the least factor of safety and the anticipated extent of the slide mass. Many stability analysis programs are in use that would serve as a level 3 analysis for either shape of failure surface. Plans are to formulate the most functional of these as subroutines for one master program. Figure 13 shows possible translational and circular arc failure surfaces for the cut slope on the cross-section of the critical site. Figure 14 shows a programmable calculator printout for a program that combines the Fellenius (ordinary method of slices), simplified Bishop, and Janbu methods of slices solution for failure along these surfaces. The master computer program will combine analyses such as these, which can be preselected by the designer in conjunction with failure surface predicting, slice generating, and optional search for minimum factor of safety subroutines. Subroutines for predicting the steady-state drained phreatic surface to be expected from an infinite slope seepage source will also be programmed to evaluate the various drainage conditions in steps 9 and 10.

Step 10

The analysis of the unstabilized case in step 9 serves as a standard of comparison for the relative stabilizatiòn technique analysis that begins with step 10. In step 10 all feasible stabilization alternatives are analyzed to determine the relative increase in factor of safety over the unstabilized case.

Step 11

Decision analysis components (6) are determined for each alternative:

1. Probability of failure,
2. Construction and maintenance costs,
3. Consequences of failure (cost of failure).

Level 3 analysis provides the design engineer a decision analysis through which to select the optimum stabilization alternative for the current constraints.

Feedback to Level 1

Step 12

The data gathered for levels 2 and 3 are fed back into the level 1 data base to improve future analy-

Figure 11. Level 2 base map showing one area selected for level 3 analysis.

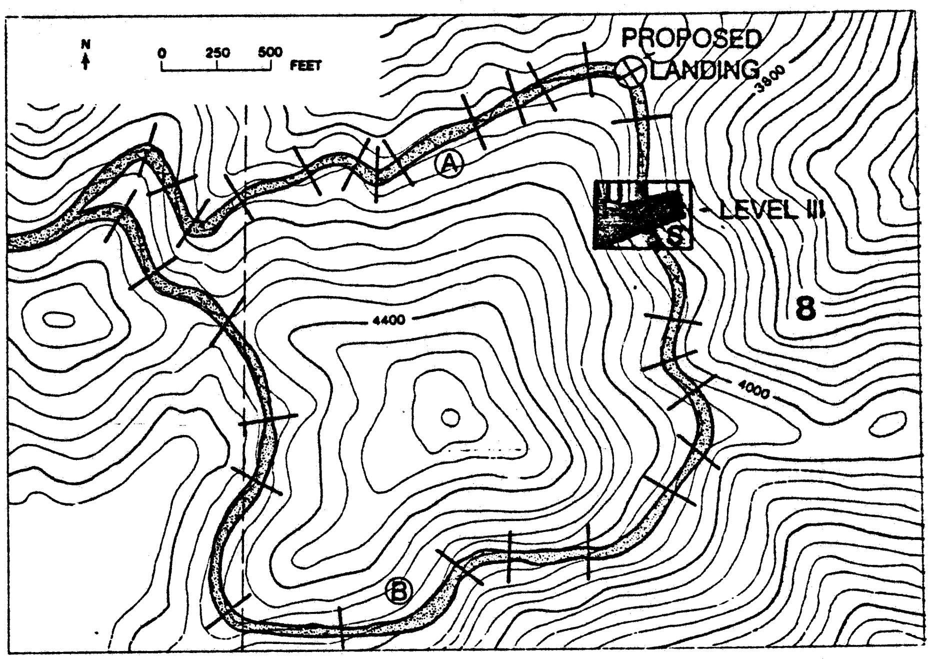


Figure 12. Level 3 analysis area showing proposed road.

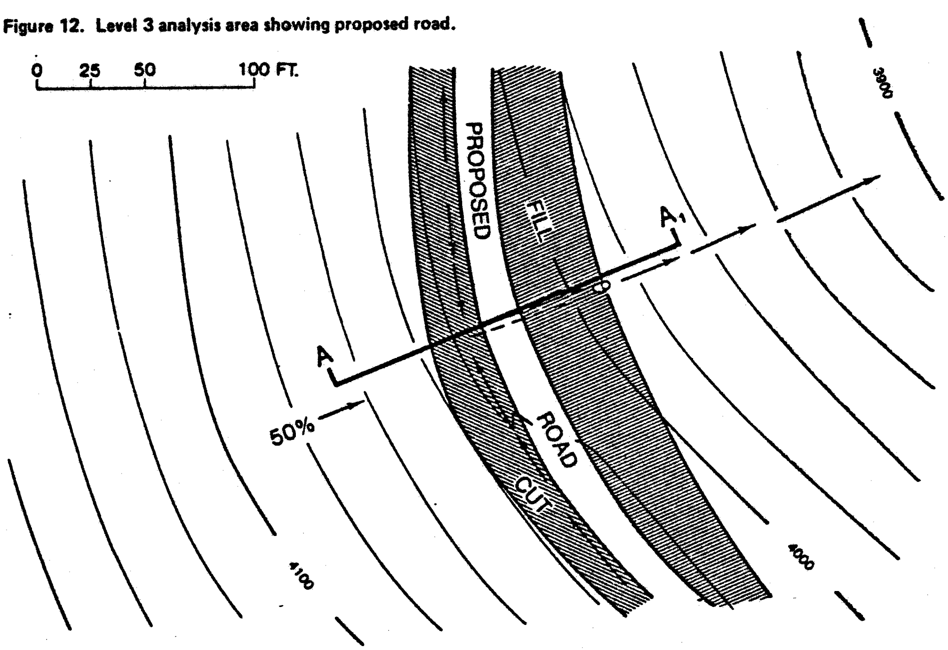


Figure 13. Cut slope portion of cross-section A-A' from Figure 12 showing possible circular arc and translational failures analyzed in Figure 14.

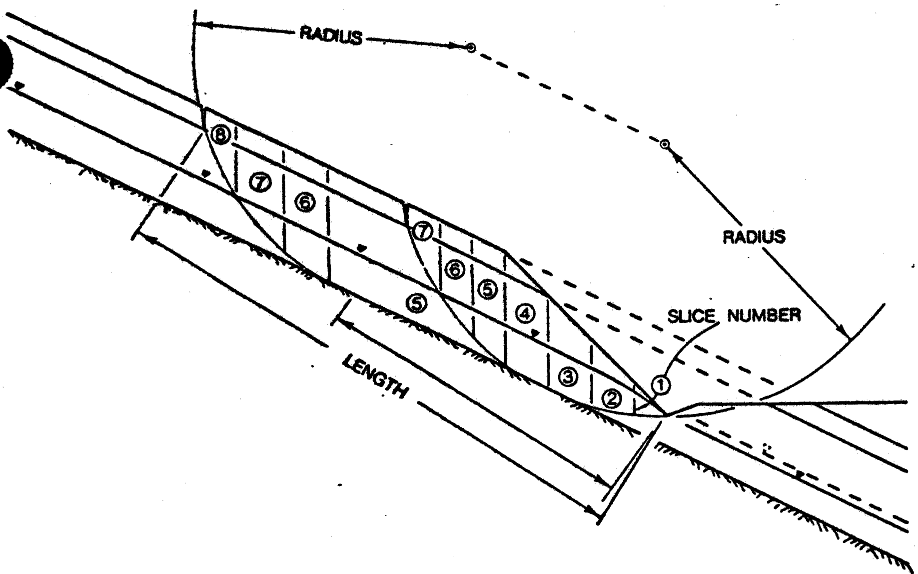


Figure 14. Printout of the level 3 analysis of Figure 13 cross-section data.

```
I.D.: CIRCULAR ARC

OMS BISHOP JANBU
          chord d= 4.5
          chord l= 29.5
TENSION CRACK
               Zw= 2.0
                a= 6.6
                R= 25.6
MIN FS= 0.78
MAX FS= 0.90

SLICE 1
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 110.0 125.0 32.0 0.0
THETA  d1  dW  X  AL.W
 4.0 0.5 1.0 3.0 33.5

SLICE 2
THETA  d1  dW  X  AL.W
 11.5 1.7 2.8 4.0 28.0

SLICE 3
THETA  d1  dW  X  AL.W
 20.9 3.6 3.3 4.0 26.6

SLICE 4
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 106.0 125.0 32.0 0.0
THETA  d1  dW  X  AL.W
 31.0 5.5 3.6 4.0 26.6

SLICE 5
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 103.0 125.0 32.0 0.0
THETA  d1  dW  X  AL.W
 41.5 6.8 2.1 3.6 26.6

SLICE 6
THETA  d1  dW  X  AL.W
 51.5 6.8 1.1 3.0 26.6

SLICE 7
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 101.0 125.0 32.0 60.0
THETA  d1  dW  X  AL.W
 65.0 4.6 0.0 3.0 0.0

  OMS FS=0.78

  BMS FS=0.87

  JMS FS=0.87

I.D.: TRANSLATIONAL

OMS BISHOP JANBU
          chord d= 6.1
          chord l= 49.5
TENSION CRACK
               Zw= 2.0
                a= 6.6
                R= 25.0
MIN FS= 0.00
MAX FS= 1.00

SLICE 1
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 110.0 125.0 32.0 0.0
THETA  d1  dW  X  AL.W
 4.0 0.5 1.0 3.0 33.5

SLICE 2
THETA  d1  dW  X  AL.W
 11.5 1.7 2.8 4.0 28.0

SLICE 3
THETA  d1  dW  X  AL.W
 20.9 3.6 3.8 4.0 26.6

SLICE 4
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 106.0 125.0 32.0 0.0
THETA  d1  dW  X  AL.W
 26.6 5.5 4.0 4.0 26.6

SLICE 5
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 103.0 125.0 32.0 0.0
THETA  d1  dW  X  AL.W
 26.6 6.8 4.0 16.0 26.6

SLICE 6
THETA  d1  dW  X  AL.W
 36.2 6.8 3.5 4.0 26.6

SLICE 7
THETA  d1  dW  X  AL.W
 49.5 6.8 1.7 4.0 26.6

SLICE 8
NEW SOIL
 DEN.1 DEN.2 PHI  COH.
 101.0 125.0 32.0 60.0
THETA  d1  dW  X  AL.W
 65.0 4.6 0.0 3.0 0.0

  OMS FS=0.88

  BMS FS=0.92

  JMS FS=0.94
```

ses. Techniques for data storage and analysis that upgrade the values for geotechnical variables for each landtype as the sample size is expanded (10) will be used.

SUMMARY AND CONCLUSIONS

The concept for a three-level landslide analysis system has been outlined. Important points regarding the system are as follows:

1. Each level of analysis is designed to require its own data base and to provide guidance for land management decisions at that level only. The level of analysis complexity, data required, and accuracy must be commensurate with the type of management decision they are intended to support.
2. A loop that channels levels 2 and 3 data back into the level 1 data base will upgrade the accuracy for future analyses.
3. Although the system described is for soil-mantle failures common in residual and colluvial soils, the concept is a series of building blocks that may be made applicable to rock slope failures by the proper substitutions.
4. Current restrictions on use of this system are not in the analysis techniques that are either in existence or at least feasible for development. The current restrictions are (a) the general lack of a dynamic and easily upgraded storage system and (b) the present state of the art for determining the values for certain geotechnical variables such as ground water concentration and effective tree root strength.

REFERENCES

1. D.J. Varnes. Slope Movement Types and Processes. In Landslides Analysis and Control. TRB, Special Rept. 176, 1978, pp. 11-33.
2. L.K. Moulton. Control of Groundwater. In Highway Subdrainage Design, FHWA, Rept. FHWA-TS-80-224, 1980, pp. 114-140.
3. R.W. Prellwitz. Analysis of Parallel Drains for Highway Cut-Slope Stabilization. TRB, Transportation Research Record 705, 1979, pp. 2-7.
4. D.B. Simons, R.M. Li, and T.J. Ward. Mapping of Potential Landslide Areas in Terms of Slope Stability. Colorado State Univ., Fort Collins, Colo., Rept. CER78-79 DBS-RML-TJW19, 1978, pp. 1-75.
5. R.W. Prellwitz. Simplified Slope Design for Low-Standard Roads in Mountainous Areas. In Low Volume Roads, TRB, Special Rept. 160, 1975, pp. 65-74.
6. D.C. Wyllie, N.R. McCammon, and W. Brumund. Planning Slope Stabilization Programs Using Decision Analysis. TRB, Transportation Research Record 749, 1980, pp. 34-39.

7. J.V. DeGraff. Quantitative Approach to Assessing Landslide Hazard to Transportation Corridors on a National Forest. TRB, Transportation Research Record 892, 1982, pp. 64-68.
8. W.D. Wilson, R. Patten, and W.F. Megahan. Systematic Watershed Analysis Procedure for Clearwater National Forest. TRB, Transportation Research Record 892, 1982, pp. 50-56.
9. R.E. Campbell and T.J. Ward. Predicting Water and Sediment Yields from Forest Watersheds: a Methodology for Delineating Potential Landslides. Rocky Mountain Forest and Range Experimental Station, U.S. Department of Agriculture, Fort Collins, Colo. (in process).
10. C.L. Vita. A Landform-Based Probabilistic Methodology for Site Characterization. Proc., 19th Annual Idaho Engineering Geology and Soils Engineering Symposium, Pocatello, 1982, pp. 339-354.

TRANSPORTATION RESEARCH RECORD 919

Evaluating Strength Parameters of Simple Clays: Geotechnical Consideration of Residual Soils

TRANSPORTATION RESEARCH BOARD

NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES
WASHINGTON, D.C. 1983

STREAMBANK PROTECTION MEASURES

Soil Bioengineering methods and design criteria for protecting streambanks and controlling erosion

Definitions:

Soil Bioengineering is a method which uses specific live native plant materials as its main structural components. Live plant materials which are unrooted cuttings, are placed on or in the ground, i.e., on streambank slopes in such a manner that they serve as erosion and sedimentation controlling devices. These plants are able to grow and assist in streambank stabilization.

Branches or Live Cut Branches refer to material that has been cut from native growing material. This live cut plant material is intended to root and grow. Live rootable plant material is used in all soil bioengineering structures, i.e., live cribwalls, live stakes, joint planting, live soft gabions, brushlayering, brushmattress and live fascines. These are used either singularly or in conjunction with dead conventional parts.

Growing Tips refer to the top ends of the live cut branches which are expected to produce leaf development.

Basal Cut Ends refer to the base or butt end of the live branches which are expected to produce root development.

Biotechnically Suitable Plants refers to the suitability of a plant and its root formation for soil bioengineering purposes, (live constructions), eg. its ability to root through cuttings, its ability to withstand dry or wet conditions etc.

Pioneer Plants are plants that normally colonize or invade a disturbed land site or raw mineral soils and modify or prepare them for succeeding plants.

Vegetative Propagation is propagation without pollination by way of seperating vegetative parts (branches, stolons, buds) from the mother plant and planting them so that they may take root and grow.

Bank Slope is an artificially established site having a certain angle with flattened or steepened sections on the top and bottom.

Cut Slope is a bank or area excavated or eroded to a certain angle.

Fill Slope is an embankment that is created by the placement of earth at a certain angle.

LIVE SYSTEM FIGURES

LIVE SYSTEMS

Live Stakes: (Figure located on sheet No.1)

Live Stakes are sticks that have been cut and pruned from living plant material. All of the side branches are trimmed. They shall root and grow, to consolidate soil particles, remove moisture from a bank and create siltation deposition. These live pieces of plant material are used singularly or in combination to join parts within other systems, i.e., brushmattess or fascine construction.

Joint Planting: (Figure located on sheet No.1)

Joint Planting is similar to live staking work except that the live stakes are tamped into the ground between riprap stones. These live stakes are intended to root and form a "live root mat" in the base upon which the riprap stone has been placed. This shall allow for the natural release of waters from the bank. The top growth shall cause siltation to occur on the bank.

Brushlayering: (cut slopes) (Figure located on sheet No.2)

Brushlayer construction on cut slopes, is a proceedure used that consists of cutting terraces in a slope. Live cut plant materials, in the form of live branches, are placed on the terraces to form a brushlayer. The portions of the brush that protrudes from the terrace on the slope will assist in retarding surficial runoff erosion.

Brushlayering: (fill slopes) (Figure located on sheet No.2)

Brushlayer construction on fill slopes, is a proceedure used that consists of placing live plant mateial, in the form of living branches, on terraces. These terraces are created as the fill is conventionally placed. When the fill operations are completed, the protruding sections of the brushlayers shall assist in retarding surficial runoff erosion. The branch parts in the fill shall serve immediately as soil reinforcing units.

Live growing brushlayer systems are intended, through the live root system to consolidate the soil particles, and through transpiration, "pump" water out of the bank. The pioneer species used in the brushlayers shall improve and stabilize the soils. This shall encourage the invasion of the surrounding plant species, thus increasing the strength of the live system with age.

Live Fascine: (Figure located on sheet No.3)

A Live Fascine is a collection of live cut branches grouped together into a sausage or cigar-like structure, with the growing tips all placed in the same direction. The unit is placed in a dug trench and serves immediately as a pole drain, and as a securing device. The live structural parts shall root and "pull" water out of the bank, through transpiration.

Live Soft Gabions: (Figure located on sheet No.3)

Live Soft Gabions are gabion-like structures that are made of geogrids, a polymer material. They are used for rapid repairs, in conjunction with granular fills, where immediate reinforcement is required. These are often useful in areas where heavy loads are to be applied to an upper surface. This geogrid is used in conjunction with live branch layers. The live plant stock serves to give immediate mechanical reinforcement, and long term soil consolidation, that is, "soil binding" and water pumping capabilities. Additionally the final growing product is beautiful.

Live Cribwall: (Figure located on sheet No. 4)

A Live Cribwall consists of a hollow, boxlike, interlocking arrangement of logs, timbers etc., filled with soil/granular fill, and cut, living, unrooted plant material. The plants are intended to root. This will consolidate soil particles and through transpiration remove water out of the slope, and cause siltation to occur. It is capable of binding the toe structure to the natural bank. This creates a complete, natural and flexible unit.

Branchpacking: (Figure located on sheet No.4)

Branchpacking is a proceedure used to repair gullies, washouts and scours. This is especially useful to repair damage adjacent to bridge wing wall units. It is similar to brushlayering except that the live branches or live brush is "packed" closer together. It functions immediately, to become the new edge of the bank. During the flood stage it shall capture debris and sediments.

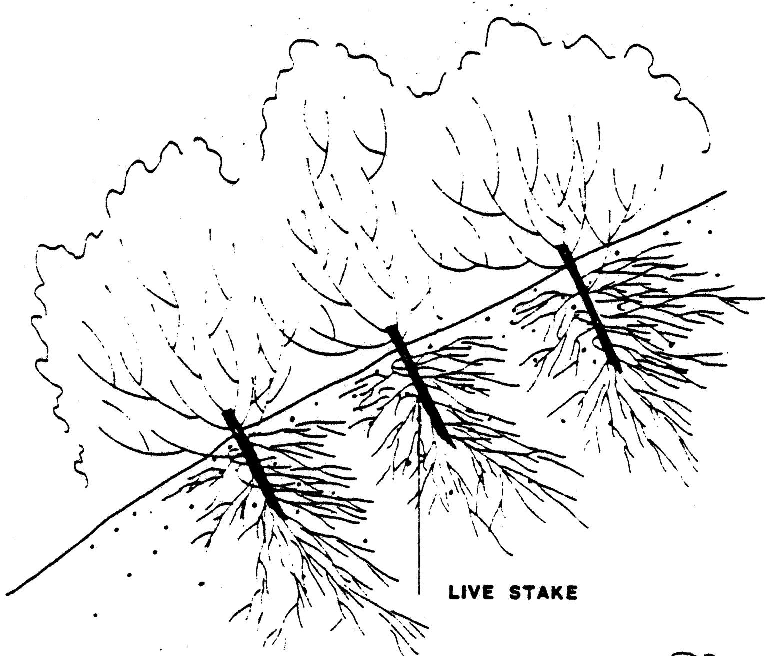
Live Siltation: (Figure located on sheet No.5)

Live Siltation construction is a proceedure used to capture silts and sediments during flood conditions. It is therefore necessary that it be constructed in an area that the waters during flood stages will pass over. The system is used to rebuild a lost bank or land section along a stream bank. It consists of several live brush layer barriers, over which the flood waters will flow. The intent is that the velocity shall be slowed in this immediate zone, and the sediments will be dropped, thus naturally rebuilding the site.

Brushmattress: (Figure located on sheet No.5)

Brushmattress refers to a live construction that places living unrooted branches close together to form a "mattress" or cover over the ground. This brushmattress immediately serves to cover and protect the bank. The live branches shall root and produce leaves along their entire length. The brushmattress shall grow and afford greater protection as it ages. Under normal conditions it will function immediately and cause sediments to be dropped during flood conditions. This will then rebuild the bank naturally. During flood conditions the soft flexible live material lies down, against the bank. This creates a natural flexible layer that further protects the bank. Velocities are lowered in this immediate zone, thus bank protection is rendered.

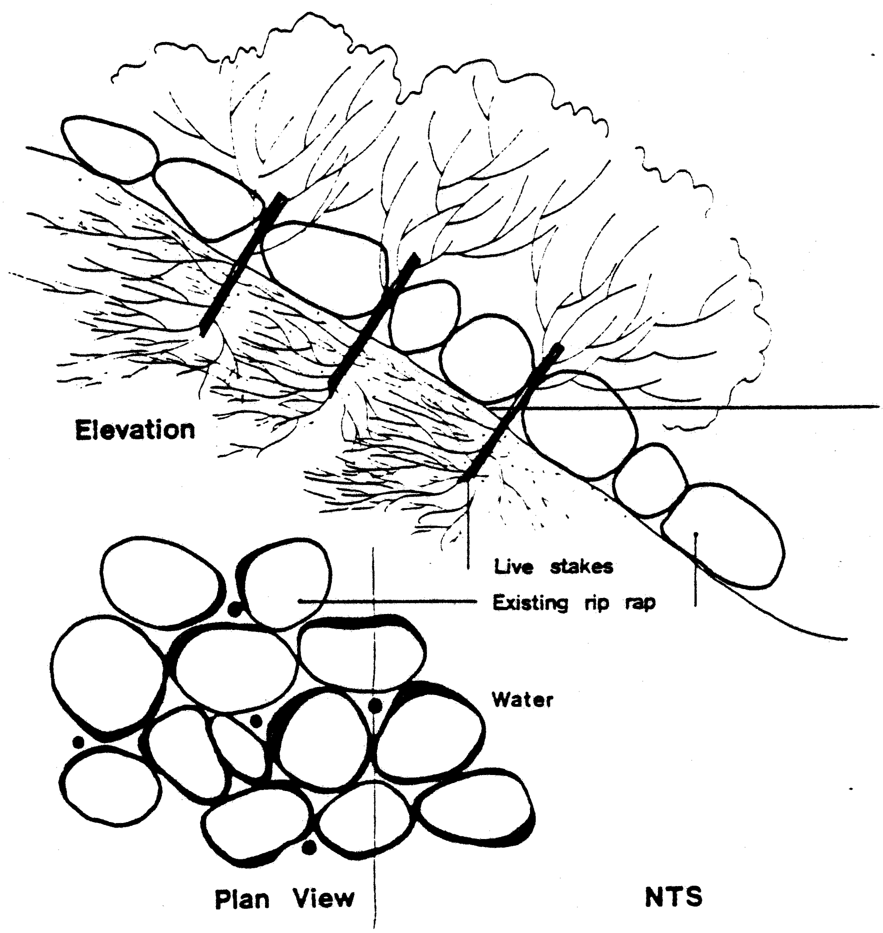
LIVE STAKING



Elevation

NTS

JOINT PLANTING



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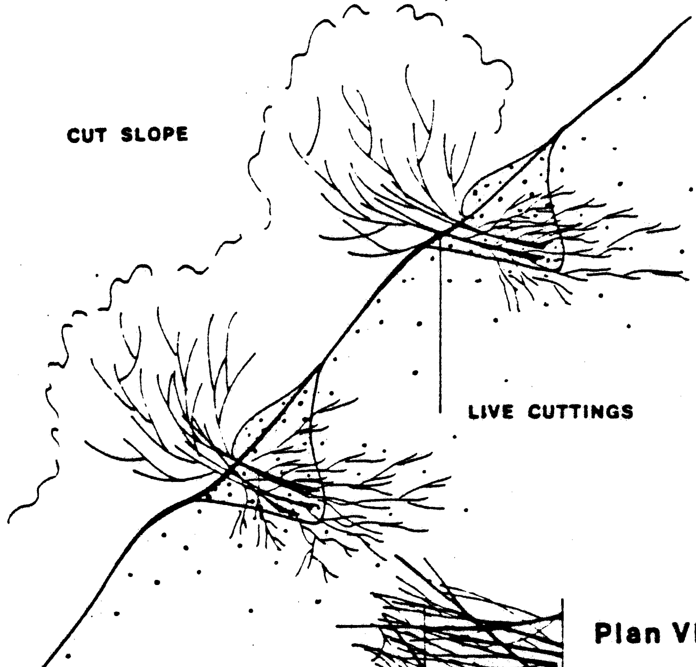
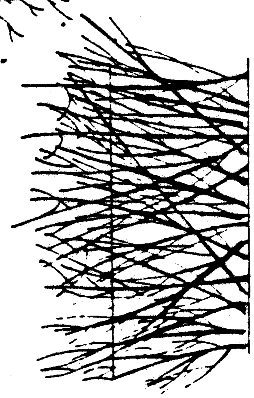
BRUSHLAYERING

CUT SLOPE



LIVE CUTTINGS

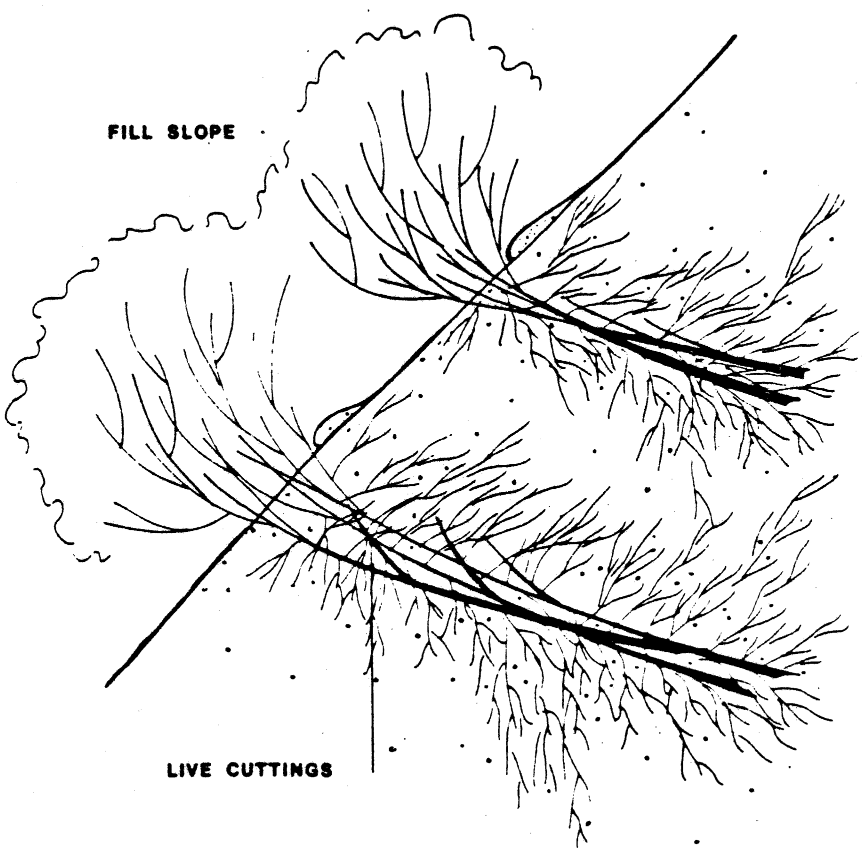


Plan View

Configuration

Section

FILL SLOPE



LIVE CUTTINGS

Section

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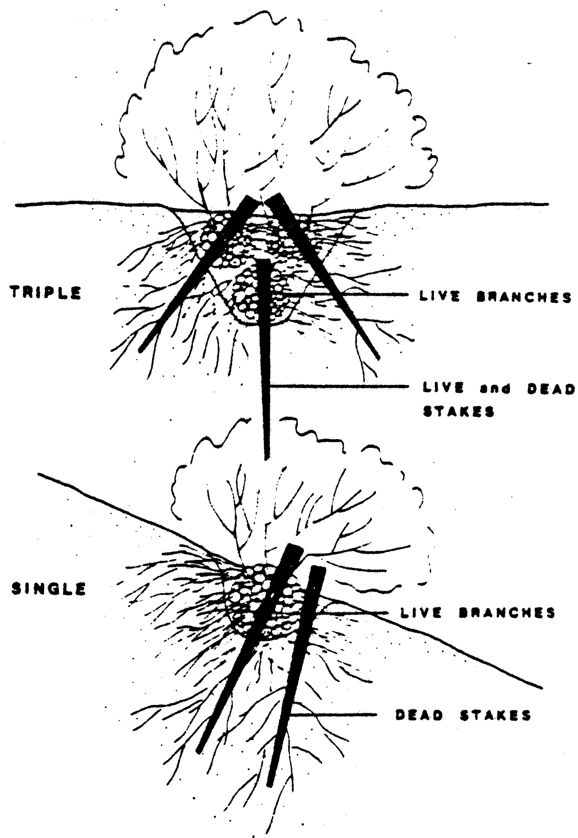
LIVE FASCINE



Elevation
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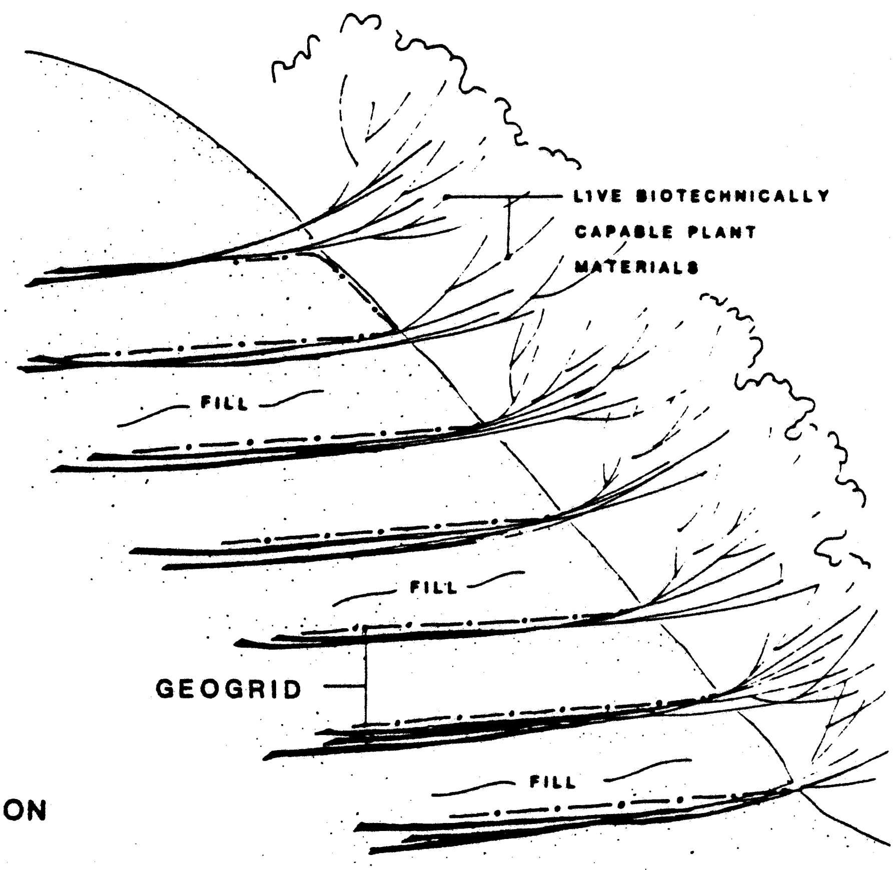
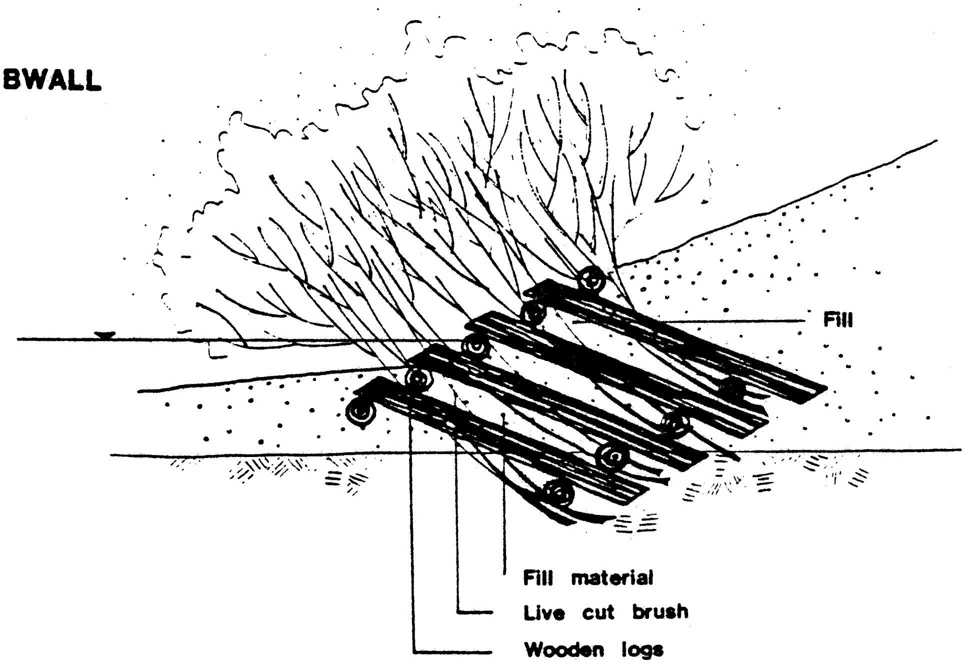


LIVE SOFT GABION

LIVE CRIBWALL



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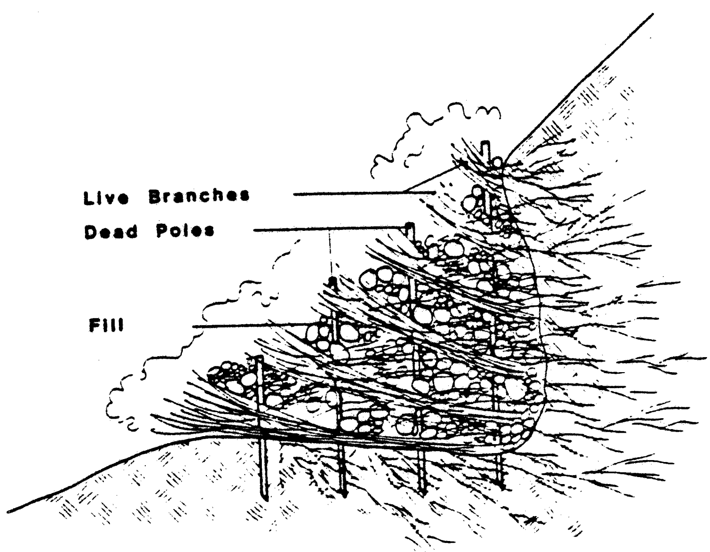
BRANCHPACKING

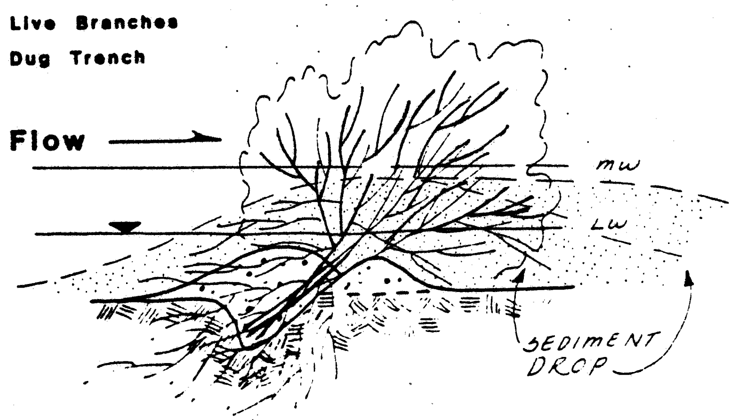


Typical Cross Section of a Filled Area

Elevation

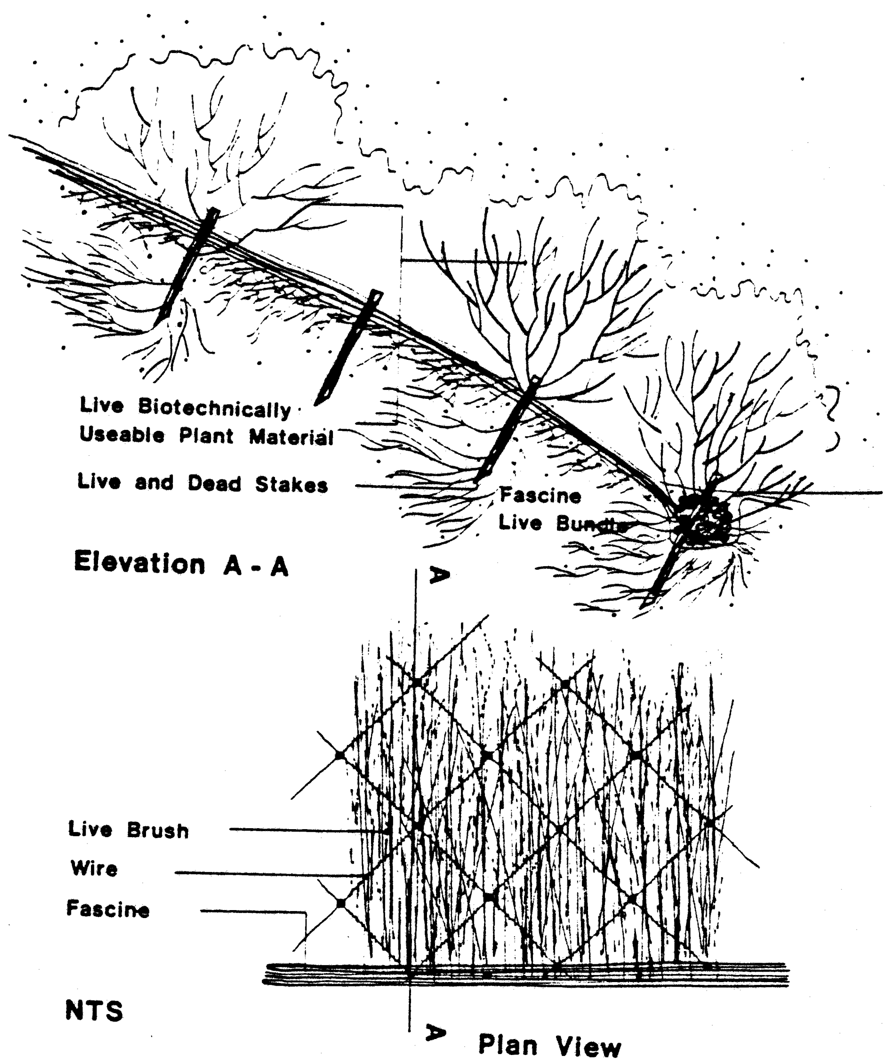
NTS

LIVE SILTATION CONSTRUCTION



Cross Section of a Typical Row

BRUSHMATTRESS



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A BRIEF HISTORY OF SOIL BIOENGINEERING

Soil bioengineering is an applied science that combines engineering, biological, and ecological concepts to construct living structures for erosion, sediment and flood control. It is a tested and proven system. Written proclamations and directives to use live plants to control flooding, river meandering, erosion and sedimentation problems, on river and stream banks, dikes, etc., date back to the 1500's. The techniques have steadily developed and improved since these times. Today soil bioengineering is a widely accepted biotechnical discipline, one which is rapidly gaining respect in the United States. Soil bioengineering is used in over thirty countries in the world at this time.

These early natural materials construction methods of erosion control, practiced for centuries in Europe, Asia and America, became unpopular as the mechanization that marked the progress of the Industrial Revolution continued into the 20th Century. The age of machines and the development of concrete and steel technology encouraged the use of rigid, dead construction materials in engineered projects. These materials allowed for exact geometric measurements and designs and suited precise hydraulic and stress calculations. The hard systems also initially promised to be longer lasting, cheaper and safer.

In the United States the abandonment of vegetation and living structures for erosion and sediment control was rapid and complete. The low cost of energy, the high cost of labor, and the wide distribution and abundance of raw materials needed in the fabrication of steel and concrete, encouraged this trend in America and other countries.

Europe, on the other hand, continued to use and improve live construction methods. By the 1930's, professionals in various disciplines, such as Alwin Seiffert and Alexander Von Kruedner, held that natural construction, the use of biological strengths, and the careful consideration of natural existing relationships, was the most reasonable method of erosion and sedimentation control. In 1937, Eduard Keller of Austria undertook the first recorded scientific experiment with willows used as a live construction element and originated the term "Living Construction." In 1936 similar work was being done by Krabel in the United States. In Europe in 1943, the term "Natural Construction" was used in print by another group of scientists to describe methods of water retention, water distribution and protection of inflexible structures with planted materials. From these early experiments and projects evolved the concepts of what is now referred to as "Soil Bioengineering."

One of the first major advances in the area of Soil Bioengineering occurred during the Great Depression while Wilhelm Hassenteufel was construction supervisor for mountain stream and avalanche protection at Reiutte in the Tyrol/Austria. At the same time that other areas were using lean-mix concrete, Hassenteufel systematically investigated the potential of natural materials located near the construction sites. They were free and required minimum transportation costs. According to Hugo Schiecthl, a world renowned Soil Bioengineer, these emergency measures met with almost complete success and resulted in new construction methods such as the planting of dry stone wall joints with woody cuttings, cribwall construction with branchlayering and vegetation wall construction.

After World War II, a number of investigators including Schiechtl, Henson, Pruckner, Kirwald, Kruedener and Bittman studied, developed and evaluated "Living Construction" and published their experiences. During this period a comprehensive construction technology using live materials was developed, and numerous experiments and projects utilizing current biological research were carried out, to determine the permanence and maintenance requirements of each technique.

In order to develop a uniform nomenclature for what was variously called "Living Construction," "Biological Construction," and "Landscape Construction," and to enable this concept to be easily adopted by public agencies, a committee of experienced persons was formed in 1950. This committee which represented several of the German, Austrian and Swiss states, worked to place all methods of shoreline protection, with both dead and live materials, into the German system of standards and measurements (DIN).

U.S. Army Corps of Engineers, Mobile District
June 1984 Volume IV, Number 6

Soil bioengineering tested by District

By Winnie L. Smith

Designated reaches of riverbank and canal slopes on the Tennessee-Tombigbee Waterway at Aliceville, Aberdeen, and Pool A have undergone intensive erosion control measures this year in a program to test the effectiveness of soil bioengineering to control erosion and sedimentation.

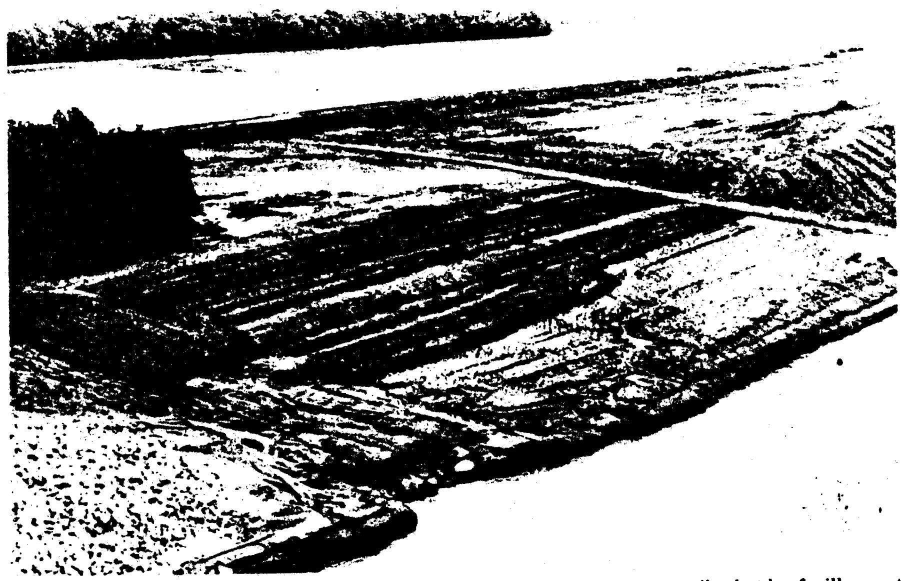
Soil bioengineering is relatively new in this country although it has been used extensively in Europe. Nothing of the magnitude of what the Mobile District is doing on the Tenn-Tom has been undertaken in the United States. However, some work on a smaller scale has been going on out west.

Soil bioengineering is a construction method which initially requires reshaping eroded slopes to a practical workable degree, controlling top of slope drainage, and using live plant material in conjunction with traditional structural materials. The District's test project has aroused interest in the South Atlantic Division (SAD). Representatives of the Savannah and Jacksonville Districts and from Memphis visited the area to see the work in progress in the spring.

More than 15 acres of slopes are involved in this test project. They include the left bank of the river below Aliceville in the Gainesville Pool, the left bank of the canal in Pool A, and the left bank of the river in the Columbus Pool below the Aberdeen Lock. The three areas were selected to determine effectiveness in several types of soil and for accessibility to contractors and Corps representatives making inspections.

The slopes were in bad condition and were selected to show what this system can do for a severely eroding river bank, with the idea that if it worked on the worst, it should work on others. Aliceville and Aberdeen sites are highly subject to flooding in the spring, while Pool A is not. Aberdeen had a massive amount of earthwork to be done to fill deep gullies, provide drainage swales and lay the slopes back before work to install the system could begin.

District personnel working on the project include: Ray Gustin and Beverly Winn, of Engineering Division's soil design section; Don Graham, project manager, of Engineering Division's civil works project management section; Edward J. Horder, Lynn Bradford and Carol Cromer, of Planning Division's recreation resources section; Joe Clark, Paul Perkins and Albert Lee of Construction Division's Aberdeen Resident Office. In addition, John Lambert, Engineering Divison, SAD, has been extremely supportive of the project and is a member of the erosion control task force for the Tenn-Tom.



View from Highway 45 bridge below Aberdeen Lock and Dam. Six tractor trailer loads of willow cuttings were used at this one site. Between riprap and vegetation is one of the areas that will be done in the fall.

In this particular test project hundreds of flatbed truckloads of black willow tree cuttings, branches, and poles were used. These ranged from 6 to 15 feet in length and were one to four inches in diameter. Some cottonwood, sycamore, river birch and red maples, among others, were also used, Horder said, but there were not nearly enough of these available to be significant.

In addition to the massive planting of tree cuttings, "We're using as many soil bioengineering systems as possible," Horder said. The result will be a varied amount of erosion control measures and a controlled situation for running cost evaluations on the different kinds of systems.

He outlined the various systems as follows:

- Live stakes-pieces of one-and-a-half to three-inch caliper willow trunks, about two feet long which are driven into the soil for about three-fourths of their lengths to root in place.
- Live cribwalls-wooden structures fastened together in a cage-like fashion and containing soil and live plant material, in layers throughout the structure.
- Live soft gabions-a system composed of filter fabric, gridded "Tensar" and willow bundles held together using the reinforced earth principle.
- Joint plantings-live stakes driven into riprap.
- Brush layers-long rows of branchy willow cuttings covered with soil except for the ends, which protrude into sunlight.
- Brush mattresses-live willow cuttings and branches laid on the ground in criss cross fashion, covered lightly with soil and held down with wire netting and live stakes.
- Facines-bundles of willow cuttings bound together and laid in trenches in various configurations on a slope.
- Wattles-same as facines except using longer trunks without leaving brushy tops intact.
- Lorenz drains-drainage swale covered with rocks one layer deep, choked with cobbles, and then provided with facines along the edges and live stakes at random.

The work is being done under a $964,000 cost reimbursable performance contract with Jack Larmour's Nursery of Caledonia, Miss., near Columbus, in partnership with Ron Ragland of Ragland Construction, Co., of Tupelo, Miss. The contract began the first week in March and continued to May 4.

Brush layering.

Beverly Winn checks slope seepage at base of a crib wall.

Live gabion after a flood.

Lorenz drain.

Consultant on the project is Robbin Sotir, president of Soil Bioengineering Corporation, Marietta, Ga.

Graham said the cooperation of Edward M. Slana of Procurement and Supply Division's procurement branch in working out the cost reimbursable contract has been beneficial in a number of ways. "This type of contract allows us to actually pin down the exact costs on every aspect of the project so that in future work we will have accurate financial data to use in our planning," he said.

The District expects some of the soil bioengineering systems to be less expensive than previous erosion control measures and cost effective through the expected value in reducing sedimentation, thus cutting down on the amount of dredging needed in the waterway.

"Most of the riverbanks can be expected to stabilize naturally," Gustin said, "however, some will need help, and bioengineering can be expected to play a significant role in providing bank stabilization.

"Several side benefits of bioengineering may result. Heavy streambank vegetation provides good cover and food sources for fish and wild life. Heavy root mats add stability to the bank slopes by tying the soil together and reducing the saturation level in the banks. Reduced water velocities adjacent to the bank reduce erosion during high water. Some types of grasses and other plants at the shoreline are effective in dissipating wave wash energy from river traffic. Additionally, one of the reasons bioengineering is so attractive is that it is labor intensive (little or no heavy equipment required), can be done in small increments, can be easily repaired, cleans up waterways by catching debris, and can be done by private individuals with very little cash outlay and with only minimal guidance. This type of program gets land owners to thinking about soil conservation measures not only as they may apply to a river bank but as they may apply on other portions of their land.

"It should be recognized that we in the District do not want to indicate that bioengineering is a panacea but rather is another tool that can possibly be used to control erosion," Gustin said.

Other work is planned in the fall.

Mr. John Lambert
Engineering Division SAD

404 221 - 6720

Tab 11

BIOTECHNICAL SLOPE PROTECTION AND EROSION CONTROL

Outline of presentation on GRADE STABILIZATION FOR EROSION CONTROL

William Weaver, geologist
1882 Archer Road
McKinleyville, CA 95521

September, 1984

GRADE STABILIZATION FOR EROSION CONTROL

I. Factors affecting erosion in stream channels.

II. Gully networks.

- A. Continuous and discontinuous gully systems.
- B. Gully development and growth.

III. Grade stabilization: controlling erosion in channels, small streams and adjacent hillslopes.

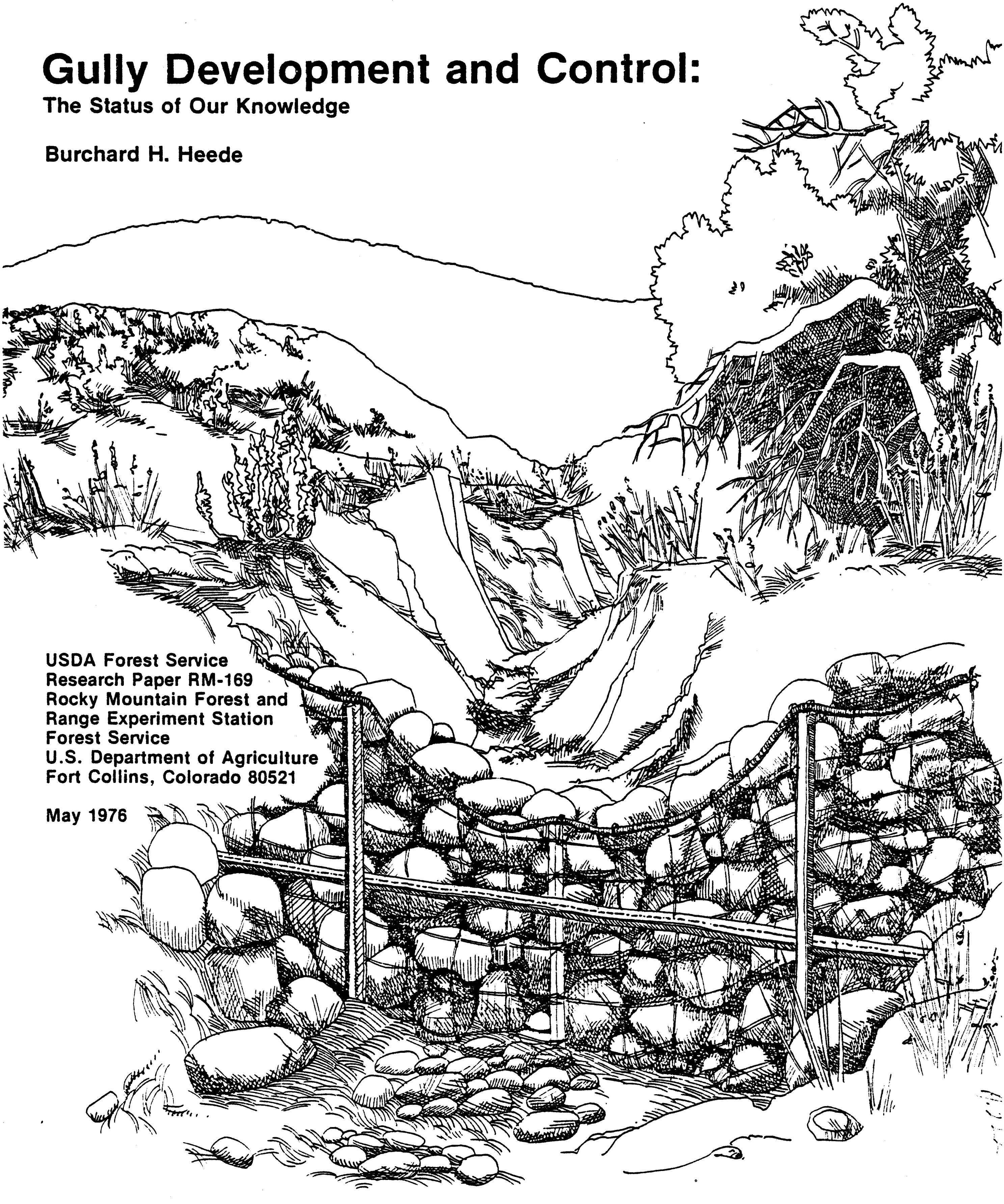
- A. Treatment of gully networks: prioritizing treatment sites.
- B. Single channel treatments.
  1. Primary stabilization practices
     - a. definition and advantages of primary practices
     - b. utilizing natural baselevels
     - c. use of heavy earth moving equipment for achieving stable grade
     - d. role of vegetation in grade stabilization
  2. Secondary techniques for stabilizing natural or artifical channel grades
     - a. purpose and function of secondary grade control measures
     - b. choice and design of stabilization systems and structures
     - c. techniques (checkdams, armor, other structures), applications, design criteria, costs, operation and maintenance

IV. Summary and conclusion.

Gully Development and Control:

The Status of Our Knowledge

Burchard H. Heede



USDA Forest Service
Research Paper RM-169
Rocky Mountain Forest and
Range Experiment Station
Forest Service
U.S. Department of Agriculture
Fort Collins, Colorado 80521

May 1976

Abstract

Heede, Burchard H.

1976. Gully development and control: The status of our knowledge. USDA For. Serv. Res. Pap. RM-169, 42 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. 80521

Gully formation is discussed in terms of mechanics, processes, morphology, and growth models. Design of gully controls should draw on our understanding of these aspects. Establishment of an effective vegetation cover is the long-term objective. Structures are often required. The least expensive, simply built structures are loose-rock check dams, usually constructed with single- or double-wire fences. Prefabricated concrete dams are also effective. Functional relationships between dams, sediment catch, and costs, as well as a critical review of construction procedures, should aid the land manager in design and installation of gully treatments.

Keywords: Gullies, erosion, geomorphology, erosion control, dams.

USDA Forest Service
Research Paper RM-169

May 1976

GULLY DEVELOPMENT AND CONTROL:
The Status of Our Knowledge

Burchard H. Heede,
Principal Hydraulic Engineer
Rocky Mountain Forest and Range Experiment Station[1]

[1]*Central headquarters is maintained at Fort Collins, in cooperation with Colorado State University; author is located at the Station's Research Work Unit at Tempe, in cooperation with Arizona State University.*

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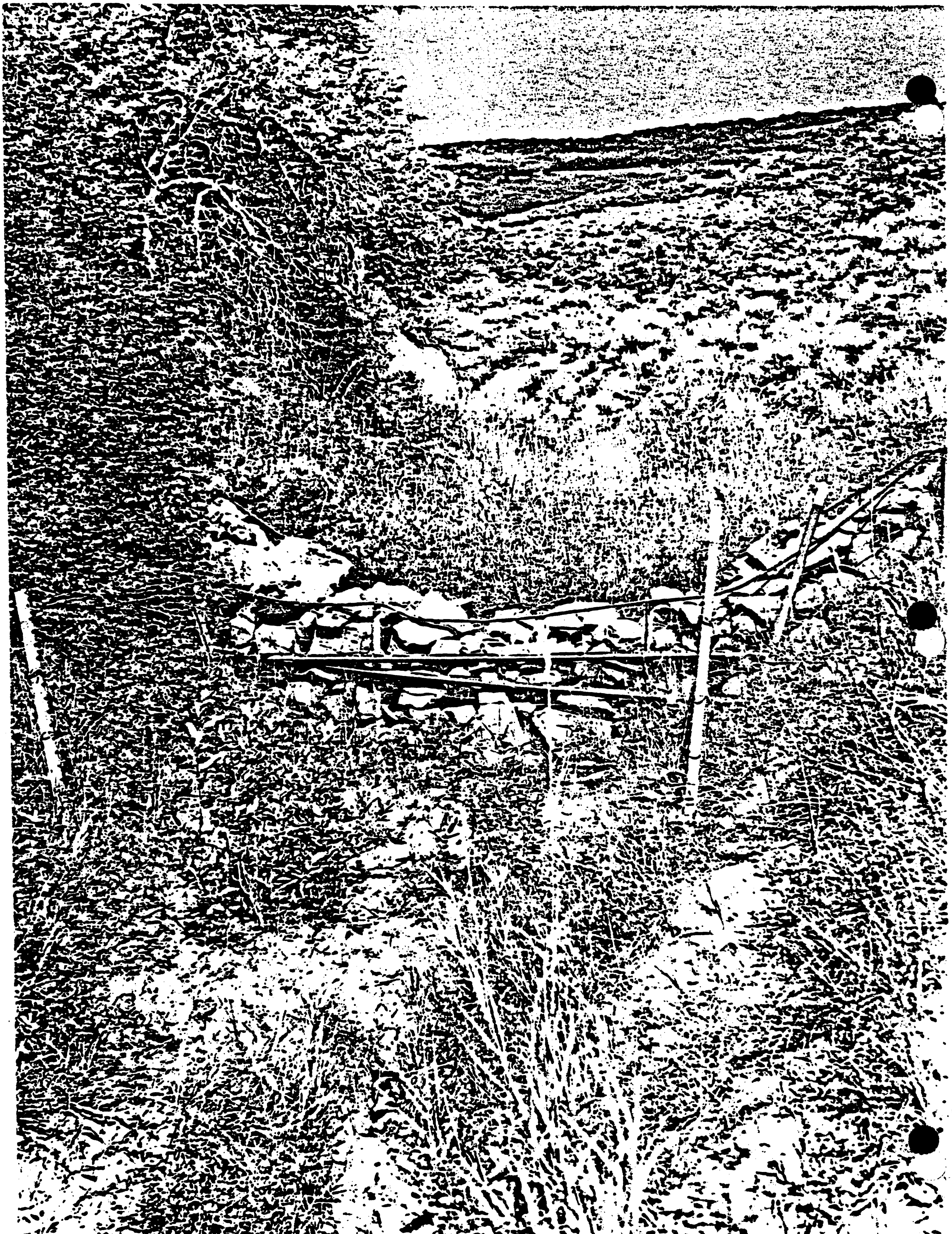


GULLY DEVELOPMENT AND CONTROL: The Status of Our Knowledge

Burchard H. Heede

HISTORICAL BACKGROUND

Early man was less mobile and more dependent on the surrounding land than his modern descendant. In many desert and semidesert regions, he not only learned to live with gullies, but utilized them for the collection of water and the production of food. Such desert agriculture was practiced in North Africa, Syria, Transjordan, southern Arabia, and North and South America. Thus, many areas in the world once supported more people than today. The Sierra Madre Occidental, Mexico, had a much higher population density 1,000 years ago than at present (Dennis and Griffin 1971), as did the Negev Desert in Israel (Evenari 1974).

Gully control on these ancient farms was not an end in itself, but a means for food production. Evenari et al. (1961) found well-defined "runoff farms" in the Negev Desert of Israel dating back to the Iron Age, 3,000 years ago. The climate, undoubtedly not different today, is characterized by an average yearly precipitation of 95 mm (3.7 inches), most of which falls in relatively small showers. Precipitation exceeds 10 mm (0.39 inch) on an average of only 2 days per year. Still, runoff farms, using check dams and water spreaders in wadis, gullies, and on hillsides, were able to support dense populations until the Negev was occupied by nomadic Bedouins after the Arab conquest in the 7th century A.D.

At least 900 years ago, the aborigines of the northern Sierra Madre Mountains of Chihuahua and Sonora, Mexico, developed an intensive field system by altering the natural environment with the help of trincheras (Herold 1965). Trincheras of the Sierran type—check dams built from loose rock—created field and garden plots within gullies and valleys by sediment accumulation, increased water storage within the deposits, and spread the flows on the deposits during storms. Similar but less developed systems were built sporadically in Arizona, New Mexico, and southwestern Colorado. About 1450 A.D. this flourishing agriculture disappeared.

With the age of industrialization, man lost his close dependence on the land. Population densities increased, land was fenced, and roads and communication systems mushroomed. This rapid change caused a different philosophy in the approach to gullies. Gullies were visualized as destroyers of lives and property, and as barriers to speedy communication. It is not surprising, therefore, that the first textbook on gullies or torrents, published in the 1860's in France, dealt with control only. Others followed quickly in Austria, Italy, Germany, and later in Japan.

It is not surprising that our knowledge on the mechanics of gullying is meager if we consider that, during the last 100 years, torrent and gully control were emphasized. Gully control research focused on engineering aspects—structural dimensions, types of structures, and adaptation of advances in civil engineering elsewhere. When in the middle 1950's interest was awakened in gully processes, efforts concentrated on mathematical and statistical, rather than physical, relationships.

The time has come to concentrate our efforts on understanding gully mechanics, and to reassess our philosophy on gully control. The objectives must be broadened beyond those of defense, and incorporate those of agricultural production, water yield, and environmental values. This task will not be easy, and in many cases tradeoffs will be required.

In areas of food shortage, the most pressing objective in gully control may be agricultural production. Food-short areas are often arid or semiarid, where gullies are the only streambeds supporting flow at times, and gully bottoms are closest to the low-lying water table. Gully flows as well as moisture storage both were utilized for plant growth by ancient man. Modern man may have to relearn the forgotten art of gully management in desert farming. This possibility is better for many developing countries; in highly industrialized countries, the present cost-price structure will seldom permit successful gully management for food production in deserts and mountain lands.

In the United States, however, gully management has been successfully practiced on agricultural lowlands at least since the 1930's, when conservation farming was introduced on a large scale. Farmers converted gullies into grassed waterways to serve the dual purpose of safe conveyance of surplus irrigation water and forage production. Often the Federal Government subsidized this work by extending technical and monetary help.

In contrast to agricultural lowlands, we know very little about gully management on mountain lands, where we have been mainly concerned with control. In the United States, a first approach to gully management on mountain slopes was Heede's (1968a) installation of vegetation-lined waterways in the Colorado Rocky Mountains. Converted areas lost 91 percent less soil than untreated gullies, and the unpalatable plant cover, consisting mainly of sagebrush, was changed to a palatable one, adding to the grazing resource.

In Italy, intensive hand labor, plowing, and manmade torrent streams reshaped gullied mountain slopes of the Apennines into gentle hillsides that could support pastures, vineyards, and orchards. The reshaping, called hydraulic reclamation (Heede 1965a), was justified by efforts to place Italian agriculture on a competitive basis when it would join the Common Market Community in the late 1960's.[2]

Modern check dam systems can also benefit water yield. Brown (1963) reported on the conversion of ephemeral flows to perennial streams below check dams. Heede[3] obtained perennial flow 7 years after installation of a check dam system where only ephemeral flow had occurred during the previous 50 years. It is postulated that this change is due to water storage in the sediment accumulations above the dams. Considerable vegetation develops within the gullies as well as on the watershed. Although this additional vegetation undoubtedly uses water, the evapotranspiration loss is more than offset by increased soil infiltration rates, resulting from vegetation cover improvement, which benefit soil water storage at times of high flows. The duration of significant flows increased, but total water yield did not.

[2]*Heede, Burchard H., 1962. A report on a visit of research stations, torrent control, and land reclamation projects in France, Italy and Austria. 73 p. (On file, Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)*

[3]*Heede, Burchard H. Evaluation of an early soil and water rehabilitation project—Alkali Creek watershed, Colorado. (Research Paper in preparation at Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)*

The environmental value of gullies is assessed differently by different people. To some, gullies may represent a typical landform of the Old West, a dear sentiment, adding to environmental quality. To them, gully control should be attempted only if needed to meet pressing land management objectives. To others, gullies may offer only an unsightly scene, and the conversion of raw gully walls into green stable slopes is a desirable goal. Our approaches to gully management must therefore remain flexible.

It is the objective of this paper to show progress and limits in our knowledge of gullies and their control, and thus to help the land manager achieve his goals.

SCOPE

This paper attempts to summarize the available body of knowledge and hypotheses on gully formation and control. As illustrated by the historical development of gully management, gully control currently comprises the larger body of knowledge. Of necessity, the discussion of gully control will be based mainly on works in the Colorado Rocky Mountains, where considerable effort has been invested since the work of the Civilian Conservation Corps in the 1930's.

Gully formation will be divided into three aspects: mechanics, processes and resulting morphology, and growth models. The individual aspects of gully formation must be considered not only by the control engineer, but also by the land manager who may decide not to interfere. If noninterference is the decision—and it will be in most cases—the consequences should be considered in the management plan to avoid future "surprises." Should gully management be planned for food or forage production, however, knowledge of these aspects of gullying will improve the design. Thus this report should be a helpful tool, whatever the land management decision may be.

GULLY FORMATION

Gullies develop in different vegetation types. In the West, gullies often develop in open ponderosa pine forests (fig. 1) or grasslands (fig. 2), the latter often heavily mixed with sagebrush (Heede 1970).

Gully development and processes have been studied by many investigators. A basic question raised was, why did gully cutting accelerate in the 1880's in the West, as documented for many locations? Schumm and Hadley (1957) argued that the sudden rapid development of gullies followed the sharp increase in cattle grazing around 1870. Leopold (1951) cited an additional influential factor—exceptionally frequent high-intensity storms at this time. Thus overgrazing may only have been the trigger. Yet Peterson (1950) stated that gully formation started in some locations before they were overgrazed, while other areas never experienced gully erosion after grazing. Other investigators stressed climatic change as the chief cause (Gregory 1917, Bryan 1925, Richardson 1945).

Neither the short-time climatological records, nor other approaches such as tree ring studies and pollen analysis, permit us to realistically assess the possible relationship between climatic change and gully cutting. I agree with Hastings (1959) that, recognizing the fragile condition of much western plant cover, any trigger effect could damage the cover to an extent where bare soil and runoff could increase drastically. Overgrazing and other land abuses such as poor road construction and location certainly were triggers. Once gully scarps formed, the development of gully networks was inevitable, because during the last half of the 19th century, the agricultural industry of the West was one of exploitation, not conservation.

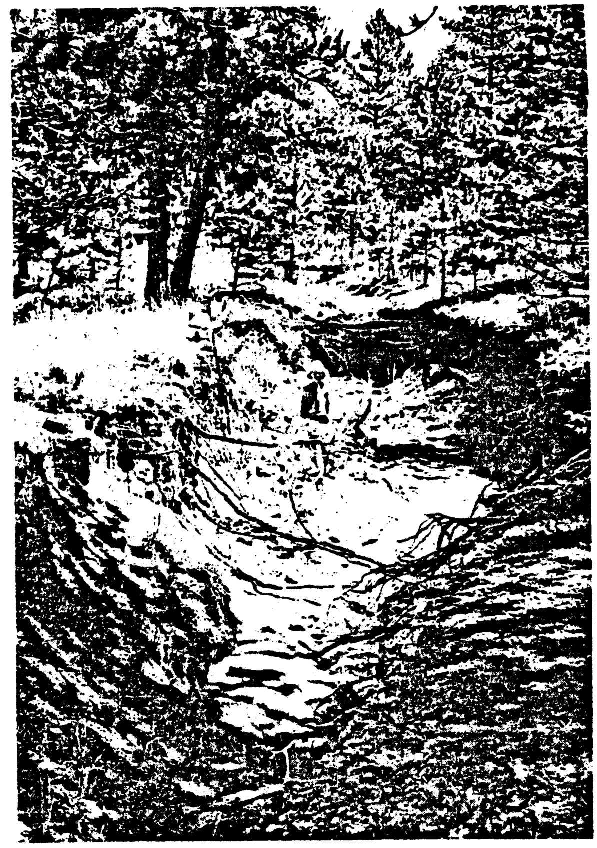


Figure 1.—This discontinuous gully advances through a ponderosa pine forest with an understory of grasses and other herbaceous vegetation. Location is the Manitou Experimental Forest on the eastern slope of the Rocky Mountains in the Colorado Front Range.

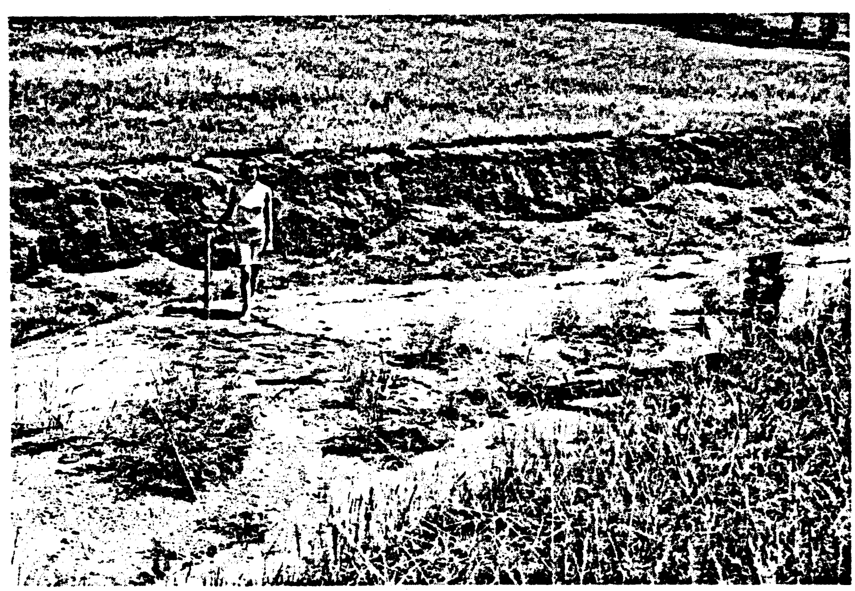


Figure 2.—This gully developed on a valley bottom covered by a fine stand of bunchgrasses on the Manitou Experimental Forest, Colorado Front Range. The view is across the reach close to the gully mouth.

Mechanics

Piest et al. (1973, 1975) deserve the credit for beginning gully mechanics investigations on agricultural croplands. Their studies showed that tractive force and stream power of the flow were not sufficient for a significant detachment of erodible loess soil overlying glacial till in the rolling countryside of western Iowa. Tractive force (τ) was defined as:

$$\tau = \gamma R_1 S_1 \quad (1)$$

where γ is the specific weight of the fluid, R_1 is the hydraulic radius, and S_1 represents the slope of the energy gradient. The investigators determined the stream power per unit length of gully (ω) by

$$\omega = \tau P V \quad (2)$$

where P is the wetted perimeter and V is the mean stream velocity. Since flow width (w) and wetted perimeter were approximately equal, w, the factor usually included in the equation, was substituted with P.

Calculations of unit stream power gave estimated values only, since the roughness coefficient (n) had to be estimated in the Manning's equation. Stage-discharge records as well as current meter measurements were used as checks, however. These calculations explain much of the "abnormal" behavior of flow and sediment relations observed by Heede (1964,[4] 1975a) and Piest et al. (1973, 1975): flow and sediment concentration in gullies are not necessarily related.

Concentration is related to the time since beginning of the particular flow event, however (table 1). During early flow, sediment concentrations and loads are high and then decrease with time until the easily available sediment derived from mass wasting processes within the gully has been removed. The last recession flows may be nearly clear water. This time-dependent characteristic of sediment concentration makes it possible that a high stream discharge may carry a much smaller load than a small one if the former occurs at a later date. Thus, if concentration is plotted over discharge, a hysteresis effect becomes visible.

Piest et al. (1975) stress that a sediment concentration parameter is usually a better erosion indicator than sediment discharge for testing erosion-causing variables. Two main reasons were given: (1) Sediment discharge is the product of flow and sediment concentration, which introduces a statistical bias into any relationship that may be runoff correlated; (2) runoff is not a basic variable, and would mask other, more basic variables since it usually is well correlated with the erosion condition of the watershed.

In the Iowa study, mass wasting of gully banks and headcuts were the prime erosion processes, not tractive force or stream power. Piest et al. (1973) found that height of water table, soil cohesive strength, and rate of water infiltration were the main factors controlling stability of gully banks. At Alkali Creek in western Colorado, where soils have up to 60 percent clay, mass wasting of gully banks takes place mainly during rainfalls that are sufficient to wet and thus change the cohesiveness of the banks, but insufficient to cause gully flows (fig. 3).

Processes and Morphology

Discontinuous Gullies

Leopold and Miller (1956) classified gullies as discontinuous or continuous. Discontinuous gullies may be found at any location on a hillslope. Their start is signified by an abrupt headcut. Normally, gully depth decreases rapidly downstream. A fan forms where the gully intersects the valley. Discontinuous gullies may occur singly or in a system of chains (Heede 1967) in which one gully follows the next downslope. These gullies may be incorporated into a continuous system either by fusion with a tributary, or may become a tributary to the continuous stream net themselves by a process similar to stream "capture." In the latter case, shifts on the alluvial fan cause the flow from a discontinuous gully to be diverted into a gully, falling over the gully bank. At this point, a headcut will develop that proceeds upstream into the discontinuous channel where it will form a nickpoint. Headward advance of the nickpoint will lead to gully deepening.

A chain of discontinuous gullies can be expected to fuse into a single continuous channel. Heede (1967) described the case history of such a fusion. Within three storm events of less than exceptional magnitude, the headcut of the downhill gully advanced 13 m to the next uphill channel, removing 70 m^3 of soil and forming one gully.

Vegetation types on the eastern and western flanks of the Colorado Rocky Mountains have not controlled the advance of headcuts of discon-

[4]Heede, Burchard H., 1964. A study to investigate gully-control measures on the Alkali Creek watershed, White River National Forest. Progress Report No. 3. 29 p. (On file at Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)

Table 1.--Suspended sediment samples from gully flows on Alkali Creek watershed, Colorado Rocky Mountains, 1964[1] and 1975[2]

Sampling station	Watershed area	Date	Flow: Average velocity	Flow: Discharge	Sediment: Concentration	Sediment: Discharge	Sand	Silt	Clay
	km^2		*m/s*	m^3/s	*p.p.m.*	*kg/s*	*Percent*		
		1964							
Gully 3	0.5	April 16	0.7	0.20	35,706	7.17	12.9	53.4	33.7
		April 29	.5	.02	12,402	.23	7.5	54.6	37.9
Main Gully A	2.8	April 14	.3	.23	20,766	4.81	1.5	57.9	40.6
		April 16	.4	.55	13,432	7.39	2.5	62.3	35.2
		April 28	.5	.19	4,499	.86	5.9	63.5	30.6
		May 26	.3	.02	19	.0003	--	--	--
Main Gully B	26.9	April 15	1.5	2.25	63,855	143.52	34.1	45.1	20.8
		April 16	2.0	3.04	35,134	106.59	51.4	24.9	23.7
		April 29	1.0	.99	4,628	4.58	17.3	73.8	8.9
		May 26	.4	.03	12	.0004	--	--	--
		1975							
Gully 3	0.5	April 25	.4	.06	2,775	.18	--	--	--
		April 26	.6	.04	1,255	.05	--	--	--
Main Gully A	2.8	April 24	.7	.19	2,377	.44	--	--	--
		April 26	.7	.25	932	.23	--	--	--
		April 28	.6	.04	178	.01	--	--	--

[1]The flow of 1964, caused by snowmelt, was preceded by a dry channel period of 1 year.
[2]Since 1971, the flows are perennial but decrease to magnitudes of less than 0.028 m^3/s by midsummer, except after intense rains. The 1975 flow was mainly snowmelt runoff.

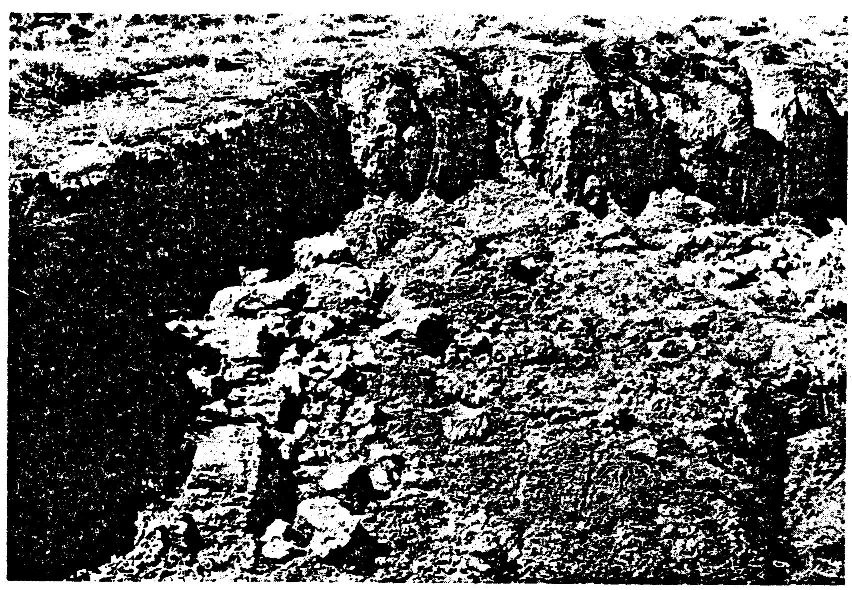


Figure 3.—Bank sloughing in gully on Alkali Creek watershed, western Colorado, during a period with no channel flow.

tinuous gullies (fig. 4). Ponderosa pine and Douglas-fir types, both with understory of grasses and other herbaceous vegetation, grew on the eastern flank; grass and sagebrush dominated the western flank. Since dense root mats of all these species occur at a depth below ground surface of only 0.3 to 0.6 m, undercutting by the waterfall over the headcut lip renders the mats ineffective.

Investigation of valley fill profiles and discontinuous gullies in Wyoming and New Mexico showed that discontinuous gullies formed on reaches of steeper gradient within a valley (Schumm and Hadley 1957). The authors postulated that overly steep gradients within alluviated valleys could be explained by deficiency of water in relation to sediment. In arid and semiarid areas, water losses along stream courses are well known (Murphey et al. 1972). The maintenance of stable alluvial streambeds is related to the quantity of water and the quantity and type of sediment moving through the system (Schumm 1969).

On the Alkali Creek watershed, evidence suggests that discontinuous gullies began to form at locations on the mountain slopes that were characterized by a break in slope gradient. This observation coincides with Schumm and Hadley's (1957) survey on valley floor and discontinuous gully profiles in Wyoming and New Mexico, and with Patton and Schumm's (1975) investigations of gullies in the oil-shale mountains of western Colorado. There, the breaks in valley gradients constituted a critical oversteepening of the valley slope. The oversteepening was the product of tributary streams that deposited large alluvial fans on the valley floor. Since flow data were not available, Patton and Schumm related the valley slope to drainage area. Discriminant-function analysis showed that, for areas larger than 10 km^2, a highly significant relationship existed between slope gradient, drainage area, and gullying. Discontinuous gullies occurred only above a critical slope value for a given area. The authors suggested that the results may be applicable only for the study region, since climate, vegetation and geology were considered constants. Yet for this particular region, the land manager obtained a valuable tool that tells him where discontinuous gullies may form.

The initiation of a discontinuous gully may also be explained by piping collapse (Hamilton 1970). Leopold et al. (1964) reported soil pipes to be an important element in the headward extension of this gully type. Since soil piping may be related to soil sodium, soil chemistry must also be regarded as a factor in gully formation, as demonstrated on the Alkali Creek watershed (Heede 1971). Piping soils (fig. 5), which caused gully widening and the formation of tributary gullies (fig. 6), had a significantly higher exchangeable sodium percentage (ESP) than nonpiping soils. The sodium decreased the layer permeability of the soils by 88 to 98 percent. Other prerequisites for the occurrence of pipes were low gypsum content, fine-textured soils with montmorillonite clay, and hydraulic head.

Older soil piping areas showed that extensive presence of pipes leads to a karstlike topography (fig. 7). The mechanical breakdown of the soils under such conditions facilitates leaching of the sodium from the soils, which in turn benefits plants. The new topography, characterized by more gentle gully side slopes compared with the former vertical walls of sodium soils, permits increased water infiltration, and natural rehabilitation of the gully by vegetation.

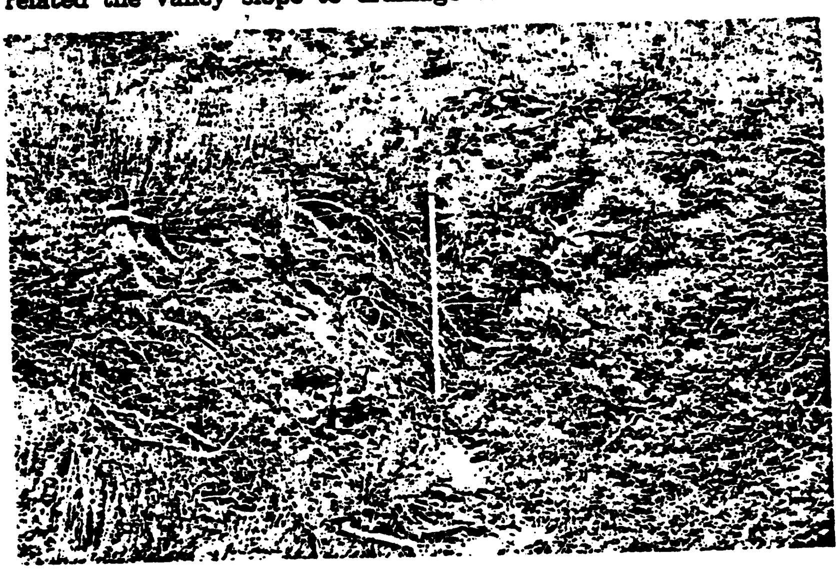


Figure 4.—Upstream view of headcut in gully 4, Alkali Creek watershed, before treatment. Length of rod is 1.7 m.

Figure 5.— Soil pipes on the Alkali Creek watershed drain runoff into the gully.

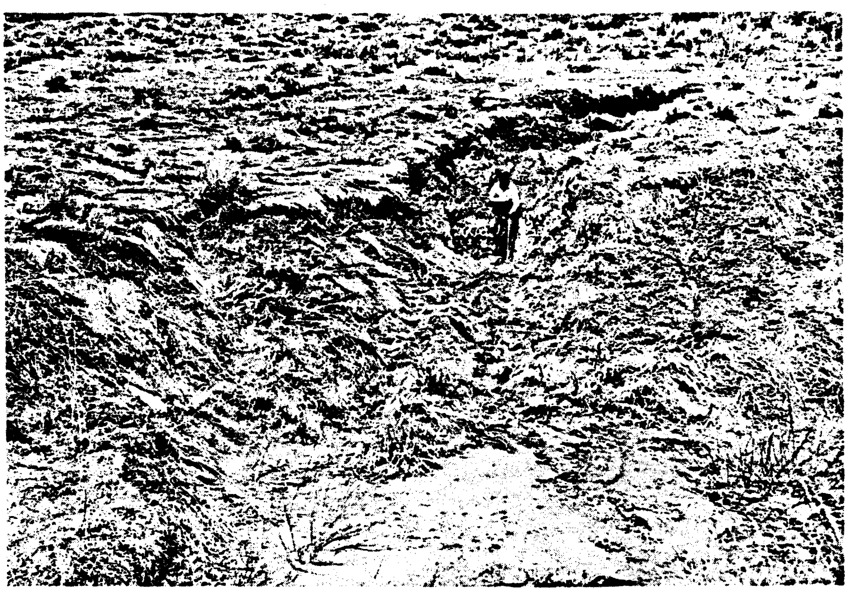


Figure 6.—This tributary to the main gully of Alkali Creek developed after the roof of the soil pipe collapsed.

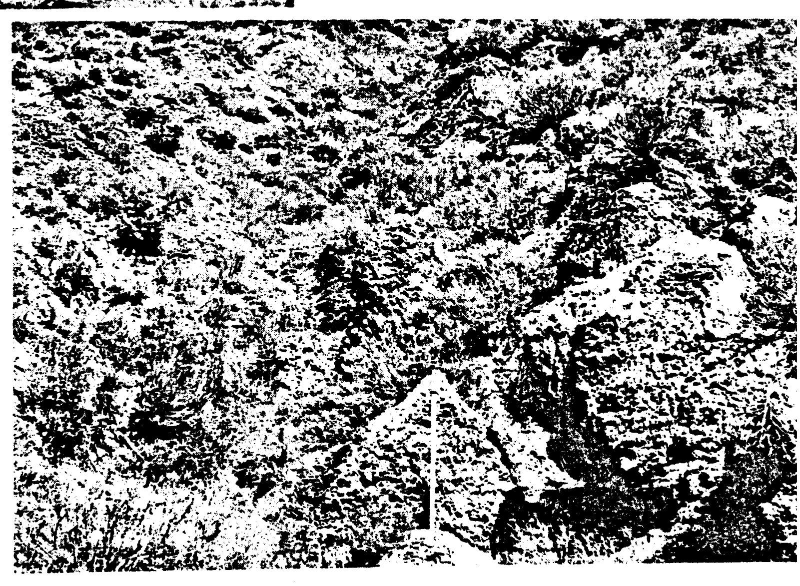


Figure 7.— After the collapse of the soil pipes, lining several reaches of gullies on the Alkali Creek watershed, the vertical gully walls have begun to break down. The resultant topography begins to resemble that of a karst surface of mature to old-age stage.

Continuous Gullies

The continuous gully begins with many fingerlike extensions into the headwater area. It gains depth rapidly in the downstream direction, and maintains approximately this depth to the gully mouth. Continuous gullies nearly always form systems (stream nets). They are found in different vegetation types, but are prominent in the semiarid and arid regions. It appears that localized or regional depletion of any vegetation cover can lead to gully formation and gully stream nets, if other factors such as topography and soils are conducive to gully initiation. Several studies have demonstrated, however, that vegetation and soil type predominantly influence the morphology of gullies.

Schumm (1960) found that, in western channels, the type of material in banks and bottoms controls the cross-sectional channel shape. When the mechanical analysis of the soils was related to the width-depth ratio (upper width versus mean depth), linear regression indicated that increases in the ratio conformed with the increases of the average percent sand in the measured load. This relationship was also established by the Soil Conservation Service at Chickasha, Oklahoma (unpublished report). On Alkali Creek, where extensive sampling showed no significant differences in the texture of the soils, meaningful correlations between the width-depth ratio and thalweg length could not be established (Heede 1970).

Tuan (1966) reported, in a critical review of literature on gullies in New Mexico, that channels developed in a semiarid upland environment were of moderate depth, and cut into sandy alluvium. Deep trenches were rare. The influence of sand in gully bank material on sediment production was shown when upland gullies were studied in the loess hills of Mississippi (Miller et al. 1962). They found that the annual volume of sediment produced ranged from 0.091 to 0.425 m^3 per hectare of exposed gully surface. The lower rate was associated with an average 6-m vertical gully wall having a low percentage of uncemented sand, while the higher rate was found in gullies with 12 m vertical walls and a high percentage of uncemented sand. As illustrated by gully erosion on the Alkali Creek watershed, sediment production may also be related to the chemical composition of the soils (Heede 1971).

That vegetation surrounding a gully may exert stronger influences on the channel morphology than the soils was shown for small streams in northern Vermont (Zimmerman et al. 1967), and also for a large gully in California (Orme and Bailey 1971). The California gully occupied a 354-ha watershed in the San Gabriel Mountains. In an experiment to increase water yield, the riparian woodland was removed and replaced by grasses. Two years later, a wildfire destroyed all vegetation on the watershed, and 57 ha of side slopes (16 percent of total area) were seeded to grass. The vegetative conversion was maintained by aerial sprays with selective herbicides. When high-intensity storms hit the watershed 6 and 9 years after conversion, stream discharge rates and sediment loads increased to previously unknown magnitudes. Changes in longitudinal profile and channel cross sections were spectacular. The gully "survived" as a relict feature partially clogged with storm debris, but had not regained its hydraulic efficiency by mid-1971.

In the Vermont streams, encroachment and disturbance by vegetation eliminated the geomorphic effect of channel width increase in the downstream direction, a normal stream behavior. In contrast, on the Alkali Creek watershed where gullies did not experience severe encroachment or disturbance by the sagebrush-grass cover, this geomorphic effect was eliminated by local rock outcrops and soils with low permeability due to high sodium (Heede 1970).

To establish gully morphology and possible stages of gully development, Heede (1974) analyzed the hydraulic geometry of 17 Alkali Creek gullies. Stream order analysis showed that 67 percent of the area of a fourth-order basin was drained by first-order streams. This is contrasted to 1 percent, the average for similar river basins in the United States (Leopold et al. 1964). Since the Alkali Creek gully system is still in the process of enlargement toward headwaters, the drainage area of the first-order streams will decrease with time. The longitudinal profiles of the gullies exhibited weak concavities, and it was argued that concavity would increase with future gully development.

The shape factor of the gullies, relating maximum to mean depth and expressing channel shape, had relatively high values (average 2.0). These values represent cross sections with large wetted perimeters that in turn indicate hydraulic inefficiency of the gullies.

The tested hydraulic parameters—drainage net, profile, and shape factor—were interpreted as indicating juvenile stages of gully development (termed youthful and early mature). Thus it can be argued that gully development should be recognized in terms of landform evolution, proceeding from young to old age stages. If stages of development could be expressed in terms of erosion rates and sediment yields, a useful tool would be provided for the watershed manager.

When the hydraulic geometry of the gullies was compared with that of rivers, it was suggested that the mature gully stage should be characterized by dynamic equilibrium. The condition of dynamic equilibrium does not represent a true balance between the opposing forces, but includes the capability to adjust to changes in short timespans, and thus regain equilibirum (Heede 1975b). Although some gullies of the Alkali Creek watershed approached this condition, it must be realized that in ephemeral gullies, a mature stage may not be defined by stream equilibrium alone, but may include other aspects of stability such as channel vegetation. Invasion of vegetation into the gully is stimulated during dry channel periods.

During the youthful stage, gully processes proceed toward the attainment of dynamic equilibrium, while in the old age stage, a gully loses the characteristics for which it is named, and resembles a river or "normal" stream. Gully development may not end with old age, however. Environmental changes such as induced by new land use (Nir and Klein 1974) and climatic fluctuation or uplift, may lead to rejuvenation, throwing the gully back into the youthful stage.

The condition of steady state, representing true equilibrium, is a theoretical one and can hardly be conceived to apply to gully systems, with the possible exception of very short timespans. Schumm and Lichty (1965) expressed a similar view when they stated that only certain components of a drainage basin may be in steady state.

We must also recognize that gully development is not necessarily an "orderly" process, proceeding from one condition to the next "advanced" one. Erosion processes accelerate at certain times, and at others apparently stand still. For example, Harris (1959) established four epicycles of erosion during the last 8,000 years for Boxelder Creek in northern Colorado. During the interims, the stream was in dynamic equilibrium most of the time. In a case study on ephemeral gullies, it was demonstrated that flows alter the channel, at times leaving a more stable, at others a very unstable, condition (Heede 1967). The latter internal condition leads to the well-known explosive behavior of geomorphic systems (Thornes 1974). External events, however, such as flooding in natural streams, may also lead to rapid, drastic changes (Schumm and Lichty 1963).

Growth Models

At present, no physical formula or model is available that describes the advancement of gullies, although several statistical models have been devised. In the badlands of southern Israel, which are severely dissected by gullies, field data were statistically analyzed and a simple model for gully advance established (Seginer 1966). Seginer tested three geometric parameters of the watershed that can easily be measured: watershed area, length of watershed along the main depression, and maximum elevation difference in the watershed. Of course, these parameters are interrelated. Regression analyses for several combinations indicated that watershed area was the most important single factor explaining the deviations about the mean; additional factors did not supply more information.

The prediction equation derived was as follows:

$$E = C_1A^{0.50} \tag{3}$$

where E is the advancement rate of the gully headcut, A is the watershed area draining into the headcut, and C_1 is a constant that varies from watershed to watershed.

It is obvious that a simplified approach to the quantification of gully processes, such as described above, at best presents empirical relationships valid for a given watershed at a given point in time. Assumptions of uniform distribution of rainfall (expressed by watershed area), uniform geology, soils, and vegetation, unchanged land uses, to name just a few, do not permit formulation of meaningful predictions.

The limitation of prediction equations based on statistical relations of a few selected parameters and factors was also illustrated by other studies. Thompson (1964) investigated the quantitative effect of independent watershed variables on rate of gully-head advancement. Variables were: drainage area above the gully head, slope of approach channel above the gully head, summation of rainfall from 24-hour rains equal to or greater than 13 mm, and a soil factor—the approximate clay content (0.005 mm or smaller) of the soil profile through which the head cut is advancing. Regression analysis showed that 77 percent of the variance was explained by the four variables. The t-test indicated that only drainage area, precipitation, and soils were highly significant in the regression equation at the 5 percent level to express the rate of headcut advancement. An R^2 value of 0.77 appears to signify an efficient relationship, yet about one-fourth of the variance is due to other, not measured variables. This unexplained fourth will prohibit the use of the prediction equation for most projects.

While Thompson (1964) chose the linear advancement of gully headcuts, Beer and Johnson (1963) selected the changes in gully surface area as the dependent variable. In addition to the independent variables used in the 1964 study, Beer and Johnson included an estimate of an index of surface runoff. The results showed that the gullying process was best represented by a logarithmic model, as contrasted with Thompson's linear model. All variables were evaluated from the past growth of the gullies. No controlled studies of the individual components responsible for the gullying process have been made.

The above-mentioned statistical investigations threw light on the important variables in gully growth, and thus added to our understanding of gullying. But quantification and prediction of growth still lack precision because past rates of gullying do not necessarily indicate future rates. Stages can be recognized in the development of gullies, and erosion and sediment production change between the stages (Heede 1974). Gully growth predictions without recognition of stage development may not be meaningful.

A deterministic growth model for gullies was proposed based on investigations in the badlands of S.E. Alberta, Canada, where climate, lithology, and total available relief are uniform (Faulkner 1974). Vegetation is practically absent. The constraints on the model are quite drastic in view of the variability of environments supporting gully systems. The model is an extension of Woldenberg's (1966) gradient derived from the allometric growth law (Huxley 1954), defined as

$$x = c_1 y^{d^1} \qquad (4)$$

where x is the size of an organ, y represents the size of the organism to which the organ belongs, and c_1 and d^1 are constants.

Usually, nonuniformity of environmental factors such as soils and vegetation is the rule. The intermittent flow of ephemeral gullies adds another formidable task in making the present law sufficiently flexible to take care of the numerous field combinations. For most situations, the present model will therefore not yield results useful to the land manager.

The above compendium illustrates that our knowledge on gully mechanics and processes is limited. As we will recognize in the following chapters, art and judgment are still required in many phases of gully control.

OBJECTIVES IN GULLY CONTROL

Main Processes of Gully Erosion As Related To Control

The mechanics of gully erosion can be reduced to two main processes: downcutting and headcutting. Downcutting of the gully bottom leads to gully deepening and widening. Headcutting extends the channel into ungullied headwater areas, and increases the stream net and its density by developing tributaries. Thus, effective gully control must stabilize both the channel gradient and channel headcuts.

Long-Term Objective of Controls—Vegetation

In gully control, it is of benefit to recognize long- and short-term objectives because often it is very difficult or impossible to reach the long-term goal—vegetation—directly; gully conditions must be altered first. Required alterations are the immediate objectives.

Where an effective vegetation cover will grow, gradients may be controlled by the establishment of plants without supplemental mechanical measures. Only rarely can vegetation alone stabilize headcuts, however, because of the concentrated forces of flow at these locations. The most effective cover in gullies is characterized by great plant density, deep and dense root systems, and low plant height. Long, flexible plants, on the other hand, such as certain tall grasses, lie down on the gully bottom under impact of flow. They provide a smooth interface between flow and original bed, and may substantially increase flow velocities. These higher velocities may endanger meandering gully banks and, in spite of bottom protection, widen the gully. Trees, especially if grown beyond sapling stage, may restrict the flow and cause diversion against the bank. Where such restrictions are concentrated, the flows may leave the gully. This is very undesirable because, in many cases, new gullies develop and new headcuts form where the flow reenters the original channel.

Engineers' Measures—An Aid to Vegetation Recovery

If growing conditions do not permit the direct establishment of vegetation (due to climatic or site restrictions, or to severity of gully erosion) engineering measures will be required. These measures are nearly always required at the critical locations where channel changes invariably take place. Examples are nickpoints on the gully bed, headcuts, and gully reaches close to the gully mouth where deepening, widening, and deposition alternate frequently with different flows (see fig. 2). Nickpoints signify longitudinal gradient changes; a gentler gradient is being extended toward headwaters by headcutting on the bed (fig. 8). Normally, critical locations are easily definable since the active stage of erosion at these sites leaves bed and banks in a raw, disturbed condition.

The designer must keep in mind that well-established vegetation perpetuates itself and thus represents a permanent type of control. In contrast, engineering measures always require some degree of maintenance. Because maintenance costs time and money, projects should be planned so that maintenance is not required indefinitely.

An effective engineering design must help establish and rehabilitate vegetation. Revegetation of a site can be aided in different ways. If the gully gradient is stabilized, vegetation can become established on the bed. Stabilized gully bottoms will make possible the stabilization of banks, since the toe of the gully side slopes is at rest (fig. 9). This process can be speeded up mechanically by sloughing gully banks where steep banks would prevent vegetation establishment. Banks should be sloughed only after the bottom is stable, however.

Vegetation rehabilitation is also speeded if large and deep deposits of sediment accumulate in the gully above engineering works. Such alluvial deposits make excellent aquifers, increase channel storage capacity, decrease channel gradients, and thus, decrease peak flows. Channel deposits may also raise the water table on the land outside the gully. They may reactivate dried-up springs, or may convert ephemeral

Figure 8.—The nickpoint, located on the gully bottom and indicated by a survey rod, has a depth of about 0.5 m. Although this gully appears to be stabilized by the invasion of vegetation, rejuvenation must be expected by the upstream advance of the nickpoint. The root systems will be undercut and gully depth and width will increase. Length of the rod is 1.7 m.

Figure 9.—The bank of Main gully, Alkali Creek watershed, 12 years after installation of check dams. Stabilization of the gully bottom made possible the invasion of dense vegetation that now is creeping up the bank. The man stands at the toe of the bank.

springs to perennial flow. All these results create conditions much more favorable to plant growth than those existing before control.

Watershed Restoration Aids Gully Control Measures

Measures taken outside the channel can also aid revegetation processes in the gully. Improvements on the watershed that (1) increase infiltration and decrease overland flow, and (2) spread instead of concentrate this flow, will benefit gully healing processes. A study on sediment control measures showed that sediment yields were reduced 25 to 60 percent by land treatment and land use adjustments, as surveyed at 15- to 20-year-old flood-water-retarding structures in the southern Great Plains (Renfro 1972). But when combined land treatment and structural measures were applied, sediment yields were reduced 60 to 75 percent.

Normally, however, gully improvements can be attained quicker within the gully than outside, because of concentration of treatment and availability of higher soil moisture in the defined channel.

Many types of watershed restoration measures have been devised, and the literature on the subject is abundant: Poncet (1965) described an integrated approach to erosion control on the watershed and in gullies; Copeland (1960) presented a photo-record of watershed slope stabilization in the Wasatch Mountains of Utah; and Bailey and Copeland (1961) analyzed the behavior of slope stabilization structures.

Since watershed restoration measures are only supplemental to gully control, some examples will suffice here: seeding and planting with and without land preparation and fertilization; vegetation cover conversions; and engineering works such as reservoirs, water diversions, benches, terraces, trenches, and furrows.

Immediate Objectives of Control

Different types of measures benefit plants in different ways. It is therefore important to clarify the type of help vegetation establishment requires most. Questions should be answered such as: Is the present moisture regime of the gully bottom sufficient to support plants, or should the bottom be raised to increase moisture availability? One must recognize that a continuous, even raising of the bottom is not possible. Due to the processes of sedimentation above check dams, deposits have a wedge-shaped cross section if plotted along the thalweg.

The immediate objectives of a gully treatment must consider other aspects in addition to plant cover. Usually, these considerations involve hydraulics, sedimentation, soils, and sometimes the logistics required for the management of the watershed. For instance, management may call for deposits of maximum possible depth at strategic locations to provide shallow gully crossings. Thus, if sediment catch is a desirable objective, large dams should be built. But if esthetic considerations make check dams undesirable (and watershed logistics and revegetation offer no problems), the gully bottom may be stabilized with dams submerged into the bed, and thus invisible to the casual observer.

These examples illustrate how important it is to clarify the immediate and overall objectives of a planned treatment before deciding on approaches and measures. The objectives determine the measures; the measures, the type of result.

GULLY CONTROL STRUCTURES AND SYSTEMS

Types of Porous Check Dams

The most commonly applied engineering measure is the check dam. Forces acting on a check dam depend on design and type of construction material. Nonporous dams with no weep holes, such as those built from concrete (Poncet 1963, Heede 1965b, Kronfellner-Kraus 1971), sheet steel, wet masonry, and fiberglass, receive a strong impact from the dynamic and hydrostatic forces of the flow (fig. 10). These forces require strong anchoring of the dam into the gully banks, to which most of the pressure is transmitted. In contrast, porous dams release part of the flow through the structure, and thereby decrease the head of flow over the spillway and the dynamic and hydrostatic forces against the dam (fig. 11). Much less pressure is received at the banks than with nonporous dams. Since gullies generally are eroded from relatively soft soils, it is easier to design effective porous check dams than nonporous ones. Once the catch basin of either porous or nonporous dams is filled by sediment deposits, however, structural stability is less critical because the dam crest has become a new level of the upstream gully floor.

Loose rock can be used in different types of check dams. Dams may be built of loose rock only, or the rock may be reinforced by wire mesh, steel posts, or other materials. The reinforce-

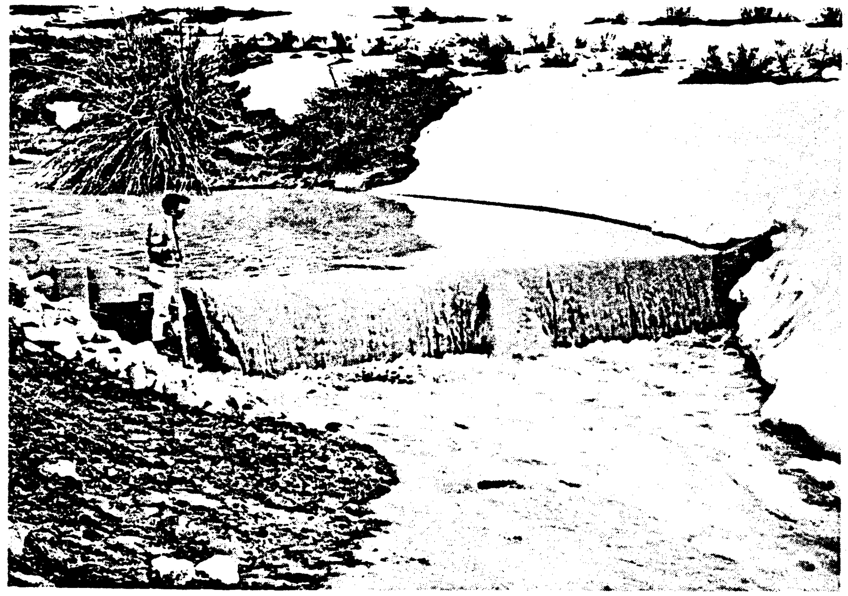


Figure 10.—This prefabricated, prestressed concrete check dam accumulated sediment readily because the structure is not porous. At the same time, dynamic and hydrostatic forces of the flow on the dam are much stronger than those at a porous rock check dam. The discharge over the spillway of this structure, installed on the Alkali Creek watershed, is about 0.4 m^3/s.

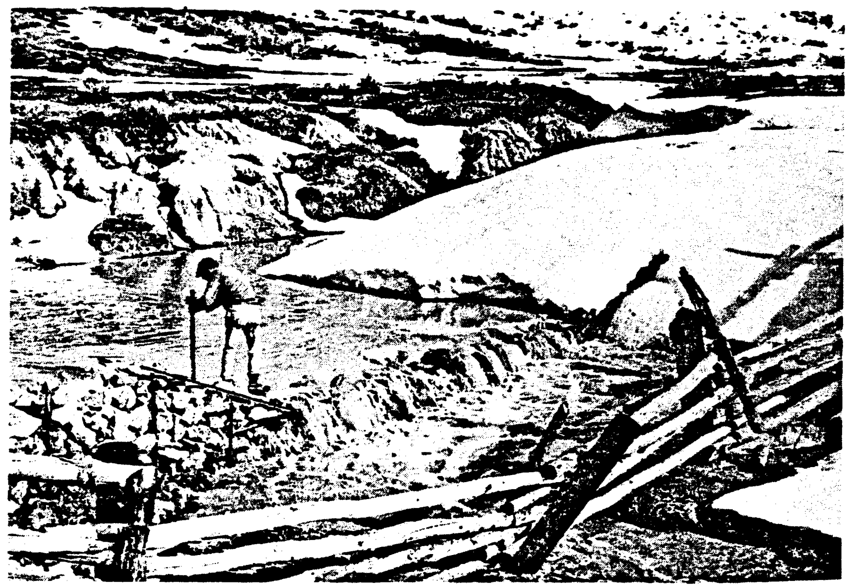


Figure 11.—As contrasted to impervious dams, rock check dams such as this double-fence structure release much of the flow, and hence hydrostatic pressure, through the structure.

ments may influence rock size requirements. If wire mesh with small openings is used, rocks may be smaller than otherwise required by the design flow.

Some different types of check dams will be described, but the field of check dam design is wide open. Many variations are possible. The torrent-control engineers of Europe have been especially successful with filter or open dams. Most of their designs are for large torrents where stresses on the structures are much greater than those in gullies, generally. Clauzel and Poncet (1963) developed a concrete dam whose spillway is a concrete chute with a steel grid as the chute bottom. This grid acts as a filter for the bedload. Periodic cleaning of the dam is required, however.

Other types of filter dams have vertical grids, or grids installed at an angle to the vertical. Such dams are described by Puglisi (1967), Kronfellner-Kraus (1970), and Fattorelli (1971).

All the torrent control dams are quite sophisticated, and thus costly. Such high costs are often justified in Europe, however, since population densities require the most effective and lasting control measures. These qualities are especially important if the basic geologic instability of the alpine torrents is considered. In contrast, most gullies in the western United States are caused by soil failure, and life and high-cost property are not usually endangered. Simpler, low-cost structures will therefore be preferable. Some of the most effective and inexpensive dams are built mainly from loose rock. They will, therefore, be emphasized in the descriptions that follow.

Loose Rock

The basic design of a loose-rock check dam is illustrated in figure 12. If facilities are not available to use the computer program developed by Heede and Mufich (1974), volumes of excavation and of rocks required in the construction can be calculated from the drawings. Rock volumes can also be obtained from an equation discussed in the section on Equations for Volume Calculations. In a Colorado project, the drawings also served well in the field as construction plans (Heede 1966).

Since loose-rock dams are not reinforced, the angle of rest of the rock should determine the slopes of the dam sides. This angle depends on the type of rock, the weight, size, and shape of the individual rocks, and their size distribution. If the dam sides are constructed at an angle steeper than that of rest, the structure will be unstable and may lose its shape during the first heavy runoff. For the design of check dams, the following rule of thumb can be used: the angle of rest for angular rock corresponds to a slope ratio of 1.25 to 1.00; for round rock, 1.50 to 1.00. Figure 13 illustrates a dam built from angular loose rock.

Wire-Bound Loose Rock

A wire-bound check dam is identical in shape to that of a loose-rock dam, but the loose rock is enclosed in wire mesh to reinforce the structure. The flexibility within the wire mesh is sufficient to permit adjustments in the structural shape, if the dam sides are not initially sloped to the angle of rest. Therefore, the same rock design criteria are required for a wire-bound dam as for a loose-rock structure.

The wire mesh should: (1) be resistant to corrosion, (2) be of sufficient strength to withstand the pressure exerted by flow and rocks, and (3) have openings not larger than the average rock size in the dam. Wire mesh may not be effective in boulder-strewn gullies supporting flows with heavy, coarse loads.

Single Fence

Single-fence rock check dams (figs. 14, 15) differ greatly in shape and requirements of construction materials from the loose-rock and wire-bound dams. These structures consist of (1) a wire-mesh fence, fastened to steel fenceposts and strung at right angles across the gully, and (2) a loose-rock fill, piled from upstream against the fence. The rock fill can be constructed at an angle steeper than that of rest for two reasons:

1. The impact of flows will tend to push the individual rock into the fill and against the dam.
2. Sediment deposits will add stability to the fill and will eventually cover it.

The design of this type of check dam should emphasize specifications for the wire mesh, and the setting, spacing, and securing of the steel fenceposts. The wire mesh specifications will be the same as those for the wire-bound dams.

The steel fenceposts should be sufficiently strong to resist the pressure of the rock fill and the flows, and must be driven into the gully bot-

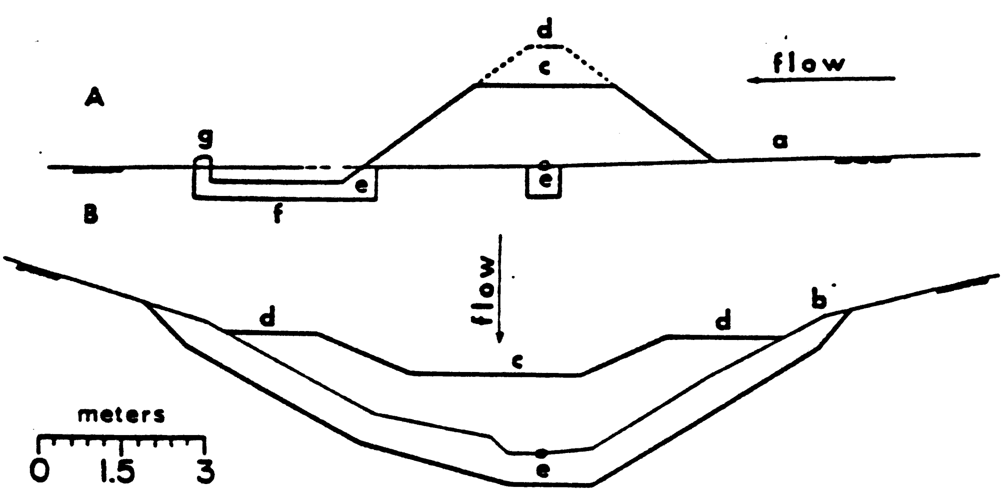
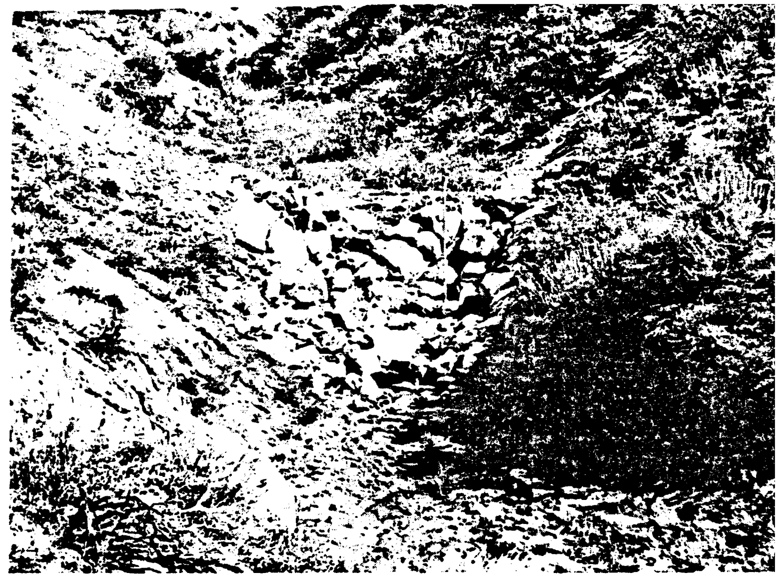


Figure 12.—Construction plans for a loose-rock check dam. *A*, Section of the dam parallel to the centerline of the gully. *B*, Section of the dam at the cross section of the gully. a = original gully bottom; b = original gully cross section; c = spillway; d = crest of freeboard; e = excavation for apron; g = end sill.

Figure 13.—Upstream view of a loose-rock check dam. The catchment basin filled with sediment during the first spring runoff after construction. Rod is 1.7 m high.



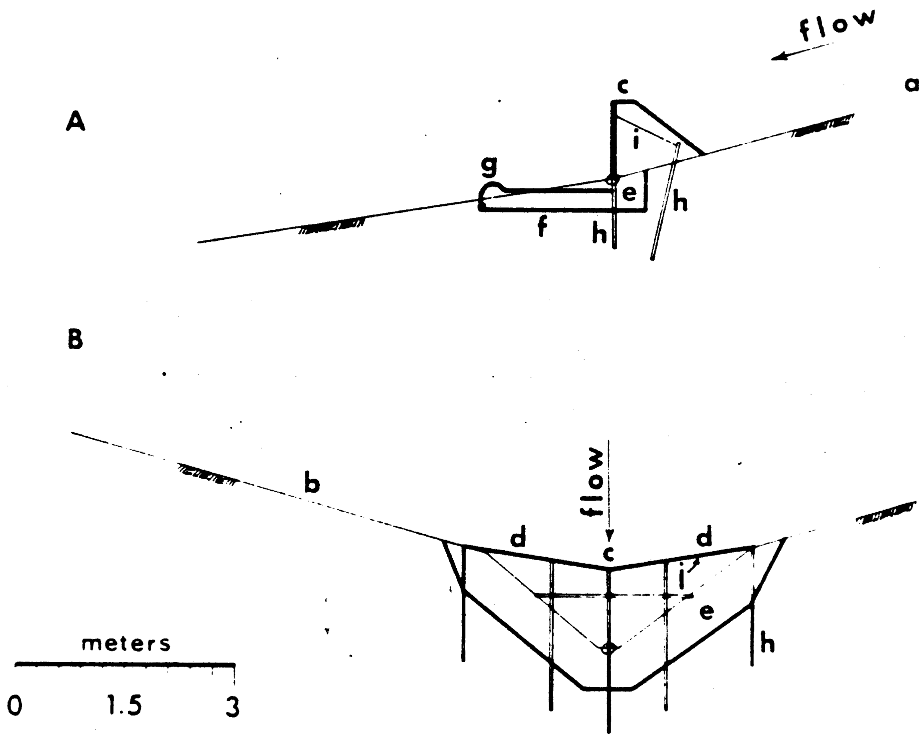
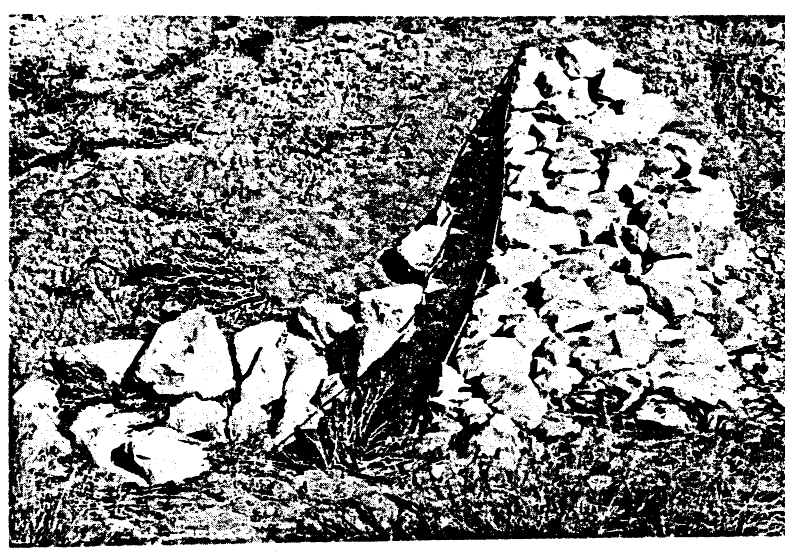


Figure 14.—Construction plans for a single-fence rock check dam.
A, Section of the dam parallel to the centerline of the gully.
B, Section of the dam at the cross section of the gully. a = original gully bottom; b = original gully cross section; c = spillway; d = crest of freeboard; e = excavation for key; f = excavation for apron; g = end sill; h = steel fencepost; k = guys; j = rebar, 13 mm in diameter.

Figure 15.—View across a single-fence dam. Apron and gully bank protection are to the left of the dam crest.



tom and side slopes to a depth that insures their stability in saturated soil. If it is impractical to drive posts to sufficient depths, the stability of the posts should be enhanced by guys. These guys should be anchored to other posts that will be covered and thus held in place by the rock fill.

In general, spacing between the fenceposts should not be more than 1.2 m to prevent excessive pouching (stretching) of the wire mesh. Where conditions do not allow this spacing, a maximum of 1.5 m can be used but the fence must be reinforced by steel posts fastened horizontally between the vertical posts. Excessive pouching of the wire mesh reduces the structural height and impairs the stability of the dam.

Double Fence

The double-fence rock check dam has two wire mesh fences, strung at a distance from each other across the channel (fig. 16). In this type of dam, a well-graded supply of rocks is essential, otherwise the relative thinness of the structure would permit rapid throughflow, resulting in water jets. Double-fence dams should only be built if an effective rock gradation can be obtained.

In Colorado, parallel fences were spaced 0.6 m (Heede 1966). Peak flows did not exceed 0.7 m^3/s, and loads consisted mainly of finer material. Dams were no taller than 1.8 m (fig. 17). At many dam sites, maintenance and repairs

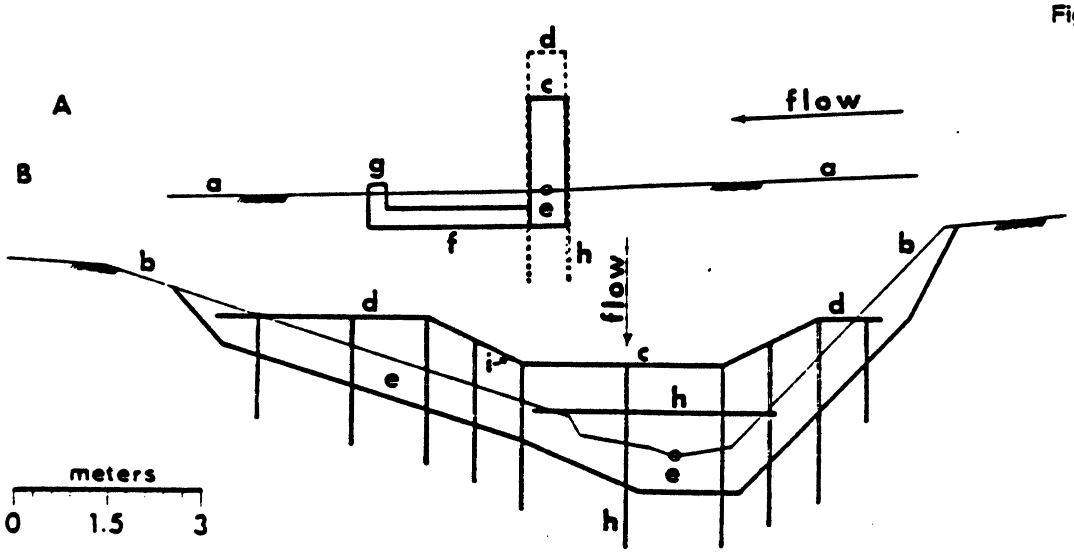


Figure 16.—Construction plans for a double-fence rock check dam.
A, Section of the dam parallel to the centerline of the gully.
B, Section of the dam at the cross section of the gully.
a = original gully bottom; b = original gully cross section; c = spillway; d = crest of freeboard; e = excavation for key; f = excavation for apron; g = end sill; h = steel fencepost; i = rebar, 13 mm in diameter.

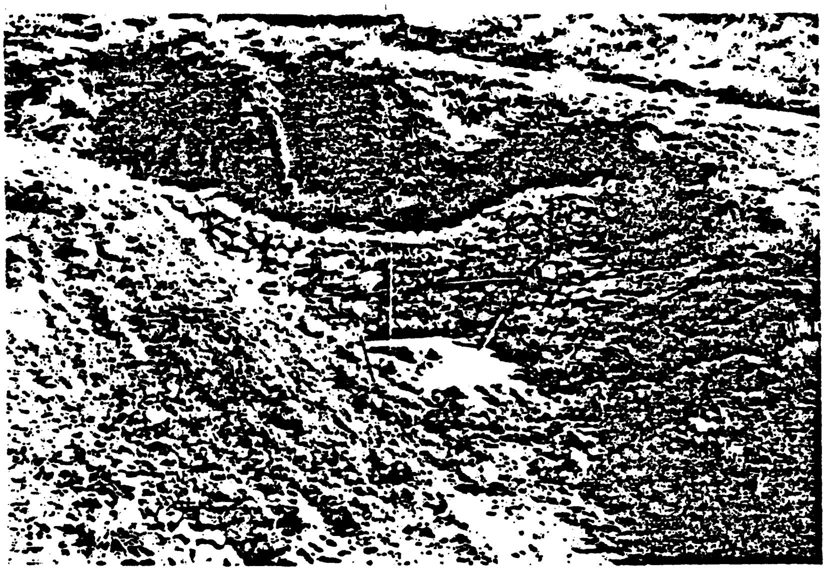


Figure 17.—Upstream view of a double-fence dam. Note the bank protection work. The apron is covered by water. Length of rod is 1.7 m.

were required because excessive water jetting through the structures caused bank damage. The percentage of small rock sizes was too low.

When flows of large magnitude, say 2 m^3/s, or gullies on steep hillsides are encountered, the base of the double-fence dam should be wider than the crest. This will add structural stability and increase the length of the flow through the lower part of the dam.

Gabion

A gabion check dam consists of prefabricated wire cages that are filled with loose rock. Individual cages are placed beside and onto each other to obtain the dam shape. Normally, this dam is more esthetically pleasing, but it is more costly than loose-rock or wire-bound rock check dams.

Headcut Control

Headcuts can be stabilized by different types of structures, but all have two important requirements: (1) porosity in order to avoid excessive pressures and thus eliminate the need for large, heavy structural foundations; and (2) some type of inverted filter that leads the seepage gradually from smaller to the larger openings in the structure. Otherwise, the soils will be carried through the control, resulting in erosion. An inverted filter can be obtained if the headcut wall is sloughed to such an angle that material can be placed in layers of increasing particle size, from fine to coarse sand and on to fine and coarse gravel. Good results may also be obtained by use of erosion cloth, a plastic sheet available in two degrees of porosity.

If rock walls reinforced by wire mesh and steel posts are used, site preparation can be minimized. Loose rock can be an effective headcut control (Heede 1966) if the flow through the structure is controlled also. As in loose-rock check dams, the size, shape, and size distribution of the rock are of special importance to the success of the structure. The wall of the headcut must be sloped back so the rock can be placed against it.

If the toe of the rock fill should be eroded away, the fill would be lost. Therefore, stabilization of this toe must be emphasized in the design. A loose-rock dam can be designed to dissipate energy from the chuting flows, and to catch sediment (fig. 18). Sediment depositions will further stabilize the toe of the rock fill by encouraging vegetation during periods with no or low channel flow.

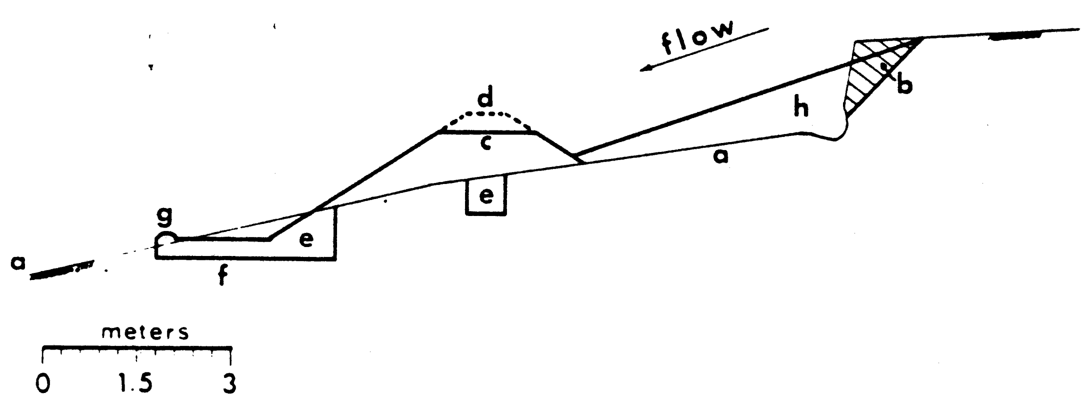


Figure 18.—Construction plan for a gully headcut control with a loose-rock check dam. The section of the structure is parallel to the centerline of the gully. a = original gully bottom; b = excavated area of headcut wall; c = spillway; d = crest of freeboard; e = excavation for key; f = excavation for apron; g = end sill; h = rock fill.

General Design Criteria

Loose Rock

Loose rock has proved to be a very suitable construction material if used correctly. Often it is found on the land and thus eliminates expenditures for long hauls. Machine and/or hand labor may be used. The quality, shape, size, and size distribution of the rock used in construction of a check dam affect the success and lifespan of the structure.

Obviously, rock that disintegrates rapidly when exposed to water and atmosphere will have a short structural life. Further, if only small rocks are used in a dam, they may be moved by the impact of the first large water flow, and the dam quickly destroyed. In contrast, a check dam constructed of only large rocks that leave large voids in the structure will offer resistance to the flow, but may create water jets through the voids (fig. 19). These jets can be highly destructive if directed toward openings in the bank protection work or other unprotected parts of the channel. Large voids in check dams also prevent the accumulation of sediment above the structures. In general, this accumulation is desirable because it increases the stability of structures and enhances stabilization of the gully.

Large voids will be avoided if the rock is well graded. Well-graded rock will permit some flow through the structure. The majority of the rock should be large enough to resist the flow.

Since required size and gradation of rock depend on size of dam and magnitude of flow, strict rules for effective rock gradation cannot be given. The recommendations given below are empirical values derived from gully treatments in the Colorado Rocky Mountains, and should be evaluated accordingly. The designer should use these values only as a guide for his decision.

As a general rule, rock diameters should not be less than 10 cm, and 25 percent of all rocks should fall into the 10- to 14-cm size class. The upper size limit will be determined by the size of the dam; large dams can include larger rock than small ones. Flat and round rock, such as river material, should be avoided. Both types slip out of a structure more easily than broken rocks, which anchor well with each other.

In general, large design peak flows will require larger rock sizes than small flows. As an example, assume that the designed total dam height ranges between 1 and 2 m, where total height is measured from the bottom of the dam to the crest of the freeboard. Type of dam is loose rock without reinforcement. Design peak flow is estimated not to exceed 1 m^3/s. An effective rock gradation would call for a distribution of size classes as follows:

Size	Percent
10–14 cm	25
15–19 cm	20
20–30 cm	25
31–45 cm	30

If, on the other hand, dam height would be increased to 3 m, rock up to 1 m diameter, constituting 15 percent of the volume, could be placed into the base of the dam and the second size class decreased by this portion. If peak flow

Figure 19.—Because this double-fence rock check dam was built with an insufficient portion of small rocks, many large voids allow water jets through the structure. Note that water is not running over the spillway. The jets endanger the stability of the structural keys and bank protection work.

was estimated not to exceed 0.75 m^3/s, the 31- to 45-cm size class could be eliminated and 55 percent of the volume could be in the 20- to 30-cm class.

In ephemeral gullies, only in exceptional cases will meaningful flow information be available that permits a realistic estimate of average velocities at the dam sites. If flow information is available, an equation developed by Isbach and quoted by Leliavsky (1957) may be used to check the suitability of the larger sizes. The equation relates the weight of rock to the mean velocity of the flow as follows:

$$W = 2.44(10^{-5})V^6 \qquad (5)$$

where W is the weight of rock related to D_{65} of the rocks, and V is stream velocity. D_{65} is the sieve size that allows 65 percent of the material to pass through. This equation states that 65 percent of the rocks can be smaller and 35 percent larger than the calculated weight. As stated above, the smallest size should have a diameter of 10 cm.

Spacing

The location of a check dam will be determined primarily by the required spacing of the structures. Requirements for spacing depend on the gradients of the sediment deposits expected to accumulate above the dams, the effective heights of the dams, the available funds, and the objective of the gully treatment. If, for instance, the objective is to achieve the greatest possible deposition of sediment, high, widely spaced dams would be constructed. On the other hand, if the objective is mainly to stabilize the gully gradient, the spacing would be relatively close and the dams low.

In general, the most efficient and most economical spacing is obtained if a check dam is placed at the upstream toe of the final sediment deposits of the next dam downstream. This ideal spacing can only be estimated, of course, to obtain guidelines for construction plans.

Normally, objectives of gully control require spacings of check dams great enough to allow the full utilization of the sediment-holding capacity of the structures. Determination of this spacing requires definite knowledge of the relationship between the original gradient of the gully channel and that of sediment deposits above check dams placed in the gully. This relationship has been hypothesized by several authors.

Kaetz and Rich[5] were the first known investigators to propose a relationship between the slope of sediment deposits above structures and that of the original thalweg. They concluded that the ratio varied between 0.3 and 0.6. The steeper deposition slopes were found in channels carrying coarse gravel, in contrast to the flatter slopes associated with fine loads. When some of the same structures were resurveyed 22 years later (Myrick Survey, as quoted by Leopold et al. 1964), the sediment wedge had lengthened only slightly since the time of the first survey. The increase in length was accompanied by a slight steepening of the deposition slope.

The Los Angeles County Flood Control District, engaged in gully control since the 1930's, used an empirically established ratio of 0.7 between deposition and original bed slope (Ferrell 1959, Ferrell and Barr 1963). In a sediment trend study, conducted 9 years after installation of a check dam treatment, the validity of this ratio could not be confirmed (Ruby 1973). It appears that a 9-year period is not sufficiently long to prove or disprove the rule of thumb.

Deposition of sediment above dams is a dynamic process dependent on regimen and magnitudes of flows during the treatment period. In a laboratory study on low-drop structures for alluvial flood channels, it was demonstrated that the regimen of flow exerts an overriding influence on channel grade (Vanoni and Pollak 1959). Also, Ruby (1973) stated that the system is constantly changing. But it is important to note that in the Los Angeles treatment, all sediment deposits have consistently aggraded, and not one has yet degraded. This suggests that sediment is still accumulating above the check dams.

Heede (1960) evaluated 20- to 26-year-old check dams in the Colorado Front Range (eastern flank) of the Rocky Mountains, and found the ratio of deposition to original bed slope fluctuating between 0.5 and 0.65. The soils had a large amount of coarse particles, and clay content was low. A check of 15-year-old earth check dams and stock pond structures on the western flank of the Colorado Rocky Mountains showed an average ratio of 0.7 (Heede 1966). This ratio was applied to an extensive watershed restoration

[5]Kaetz, A. G., and L. R. Rich, 1939. Report of survey made to determine grade of deposition above silt and gravel barriers. (Unpublished memo, dated Dec. 5, 1939, on file, U.S. Soil Conserv. Serv. library, Albuquerque, N.M.)

project on the western slope of the Rocky Mountains in 1963. That project is now being evaluated.[6]

Channel structures were investigated in Arizona washes by Hadley (1963). He concluded that a rise in base level, as represented by a dam, reduced the channel slope and caused aggradation upstream to a higher elevation than that of the channel control (dam). From the field observations, he inferred that the extent of deposition is determined by valley width, channel slope, particle size of the material, and vegetation. A ratio was not established.

The deposition slopes behind the impermeable structures of the Arizona washes were compared with those of permeable structures in the upper Rio Puerco Basin of New Mexico (Lusby and Hadley 1967). The latter developed steeper slopes than the impermeable dams. Impermeable structures, placed on gentle hill slopes, consisted of wooden fenceposts and woven-wire fencing material, and were set into the ground so that 0.3 m was above the original land surface.

A general flattening of the deposition slope, as compared with the original thalweg, was also found in field investigations on 25-year-old gully control structures in Wisconsin (Woolhiser and Miller 1963). The ratio ranged between 0.29 and 1.22. Interestingly, the authors recognized the classic aggradation-degradation pattern between structures; it showed degradation and the associated flattening of the channel slope caused by a reduction in the sediment load.

Woolhiser and Lenz (1965) also demonstrated that not only the original channel gradient influences the deposition slope, but also the width of the channel at the structure, and the crest height of the spillway above the original channel bottom. These authors found an average slope ratio of 0.52. Where original slopes were less than 14 percent, the average ratio was raised to 0.66; the ratios tended to be smaller as the original slope increased.

As the above discussion demonstrated, relationships developed so far have been entirely empirical, and further research is necessary to establish the theoretical basis.

In Colorado, earth dams were examined for guidance in determining the spacing of dams (Heede 1966). Data indicated that, in gullies of less than 20 percent gradient, the dams would not interfere with sediment catch if their spacing was based on the expected slope of the deposits being 0.7 of the original gully gradient. For gully gradients exceeding 20 percent, expected sediment deposits would have a gradient of 0.5 that of the gully. Heede and Mufich (1973) developed an equation to simplify the calculation of spacing as follows:

$$S = \frac{H_E}{K\,G\cos\alpha} \tag{6}$$

where S is the spacing, H_E is effective dam height as measured from gully bottom to spillway crest, G represents the gully gradient as a ratio, α is the angle corresponding to the gully gradient ($G = \tan\alpha$), and K is a constant. The equation is based on the assumption that the gradient of the sediment deposits is $(1-K)G$. In the Colorado example, values for K were:

$$K = 0.3 \text{ for } G \leq 0.20 \tag{7}$$
$$K = 0.5 \text{ for } G > 0.20 \tag{8}$$

The generalized equation (6) can be used by the designer, after the applicable K value has been determined for the treatment area. Works older than 10 years should be inspected for this determination. Figure 20 illustrates the relationship between dam spacing, height, and gully gradient. For a given gully, the required number of dams decreases with increasing spacing or increasing effective dam height, and increases with increasing gully gradient. An example for a 600 m gully segment is given in figure 21.

Keys

Keying a check dam into the side slopes and bottom of the gully greatly enhances the stability of the structure. Such keying is important in gullies where expected peak flow is large, and where soils are highly erosive (such as soils with high sand content). Loose-rock check dams without keys were successfully installed in soils derived from Pikes Peak granite, but estimated peak flows did not exceed 0.2 m^3/s (Heede 1960).

The objective of extending the key into the gully side slopes is to prevent destructive flows of water around the dam and consequent scouring of the banks. Scouring could lead to gaps between dam and bank that would render the structure ineffective. The keys minimize the danger of scouring and tunneling around check dams because the route of seepage is considerably lengthened. As voids in the keys become plugged, the length of the seepage route increases. This in-

[6] *Heede, Burchard H. Evaluation of an early soil and water rehabilitation project—Alkali Creek watershed, Colorado. (Research Paper in preparation at Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.)*

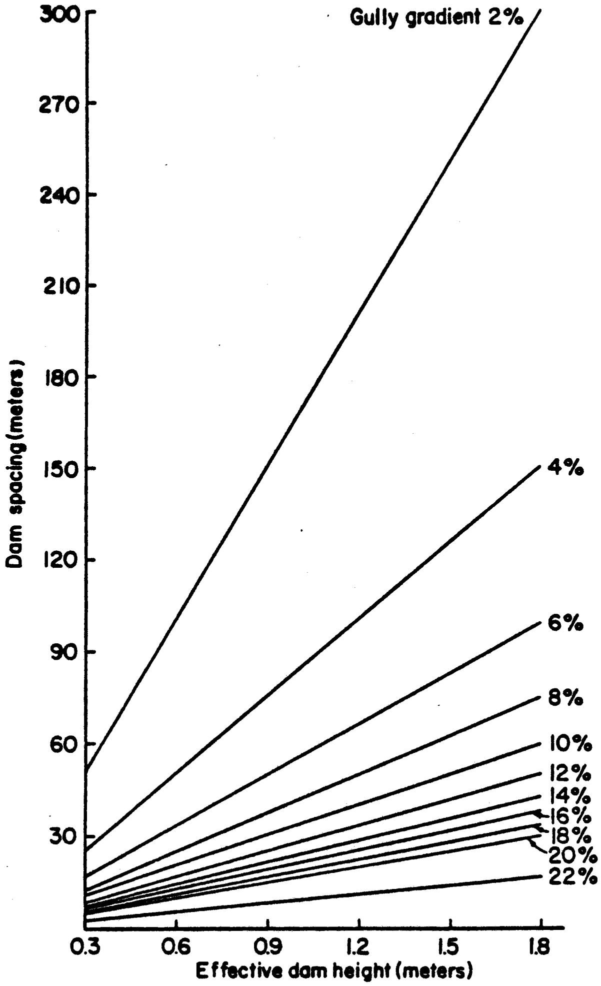


Figure 20.—Spacing of check dams, installed in gullies with different gradients, as a function of effective dam height.

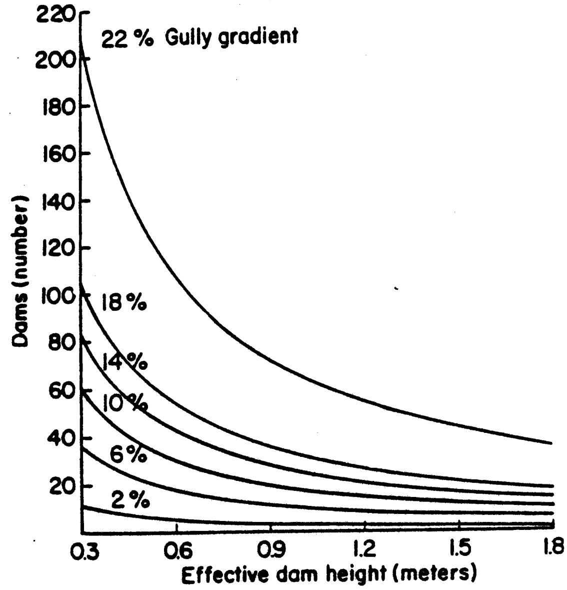


Figure 21.—Number of dams required in gullies, 600 m long and with different gradients, as a function of effective dam height.

crease causes a decrease in the flow velocity of the seepage water and, in turn, a decrease of the erosion energy.

The part of the key placed into the gully bottom is designed to safeguard the check dam against undercutting at the downstream side. Therefore, the base of the key, which constitutes the footing of the dam, must be designed to be below the surface of the apron. This is of particular importance for fence-type and impervious structures because of the greater danger of scouring at the foot of these dams. The water flowing over the spillway forms a chute that creates a main critical area of impact where the hydraulic jump strikes the gully bottom. This location is away from the structure. The sides of loose-rock and wire-bound check dams slope onto the apron, on the other hand, and no freefall of water occurs.

The design of the keys calls for a trench, usually 0.6 m deep and wide, dug across the channel. Where excessive instability is demonstrated by large amounts of loose materials on the lower part of the channel side slopes or by large cracks and fissures in the bank walls, the depth of the trench should be increased to 1.2 or 1.8 m.

Dam construction starts with the filling of the key with loose rock. Then the dam is erected on the rock fill. Rock size distribution in the key should be watched carefully. If voids in the key are large, velocities of flow within the key may lead to washouts of the bank materials. Since the rock of the keys is embedded in the trench and therefore cannot be easily moved, it is advantageous to use smaller materials, such as a mixture with 80 percent smaller than 14 cm.

Height

The effective height of a check dam (H_E) is the elevation of the crest of the spillway above the original gully bottom. The height not only influences structural spacing but also volume of sediment deposits.

Heede and Mufich (1973) developed an equation that relates the volume of sediment deposits to spacing and effective height of dam:

$$V_S = \tfrac{1}{2} H_E S \cos \alpha L_{HE} \qquad (9)$$

where V_S is the sediment volume, S represents the spacing, and L_{HE} is the average length of dam, considered for effective dam height and calculated by the equation:

$$L_{HE} = L_B + \frac{L_U - L_B}{2D} H_E \qquad (10)$$

where L_B is the bottom width and L_U the bank width of the gully, measured from brink to brink, and D is the depth of the gully. If S in equation (9) is substituted, then

$$V_S = \frac{H_E^2}{2KG} L_{HE} \qquad (11)$$

where the constant K has the values found to be applicable to the treatment area. Equation (11) indicates that sediment deposits increase as the square of effective dam height (fig. 22).

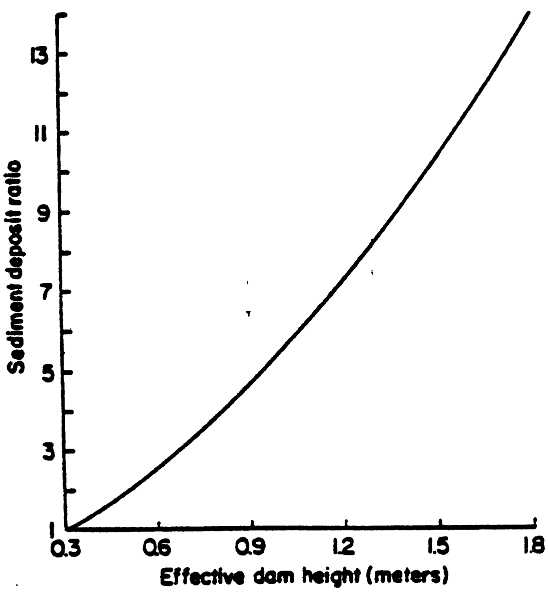


Figure 22.—Expected sediment deposits retained by check dam treatment as a function of effective dam height. The sediment deposit ratio relates the volume of sediment deposits to the volume of sediment deposits at effective dam height of 0.3 m. Thus, deposits in a treatment with 1.2 m dams are more than seven times larger than those caught by 0.3 m dams.

For practical purposes, based on the sediment deposit model, the sediment curve in figure 22 is valid for treatments in gullies with identical cross sections and gradients ranging from 1 to 30 percent. At this range, the difference is 4.5 percent with smaller deposits on the steeper gradients, a negligible fraction in such estimates. The volume of deposits, compared with that on a 1 percent gradient, decreases by 10 percent on a gradient of 45 percent, if the cross sections are constant. Magnitudes of cross sections, of course, exert strong influences on sediment deposition.

In most cases, dam height will be restricted by one or all of the following criteria: (1) costs, (2) stability, and (3) channel geometry in relation to spillway requirements. Cost relations between different types of rock check dams will be discussed later. Stability of impervious check dams should be calculated where life and/or property would be endangered by failure. Heede (1965b) presented an example for these calculations which can be easily followed. Pervious dams such as rock check dams cannot be easily analyzed for stability, however, because of unknowns such as the porosity of a structure.

Severely tested check dams in Colorado (Heede 1966) had maximum heights as follows: loose-rock and wire-bound dams, 2.2 m; and fence-type dams (thickness of 0.6 m), 1.8 m.

In gullies with small widths and depths but large magnitudes of flow, the effective height of dams may be greatly restricted by the spillway requirements. This restriction may result from the spillway depth necessary to accommodate expected debris-laden flows.

Spillway

Since spillways of rock check dams may be considered broad-crested weirs (fig. 23), the discharge equation for that type of weir is applicable:

$$Q = CLH^{3/2} \qquad (12)$$

where Q = discharge in m^3/s, C = coefficient of the weir, L = effective length of the weir in m, and H = head of flow above the weir crest in m.

The value of C varies. The exact value depends on the roughness as well as the breadth and shape of the weir and the depth of flow. Since in rock check dams, breadth of weir changes within a structure from one spillway side to the other, and shape and roughness of the rocks lining the spillway also change, C would have to be determined experimentally for each dam. This, of course, is not practical and it is recommended, therefore, to

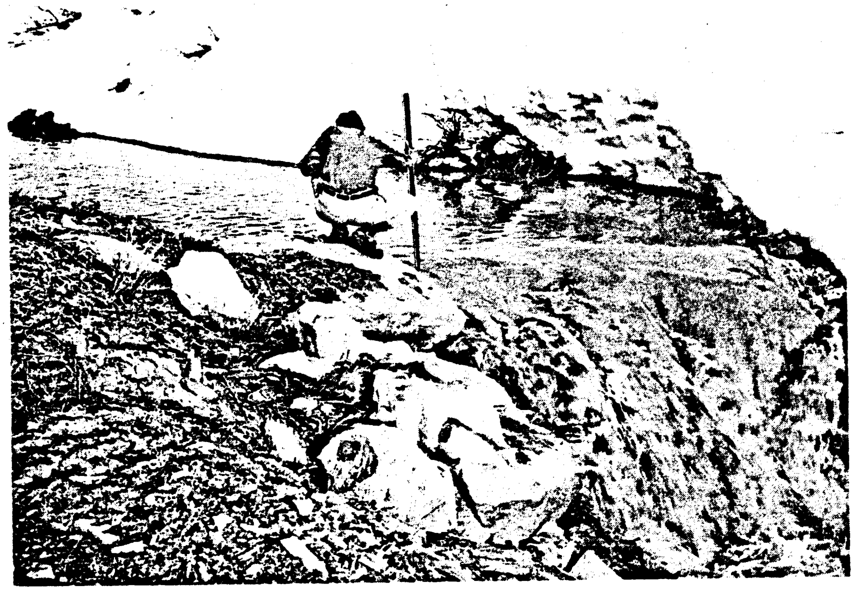


Figure 23.—Upstream view of a loose-rock check dam supporting a discharge of about 0.3 m³/s. Effective dam height is 1.7 m.

use a mean value of 1.65. This value appears reasonable in the light of other inaccuracies that are introduced in calculating the design storm and its expected peak flow. For this reason also, the discharge calculations would not be significantly improved if they were corrected for the velocity of approach above a dam. Such a correction would amount to an increase of 5 percent of the calculated discharge at a head of flow of 0.6 m over a dam 0.75 m high, or 8 percent if the flow had a 0.9 m head.

Most gullies have either trapezoidal, rectangular, or V-shaped cross sections. Heede and Mufich (1973) developed equations for the calculation of spillway dimensions for check dams placed in these gully shapes. In trapezoidal gullies, the equation for length of spillway can be adjusted to prevent the water overfall from hitting the gully side slopes, thus eliminating the need for extensive bank protection. In V-shaped gullies this is not possible, generally. In rectangular ones, adjustment of the equation is not required because the freeboard requirement prevents the water from falling directly on the banks. One equation was established, therefore, for V-shaped and rectangular gullies as follows:

$$H_{SV} = \left(\frac{Q}{CL_{AS}}\right)^{2/3} \qquad (13)$$

where H_{SV} is spillway depth, the constant C is taken as 1.65, and L_{AS}, the effective length of spillway, was derived from the equation

$$L_{AS} = \frac{L_U}{D} H_E - f \qquad (14)$$

in which f is a constant, referring to the length of the freeboard. In gullies with a depth of 1.5 m or less, the f value should not be less than 0.3 m; in gullies deeper than 1.5 m, the minimum value should be at least 0.6 m.

For structural gully control, design storms should be of 25 years magnitude, and, as a minimum, spillways should accommodate the expected peak flow from such a storm. In mountainous watersheds, however, where forests and brushlands often contribute large amounts of debris to the flow, the size and the shape of spillways should be determined by this expected organic material. As a result, required spillway sizes will be much larger than if the flow could be considered alone. Spillways designed with great lengths relative to their depths are very important here. Yet, spillway length can be extended only within limits because a sufficient contraction of the flow over the spillway is needed to form larger depths of flows to float larger loads over the crest. The obstruction of a spillway by debris is undesirable since it may cause the flow to overtop the freeboard of the check dam and lead to its destruction.

The characteristics of the sides of a spillway are also important for the release of debris over the structure. Spillways with perpendicular sides will retain debris much easier than those with sloping sides; in other words, trapezoidal cross sections are preferable to rectangular ones. A trapezoidal shape introduces another benefit by increasing the effective length of the spillway with increasing magnitudes of flow.

The length of the spillway relative to the width of the gully bottom is important for the protection of the channel and the structure. Normally, it is desirable to design spillways with a length not greater than the available gully bottom width so that the waterfall from the dam will strike the gully bottom. There, due to the stilling-basin effects of the dam apron, the turbulence of the flow is better controlled than if the water first strikes against the banks. Splashing of water against the channel side slopes should be kept at a minimum to prevent new erosion. Generally, spillway length will exceed gully bottom width in gullies with V-shaped cross sections, or where large flows of water and debris are expected relative to the available bottom width. In such cases, intensive protection of the gully side slopes below the structures is required.

Equation (13) includes a safety margin, because the effective length of the spillway is calculated with reference to the width of the gully at the elevation of the spillway bottom, instead of that at half the depth of the spillway. At spillway bottom elevation, gullies are generally narrower than at the location of the effective spillway length. This results in somewhat smaller spillway lengths, which will benefit the fit of the spillway into the dam and the gully.

If the spillway sides are sloped 1:1, it follows that in V-shaped and rectangular gullies, the bottom length of the spillway (L_{BSV}) is derived from the equation

$$L_{BSV} = L_{AS} - H_{SV} \tag{15}$$

and the length between the brinks of the spillway (L_{USV}) is given by the equation

$$L_{USV} = L_{AS} + H_{SV} \tag{16}$$

In trapezoidal gullies, the effective length of the spillway equals the bottom width of the gully. From the discharge equation for broad-crested weirs, it follows that the depth of spillway (H_S) in these gullies is given by the equation

$$H_S = \left(\frac{Q}{CL_B}\right)^{2/3} \tag{17}$$

in which the coefficient of the weir (C) is taken as 1.65.

Lengths at the bottom (L_{BS}) and between the brinks of the spillway (L_{US}) are calculated by the equations

$$L_{BS} = L_B - H_S \tag{18}$$

and

$$L_{US} = L_B + H_S \tag{19}$$

respectively.

Rock-fill dams were also designed with builtin spillways (Parkin 1963). At minimum depth of flow, the flow passes on a plane through the crest of the spillway and inclines at 45° to the vertical at the downstream side. The design does not appear to be suitable for most gully control situations due to the high sediment loads, which rapidly clog the structural voids. The detailed discussions and design equations could be helpful, however, in testing rock stability as related to specific gravity and diameter of rock as well as in estimating the void ratio. The interested reader should, however, be aware that the design criteria are for large dams, supporting discharges between 28 and 85 m^3/s, and that flow information must be available. In most situations, conservation programs have to be started to meet public demand even though adequate hydrologic data are not available. Not much has changed since this observation was made by Peterson and Hadley (1960).

Apron

Aprons must be installed on the gully bottom and protective works on the gully side slopes below the check dams, otherwise flows may easily undercut the structures from downstream and destroy them.

Apron length below a loose-rock check dam cannot be calculated without field and laboratory investigations on prototypes. Different structures may have different roughness coefficients of the dam side slope that forms a chute to the flow if tailwater depth is low. Differences in rock gradation may be mainly responsible for the different roughness values.

The design procedures for the loose-rock aprons were therefore simplified and a rule of thumb adopted: the length of the apron was taken as 1.5 times the height of the structure in channels where the gradient did not exceed 15 percent, and 1.75 times where the gradient was steeper than 15 percent. The resulting apron lengths included a sufficient margin of safety to prevent the waterfall from hitting the unprotected gully bottom. The design provided for embedding the apron into the channel floor so that its surface would be roughly level and about 0.15 m below the original bottom elevation.

In contrast, for straight-drop structures such as dams built from steel sheets or fence-type dams, apron length can be calculated if gully flows are known. In such a case, the trajectory of the nappe can be computed as follows:

$$V_o = x \sqrt{\frac{g}{2(-z)}} \tag{20}$$

in which x and z are the horizontal and vertical coordinates of a point on the trajectory referred to the midpoint of the spillway as the origin, and g is the acceleration due to gravity, taken as 9.81 m/s^2 (Howe 1950). Thus, if V_o is substituted for V_c, the critical velocity at dam crest, and z is the effective dam height, x will yield the distance from the structure at which the waterfall will hit the apron. Depending on magnitude of flow, one or several meters should be added to this distance.

The procedure for calculating critical depth and critical velocity over a check dam is as follows: The critical depth equation is

$$\frac{V_c^2}{2g} = \frac{Y_c}{2} \tag{21}$$

where V_c is the critical velocity, and Y_c is the critical depth. The continuity equation for open channel flow is

$$q = AV \tag{22}$$

where q is the flow rate of unit width of flow, A is the cross section of flow, and V represents the average velocity in the cross section. q is derived from the estimated rate of flow Q by dividing Q by L_{AS}, the effective length of spillway. Since q refers to unit width of flow, A can be replaced by Y_c and equation (22) becomes

$$V_c = \frac{q}{Y_c} \tag{23}$$

If V_c in equation (21) is replaced by (23),

$$\frac{q^2}{g} = Y_c^3 \tag{24}$$

By placing the value of Y_c, the depth of flow over the spillway, into equation (23), the critical velocity (V_c) can be obtained.

At the downstream end of the apron, a loose-rock sill should be built 0.15 m high, measured from channel bottom elevation to the crest of the sill. This end still creates a pool in which the water will cushion the impact of the waterfall.

The installation of an end sill provides another benefit for the structure. Generally, aprons are endangered by the so-called ground roller that develops where the hydraulic jump of the water hits the gully bottom. These vertical ground rollers of the flow rotate upstream, and where they strike the gully floor, scouring takes place. Thus, if the hydraulic jump is close to the apron, the ground roller may undermine the apron and destroy it (Vanoni and Pollak 1959). The end sill will shift the hydraulic jump farther downstream, and with it the dangerous ground roller. The higher the end sill, the farther downstream the jump will occur. Since data on sediment and flow are not usually available, a uniform height of sill may be used for all structures.

Ephemeral gullies carry frequent flows of small magnitudes. Therefore, it is advisable not to raise the crest of the end sills more than 0.15 to 0.25 m above the gully bottom. End sills, if not submerged by the water, are dams and create waterfalls that may scour the ground below the sill. At higher flows, some tailwater usually exists below a sill and cushions to some extent the impact from the waterfall over the sill.

Where the downstream nature of the gully is such that appreciable depth of tailwater is expected, the installation of end sills is not critically important. The hydraulic jump will strike the water surface and ground rollers will be weak.

Bank Protection

Investigations have shown (Heede 1960) that check dams may be destroyed if flows scour the gully side slopes below the structures and produce a gap between the dam and the bank. Since water below a check dam is turbulent, eddies develop that flow upstream along each gully side slope. These eddies are the cutting forces.

Several types of material are suitable for bank protection. Loose rock is effective, but should be reinforced with wire-mesh fence, secured to steel posts, on all slopes steeper than 1.25 to 1.00 (see fig. 17). The design should provide for excavation of the side slopes to a depth of about 0.3 m so that the rock can be placed flush with the surrounding side slope surface to increase stability of the protection. Excavation of surface materials also assures that the rock would not be set on vegetation. Banks should be protected for the full length of the apron.

The height of the bank protection depends on the characteristics of channel, flow, and structure. Where gullies have wide bottoms and spillways are designed to shed the water only on the channel floor, the height should equal total dam height at the structure, but can rapidly decrease with distance from the structure. In contrast, where the waterfall from a check dam will strike against the gully banks, the height of the bank protection should not decrease with distance from dam to prevent the water from splashing against unprotected banks.

In gullies with V-shaped cross sections, the height of the bank protection should be equal to the elevation of the upper edges of the freeboards of the dam. In general, the height of the bank protection can decrease with increasing distance from the dam.

Equations for Volume Calculations

After the dam locations have been determined in the field, based on spacing requirements and suitability of the site for a dam, gully cross sections at these locations should be surveyed and plotted. If possible, use the computer program developed by Heede and Mufich (1974) to design the dams. Otherwise the dams must be designed from the plotted gully cross sections. Structural and gully dimensions can be used in equations developed by the above authors.

Loose-Rock and Wire-Bound Dams

The volume equation for the dam proper of loose-rock and wire-bound dams considers either angular or round rock, because the angle of repose varies with rock shape and influences the side slopes of the dam. The generalized equation is

$$V_{LR} = \frac{H_D^2}{\tan A_R} + 0.6\,HL_A - V_{SP} \qquad (25)$$

where V_{LR} is the volume of the dam proper, H_D represents dam height, 0.6 is a constant that refers to the breadth of dam, L_A is the average length of the dam, $\tan A_R$ is the tangent of the angle of repose of the rock type, and V_{SP} is the volume of the spillway. It is assumed that the angle of repose for angular rock is represented by a slope of 1.25:1.00, corresponding to a tangent of 0.8002; for round rock, the slope is 1.50:1.00 with a tangent of 0.6590. L_A is given by the equation

$$L_A = L_B + \frac{L_U - L_B}{2D} H_D \qquad (26)$$

where L_B is the length of dam at the bottom, L_U represents the length of dam measured at freeboard elevation, and D is the depth of the gully. V_{SP} is calculated by the equation

$$V_{SP} = H_S L_{AS} B_A \qquad (27)$$

where H_S is the depth and L_{AS} is the effective length of the spillway; B_A is the breadth of the dam, measured at half the depth of the spillway and derived from the equation

$$B_A = \frac{H_S}{0.70711 \tan A_R} + 0.3 \qquad (28)$$

where 0.70711 is the sine of 45°, and 0.3 is a constant derived from a breadth of dam of 0.6 m.

Angular rock is preferable to round rock because less is required, and it enhances dam stability.

The equation for volumes of loose-rock and wire-bound loose-rock dams (eq. 25) was simplified by assuming a zero gully gradient. This assumption results in an underestimate of volumes in gullies with steep gradients. To offset this underestimate on gradients larger than 15 percent, 10 percent should be added to the calculated volume.

If the design peak flow is larger than 0.3 m^3/s, all types of check dams must be keyed into the gully banks and bottom. Under varied conditions in Colorado, it was found that a bottom key of 0.6 m depth and width was sufficient for check dams up to 2 m high. A width of 0.6 m was also adequate for the bank keys. The depth of the keys, however, must be adjusted according to characteristics of the soils. Thus, the equation for the volume of the key is generalized as follows:

$$V_K = (L_A + 2R)(0.6H_D + 0.36) - 0.6H_D L_A \qquad (29)$$

where R represents the depth of key and 0.6 and 0.36 are constants in meter units, referring to depth and width of bottom key and width of bank key, respectively.

In the construction plan, the volume V_K should be kept separate from that of the dam proper because, generally, a finer rock gradation is required for the keys.

Apron and bank protection below the structure are always required at check dams. The equation developed for the volume calculations is:

$$V_A = cH_D L_B + dH^2 \qquad (30)$$

in which V_A is the rock volume of the apron and bank protection, and c and d are constants whose values depend on gully gradient. For gradients $\leq$ 15 percent, $c = 1.5$ and $d = 3.0$; for gradients $>$ 15 percent, $c = 1.75$ and $d = 3.5$.

The total rock volume required for a loose-rock dam with keys is the sum of equations (25), (29), and (30).

Besides rock, wire mesh and steel fenceposts are used in most of the dams. If dam height is equal to or larger than 1.2 m, reinforcement of the bank protection work by wire mesh and fenceposts will generally be required. The equation for amount of wire mesh and number of posts includes a margin of safety to offset unforeseen additional needs. To assist in construction, dimensions of the mesh are given in length and width. The length measured along the thalweg is

$$M_{LB} = 3.50\, H_D \tag{31}$$

where M_{LB} is the length of the wire mesh for the bank protection, and 3.50 is a constant. The width of the wire mesh, measured from the apron to the top of the bank protection at the dam, equals the total dam height.

The number of fenceposts is calculated by the equation

$$N_B = 3H_D + 2 \tag{32}$$

where N_B is the number of fenceposts for the bank protection, rounded up to a whole even number, and 2 and 3 are constants, the latter derived from a 1.2 m spacing. Of the total number of posts, half should be 0.75 m taller than the dam; the other half are of dam height.

For wire-bound dams, the length of the wire mesh is taken as that of the dam crest, which includes a safety margin and is calculated by the equation

$$M_L = L_B + \frac{L_U - L_B}{D} H \tag{33}$$

where M_L is the length of the wire mesh. The width of the mesh, measured parallel to the thalweg, depends not only on dam height but also on rock shape. The equation for the width of the wire mesh is

$$M_W = \frac{2H_D}{\tan A_R} + \frac{2H_D}{\sin A_R} + 3 \tag{34}$$

where M_W is the width and A_R the angle of repose of the rock. For angular rock, this angle is assumed to be 38° 40′, corresponding to a dam side slope of 1.25:1.00, and for round rocks 33° 25′, representing a slope of 1.50:1.00. The term 3 is a constant, in meter units. Equation (34) provides for an overlapping of the mesh by 1.8 m.

Single-Fence Dams

A zero gully gradient was assumed for calculating rock volume for the dam proper of single-fence dams. This results in overestimates that compensate for simplification of the equation for volume calculation. If the construction plan calls for a dam with a 0.6 m breadth, for ease of calculation, the cross section of the dam parallel to the thalweg is taken as a right triangle with a dam side slope of 1.25:1.00 in the equation:

$$V_{SF} = \frac{H_D^2}{2(0.80020)} L_A - V_{SSF} \tag{35}$$

where V_{SF} is the rock volume of the dam proper, 2 is constant, and 0.80020 represents the tangent of a slope of 1.25:1.00. V_{SSF} is the volume of the spillway, calculated by the equation

$$V_{SSF} = H_S L_{AS} B_{SF} \tag{36}$$

where B_{SF} is the breadth of the dam, measured at half the depth of the spillway and given by the equation

$$B_{SF} = \frac{H_S}{2(0.80020)} \tag{37}$$

The length of wire mesh for a single-fence dam is given by equation (33), while the width equals dam height. The number of fenceposts is calculated by the equation

$$N_{SF} = \frac{L_B}{1.2} + \frac{L_U - L_B}{2.4D} H_D + 1 \tag{38}$$

where N_{SF} is the number of posts of the dam proper of a single-fence dam, rounded up to a whole number: 1.2 signifies a distance of 1.2 m between the posts; 2.4 and 1 are constants. Of the total number of posts, half are 0.75 m taller than the dam; the other half are dam height. This equation contrasts with the original (Heede and Mufich 1974), because the high safety margin for number of posts was reduced.

Double-Fence Dams

The equation for rock volume of a double-fence dam with vertical fences, 0.6 m apart, is:

$$V_{DF} = 0.6H_DL_A - V_{SDF} \quad (39)$$

where V_{DF} is the volume, 0.6 is a constant, and V_{SDF} is the volume of the spillway, computed by the equation

$$V_{SDF} = 0.6H_SL_{AS} \quad (40)$$

where 0.6 represents the standard breadth of the dam, in meters.

The length of wire mesh is given by

$$M_{LD} = 2L_B + \frac{L_U - L_B}{D} 2H_D \quad (41)$$

where M_{LD} is the length of the mesh. The width of the wire mesh equals dam height. The number of fenceposts is computed by the equation

$$N_{DF} = \frac{L_B}{0.6} + \frac{L_U - L_B}{1.2D} H_D + 2 \quad (42)$$

where N_{DF} is the number of posts of the dam proper of a double-fence dam, rounded up to a whole even number, and 0.6, 1.2, and 2 are constants. The equation is based on a post spacing of 1.2 m. Half of the posts are dam height, while the other half are 0.75 m taller than the dam. Reduction of the high safety margin contained in the original equation (Heede and Mufich 1974) resulted in a new equation.

Headcut Control

The volume requirements for a headcut control structure are given by the equation

$$V_{HC} = \left(\frac{D^2}{2(0.33333)}\right)\left(\frac{L_U + 3L_B}{4}\right) \quad (43)$$

where V_{HC} is the rock volume, D is the depth of the gully at the headcut, and 0.33333 is the tangent of the angle that refers to a structure with a slope gradient of 3:1. If a slope gradient different from 3:1 is selected, the value of the tangent in the equation should be changed to correspond to that gradient.

Rock Volume Relations Among Dam Types

In the Colorado project (Heede and Mufich 1973), rock volumes required for the various types of check dams were expressed graphically (fig. 24). If this graph is used for decisionmaking, it must be recognized that double-fence dams had parallel faces 0.6 m apart. Where double-fence structures with bases wider than the breadth of dam are required, rock volume requirements will be larger. The graph shows that a loose-rock or wire-bound dam with effective height of 2 m requires 5.5 times more rock than a double-fence dam.

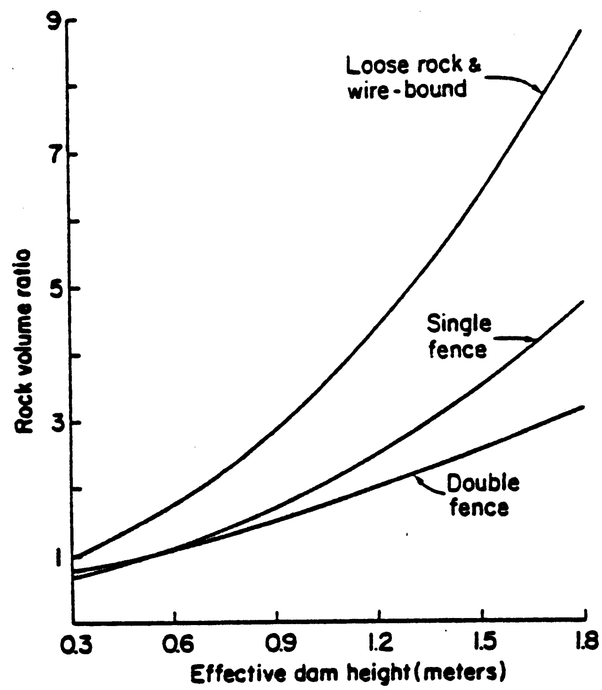


Figure 24.—Required volumes of angular rock for four different dam types as a function of effective dam height. The rock volume ratio relates the rock volume to that of a loose-rock dam 0.3 m high.

Construction Procedures

Before construction starts, the following design features should be staked and flagged conspicuously:

1. Mark the centerline of the dam and the key trenches, respectively, on each bank. Set the stakes away from the gully edge to protect them during construction.

2. Designate the crest of the spillway by a temporary bench-mark in the gully side slope sufficiently close to be of value for the installation of the dam.

3. Mark the downstream end of the apron.

4. For loose-rock and wire-bound dams, flag the upstream and downstream toes of the dam proper.

Caution is required during excavation to avoid destroying the stakes before the main work of installation begins.

The construction of all dams should start with the excavation for the structural key (fig. 25), the apron, and the bank protection. This very important work can be performed by a backhoe or hand labor. Vegetation and loose material should be cleaned from the site at the same time.

The trenches for the structural keys will usually have a width of 0.6 m, therefore a 0.5-m-wide bucket can be used on the backhoe. If the construction plan calls for motorized equipment, two types of backhoes can be used. One, mounted on a rubber-wheeled vehicle and operating from a turntable, permits the backhoe to rotate 360°. This machine travels rapidly between locations where the ground surfaces are not rough, and works very efficiently in gullies whose side slopes and bottoms can be excavated from one or both channel banks. The other type can be attached to a crawler tractor. This type proves to be advantageous at gullies that are difficult to reach, and with widths and depths so large that the backhoe has to descend into the channel to excavate. In deep gullies with V-shaped cross sections, temporary benches on the side slopes may be necessary. Often, the bench can be constructed by a tractor with blade before the backhoe arrives.

The excavated material should be placed upstream from the dam site in the gully. The excavated trench and apron should then be filled with rock. Since a special graded rock is required for the keys, rock piles for keys must be separate from those used in the apron and dam proper. Excavations can be filled by dumping from a dump truck or by hand labor. During dumping operations, the fill must be checked for voids, which should be eliminated.

If dump trucks are loaded by a bucket loader, some soil may be scooped up along with the rock. Soil is undesirable in a rock structure because of the danger of washouts. To avoid soil additions, use a bucket with a grilled bottom that can be shaken before the truck is loaded. Other devices such as a grilled loading chute would also be appropriate.

Dumping rock into the dam proper has two advantages: The structure will attain greater density, and rocks will be closer to their angle of repose than if placed by hand. Hand labor can never be completely avoided, however, since plugging larger voids and the final dam shape require hand placement. Where gullies are deep and dumping is impractical, rock chutes may be used.

Often, gully control projects are planned to provide employment for numbers of people. This objective can easily be accomplished if sufficient supervision is available for the individual steps in the construction. Special attention is needed at the spillway and freeboard. In loose-rock and wire-bound structures, where the shape of the dam is not outlined by a fence as in the other types, experience shows there is a tendency to construct the spillways smaller than designed.

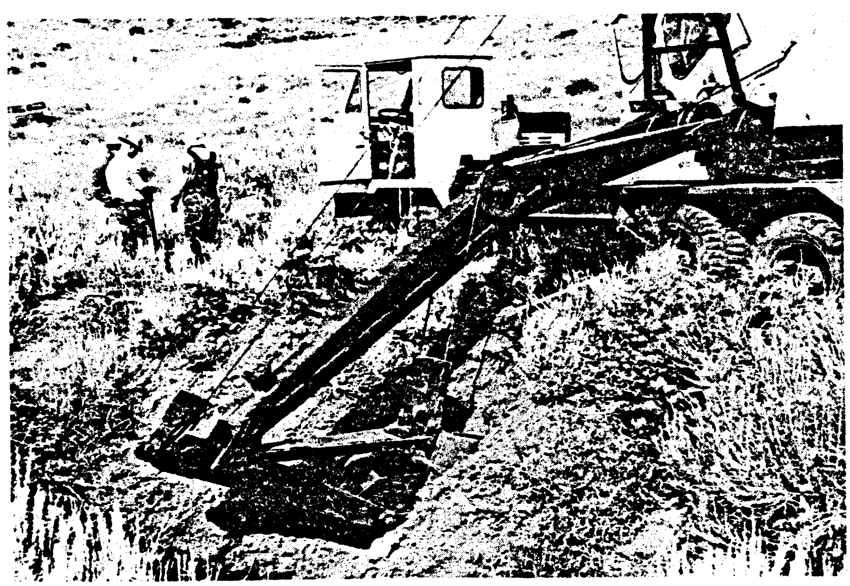


Figure 25.—The key for a rock check dam is efficiently excavated with a backhoe.

In wire-bound dams, a commercial, galvanized stock fence, usually about 1.2 m wide, can be used. The stay and line wires should not be less than 12½-gage low-carbon steel, the top and bottom wires 10-gage low-carbon steel, and the openings in the mesh 0.15 m. To connect ends of the fence or to attach the fence to steel posts, a galvanized 12½-gage coil wire is sufficiently strong.

The wire mesh of required length and width should be placed over the gully bottom and side slopes after the trench and apron have been filled with rock (fig. 26). Generally, several widths of mesh will be needed to cover the surface from bank to bank. If several widths are required, they should be wired together with coil wire where they will be covered with rocks. The parts not to be covered should be left unattached to facilitate the fence-stringing operations around the structure.

Before the rock is placed on the wire mesh for the installation of the dam proper, the mesh should be temporarily attached to the gully banks. Otherwise, the wire mesh lying on the gully side slopes will be pushed into the gully bottom by the falling rock and buried. Usually, stakes are used to hold the wire mesh on the banks.

After the dam proper is placed and shaped, the fence can be bound around the structure. Fence stretchers should be applied to pull the upstream ends of the fence material down tightly over the downstream ends, where they will be fastened together with coil wire. Then the bank protection below the dam should be installed.

The installation of single- and double-fence dams begins with the construction of the fences after excavation is completed (fig. 27). Construc-

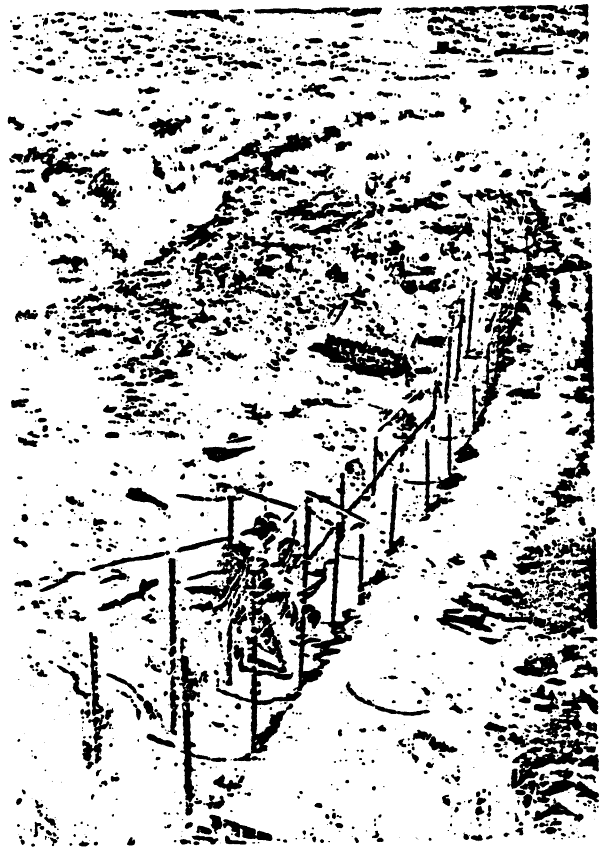


Figure 27.—Parallel fences for double-fence dam (see fig. 11) are being installed. Note the excavations for key, apron, and bank protection (the latter two to right of the structure).

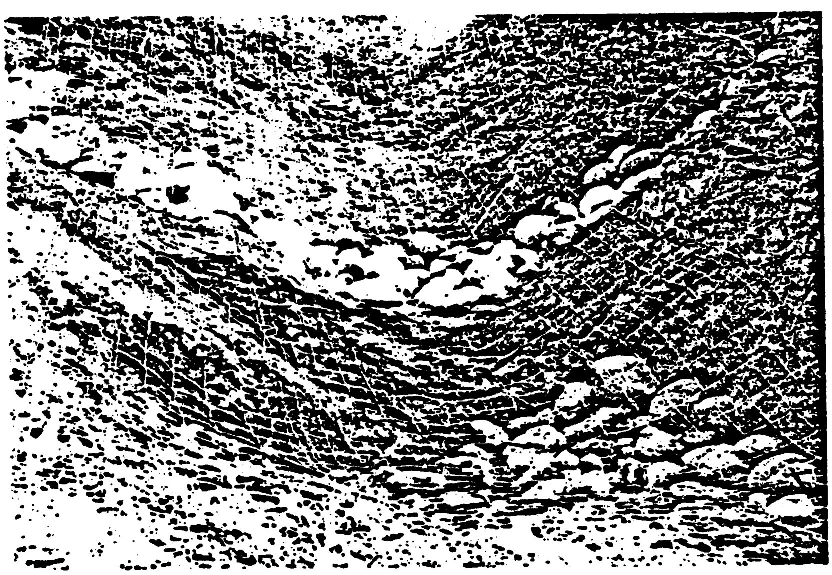


Figure 26.—Upstream view of a construction site for a wire-bound dam. Note that key and apron excavations were filled with rock before wire mesh was placed on the bed banks. The site is prepared for the construction of the dam proper.

tion drawings should be followed closely here, because the final shape of the dams will be determined by the fences. Conventional steel fenceposts can be used. In some locations, the great height of posts may offer difficulties for the operator of the driving equipment, and scaffolds should be improvised. A pneumatically driven pavement breaker with an attachment designed by Heede (1964) can be used to ease the job of driving. Since relatively great lengths of hose may be attached, this tool may be used in deep gullies and on sites with difficult access.

At single-fence dams, dumping of rock is practical if the gully is not excessively deep or wide. At double-fence structures, hand labor, or a backhoe or clamshell (fig. 28) will be required. The rock should be placed in layers and each layer inspected for large voids, which should be closed manually by rearranging rocks.

Much time and effort can be saved during construction if a realistic equipment plan is established beforehand. Such a plan requires an intimate knowledge of the cross-sectional dimensions of the gullies and their accessibility to motorized equipment. Pioneer roads that might be needed because of lack of access are not only important for equipment considerations, but will also enter into the cost of the construction.

If equipment is to be used, as a general rule, it appears to be advantageous to use heavier and larger machines if their mobility is adequate. Although hourly costs for heavier machines are usually greater, the total cost for a job is reduced.

With few exceptions, conventional construction equipment is not sufficiently mobile to operate in rough topography without pioneer roads. In watershed rehabilitation projects such as gully control, road construction is undesirable because it disturbs the ground surface and may lead to new erosion. It is therefore desirable to consider crawler-type equipment only.

Cost Relations

Relationships between the installation costs of the four different types of rock check dams described here are based on research in Colorado (Heede 1966). The relationships are expressed by ratios (fig. 29) to avoid specific dollar comparisons. When considering the cost ratio, one must keep in mind that differential inflation may have offset some finer differences in cost. It is advisable, therefore, to test the cost of individual structures by using material and volume requirements as given by the equations. The cost ratios in figure 29 can then be adjusted, if necessary.

In a given gully, for example, a double-fence dam with an effective height of 1.8 m costs only about four times as much as a 0.3 m loose-rock dam, while a wire-bound dam 1.8 m high costs 8.5 times as much. Costs will change with different sizes and gradients of gullies, but the general relationships will not change.

It is obvious that the cost of installing a complete gully treatment increases with gully gradient because the required number of dams increases. Figure 30 indicates there is one effective dam height at which the cost is lowest. In the sample gully, this optimum height for loose-rock dams is about 0.6 m, for single-fence dams 0.7 m, and for double-fence dams 1.1 m. A con-

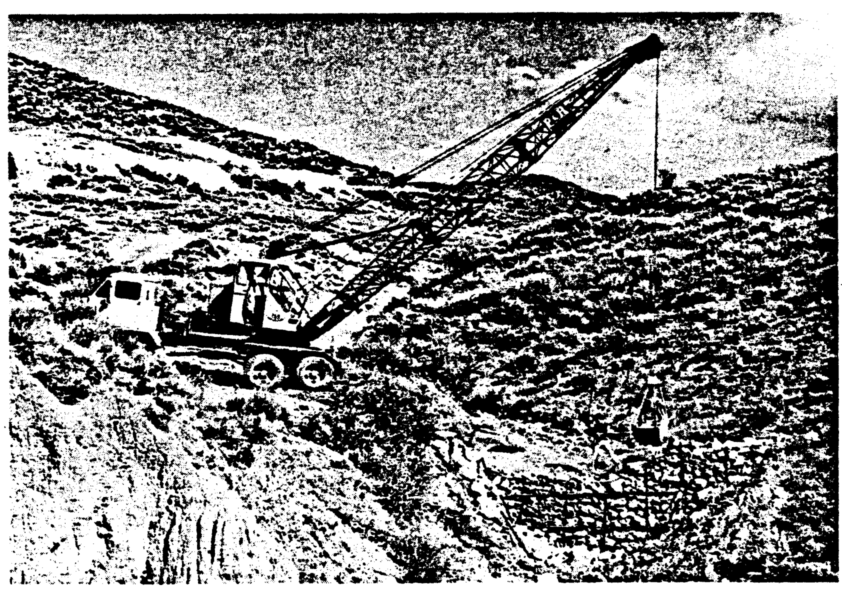


Figure 28.—Using clamshell to place rock into a double-fence dam. The man steadies the clamshell with a long rope.

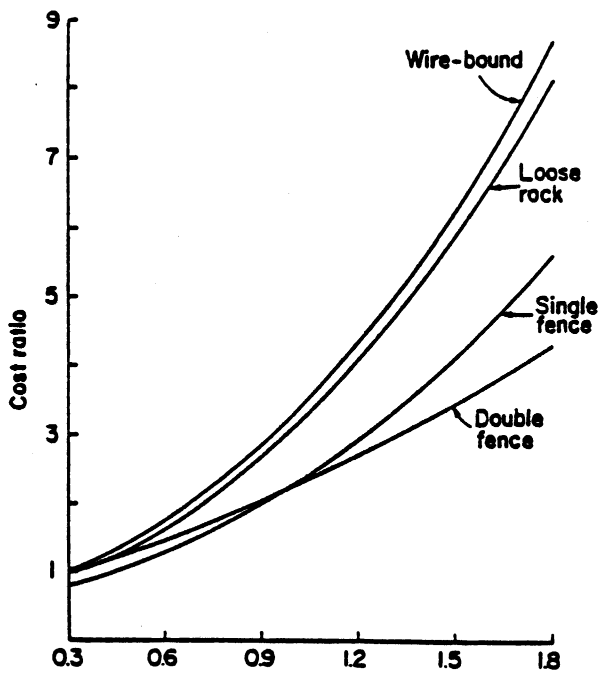


Figure 29.—Installation cost of four different types of check dams as a function of effective dam height. The cost ratio is the cost of a dam related to the cost of a loose-rock dam, 0.3-m-high, built with angular rock.

stant gully cross section was assumed. In reality, of course, gully cross sections usually change between dam sites. The optimum height for lowest treatment costs is not a constant, but changes between gullies, depending on shape and magnitude of the gully cross sections at the dam sites.

Since the cost of the dam is directly proportional to the rock volume, figure 30 also expresses the relationship between rock requirement and effective dam height. This means that, in a given gully, there is one dam height at which rock requirements for a treatment are smallest.

A treatment cannot be evaluated on the basis of cost of installation alone, because recognition of benefits is part of the decisionmaking process. Sediment deposits retained by check dams can be incorporated into a cost ratio that brings one tangible benefit into perspective. Sediment has been cited as the nation's most serious pollutant (Allen and Welch 1971). The sediment-cost ratio increases (treatment is increasingly beneficial) with dam height and decreases with increasing gradient (fig. 31). The example in figure 31 shows that a treatment consisting of loose-rock dams on a 2 percent gradient has a sediment-cost ratio larger than 1.0 for effective dam heights of 0.75 m and above.

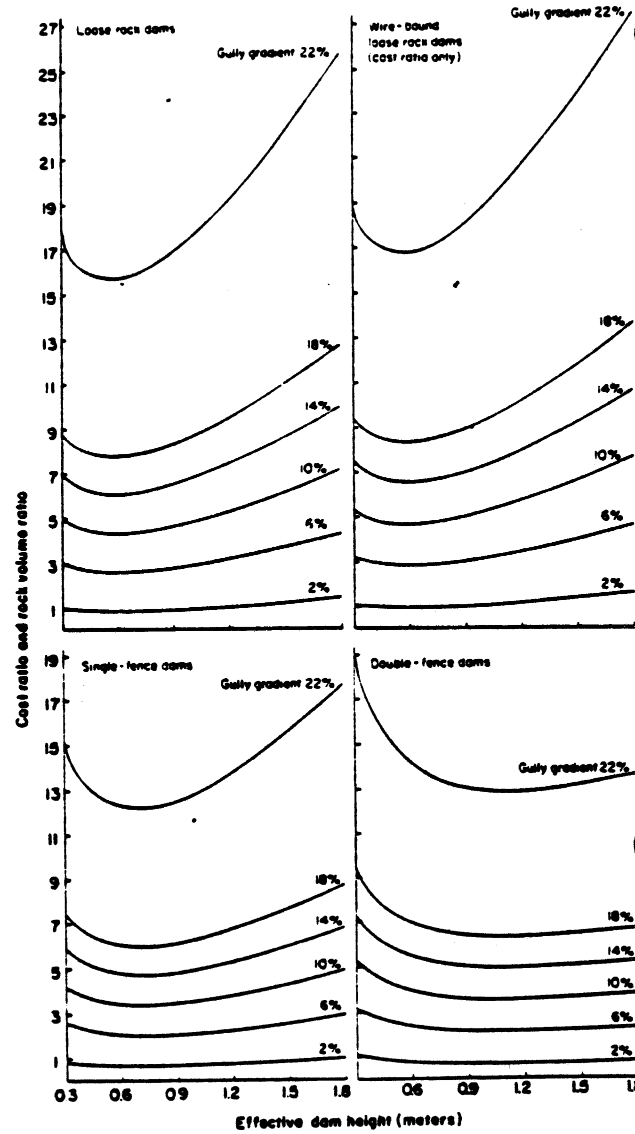
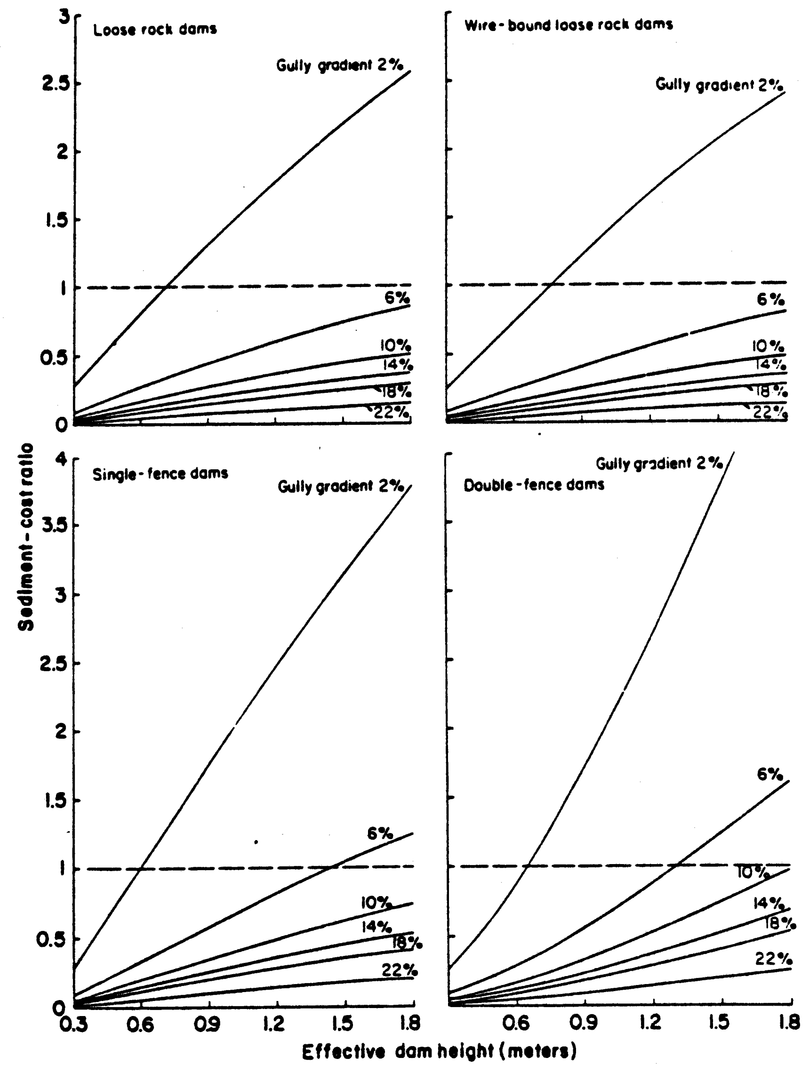


Figure 30.—Relative cost of installation of check-dam treatments and relative angular rock volume requirements in gullies with different gradients as a function of effective dam height. The cost and rock volume ratios relate the cost and rock volume of a treatment to those of a treatment with loose-rock dams 0.3 m high installed on a 2 percent gradient.

The large ratio is explained by the fact that a gully with a 2 percent gradient requires only a small number of dams (see fig. 21), while volumes of sediment deposits do not decrease significantly with number of dams or with gradient.

Since single-fence and double-fence dams cost less than loose-rock and wire-bound loose-rock dams for an effective height greater than 0.3 m, the sediment-cost ratio is more favorable for the

Figure 31.—The sediment-cost ratio relates the value of the expected sediment deposits to the cost of treatment. The graphs show this ratio as a function of effective dam height on gully gradients ranging from 2 to 22 percent. The base cost was taken as $20/m³ of angular rock dam; the value of 1 m³ of sediment deposits was assumed to be one-tenth of that cost.



fence-type structures. The ratios remain smaller than 1.0 on all gradients larger than 5 percent for treatments with loose-rock and wire-bound loose-rock dams, and on gradients larger than 7 and 9 percent for treatments with single-fence and double-fence dams, respectively.

The importance of sediment-cost ratios in relation to gully gradient and effective dam height becomes apparent in situations where not all gullies of a watershed can be treated. Gullies with the smallest gradient and largest depth, and highest possible fence-type dams should be chosen if other aspects such as access or esthetic value are not dominant.

Other Gully Control Structures and Systems

Nonporous Check Dams

Rock can be used for the construction of wet masonry dams. Limitations in available masonry skills, however, may not permit this approach. A prefabricated concrete dam was designed (Heede 1965b) and a prototype installed in Colorado (see fig. 10). It required very little time and no special skills for installation (fig. 32). The capital investment for this dam is larger than for a rock structure, however. A prestressed concrete manufacturer must be available reasonably close to the project area, and the construction sites must be accessible to motorized equipment. Where esthetic considerations and land values are high—recreational sites and parks, for example—a prestressed, prefabricated concrete check dam may be the answer.

Many different designs of concrete dams for torrent control were published in recent years. Some references are: Fattorelli (1970, 1971), Puglisi (1970), Benini et al. (1972), IUFRO-Working Group on Torrents, Snow and Avalanches (1973). Nearly all torrent dams would be overdesigned if installed in western gullies, however.

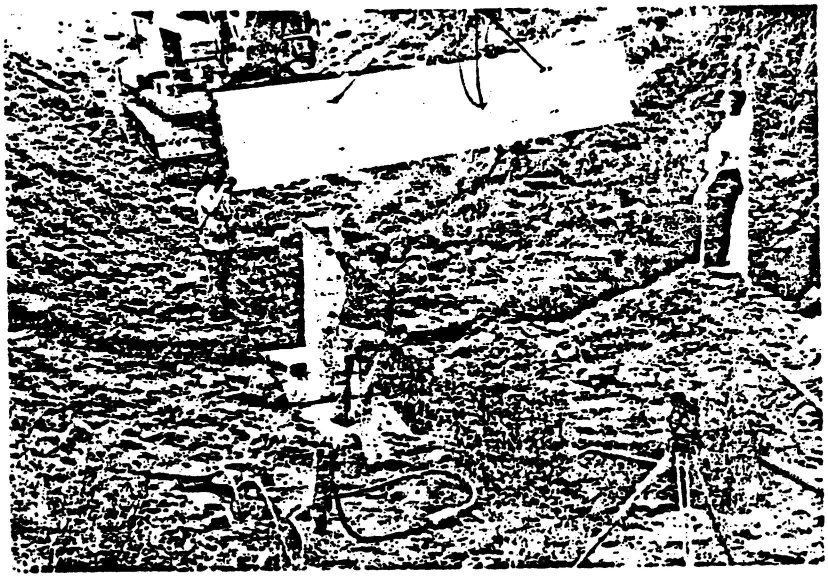


Figure 32.—Placement of a prestressed concrete slab against the buttresses of a prefabricated dam on Alkali Creek watershed. Backhoe proved to be sufficient for excavation of key and for structural installation. View is downstream.

Check dams may also be built from corrugated sheet steel. For successful application, a pile driver is required to assure proper fit of the sheets. Excavating trenches for the sheets jeopardizes dam stability if the refill is not compacted sufficiently. Quite often, insufficient depth of soil above the bedrock does not permit this dam type.

Earth Check Dams

Earth check dams should be used for gully control only in exceptional cases. Basically, it was the failure of the construction material, soil, that—in combination with concentrated surface runoff—caused the gully. Gullies with very little flow may be an exception if the emergency spillway safely releases the flow onto the land outside the gully. The released flow should not concentrate, but should spread out on an area stabilized by an effective vegetation cover or by some other type of protection such as a gravel field. Most gullied watersheds do not support areas for safe water discharge.

Standpipes or culverts in earth check dams generally create problems, because of the danger of clogging the pipe or culvert inlet, and the difficulty in estimating peak flows. Therefore, additional spillways are required.

If soil is the only dam material available, additional watershed restoration measures (such as vegetation cover improvement work and contour trenches) should be installed to improve soil infiltration rates, to enhance water retention and storage, and thus decrease magnitude and peak of gully flows.

Vegetation-Lined Waterways

With the exception of earth check dams, gully control measures described previously treat the flow where it is—in the gully. In contrast, treatments by waterways take the water out of the gully by changing the topography (figs. 33, 34). Check dams and waterways both modify the regimen of the flow by decreasing the erosive forces of the flow to a level that permits vegetation to grow. In waterways, however, flow is modified compared with the original gully, in two ways (Heede 1968a): (1) Lengthening the watercourse results in a gentler bed gradient; and (2) widening the cross section of flow provides very gentle channel side slopes. This latter measure leads to shallow flows with a large wetted perimeter (increase in roughness parameter). Both measures substantially decrease flow velocities, which in turn decrease the erosive forces.

Contrasted with check dam control, waterway projects strive to establish a vegetation cover when land reshaping is finished. Indeed, a quick establishment of an effective vegetation lining is the key to successful waterways. It follows that the prime requisites for a successful application are precipitation, temperature, and fertility of soils, all favorable to plant growth. Other requisites are:

1. Size of gully should not be larger than the available fill volumes;
2. Width of valley bottom must be sufficient for the placement of a waterway with greater length than that of the gully;

Figure 33.—Looking upstream on gully No. 6 of Alkali Creek watershed before treatment, November 14, 1961. Mean gully depth was 0.9 m and mean width from bank to bank 4.0 m.

Figure 34.—Repeat photograph of figure 33 taken on September 2, 1964, three growing seasons after conversion of gully to vegetation-lined waterway. The annual pioneer cover, consisting mainly of ryegrass (*Lolium* sp., annual variety) has been replaced by perennial herbaceous plants — smooth brome (*Bromus inermis*) and intermediate wheatgrass (*Agropyron intermedium*) are the main species.

3. Depth of soil mantle adequate to permit shaping of the topography; and
4. Depth of topsoil sufficient to permit later spreading on all disturbed areas (fig. 35).

Design criteria or prerequisites in terms of hydraulic geometry are not yet available, but the literature discussed below is relevant.

Few studies are available on flow in vegetation-lined channels or waterways. The investigation by Ree and Palmer (1949) may be a classic. They planted grasses that are widespread in the southeastern and southcentral States. Outdoor test channels and flumes were located in the Piedmont plateau, South Carolina. Permissible velocities (threshold values before beginning of erosion) were established. The study gained valuable insight into the change of the roughness parameter (n) with the growth of the grasses. The species they used do not normally grow in the West, however.

Parsons (1963), basing his work on that of Ree and Palmer (1949), established equivalent stone sizes for Bermudagrass streambank linings by relating the allowable shear stress on the grass lining to equivalent stone diameter. Useful guiding principles for successful application of vegetation for stream bank erosion control were given.

Kouwen et al. (1969) avoided the Ree and Palmer (1949) method of empirically representing the functional relationships between Manning's (n) and the relevant flow parameters. Instead,

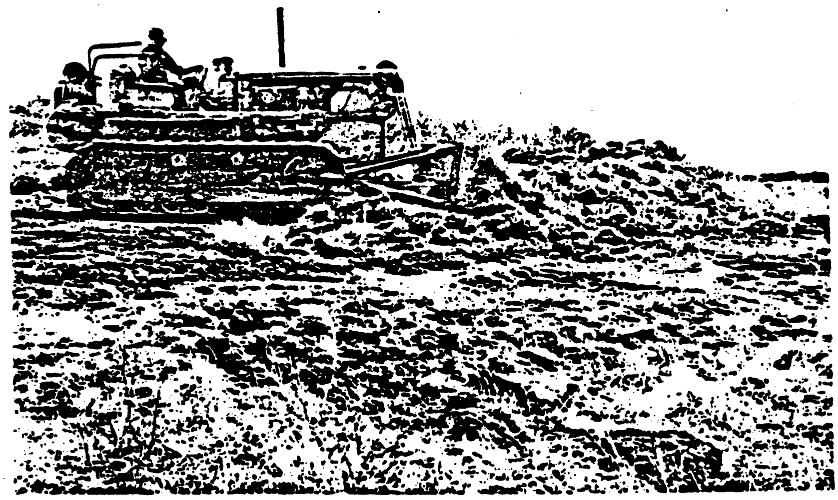


Figure 35.—Topsoil is removed from the construction area and saved, to be spread later on the finished waterway.

they derived a quasi-theoretical equation for flow and vegetation condition in a channel as follows:

$$\frac{V}{u_*} = C_1 + C_2 \ln\left(\frac{A}{A_V}\right) \qquad (44)$$

where V is the mean velocity of flow, and u_* is the shear velocity defined as $(gR_1S_1)^{1/2}$ (g represents acceleration due to gravity, R_1 is the hydraulic radius, and S_1 is the energy gradient). C_1 is a parameter that depends on the density of the vegetation, while C_2 is a parameter that depends on the stiffness of the vegetation. A is the area of the channel cross section; A_V represents the area of the vegetated part of the cross section.

The investigators could not establish design curves or tables, thus practical application is not yet feasible.

Vegetation-lined waterways require exact construction and therefore close construction supervision (fig. 36), and frequent inspections during the first treatment years. The risk, inherent to nearly all types of erosion control work, is greater for waterways at the beginning of treatment than for check dam systems. To offset this risk, in Colorado 19 percent of the original cost

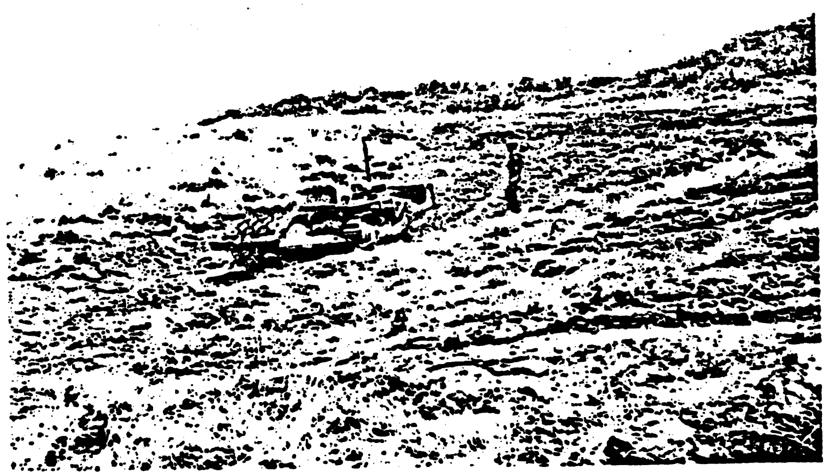


Figure 36.—A sheep-foot roller pulled by a small tractor compacts the fill in the gully. Fill was placed in layers 0.15 to 0.30 m thick.

of installation was expended for maintenance, while for the same period of time, only 4 percent was required at check dams (Heede 1968b).

Eight percent less funds were expended per linear meter of gully for construction and maintenance of grassed waterways than for check dams. This cost difference is not significant, especially if the greater involvement in waterway maintenance is recognized. In deciding on the type of gully control, one should consider not only construction costs but also risk of and prerequisites for vegetation-lined waterways.

Summary of Design Criteria and Recommendations

Spacing decreases with increasing gully gradient and increases with effective dam height (see fig. 20). Number of check dams increases with gully gradient and decreases with increasing effective dam height (see fig. 21). Expected volumes of sediment deposits increases with effective height (see fig. 22).

For practical purposes, gully gradients ranging from 1 to 30 percent do not influence volumes of sediment deposits in a treatment. On gradients larger than 30 percent, sediment catch decreases more distinctly with increasing gradient.

Rock volume requirements are much larger for loose-rock and wire-bound loose-rock dams than for fence-type dams. At effective dam heights larger than 0.6 m, treatments with double-fence dams require smallest amounts of rock (see fig. 24).

At effective dam heights larger than about 0.5 m, loose-rock and wire-bound loose-rock dams are more expensive than fence-type dams. The difference in cost increases with height (see fig. 29). Single-fence dams are less expensive than double-fence dams at effective heights up to 1.0 m.

Regardless of gradient, in a given gully, there is one effective dam height for each type of structure at which the cost of treatment is lowest (see fig. 30). For each type of treatment, rock requirements are smallest at the optimum effective dam heights for least costs (see fig. 30). The sediment-cost ratio (the value of expected sediment deposits divided by the cost of treatment) increases with effective dam height and decreases with increasing gully gradient (see fig. 31). At effective dam heights of about 0.6 m and larger, single-fence dams have a more pronounced beneficial sediment-cost ratio than loose-rock or wire-bound loose-rock dams. At effective dam heights of 1.1 m and larger, treatments with double-fence dams have the largest sediment-cost ratios (see fig. 31).

LITERATURE CITED

Allen, Paul B., and Norman H. Welch.
1971. Sediment yield reductions on watersheds treated with flood-retarding structures. Trans. ASAE 14(5):814–817.

Bailey, Reed W., and Otis L. Copeland.
1961. Vegetation and engineering structures in flood and erosion control. Int. Union For. Res. Organ., 13th Congr. [Vienna, Austria, Sept. 1961]. Proc. Pap. 11-1, n.p. [23 p.]

Beer, C. E., and H. P. Johnson.
1963. Factors in gully growth in the deep loess area of western Iowa. Trans. ASAE 6(3): 237–240.

Benini, G., S. Puglisi, G. C. Calabri, and S. Fattorelli.
1972. Opere per la correzione dei torrenti. Moderne techniche costruttive e nuovi procedimenti di calcolo. [Structures for torrent control. Modern construction designs and new stability calculations.] Publ. Dep. Sistemazioni idraulico-forestali, Univ. Padua, Italy, 172 p.

Brown, J. B.
1963. The role of geology in a unified conservation program, Flat Top Ranch, Bosque County, Texas. Baylor Geol. Stud. Bull. 5, 29 p. Baylor Univ., Waco, Tex.

Bryan, Kirk.
1925. Date of channel trenching (arroyo-cutting) in the arid Southwest. Science 62:344–388.

Clauzel, L., and A. Poncet.
1963. Barrages filtrants et correction torrentielle par ségrégation des matériaux charriés. [Filter dams and torrent control by bed load retention.] Rev. For. Fr. (Nancy) 15(4):280–292.

Copeland, Otis L.
1960. Watershed restoration. A photo-record of conservation practices applied in the Wasatch Mountains of Utah. J. Soil Water Conserv. 15(3):105–120.

Dennis, Howard W., and Ernest C. Griffin.
1971. Some effects of trincheras on small river basin hydrology. J. Soil Water Conserv. 26(6):240–242.

Evenari, Michael.
1974. Desert farmers: Ancient and modern. Nat. Hist. 83(7):42–49.

Evenari, M., L. Shanan, N. Tadmor, and Y. Aharoni.
1961. Ancient agriculture in the Negev. Science 133(3457):979–996.

Fattorelli, Sergio.
1970. Sul dimensionamento delle briglie in conglomerato cementizio. [Design of concrete check dams.] Monte e Boschi, 21:3–20.

Fattorelli, Sergio.
1971. Opere per la correzione dei torrenti nelle Alpi francesi. [Torrent control structures in the French Alps.] Monte e Boschi 22(5):33–44.

Faulkner, Patricia H.
1974. An allometric growth model for competitive gullies. Z. Geomorphol. Suppl. 21:76–87.

Ferrell, William R.
1959. Report on debris reduction studies for mountain watersheds. 164 p. Los Angeles County Flood Control Dist., Dams and Conserv. Branch, Los Angeles, Calif.

Ferrell, W. R., and W. R. Barr.
1963. Criteria and methods for use of check dams in stabilizing channel banks and beds. *In* Proc. Fed. Inter-Agency Sediment. Conf., 1963. U.S. Dep. Agric., Misc. Publ. 970, p. 376–386.

Gregory, H. E.
1917. Geology of the Navajo County. U.S. Geol. Surv. Prof. Pap. 93, 161 p.

Hadley, R. F.
1963. Characteristics of sediment deposits above channel structures in Polacca Wash, Arizona. *In* Proc. Fed. Inter-Agency Sediment. Conf., 1963. U.S. Dep. Agric., Misc. Publ. 970, p. 806–810.

Hamilton, Thomas M.
1970. Channel-scarp formation in western North Dakota. U.S. Geol. Surv. Prof. Pap. 700-C, p. C229–C232.

Harris, David Vernon.
1959. Late quaternary alluviation and erosion in Boxelder Creek valley, Larimer County, Colorado. Ph.D. Diss., Univ. Colo., Boulder. 104 p.

Hastings, James Rodney.
1959. Vegetation change and arroyo cutting in southeastern Arizona. J. Ariz. Acad. Sci. 1(2):60-67.

Heede, Burchard H.
1960. A study of early gully-control structures in the Colorado Front Range. U.S. Dep. Agric., For. Serv., Rocky Mt. For. and Range Exp. Stn., Stn. Pap. 55, 42 p. Fort Collins, Colo.

Heede, Burchard H.
1964. A pavement breaker attachment to drive steel fenceposts. J. Soil Water Conserv. 19(5):181–182.

Heede, Burchard H.
1965a. Hydraulic reclamation: A unique Italian method in watershed rehabilitation. J. Soil Water Conserv. 20:216–219.

Heede, Burchard H.
1965b. Multipurpose prefabricated concrete check dam. U.S. For. Serv. Res. Pap. RM-12, 16 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.

Heede, Burchard H.
1966. Design, construction and cost of rock check dams. U.S. For. Serv. Res. Pap. RM-20, 24 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.

Heede, Burchard H.
1967. The fusion of discontinuous gullies—a case study. Bull. Int. Assoc. Sci. Hydrol. 12:42–50.

Heede, Burchard H.
1968a. Conversion of gullies to vegetation-lined waterways on mountain slopes. USDA For. Serv. Res. Pap. RM-40, 11 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.

Heede, Burchard H.
1968b. Engineering techniques and principles applied to soil erosion control. U.S. For. Serv. Res. Note RM-102, 7 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.

Heede, Burchard H.
1970. Morphology of gullies in the Colorado Rocky Mountains. Bull. Int. Assoc. Sci. Hydrol. 15:79–89.

Heede, Burchard H.
1971. Characteristics and processes of soil piping in gullies. USDA For. Serv. Res. Pap. RM-68, 15 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo.

Heede, Burchard H.
1974. Stages of development of gullies in western United States of America. Z. Geomorphol. 18(3):260–271.

Heede, Burchard H.
1975a. Stages of development of gullies in the West. *In* Present and prospective technology for predicting sediment yields and sources. U.S. Dep. Agric., Agric. Res. Serv., ARS-S-40, p. 155–161.

Heede, Burchard H.
1975b. Mountain watersheds and dynamic equilibrium. Watershed Manage. Symp., ASCE, Irrig. Drain. Div. [Logan, Utah, Aug. 1975] Proc. 1975:407–420.

Heede, Burchard H., and John G. Mufich.
1973. Functional relationships and a computer program for structural gully control. J. Environ. Manage. 1:321–344.

Heede, Burchard H., and John G. Mufich.
1974. Field and computer procedures for gully control by check dams. J. Environ. Manage. 2:1–49.

Herold, Laurance C.
1965. Trincheras and physical environment along the Rio Gavilan, Chihuahua, Mexico. Publ. Geogr., Tech. Pap. 65-1, 233 p. Dep. Geogr., Univ. Denver.

Howe, J. W.
1950. Flow measurement. p. 177–228. *In* Hunter Rouse, ed. Engineering hydraulics. 1039 p. John Wiley & Sons, Inc., New York.

Huxley, J. S.
1954. Constant differential growth ratios and their significance. Nature 114:895–896.

IUFRO. Working Group on Torrents, Snow and Avalanches.
1973. Kolloquium über Wildbachsperren. [Colloquium on torrent dams.] [Vienna, Austria, Apr. 1972.] Mitt. Forstl. Bundes-Versuchsanst. Wien, Austria. (102), 419 p. (parts in German, others in French).

Kouwen, Nicholas, T. E. Unny, and Harry M. Hill.
1969. Flow retardance in vegetated channels. Proc. ASCE J. Irrig. Drain. Div. 95(IR2): 329–342.

Kronfellner-Kraus, Gottfried.
1970. Über offene Wildbachsperren. [Open torrent control dams.] Mitt. Forstl. Bundes-Versuchsanst. Wien, Austria. (88):7–76.

Kronfellner-Kraus, Gottfried.
1971. Untersuchungen and Messungen an Wildbachsperren. [Studies and measurements on torrent check dams.] Forstl. Bundes-Versuchsanst. Wien, Austria. Pap. presented at Int. Union For. Res. Organ., 15th Congr. [Gainesville, Fla., Mar. 1971.] 63 p.

Leliavsky, Serge.
1957. Irrigation and hydraulic design. Vol. 2, Irrigation Works. p. 330. Chapman and Hall Ltd., London.

Leopold, Luna B.
1951. Rainfall frequency: An aspect of climatic variation. Trans. Am. Geophys. Union 32: 347–357.

Leopold, Luna B., and J. P. Miller.
1956. Ephemeral streams—hydraulic factors and their relation to the drainage net. U.S. Geol. Surv. Prof. Pap. 282-A, 37 p.

Leopold, Luna B., M. Gordon Wolman, and John P. Miller.
1964. Fluvial processes in geomorphology. 522 p. W. H. Freeman and Co., San Francisco, London.

Lusby, G. C., and R. F. Hadley.
1967. Deposition behind low dams and barriers in the southwestern United States. J. Hydrol. (N.Z.) 6(2):89–105.

Miller, Carl R., Russell Woodburn, and Herschel R. Turner.
1962. Upland gully sediment production. Symp. of Bari, Comm. of Land Erosion. Int. Assoc. Sci. Hydrol. Publ. 59, p. 83–104.

Murphey, J. B., L. J. Lane, and M. H. Diskin.
1972. Bed material characteristics and transmission losses in an ephemeral stream. *In* Hydrology and Water Resources in Arizona and the Southwest, Proc. Ariz. Sect. Amer. Water. Resour. Assoc., Hydrol. Sect. Ariz. Acad. Sci. 2:455–472.

Nir, D., and M. Klein.
1974. Gully erosion induced in land use in a semi-arid terrain (Nahal Shigma, Israel). Z. Geomorphol. Suppl. 21:191–201.

Orme, Antony R., and Robert G. Bailey.
1971. Vegetation conversion and channel geometry in Monroe Canyon, southern California. Assoc. Pac. Coast Geogr. Yearb. 33:65–82.

Parkin, A. K.
1963. Rockfill dams with inbuilt spillways. Part I. Hydraulic characteristics. Dep. Civil Eng., Univ. Melbourne, Australia, 88 p. (mimeogr.).

Parsons, Donald A.
1963. Vegetative control of streambank erosion. *In* Proc. Fed. Inter-Agency Sediment. Conf., 1963. U.S. Dep. Agric., Misc. Publ. 970, p. 130–136.

Patton, Peter C., and Stanley A. Schumm.
1975. Gully erosion, northwestern Colorado: A threshold phenomenon. Geology 3(2):88–90.

Peterson, H. V.
1950. The problem of gullying in western valleys. p. 407–433, *In* P. D. Trask, ed., Applied sedimentation. 707 p. John Wiley & Sons, Inc., New York.

Peterson, H. V., and R. F. Hadley.
1960. Effectiveness of erosion abatement practices on semiarid rangelands in western United States. Bull. Int. Assoc. Sci. Hydrol. 53:182–191.

Piest, Robert F., Joe M. Bradford, and George M. Wyatt.
1973. Soil erosion and sediment transport from gullies. Am. Soc. Civil Eng. Annu. and Natl. Eng. Meet., [New York City, Oct.–Nov. 1973] 20 p. (mimeogr.).

Piest, R. F., J. M. Bradford, and R. G. Spomer.
1975. Mechanisms of erosion and sediment movement from gullies. *In* Present and prospective technology for predicting sediment yields and sources, U.S. Dep. Agric., Agric. Res. Serv., ARS-S-40, p. 162–176.

Poncet, A.
1963. Tendances autrichiennes en matière de calcul de barrages de correction torrentielle. [Austrian approaches to stability calculations of torrent control dams.] Rev. For. Fr. (Nancy) 15(3):208–216.
Poncet, A.
1965. Notes sur la lutte contre l'érosion et l'amenagement des bassins versants montagnards au nord de la Méditerranée. [Notes on erosion control and watershed management in the mountains north of the Mediterranean.] Rev. For. Fr. (Nancy) 17(10):637–661.

Puglisi, Salvatore.
1967. L'impiego di dispositivi filtranti nella correzione dei torrenti. [The use of filter dams in torrent control.] L'Italia Forestale e Montana 22 (1):12–24.
Puglisi, Salvatore.
1970. Rapport sur la préfabrication des ouvrages de correction des torrents. [Report on the prefabrication of torrent control structures.] FAO and Eur. For. Comm., 9th Sess. [Munich, June 1970] 30 p. (mimeogr.).
Ree, W. O., and V. J. Palmer.
1949. Flow of water in channels protected by vegetative linings. U.S. Dep. Agric., Tech. Bull. 967, 115 p.

Renfro, Graham W.
1972. Sediment control measures and effects, southern Great Plains. *In* Control of agriculture-related pollution in the Great Plains. Water Resour. Comm., Semin., [Lincoln, Nebr., July 1972] Great Plains Agric. Counc. Publ. 60, p. 41–48.
Richardson, H. L.
1945. Discussion: The significance of terraces due to climatic oscillation. Geol. Mag. 82: 16–18.
Ruby, Earl C.
1973. Sediment trend study, Los Angeles River watershed. An analysis of the response of Dunsmore Canyon to check dam treatment. USDA For. Serv., Calif. Reg., Angeles Nat. For., Pasadena. 130 p. (mimeogr.).
Schumm, S. A.
1960. The shape of alluvial channels in relation to sediment type. U.S. Geol. Surv. Prof. Pap. 352-B:16–30.
Schumm, Stanley A.
1969. River metamorphosis. Proc. ASCE, J. Hydraul. Div., 95(HY1):255–273.
Schumm, S. A., and R. F. Hadley.
1957. Arroyos and the semi-arid cycle of erosion. Am. J. Sci. 255:161–174.
Schumm, S. A., and R. F. Lichty.
1963. Channel widening and flood plain construction along the Cimarron River in southwestern Kansas. U.S. Geol. Surv. Prof. Pap. 352-D:71–88.
Schumm, S. A., and R. W. Lichty.
1965. Time, space, and causality in geomorphology. Am. J. Sci. 263:110–119.
Seginer, Ido.
1966. Gully development and sediment yield. J. Hydrol. (Amsterdam) 4:236–253.
Thompson, James R.
1964. Quantitative effect of watershed variables on rate of gully-head advancement. Trans. ASAE 7:54–55.
Thornes, John B.
1974. Speculations on the behavior of stream channel width. Discuss. Pap. 49, 17 p. Grad. School Geogr., London School Econ., Engl. (mimeogr.).
Tuan, Yi-Fu.
1966. New Mexican gullies: A critical review and some recent observations. Ann. Assoc. Am. Geogr. 56(4):573–597.
Vanoni, Vito A., and Robert E. Pollak.
1959. Experimental design of low rectangular drops for alluvial flood channels. Calif. Inst. Technol. Tech. Rep. E-82, 122 p.
Woldenberg, Michael Y.
1966. Horton's laws justified in terms of allometric growth and steady state in open systems. Geol. Soc. Am. Bull. 77:431–434.
Woolhiser, David A., and Carl R. Miller.
1963. Case histories of gully control structures in southwestern Wisconsin. U.S. Dep. Agric., Agric. Res. Serv., ARS 41-60, 28 p.
Woolhiser, David A., and Arno T. Lenz.
1965. Channel gradients above gully-control structures. Proc. ASCE, J. Hydraul. Div. 91(HY3):165–187.
Zimmerman, R. C., J. C. Goodlett, and G. H. Comer.
1967. The influence of vegetation on channel form of small streams. Symp. River Morphol., Int. Assoc. Sci. Hydrol. Publ. 75, p. 255–275.

SYMBOLS

α = angle corresponding to the gully gradient.
A = area of the channel cross section.
A_R = angle of repose of rock.
A_V = area of the vegetated part of the cross section.
B_A = breadth of loose rock or wire-bound loose-rock dams, measured at one-half of the depth of the spillway.
B_{SF} = breadth of single-fence dams, measured at one-half of the depth of the spillway.
C = discharge coefficient, taken at 1.65.
C^1 = constant whose value depends on the watershed configuration.
C_1 = parameter depending on density of vegetation.
C_2 = parameter depending on stiffness of vegetation.
c = constant whose value changes with groups of gully gradients.
c_1 = constant in Huxley's growth law.
D = depth of gully.
D_{65} = sieve size which allows 65 percent of rocks to pass through.
d = constant whose value changes with groups of gully gradients.
d^1 = constant in Huxley's growth law.
E = advancement rate of the gully.
f = constant whose value changes with groups of gully gradients.
G = gully gradient in percent.
g = acceleration due to gravity, taken as 9.81 m/s^2.
γ = specific weight of the fluid.
H = head of flow above weir crest.
H_D = total height of dam.
H_E = effective height of dam, the elevation of the crest of the spillway above the original gully bottom.
H_S = depth of spillway of a dam installed in a rectangular or trapezoidal gully.
H_{SV} = depth of spillway for a dam installed in a V-shaped gully.
K = constant, referring to the expected sediment gradient.
L = effective length of the weir.
L_A = average length of dam.
L_{AS} = effective length of spillway.
L_B = bottom width of the gully.
L_{BS} = bottom length of the spillway of a dam installed in a rectangular or trapezoidal gully.
L_{BSV} = bottom length of the spillway of a dam installed in a V-shaped gully.
L_{HE} = average length of dam.
L_U = width of the gully between the gully brinks.
L_{US} = length between the brinks of the spillway of a dam installed in a rectangular or trapezoidal gully.
L_{USV} = length between the brinks of the spillway of a dam installed in a V-shaped gully.
M_L = length of the wire mesh of a wire-bound dam.
M_{LB} = length of the wire mesh of the bank protection, measured parallel to the thalweg.
M_{LD} = length of the wire mesh for a double-fence dam.
M_W = width of the wire mesh of a wire-bound dam, measured parallel to the thalweg.
N_B = number of fenceposts of the bank protection work.
N_{DF} = number of fenceposts of the dam proper of a double-fence dam.
N_{SF} = number of fenceposts of the dam proper of a single-fence dam.
n = Manning's roughness coefficient.
P = wetted perimeter.
Q = rate of the peak flow in m^3/s, based on the design storm.
q = rate of the peak flow in m^3/s per unit width of spillway.
u_* = shear velocity $[(g R_1 S_1)^{1/2}]$.
R = constant, representing the depth of key.
R_1 = hydraulic radius.
ω = stream power per unit length of gully.
S = spacing of check dams.
S_1 = energy gradient.
τ = tractive force.
V = mean stream velocity.
V_A = volume of rock for the apron and bank protection.
V_c = critical velocity at dam crest.
V_{HC} = volume of a headcut control structure.
V_{DF} = volume of the dam proper of a double-fence dam.
V_K = volume of the key.
V_{LR} = volume of the dam proper of a loose-rock dam.
V_O = approach velocity of flow.
V_S = volume of sediment deposits above check dams.
V_{SF} = volume of the dam proper of a single-fence dam.
V_{SP} = volume of the spillway of loose rock and wire-bound loose-rock dams.

V_{SDF} = volume of the spillway of a double-fence dam.
V_{SSF} = volume of the spillway of a single-fence dam.
W = weight of rock related to D_{85}.
w = flow width.
X = size of a biological organ.
x = horizontal coordinate of a point on the trajectory, here the horizontal distance between the downstream side of the spillway and the point where the waterfall hits the apron.
y = size of an organism.
Y_c = critical depth of flow at dam crest.
z = vertical coordinate of a point on the trajectory, here the effective dam height.

Heede, Burchard H.
1976. Gully development and control: The status of our knowledge. USDA For. Serv. Res. Pap. RM-169, 42 p. Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. 80521

Gully formation is discussed in terms of mechanics, processes, morphology, and growth models. Design of gully controls should draw on our understanding of these aspects. Establishment of an effective vegetation cover is the long-term objective. Structures are often required. The least expensive, simply built structures are loose-rock check dams, usually constructed with single- or double-wire fences. Prefabricated concrete dams are also effective. Functional relationships between dams, sediment catch, and costs, as well as a critical review of construction procedures, should aid the land manager in design and installation of gully treatments.

Keywords: Gullies, Erosion, geomorphology, erosion control, dams.

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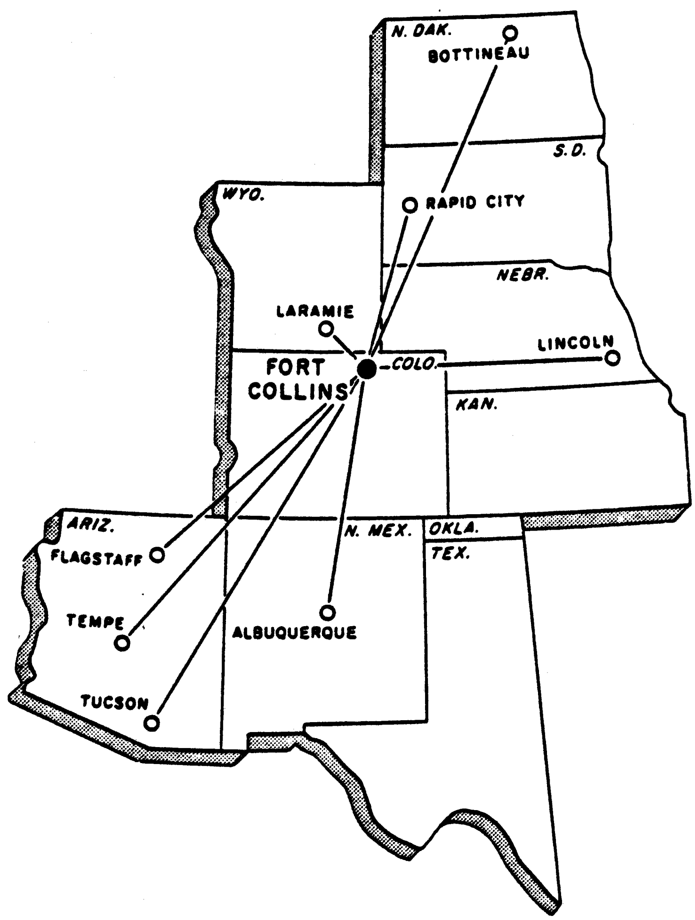
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RESEARCH

Designing Gully Control Systems for Eroding Watersheds

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ABSTRACT / Effective design of gully control systems must consider the gully network as a whole and be based on geomorphologic indicators such as type of network, stream order, and stage of development. Consideration of geomorphologic characteristics allows a ranking of gully treatment priorities that, in turn, promises the highest return for expenditures. Relationships between sediment catch, channel gradient, treatment cost, and height of check dams in a treatment system are presented. Return is considered within a physical rather than economic framework. Future soil savings are the main focus.

The Problem

Past research on engineered gully control emphasized the design and construction of individual structures. Yet, the individual hydraulic installation represents only one component of a treatment system; the quality of the treatment as a whole determines success or failure. One gully on a watershed cannot be singled out for treatment and the rest of the gully network neglected. Recent research has demonstrated strong relationships among the gullies in a network (Heede 1977). Furthermore, this research showed that by a combination of vegetation and engineering measures, a deteriorated watershed was restored within 12 years (Fig. 1). Perennial streamflow resumed after 7 years treatment.

Another important finding was that some tributary gullies can be controlled by vegetation management alone if their base levels are controlled by gullies that are structurally treated (Fig. 2). This finding can save considerable money, since generally only one-third of the total gully network length will require structures.

This report, therefore, treats gully control from a systems approach that combines vegetation management and structures.

Analysis of Gully Network

Before designing a restoration system, the stream network must be analyzed to determine gully types. Each type has its own characteristics that indicate the critical locations within the system and relationships among the gullies. This information is essential to a sound design.

Gully systems Three types of gully networks can be differentiated. One consists of continuous gullies only; another consists of discontinuous gullies only; and the third consists of a mixture where the discontinuous gullies may be in various stages of fusion with the network (Fig. 3).

KEY WORDS: Gully control, Soil Conservation, Erosion, Sedimentation.

Note: This report was presented at the Tenth International Congress on Sedimentology, Jerusalem, Israel, July 9–14, 1978.

In a network of continuous gullies, each gully is connected with the adjacent, downstream one. The gullies always start high up on the mountainside beginning with many small rills. Gully depth is quickly reached and maintained until the lowest segment above the gully mouth is approached. There, depth decreases rapidly along a concave profile. In the lowest segment, shallower depth leads to channel widening, to frequent deposition of sediment (Fig. 4), and to less frequent retransport of the depositions (extreme high flows). Pronounced channel changes must be expected, therefore, in this section close to the gully mouth. Since the local base level of a tributary gully is the bed level of the higher-order stream at the point of junction, changes of this bed level invariably influence events in the tributary.

If a gully system consists of discontinuous gullies only, the term network may not be entirely appropriate, although these gullies also occur in series along the length of the drainageway, forming a system (Heede 1960). In contrast to the continuous gully, the discontinuous counterpart begins its downstream course with an abrupt headcut at any location on the mountainside or valley floor (Fig. 5). The depth decreases rapidly downstream (Fig. 6), and thus develops a gully bottom gradient much gentler than that of the original valley floor. Where both gradients intersect, a sediment fan develops. This fan represents the local base level of the gully. If the fan becomes overly steep, the threshold value for safe conveyance of the flow is surpassed, and a headcut and new discontinuous gully will form (Patton and Schumm 1975). Headcuts advance upstream, fusing the discontinuous gullies (Fig. 7). With time, a continuous gully may evolve from fusion of several discontinuous gullies (Heede 1967). Thus a discontinuous gully has two main critical

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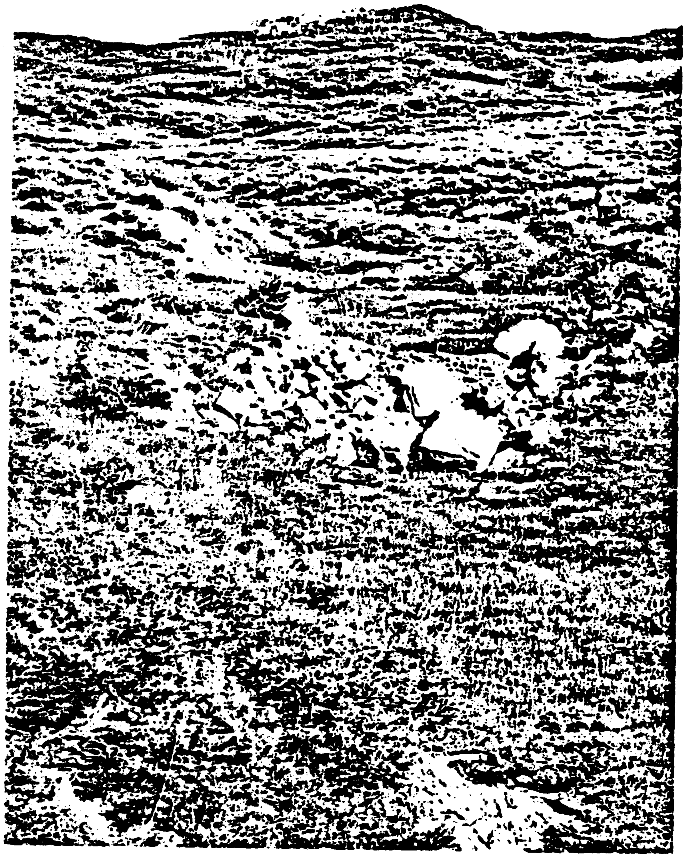

Figure 1. Twelve years after treatment at Alkali Creek watershed in the Colorado Rocky Mountains, a dense herbaceous cover protects the gully bottoms and most steep gully banks. Treatment consisted of constructing loose rock check dams, regulating grazing, and reseeding areas disturbed by construction.

Figure 2. A 25 m bank-wide tributary gully of Alkali Creek where no structures were installed but the main stem was treated by check dams 12 years ago. The well vegetated gully bottom and banks indicate that erosion has almost stopped.

locations—one at the headcut, the other at the gully mouth.

In less developed continuous gully networks, discontinuous gullies may occur in the fringe areas. Alluvial fans form below the mouths of these gullies. Where the valley bottom is narrow, limiting the shifting of flows on the fan (delta formation), rapid oversteepening of the deposits leads to the formation of a new discontinuous channel. The process recurs down the valley until the lowest channel reaches a gully of the continuous network. At the junction, the overflow into the network channel creates a headcut that in turn advances upstream and fuses the individual discontinuous gullies into one continuous channel joining the network (Fig. 3).

If discontinuous gullies are located on relatively broad valley bottoms, these processes either take a relatively long time, or the process will be modified by excessive delta formation below the gully mouth. During exceptional flows, large concentrations of flow may develop a headcut where this flow joins the next gully of the network, setting the stage for fusion with the network (Heede 1976).

As a result of the intricate processes of gully fusion, a continuous gully network may include discontinuous gullies in different stages of development. Some discontinuous channels may still be fully independent, while others

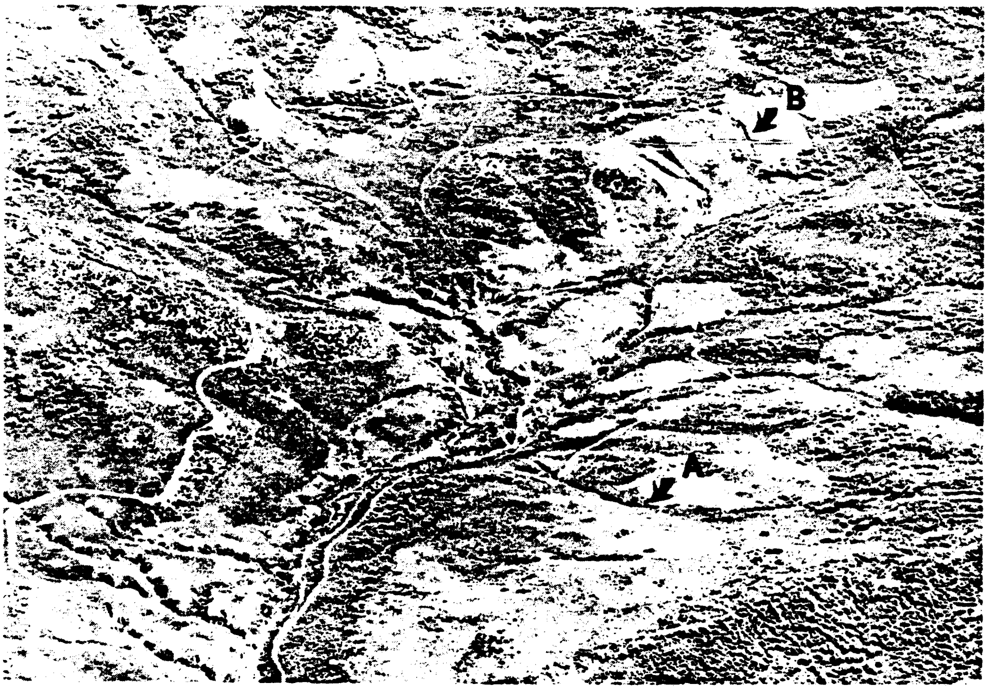

Figure 3. A gully network containing both continuous and discontinuous gullies where the discontinuous gullies are in various stages of fusion. At A a discontinuous gully has joined the network. At B a discontinuous gully is still independent of the network.

Figure 4. Upstream view of a tributary gully where heavy sediment deposits in the lowest reach have produced channel widening. A sediment fan has formed where the tributary joins the main stem because past flows were too small to remove the deposits.

Figure 5. Start of a discontinuous gully, denoted by a headcut, in the middle of a Valley floor.

Figure 6. Downstream view of a discontinuous gully. Gully depth decreases rapidly toward gully mouth. Rod in middle ground is 1.7 m high.

Figure 7. A headcut advancing upstream has fused two discontinuous gullies producing one channel and a bed scarp. Future advancing bed scarps will cut the upstream channel enough that the gradient of the upper channel adjusts to that of the lower. Stake in foreground is 1 m high.

show beginning base level changes introduced by an adjacent, downstream gully. Still others may have joined the network, but the headcut signifies its former stage. Thus, the critical locations in discontinuous gullies in different stages of development may be at different sites. These critical locations need special attention in the treatment plan.

Ranking of Gullies

Stream Ordering

Stream ordering is a system of ranking the relationships among gullies. A first-order gully does not control another tributary, but a second-order gully influences the local base level of one or more first-order gullies. The highest-order gully of a system influences all other gullies. The higher the order, the larger the number of gullies controlled by one gully. Horton's method (1945), although subjective in the selection of the headward extension of the higher-order stream, is superior for control purposes to Strahler's (1957, p. 914), which confines orders to stream segments only.

Each gully should be designated by a letter and a number representing the number of tributaries dependent on the gully (such as F_3 or K_{14}). Tabulation of the gullies by stream orders and dependent tributaries gives a measure of the impact of one gully on others. Figure 8 presents a sample gully network and Table 1 is based on this network.

Stage of Development

Stages of gully development must be recognized in the design of control measures, because these stages indicate future changes in channel morphology and expected rates of erosion (Heede 1974). The second step in ranking network gullies should, therefore, be based on stages of development, determined not only from aerial photographs but also verified at the site. This step brings a new aspect into the rank evaluation, because it considers future erosion and the benefit to be derived from treatment. The best way to rank gullies is to compare the channel morphology of the network gullies. Three or more stages may be differentiated (for example, young, mature, and old, each signifying relatively large,

Table 1. Stream orders of gullies shown in Fig. 8.

Stream orders				
1	2	3	4	0*
A	F_3	B_6	K_{14}	I
C	N_1			Q
D				
E	P_1			
G				
H				
J				
L				
M				
O				

Subscripts indicate number of tributaries.
*Gullies not fused with network.

Table 2. Stages of development of gullies shown in Fig. 1.

Young	Mature	Old
	B_6	K_{14}
I	C	A
Q	F_3	D
	G	E
		J
	H	L
	N_1	M
	P_1	O

medium, and small volumes of expected erosion, respectively). If applicable and desirable, each stage could be divided further (early mature, late mature, and so forth).

Gully classes (continuous, discontinuous, formerly discontinuous but in process of fusion) provide an objective measure to determine the critical locations where evaluation must focus. For example, pronounced headcut of a tributary indicates that a discontinuous gully joined the network. Frequent bed scarps demonstrate that gradient is still adjusting to the local base level (Fig. 9). The headcut is located only half-way upstream on the valley floor. We may conclude that a formerly discontinuous gully has reached an early mature stage, and substantial headward advance must be expected. Its young stage would have been characterized by lack of fusion within the system, while its old age stage would exhibit few bed scarps and a headcut relatively close to the drainage divide.

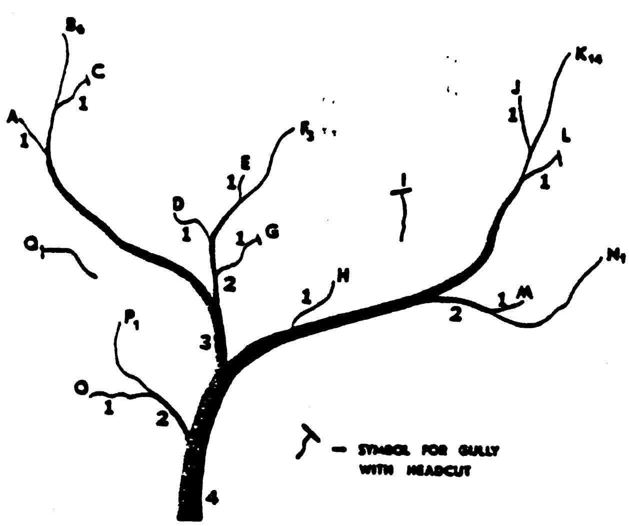

Figure 8. A schematic gully network consisting of continuous gullies, independent discontinuous gullies (I, Q), and discontinuous gullies in fusion with the network (C, G, L). Fused discontinuous gullies are indicated by the headcut symbol.

In continuous gullies, stage indicators may be found on gully bottoms and banks. Frequent and pronounced bed scarps, and beginning meandering in relatively straight reaches shown by new, steep, and undercut concave gully walls may signify youthful stage (Fig. 10). A gully with bedrock or vegetated bottom and sloped, vegetated banks could be safely classed as old age (Fig. 11); relatively small future erosion rates would be expected. Table 2 summarizes the stages of development for the network illustrated in Fig. 8.

Treatment Potential

Step three develops the final treatment priority of the individual gullies within the network hierarchy. The bases for this final ranking are stream order, number of tributaries, and stage of development (future erosion rates).

Treatment of gullies with expected large erosion rates will yield larger returns than those with insignificant rates. This relationship demonstrates that highest returns can be expected from the control of discontinuous gullies that have not yet joined the network, and whose headcuts are located in a sizable valley (Fig. 5). Gullies with sloped and vegetated banks, and bottoms on bedrock, must be classed as low priority, unless secondary considerations such as influence on tributaries call for higher priority. Such a case may exist, for instance, if local base level raises in the tributaries are desirable.

Figure 9. Severe lowering of the local channel base level has produced a bed scarp at the waterfall, which will advance upstream and cause deep cutting in the upper channel. This deep cutting causes channel widening, as indicated in the lower section of the channel.

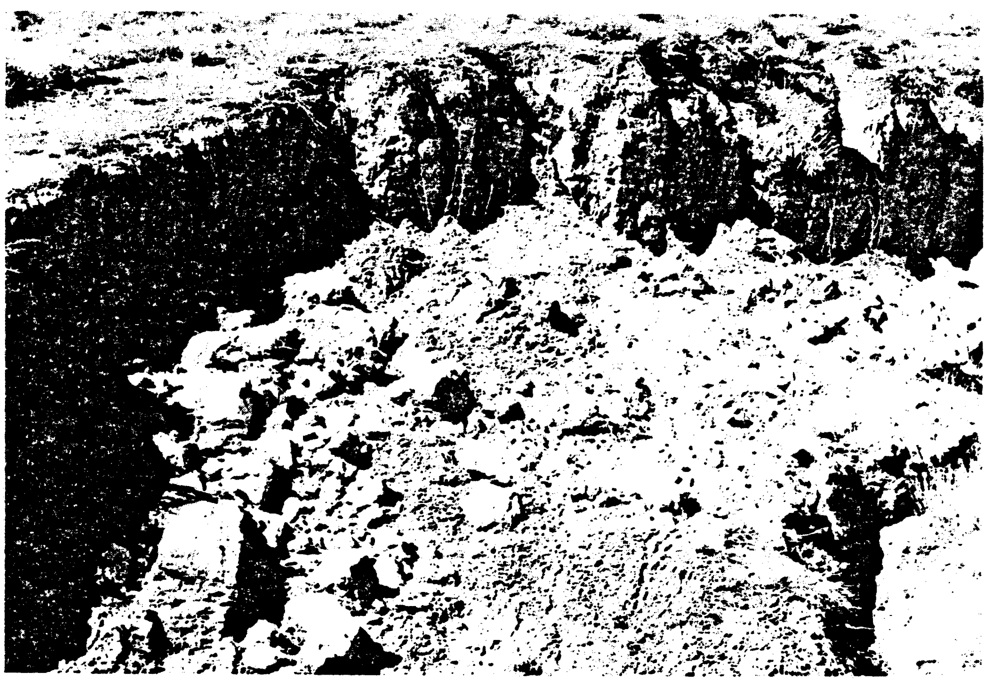

Figure 10. In young stages of gully development, meanders may undercut banks and cause bank cleavage and fall. Large amounts of sediment must be removed and bank slope gradients reduced before some kind of equilibrium can exist between flow and channel.

Figure 11. This gully bed is protected by a vigorous stand of grass and most banks are stabilized at the angle of repose. Stable banks and channel bed indicate an advanced state of gully development.

Assigning Priorities

Although all available morphologic indicators should be examined, individual judgment is needed to set treatment priorities. Priority lists allow intelligent selection of gullies for treatment with limited funds.

Table 3, based on information gained from Fig. 8 and Tables 1 and 2, illustrates both objective as well as subjective approaches to gully treatment ranking. Only the highest two priorities were recorded, because indicators suggested that effective vegetation management will lead to stabilization of the remaining gullies, once their local base levels are maintained by structurally treated gullies.

Table 3. Ranking of gullies by treatment priorities.

Priority 1	Priority 2	Headcut treatment only
I	F_3	C
Q	P_1	G
K_{14}		L
B_4		

In this case, four gullies were selected for first priority treatment. Although gully K_{14} has reached the old-age stage of development, it represents the main stem of the network. Its treatment will benefit all other gullies. Gully B_6, of next lower stream order, also has several tributaries and is still in the mature stage. Gullies I and Q are discontinuous, with headcuts located halfway up the valley bottoms. For second priority, gully F_3 was chosen because of severity of expected erosion and a greater number of tributaries compared with gully N_1. The latter could have been classed as late mature. Within the priority classes, rank decreases from top to bottom.

Table 3 also lists formerly discontinuous gullies that fused with the network but are still in the process of headcutting toward the watershed divide. Only the headcuts require treatment.

Cost Analysis

Funds available for the restoration of a depleted watershed are usually limited. To determine how many gullies can be treated with the available resources, a short-cut procedure, proposed by Heede and Mufich (1974), can be used to quickly establish an estimate of the costs of treatment with check dams. From the priority list (Table 3), representative gullies may be selected, and the few data required for Phase I of the computer program may be obtained. Considering Table 3 as an example, gully I would be selected for the discontinuous group and gully K_{14} for the gullies of the network proposed for treatment. Since costs for a headcut control structure will be part of the computer output for gully I, an estimate for headcut controls in gullies C, G, and L can be obtained. In the final selection of the gullies to be treated, at least all headcuts, all discontinuous gullies, and the mainstem gully should be included. These are given in order of treatment priority. With the exception of the mainstem, discontinuous gullies should take treatment priority over continuous gullies, because large gains can be expected from the treatment. Not all treatments need be done at

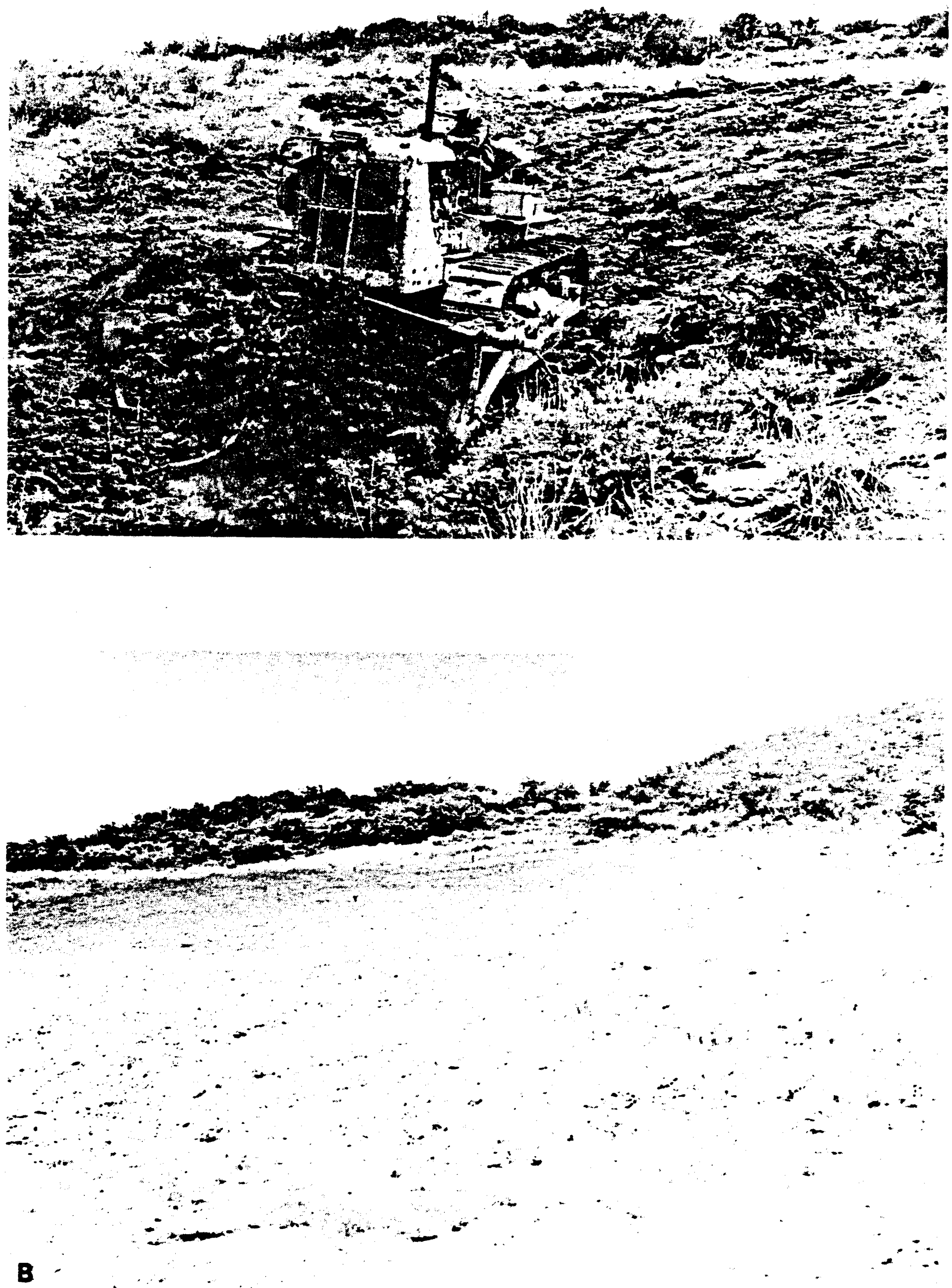

Figure 12. A discontinuous gully is converted to a vegetation-lined waterway by (a) removing brush and scarifying the gully walls and bottom, (b) filling and smoothing the bottom, and (c) planting and establishing a perennial grass cover, as indicated by this 2-year-old stand.

Figure 12. (*Continued*)

once, but it is of ultimate importance that treatment proceeds from the highest to the lower-order gullies because of local base level controls.

Discontinuous gullies, located independently of the network on broad valley bottoms and of small or medium size (not more than 3 m deep and 8 m wide), often are suitable for conversion to vegetation-lined waterways (Figs. 12a, b, c). Field tests showed that the installation of such waterways cost 8 percent less per meter of channel length than check dam treatments (Heede 1968).

Analysis of watershed Vegetative Cover Conditions and Potential for Rehabilitation

Aerial photographs, supplemented by field inspections, generally present an overall view of the different vegetation complexes of a watershed, that is sufficient for management purposes (Fig. 3). Within these complexes, areas of different vegetation densities are easily delineated on the photographs and mapped for treatment.

If feasible, planting should be considered for severely depleted sites. Reduction of grazing or temporary exclusion of the animals should be considered. Other uses, such as animal driveways, recreation, and major thoroughfares, must be adjusted to conform with the goals of vegetation management for rehabilitation.

Planting methods and fertilization requirements largely depend on the natural potentials for vegetation restoration. At harsh sites, container planting may be selected, others may require disking and rilling of seed, while broadcasting of seed may suffice on roads and areas disturbed by the treatment operations. Nutrient soil tests will yield information on fertilizer requirements.

Where infiltration data are available, rates of infiltration can be used as indices of relative potentials for overland flow and erosion. Generally, well established, dense, vigorous vegetative covers have high infiltration rates, while bare areas may shed practically all water (Dortignac and Love 1961, Meiman 1973). If possible, the most hydrologically beneficial vegetation complexes should not be disturbed during treatment in order to maintain the full potential of these sites. One may argue that these sites can recover more quicky than those with lesser potentials and, therefore, should receive preference for location of pioneer roads, etc. Yet, their role as nuclei for later vegetation increases on the watershed may outweigh this argument manyfold.

Disturbed ground surfaces must be planted after construction is finished. Generally, herbaceous species are preferable in the West, because such cover controls soil erosion faster and more efficiently than trees or brush.

Figure 13. Loose rock check dams, reinforced by wire mesh and steel posts, were installed in the headwaters of a gully control system. Before treatment at the mouth of this gully, the maximum depth of the reach was 12 m and maximum bank width 15 m.

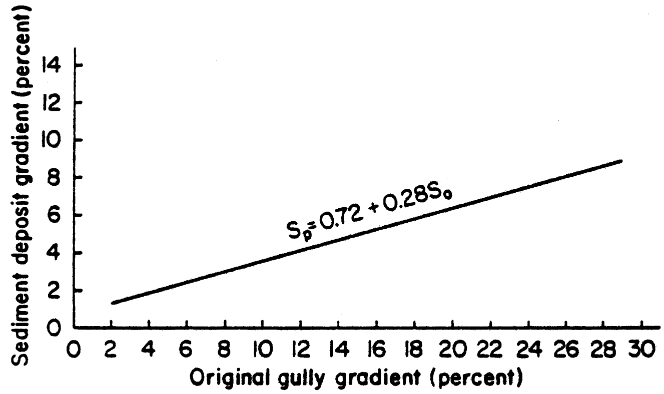

Figure 14. The relationship between original gully and sediment deposit gradients at Alkali Creek watershed, Colorado.

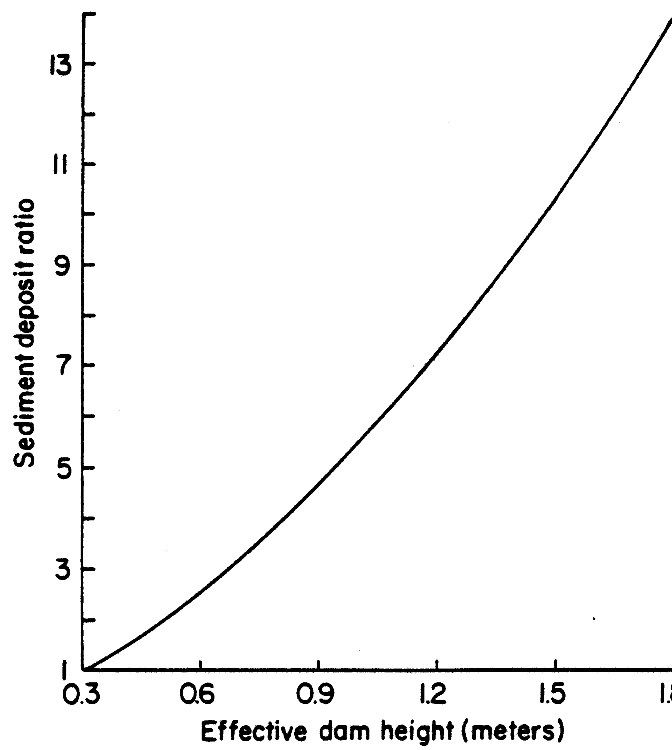

Figure 15. Expected sediment deposits retained by check dam treatment as a function of effective dam height. The sediment deposit ratio relates the volume of sediment deposits to the volume of sediment deposits at effective dam height of 0.3 m. Thus, deposits in a treatment with 1.2 m dams are more than seven times larger than those caught by 0.3 m dams.

Design of Treatments

Check Dam Systems

Rarely will a single check dam suffice for gully control. In fact, one dam installed alone may do more harm than good, because the future sediment deposits above the dam may cause a break in gradient (nickpoint) where the deposits intersect the original bed. Deposit gradients are gentler than the original gully gradient. A scarp may develop that, like a headcut, advances upstream, thereby increasing gully depth and with. Only in exceptional cases will it be possible to design for a smooth transition between deposit and gully gradients. It follows that, generally, check dams must be installed in systems (Fig. 13), proceeding upstream from the mouth until a natural control, such as given by bedrock outcrop, has been reached. Obviously, the required number of dams increases with increasing gully gradient, but decreases with increasing dam height (Heede and Mufich 1973).

The determination of dam spacing usually requires individual judgment, unless Heede's (1977) equation is applicable (Fig. 14). This equation can be used to calculate the gradient of the expected sediment deposits above check dams from the original gully gradient for projects located in the Colorado Plateau Province of western Colorado, but not necessarily elsewhere. In other regions, field surveys of older alluvial deposits above natural or artificial barriers, such as earth dams for water diversion and stock ponds, will be required to establish a ratio between original and deposit gradients. If this ratio can not be obtained confidently, Alkali Creek experience (Heede 1977) suggests it is better to use larger rather than smaller spacings. Too small spacings may prevent full utilization of the dam catchment potentials. Too large spacings, on the other hand, may cause serious bed scarps above the upstream toe of the sediment deposits. As a secondary treatment, these can easily be controlled by small structures.

Where it is advantageous to control the gradient of a given reach only, avoiding the accumulation of sediment deposits at dams, the designer may apply a submerged structure. Its spillway should be shaped like the channel, crest and side slopes level with the bed and banks, respectively. Such a structure, submerged to a certain depth, will prevent a bed scarp from advancing upstream, but will not accumulate sediment. An example of effective use of the structure is a situation in which depth cutting is expected in a tributary, caused by erosion of the higher-order streamed. The structure, installed close to the tributary's mouth, will maintain the local base level and prevent headward cutting.

Generally, conventional check dams should be designed. If sediment accumulation is one of the objectives, it must be realized that a system of high dams accumulates more deposits than one of low dams (Fig. 15). Yet, stability and cost considerations limit the heights of simply built dams. As Fig. 16 shows, loose rock dams require more rock than fence-type structures of the same height and are, therefore, more costly. Although loose rock dams could safely be built higher than the other types, the higher costs could outweigh the savings in number of dams. This is illustrated by the sediment-cost ratio, which includes expected deposits above the dams as benefit. Loose rock and wire-bound structures have less beneficial ratios than fence-type dams (Fig. 17). Yet, despite lower ratios, physical treatment requirements should take precedence over initial cost, because immediate savings may lead to high maintenance costs later.

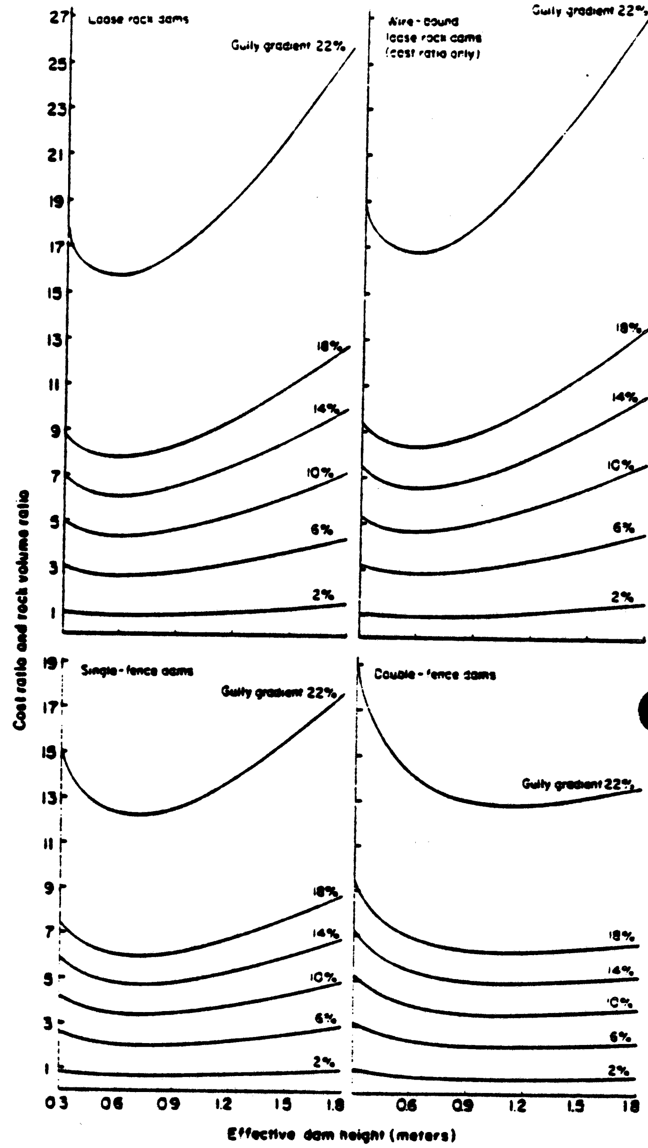

Figure 16. Relative cost of installing check dam treatments and relative angular rock volume requirements in gullies with different gradients as a function of effective dam height. The cost and rock/volume ratios relate the cost of a treatment to those of a treatment with loose rock dams 0.3 m high installed on a 2% gradient.

The Alkali Creek experiment demonstrated that loose rock and wire-bound dams are superior to fence-type structures, if an effective gradation of rock sizes is not available (Heede 1977). Under such conditions, the less expensive fence dams should not be designed, and more rock and more money will be needed.

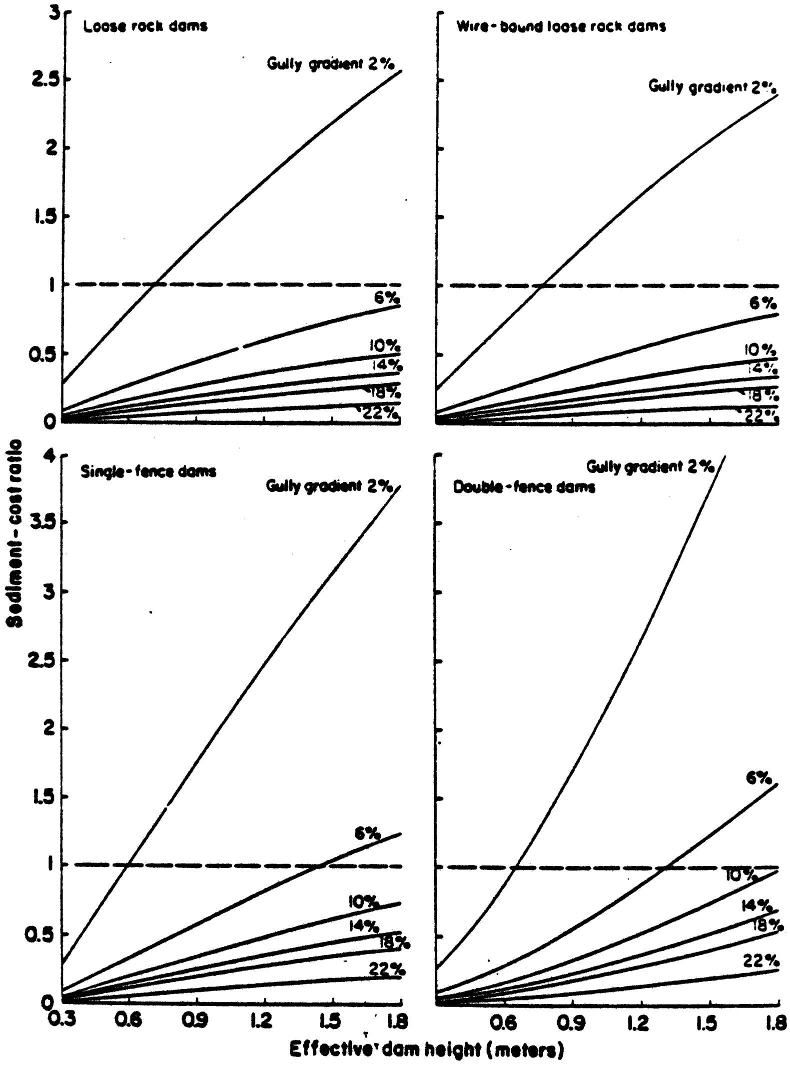

Figure 17. The sediment-cost ratio relates the value of the expected sediment deposits to the cost of treatment. The graphs show this ratio as a function of effective dam height on gully gradients ranging from 2–22%. The base cost was taken as $20/m$_3$ of angular rock dam: the value of storing 1 m$_3$ of sediment was assumed to be $2.00

Installation cost is also a function of gully gradient (Fig. 16), since steeper gradients demand more dams. Yet, more steeply sloped gullies often erode faster than gentler ones, and are in a younger stage of development. They, therefore, have high priority for treatment. Again, the physical aspects should be overriding. Where funds are seriously limited, a stepwise approach over time is preferable to a one-shot but less effective one.

For details of individual check dam design, see Heede 1965 and 1966.

Vegetation-Lined Waterways

Design of vegetation-lined waterways will be considered within a network of gullies, either consisting of continuous channels with discontinuous ones in the network fringe areas, or a system of discontinuous gullies only. Detail design criteria were published earlier (Heede 1968).

In general, discontinuous gullies are better suited for treatment by waterways than continuous gullies. The latter start high up on mountain slopes where it is more difficult to design for smooth transition into the waterway than on the valley or depression floor. Also, discontinuous gullies are often still small. If a continuous gully or former discontinuous gully of a network is treated by topographic reshaping, the design must include a check dam at the mouth of the waterway to safely discharge the flow into the network. This design may lead to undesirable flow concentrations on the waterway above the dam, because waterway cross sections should be broad and shallow to spread the flow.

Treatment of individual discontinuous gullies of the network fringe areas offers no difficulties if basic design criteria are observed and enough fill material and topsoil are available. Where these gullies appear in series, however, the sediment fans below the mouth of the gullies need special attention. These fans may show either small individual gullies or just small headcuts, marking the beginning stage of a channel. The design for reshaping the topography should, therefore, include the alluvial fans.

Vegetation-lined waterways must be designed for inefficient sediment transport, because, until an effective plant cover has been established, the reshaped ground is extremely susceptible to renewed erosion. Incompetence may best be achieved by relatively low gradients and wide cross sections that lead to small depth of flow and thus to high roughness parameters. It follows that sediment may collect in undesirable amounts on the waterway, if source areas exist nearby. The deposits may lead to flow concentrations, and maintenance must, therefore, be anticipated.

Conclusion

Effective gully control design recognizes geomorphologic indicators such as type of gully network, stream order, stage of development, and expected erosion rate. But, because knowledge of gully mechanics is limited, some degree of judgment is required to rank the gullies within the hierarchy of tributaries and to establish treatment priorities. Earlier research has shown that not all gullies of a network require treatment, if the vegetation cover is effectively managed for rehabilitation. Structural and vegetative treatments should, supplement each other.

Since gully networks can be treated stepwise over time if treatment starts at the network mouth, within limits, the physical requirements for control should override installation costs.

Discontinuous gullies should generally be ranked first priority for control, because they have greater potential for rapid growth than do continuous gullies.

Literature Cited

Dortignac, E. Y., and L. D. Love. 1961. Infiltration studies on ponderosa pine ranges of Colorado. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Stn. Pap. 59. 34 pp.

Heede, Burchard H. 1960. A study of early gully-control structures in the Colorado Front Range. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Stn. Pap. 55. 42 pp.

Heede, Burchard H. 1965. Multipurpose prefabricated concrete check dam. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Res. Pap. RM-12. 16 pp.

Heede, Burchard H. 1966. Design, construction and cost of rock check dams. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Res. Pap. RM-20. 24 pp.

Heede, Burchard H. 1967. The fusion of discontinuous gullies—a case study. Bull. Int. Assoc. Sci. Hydrol. 12:42–50.

Heede, Burchard H. 1968. Conversion of gullies to vegetation-lined waterways on mountain slopes. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Res. Pap. RM-40. 11 pp.

Heede, Burchard H. 1974. Stages of development of gullies in western United States of America. Z. Geomorphol. 18(3):260–271.

Heede, Burchard H. 1976. Gully development and control: The status of our knowledge. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Res. Pap. RM-169. 42 pp.

Heede, Burchard H. 1977. Case study of a watershed rehabilitation project: Alkali Creek, Colorado. USDA For. Serv., Rocky Mt. For. and Range Exp. Stn., Fort Collins, Colo. Res. Pap. RM-189. 18 pp.

Heede, Burchard H., and John G. Mufich. 1973. Functional relationships and a computer program for structural gully control. J. Environ. Manage. 1:321–344.

Heede, Burchard H., and John G. Mufich. 1974. Field and computer procedures for gully control by check dams. J. Environ. Manage. 2:1–49.

Horton, Robert E. 1945. Erosional development of streams and their drainage basins. Geol. Soc. Am. Bull. 56:175–370.

Meiman, James R. 1975. Infiltration studies in the Piceance Basin, Colorado. Thorne Ecol. Inst., Boulder, Colo. Tech. Pub. 12. 42 pp.

Patton, Peter C., and Stanley A. Schumm. 1975. Gully erosion, northwestern Colorado: a threshold phenomenon. Geology, Feb. 75. p. 88–90.

Strahler, A. N. 1957. Quantitative analysis of watershed geomorphology. Am. Geophys. Union Trans. 38:913–920.

Excerpt from FAO Conservation Guide
Vol. 1, Rome, Italy, 1977
Kunkle & James, eds.

Ch. VI. Evaluation of Erosion Conditions and Trend
by Thomas Dunne
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3. MEASUREMENT OF EROSION WITHIN CATCHMENTS

3.1 Sediment Sources

Sediment yields from drainage basins provide a useful index of erosion conditions and trends. Often, however, the land manager needs more detailed information about where the sediment comes from within a catchment, what the major processes are that mobilize the sediment, and what the relation is between the intensity of erosion and various possible controlling factors. The techniques used for measuring erosion within catchments vary with the erosion processes themselves. The processes to be discussed here include: sheetwash erosion; rilling and gullying; river channel changes; mass movements and wind erosion.

3.2 Sheetwash erosion

3.2.1 Plots

Current sheetwash erosion can be monitored by measuring the amount of sediment washed from hillside plots, or by measuring the rate of lowering of the ground surface at stakes. An erosion plot can be established simply by installing a collecting trough along the contour and connecting it to a tank in which the eroded sediment and runoff can be measured. Also note example given in the paper by Djorovik in this publication. The length of the trough may vary according to the wishes of the investigator. Longer troughs sample a larger section of hillslope, which minimizes errors due to the spatial variability of erosion caused by minor rills. Small plots are often defined by inserting metal or plastic walls a few centimetres into the soil. Such small plots, varying in width from about 0.5 m to 2 m are often used for studying infiltration and soil erosion under controlled artificial rainfall (35). Larger plots are usually left unbounded and are defined approximately by a topographic survey.

The construction of the collector trough is usually simple, but the trough must be carefully installed if it is to function properly. Figure 7 shows some suggested designs. The crucial factor is to provide good contact between the lip of the trough and the soil surface, so that runoff enters the trough satisfactorily and does not erode the lip and by-pass the trough. Gerlack used a small (50 cm long), sheet-metal trough placed in a shallow trench (Figure 7a). The trough has a hinged lid to prevent the ingress of rainfall, and a 3.5 cm wide lip which is pushed under the soil surface. A drain from one corner of the trough conducts sediment laden runoff to a storage tank. Such a short trough is satisfactory if there is no significant spatial variability across the slope. The ease of construction and installation allows several troughs to be installed en echelon down a hillside to study the effects of hillslope length upon erosion.

If small rills are present, the investigator may wish to use a longer trough. A collector can be constructed, as shown in Figure 7b, by driving stakes into the soil along the slope, and nailing to them 20 cm wide boards. The boards can then be used to support a collector. If the plot is only several feet long and if the cross-sectional profile of the hillslope is smooth, sections of sheet metal can be laid side-by-side and soldered together. Their uphill edge can be inserted into the topsoil as shown. If the hillside is not smooth, however, and if the collector needs to be a long one, thick industrial polyethylene sheet can be used. One edge of the sheet can be fastened to the supporting backboard and the sheet spread down the board and uphill along the ground. A short slit is made about 2 cm into the soil surface, and the edge of the polyethylene sheet is tucked into this slit with a piece of wood. The base of the polyethylene channel can then be covered with a bituminous roofing compound to keep it smooth. It is also advisable to provide for a considerable gradient on such a channel so that it drains the runoff water and sediment efficiently toward the storage tank. A plastic collector of this type is less durable than other materials, but is cheaper and easier to install if long troughs are needed. Inserting a lip under the topsoil works well if the soil has a moderately good vegetative cover, but where the root mat is not strong, the soil tends to disintegrate

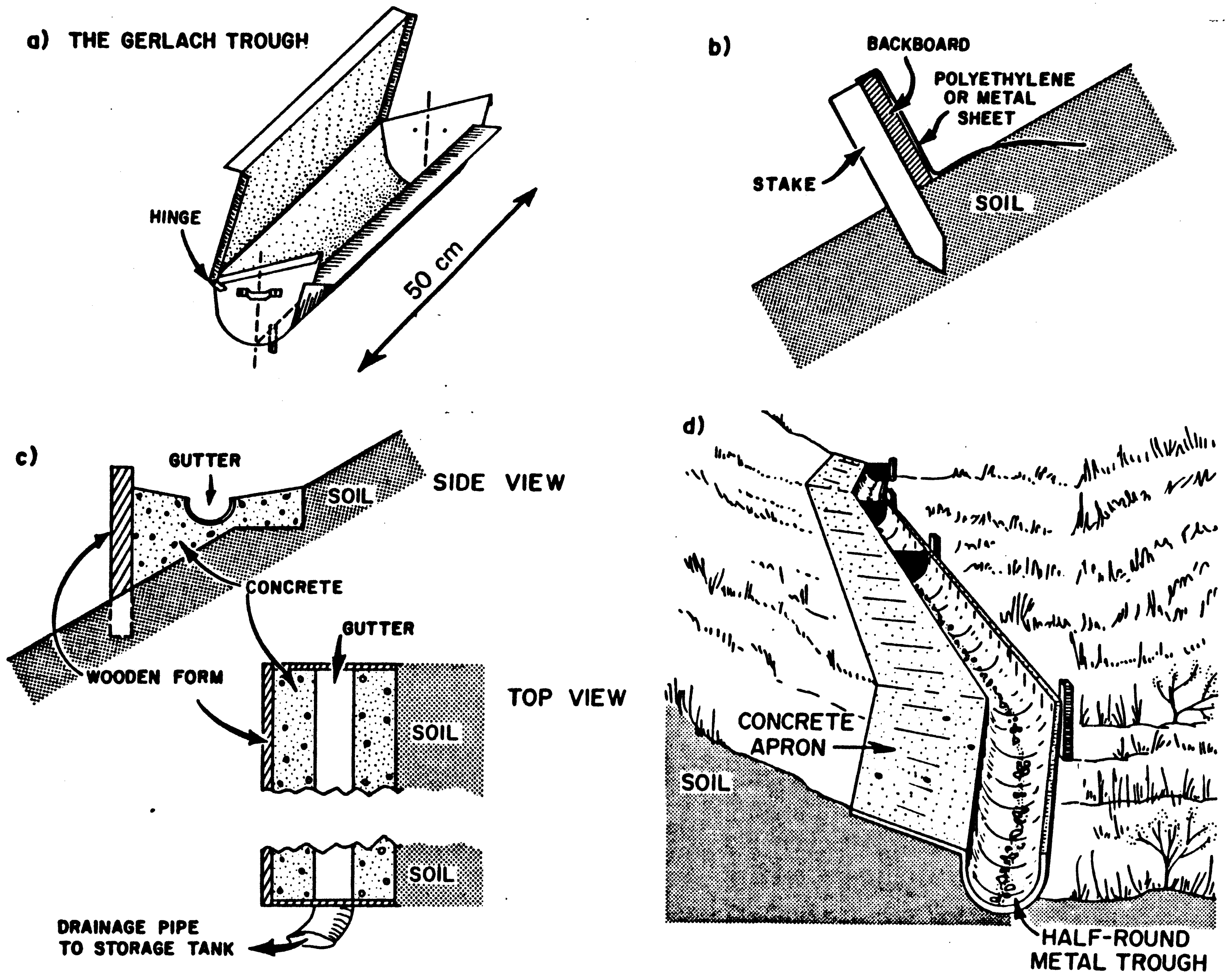


Figure 7. Collector troughs for measuring runoff and soil eroded from hillside plots. (a) Gerlach trough; (b) Channel consisting of a backboard and sheet; (c) concrete gutter; (d) large debris trough.

immediately above the trough. In such cases, it is better to install a concrete trough, as shown in Figure 7c. The width and depth of the concrete channel must vary with the magnitude of flows to be expected from the plot.

Where erosion rates are extremely high and where eroded debris may be very coarse, a larger collecting trough is required. Anderson et al (2) used half-round steel troughs to catch debris from steep hillslopes in Southern California (see Figure 7d). The upslope edge of the trough was connected to the soil surface by a concrete apron, and one-metre high reinforced wooden boards were positioned behind the trough to catch bouncing rocks.

If the plot must be large, the design of the storage tank must take into account the potentially large volumes of sediment and runoff to be collected. One inch of runoff from a hillside plot 200 feet long and 50 feet wide would total 850 cubic feet of water (one foot = 0.30 metre). Rather than construct such a large storage tank, some investigators install devices that divert only a small portion of the runoff and sediment into the measuring tank (37).

3.2.2 Stakes and Pins

An alternative method of measuring sheetwash erosion involves repeated measurement of the height of the ground surface at stakes or erosion pins. The instrumentation shown in Figure 8a consists of a 25 cm long nail and a large washer. At the time of installation, the nail and washer are driven into the soil. The distance from the head of the nail to the top of the washer is then measured with a millimetre scale. Erosion removes material from around and beneath the washer which is lowered to the position shown in Figure 8b. Remeasurement of the distance between the top of the nail and the top of the washer provides a measure of the erosion rate during the intervening period. If the washer has protected the soil from raindrop impact, so that it now stands on a small pedestal, the pedestal must be removed before measurement, so that the washer lies at the general level of the ground surface. The advantage of using the washer is that it gives a firm surface from which to measure. Such monitoring devices are cheap and easy to install in large numbers. A wide range of conditions can therefore be sampled at small cost. The most waluable illustration of the use of such data in understanding erosion processes and their relationship to the sediment budget of a drainage basin is the work of Leopold et al (33).

Marking the pins with bright red paint and placing them in a grid pattern or along a line facilitates relocation. Near each group of pins, should be an easily located bench mark, clearly identified by a numbered tag or engraved plate. The records of installation should include detailed instructions on how to locate the bench mark, and how to locate the groups of pins from the benchmark. The value of these simple measurements lies in their repetition, and the work is wasted if records of the pin locations is lost when the original investigator changes his job. At the time of each erosion-pin survey, the elevation of the top of each pin should be checked by running a line of levels from the bench mark. This will indicate whether the pins have been displaced by frost heaving or trampling.

3.2.3 Other Erosion Indicators

In addition to measuring current rates of soil loss, it is also sometimes possible to reconstruct the recent erosional history of an area from truncations of soil profiles, from the height of residual soil pedestals, or from the exposure of tree roots. A lot of care must be exercised when using these methods, and local undisturbed soils and tree roots must be examined in detail. Normal spatial variations of soil-profile depth with hillslope gradient and other factors, for example, should be considered when using measurements of truncated soil profiles. Haggett (18) measured the combined thickness of the A and B horizons of a soil on the mid-sections of slopes that had been planted to coffee in Brazil. By comparison with similar measurements under nearby undisturbed forest, he was able to show that 20 cm ($\pm$ 5 cm) of soil had been removed from the cultivated slopes since 1850. A comparison of the textures of the two sets of soil profiles confirmed this conclusion.

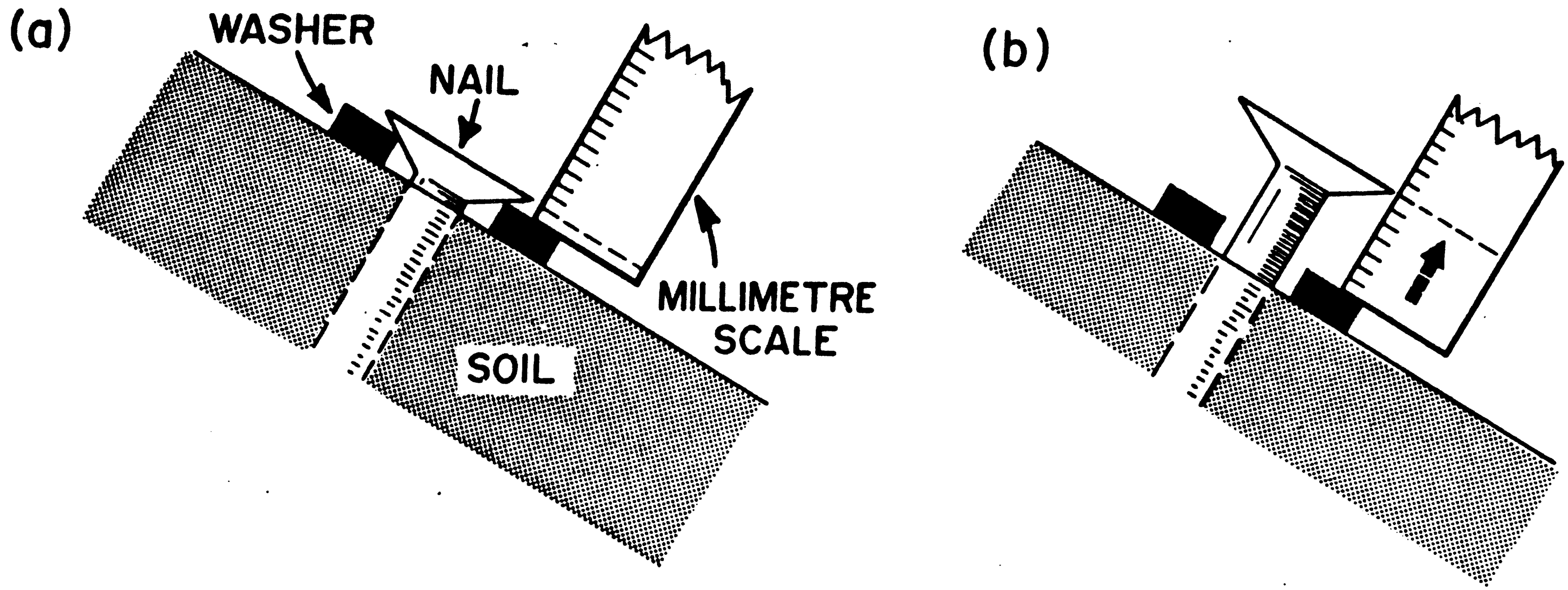


Figure 8. Measurement of erosion and deposition at stakes: (a) Installation; (b) Remeasurement.

a)

b)

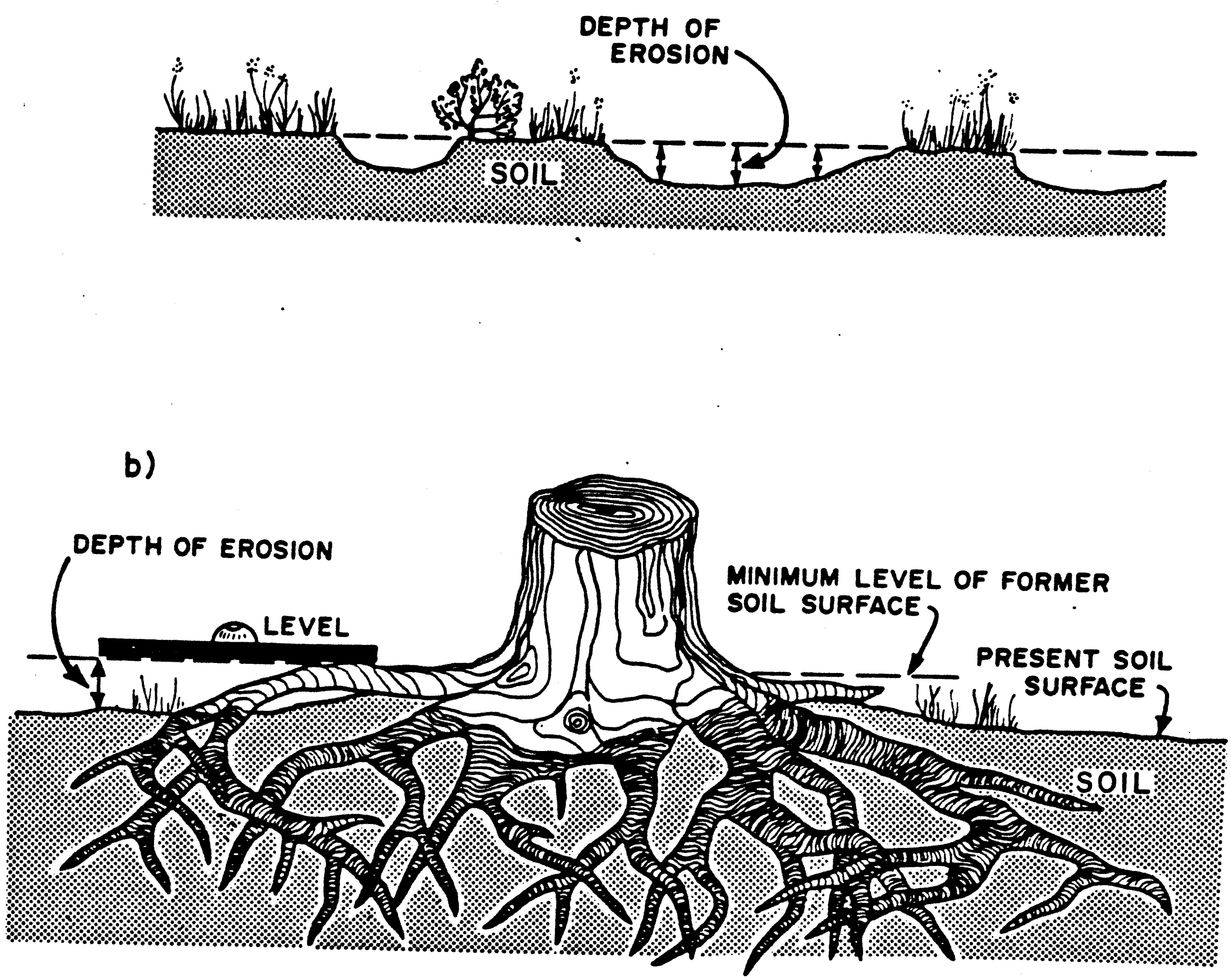
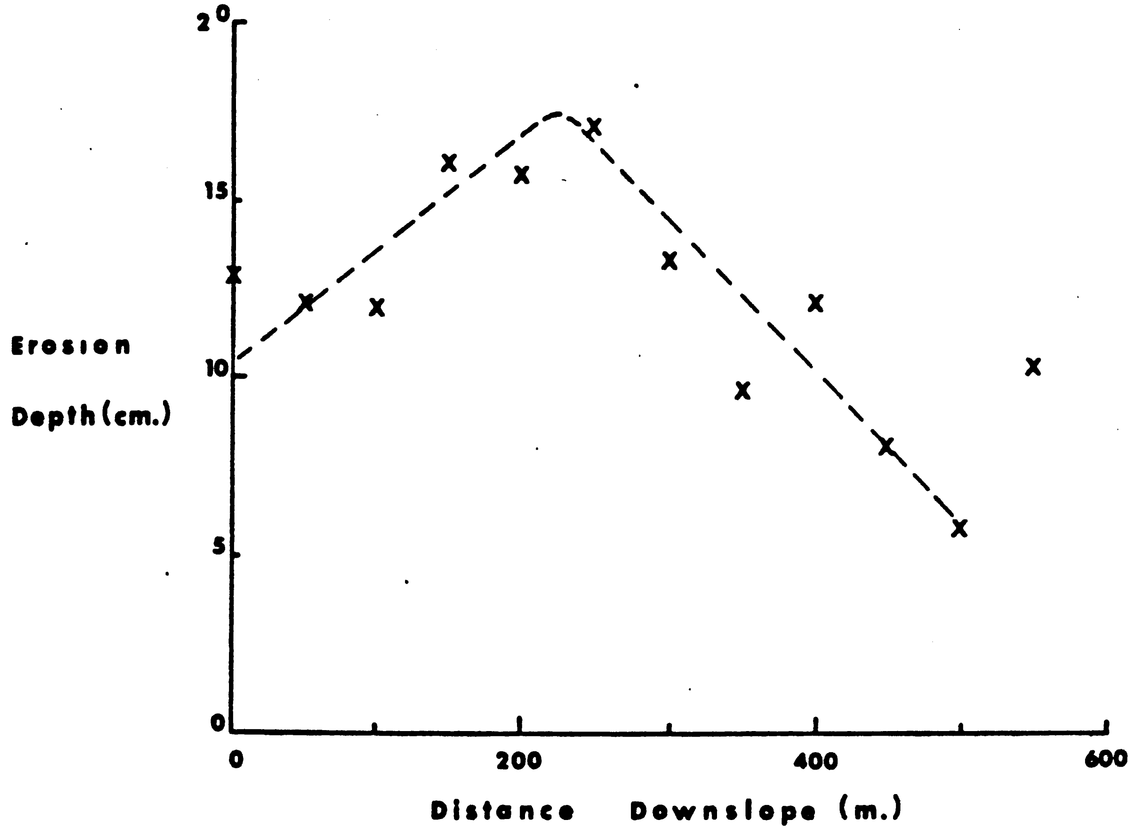


Figure 9. (a) Measurement of recent sheetwash erosion between vegetated remnants of the former soil surface; (b) Measurement of erosion around tree roots.



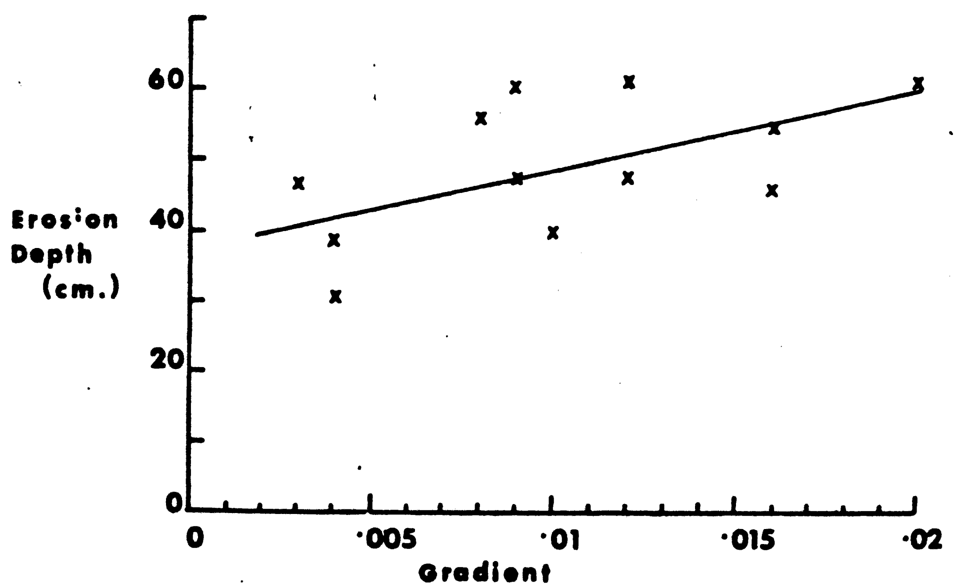


Figure 10. Depths of erosion measured from tree root exposures. Each point represents an average of 5-10 trees within a small area.

Particularly if soil erosion is rapid following the destruction of a vegetative cover, remnants of the former surface may be left, as shown in Figure 9a. A ruler, frame, or tape laid across the former surface can be used as a reference from which to measure the average depth of erosion. Various other indices, such as the average distance between remnants or the average width of remnants along a transect can also provide sensitive information that will indicate changes between repeated surveys of the same area. The exposure of tree roots can be measured, as shown in Figure 9b. Before such measurements are made, the investigator should examine roots of trees from the same species in neighbouring areas. Some trees, even on undisturbed sites, grow with part of their roots above the ground surface. The average depth of the basal flare of the trunk below ground should also be examined at undisturbed sites; in Figure 9b, only a minimum depth of erosion is indicated for a tree species whose roots lie wholly within the soil at undisturbed sites. The methodology and limitations of using tree-root exposures for measuring erosion has been discussed in detail by Lamarche (29). Measurements of root exposures at various sites produces data of the kind shown in Figure 10 and allows the investigator to relate erosion rates to their controlling variables.

A minimum date for the duration of the accelerated erosion can sometimes also be obtained from the age of the tree, which in some climates can be measured by counting the number of annual growth rings on newly-cut stumps, or in cores taken from the living tree with a Swedish increment borer (obtainable from major engineering supply houses). In many tropical regions, however, annual growth rings do not form, and the trees must be aged by some other method such as measurement of their diameter. The relationship between diameter and age can be obtained by measuring annual increments of diameter on a sample to trees. Pedestals can also be dated from the ages of long-lived shrubs which grow on them, but the techniques for aging shrubs are not as well developed as those for trees. If the tree has been cut down, the age of the surface can sometimes be obtained from aerial-photographic evidence of the data of destruction of the woodland, from local records, or from local oral tradition. Rough estimates can be made from the extent of weathering of the wood, or from the nature of the charcoal if the stump has been burnt. All such estimates would be affected by such variables as the climate and the nature of the wood (i.e. the tree species). In making the estimates, therefore, there is no substitute for local field experience.

3.2.4 Classification Approach

In some studies there is not sufficient time or manpower to make detailed measurements of erosion depths. The intensity of erosion is estimated by inspection of large areas, and is mapped onto aerial photographs. This technique usually involves the establishment of three or four catagories of erosion intensity. Areas of perhaps 10-200 acres, or individual hillslopes are then classified into one of these categories (note 1 acre = 0.4 ha). The erosion classes should be well-defined and carefully described. Photographic documentation of type localities will give other investigators, land managers and planners a clear idea of the erosion conditions represented by each category. The classification may vary with the form that erosion takes in a particular region, but it is useful to seek agreement between all the workers surveying erosion in a region. As an example, the classification adopted by the U.S. Soil Conservation Service (48) for classifying water erosion is listed below.

Class 1: Up to 25 percent of the original A horizon, or original ploughed layer in soils with thin A horizons, removed from most of the area.

Class 2: Approximately 25 to 75 percent of the original A horizon or surface soil lost from most of the area.

Class 3: More than 75 percent of the original A horizon or surface soil, and commonly part or all of the B horizon or other underlying layers, lost from most of the area.

Class 4: The land has been deeply eroded until it has an intricate pattern of moderately deep or deep gullies. Soil profiles have been destroyed except in small areas between the gullies.

Because of the association between the density of the vegetative cover and erosion rate, the vegetation can sometimes be used as an indicator of erosion if all other important controls are fairly constant. Thus, the percentage of bare soil, the canopy density, or density of ground cover have all been used as indices. They do not, of course, take into account that for a fixed cover density, erosion will be faster on steeper slopes and more erodible soils. They are, however, easy to measure repeatedly and are generally incorporated into any inventory of range condition. Although the use of erosion indicators is a relatively crude technique it can be used to define quantitatively the conditions of geology, soils, topography climate and land use which retard or accelerate erosion. Measurement of these controlling factors for each site at which erosion conditions are classified allows the investigator to study the association between various degrees of erosion and values of the controlling variables using statistical techniques that are appropriate for ordinal and nominal scale variables (20, 26). (Note also paper by Stevens in this publication).

3.3 Rilling and Gullying

If large gullies have grown or are growing rapidly in a region, their development can be measured on sequences of aerial photographs (3,44). The date of initiation of gullying can also often be established from aerial photographs. Many gully systems, however, can generate large amounts of sediment by only small enlargements of their headcuts or by minor retreat of their sidewalls. The measurement of these processes is not possible from aerial photographs. Even plane-table maps or pace-and-compass maps are not generally accurate enough for this purpose. Changes in gullies, and in smaller features such as rills, should be measured by repeated level-surveys at benchmarked cross-section and along the profile of the gully. Iron stakes driven 20 cm into the ground provide adequate benchmarks for this purpose. Widening, deepening, and the migration of headcuts can be quantified in this way. An example of the results of repeated surveys is shown in figure 11a. Several such cross-sections should be installed along the gully. The average net change at two adjacent cross-sections should be multiplied by the distance between the cross-sections to obtain the net volume of erosion or deposition.

For detailed monitoring of the behaviour of vertical headcuts, an arrangement of stakes, such as that shown in Figure 11b will allow repeated tape measurements to be made. In this way, retreat averaging a few inches per year can be measured with ease. The amount of sediment mobilized by rills can be measured either at lines of erosion pins, or by repeatedly running lines of levels along the contour between benchmarks (see Figure 12).

3.4 River channel changes

Some changes in river channels can be observed from sequences of aerial photographs. Disappearance of riparian vegetation, bank caving, or direct measurements of width and sinuousity may all provide quantitative evidence of net erosion if it is large. On the ground, repeated mapping of river channels onto aerial-photographic mosaics, or by plane-table, chaining, or pace-and-compass mapping can indicate large increases of channel width, sinuousity, or shifts of position. Lateral movement, however, and the presence of raw, undercut banks do not necessarily indicate a net loss of sediment from the valley floor. By repeated levelling of benchmarked cross-sections of a small stream, Leopold et al (33) were able to show that the sediment eroded from the undercut bank was approximately in balance with the amount of sediment deposited on the opposite streambank as the channel shifted laterally (see Figure 13). The deposited material came from the drainage basin or from the valley floor upstream, the net amount of erosion or sediment production within the surveyed reach was approximately zero. If the bank being undercut is higher than the

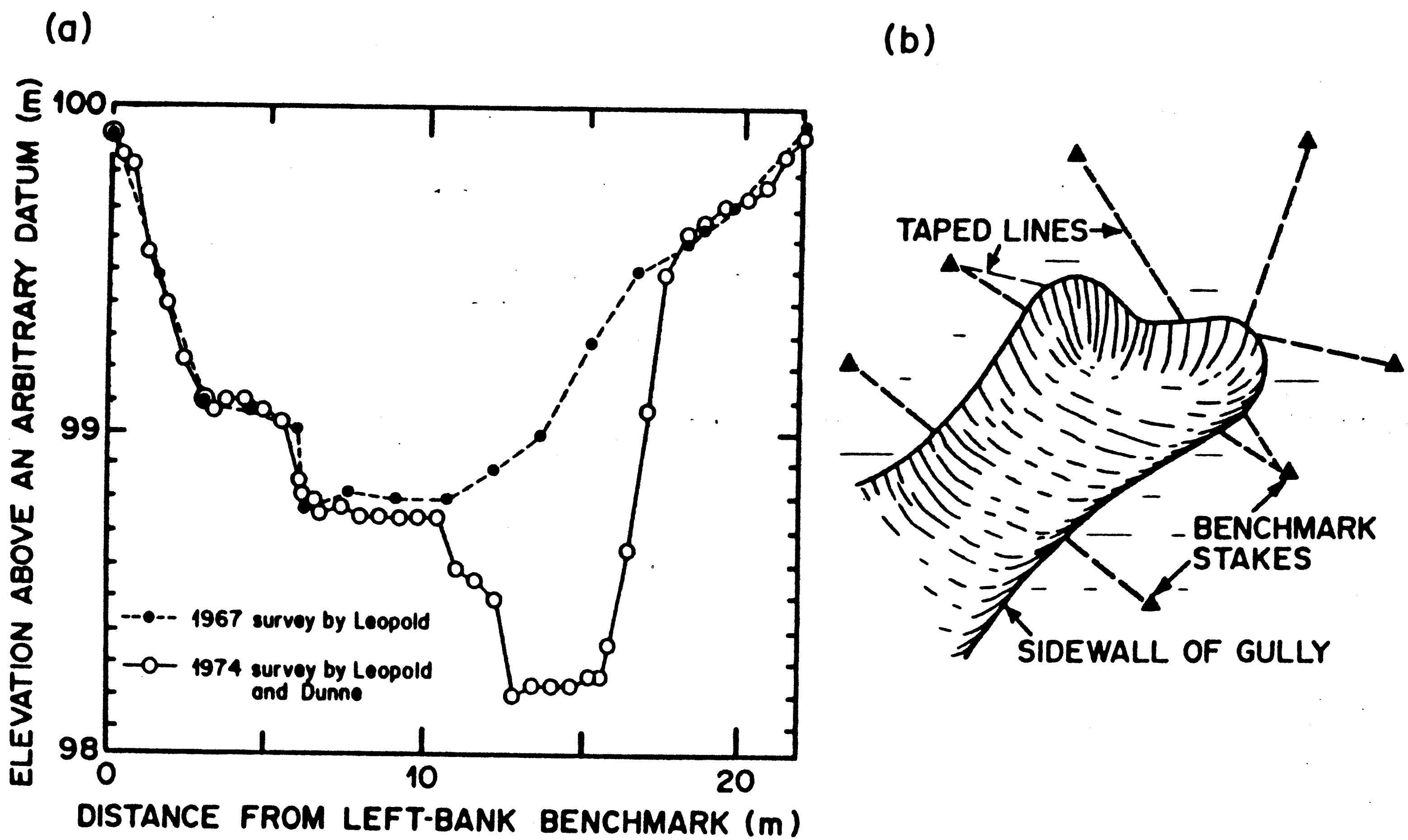


Figure 11. Measurement of gully changes: (a) by a level survey at a bench-marked cross-section; (b) by taping distances from a number of bench marks.

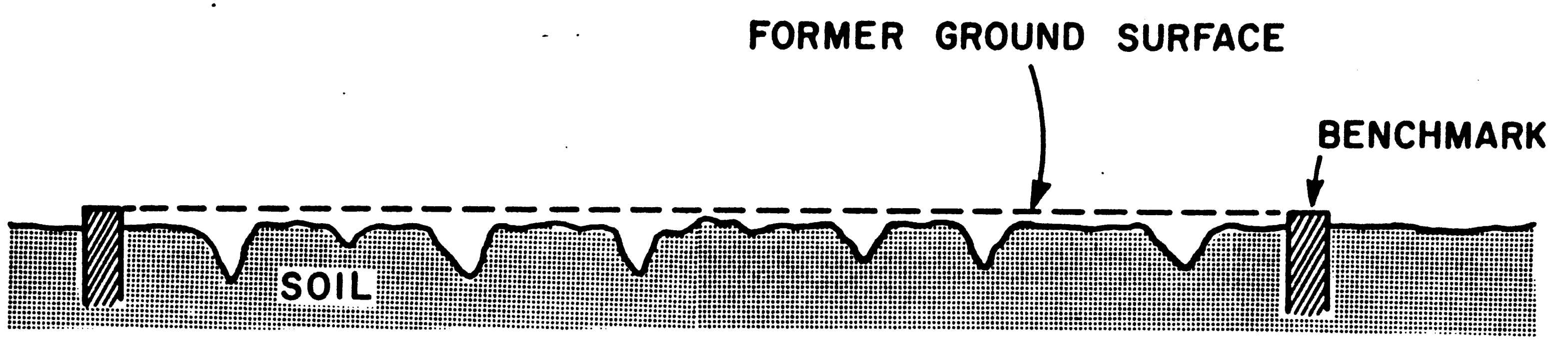


Figure 12. Measurement of rills by levelling along the contour between two bench marks.

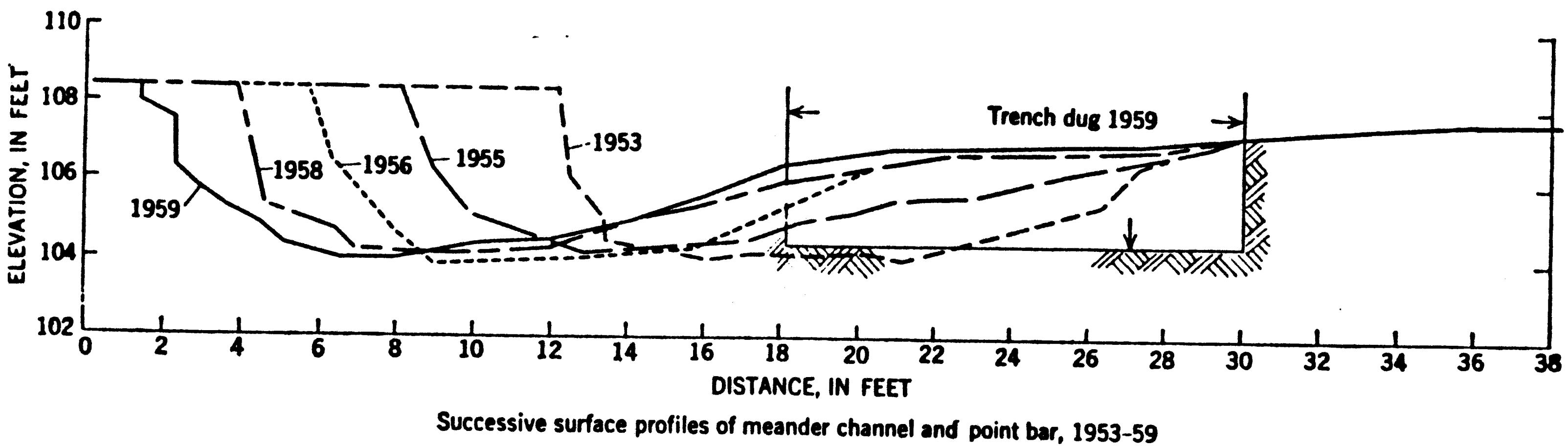


Figure 13. Repeated measurements of the channel cross-section of a stream, showing bank erosion and channel shifting without net loss of sediment. Source: Leopold et al (1964).

one being deposited, however, there is a net loss of sediment from the reach. This erosion can best be quantified by establishing several bench-marked cross sections on a stream and multiplying the average net loss of sediment from adjacent cross-sections by the distance between the sections. If bank recession is very rapid, or is highly variable, it can be monitored by repeatedly measuring the distance from the bank to lines of stakes well back from the stream. Grey (14) has provided a detailed description of methods for measuring bank erosion during snowmelt floods.

Tab 12

BIOTECHNICAL SLOPE PROTECTION AND EROSION CONTROL

Outline of presentation on Case Histories and Applications of Biotechnical Slope Protection: WATERSHED REHABILITATION IN REDWOOD NATIONAL PARK

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September, 1984

WATERSHED REHABILITATION IN REDWOOD NATIONAL PARK

I. Natural and cultural influences on erosion rates in north coastal California.

II. Sources of sediment from logged lands in the Redwood Creek basin.

III. Purpose and scope of the watershed rehabilitation program.

- A. Objectives
- B. Site selection
- C. Prescription development
- D. Primary erosion control practices
  - 1. heavy equipment
  - 2. labor intensive
- E. Secondary techniques
  - 1. control of surface erosion
    - a. terracing structures
    - b. mulches and blankets
  - 2. control of channel erosion
    - a. checkdams and submerged spillways
    - b. channel armor
    - c. other
- F. Vegetation treatments
  - 1. cuttings, wattling, seeding, transplanting, container stock
  - 2. formula for success (in the Redwood Region)
- G. Overview of erosion control treatment costs and effectiveness
- H. Before/after comparisons
- I. Time lapse movies of primary treatments (time allowing)

IV. Summary and conclusion.

RELATIVE COST-EFFECTIVENESS OF EROSION CONTROL FOR FOREST LAND REHABILITATION, REDWOOD NATIONAL PARK, NORTHERN CALIFORNIA

William Weaver[1] and Ronald A. Sonnevil[1]

I. ABSTRACT

In 1978 the U.S. Congress expanded Redwood National Park. About 35,000 acres of this area, which had previously been modified by road building and timber harvest, is now the focus of a large-scale rehabilitation program aimed at reducing accelerated erosion rates and speeding the vegetative recovery of cutover lands.

For each rehabilitation site, detailed geomorphic maps delineated natural and disturbed drainages, slope instabilities, and other erosional problems. Next, heavy equipment disaggregated and outsloped logging roads, excavated road fill from stream channels, removed unstable road fill from road prisms, and restored altered drainages to their natural patterns. After heavy equipment work was completed, labor-intensive work crews constructed erosion control structures to stabilize gullies and stream channels, minimize rainsplash erosion and rilling, and promote revegetation of disturbed areas. Checkdams, water ladders and flumes, wattling, contour trenches, wooded terraces, ravel catchers, mulches and vegetative techniques were used.

Redwood National Park, 791 Eighth Street, Arcata, CA 95521

Unfortunately, traditional cost-benefit analyses cannot be routinely applied to such forest land rehabilitation practices because soil and many other watershed amentities have little conventional net economic value. Thus, at Redwood National Park a quantitative measure of erosion control cost-effectiveness has been developed which evaluates individual techniques on the basis of treatment costs and the amount of sediment prevented from entering and being transported by stream systems.

Erosion control work at the park is divided into primary and secondary treatments. Most primary practices (those aimed at controlling erosion caused by past logging or road building activities) are accomplished at a cost-effectiveness of from \$1 to \$10/yd^3 of sediment "saved". Secondary erosion control practices (those designed to minimize erosion on areas disturbed during primary treatment) are from one to three orders of magnitude less cost-effective than primary treatments. Primary practices, typically performed by heavy earth-moving machinery, are intended to minimize stream channel erosion and landsliding while secondary treatments, consisting of heavy equipment or labor intensive work, are intended to control stream channel scour or surface erosion and promote revegetation on areas disturbed during primary treatment.

On logged land in Redwood National Park, secondary treatments used to control stream channel erosion are generally much more cost-effective than treatments designed to control surface erosion. This is primarily a reflection of the relative contribution of the two erosion processes to total sediment yield. Even treatments designed to control similar erosional problems display cost-effectiveness differences of over one order of magnitude. Whether in conjunction with the original land use or as part of subsequent rehabilitation activities, prevention is clearly the most cost-effective technique for minimizing sediment production and yield.

II. INTRODUCTION

Redwood National Park was established by P.L. 90-245 in 1968 to preserve significant examples of primeval coastal redwood forests and the streams and seashores with which they are associated. The Park's redwood forests include the world's tallest measured tree along with several others nearly as tall on alluvial flats adjacent to Redwood Creek. From the moment of Park creation, timber harvesting and road construction disturbances in the Redwood Creek watershed outside the park combined with natural processes to pose imminent threats to Park resources. Interactions between inherently unstable, highly erodible hillslopes, exceptionally severe storms and intensive land use practices caused Redwood Creek and many of its major tributaries to transport far more sediment than natural.

Roughly 90% of the forests in the Redwood Creek basin have been logged since 1945. Logging practices resulting in vegetation removal, pervasive alteration of hillside drainages, development of an extensive logging road network, and construction of thousands of miles of tractor log-skidding trails, greatly accelerated erosional processes (Janda and others, 1975). Changes included increased runoff, modification of the natural hydrologic regime and increased sediment yield. Other problems such as increased landsliding, filling and widening of stream beds, erosion of stream banks, damage to streamside vegetation, and overall degradation of natural aquatic ecosystems have been documented (Nolan and others, 1976; Harden and others, 1978).

Congress, in establishing Park boundaries in the 1968 Act, had not anticipated the magnitude of adverse impacts resulting from continued logging in the watershed upstream from parklands. In 1978, Congress amended the Redwood National Park Act through P.L. 95-250 to enlarge the Park by 19,500 ha (48,000 ac). It also mandated preparation and implementation of a watershed rehabilitation program aimed at accelerating erosional recovery of lands within the Redwood Creek basin, thereby eventually diminishing threats to Park resources (fig. 1).

This paper will review the methodology and specific techniques Redwood National Park has employed in its rehabilitation program to control accelerated erosion on lands which have been impacted by timber harvest and road building. In the second part of the paper, results from cost analyses, post-rehabilitation erosional inventories and plot studies have been combined to yield a quantitative measure of short term erosion control cost-effectiveness for forest land restoration practices.

III. GENERAL DESCRIPTION OF AREA

Redwood Creek drains a 725 km^2 (280 mi^2) watershed in the mountainous, coastal region of Northern California. The headwaters begin near elevations of 1,525 m (5,000 ft.) and the creek flows north-northwest to the Pacific Ocean near Orick, California. Through most of the parklands, Redwood Creek follows the trace of a major geologic structure, the Grogan Fault, which divides the land into two different terrain types. The basin's westside is underlain by well-foliated, mica-quartz-feldspar schist, and generally has steeper hillslopes and a higher drainage density than the east slope. In contrast, the east side is primarily underlain by pervasively sheared sandstones and siltstones and it supports more gentle slopes, loamy soil, locally large earthflow prairie areas, and a lower drainage density.

Annual precipitation in the lower one-third of the Redwood Creek basin between 1938 and 1980 has averaged approximately 2000 mm (79 in.). Four major storms (1964, 1972 (2), 1975) have occurred since logging began on most of the area. Harvesting and road construction on many sites occurred after 1964, but prior to or immediately after the storms of 1972. While the magnitudes of the 1972 storms were probably less than either the 1964 or 1975 rainfall events (Harden and others, 1978), their erosive impact appears to have been greatest, perhaps because clearcutting and tractor yarding on many of the areas had been just completed.

IV. LANDUSE AND EROSION

A significant problem associated with timber harvesting and road building in mountainous terrain is an increase in soil erosion rates and resultant sediment yield (Anderson, 1979; Kelsey, 1980; Swanson, 1981). Few places in North America display this more graphically than the Redwood Creek basin where physiographic, geologic and climatic factors, together with complex land use patterns, have contributed to exceptionally high rates of erosion (Janda and others, 1975). For example, during six years of record beginning in 1971, Redwood Creek at Orick, California, transported a mean annual suspended sediment load of 2,619 t/km^2 (7,500 ton/mi^2), 32 percent higher than the Eel River at Scotia, California (Janda, 1978), which has previously been characterized as the most rapidly eroding, non-glaciated basin of comparable size in North America (Brown and Ritter, 1969). While Redwood Creek's suspended sediment discharge has been estimated to be 8.6 times greater than the expected normal rate of delivery (Anderson, 1979), synoptic storm sampling indicates that some tributary basins displaying severe ground disruption from recent timber harvesting have yielded as much as 17 times the suspended sediment, per unit area, as nearby unharvested basins (Janda, 1978).

Logging Methods

When in private ownership, cutover lands now a part of Redwood National Park were logged by one of three methods. Early logging (c. 1930's) utilized "steam donkey" winches to yard (drag) trees over great distances to ridge top areas. Logs were then loaded and hauled to nearby mills over railroad systems. Although this early clear cut logging left behind great quantities of logging slash and removed most of the vegetation of cutover areas, soil disturbance and drainage pattern disruption were not severe.

More modern cable yarding practices required extensive road construction, yet such logging became common in Redwood Creek only after the last major storm in 1975. Field-observations and data from erosional inventories suggests that although sediment production from these recently clear cut, cable yarded hillslopes has also been negligible so far, landslides and gully erosion could become important within the next several decades.

In total, steam donkey and recent cable yarded clear cut lands comprise 23 percent of the 13,000 ha (32,000 ac.) of cutover area in the Redwood Creek portion of Redwood National Park (Best, 1984). However, most of the increased erosion from land use disturbances has been derived from 800 ha (2,000 ac.) of roaded prairie grasslands and 10,000 ha (25,000 ac.) of tractor yarded forest lands (Weaver and others, 1984).

When logging was done by tractors, felled trees were yarded downhill to the nearest logging road and loaded on trucks. In the process, yarding tractors constructed a network of interconnecting skid trails which crossed nearly every hillslope stream channel at frequent intervals. In every case, this method of logging was marked by extreme ground disturbance (fig. 2). Typically, upon completion of clear-cut operations, from 80 to 85 percent of the ground surface had been bared and roughly 40 percent of the site was covered by areas of severe ground disturbance including roads, landings and skid trails (Janda and others, 1975). The effect was a near total disruption of the microtopographic features of a site and obliteration of all but the major channels of the original drainage network (fig. 3). Compacted areas (roads, trails and landings) generated rapid surface runoff during winter storms and diverted streams find new paths over the disrupted landscape.

Erosion Problems on Logged Land in Redwood National Park

The primary erosional impacts of timber harvesting and related activities were: (1) massive soil disturbance on steep slopes logged by crawler tractors, (2) road cuts and fills which are prone to mass failure, (3) widespread alteration of natural drainage networks, (4) cutbank interception of groundwater, and an increase in impermeable bare soil areas associated with logging roads, causing increased runoff during storms, (5) surface runoff

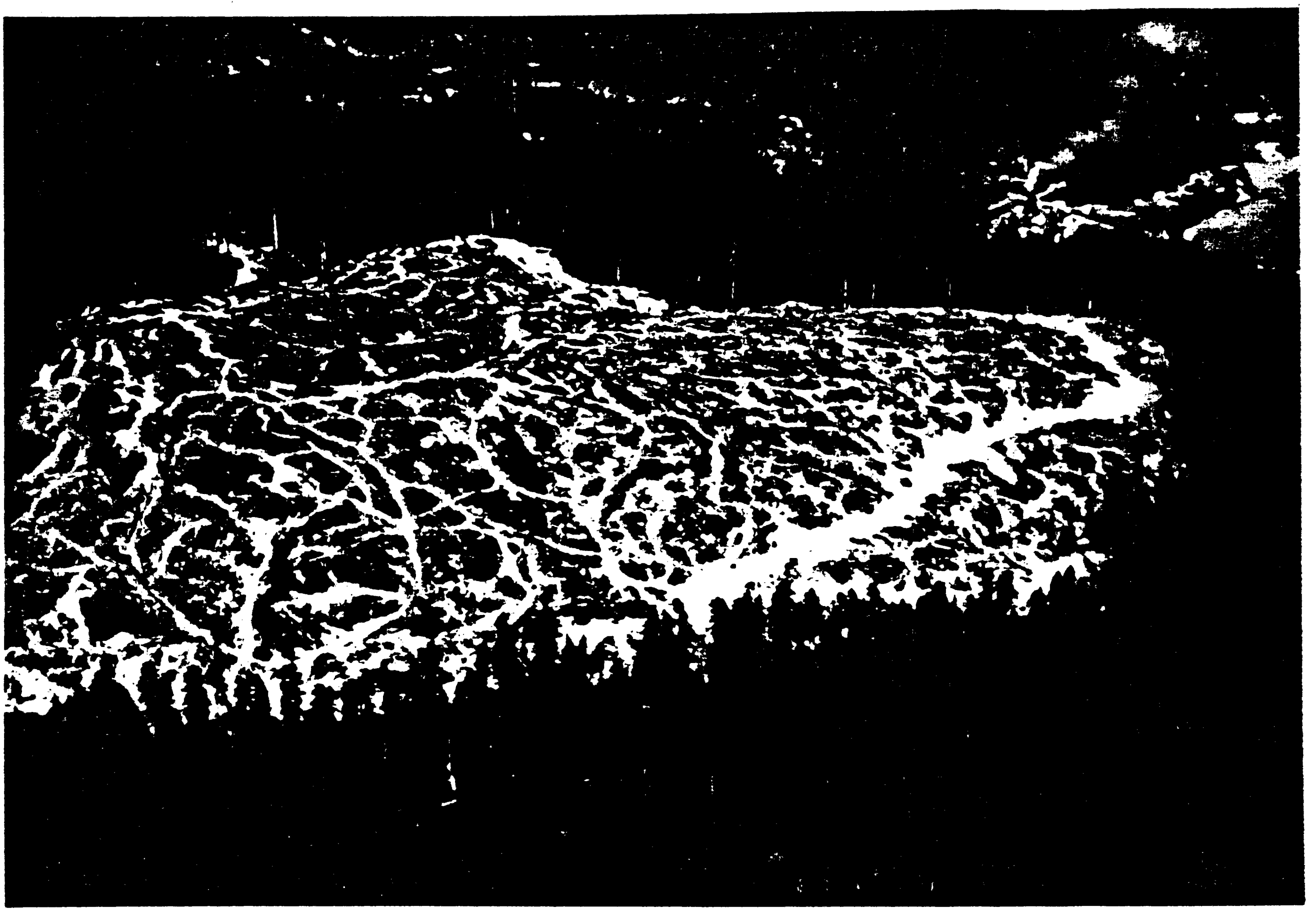

Figure 2. Oblique aerial photograph of tractor logged clear cut area now within Redwood National Park. Note logging road traversing lower slope and the intricate web of recently constructed skid trails.

Figure 3. Close-up oblique aerial view of tractor logged hillslope which was clear cut in 1975-1976. By 1978 (date of photo), areas between skid trails had already begun to revegetate. Photograph shows two road crossings and at least seven skid trail crossings of the intermittent stream which flows from top-center to bottom-center of the hillslope. Note uncut, old growth forest below the lower road.

disruption leading to development of extensive gully systems, particularly on areas with deep, fine grained soils, and (6) direct deposition of sediment and organic debris in stream channels. In addition, stream channels adjusting to the above impacts have shown an increase in streamside landsliding.

Because of the tremendous size of the trees in old growth redwood and Douglas-fir forests, tractors created deep cuts while making skid roads to drag out the logs. These deep tractor cuts on the hillslopes created major erosion problems and hampered revegetation. Skid roads frequently became gully courses when they intercepted and concentrated ground water and increased surface runoff during rain storms. The spider-web network of bulldozer skid roads severely disrupted the natural drainage network. Gullies formed from diverted streams flowing over bare hillslopes. Concentrated water also caused saturation of soil fills and led to destructive debris avalanches and debris torrents moving down steep stream channels.

Logging roads and their effect on diverting surface runoff constitute one of the greatest erosion problems on the logged watersheds in Redwood National Park (Weaver and others, 1984). Roads have increased erosion principally through the mass failure of cut and fill slopes and by diverting streams and concentrating surface erosion to produce hillslope gully systems (fig. 4). Stream side landslides, as well as slope failures triggered by road construction and logging-related disturbances, have accounted for substantial quantities of sediment directly delivered to perennial streams (LaHusen, 1984; Kelsey and others, 1984).

Redwood National Park also has 800 ha (2,000 acres) of grass prairie, and

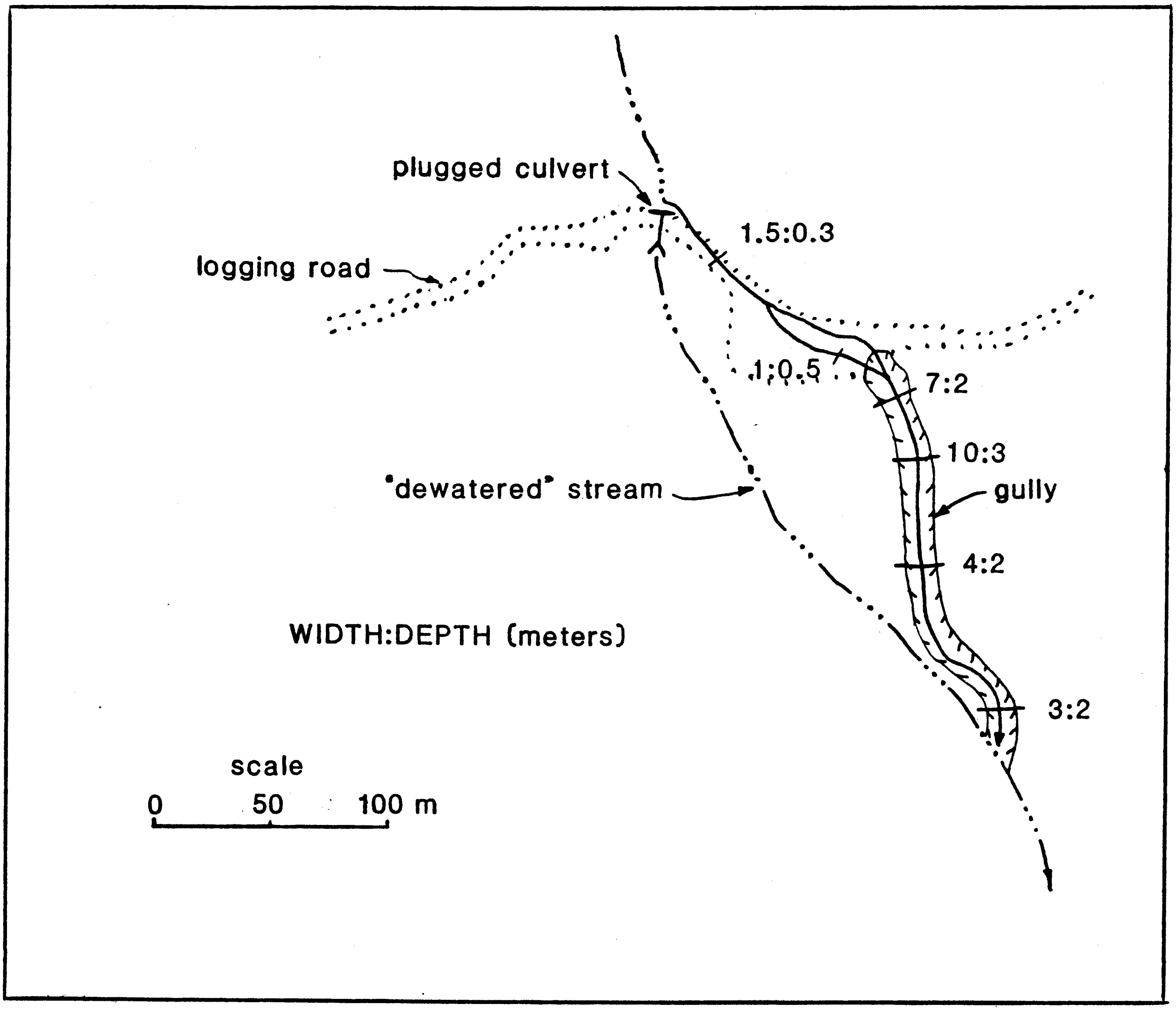

Figure 4. Planimetric map of typical gully developed on logged land in Redwood National Park. Perennial stream was diverted out of its natural channel when organic debris plugged the culvert entrance. Once water spilled over the outside of the road, a large gully developed on the clear cut hillslope below. Occassional channel cross-sections are shown. Total measured erosion from this gully was 3660 cubic yards (2800 m^3).

practically all these prairies are traversed by ranch or logging roads. Many of the roads have altered natural drainage patterns established on these erosionally sensitive slopes by intercepting and diverting upslope runoff. Gullies several tens of feet deep and thousands of feet long, as well as newly activated slumps and earthflows, formed as slopes adjusted to altered hydrologic conditions.

V. EROSION CONTROL TECHNIQUES USED IN REDWOOD NATIONAL PARK

The rehabilitation program, begun in 1978, is a multifaceted effort designed to meet the following objectives: (1) to minimize the amount of sediment delivered to stream channels from areas disturbed by logging, (2) to remove over 400 km (250 mi) of logging roads, (3) to protect or restore aquatic and riparian resources, (4) to accelerate the conversion of logged timberlands to a reasonable mimic of old growth coastal redwood forests, and (5) to encourage the prevention and control of accelerated erosion on private lands where timber is still being harvested upstream from the park (U.S.D.I., 1981a).

The rehabilitation program, expected to span approximately 15 years, is addressing each of the above objectives. During the first five years, erosion control efforts were concentrated on recently cutover terrain where severe erosional problems were found adjacent to streams or threatened to deliver sediment directly to streams. Rehabilitation on such sites progressed in three stages: (1) geomorphic mapping and erosion inventory, (2) major earth moving using mechanized heavy equipment, and (3) installation of erosion control structures and revegetation by manual labor.

The first phase, detailed geomorphic mapping, delineated surface drainage patterns and gully systems, areas of emerging groundwater, active and inactive landslides, culvert locations along roads, tractor-constructed fill crossings of stream channels, and excessive organic or inorganic debris in streams. Erosion mapping was done during the wet winter months, at a rate of roughly 6 ha (15 ac.) per person-day. This information was then used to prioritize potential work sites, prescribe site specific erosion control treatments and prepare cost estimates for the work to follow.

Heavy Equipment Rehabilitation Work

The second major stage of each rehabilitation project occurred in the dry summer months. In this phase, heavy equipment was used for major earth moving tasks. Bulldozers, dragline cranes, hydraulic excavators and backhoes removed road fill crossings from stream channels and reestablished the approximate pre-road channel configuration and gradient (fig. 5). Bulldozers and hydraulic excavators or backhoes worked in tandem or alone to excavate debris and skid trail fill from incised stream channels. Where streams had been artificially diverted by logging activity, or where groundwater emerged from cutbanks, this same equipment redirected water to natural channels. Backhoes and excavators also placed large rock in the bottom of newly excavated channels to protect the streambed from downcutting during winter flows (fig. 6).

Heavy earth moving equipment was used extensively to remove logging roads and stabilize log landings. Typically, the rock-surfaced road bench was first decompacted using chisel teeth rippers mounted on a bulldozer (fig. 7).

Figure 5. Hydraulic excavator and bulldozer completing stream crossing excavation on a former logging road. The excavator removed soil and organic debris from the stream channel while the bulldozer distributed spoil material to stable locations farther down the road.

Figure 6. Before and after photographs of skid trail crossing excavation on tractor logged hillslope. Top picture shows wedge shaped mass of soil and logging slash which was bulldozed into the channel during logging operations in 1975. In lower photo, taken after rehabilitation work was completed in 1980, debris has been removed from the watercourse. Straw mulch was applied to the bare sideslopes and the channel was armored with coarse rock fragments to control post-rehabilitation erosion.

Figure 7. Photographs showing mechanical ripping of former logging roads in Redwood National Park. Compacted road surfaces are disaggregated to a depth of 30 inches (75 cm) by several passes of large, specially equipped bulldozers. Ripping results in increased infiltration, decreased surface runoff and quicker revegetation.

Several passes of the tractor successfully disaggregated the upper 0.5 m (1.5 ft.) of material and increased the infiltration rate of the previously impermeable surface. Then, oversteepened fill material and organic debris from the outside edge of the road or landing was placed by a dragline crane or excavator, or pushed by a bulldozer, onto the stable inside half of the road bench (fig. 8). To "outslope" a road, a bulldozer then graded this material against the cutbank at a 3° to 20° gradient, obliterating the inboard ditch and directing surface runoff across the former road alignment (fig. 9). For road segments located on extremely steep, unstable slopes, excavated road fill was loaded into dump trucks and "end-hauled" to a stable disposal site. In contrast, decompaction (ripping) of the road surface , together with the construction of irregularly spaced cross road drains, or ditches, was the main treatment for many roads and landings found on gently sloped or stable hillslope locations. Roads accounted for less than five per cent of the area of rehabilitation sites, but often required up to 90 percent of total equipment time for correction of erosional problems.

Labor Intensive Rehabilitation Work

Following the geomorphic mapping and heavy equipment earthmoving phases of rehabilitation, labor crews refined the earthwork, constructed erosion control devices and replanted areas bared during earlier heavy equipment operations. At locations inappropriate for heavy equipment activity, labor crews constructed waterbars or ditches to disperse concentrated surface runoff or redirect diverted water. Hand constructed erosion control devices, installed prior to the first heavy winter rains, dissipated the erosive force of falling rain and flowing water in order to control rainsplash, and sheet, rill and gully erosion.

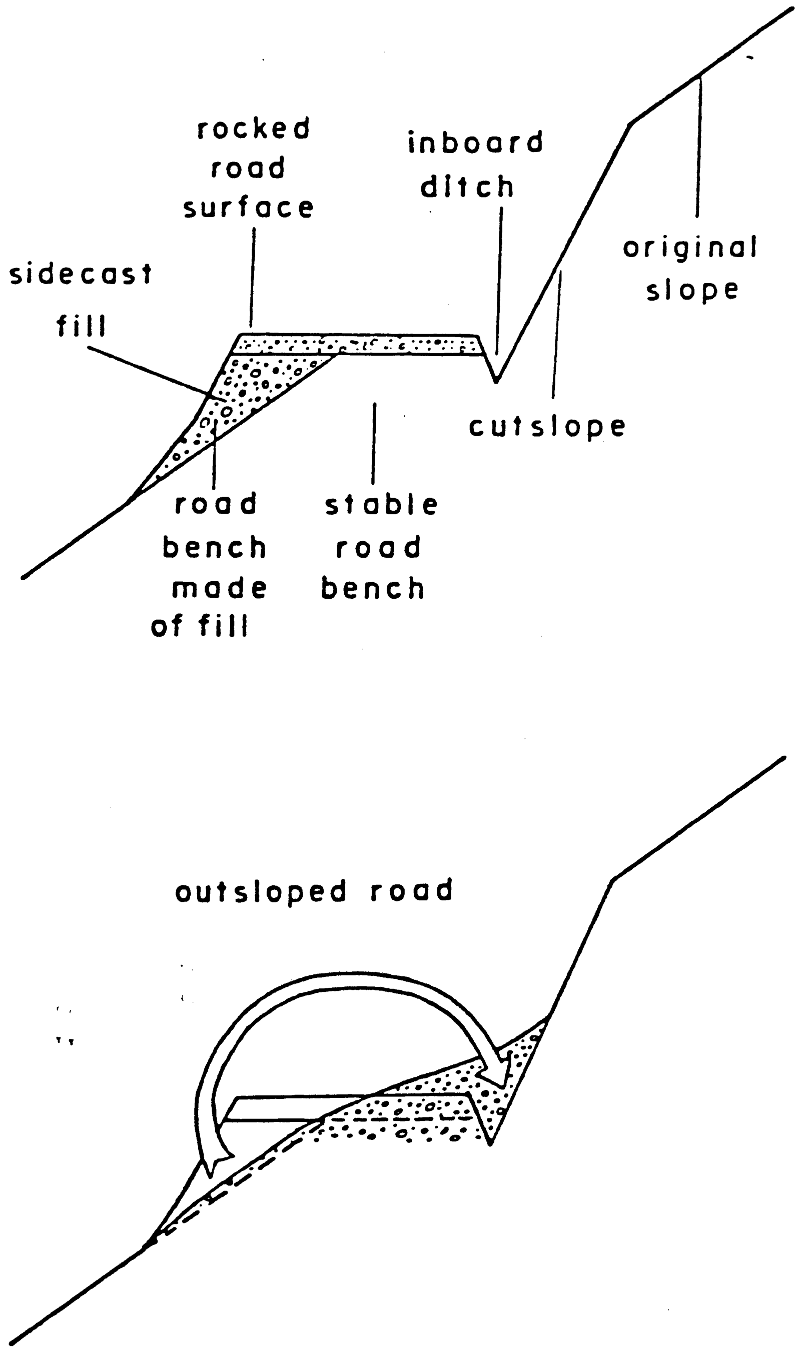

Figure 8. Top diagram shows typical logging road in cross section. Below, slope configuration after road outsloping. Sidecast fill material has been transferred to the inside of the road bench.

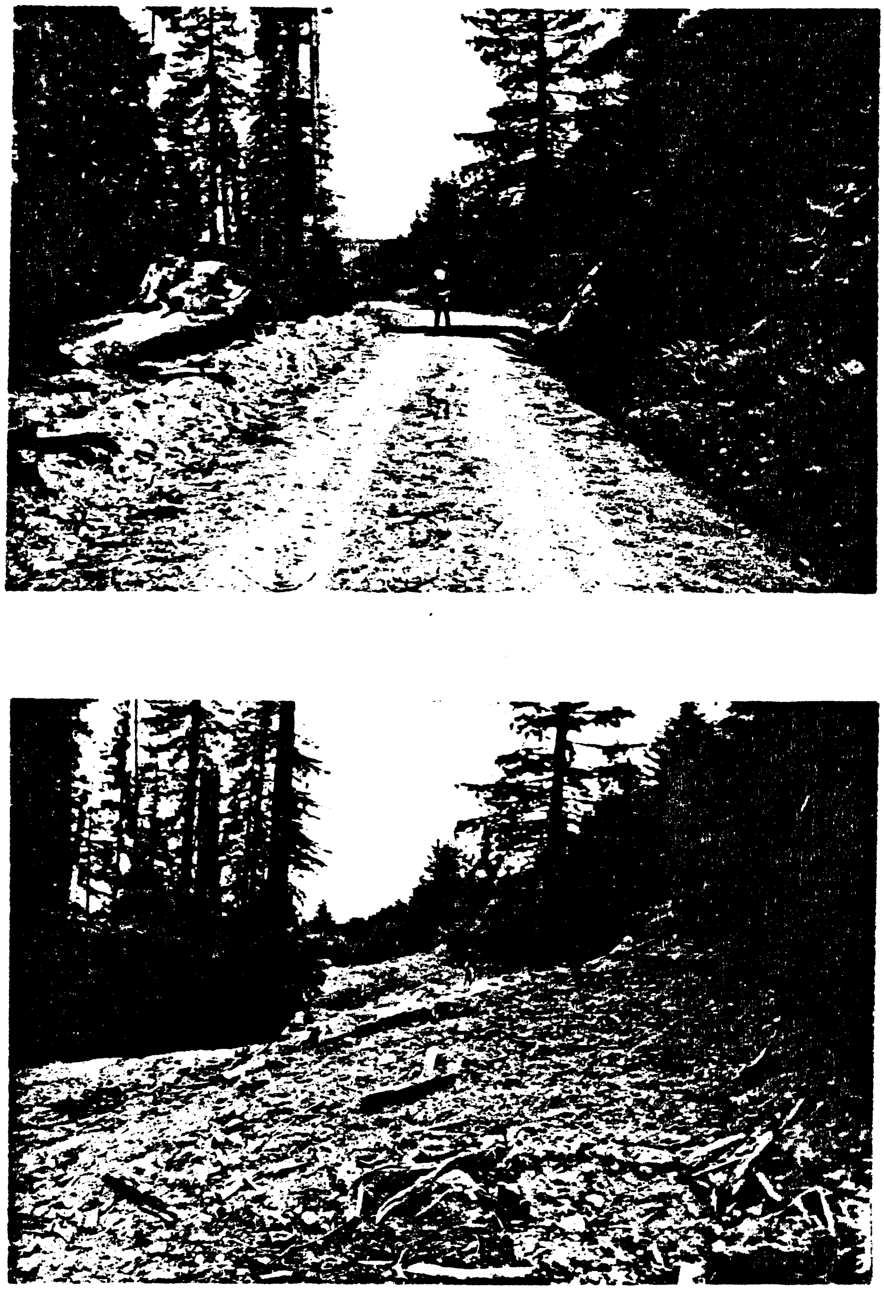

Figure 9. Before and after photographs of typical outsloped logging road in Redwood National Park.

These structures included wooded terraces, contour trenches, wattles, ravel catchers, mulches check dams rock aggregate, water ladders, flumes and energy dissipators.

Techniques for controlling erosion in gullies and small stream channels. Bank erosion and channel downcutting in gully systems and recently excavated stream channels were a substantial source of sediment production from rehabilitated clear cut areas. Both check dams and rock armor were used to protect highly erodible channel banks and beds. In 1978-1980, check dams were confined to streams with less than about 0.6 cms (20 cfs), mean annual peak winter discharge. Most structures were constructed from redwood slabs which were split or milled at each work site (fig. 10).

Installed in a sequence such that the sediment basin behind one dam abuted the base of the next upstream, check dams effectively prevented further channel downcutting, helped stabilize adjacent stream banks and provided a fertile substrate for vegetation (fig. 11). Once erosion was retarded, vegetation will become reestablished and provide root support and surface protection when the check dams begin to deteriorate after their minimum expected lifetime of 10 years. Periodic maintenance inspections following winter storms has largely prevented premature failure of check dams.

Check dams were typically 0.25-1.0 m (0.75-3.0 ft.) high, and 0.5-3.0 m (1-10 ft.) wide, depending on stream gradient and width (fig. 12). They required 5-15 person-hours, or more, to construct and install. Compared to other erosion control techniques, check dams were relatively expensive, but their high effectiveness in stabilizing rapidly eroding watercourses warrented their use in many cases.

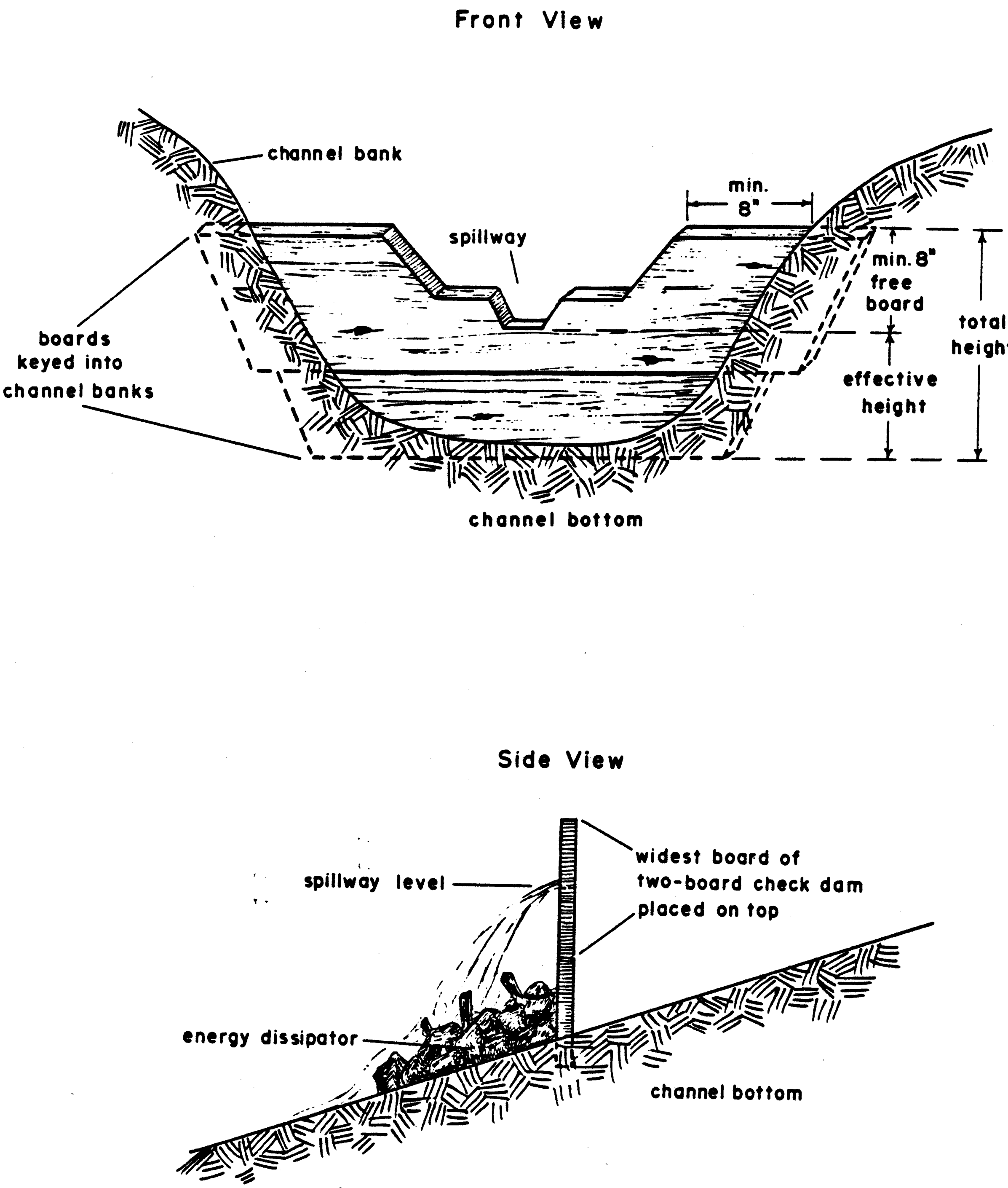

Figure 10. Schematic drawing of split or milled redwood board check dam.

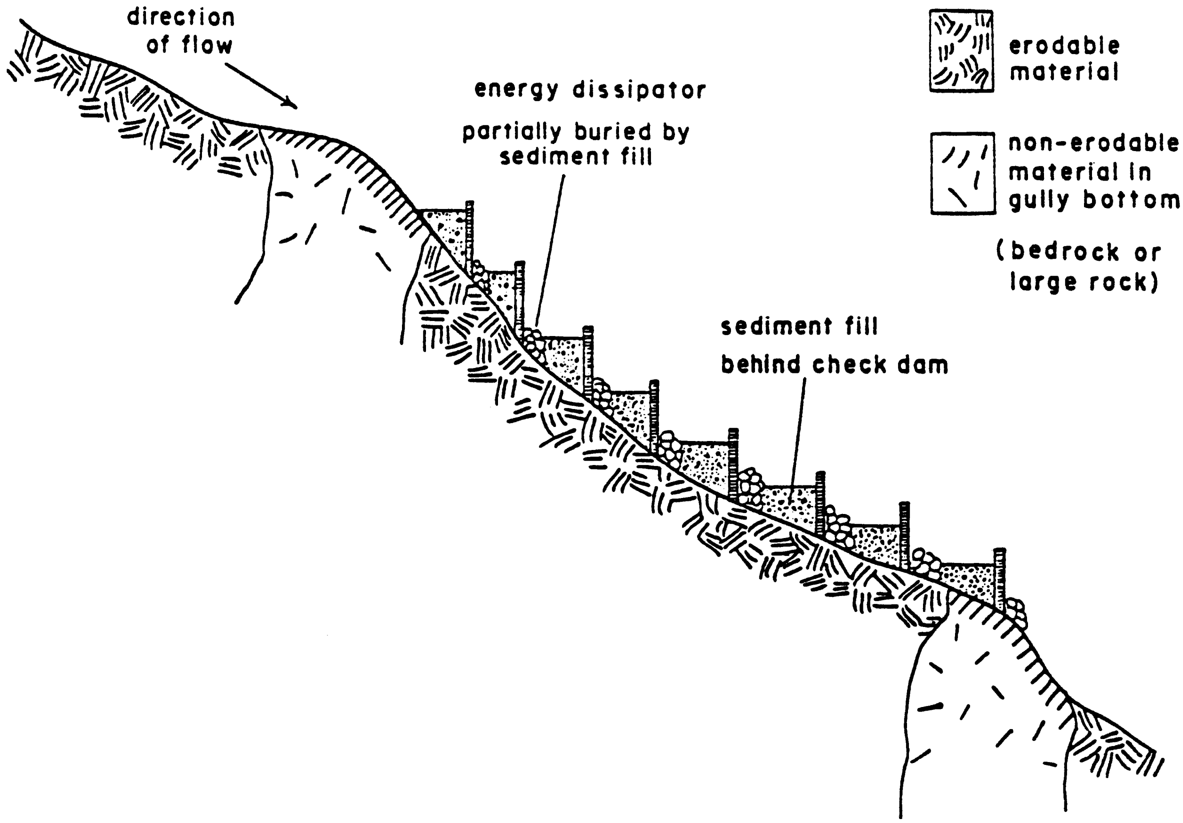

Figure 11. Profile along a gully bottom showing proper placement of a series of check dams with lower-most check dam built on a non-erodible base (called base level).

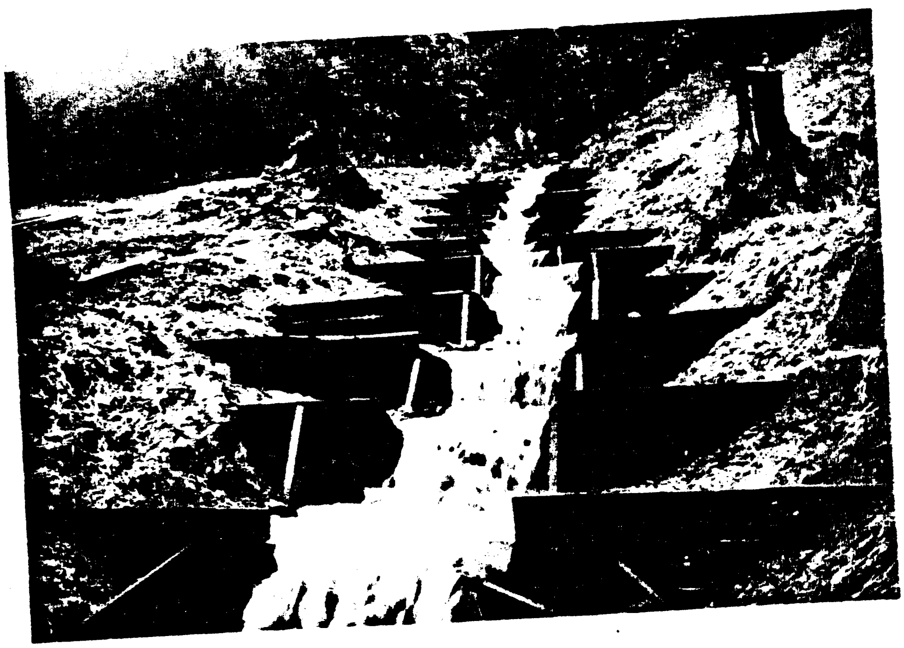

Figure 12. Photographs showing small and large redwood board check dams constructed on rehabilitation sites in Redwood National Park. Effective spillway heights (see fig. 10) are roughly 1.0 feet (0.3 m) and 3.0 feet (1.0 m), respectively. Spillway capacities were designed for the 20-year flood.

Armoring newly excavated stream channels with rocks also inhibited downcutting and lateral erosion (fig. 13). Like check dams, rocking promoted immediate channel bank and bed stabilization while allowing time for vegetation to become established. Typically, rock aggregate was quarried, transported and placed in channels by heavy equipment. Stream crossings required 10-300 m^3 (15-400 yd^3) of rock ranging from 20-100 cm (0.5-3.5 ft.) in diameter. Manual rocking was used in channels inaccessible to dump trucks and in first order streams where large boulders were not essential. Although rocks shifted slightly in the first winter season until they were firmly bedded in the channel, minor erosion occurred during this adjustment period.

In channels that were too steep to use check dams or rocking, water ladders or flumes were used instead to arrest headcut erosion at stream knickpoints. Water ladders are stair-stepped wooden structures that convey concentrated runoff over short reaches of steep, unstable slopes, while dissipating the kinetic energy of the water (fig. 14). Water flumes are analogous structures which convey water in wooden chutes with baffles instead of ladder treads. Both were effective erosion control tools in small watercourses, provided that 1) all flow was diverted into the structures, 2) they could accommodate peak flows, and 3) energy dissipators were installed at the outlets. Because of their high cost ($300-$800 per structure), water ladders, and flumes were only used where less expensive methods were not feasible.

To prevent erosion by flowing water at the outlets of structures, wood boards, coarse rock, and anchored logging slash were used to dissipate the energy of

Figure 13. Excavated stream channel protected by a blanket of coarse rock armor. Care must be taken to protect a channel width corresponding to the design wetted perimeter and to use rock which will remain in place during the design flood.

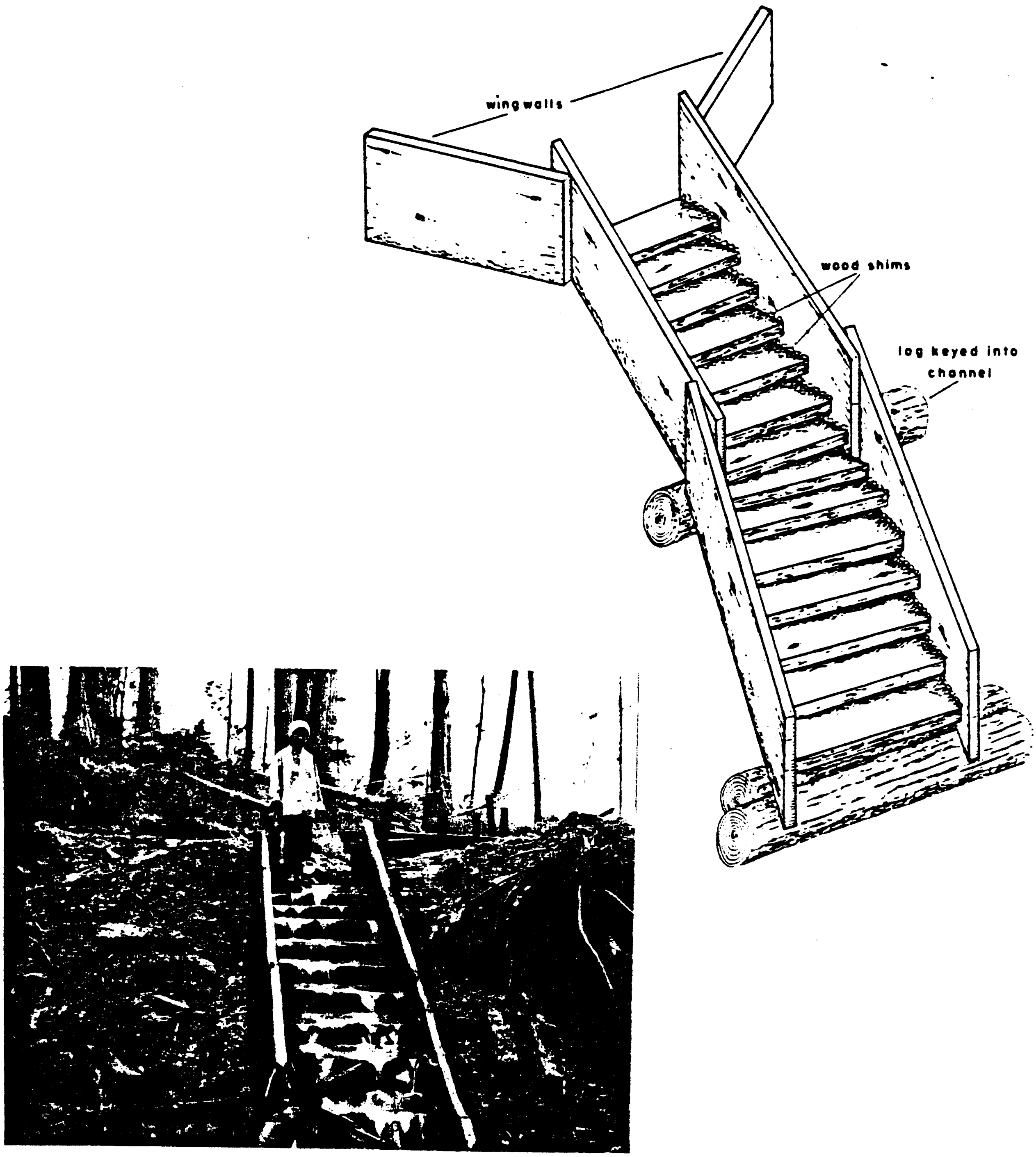

Figure 14. Typical water ladder design and installation.

cascading water. Energy dissipators were an integral part of the design and cost of structures such as water ladders, checkdams and flumes.

Techniques for controlling surface erosion. Two general categories of treatments were applied to control surface erosion. These included: 1) treatments consisting of contour terracing structures intended to disperse concentrated runoff and cause deposition of eroded sediment (wooded terraces, contour trenches, ravel catchers and wattles) and 2) treatments applied as a protective ground cover to prevent soil from eroding (mulches and seeding).

While waterbars diverted concentrated surface runoff, wooded terraces, wattles and contour trenches acted to disperse runoff or prevent surface water from concentrating (fig. 15). Wooded terraces (soil benches constructed on a contour and supported on the downslope edge by woody material) dispersed runoff, and through the terracing effect, trapped soil particles transported from bare upslope areas. Contour trenches, discontinuous ditches dug on contour into bare hillslopes, acted as small trap basins for surface runoff and eroded sediment. Both structures promoted infiltration of surface runoff into the soil, but were relatively expensive to install. Because preliminary data suggested that rainsplash, sheet and rill erosion played a minor role in total sediment delivery from rehabilitation sites, these structures were only used on critical areas during early years of the program.

Physical erosion control structures eventually deteriorate. Vegetation must be reestablished on disturbed sites to assure a long-term reduction in sediment yield. Wattling combines physical and vegetative protection for

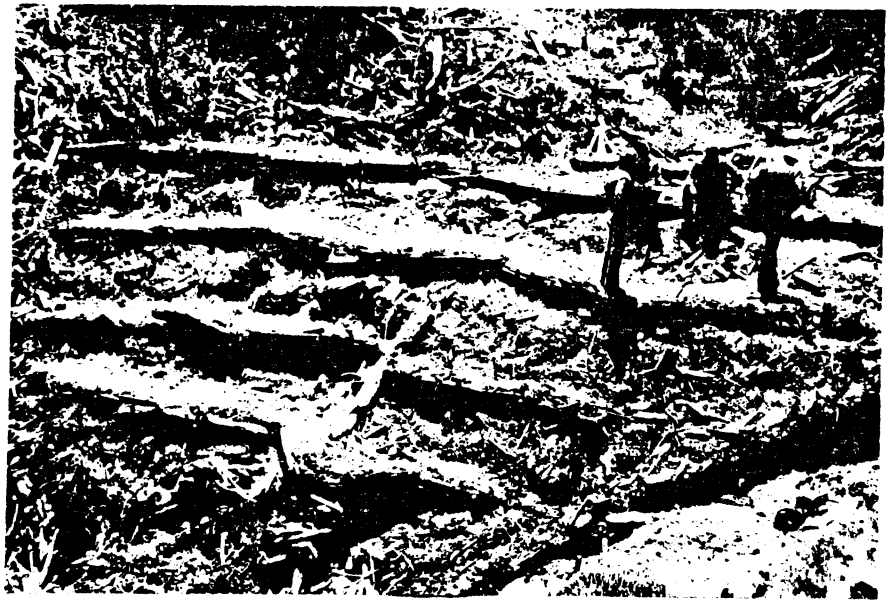

Figure 15. Examples of three types of contour terracing "structures" used in the park's erosion control program. Wooded terraces (upper left), willow wattles (upper right) and ravel catchers (lower left) were all built with locally derived, native materials. Ravel catchers were partially backfilled with soil and planted with with willow stem cuttings.

disturbed slopes. Wattles, bundles of small branches and stems partially buried in contour trenches on hillslopes, formed small terraces that trapped fine sediment derived from slope wash and dispersed runoff before it began to rill (fig. 16). If composed of sprouting species such as willow (*Salix spp*), wattles provided the added stability of a root structure as well as ground cover. Wattles were effective on moist, steep fine-grained soils, but also proved relatively expensive to install.

Mulches were applied to disturbed ground to provide immediate protection from sheet and rill erosion, as well as to preserve an open soil structure, high infiltration rates and moist soil conditions. Wood chips, logging debris (slash), straw, jute netting (loosely woven hemp), and excelsior blankets (shredded wood covered by a plastic netting), were used alone and in combination (fig. 17).

Broadcast grass-legume seeding and fertilization were used in some areas to provide a temporary ground cover as protection from rainfall. Application rates varied from 45 to 100 kg/ha (250-550 lbs/ac). Locally, vigorous grass growth bound loose surface soil and retarded ravelling and rill development; however, due to the seasonal occurrance of heavy rains, a complete ground cover was usually not established until late winter. The effectiveness of grass as a surface erosion control treatment was strongly correlated with cover density at the time significant winter rainfall began.

Other revegetation efforts included dense plantings of stem cuttings of sprouting species, and transplanting and seeding ground cover, brush and tree species (Reed and others, 1984). The primary purpose of these efforts was to accelerate the development of a cohesive root system to stabilize stream

Figure 16. Before and after photos of rehabilitation work completed along an unstable logging road. Top picture shows developing landslide. In lower photo, unstable soil and logging debris has been excavated. The bare hillslope was contour wattled with a variety of sprouting plant species.

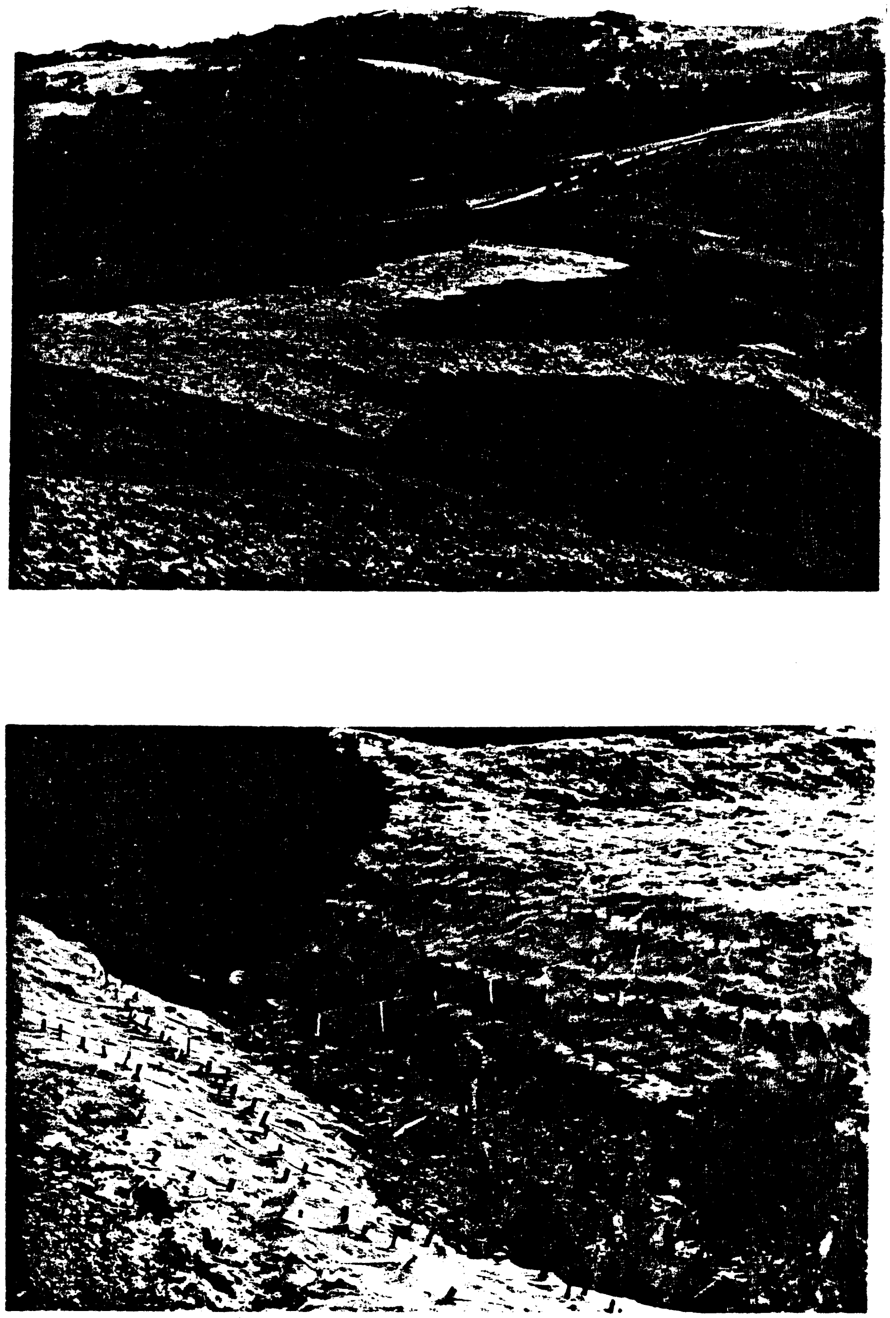

Figure 17. Straw mulch has been routinely used to cover bare soil areas on reshaped logging roads (top photo) and at excavated stream crossing sites (lower photo). It acts to control erosion and improve site conditions for revegetation. In the lower photo, stream channel side-slopes were wattled, mulched with straw and covered by jute netting.

banks and beds, and shallow slope instabilities. Although revegetation success of early projects has been studied (Reed and Hektner, 1984), evaluation of the effectiveness of revegetation for erosion control must necessarily await several more years of observation.

VI. COST-EFFECTIVENESS OF WATERSHED REHABILITATION

The effectiveness of watershed rehabilitation, like any other work, is primarily dependent upon the degree to which stated goals have been obtained. Success can only be judged when clearly defined objectives have been established against which results can then be compared and evaluated.

In isolated situations where the end-product of such work has an immediate and directly quantifiable market value, a cost-benefit analysis will help determine the advisability of initiating, continuing and/or modifying restoration activities. However, classic cost-benefit analyses cannot be routinely assigned to erosion control practices because forest soils have no easily quantifiable economic value.

Since unitless cost-benefit ratios for erosion control work at the park would have require the assignment of unique monetary values to project benefits, cost-effectiveness ratios were developed instead. Thus, rehabilitation cost-effectiveness has been judged on the degree to which the stated goal of keeping soil out of streams has been or is expected to be attained rather than on economic return. In this manner, various techniques used to attain this goal were compared on the basis of their cost and measured effectiveness.

Controls on Cost-Effectiveness

The controls on rehabilitation cost-effectiveness are derived from three principal sources. First, the goals of the program are of greatest overall importance in determining how cost-effectiveness will be measured and evaluated. The other two controls are, by definition, those factors which influence the effectiveness of treatments and work procedures, and those factors which determine project costs. The remainder of this section will describe the more important variables which have influenced each of the three controlling elements, as they have related to erosion control work at Redwood National Park.

Goals. The two fundamental goals of the park's watershed rehabilitation program are: 1) to restore the acquired area to a natural, self-functioning redwood forest ecosystem, and 2) to reduce accelerated erosion rates and sediment yields which continue to impact park resources (United States Department of Interior, 1981a). Although revegetation and restoration of the biological system are important elements of the program, primary emphasis has been placed on the reduction of accelerated sediment production and delivery. With this objective for the park's erosion control program, cost-effectiveness has been measured, and techniques evaluated, on the basis of treatment costs and the amount of sediment removed or prevented from entering active watercourses where it could have been transported downstream. The measure of cost-effectiveness used in the park's program is the unit cost-per-volume of sediment "saved" from sediment yield ($\$/yd^3$) over a specified period of time.

The dynamic nature of immediate goals in a long-term rehabilitation program sometimes makes cost-effectiveness evaluations most applicable to limited, short-term objectives. For example, in the park's program, the immediate goal of erosion-control technique-development temporarily supplanted the long range goal of minimizing increased sediment yield (Sonnevil and Weaver, 1982). Thus, in 1978, rehabilitation sites were selected to provide numerous opportunities for controlling a wide variety of erosional problems. Cost-effectiveness during this experimental phase was not of overriding importance in determining work site locations or technical prescriptions for erosion control. When objectives and other conditions do not change through time (as has been the case in the park program since 1979), technical improvements, increased efficiency and experience aid in improving effectiveness, decreasing costs and raising overall levels of cost-effectiveness.

<u>Effectiveness</u>. By definition, factors which influence the effectiveness of erosion control work also partially determine the cost-effectiveness of these techniques. Many treatments may show little or no change in their effectiveness through time (e.g. road outsloping, channel excavations, waterbars, rock armor). Other treatments, however, exhibit temporal variability that ultimately affects the effectiveness (and cost-effectiveness) of erosion control work. For example, successful revegetation will provide additional stability and protection to a disturbed site as the plants increase in size and number (Reed and Hektner, 1981). Thus, revegetation represents an erosion control treatment whose effectiveness increases through time, making it one of the most cost-effective long-term treatments. In direct contrast, some erosion control treatments become less effective through time, especially mulches and structural devices such as wood check-dams, flumes, and water

ladders. Even though some structural measures provide highly effective, immediate protection against accelerated erosion, their limited life-spans result in continuing maintenance costs and generally decreasing levels of effectiveness. For those treatments which display temporal variability in effectiveness, only their short term effectiveness (1-2 years) was considered for cost-effectiveness determinations in this report.

In addition to changes in the effectiveness of erosion control work through time, the type and magnitude of erosion processes can also exert substantial control on the effectiveness (and cost-effectiveness) of watershed rehabilitation (e.g. see Kelsey and others, 1981). Some erosional processes (e.g. raindrop and sheet erosion) are highly amenable to treatment and effective control, yet their relative contribution to sediment production and yield may be minimal. On the other hand, deep-seated mass movement features, while perhaps contributing a proportionately larger quantity of sediment directly to the stream system, could require huge expenditures to treat. In many cases, these sediment sources may not even be controllable. Because of this, cost-effectiveness should not be the only management tool used to influence the decision-making process. Either cost or effectiveness may be of overriding importance depending on the importance, relative size and complexity of the delivery mechanisms.

Perhaps the greatest single factor determining the ultimate effectiveness (and cost-effectiveness) of watershed rehabilitation relates to the relative timing of the original land use or ground disturbance and the onset of erosion control activities (Kelsey and others, 1981). A simplified, schematic representation of this concept is shown in Figure 18. Some erosion features

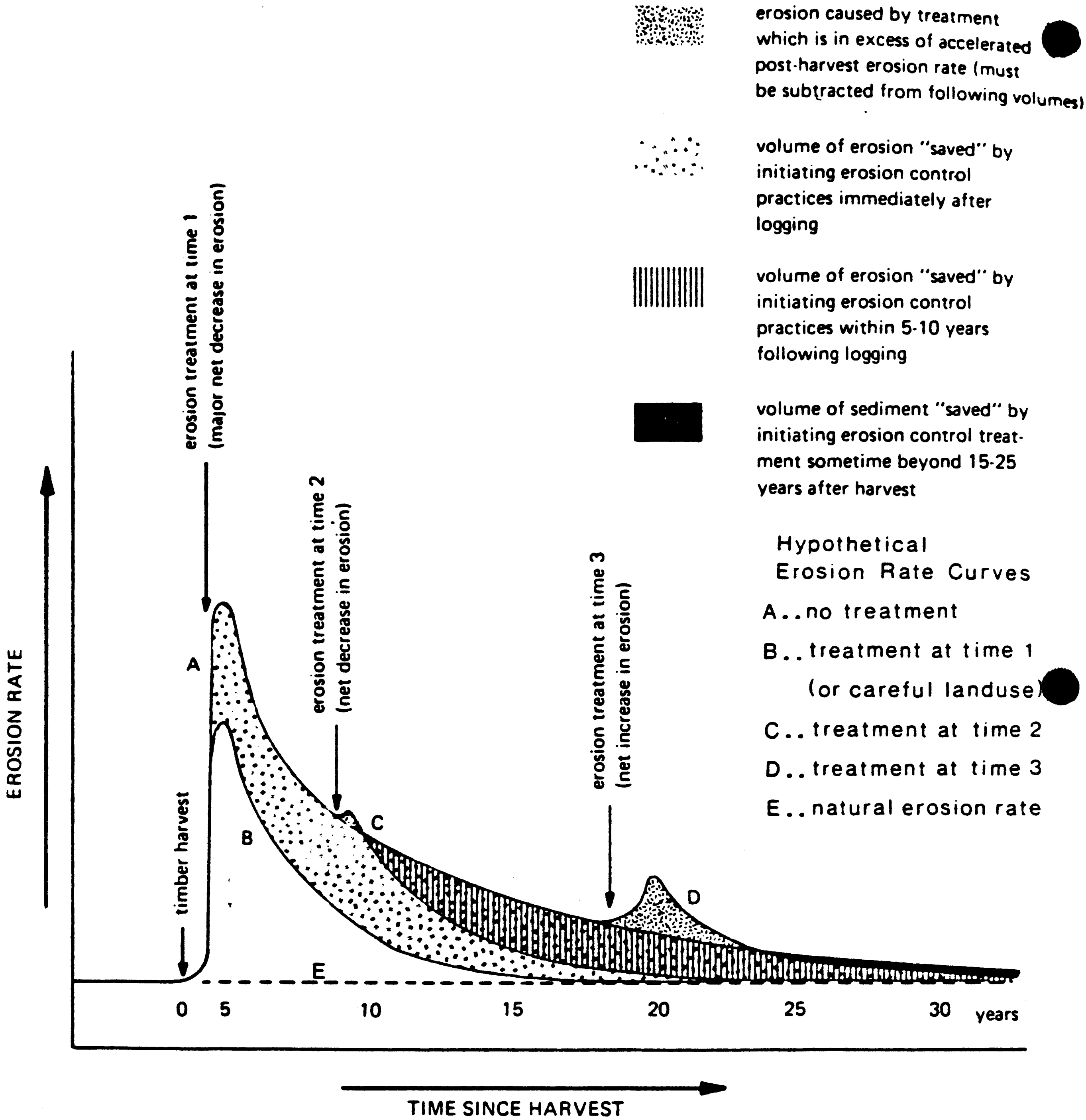

Figure 18. Schematic representation of fluvial erosion rates as affected by erosion control. Diagram is simplified for illustrative purposes. It excludes such factors as the effect of major storms and the delayed occurrence of mass movement processes following timber harvest. All else constant, the greatest rates of post-harvest erosion are expected in the first five years following land use. Time scale is included only for general reference. Depending upon erosional processes, climatic conditions and land use history, each area will display unique sediment yield curves.

may be so far advanced by the time treatment is contemplated that they are either beyond one's ability to effectively treat or they are no longer generating significant quantities of sediment. For example, in the Copper Creek drainage basin, eight years after logging, nearly 50 percent of the gully systems were no longer active (Weaver and others, 1982). Those gullies which still carried their channel-forming discharges were probably yielding sediment at only a fraction of the initial rate following management-related disturbance.

Depending on the timing of major storms, if roads are not maintained, and cutover hillslopes and erosional features in the park are allowed to remain untreated for long periods of time (approximately 20-30 years), the disturbance to soil and newly established vegetation caused by rehabilitation activities could potentially outweigh the benefits derived from these erosion control efforts. This results from rapid revegetation and a rapid rise, and then decline, in rates of elevated fluvial sediment production following timber harvest and road construction in the coastal region of northern California. Where logging roads could continue to cause stream diversions and consequent gullying in the park, treatment benefits may still outweigh impacts for several decades. For these reasons, it is critical either to plan and conduct land-use practices in a fashion to strictly minimize subsequent erosion, or to initiate rehabilitation and erosion control work immediately upon completion of operations. In this manner, treatment costs can be substantially reduced and the bulk of erosion may be altogether avoided.

Costs. Unlike factors which control the effectiveness of rehabilitation work, those elements which influence costs are more amenable to quantification and manipulation. In practice, realistic project objectives are frequently developed only after available financial resources have been determined. The stated program goal(s) indirectly assign a desired minimum level of erosion control protection the land manager is willing to accept. Specific objectives to work towards the goal(s), as well as the intensity of work activity, are then established in relationship to the funding available.

Several other considerations which affect the cost of rehabilitation work include: 1) the magnitude of indirect costs (access, administrative overhead, profit, supplies and materials, etc.) which do not specifically result in attainment of objectives but which represent unavoidable costs; 2) subjective professional judgment used to outline the problems and the desired methods of rehabilitation; and 3) the size or intensity of hydrologic event which erosion control treatments are designed to successfully withstand.

Under some circumstances, indirect costs can become prohibitively large. Depending on the method of contracting, costs not directly involved with onsite labor and heavy-equipment erosion control work can exceed 50 percent of total rehabilitation expenditures for an area (Kelsey and Stroud, 1981). Similarly, but on a smaller scale, re-opening abandoned road systems to treat continuing erosion problems along or adjacent to the roads can also increase costs and have an adverse impact on the cost-effectiveness of the overall effort.

Errors or differences in professional judgment (generally the result of a lack of relevant experience) can also result in rapid cost escalations which thereby significantly reduce rehabilitation cost-effectiveness. For example, in 1977, and to a lesser extent in 1978, project supervisors in the park perceived surface erosion to be a critical problem on rehabilitation sites. Extensive treatments were applied to control this source of soil loss (Madej and others, 1980). In actuality, after close measurement and subsequent field observations it was not found to be as significant a process as originally thought (Weaver and Seltenrich, 1981). While the treatments may have been effective, they were also of very low cost-effectiveness ($\$/yd^3$ "saved").

Other professional judgments applied during field operations can also affect levels of cost-effectiveness. For example, the use of drag-line cranes or hydraulic excavators to perform stream-crossing excavations may be needlessly expensive if the same jobs could be done with equal effectiveness and at a lower cost by more efficient machinery (e.g. bulldozers). Similarly, most stream crossings in the Park, if excavated with 30 percent channel sideslopes, show few, if any, post-rehabilitation slope stability problems. However, if the sideslopes would have been equally as stable at 50 percent steepness, significantly less soil could have been excavated at no measurable loss of effectiveness. Reductions in cost-effectiveness attributable to errors in professional judgment can be largely eliminated through increased experience, and regular and repeated peer review conducted before, during and after field operations.

Due to the nature of physical and meterologic processes in northcoastal California, and high rates of sediment production and yield, the ultimate test of erosion control effectiveness is the large hydrologic event. To account for this, treatments must be designed to accommodate high stream flows while still minimizing project expenditures. In the park, channel protection devices and stream channel excavations are currently constructed to withstand the calculated 20-year return period runoff event.

Significant increases in costs commonly associated with small increases in treatment design standards or minor improvements in effectiveness will lower rehabilitation cost-effectiveness. For example, 1500 cubic yards of fill might be excavated from a stream crossing at a total cost of $4,500. Assuming that all 1500 cubic yards would have been eroded if the crossing had not been treated, the cost-effectiveness of this excavation was $3.00 per cubic yard. To stabilize the new channel and prevent local downcutting and bank erosion, rock armor or check dams can then be placed in the stream bed. To prevent an estimated additional loss of 100 cubic yards, $2,500 might be spent on this protective treatment. The comparable unit cost (cost-effectiveness) is then $25 per cubic yard "saved" from erosion. Clearly, the added protection is accomplished at significantly reduced levels of cost-effectiveness. The decision to pursue such costly measures depends on a number of factors including, but not limited to, the nature of downstream or on-site resources being protected.

Predicting Cost-Effectiveness

Areas which display advanced erosion problems may have evolved to the point where erosion control work is no longer justified. Also, an erosion problem may be of lesser or greater significance than initially perceived. It is important, therefore, to predict or estimate the cost-effectiveness of rehabilitation work before it is conducted. In this way, the greatest results can be achieved with the available funds.

The prediction process involves five basic steps. They include: 1) delineating active versus inactive erosion features; 2) identifying potential sources of future erosion; 3) defining those problems which are technically treatable; 4) delineating those active or potential erosion sources which are accessible by heavy equipment, or whatever tools are needed to treat the problems; and 5) estimating the cost-effectiveness of the proposed treatments. The listed order of these steps is one which logically follows the intensive geomorphic mapping and erosion inventory which must precede rehabilitation. Of the currently active or potential erosion sources identified in a watershed, it is likely that only a fraction of these will be both accessible and controllable, and many may no longer be cost-effectively treated.

In erosion control work, prior determination of cost-effectiveness and the decision whether or not to treat an area hinges on an evaluation of: 1) the potential volume of sediment to be lost to erosion; 2) the probability of occurrence for this sediment release; 3) the expected rate of delivery or amortization period over which soil loss would occur; 4) the expected delivery ratio (the ratio of sediment yield to sediment production); and finally, 5) the cost associated with access and effective treatment. Although many of these factors can only be qualitatively determined for many sites, recognition of their importance and usefulness in predicting the cost-effectiveness of proposed erosion control work is paramount to making educated decisions and defensible plans for watershed rehabilitation.

VII. RELATIVE COST-EFFECTIVENESS OF EROSION CONTROL AT REDWOOD NATIONAL PARK

Methods For Evaluating Cost-Effectiveness

Post-rehabilitation evaluation of completed work is the greatest available tool for improving the effectiveness and cost-effectiveness of general approaches and specific techniques for erosion control. For maximum benefit, it is thus critical to maintain detailed accounting of work performance and costs during every readily distinguishable phase or element of a project. During the heavy equipment phase of rehabilitation work in the park, project supervisors keep hourly and daily records of where work is being done, the pieces of equipment used, the job tasks (e.g., outsloping, waterbar construction, etc.), the rate work is completed (e.g., ft/day, yd^3/hr, etc.), the cost of each task (e.g., \$/yd^3), and overall task performance and quality. These detailed records are used to determine the pieces or combinations of equipment which are most cost-effective for each task and the operators who are most adept at this work. Similarly, work and cost documentation are integral parks of all labor-intensive rehabilitation work performed in the park (e.g. United States Department of Interior, 1981b).

The most useful tools for evaluating the effectiveness of erosion control techniques are those derived from field data and experience. Repeated photo-documentation, written notes and sketches based on field observations collected during storms and periods of high runoff, and group discussions and recommendations generated during organized peer review sessions are qualitative methods of evaluation which commonly result in changes in operating procedures.

Technical changes in erosion control work generally evolve in response to a quantitative evaluation of physical processes. Techniques which have been used to measure erosion and evaluate the physical effectiveness of erosion control work in the park are straight-forward and numerous. For example, surface erosion on treated and untreated sites has been documented with detailed hillslope cross-sections and rainfall-runoff/erosion plots involving the use of sediment collection troughs. Pre- and post-rehabilitation channel erosion is measured with cross-sections and longitudinal profiles, accurate morphologic maps and winter-storm discharge measurements. Additionally, detailed checklists have been used to document the effectiveness of measures used to prevent or control both surface and channel erosion. Accurate topographic surveys, stake lines, groundwater wells, piezometers and detailed mapping have been used to monitor the response of mass movement features to rehabilitation and other environmental controls.

A modest program designed to measure the absolute or relative effectiveness of various procedures and techniques can pay for itself many times over. In the Redwood National Park rehabilitation program, less than five percent of the total budget is allocated to an evaluation program, yet the results of monitoring costs and effectiveness have provided a substantive basis for making major changes in the direction, approach and details of on-the-ground restoration (Sonnevil and Weaver, 1982; Hektner and others, 1982).

Cost Effectiveness of Primary Erosion Control Treatments

Erosion control work in Redwood National Park is divided into primary and secondary treatments. Those elements designed to provide for the immediate reduction of management-caused sediment production or yield are considered primary treatments. They are most closely associated with the ultimate objectives of erosion control and landscape rehabilitation. They generally consist of heavy equipment treatments such as outsloping, road ripping, construction of cross-road drains and waterbars, removal of soil and logging debris from stream channels, landslide stabilization and stream channel rediversions. Most primary erosion control practices used in the park were accomplished at a cost-effectiveness of from one to ten dollars per cubic yard ($\$/yd^3$) of sediment removed or prevented from entering local channels and being transported downstream (table 1).

Disregarding the fact that the sources of increased erosion are associated with a variety of failure probabilities and subsequent delivery ratios, the cost-effectiveness of treating these erosion problems can vary over a range of two orders of magnitude, or more. For example, by excavating fill-crossings on natural stream channels, each cubic yard physically removed represents an equal volume saved from future erosion and sediment yield. However, in diverting streams out of rapidly eroding gullies and back into their natural channels, each cubic yard excavated at the diversion point could ultimately result in 10, 100 or more times the savings in potential future erosion that might have been generated by the untreated, active gully system. Large, rapidly eroding gully systems can be totally dewatered through simple, comparatively inexpensive excavations at the source of the diversion (Teti, 1982). It may also be technically possible to treat some large landslides at cost-effectiveness levels better than $1 - $10 per cubic yard. However, stabilization is typically difficult and/or prohibitively expensive regardless of overall cost-effectiveness.

Table 1. Cost-effectiveness of primary erosion control treatments used to prevent or minimize sediment production and yield in Redwood National Park, 1978-1980.

Treatment	Average Cost Paid in Park ($)[1]*	Cost-Effectiveness Range ($/yd³ "saved")[2]
Road ripping (decompaction)	350-450/mi	unquantified[3]
Construction of crossroad drains[4]	1000-3000/mi	unquantified[5]
Waterbar construction on skid-trails		
machine constructed	5-50 ea[6]	unquantified[5]
hand-labor constructed	30-300 ea[7]	unquantified[5]
Forest road outsloping for erosion control[8]	2500-9500/mi[9]	1-10[10]
Prairie road outsloping	7000/mi[11]	unquantified[12]
Excavation of skid-trail stream crossings	125-1350 ea[13]	1-10[10]
Excavation of logging road stream crossings[14]		
- under 750 cubic yards	2000 ea	1-10[10]
- 750-1500 cubic yards	3000-3500 ea	1-10[10]
- those requiring endhauling	4000 ea	1-10[10]
Rediversion of stream flow from gullies back into Natural stream channels	125-4000 ea[15]	0.1-0.5[16]
Gully stabilization[17]	variable	variable
Prairie gully obliteration	variable	unquantified[12]
Removal of perched debris from the perimeter of yarder pads and cable landings	1000-5000 ea[8]	1-10[10]
Large landslide excavations[18]	20,000-30,000 ea[19]	1-10[10][20]

*see succeeding page for footnote explanations

Footnotes to Table 1:

1. Cost based on 1978, 1979 and 1980 unpublished data. "Average" depends on site conditions.

2. Primary goal of Redwood National Park program is to minimize management-related sediment production and yield (i.e., to "save" soil from moving into stream channel systems and, eventually, downstream); no time frame for the eventual occurrence of the erosion has been specified for these calculations although complete loss is anticipated over a period from one decade, or less, to one century. Cost-effectiveness calculation assumes total loss, without reference to time.

3. Treatment results in increased rate of revegetation and reduced surface runoff, and produces an unknown decrease in road surface, ditch, gully, and downslope stream channel erosion. Road fill failures may also be reduced by an unknown quantity.

4. Assumes construction every 150 feet, on average; cost range dependent on type of equipment (tractor, backhoe and hydraulic excavator, in order of increasing unit costs).

5. Treatment results in reduced concentration of surface runoff and an unknown reduction in road surface, ditch and gully erosion from adjacent hillslopes.

6. Tractor-constructed waterbars, \$5 to \$20 each; backhoe-constructed waterbars, from \$5 each for areas with good access and requiring little travel time between work sites, to \$50 each where access is poor (e.g., on steep slopes) and results in high travel time.

7. Average cost was \$60 each; range dependent on length and substrate hardness at each waterbar location.

8. Costs depend upon the concentration of organic debris and the amount of endhauling required; generally, up to 75 percent of the cost may be in debris removal while the remaining 25 percent is taken up by actual outsloping of the landing.

9. Narrow roads using only tractor, \$2500/mi; narrow roads using tractor and backhoe, \$3000-\$4000/mi; roads across moderately steep terrain using tractor and hydraulic excavator, \$5000-\$7750/mi; roads built across steep, unstable ground using a dragline crane and tractor, with some endhauling required, up to \$9500/mi (all figures include ripping costs, but do not include the expense of stream crossing excavations).

10. Assumes sediment production would have occurred had the excavation not been performed, and that erosion would have been translated into sediment yield in adjacent stream channel systems. Does not include benefits realized from preventing future stream diversions, and associated gully erosion, which might have occurred without treatment. Unit costs for excavations from 1978-1980 varied from \$3-\$7/yd^3 while from 1981-1983 costs ranged from \$1-\$5/yd^3. This difference is a result of increased efficiency at performing excavations.

11. Full outsloping utilized tractor and hydraulic excavator.

12. The dual goals of rehabilitation on prairie or grassland areas include: (1) erosion control and prevention, and (2) scenic or site restoration of a high visitor-use area; thus the measure of cost-effectiveness used in this table does not apply.

13. Costs dependent upon site accessibility; x = \$400 for tractor and backhoe combination; x = \$300 for tractor and excavator tandum.

14. Excavations primarily performed by tractor and hydraulic excavator; some completed with drag-line crane.

15. Cost of re-diversion is typically associated with stream crossing excavation at point of diversion.

16. Assumes diverted stream flow would continue to cause increased erosion and had not yet formed a stable, non-eroding channel. Results derived from Teti (1982), Weaver and others, (1982) and unpublished Redwood National Park data.

17. Treatments include armoring, check dams, bank protection and gully headcut stabilization. Costs and cost-effectiveness dependent on type and extent of treatments applied, their effectiveness, and the expected rate of erosion had the erosion control work not been accomplished (see also table 3 for a more detailed discussion of the cost-effectiveness of these methods).

18. Streamside landslides in the size class of 60,000 to 100,000 cubic yards, of which approximately 4500 to 7000 cubic yards (7%) are excavated from the crown region. Other

remedial measures may also be employed on a site-specific basis, but those figures are not included in this analysis.

19. Cost includes endhauling a short distance to a local storage area (0.25 mile).

20. Assumes a one-to-one soil loss potential; that is, each cubic yard of material excavated from the slide mass is considered one cubic yard "saved" from eventual delivery to an adjacent stream channel.

The cost-effectiveness of some primary erosion control treatments is unquantifiable (table 1). Commonly these treatments are intended to disperse or restore concentrated surface runoff which can impact gully and surface erosion rates as well as slope stability. Treatments such as waterbars, cross-road drains and some forms of road outsloping are included in this category. Some treatments are intended to fulfill other park goals in addition to controlling erosion. These include road ripping, prairie road outsloping, gully shaping on prairies, and work in areas which may receive high visitor use.

Cost-Effectiveness of Secondary Erosion Control Treatments

Secondary erosion control practices are those designed to minimize erosion from areas disturbed during primary treatment. They typically consist of a variety of labor-intensive erosion control and revegetation techniques, as well as heavy equipment work needed to transport and place channel armor. A listing of average costs and the range of relative cost-effectiveness of secondary erosion control techniques used in Redwood National Park is shown in table 2 and table 3.

Perhaps the most important information to be gleaned from a comparison of the three tables are the relative ranges of cost-effectiveness. Primary erosion control techniques used in the park are from one to three orders of magnitude more cost-effective than secondary treatments. Similarly, on logged lands in Redwood National Park, secondary treatments used to control channel erosion (table 2) are generally much more cost-effective than treatments designed to control surface erosion (table 3). This difference reflects the greater

Table 2. Cost-effectiveness of secondary erosion control treatments used in Redwood National Park to minimize or eliminate short-term post-rehabilitation channel scour(1)

Channel Treatment(2)	Cost-Effectiveness Range(3) ($/yd³ "saved")	Comments
Water ladders(4)	20 - 70(5)	for short reaches
Brush check-dams(6)	10 - 30(7)	short lived; for small gullies
Small board check-dams(4)	10 - 30(8)	highly effective; may require maintenance
Large board check-dams(9)	30 - 50(10)	very expensive; require maintenance
Hand-placed rock armor(6)	20 - 70(11)	limited to small channels; low flow
Machine-placed rock armor	10 - 50(12)	very effective; requires good access

1. In certain circumstances, these techniques may also be considered primary treatments. In the park, they are typically employed at excavated skid-trail and logging-road stream crossings.

2. The treatments listed here are not interchangeable; each technique is best suited to a particular situation. Thus, treatments are not directly comparable in terms of cost-effectiveness.

3. Assumes treatment is 100% effective; most methods provide 60% to 90% effectiveness in the first year of average rainfall, and a reduced effectiveness with time (with the possible exception of machine-placed rock armor). Cost-effectiveness would therefore be somewhat lower than that listed (i.e. higher $/yd³ value). Figures refer to first year cost-effectiveness.

4. As used in the park, structures work best when confined to channels which carry a 20-year peak discharge of 6 cfs, or less.

5. Average cost = $700 (1978 data) for 30 ft. structure; erosion prevented = 10 - 40 yds³. A more cost-effective treatment would be to excavate a small channel.

6. As used in the park, treatments work best with flows of 2 cfs, or less. Brush dams are used mostly in narrow gullies, not in excavated stream crossings.

7. Treatment cost for 60 ft. channel (9 dams) = $135. (1981 wage rates); erosion prevented = 5 - 20 yds³.

8. Average cost for a 60 ft. channel (9 dams) = $320. (1978 - 1979 data); erosion prevented = 10 - 40 yds³.

9. As used in the park, large-board dams work best when used on channels which carry a 20-year peak discharge of 20-30 cfs, or less.

10. Cost to treat a 200 ft. channel = $9450. (13 dams on a 1980 work site; includes first year maintenance costs); erosion prevented = 200-300 yds³, based on nearby untreated crossings.

11. Average cost to armor a 60 ft. channel = $370. (1978-1979 data); erosion prevented = 5-20 yds³. Cost assumes rock is available on site.

12. 1980 cost to rock a large, 275 ft. channel = $4180. (includes blasting and rock delivery; size = 6-18 in.); erosion prevented = 200-300 yds³. Mean cost to treat 12 crossings (avg. 65 ft long) = $530.; erosion prevented = 10-40 yds³.

Table 3. Average costs and first year cost-effectiveness of secondary treatments used to control surface erosion in Redwood National Park.

Treatment	mean cost(1) ($/10,000ft²)	Cost-effectiveness to prevent rain splash erosion(2)(3) ($/yd³ "saved")	Cost-effectiveness to control rill erosion(2)(4) ($/yd³ "saved")
Contour Trenches(5)	430	NO DATA (ND)	NO DATA (ND)
Wooded Terraces(5)	668	ND	ND
Ravel Catchers(5)	590	ND	ND
Wattles(5)	2500	ND	ND
Grass-Legume Seed with Fertilizer	99	95	ND
Hydroseed	514	430	ND
Straw mulch (8000 lb/acre)(6)	275	195	5
Jute-secured straw mulch (8000 lb/acre)	2360	1575(7)	45(7)
Excelsior Blankets	1970(8)	1315(7)	40(7)
Wood Chips	950	ND	145

1. Based on 1978 and 1979 data.

2. Cost-effectiveness is computed by dividing mean treatment cost by the product of potential or estimated erosion and measured treatment effectiveness. For example, grass-legume seeding cost $99/10,000ft² and, in our plot studies controls about 70 percent of the first year surface erosion (compared to control plots). If 1.5 yd³ would have been eroded from the untreated area, cost-effectiveness = 99/(1.5 x 0.7) = 95.

3. Computations based on treating a 100-foot-long stream crossing excavation with bare, 50-foot-long side-slopes at a 45% gradient (total area = 10,000 ft²). Potential volume of erosion from this area is assumed to be 1.5 yds³ (all of which would leave the slope and enter the adjacent stream channel system) until living ground cover is established. This volume is somewhat in excess of erosion rates measured from sediment trough studies from similar settings within the park and produces conservative or "better-than-probable" values of cost-effectiveness.

4. Computations based on a 200-foot-wide by 50-foot-long slope at a 50% gradient (10,000 ft²) subjected to concentrated surface runoff for one winter. Volume of erosion from this area without treatment is assumed to be 50 yds³. These figures were obtained from slope treatment plots (20-foot-wide X 50-foot-long) with measured cross-sections.

5. Quantitative data concerning the effectiveness of contour terrace structures is not available. However, by definition these structures could not prevent rainsplash erosion. Additionally, field observations showed that, although these treatments all caused the deposition of sediment, they often concentrated surface runoff which resulted in some rill erosion. Thus, for this reason, and because of their high installation cost, the use of contour terrace structures was discontinued before quantitative tests were established. Treatments are listed for cost comparison only.

6. Straw mulch is routinely applied at an application rate of 6000 lb/acre and has been the dominate treatment to control surface erosion since 1980. 8000 lb/acre was inadvertently used for plot studies.

7. Jute-secured straw and excelsior blankets are of roughly equal effectiveness.

8. Based on 1980 purchase data and 1979 installation costs.

importance and contribution of erosion from post-rehabilitation channel scour and adjustment as compared to surficial soil loss from bare areas.

It is significant to note that even among methods designed to treat similar erosion problems (e.g., sheet or rill erosion), there may be well over an order of magnitude difference in their relative cost-effectiveness (table 3). This usually arises from large variations in the cost of application rather than major differences in effectiveness. For example, in the park, straw mulch and jute-secured straw mulch have provided comparable protection to bare slopes under 70 percent in steepness. However, the high cost of installing jute makes this a much less cost-effective treatment. By definition, maximizing cost-effectiveness entails a trade-off between maximum effectiveness and minimum cost. In addressing erosion control problems, it must be recognized that maximizing cost-effectiveness may result in unwarranted compromises.

Some techniques may be only marginally effective yet their cost-effectiveness is high. Grass seeding, an inexpensive erosion control treatment, is one such example (table 3). If the objective is controlling surface erosion, a slightly more effective, less cost-effective method could be chosen (e.g. straw mulch). Likewise, jute-secured straw mulch may be the most cost-effective technique in circumstances where other treatments provide an unacceptably low level of effectiveness (e.g. on slopes steeper than 70 percent; Weaver and Seltenrich, 1981).

In general, secondary treatments are characterized by one or more factors which significantly limits their ultimate cost-effectiveness. They include:

(1) the volume of potential erosion being prevented is usually small when compared to the volume treated during primary or heavy equipment rehabilitation work; (2) by nature, the total cost of a task performed by manual labor can be extremely high when compared to costs for performing the same types of work with machines; and (3) the products of labor techniques are commonly plagued by either limited life-spans, limited capacity or resiliency or, in the case of revegetation, delayed effectiveness. If program objectives demand short-, immediate-, and long-term protection, such practices can be justified. If short-term increases in sediment production would endanger downstream riparian or aquatic resources in need of protection, then labor-intensive treatments are essential (Teti, 1982). However, if ultimate concerns are only focused on the long-term reduction of accelerated sediment yields, many of the secondary treatments listed in Tables 2 and 3 would not be cost-effective.

Finally, labor costs, can represent a large percentage of watershed rehabilitation expenditures. On four 50-acre to 200-acre cutover areas treated in 1978 and 1979, labor costs along ranged from $20,000 to over $80,000 each and comprised from 75 percent to 95 percent of the total project cost (Madej and others, 1980; Kelsey and Stroud, 1981). Four methods have been used to complete labor work in the park (Sonnevil and Weaver, 1982). In order of increasing cost-effectiveness, they are: (1) request for proposal (RFP), cost-reimbursement contract; (2) request for proposal (RFP), fixed price service contract; (3) direct labor hiring (if done on a temporary or part-time basis); and (4) invitation for bid (IFB), fixed price service contract. Although the incentive to do a rapid, inexpensive, and effective job is greatest with the competitively bid, IFB, fixed price service contract;

administrative delays and legal requirements can sometimes make their use impractical when erosion control work must be completed prior to the commencement of winter rains. Hiring a part-time labor force allows the application of secondary treatments immediately following heavy equipment operations. However, the need for training and a comparatively lesser incentive to maximize work efficiency can increase the costs of this method. Thus, in-house labor must be temporary, well trained and well supervised or motivated to be cost-effective.

VIII. SUMMARY AND CONCLUSION

A number of factors have affected the cost-effectiveness of erosion control work conducted at Redwood National Park. Here, as elsewhere, cost-effectiveness can only be evaluated in terms of achieving clearly defined objectives within the framework of overall program goals and available funds. Controls on cost-effectiveness have been shown to depend upon the static or dynamic nature of short-term objectives and long-term goals, the mechanisms and "controlability" of sediment sources, and a variety of factors which show variation through time. These include the protection provided by revegetation and structural erosion control measures, and rates of post-disturbance sediment production and yield. Erosion control treatments can be selectively prescribed to provide short-, intermediate- and long-term protection to a site. Applications which ignore any of these time frames may fall short of meeting the primary objectives.

Prevention is clearly the least costly and most effective method for minimizing increased erosion and sediment yield. However, where corrective work is needed, quantitative predictions of erosion control cost-effectiveness can result in significant savings. Only those projects which can be completed within acceptable levels of cost and with beneficial results need be carried out. Cost-effectiveness calculations used in the park's program allow short-term comparisons between measures as widely divergent as the dewatering of gully systems, stream channel excavations, channel armoring, check damming, mulching and grass seeding.

Evaluations of the cost-effectiveness of specific primary and secondary treatments used in Redwood National Park show in excess of three orders of magnitude potential difference between the various techniques. Due to this large variation, extra efforts are now taken on park projects to complete the cost-effective primary treatments to the fullest extent possible. This reduces the amount of secondary protection needed to attain maximum overall cost-effectiveness. In many cases, adequate primary treatment may actually obviate the need for further protection at little or no loss of effectiveness.

Work at the park has shown that a successful erosion control program requires a rigorous evaluation and monitoring program which continually feeds information and findings back into ongoing rehabilitation work. Acceptable levels of cost-effectiveness can only be assured through the quantitative documentation of erosion processes and erosion control effectiveness. Accurate, detailed accounting of procedures, work elements and associated unit costs can then be used to establish the cost-effectiveness of watershed rehabilitation for erosion control.

REFERENCES CITED

Anderson, H. W. 1979. Sources of sediment-induced reduction in water quality appraised from catchment attributes and land use. In Proceedings of Third World Congress on Water Resources, Mexico City, April 23 - 28, v. 8, p. 3606 - 3616.

Best, D. W. 1984. History of timber harvest in the Redwood Creek basin. U.S. Geol. Survey Professional Paper (in press).

Brown, W. M., and Ritter, R. J. 1969. Phenomenal erosion rates in the Eel River basin, California, may be the highest in North American (abs.). Abstracts with Program, Geol. Soc. America Annual Meeting, Atlantic City, N.J., v. 1, p. 21 - 22.

Bundros, G. J., Spreiter, T., Utley, K., and Wosika, E. 1982. Erosion control in Redwood National Park, 1980. In Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Pacific Coastal Areas. August 25 - 28, 1981. Center for Natural Resource Studies, Sacramento, CA.

Harden, D. R., Janda, R. J., and Nolan, K. M. 1978. Mass movement and storms in the drainage basin of Redwood Creek, Humboldt County, California - A progress report. U. S. Geol. Survey Open File Report 78-486, 161 p.

Hektner, M., Reed, L., Popenoe, J., Veirs, S., Mastrogiuseppe, R., Sugihara, N., and Vezie, D. 1982. Review of revegetation treatments used in Redwood

National Park: 1977 to present. In Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Pacific Coastal Areas. August 25 - 28, 1981. Center for Natural Resource Studies, Sacramento, CA.

Janda, R. J. Nolan, K. M., Harden, D. R., and Coleman, S. M. 1975. Watershed conditions in the drainage basin of Redwood Creek, Humboldt County, California, as of 1973. U.S. Geol. Survey Open File Report 75-568, 266 p.

Janda, R. J. 1978. Summary of watershed conditions in the vicinity of Redwood National Park, California. U.S. Geol. Survey Open File Report 78-25, 82 p.

Kelsey, H. M., and Weaver, W. E. 1979. Watershed rehabilitation for erosion control on logged lands in Redwood National Park. In Guidebook for Geol. Soc. of America field trip, April 12 - 14, 1979: A field trip to observe natural and management-related erosion in the Franciscan terrain of Northern California. 14 p.

Kelsey, H. M. 1980. A sediment budget and an analysis of geomorphic process in the Van Duzen River basin, north coastal California, 1941 - 1975. Geol. Soc. America Bull., part II, v. 91, no. 4, p. 1119 - 1216.

Kelsey, H. M., Madej, M. A., Pitlick, J., Stroud, P., and Coghlan, M. 1981. Major sediment sources and limits to the effectiveness of erosion control techniques in the highly erosive watersheds of north coastal California.

_In_ Proceedings of a Symposium on Erosion and Sediment Transport in Pacific Rim Steeplands. January 25 - 31, 1981. Christchurch, New Zealand. IAHS-AISH Publication Number 132. International Association of Hydrological Sciences. Washington, D.C. p. 493-510.

Kelsey, H.M., and Stroud, P. 1981. Watershed rehabilitation in the Airstrip Creek basin. Redwood National Park Technical Report No. 2. National Park Service, Redwood National Park, Arcata, CA. 45 p.

Kelsey, H. M., Coghlan, M., Pitlick, J., and Best, D. 1984. Geomorphic Analysis of streamside landsliding in the Redwood Creek basin. U.S. Geological Survey Professional Paper (in press).

LaHusen, R. G. 1984. Characteristics of management-related debris flows, northwestern California. _In_ International Symposium on Effects of Forest Landuse on Erosion and Slope Stability, International Union of Forestry Research Organizations (in press).

Madej, M. A., Kelsey, H. M., and Weaver, W. E. 1980. An evaluation of 1978 rehabilitation sites and erosion control techniques in Redwood National Park. Redwood National Park Technical Report No. 1, National Park Service, Redwood National Park, Arcata, CA. 152 p.

Nolan, K. M., Harden, D. R., and Colman, S. M. 1976. Erosional landform map of the Redwood Creek drainage basin, Humboldt County, California; 1947-1974. U.S. Geological Survey, Water Resource Investigations, Open-file report 76-42.

Reed, L. J., and Hektner, M. M. 1981. Evaluation of 1978 revegetation techniques, Redwood National Park. Redwood National Park Technical Report No. 5, National Park Service, Redwood National Park, Arcata, CA. 70 p.

Reed, L. 1984. Revegetation of disturbed areas in Redwood National Park, northwestern California. (this volume).

Sonnevil, R. A., and Weaver, W. E. 1982. The evolution of approaches and techniques to control erosion on logged lands in Redwood National Park, In Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Pacific Coastal Areas. August 25 - 28, 1981. Center for Natural Resource Studies, Sacramento, Ca.

Swanson, F. J. 1981. New directions in assessing human impact on erosion in steeplands. Keynote address, Theme 3, Internat. Symposium on Erosion and Sediment Transport in Pacific Rim Steeplands, New Zealand Jour. of Hydrology, v. 20, no. 1, p. 3 - 7.

Teti, P. 1982. Rehabilitation of a 290 hectare site in Redwood National Park, 1980. In Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Pacific Coastal Areas. August 25 - 28, 1981, Center for Natural Resource Studies, Sacramento, Ca.

U.S. Department of the Interior. 1981a. Watershed Rehabilitation Plan. National Park Service, Redwood National Park. Denver Service Center. Denver, CO. 65 p.

U.S. Department of the Interior. 1981b. Upper Devils Creek request for proposed (RFP); contract No. CX8480-0-0009. National Park Service, Redwood National Park.

Weaver, W. E. and Seltenrich, M. S. 1981. Summary results concerning the effectiveness and cost-effectiveness of labor-intensive erosion control practices used in Redwood National Park, 1978 - 1979. Redwood National Park unpublished memorandum report. National Park Service, Redwood National Park, Arcata, CA. 19 p.

Weaver, W. E., Choquette, A. V., Hagans, D. K., and Schlosser J. P. 1982. The effects of intensive forest land-use and subsequent landscape rehabilitation on erosion rates and sediment yield in the Copper Creek drainage basin, Redwood National Park. _In_ Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Pacific Coastal Areas. August 25 - 28, 1981. Center for Natural Resource Studies, Sacramento, CA.

Weaver, W. E., Hagans, D. K. and Popenoe, J. 1984. Magnitude and causes of gully erosion in the lower Redwood Creek basin. U.S. Geol. Survey Professional Paper, (in press).

A REVIEW OF THE REVEGETATION TREATMENTS USED IN REDWOOD NATIONAL PARK - 1977 TO PRESENT

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Abstract. The revegetation program in Redwood National Park treats freshly reshaped surfaces following physical erosion control work. Revegetation prescriptions are coordinated with physical site treatments to address surficial erosion control, slope stabilization and ecosystem restoration. The program has evolved from early use of wattles and unrooted stem cuttings to current use of nursery-grown cuttings, bare root and containerized seedlings. Grass seeding for immediate erosion control is being replaced by straw mulching. Experimentation continues for technique refinements and the wider use of native species. The most successful results are attributed to treatments which mimic natural vegetation patterns.

INTRODUCTION

Redwood National Park was established in 1968 to preserve significant examples of coastal redwood forests and the streams and seashores with which they are associated. Timber harvesting and related road construction in the Redwood Creek watershed outside the park combined with natural processes to pose iminent threats to downstream Park resources (Agee 1980). Naturally high erosion rates were greatly accelerated by intensive land use practices and unusually severe storms. Vegetation removal, alteration of hillslope drainages and development of an extensive logging road/skid trail network caused increased runoff, sediment yield, and accumulation of sediment deposits in major stream channels. Other problems included increased landsliding, filling, and widening of stream beds, erosion of stream banks, damage to streamside vegetation and overall degradation of natural aquatic ecosystems (Madej *et al*. 1980).

In 1978, Congress amended the Redwood National Park Establishment Act through Public Law 95-250 to enlarge the park by 48,000 acres of which 36,000 acres were recently logged. It directed that a watershed rehabilitation program be developed to minimize man-induced erosion and to encourage the return of a natural pattern of vegetation (see USDI 1981, Watershed Rehabilitation Plan).

In anticipation of congressional authorization to rehabilitate cutover timberlands, a pilot program was begun in 1977. The rehabilitation program

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has since moved from the developmental phase into full scale implementation in 1980, with continued monitoring for technique effectiveness and refinement.

Objectives of the Revegetation Program are: 1) accelerate the restoration of redwood forests and associated vegetation systems, 2) contribute to long-term slope stability through vegetation re-establishment, and 3) aid in reduction of surface erosion. This paper describes the revegetation portion of the Vegetation Management program and examines techniques of revegetation which were implemented in Redwood National Park from 1977 to present.

SEQUENCE OF REHABILITATION ACTIVITIES

The sites chosen for rehabilitation include former logging haul roads, skid trails and stream crossings, logging decks and landings, and prairie ranch roads. Park geologists and hydrologists assess the need for erosion control, selecting the most effective physical techniques to treat critical areas. In addition to erosion control, physical treatments are designed to promote the establishment of natural and planted vegetation by: 1) disaggregating rocked roads, 2) spreading excavated fill or soil over exposed bare rock, and 3) separating and returning buried topsoil to the surface.

Heavy equipment operations (stream crossing excavations, road outsloping and ripping, and water bars) result in freshly disturbed ground susceptible to surface erosion. In areas where stream crossings have the greatest potential for contributing material to creeks, grasses, shrubs, and mulches are used to reduce streamside sediment loss, with mulches providing immediate cover until vegetation can become established.

Vegetation is generally viewed as having a minor role in initial erosion control efforts, but over time becomes the primary defense against erosion on fully rehabilitated sites. Species with fibrous root systems secure surface soil and promote soil aggregation. Rhizomes bind larger blocks of surface soil. Large, deeply-penetrating roots give the subsoil greater shear strength. Low groundcover plants reduce raindrop impact, as do trees that produce abundant litter.

The vegetation staff develops site-specific vegetation prescriptions to promote long-term erosion control by rapid revegetation. Information for developing site-specific prescriptions is derived from an inventory of existing vegetation, site relief, and soil characteristics (soil color, texture, depth of groundwater or impermeable layers) of remnant and disturbed areas. Areas of high erosion and wildlife depredation potential are noted for special attention. Species and treatments are selected for each area to maximize survival and growth. Plant materials include seed and seedlings supplied by local nurseries and seed, transplants, and cuttings collected within the park. Mulches, seed, and fertilizer are applied after heavy equipment work. Wattles, cuttings, transplants, rooted cuttings, seedlings, and tree and shrub seeds are planted in winter. Documentation and monitoring programs are conducted throughout rehabilitation, and continue for several years. Results are used to evaluate the success of treatments and refine future prescriptions.

TREATMENTS AND EVALUATION

The development of the park revegetation program is reflected in both species and techniques employed. Prescription refinements have led to changes in quantities and types of treatments utilized (Table 1).

TABLE 1. Vegetation treatments, 1977-1980.

				Containerized Seedlings and Rooted Cuttings					
Year	Wattles	Unrooted Stem Cuttings	Transplants	Redwood	Douglas-fir	Sitka Spruce	Alder	Coyote Brush	Whipplea
1977	1,820 ft	1,800	9	230	450	496	0	0	0
1978	27,155 ft	301,120 sq ft (est. 129,300)	64	2,270	2,650	0	0	0	0
1979	2,823 ft	22,000	1,919	3,400 (+67,850)*	8,900 (+58,680)*	0	800	7,471	5,066
1980	0	40	835	30,700 (2,000)*	26,820	0	59,000	23,200	13,800

* Seedlings planted for reforestation of cut-over lands.

WATTLING. Early erosion control and revegetation techniques, such as wattling, were developed elsewhere and adapted for use in the Redwood National Park rehabilitation program. Wattles are bundles of woody branches partially buried in contour trenches and are intended to revegetate the site while providing physical barriers to ravelling and rill development. Willows (*Salix*, spp.) were the primary species used because they had been used elsewhere and were abundant. Willow readily sprouted, but did not survive the dry conditions found on most rehabilitation sites. Average initial survival on 1978 sites ranged from 48 to 93 percent with vigor declining in subsequent years on all but the wettest sites. Thimbleberry (*Rubus parviflorus*), salmonberry (*Rubus spectabilis*), blackberry (*Rubus vitifolius*), coyote brush (*Baccharis pilularis* var. *consanguinea*), and redwood (*Sequoia sempervirens*) were also tried with limited success. Alder (*Alnus oregana*), blueblossom (*Ceanothus thrysiflorus*), elderberry (*Sambucus callicarpa*), and rhododendron (*Rhododendron macrophyllum*) wattles did not sprout. In general, wattles grew well on wet northern exposures with fine-textured soils. However, even on dry rehabilitation units, wattles placed close to springs or streams exhibited high survival (Reed and Hektner 1981).

As a revegetation technique, results show that wattling can be effective when restricted to readily sprouting species and placed in areas of relatively high summer moisture. The potential for successful use of wattling in Redwood National Park is limited since most of the rehabilitation areas are very dry during summer. Wattles were used extensively in 1977 and 1978, more selectively in 1979 and deleted entirely from the program in 1980. Due to high cost and ineffectiveness, Weaver and Seltenrich (1981) recommended that the use of wattling as an erosion control technique also be discontinued. Other techniques have proven to be more effective and economical for both revegetation and erosion control.

UNROOTED STEM CUTTINGS. Unrooted stem cuttings were used in early efforts to promote revegetation and root growth for slope stability. Thimbleberry, willow, salmonberry, blackberry, alder, blueblossom, coyote brush, rhododendron, elderberry, salal, big-leaf maple (*Acer macrophyllum*), evergreen huckleberry (*Vaccinium ovatum*), whipplea (*Whipplea modesta*), and bay (*Umbellularia californica*), were used as unrooted stem cuttings. Willow and coyote brush had the highest survival after one year on 1978 sites with ranges of 41 to 89 percent and 25 to 47 percent, respectively. All other species averaged less than 5 percent survival in the first year. Despite initially high survival rates for willow and coyote brush, vigor and survival are declining on all but immediate streamside sites. Willow stem cuttings are now used only along streams. Unrooted stem cuttings have not been used extensively since 1979. Other means of establishing these species are being developed.

ROOTED STEM CUTTINGS AND SEEDLINGS. Large-scaled propagation allows dense planting which is expected to establish vegetative cover more rapidly. Coyote brush seedlings and whipplea rooted stem cuttings are now the most frequently used shrub materials. Willow, thimbleberry, Sierra gooseberry (*Ribes roezlii* var. *cruentum*), ocean spray (*Holodiscus discolor*), and hazel (*Corylus cornuta* var. *californica*) cuttings are being experimentally rooted to broaden the spectrum of species available for site-specific prescriptions.

Nursery-grown alder seedlings were successfully established on 1980 rehabilitation sites and will be planted on 1981 sites. Alder enhances soil development and restoration due to the associations it forms with nitrogen-fixing Actinomycetes. Establishment of nodules on nursery stock prior to out-planting improved survival and initial growth of seedlings (Sugihara and Cromack 1981). Other hardwoods that may be nursery-grown include madrone, tanoak, big-leaf maple, and Oregon white oak (*Quercus garryana*).

One-year old containerized redwood and Douglas-fir seedlings purchased from local nurseries have been used extensively in the revegetation program. Sitka spruce (*Picea sitchensis*) has been used in limited amounts. In general, initial survival has been less than 50 percent, however by the third year, those seedlings surviving have become well established. Predation by black-tailed deer and Roosevelt elk is high on conifers and shrubs, but does not always cause mortality. Fertilizer pellets, mycorrhizal inoculation, mulching, vexar tubes and Big Game repellant are being examined as methods for increasing survival and establishment. Preliminary investigations indicate that survival of two-year old bareroot Douglas-fir and redwood seedlings is much higher under the harsh conditions typical of the park rehabilitation sites.

FIELD TRANSPLANTS. Field transplanting permits the establishment of larger plants with well-developed root systems on sites where rapid vegetative cover and root expansion are desirable. Transplanting also allows greater use of species where seed collection, propagation or rooting techniques have not been successfully developed. Most transplants are obtained on-site and are already adapted to the local environment. Transplants have been successfully used since 1977 and include: alder, redwood, coyote brush, whipplea, salal, evergreen huckleberry, rush (*Juncus*, spp.), sedge (*Carex*, spp.), madrone (*Arbutus menziesii*), 'Alta' fescue (*Festuca arundinacea*), cattail (*Typha latifolia*), coltsfoot (*Petasites palmatus*), iris (*Iris*, spp.), deer fern (*Blechnum spicant*), bracken fern (*Pterdium aquilinum* var. *pubescens*), and sword fern (*Polystichum munitum*). Moderate sized transplants have done well and survival of whipplea and coyote brush increased with top-pruning. Cost and survival comparisons for transplants, rooted cuttings, and seedlings are planned for this year.

SEEDING. Establishment of early successional vegetation can be accelerated by artificially seeding native tree, shrub, and ruderal species. Localized dense stands of coyote brush, alder, sedge, and rush have been successfully established by direct seeding. Small quantities of maple, tanoak, blueblossom, cattail, dock (*Rumex crispus*) and chinquapin (*Castanopsis chrysolepis*) were seeded with little success. Continued technique development may allow these and other species to be utilized more extensively in the revegetation program.

Grass seeding is widely used for erosion control on disturbed areas. Annual ryegrass (*Lolium multiflorum*), 'Blando' brome (*Bromus mollis*), creeping red fescue (*Festuca rubra*), and vetch (*Vicia*, spp.) were used on 1977 sites. Ryegrass dominated, with cover over 70 percent the first year. Grass coverage has decreased each successive year, to presently less than 10 percent. Quail are thought to have eaten most of the vetch seed.

Annual and perennial ryegrass, 'Potomac' orchard grass (*Dactylis glomerata* 'Potomac'), velvet grass (*Holcus lanatus*), barley (*Hordeum vulgare*), fawn tall fescue (*Festuca arundinacea*), and crimson clover (*Trifolium incarnatum*), were used alone or in combination in 1978. Only ryegrass persisted past the first year in any amount. Initial coverage was spotty, varying from less than 1 up to 40 percent.

The 1979 seed mix included perennial ryegrass, orchard grass, creeping red fescue, and 'Highland' colonial bentgrass (*Agrostis tenuis* 'Highland'). Ryegrass dominated the first year but was replaced the following year by bentgrass with little change in overall cover. In general, cover was less than 10 percent on unfertilized areas and greater than 50 percent on fertilized areas. On one unit, spring cover averaged 50 to 75 percent with 250 lbs/acre and 75 to 90 percent with 500 lbs/acre ammonium phosphate / sulfate (16-20-1-13S) fertilizer. Fertilizer also stimulated ruderal species; cover averaged 35 to 50 percent with 250 lbs/acre fertilization compared to 2 to 4 percent on control plots (Popenoe 1981).

Bentgrass, creeping red fescue, fawn tall fescue, 'Blando' brome, 'Durar' hard fescue (*F. ovina* var. *duriscula*), Zorro fescue (*F. megalura*), 'Mt. Barker' subclover (*Trifolium subterraneum*), 'Lana' woolypad vetch (*Vicia dasycarpa*), and common vetch (*Vicia*, spp.), were used on 1980 sites. Grass cover increased with fertilization, with brome the most successful. In the first season, however, even without fertilization, the legumes dominated, averaging 50 to 75 percent cover, with only vetch doing well on poorly-drained blue clay sites.

It has been noted that timing of fertilization significantly affects relative species composition. Fall applications of fertilizer improve stand cover of seeded grasses while late winter applications favor woody invader species such as coyote brush.

Grass has been locally effective for controlling frost heaving, as well as rainsplash, sheet, and rill erosion but not until late in the season. Except in wet areas, dense grass cover has not been established prior to the first rains. In 1981, limited trials of fall hydroseeding of grasses with fertilizer and mulch produced a rapid ground cover of greater than 80 percent. Water availability and vehicle access limit the potential of this technique in the park. Hydroseeding is being done on roadcuts through prairies where conventional methods cannot be used. Grass is a vigorous competitor with native woody vegetation and greatly reduces natural invasion, an objective along prairie roads in Redwood National Park. This same competition with native woody species has led to more restricted use of grass seeding in forested units. Some of the effects of grasses on invading and planted species are being investigated on 1980 rehabilitation units.

MULCHES. Mulches, when spread immediately after heavy equipment work and prior to the first rains, are used to minimize surficial erosion. Mulches also reduce and disperse runoff (promoting infiltration) and reduce evaporation. On environmentally harsh sites, these factors favor re-establishment of vegetation. Straw mulches at 2,000 and 4,000 lbs/acre were found to reduce total herbaceous cover while increasing initial invading coyote brush seedling density. In addition to straw, redwood chips, hardwood bark, whole chipped Douglas-fir and Monterey Pine (Pinus radiata) mulches were used in 1979. Excessive handling costs and poor revegetation led to the elimination of all but straw mulch for 1980. Weedy contaminants from the straw have been found but have not persisted into the second year.

COSTS

In 1978, $87,000 was spent for vegetation materials and installation. Of this, 74% ($64,353) was for wattles, 19% ($16,742) for unrooted stem cuttings and the remaining 7% ($6,305) for all other revegetation.

By 1980, a changing emphasis in treatments and techniques enabled revegetation of larger areas for similar costs. Of approximately $77,000 spent in 1980, less than 1% ($40) was spent for unrooted stem cuttings, 2% for transplants ($1,420), 26% ($20,200) for seeding and fertilizing and 72% ($55,500) for seedlings and rooted cuttings.

Table 2 shows the unit cost of major revegetation techniques used for 1978-1980, including materials and labor. Costs varied widely by site and by year

TABLE 2. Unit cost comparison of major revegetation techniques including materials and labor, 1978 - 1980.*

				Containerized Seedlings and Rooted Cuttings			
Year	Wattles	Unrooted Stem Cuttings	Transplants	Conifers	Alder	Coyote Brush	Whipplea
1978	avg. $2.47/ft ($1.00 - $2.93)	avg. $0.38 ea ($0.10 - $0.39)	$0.60 ea	$0.08 ea plus $0.12 ea labor	-	-	-
1979	avg. $1.16/ft ($1.02 - $1.79)	avg. $0.26 ea ($0.19 - $0.85)	avg. $1.48 ea ($0.49 - $2.21)	$0.10 ea	$0.10 ea	$0.35 ea	$0.35 ea
				(. . . . . plus undocumented labor costs . . . . .)			
1980	-	$1.00 ea	$1.70 ea ($1.39 - $3.00)	$0.10 ea	$0.125 ea	$0.107 ea	$0.125 ea
				(plus $0.137 - $0.216 ea for labor and overhead)			

* 1977 itemized treatment costs not available.

depending upon labor source and site conditions. Revegetation work was performed by request for bid contracts, cost reimbursable contracts, in-house labor and contract labor. Site accessibility, source of plant materials (obtained on-site vs. carried in) and difficulty of planting influenced total labor costs.

DISCUSSION

The revegetation program in Redwood National Park treats freshly recontoured surfaces following physical site treatment. In most cases the surfaces are nutrient deficient subsoils lacking native seed and micro-organisms. Environmental stresses due to summer drought and winter cold are intensified by the lack of canopy cover. Early rehabilitation projects relied upon species and techniques developed elsewhere for other conditions. Prescriptions such as willow wattling and grass seeding had been intended for immediate surficial erosion control and revegetation. These techniques often proved unsatisfactory in the environment of rehabilitation sites. Mulches have now been substituted for immediate surface protection while vegetative prescriptions address long-term erosion control through native vegetation re-establishment.

Current work concentrates on improving survival and establishment of planted vegetation and there is new emphasis on managing the seedbed environment to promote natural revegetation. Bulk native seed collection, processing, propagation, and planting techniques are being refined. Two-year old bare root conifer seedlings will be used more extensively, particularly on the harsher sites. Field trials examining the value of slow-release fertilizer pellets and treatments to minimize wildlife depredation are being conducted. Broadcast fertilization will be timed to favor establishment of native species. Experimental use of compost will begin next year and hydroseeding of native shrub species will be tested.

Vegetation treatments and techniques are most successful when they mimic the natural vegetation patterns adjacent to rehabilitation sites. Utilization of colonizing species improves prospects for successful plant establishment on harsh sites. Well-established native vegetation will assist in long-term slope stability and erosion control.

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LITERATURE CITED

Agee, J. K. 1980. Issues and impacts of Redwood National Park Expansion. Environmental Management 4:407-423.

Madej, M. A., H. M. Kelsey, and W. E. Weaver. 1980. An evaluation of 1978 rehabilitation sites and erosion control techniques in Redwood National Park. Watershed Rehabilitation Technical Report No. 1. Redwood National Park, Arcata, CA.

Popenoe, J. H. 1981. Effects of grass-seeding, fertilizer and mulches on vegetation and soils of the Copper Creek watershed rehabilitation unit: the first two years. Proceedings, Symposium on watershed rehabilitation in Redwood National Park and other Pacific coastal areas. August 25-28, 1978. Center for Natural Resource Studies, Scaramento, CA.

Reed, L. J. and M. M. Hektner. 1981. Evaluation of 1978 revegetation techniques, Redwood National Park. Watershed Rehabilitation Technical Report No. 5. Redwood National Park, Arcata, CA. In press.

Sugihara, N. G. and K. Cromack, Jr. 1981. The role of symbiotic micro-organisms in revegetation of disturbed areas - Redwood National Park. Proceedings, Symposium on watershed rehabilitation in Redwood National Park and other coastal areas. August 25-28, 1978. Arcata, CA. Center for Natural Resource Studies, Sacramento, CA.

U. S. Department of the Interior. National Park Service. 1981. Watershed Rehabilitation Plan. Redwood National Park. Denver Service Center. Denver, CO.

Weaver, W. E. and M. Seltenrich. 1981. Summary results concerning the effectiveness and cost effectiveness of labor-intensive erosion control practices used in Redwood National Park, 1978-1979. Unpublished report, Technical Services Division, Redwood National Park. Arcata, CA.

TECHNICAL SPECIFICATIONS FOR HAND-LABOR EROSION CONTROL METHODS

by
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modified from
Technical Specifications For Watershed Rehabilitation
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TECHNICAL SPECIFICATIONS FOR HAND-LABOR EROSION CONTROL METHODS.

Introduction

Erosion control works can be constructed by hand labor methods, by heavy equipment, or by a combination of the two. For example, wattling is largely done by hand labor. Coarse rock armor, on the other hand, is necessarily placed by heavy earthmoving machinery. Many techniques can be accomplished entirely by hand or entirely by mechanized procedures (e.g. spreading straw mulch). Other practices, such as constructing rock check dams, can most effectively be done by a combination of hand labor and mechanized procedures.

These technical specifications were developed for the application of a variety of erosion control measures in relatively remote steepland areas. As such, with the exception of hydroseeding, they consist entirely of labor intensive methods. All the procedures were initially used in Redwood National Park's watershed rehabilitation program from 1978 to 1980. Because of the national park setting, they emphasize the use of native, locally available raw materials which could be collected on-site.

More recent findings by National Park Service scientists indicate that certain of these practices may be much more cost effective than others for controlling surface and channel erosion (the results of tests and evaluations of erosion control cost effectiveness are available from the National Park Service). However, local conditions may warrant or dictate the use of one or more methods deemed more successful elsewhere.

If you plan on using the attached specifications for erosion control contracting, apply them loosely and use professional judgement and common sense to adapt them to your local conditions and requirements. If you have the opportunity, consult local experts and practioners. Also, try to perform at least one trial application of each method, according to specifications, that you intend to employ later. This will tell you a lot about how the contract will work and where you must remain flexible in required methods or materials. In general, however, the attached specifications should provide a good basis for developing and implementing a broad variety of erosion control prescriptions.

- OUTLINE -

TECHNICAL SPECIFICATIONS FOR HAND-LABOR EROSION CONTROL

SECTION A: SURFACE EROSION

MULCH SPECIFICATIONS

I. STRAW MULCH

A. Definition of job.

Straw from bales is spread over a predesignated area at an application rate set by contract specifications. The straw will protect the soil surface from rainfall impact and help to retain soil moisture on biologically harsh sites.

B. Specifications.

1. Straw shall be spread evenly within the flagged area. The amount to be spread will be given in number of bales (example: 3.5 bales.) or in dry pounds-per-acre.

2. Bales are provided on site but it will sometimes be necessary to transport them to the specific work area. Prospective contractors will be shown the location of the straw during the pre-bid "show-me" inspection of site.

3. Baling wire shall be removed from the site and properly disposed of.

4. Mulch shall be the last task performed on the work area, following any contour terracing, wattling, wooded terraces, transplants or grass and fertilizer application.

C. Comments.

For large areas, it's best to give a "lbs/acre" application rate. A rate of 6000 lbs/acre is good for erosion control; 8000 lbs/acre covers the ground surface completely. For small or irregular areas it may be easier to compute the number of bales needed and then just specify exactly how much goes where . . . It is your option whether to provide the bales on-site or let the contractor figure it out and do the logistics. Specify whether hay, with all its seed is a desireable or acceptable substitute for straw.

II. JUTE NETTING

A. Definition of job.

Jute netting (a loosely woven hemp) is rolled over bare soil areas to hold soil in place and prevent rilling. Since jute is tacked or stapled onto the ground it is very resistant to overland flow and disperses surface runoff. Rolls are usually 4-5 ft. wide.

B. Specifications.

1. Smooth ground surface where jute netting is to be used.

2. For ease of installation, roll jute down the fall-line of the hillslope.

3. Staple jute, or secure it with stakes, on roughly 2 to 3 foot centers.

4. Staple all low points so jute is in continuous contact with ground.

5. Roll down second strip of jute netting, overlapping adjacent strip by at least 6 inches. Staple overlapping areas.

6. Staple second roll to ground.

7. Repeat until ground is covered.

C. Comments.

Jute is usually reserved for slopes that are too steep to wet or too windy for loose straw to adhere to. As such, it is usually used as a binding cover over other loose mulches (see "Jute Secured Straw" specification). Laying strips of jute on contour should usually be avoided because it may be difficult to keep overlapping areas together under the downslope stress of soil movement.

III. JUTE-SECURED STRAW

A. Definition of job.

The bare soil is first covered with straw mulch (6000 lbs/acre) and then jute netting is secured on top. This procedure combines the effective surface protection afforded by straw mulch with the stability of the secured jute netting.

B. Specifications.

1. Apply straw mulch (as per "straw mulch" specification).

2. Secure jute netting on top of straw mulch (as per "jute netting" specification) being sure not to remove straw and expose bare soil.

C. Comments.

This has been found this to be the most effective treatment for preventing rainsplash, sheet and rill erosion from bare soil areas. Because it is much more labor intensive than straw mulching, and therefore more expensive, its use should be limited to steep (>70%) slopes or areas where wind or concentrated surface runoff would otherwise remove the straw mulch.

IV. CURLEX MULCH

A. Definition of job.

Curlex mulch, and other similar "bound mulches" are applied to prevent surface erosion (rain splash, sheet and rill erosion).

B. Specification.

1. Same procedures as for Jute Netting.

C. Comments.

Curlex is composed of shredded aspen, bound between 2 layers of biodegradable plastic netting. It lasts and performs roughly equivalent to jute secured straw mulch. It is less expensive to apply since only one step is required, but it is more expensive to purchase. Curlex should be reserved for erosion control on steep slopes.

V. WOOD CHIP MULCH

A. Definition of job.

Wood chips are spread over a designated bare soil area to retard surface erosion. Application rates are set in the contract specifications.

B. Specifications.

1. At least 50% of the wood chips used for mulching shall have at least one dimension 2 inches in length. Smaller pieces are unacceptable.

2. Wood chips shall be evenly spread over the designated area so as to cover at least 95 percent of the underlying soil surface.

C. Comments.

Wood chips are much more difficult to move and spread by hand than straw mulch. Once on the ground, they tend to move downhill by sliding or to blow across the surface during wind gusts. Some evidence suggests thick applications may retard natural revegetation. Variations of wood chips include logging slash, tree limbs and branches or chopped brush.

VI. GRASS SEED AND FERTILIZER APPLICATION

A. Definition of job.

Grass seed and fertilizer are hand spread with "belly grinders" within flagged areas. Application rates are predesignated and seed and fertilizer may be provided. Grass will serve as an immediate, temporary ground cover to decrease surface erosion.

B. Specifications.

1. When stored on-site, fertilizer is to be protected from dew and rain by plastic tarps. Grass seed must be stored under dry, cool conditions and protected from mice.

2. Application rates are listed as pounds of seed and pounds of fertilizer to be used in a specified area or, alternately, as pounds-per-acre of each.

3. Occasionally no fertilizer is to be applied. This will be noted in the site-specific instructions.

4. Scales for weighing, buckets, "belly grinders" and rakes are to be provided by the contractor.

5. When a mixture of seeds with very different sizes and weights is to be applied care must be taken to ensure that seeds are evenly distributed in the mix, insuring in an even distribution on the ground. Since smaller seeds will settle to the bottom it may be necessary to periodically shake the belly grinder to redistribute the seeds.

6. Seed and fertilizer are to be applied as soon as possible after slope work (contour terraces, wattling, wooded terraces) is completed in order to take advantage of warm temperatures accompanying the first fall rains. Seed and fertilizer are to be applied before mulching.

7. Seed and fertilizer (applied separately) <u>must</u> be spread uniformly over entire area.

8. Unless otherwise specified, seed and fertilizer are to be raked into the soil immediately after application, covering them with 1/8 to 1/4 inch of soil.

IV. GRASS SEED AND FERTILIZER APPLICATION (Continued)

C. Comments.

Grass can be an effective erosion control technique provided a thick, consistently uniform cover of grass is obtained prior to the advent of erosive rains. Its erosion control effectiveness is directly related to cover density. Unfortunately high cover density also effectively prevents the establishment of other planted or naturally seeded vegetation. Where site conditions are dry, sandy or otherwise harsh, grass may not be as successful or provide as immediate protection as mulching. The constituents of both the seed mix and fertilizer needs to be specified.

VII HYDROSEEDING

A. Definition of job.

A slurry of wood fiber, grass seed, fertilizer and water is sprayed on bare soil areas. The mulch holds the seeds in place, provides a cool, moist environment for germination and protects the ground surface from erosion. Specifications can be complex.

B. Specifications (excerpt from CalTrans Standard Specifications; 1976).

1. The work shall consist of hydro-seeding erosion control material consisting of a mixture of fiber, seed, commercial fertilizer and water to embankment slopes and excavation slopes as shown on the plans.

2. Fiber shall be produced from non-recycled wood such as wood chips or similar wood materials and shall be of such character that the fiber will disperse into a uniform slurry when mixed with water. Fiber shall not be produced from sawdust or from paper, cardboard or other recycled materials. Fiber shall be colored to contrast with the area on which the fiber is to be applied shall be nontoxic to plant or animal life, and shall not stain concrete or painted surfaces.

3. Seed shall consist of the following (names and amounts are for example only):

Botanical Name (Common Name)	Percentage (Minimum) Purity	Percentage (Minimum) Germination	Pounds per acre
Lolium multiflorum (Annual ryegrass)	99	85	51
Trifolium incarnatum (Crimson clover)	98	85	17
Festuca arundinacea 'Alta' (Alta fescue)	98	85	13
Eschscholzia californica (California poppy orange)	90	85	4

4. Before seeding, the Contractor shall furnish written evidence (seed label or letter) to the Engineer that seed not required to be labeled under the California Food and Agricultural Code conforms to the purity and germination requirements in these special provisions.

5. Seed designated without a purity or germination shall be labeled to include the name, date (month and year)

collected, and the name and address of the seed supplier. Seed at the time of sowing shall be from the previous or current year's harvest.

6. Test methods specified in "Rules for Testing Seeds" from the Proceedings of the Association of Official Seed Analysts will be acceptable for determining the germination of seed.

7. All legumes shall be inoculated with a viable bacteria compatible for use with that species of seed. The application rate for seed shall be the weight exclusive of inoculated materials. All inoculated seed shall be labeled to show the weight of seed, the date of inoculation, and the weight and source of inoculant materials.

8. Inoculated seed shall be sown within 20 days of inoculation or shall be reinoculated.

9. The legume seed shall be inoculated as provided in Bulletin AXT-280, "Pellet Inoculation of Legume Seed," of the University of California, Agricultural Extension Service, except the inoculant shall be added at the rate of 5 times the amount recommended on the inoculant package.

10. Seed shall be mixed on the project site in the presence of the Engineer.

11. Commercial fertilizer shall have the following guaranteed chemical analysis:

Ingredient	Percentage (Minimum)
Nitrogen	16
Phosphoric Acid	20
Water Soluble Potash	0

12. Water shall be of such quality that it will promote germination and growth of seeds and plants.

13. The erosion control (Type D) materials shall be mixed and applied in approximately the following proportions:

Material	Per Acre (Slope Measurement)
Fiber	1,500 pounds
Seed	85 pounds
Commercial fertilizer	400 pounds
Water	As needed for application

14. The proportion of erosion control (Type D) materials may be changed by the Engineer to meet field conditions.

15. Mixing of erosion control (Type D) materials shall be performed in a tank with a built-in, continuous agitation system of sufficient operating capacity to produce a homogeneous slurry and a discharge system which will apply the slurry to the slopes at a continuous and uniform rate. The tank shall have a minimum capacity of 1,000 gallons. The Engineer may authorize use of equipment of smaller capacity if it is demonstrated that such equipment is capable of performing all the operations satisfactorily.

16. A dispersing agent may be added provided the Contractor furnishes evidence that the additive is not harmful to the mixture. Any material considered harmful, as determined by the Engineer, shall not be used.

17. The slurry shall be applied within 60 minutes after the seed has been added to the slurry.

18. The weight of fiber to be paid for will be determined by deducting from the weight of fiber, the weight of water in the fiber at the time of weighing in excess of 15 percent of the dry weight of the fiber. The percentage of water in the fiber shall be determined by Test Method No. Calif. 226, in the same manner as provided for determining the percentage of water in straw. Commercially packaged fiber shall have the moisture content of the fiber marked on the package.

19. Before using fiber a Certificate of Compliance as provided in Section 6-1.07, "Certificates of Compliance," of the Standard Specifications, shall be furnished to the Engineer.

C. Comments.

For all the effort, hydroseeding is not a great deal more effective than broadcast seeding unless a very heavy application of wood fiber (2000-3000 16/acre) is used. This produces a true surface mulch. Still, application is considerably less effective than straw mulch. For hydroseeding to reach its optimum value considerable knowledge or experience in suitable grasses, fertilizer requirements and mulching rates for local soils is desireable.

SECTION B: SURFACE EROSION

SPECIFICATIONS FOR DITCHES AND CONTOUR STRUCTURES

VIII. CONTOUR TRENCHES

A. Definition of job.

A contour trench is a structural measure used to control surface runoff and retard erosion. Contour trenches are discontinuous ditch-like structures dug on contour into the hillslope. They act as small reservoirs which catch surface runoff (and sediment in transport) before it has a chance to concentrate and develop rills and gullys on a hillslope. Runoff, generated during a storm, is stored in the trench until the post-storm period. During this period, water seeps through the trench into the soil. It is imperative that trench dimensions (width, depth) account for soil infiltration rates and expected short duration, peak rainfall rates. Soils with slow infiltration rates will require larger trenches. Unexcavated spaces between trenches on the same contour are an integral part of the trench. These spaces prevent excessive concentrations of water should a portion of a trench fail, and protects the remaining catch of a trench should only one segment fail. The storage capacity of a trench is eventually lost by slumping and sedimentation; however, it is hoped that both surface runoff will be sufficiently reduced, and the infiltration rate increased by established vegetation that the trenches will no longer be needed.

B. Job specifications. (see sketch)

1. Work shall progress from the top of the slope to be treated downward to prevent excessive soil compaction and damage to the trenches.

2. The grade for contour trenches shall be absolutely level. The grade shall be staked with Abney level, string level, or similar devices, and shall follow slope contour (i.e. trenches shall be horizontal).

3. Contour trenches shall be 10 feet long and spaced 5 feet apart on the contour.

4. Spacing between rows of contour trenches shall be 6 feet (slope distance), or 3 feet vertically, whichever is less.

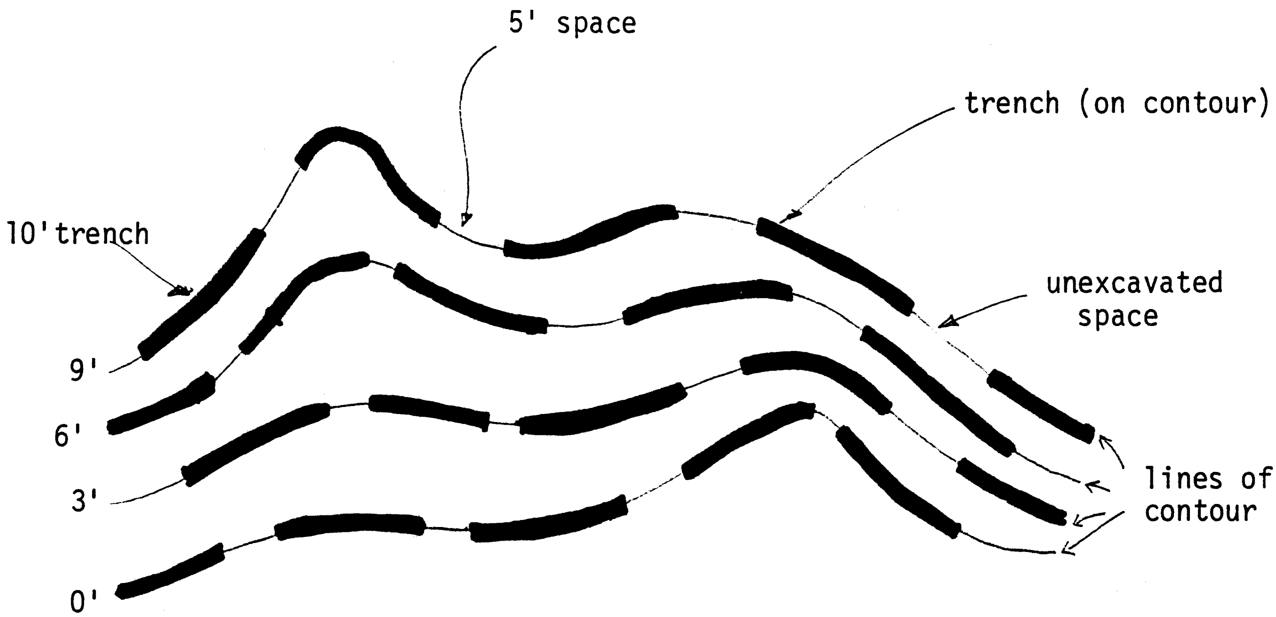
5. Trenches shall be excavated to a minimum depth of 8 inches and a width of 14 inches across the top.

6. Trenches and unexcavated spaces shall be spaced in a staggered pattern.

C. Comments.

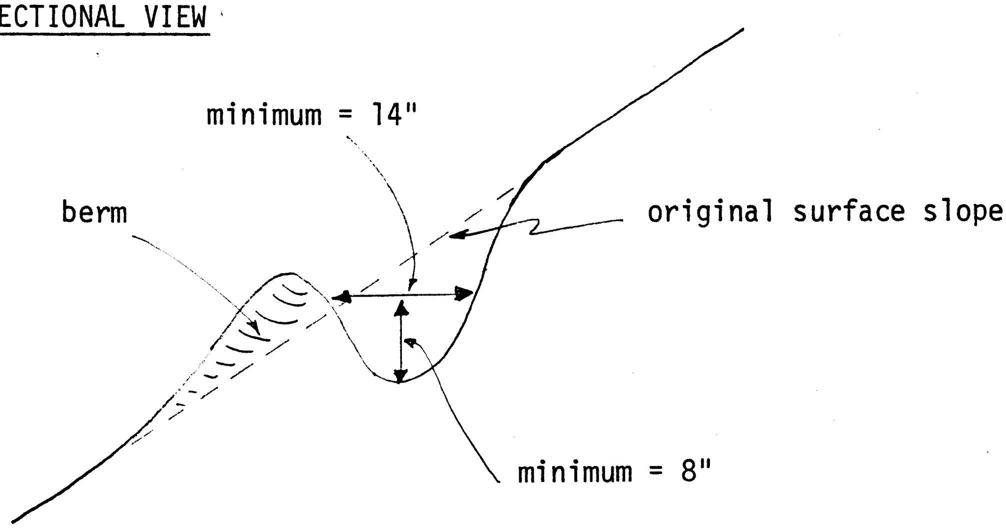
These specifications were developed for clay-loam soils; annual precipitation of 80 inches and peak 24-hour, 2-year rainfall of 5.5 inches. Designs should be modified to fit site conditions (slope, soil) and climate (peak rainfall rates).

PLAN VIEW



relative elevations of contour lines

CROSS SECTIONAL VIEW



SCHEMATIC DRAWINGS OF CONTOUR TRENCHES

IX. DITCHES

A. Definition of job.

Hand dug ditches are used to drain wet areas and divert surface runoff to stable areas. They are generally shallow compared to those dug by machines and are gently sloping.

B. Specifications.

1. Ditches shall be excavated at least 8 inches into mineral soil.

2. Top width of ditch shall be at least 12 inches.

3. The ditch shall slope gently towards direction of discharge (not so steeply as to erode its bed.

4. Ditches shall be free and clear of organic debris, soil or rocks which could block the flow of water.

5. Ditches shall discharge onto slash or rocks or similar energy dissipating materials.

6. Soil excavated during ditch construction shall be piled onto the downslope edge of ditch as a continuous berm so as to contain excess flows within ditch area.

7. In swampy areas to be drained, a number of small "feeder" channels shall be etched into the soil to drain standing water and saturated soils towards the beginning of the main drainage ditch.

C. Comments.

Drainage ditches constructed by hand are relatively expensive and lack the capacity to carry significant discharges. However, in remote areas, ditches can be useful in diverting perennial spring flows away from sensitive hillslopes and unstable areas.

VI. WATERBARS

A. Definition of job.

Waterbars serve to divert surface runoff from bare soil areas (typically trails, skid trails and roads) onto vegetated areas or other areas where the flowing water is less apt to cause soil erosion. To satisfactorily accomplish this purpose, waterbars shall:

1. Be of sufficient dimensions to accommodate the surface runoff they divert without being overtopped or otherwise failing.

2. Be located properly to successfully divert all the water they are intended to intercept (i.e., when used on a skid trail, they shall extend from the inside edge of the trail to slightly beyond the outside edge of the bare soil area.)

3. Be angled down the slope sufficiently to allow water to drain through the trough of the waterbar and freely discharge at the correct end of the structure. Thus, the slope of the waterbar shall be sufficient to drain the intercepted surface runoff without allowing ponding, yet not so steep as to cause erosion or gullying of the bottom of the trough.

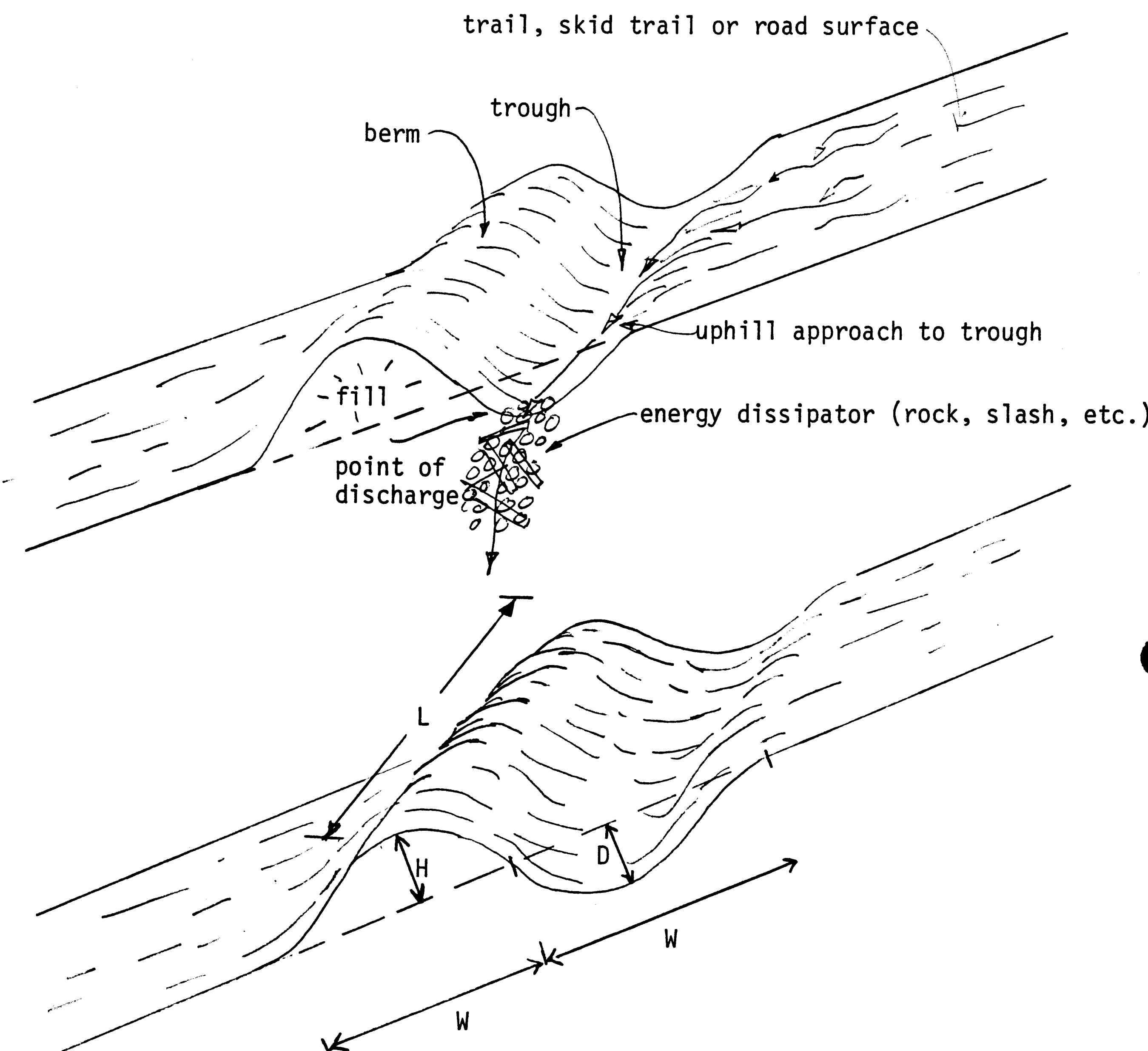
4. Be constructed so the lower or discharging end of the waterbar is clear and free from debris and allows for the free discharge of runoff.

5. Be constructed so the point of discharge is onto slash (organic debris), rock, or some other form of energy dissipation. Runoff through the downslope end of the waterbar trough shall not be allowed to erode the soil in that location or within at least three feet immediately downslope. Sufficient energy dissipation shall be provided to prevent future erosion resulting from diversion of flow by the waterbar. Waterbars which discharge on steep bare slopes may cause erosional problems if not installed with energy dissipation at their discharge ends.

B. Specifications for New Construction. (see sketch)

1. Waterbar trough shall be excavated at least 8 inches into firm substrate (d = 8").

2. Trough shall be at least 12 inches wide (w = 12"), with a gentle uphill approach to the trough.



L = length (average = 10 feet)
H = berm height (minimum = 6 in.)
D = trough depth (minimum = 6 in.)
W = trough and berm width (min. = 6 in.)

SCHEMATIC DRAWING OF WATERBARS with construction specifications)

3. Trough shall be free and clear of debris or other obstructions so as to drain freely without ponding water.

4. Trough shall have a gentle slope toward the discharging end (there shall be a total drop of 6 inches to 18 inches along the run of a typical 10-foot long trough).

5. Trough shall abut inside bank of road or trail or otherwise be constructed to assure total diversion of runoff.

6. Berm shall be at least 8 inches high (h = 8") and 12 inches wide (w = 12").

7. Berm shall be composed of on-site inorganic sediment (rock and subsoil, preferably that excavated from the trough) and shall be tamped with shovel, feet or otherwise hand-compacted.

8. Point of discharge shall be free and clear of debris so as to allow all water to drain freely from the trough.

9. Berm shall be constructed so as not to allow surface runoff to flow over or around it.

10. From point of discharge for a distance of 3 feet (slope distance) downslope, energy dissipation shall be placed in the path of the diverted surface runoff -- this shall primarily consist of rocks 5 to 12 inches in diameter and secondarily (if sufficient numbers of rocks cannot be found within 100 feet of site) of slash or other woody debris no larger than 12 inches in diameter and 24 inches in length.

C. Specifications for repairing waterbars. (see sketches)

1. Opening or unblocking point of discharge. (Open end of waterbar)

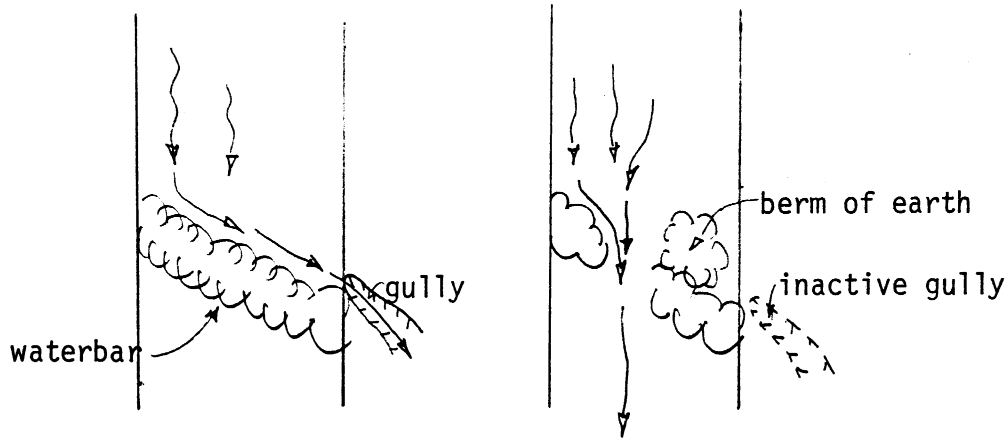
   The discharging end of the waterbar shall be cleared of organic debris, soil and rock which is preventing or hindering the free flow of water from the trough. Energy dissipation shall be placed below the point of discharge if there exists a gully over 8 inches deep and wide at that point and extending at least 3 feet downslope.

2. Clean out trough of waterbar.

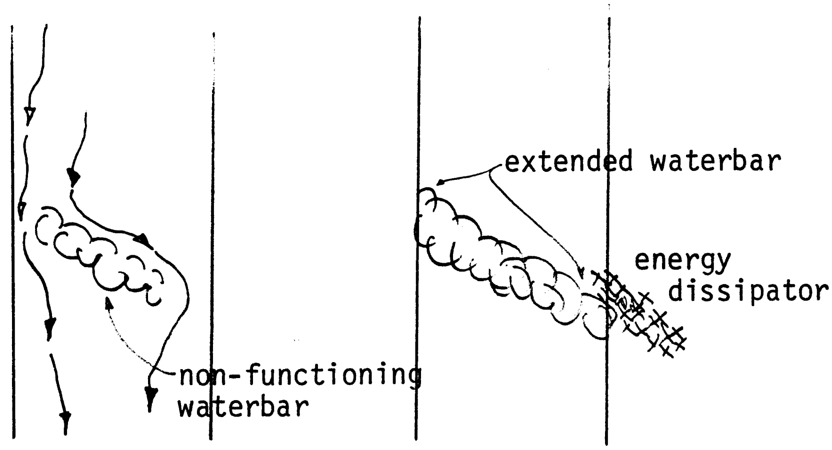
   The trough shall be cleaned of organic debris, soil and rock so as to allow free drainage through the trough and across the point of discharge. If the bare slope below

Contracted Repair

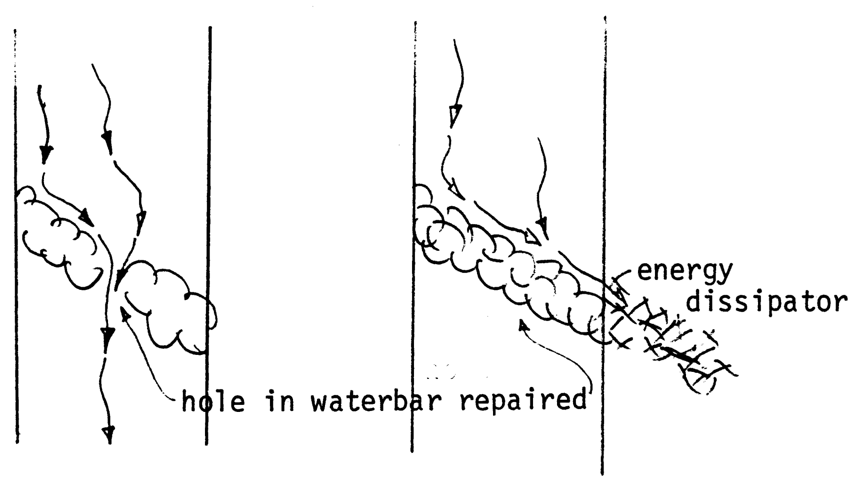
1. BREACH WATERBAR



2. EXTEND END(S) OF WATERBAR



3. REPAIR BREACHED WATERBAR



the point of discharge displays a gully greater than 8 inches in width and depth and 36 inches in length, energy dissipation shall be installed.

3. Extended end(s) of waterbar. (see sketch)

Additions to the existing waterbar shall be built at one or both ends of the waterbar so as to prevent water from flowing around the waterbar structure rather than being diverted by it. Typically, the upper end is not extended far enough downslope, so the surface runoff entering the trough presently flows around the downslope end of the waterbar rather than through the point of discharge. If not present, energy dissipation shall be provided.

4. Breach waterbars. (see sketch)

Some waterbars are doing more damage than good at their present location, and so shall be destroyed. To accomplish this, a 4 foot wide cut shall be made directly across the berm at the point opposite where most of the surface runoff is entering the trough from upslope. Excavated material shall be packed into the trough so as to assure all the water entering the pre-existing waterbar will now flow through the opened berm and not down the former trough (this point will likely be located just downslope in the trough below the new cut in the berm, thereby acting as a dam to surface flow.) The berm shall be cut down and the floor of the remaining trough built up to the level of the former surface so the new profile is smooth. The cuts in the berm should be sloped toward each other with at least 2 feet of "flat" channel between.

D. Comments.

Properly constructed and functioning waterbars are very effective in diverting and dispersing concentrated runoff from trails and other bare soil areas. They are commonly used on unsurfaced forest roads and in logging areas. Unfortunately, waterbars are both difficult and relatively expensive to construct by hand, especially on compacted surfaces.

XI. WATTLING

A. Definition of job.

Wattles are bundles of flexible twigs and branches tied together. Wattling is the process of placing wattles in contour trenches on slopes, staking the wattles in place, and then covering the wattles with soil. Once in place, wattles serve to retard surface erosion and revegetate bare slopes through sprouting of roots and branches from the bundles.

B. Specifications. (see figure)

1. Wattling bundles should be prepared from live material, native to the site. Willow (*Salix* spp.) is generally the preferred plant. Coyote brush (*Baccharis*) may be suitable for dry sites, and non-sprouting, species such as alder (*Alnus*) may be used for wattles as a physical means of erosion control. Specific species for wattling will be designated in the S.O.W.

2. Wattling bundles may vary in length, but must taper at the ends, and the longest stems shall be 1 1/2 feet longer than the average length of the stems to achieve the taper. Butts of individual stems shall not be more than 1 1/2 inches in diameter.

3. Stems should be placed alternately in each bundle so that approximately half the butt ends are at each end of the wattle.

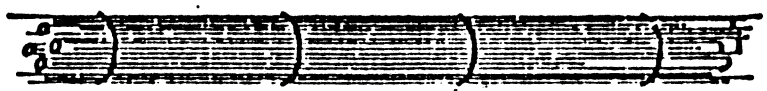
4. Bundles shall be tied at not more than 15-inch spacings with 2 wraps of binding twine, or heavy tying material, with a non-slipping knot. When compressed firmly and tied, each bundle shall be approximately 8 inches in diameter (minimum, 6 inches; maximum, 12 inches.)

5. Bundles shall be cut and tied not more than one day in advance of placement and the bundles shall be kept covered and wet between the time of cutting and installation. Cutting, tying and placing in trenches on the same day is desirable.

6. The grade for the wattling trenches should be staked out (see specifications 10 and 11 below) with an Abney level, string level, or similar device, and shall follow slope contours, (i.e., horizontal trenches.)

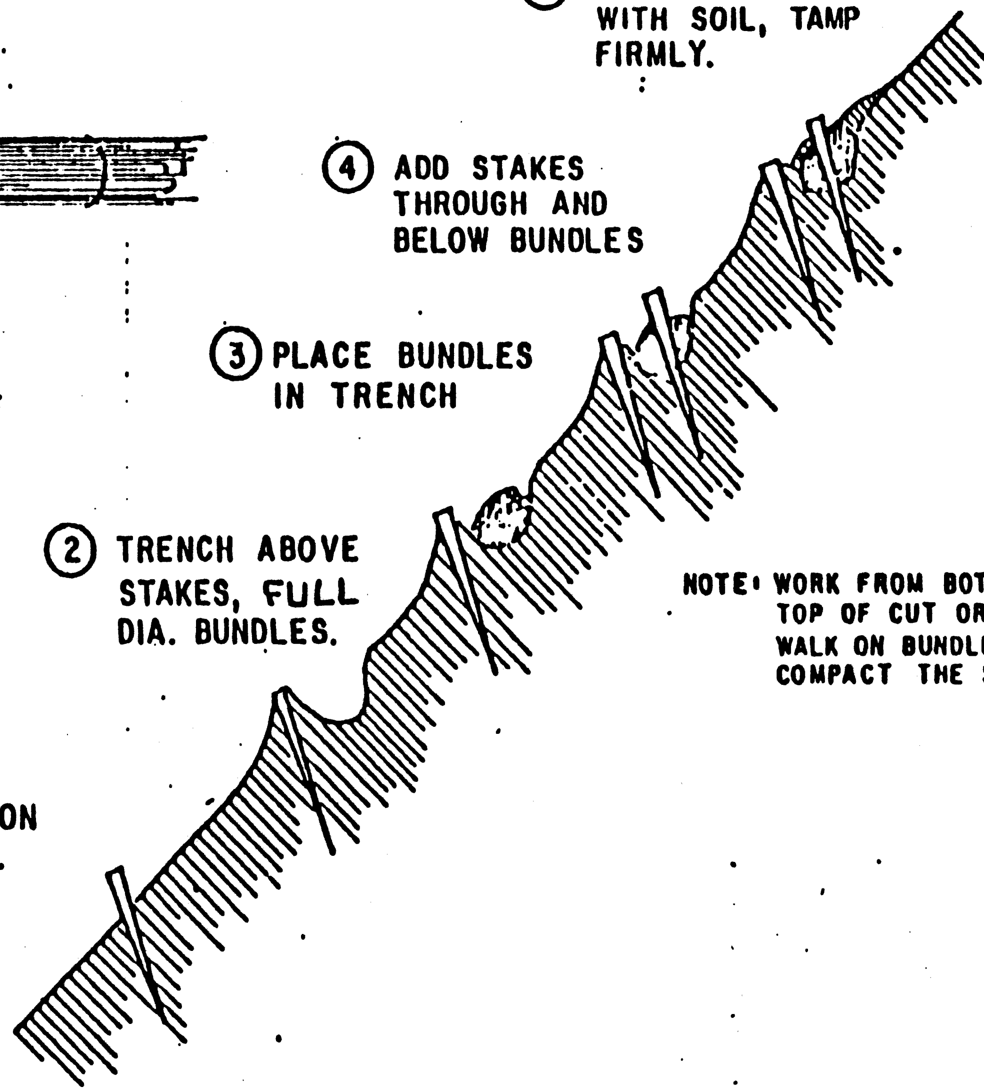
7. Trenches shall be spaced three feet apart, vertically, unless otherwise specified in the S.O.W.

PROCEDURE FOR WATTLING A BARE HILLSLOPE



WATTLING BUNDLE

CIGAR SHAPED BUNDLES OF LIVE BRUSH WITH BUTTS ALTERNATING,



NOTE: WORK FROM BOTTOM TO TOP OF CUT OR FILL. WALK ON BUNDLES TO COMPACT THE SOIL.

NOTE: THIS DRAWING IS BASED ON INFORMATION PROVIDED BY DR. A. LEISER WITH DEPT. of ENVIRONMENTAL HORTICULTURE of U.C. DAVIS.

XI.B. (Con't)

8. Bundles shall be laid in trenches dug to a depth equal to the diameter of the bundles, with ends of the bundles overlapping at least 12 inches. The overlap shall be as long as necessary to permit staking as specified below.

9. Bundles shall be staked firmly in place with vertical stakes on the downhillside of the wattle at no more than 36-inch spacing or closer if stated in the S.O.W. and at least one stake shall be driven through each bundle. In any case, a bottom stake should be placed at the mid-point of the bundle overlap.

10. Stakes shall be greater than 1 1/4 inches in diameter and 24 inches long.

11. All stakes shall be driven to a firm hold and at least 15 inches deep. Where soils are soft and 24-inch stakes are not solid, longer stakes as necessary should be used. Where soils are rocky and/or compacted, steel bars should be used to open up stake holes for the stakes. Stake depths may be waived by the Contracting Officer or his/her representative on a site-specific basis at difficult sites where it is impossible to always meet minimum stake depths.

12. Work shall progress upward from the bottom of the slope to be wattled. The buried wattles shall have soil firmly tamped around them to minimize the possibility of drying out, however, the terracing effect created by the contour trenching shall be preserved.

C. Comments.

The effectiveness of wattles is largely dependent: on 1) choice of proper plant materials, 2) proper installation techniques, and 3) favorable soil and environmental conditions. In many areas wattling is an extremely effective erosion control practice. In other localities, less expensive procedures (e.g., straw mulch) can be more cost-effective.

XII. WOODED TERRACES

A. Definition of job.

A wooded terrace is a terrace constructed on a contour and supported by woody material. A wooded terrace is a structural measure which can retard surface erosion and hasten the establishment of vegetation.

B. Specifications.

1. Woody material is defined as limb, split product material or bark.

2. Statement of Work (S.O.W.) shall specify spacing (slope or vertical) of wooded terrace rows. If not specified a vertical spacing of 3 feet shall be used.

3. The grade for wooded terraces shall be level. It shall be staked with an Abney level, string level, or similar device to follow slope contours (i.e., horizontal terraces).

4. Cumulative diameter of woody material placed in a terrace shall be at least 8 inches. There is no maximum length for woody material; however, wood must contact the slope along its entire length.

5. Wooden stakes, driven vertically into the hillslopes, shall anchor the wooded terraces. Stakes shall be greater than 1 1/4 inches in diameter and at least 24 inches long.

6. The maximum allowable spacing for stakes is 20 inches for woody material less than 40 inches in length; 30 inches for woody material greater than 40 inches in length. All ends of woody material must overlap stakes a minimum of 1 foot.

7. All stakes should be drive to a firm hold and at least 15 inches deep. Where soils are soft and 24 inch stakes are not solid, deeper stakes as necessary shall be used. Stake depths may be waived by the Contracting Officer or his/her representative on a site-specific basis at difficult sites where it is impossible to always meet minimum stake depths.

8. Procedure for constructing multiple level wooded terraces:

    a. begin at bottom of slope to be terraced and work upward.

XII.B. (Cont'd)

b. stake grade of the first terrace.

c. lay a row of woody material and drive stakes against the downhill side along the entire row.

d. back fill and cover the row of woody material with clean soil found immediately upslope from the row until a flat terrace is formed. Tamp soil.

e. repeat on next upslope level.

C. Comments.

A vertical spacing of 3 feet seems to work well (Spec. #2). Vertical spacings ensure closer spacings on steep slopes and wide spacings on gentle slopes. Stakes made of cuttings of sprouting species (e.g. Willow) can aid in revegetation. Because of their wide level benches, wooded terraces are effective at trapping sediment eroded from upslope areas. However, if they are not constructed absolutely on the contour they may actually collect and concentrate hillslope runoff. In addition, they are relatively expensive to install.

XIII. RAVEL CATCHERS

A. Definition of job.

Ravel catchers are boards, dug slightly into the hillside, and placed on contour (see figure). They catch and store dry ravel during the summer and sheet and rill erosion products. Ravel catchers can be placed on steep slopes where soil dry-ravels or can be easily washed downhill. When placed on cutbanks and/or other exposed subsoil, and then partially backfilled with fertile soil, ravel catchers can also act as planting sites for woody vegetation.

B. Specifications.

1. Ravel catchers shall be made of split or milled boards, or other suitable material specified by the S.O.W.

2. Boards shall be 16 inches - 18 inches wide, at least 1 1/4 inches thick and 4 feet to 5 feet long. In some cases, ravel catchers shall be continuous and the length of boards will be determined by hillslope micro-topographic characteristics.

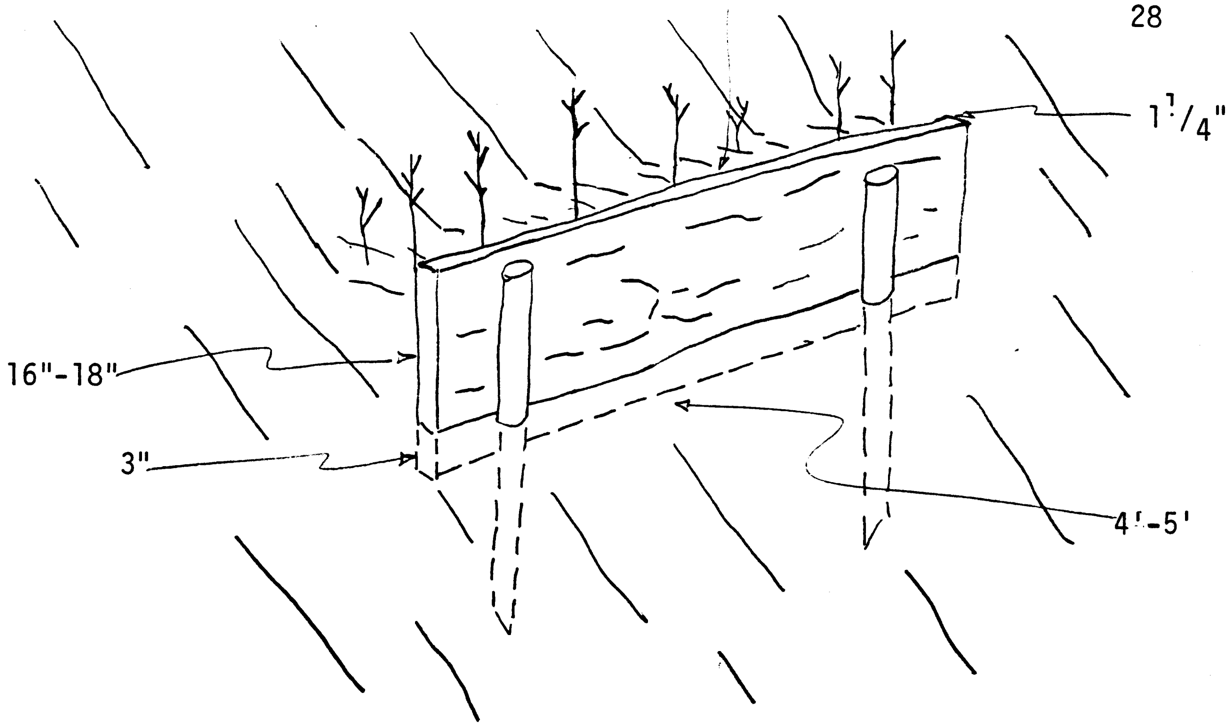
3. A trench at least 3 inches deep shall be dug the length of the board; the board shall be placed vertically and anchored by wood stakes.

4. Stakes greater than 24 inches long shall be driven on the downslope side of the board, spaced no greater than 30 inches apart.

5. Once placed, the boards shall be partially backfilled with soil.

C. Comments.

Ravel catchers should not be constructed long enough to collect and divert significant quantities of surface runoff. Ten feet is an upper limit, with 3 to 5 feet lengths preferable. They work best on steep slopes (>50%) which are prone to dry ravel.



SCHEMATIC DRAWING OF RAVEL CATCHER ON BARE SLOPE

SECTION C: REVEGETATION

XIV. STEM CUTTINGS

A. Definition of job.

A stem cutting is a shoot, or cane, cut from a live tree or shrub. Cuttings from sprouting plant species will grow if planted in the ground under certain conditions.

B. Specifications.

1. Prepared cuttings shall have the following characteristics:

   a. From healthy wood of a sprouting plant species native to the planting site.

   b. Reasonable straightness.

   c. Clean cuts with unsplit ends.

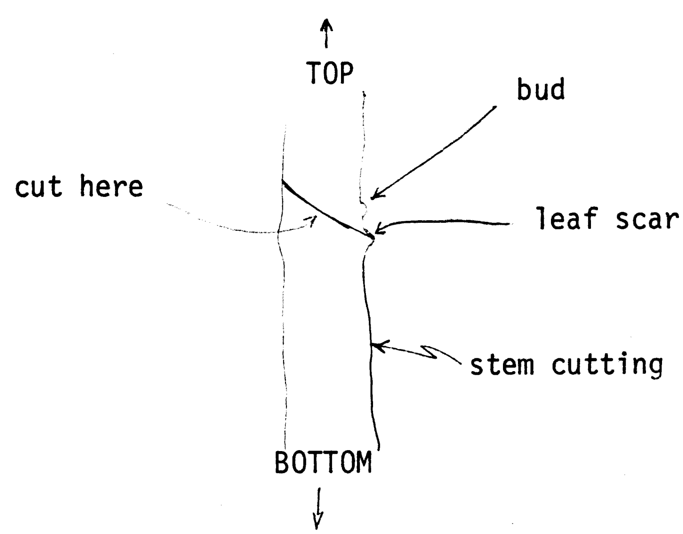
   d. Length: 12-inch minimum length.

   e. Diameter 1/4 - inch minimum diameter; the thicker the cutting, the greater the reserves. Therefore, cuttings greater than 1 inch are desirable, though their numbers may be limited by the supply.

   f. Stem cuttings shall not be from the tips of branches but rather farther back on the stems. The top of each cutting shall be just above a leaf bud, the bottom cut just below one.

   g. Trim branches from cuttings as close as possible.

   h. At least 2 lateral buds shall be above the ground after planting. (see sketch)



XIV.B. (Con't)

2. Leaves shall be stripped from cuttings used before normal leaf fall occurs.

3. Handling of cuttings between cutting and planting: Cuttings must not be allowed to dry out. Cuttings may be planted the same day, and at all times must be kept covered and moist during transport and storage before planting. Under certain dry conditions of either the cutting site or the planting site, the Contracting Officer or his/her representative may require that cuttings be soaked at least 1 day prior to planting, though mandatory soaking will be uncommon. At no time shall a cutting be left exposed to the air to dry out prior to planting.

4. Planting of cuttings: Cuttings must be planted right-side-up. At least 50% of the cuttings length should be planted in the ground; it is preferable if 75% of the cutting length is in the ground, but at least two budding nodes shall be left exposed above ground. Deep planting minimizes loss of water due to transpiration. Soil shall be firmly pressed around cutting to reduce moisture loss.

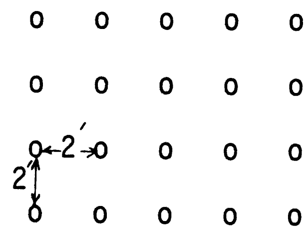
5. Time of cutting planting: Basically, planting time is between September and April; earliest possible planting time for wet sites is after first major storm in fall (greater than 1 inch rain.) For dry sites, earliest planting time is after second major storm. Latest possible date is dependent on the particular year, but approximately March 1st. Additional soaking prior to planting may be required for late plantings. Optimum planting time is October through February, when ground is wet and plant material is dormant.

6. Cutting Willow and other brushy species for planting: Cutting of plant material for use as wattles or cuttings will be done to minimize disturbance of vegetation and soil adjacent to the willow stands. Conifers must not be damaged. Ground cover must be preserved as much as possible; trails with bare soil from trampling the brush must not occur. Willows should be used as efficiently as possible; i.e., when stakes for wattles are cut, excess branches should be used as cuttings or wattle bundle material. Willow shoots must be cut by either pruning shears, hand saw or chain saw. Branches from willow must be cut diagonally to expose more surface area to water and to provide a pointed end for stake driving and sprig planting. The basal ends of the shoots must be marked clearly in some manner so workers can determine which end to plant. Correct species identification is essential,

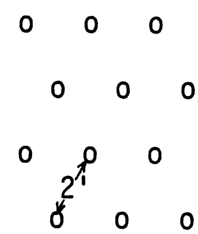
XIV.B. (Con'd)

particularly in the willows and alders which often look similar but have different habitat requirements which in turn may result in different survival success. Species identification should be confirmed by qualified personnel before collection.

7. Placement of stem cuttings and transplants: The required planting distance between transplants and/or stem cuttings will be stated in the S.O.W. as "2 foot spacing" or "3 foot spacing" etc. The rows must be staggered rather than be in columns:



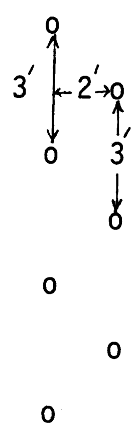
2-foot spacing
Wrong Way



2-foot spacing
Right Way

Where the contract specifies planting in a zigzag pattern, x foot spacing, y foot offset, a double row is desired with x number of feet between each cutting or transplant in that row and the second row y number of feet to the side,

Ex: zigzag pattern, 3-foot spacing, 2-foot offset



C. Comments.

Planting stem cuttings of sprouting species can be an inexpensive, successful method for revegetating disturbed sites, especially if planting stock is composed of locally

XIV.C. (Con't)

abundant native species. Proper identification of native species can be assured by consulting local experts in plant taxonomy or by using references such as Abrams Illustrated Flora of The Pacific States (Stanford Univ. Press).

It is imperative, for the success of any revegetation project utilizing stem cuttings, (for example direct planting or wattling), that you select only sprouting species which will survive in the project area's micro-environment. It is surprisingly easy to waste time and money by using species which have low or no sprouting potential, or by planting sprouting species where they will not survive. Common sense, professional advice, and simple field or greenhouse experiments can virtually eliminate these problems.

Stem cuttings are often planted in the following types of locations:

1. Slopes: bare soil areas that show evidence of recent movement or active erosion of surface particles. Especially well suited for cuttings are persistent wet areas, road-cut slopes where soil conditions permit, and raw soil areas on slumps.

2. Gullies and channels: areas best suited to use of cuttings are the floors and banks of small incipient gullies, sediment fill behind check dams, raw gully banks, berms of waterbars and the area just below waterbar outlets, if suitable soil conditions exist.

3. In addition, any other location where cuttings may be deemed useful in establishing vegetation for minimizing erosion.

XV. TRANSPLANTS

A. Definition of job.

Transplanting is the intact removal of an individual plant from one place and replanting it in another.

B. Specifications.

1. Although determining the size of an adequate root ball is necessarily a judgemental decision best made on a plant-by-plant basis in the field, all plants must be dug with a ball of soil containing at least 60% of their roots. If the soil is dry, the soil around the plant shall be soaked prior to digging so that the root ball will hold together. Plants must be transported to the site in such a way that the root ball does not shatter, exposing the roots. (Size of transplant and root ball varies with species; see species specific specifications below.)

2. All species shall be replanted within a maximum of 24 hours of being dug up. The root ball must be kept moist at all times to keep the roots from drying out.

3. The planting hole shall be large enough to accommodate the root ball easily, without cramping, bending or cutting roots. Adjust planting depth so that the old soil line (usually visible near the base of trunk or stems) is at the surface level of soil surrounding the planting hole.

4. The hole shall then be refilled about 3/4 full with soil, firmed around the roots and thoroughly watered. If settling occurs, the plant shall be readjusted and the remaining soil added, again firming the soil to eliminate any air pockets.

5. Transplants shall be obtained in such a way that at least one half of the original plants of the species remain scattered within the collection area. The source area <u>must not be</u> denuded of plants.

6. Holes created by removal of plants shall be filled with soil to the original soil surface.

7. Alder (<u>Alnus oregana</u>), coyote brush (<u>Baccharis pilularis</u> var. <u>consanguinea</u>) and rhododendron (<u>Rhododendron macrophyllum</u>) transplants: Minimum size plants shall be 6 inches high; maximum, 24 inches high. The larger the plant, the larger the root ball. At a minimum the surface circumference of the root ball shall equal the circle made at the drip line of the plant's canopy.

XV. B. (Con't)

8. Deerfer~~m~~n (*Blechnum spicatum*) and sword fern (*Polystichum munitum*) transplants: Minimum basal diameter of fern dump shall be 4 inches, root ball shall include a minimum of 75% of plant's roots.

9. Rush "plugs": Correct species identification is essential. Species identification shall be confirmed by qualified personnel before collection. *Juncus* "plugs" each with a 2 inch minimum basal diameter, may be obtained by dividing larger clumps.

10. Salal (*Gaultheria shallon*) and yerba de selva (*Whipplea modesta*) transplants: Both species root at the nodes though salal does so less frequently. Transplants shall have root balls at least 8 inches in diameter and it is desirable to include at least 10 inches of the underground stems whenever encountered. Large plants may be divided, provided each division has the 8" root ball.

11. *Placement of transplants*: See XIV B. 7. Placement of stem cuttings and transplants.

C. *Comments.*

Transplant specifications used in #7, #8, and #9 above are examples for species in north coastal California. Similar specifications can be prepared for virtually any native species. Alternatives to field transplanting include direct seeding of native species or, for more rapid results, contracting at least one year in advance for someone to grow bulk numbers of containerized stock which can then be out-planted with excellent success. Example contract specifications for this method can be requested from Redwood National Park or many agricultural colleges.

SECTION D: CHANNEL EROSION

XVI. ROCK ARMOR (hand placed)

A. Definition of job.

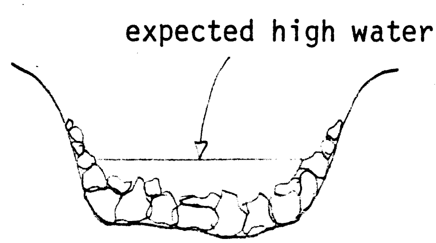
Rock armor is placed in small stream channels, gullies or other expected flow courses to increase turbulence and energy expenditures, slow velocities and eliminate scour of channel banks and beds.

B. Specifications.

1. Peak 20-year discharges for the channel reach shall be calculated using acceptable formulas (Rational method, SCS, etc.) Estimates must be substantiated by field evidence.

2. For newly constructed channels, the channel bottom shall be made wide enough to handle peak flows. Wide, shallow channels are preferable to deep narrow cross sections.

3. When the S.O.W. calls for channel excavation and rocking channel bed, the channel will be excavated in such a way that the bed is slightly concave, and rocks will be placed far enough up the channel banks to contain anticipated heavy flow. This is an effort to correct failures due to flat-bottomed, rocked channels, where bank cutting can occur during high water.



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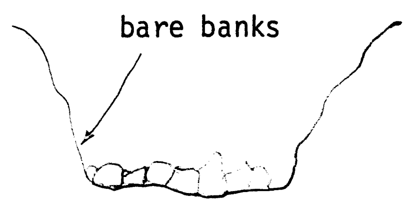


NOT THIS...

4. Sufficient quantities of rock shall be used to adequately protect and armor the bed of the channel.

5. Rock sizes and/or securing techniques shall be employed to assure that peak flows do not remove the protective material. A heterogeneous mixture of rock sizes shall be used which contains enough large rocks (rocks which cannot be moved during peak flows) to keep smaller rocks in place. Where only small rocks are available, securing techniques such as staking or wire reinforcing shall be used to anchor the armor material to the bed.

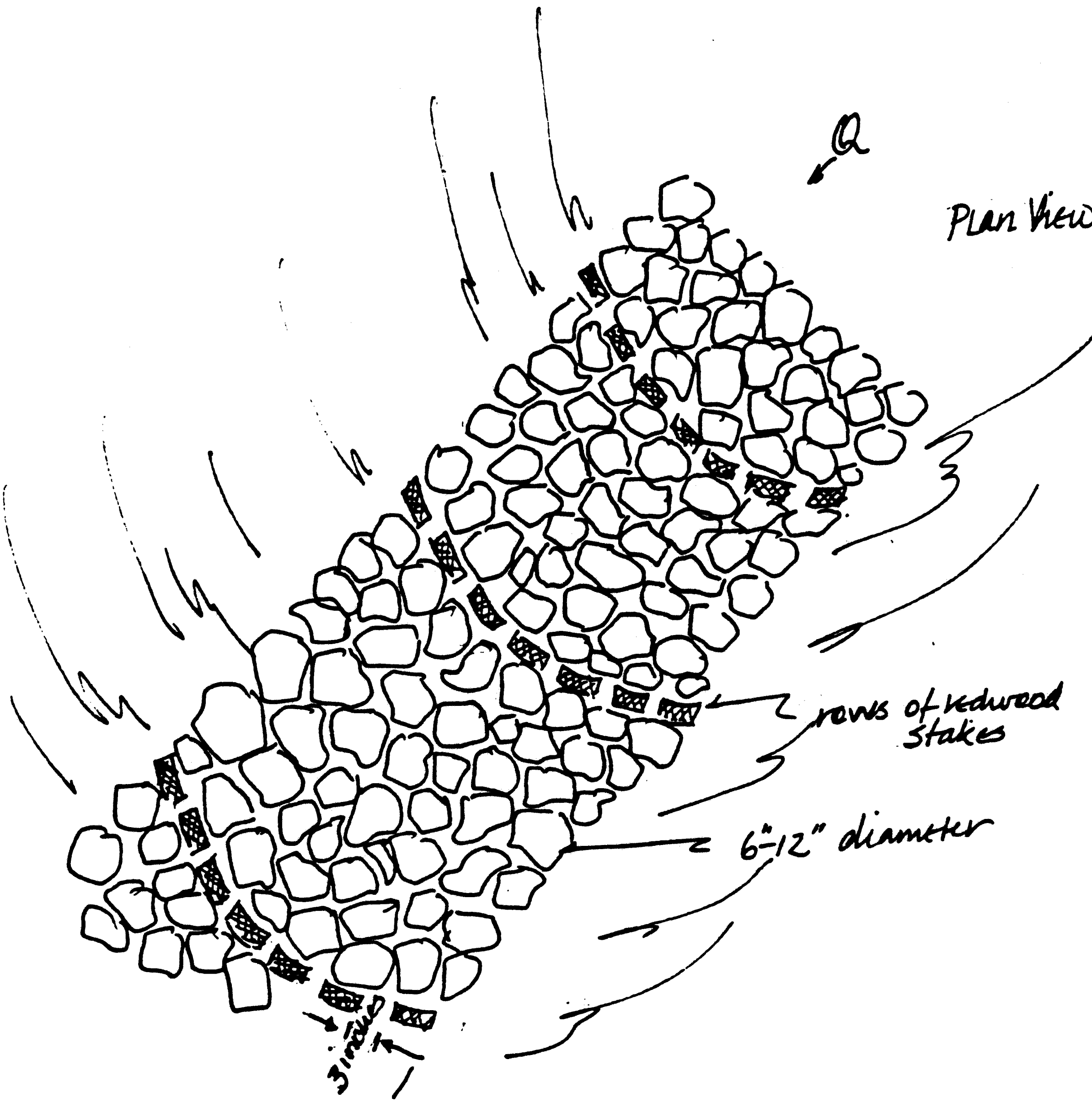
XVI. B. (Con't)

6. Rocks shall not be so large as to deflect streamflow into the banks.

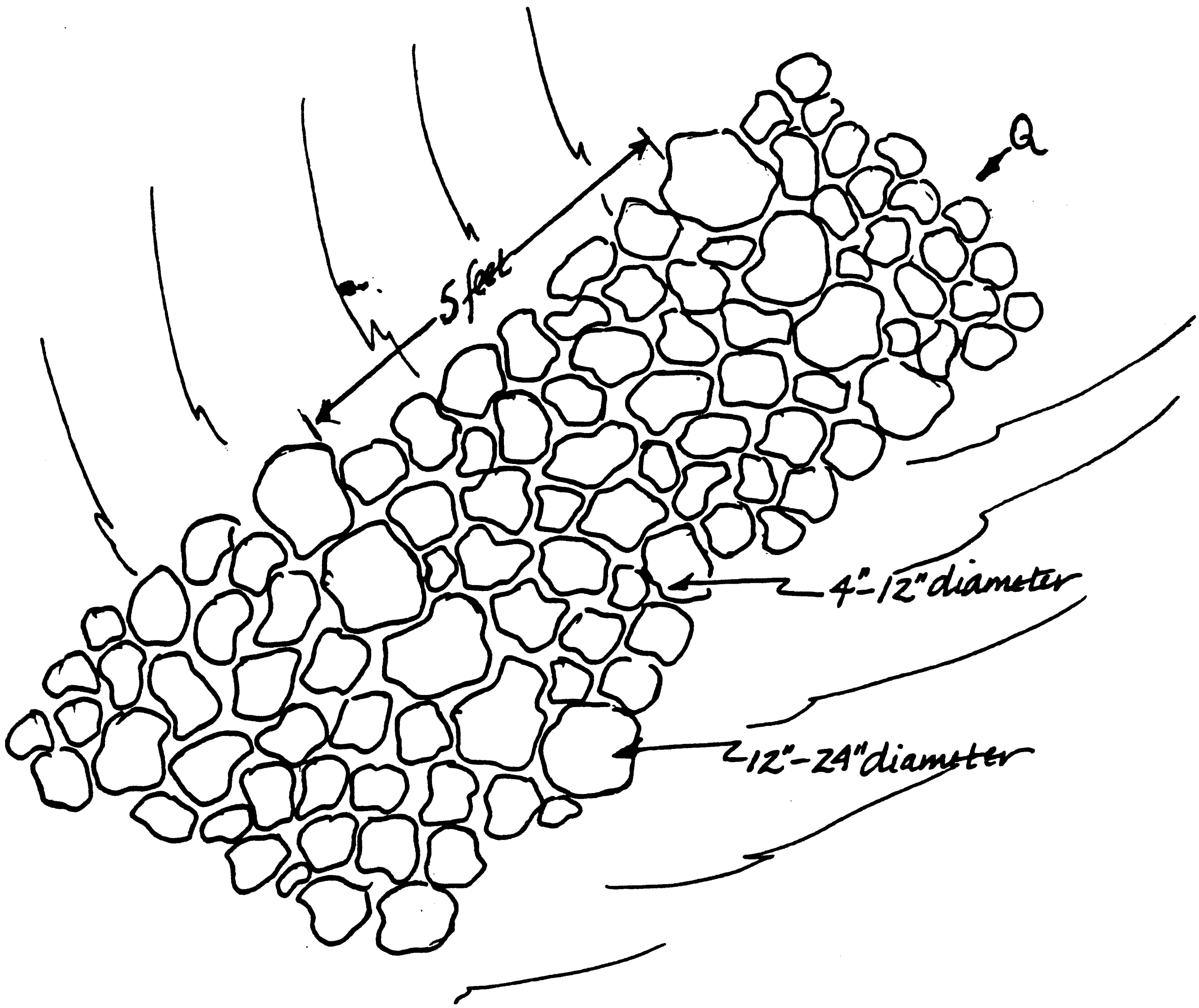
C. Comments.

The successful application of hand placed rock armor is limited by the maximum size of rock that can be moved in the channel. Rocks larger than 18 inches diameter are difficult to handle. In addition, in remote areas adequate sources of rock may not be locally available. Armoring with insufficient rock coverage or with rocks which will be transported by peak flows provides little channel protection. Finally, as with most erosion control work, regular maintenance is needed for several seasons following installation.

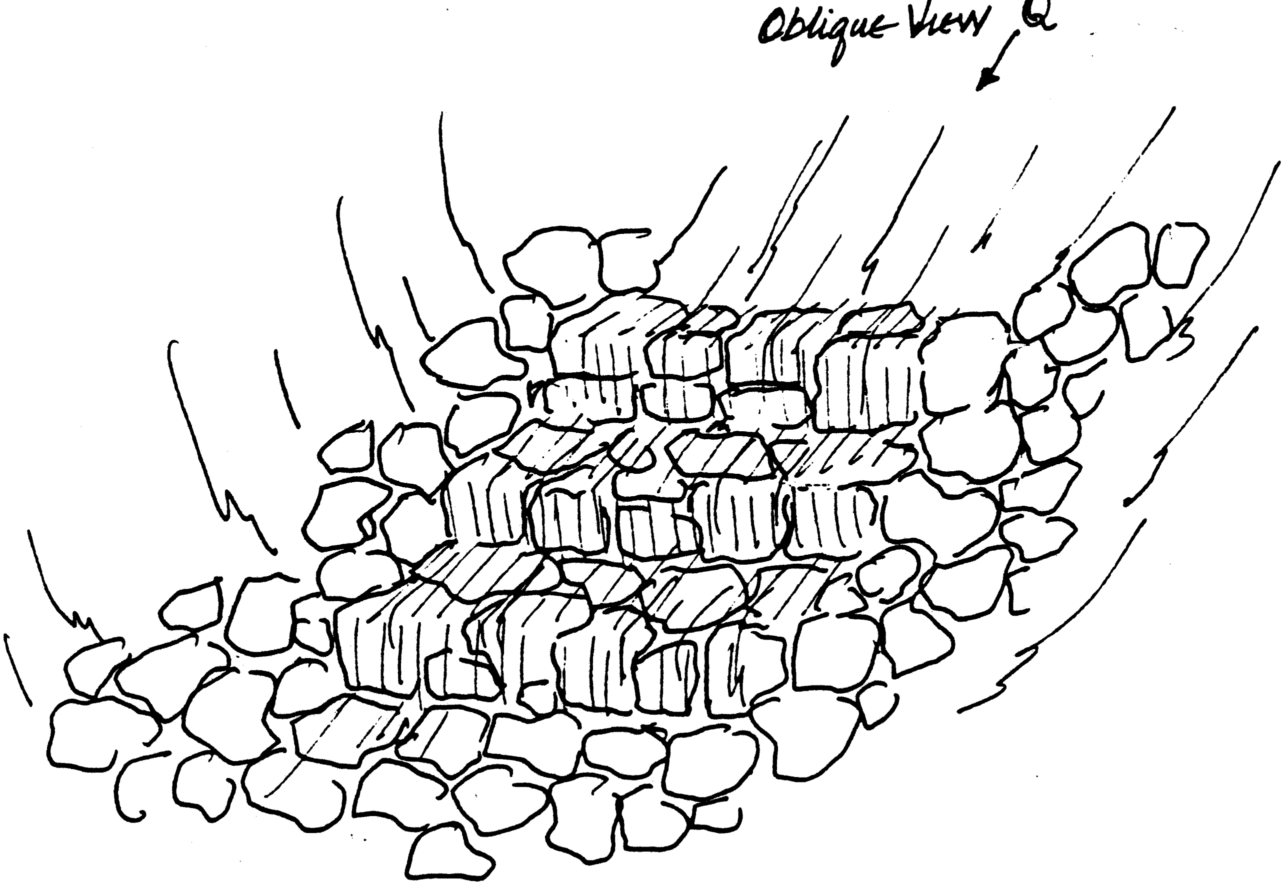
ROCKED & STAKED CHANNEL
Schematic Drawing

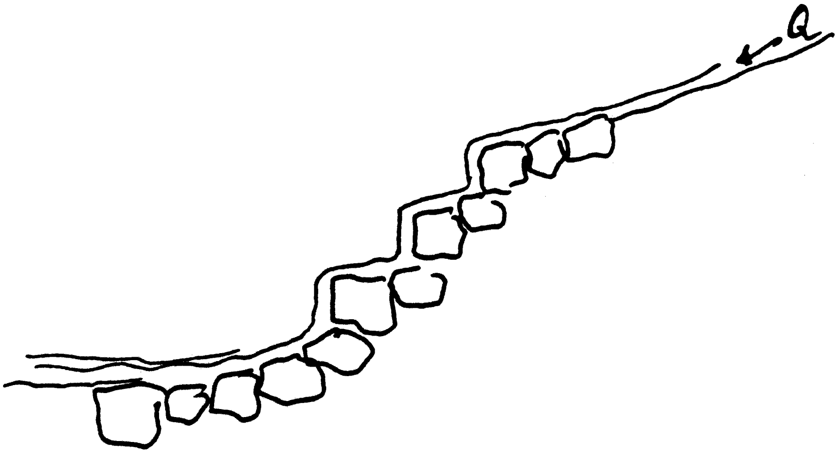


RIBBON of LARGE ROCKS in ROCKED CHANNEL
Schematic Drawing



Rock Steps
Schematic Drawing





XVII. CHECK DAMS

A. Definition of job.

Check dams are constructed in gullies and stream channels to prevent scour of the bed and banks. By raising local base levels, the sediment fill behind dams can stabilize the adjacent channel bank by preventing gully downcutting and lateral cutting; provided runoff is directed through the spillway of the check dam and the dam is not undermined by channel downcutting from below the dam. The sediment fill behind the check dams and the raw soils on the adjacent channel banks (slopes) are planted heavily with cuttings or transplants after the check dams have been installed.

B. Specifications.

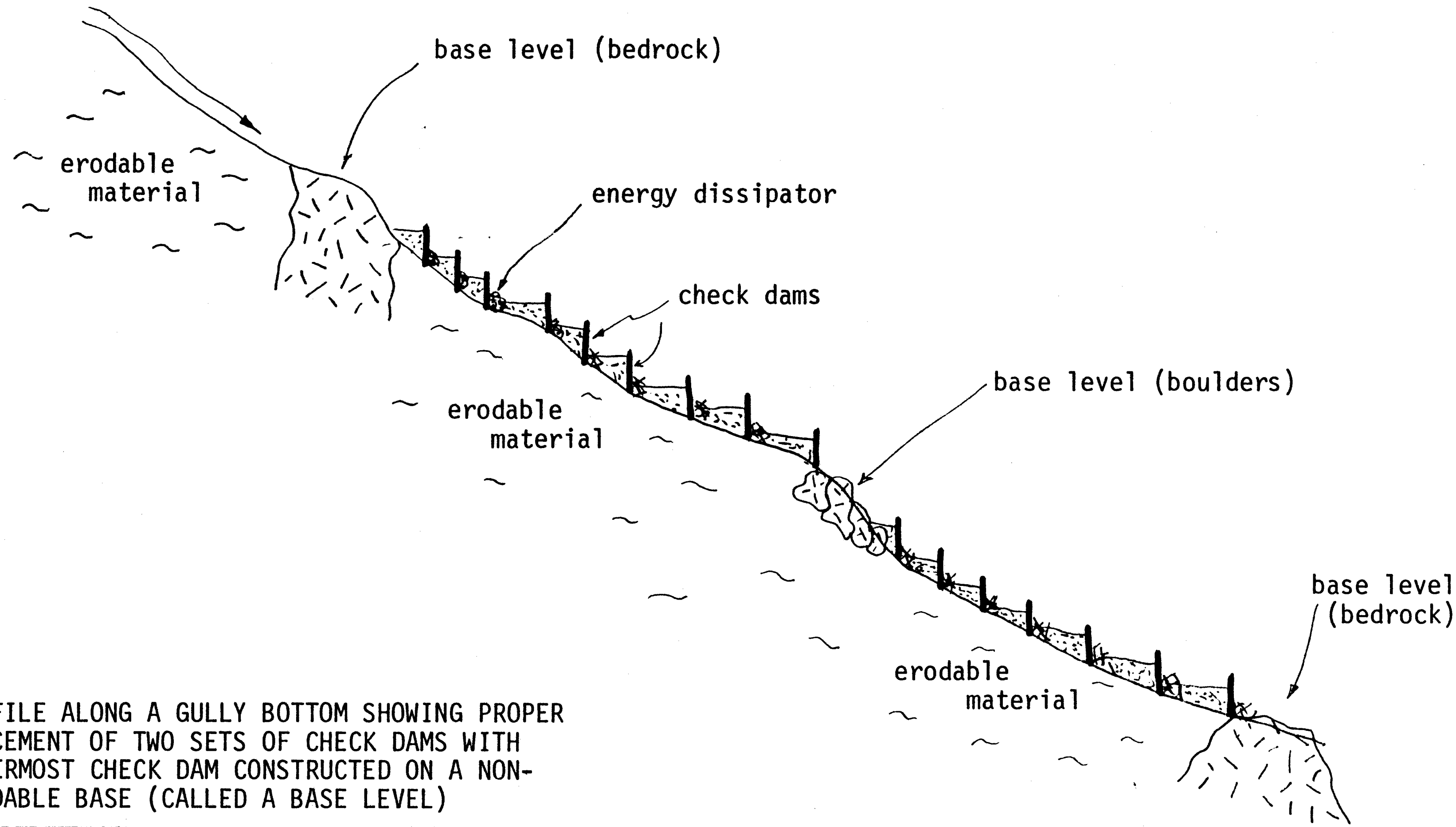
1. Composition of check dams.

Check dams can be constructed from on-site materials, such as split redwood or cedar boards from downed logs on the site or on nearby areas, conifer boughs, rock, or other suitable material specified in the S.O.W. The choice of material will be determined by availability of the material at or near the site and the suitability of the material for the particular gully or stream. Design criteria for check dams may only be altered with written approval from the Contracting Officer, or his/her representative. In all other cases, the listed specifications shall be adhered to.

2. Proper placement of check dams in a gully or stream channel. (see sketch)

All check dams shall be placed properly in a gully or stream channel, otherwise downcutting will continue and will undermine the dams. Check dams shall be installed as integrated units, each of which acts to stabilize neighboring dams. Check dams shall be aligned perpendicular to the channel. This will prevent concentrating flow at either bank.

Dam construction shall begin from the bottom of a gully or stream reach to be check dammed, and must begin at a "stable" point. Ideally, the lowermost check dam should be constructed on a non-erodable material such as bedrock, large boulders which the gully or stream cannot transport, or large logs partially buried in the gully or stream bottom. All check dams constructed upstream from the lowermost dam shall be placed so that the sediment fill behind the downstream dam (after it fills to the spillway level) abuts against the base of the upstream check dam (see sketch). To assure this condition is met,



PROFILE ALONG A GULLY BOTTOM SHOWING PROPER PLACEMENT OF TWO SETS OF CHECK DAMS WITH LOWERMOST CHECK DAM CONSTRUCTED ON A NON-ERODABLE BASE (CALLED A BASE LEVEL)

XVII.B. (Con't)

use a line level to place upstream dams. Stretch a level line from the spillway level until it contacts the channel bottom upstream (see sketch). This point of contact denotes the location of the next upstream check dam. Construct that check dam and continue this process up the gully or stream reach to be check dammed. Each check dam's spillway shall be constructed before the next upstream check dam is placed.

3. Split or milled board check dams for small gullies and streams.

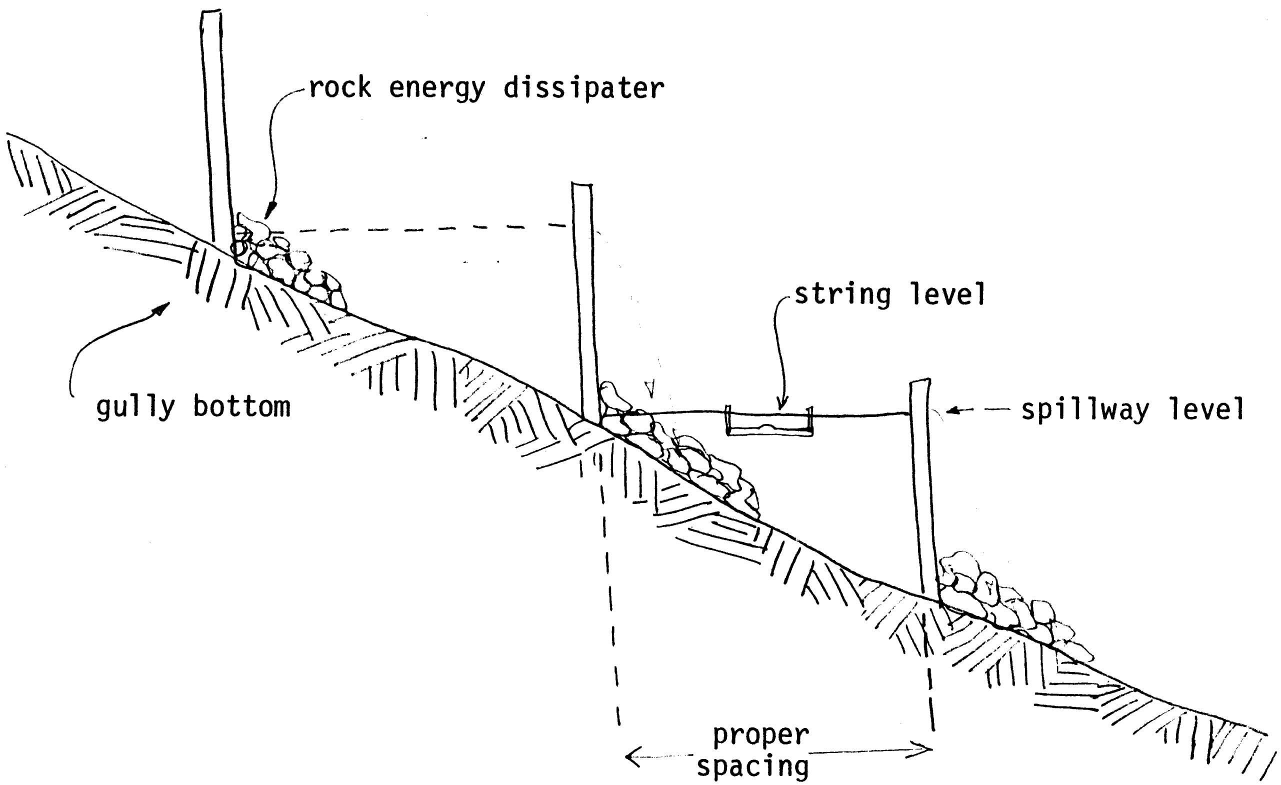
   a. Thickness and length of check dams. Check dams shall be constructed of redwood or cedar boards long enough to span the entire width of the gully or stream channel and shall be keyed into the banks (see h. below). Boards shall be 1 inch thick. However, if dams are from 6 to 10 feet in length, allowable thickness shall be at least 1 1/4 inches - 1 1/2 inches.

   b. Free-board height. Check dam free-board height is the vertical distance between the spillway level and the lowest point of the top of the check dam (see sketch). Free-board prevents high flows from cutting laterally into the channel banks and causing a check dam to fail. Free-board height shall be at least 8 inches.

   c. Effective height. The effective height is the height of a check dam which actively traps and stores sediment (see sketch) It is the distance between the channel bottom and the top of the spillway. Effective height shall be at least 8 inches and maximized whenever possible.

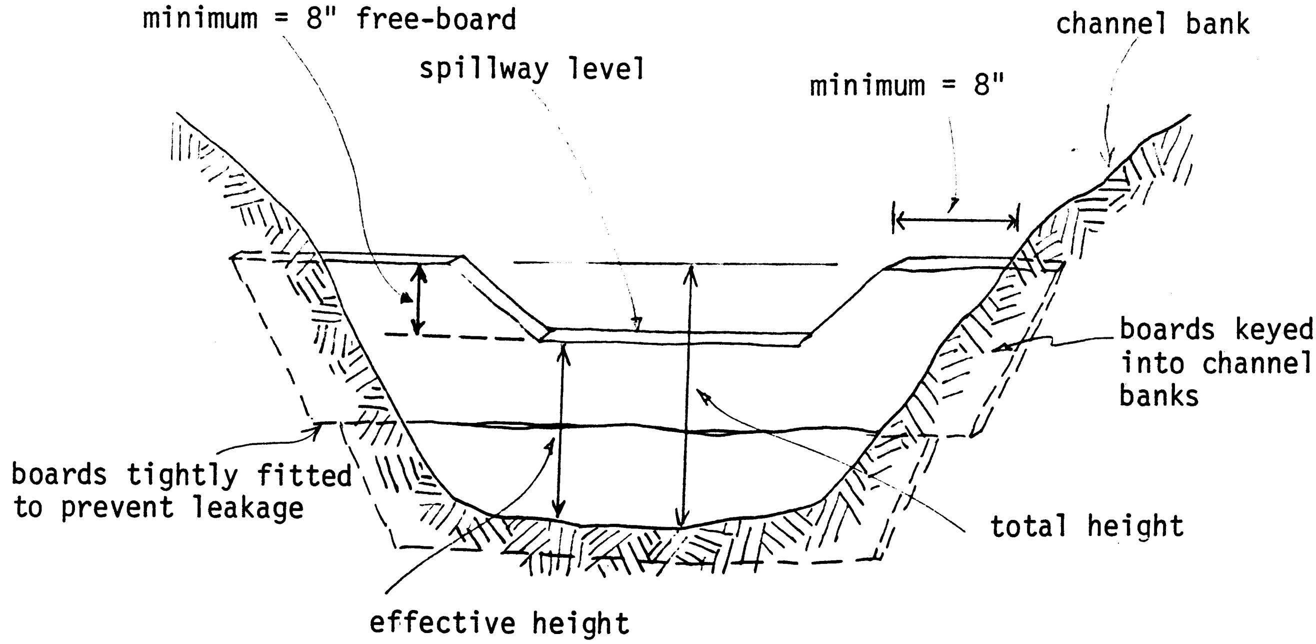
   d. Total height. Total check dam height is the sum of effective height and free-board height, and is dependent upon channel bank height. Generally, the higher the banks, the higher total check dam height can be. Maximum total check dam height shall be 40 inches.

   e. Multiple board check dams. Two boards may be used in order to attain maximum total check dam height. However, the widest board shall be placed on top and shall never be cut through entirely in order to construct a spillway.

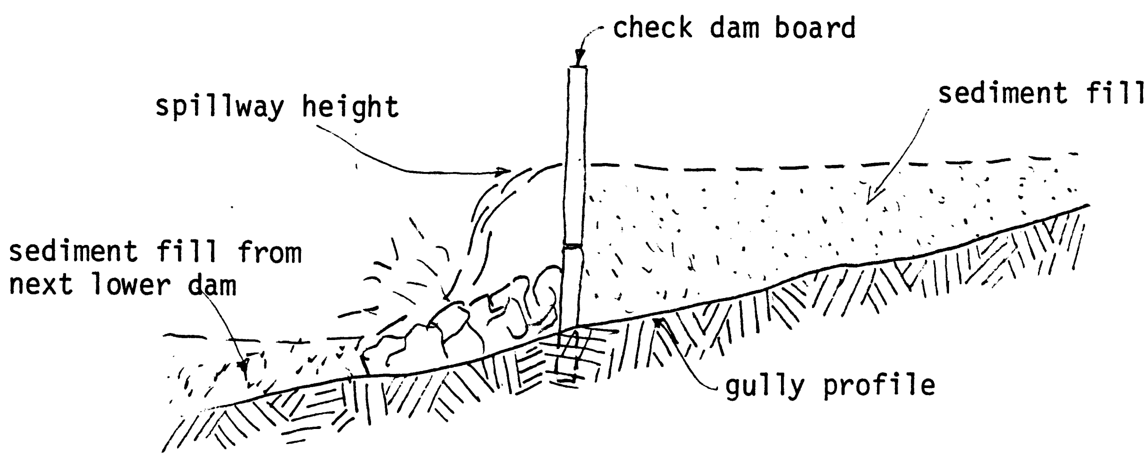


USE OF LINE LEVEL TO DETERMINE CONSERVATIVE DISTANCE BETWEEN CHECK DAMS IN A GULLY

FRONT VIEW (cross section)



SIDE VIEW (profile)



SCHEMATIC DRAWINGS OF BOARD CHECK DAMS

XVII.B.3. (Con't)

f. Check dam spillway. Board check dams must have adequate capacity spillways to accommodate high flows in the gully or stream channel. The S.O.W. shall specify the spillway area for check dams to be constructed in each channel reach.

g. Optimizing spillway design. Optimizing spillway design is important to the efficient placement and spacing of check dams in a channel. Spillway design shall proceed as follows (refer to sketch).

1) Based on channel configuration determine the maximum total check dam height.

2) Place check dam perpendicular to channel and secure to channel.

3) Measure an 8 inch free-board line onto dam (line C).

4) Measure at least 8 inches from both banks where the check dam board enters the channel bank (points "d").

5) Draw 45^{o} to 75^{o} side-walls from points "d" through line C.

6) Compute the spillway area.

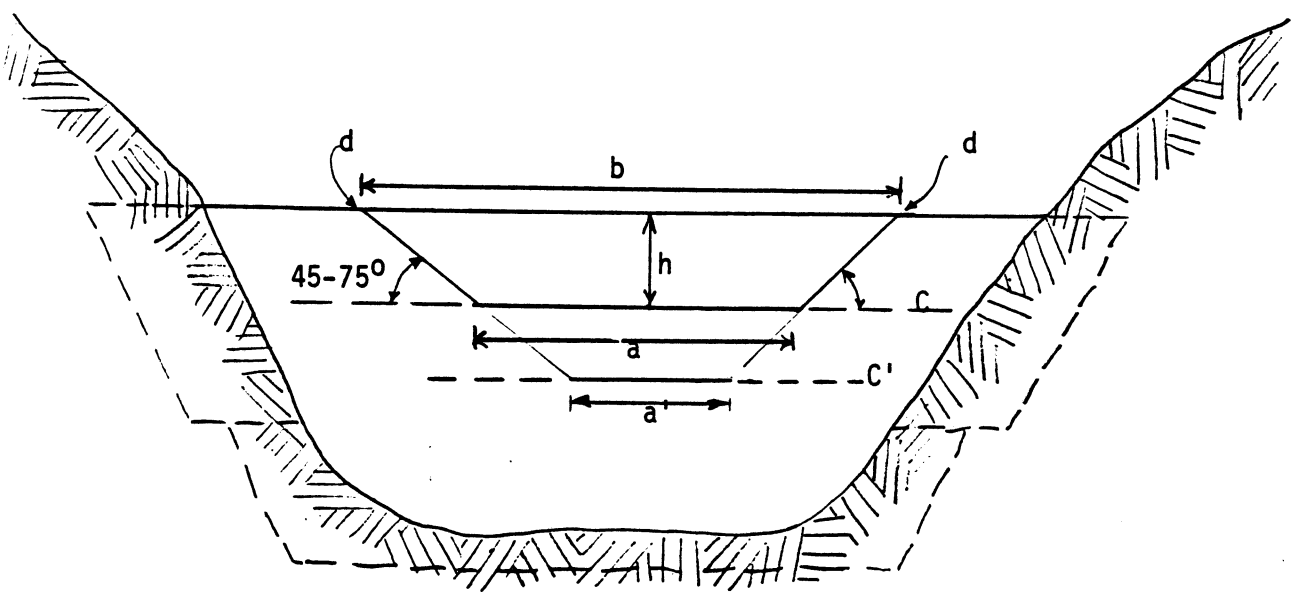
7) If spillway area is less than the specified area, increase the spillway side-wall angle to a maximum of 75^{o}.

8) Compute the spillway area.

9) If spillway area is still not adequate, lower the spillway level (line C) and vary side-wall angle to attain desired spillway area.

An important point to remember about spillway design is that a spillway should never be wide enough to allow flow to impact upon channel banks at the base of a check dam.

h. Excavation into channel banks. Boards shall be keyed into (inset into) channel banks to provide strength and prevent lateral breaching of the dam. Banks shall be neatly excavated (notched) only enough to inset the boards to a minimum depth of 6 inches. Excavate channel bottom to a minimum depth of 3 inches. The only exception shall be if channel



PROCEDURE FOR DEVELOPING SPILLWAY CAPACITY ON BOARD CHECK DAMS (see text)

XVII.B.3.h. (Con't)

bank excavation threatens to collapse the bank, or if the bank is composed of rock, or wood. If bank collapse is a problem, a compromise between enough excavation to prevent lateral breaching and a minimum amount of excavation to preserve the integrity of the bank shall be reached by on-site decisions with the Contracting Officer or his/her representative. Once a dam has been placed and inset into bank, clean fill material (i.e., fill containing no large rocks and/or woody debris) shall be packed into the channel bank where the dam is inset and along the upstream bottom of the dam. Clean fill must be used to seal the dam.

i. <u>Anchoring board check dams.</u> Check dams shall be securely anchored to the channel by either wood or metal rebar stakes. Both shall be driven at least 2 feet into the channel bottom and/or banks, and still have sufficient length to span at least 3/4 of the total check dam height. A minimum of 4 stakes shall be driven; two on each bank, with one against the upstream and one against the downstream side of the dam. Stakes shall contact the surface of the check dam and shall not interfere with flow through the spillway. When check dams exceed 6 feet in length, two additional stakes shall be driven against the downstream side of the dam evenly spaced across its length.

j. <u>Energy dissipation.</u> All board dams must have adequate energy dissipation devices installed in the channel bottom immediately below the spillway. The energy dissipation can consist of rock, conifer or hardwood boughs, small woody slash, split or milled boards or a combination of the above. Dissipators shall be: 1) firmly secured to the channel bottom, 2) located immediately below the spillway; and 3) as wide as the widest portion of the spillway notch. There should be no gap between the check dam boards and dissipators. Energy dissipators must extend continuously downstream at least 1 1/2 times the effective height of the check dam.

4. <u>Rock check dams for small channels.</u>

a. <u>Size of rock.</u> The largest rocks which can be transported manually and which are available from a nearby locality shall be used to build up the dams. Smaller rocks shall also be used in the rock dam so that as many large holes as possible are filled in to reduce porosity.

XVII.B.4. (Con't)

b. Rock dam height. Rock dams shall be between 12 and 36 inches high.

c. Spillway. Rock dams shall be built with an adequate spillway notch at least five inches deep and five inches wide. Most importantly, the height of the rock dam shall increase from the spillway toward the gully bank so that all flow is channeled through the spillway region. It is recognized that spillway notches will be highly irregular and variable because of varying rock sizes.

d. Excavation into gully banks and gully bottom. Side banks shall be excavated at least 4 inches unless the ground is too rocky, or unless excavation threatens to collapse the bank. Gully bottoms shall be excavated at least 3 inches. These specifications may be altered by the Contracting Officer or his/her representative on a site-specific basis.

e. Energy dissipation. The slope of the rock dam on the downstream side generally provides adequate energy dissipation below the spillway. The downstream side of the rock dam shall not be so steep as to allow the free fall of water from the spillway notch onto the gully bottom, i.e., the rock dam shall also serve as an energy dissipator.

f. Anchoring rock check dams in place with wire mesh. All rock check dams shall be anchored securely in place using corrosion-resistant wire mesh. The wire mesh shall cover the rock dam, be fastened together with baling wire, and be secured to the gully bottom and side with wooden stakes or metal rebar. The entire rock dam, including the base, may also be enclosed in wire mesh, thereby forming an irregular shaped gabion. The wire mesh shall in all cases be securely anchored to the banks.

4. Bough dams for small channels.

a. Utilization of bough dams. Bough dams can be an effective type of check dam in certain localities. No specifications for bough dams are given; however, the use of bough dams in a gully or stream reach shall be discussed with, and approved by, the Contracting Officer in writing prior to any bough dam installation.

XVII.B.4. (Con't)

b. Anchoring bough dams to gully. Because bough dams totally lose their leaves as quickly as 4 months after being cut and installed in gullies, it is important that the boughs be bound tightly together and staked firmly in the ground so that the bough dam does not become loose. Rocky gullies that do not allow adequate staking are generally unsuited for bough dams.

C. Comments.

The above specifications are applicable to channel stabilization measures for small coastal streams (maximum drainage area of about 50 to 100 acres) in Northern California. A multitude of check dam construction techniques have been developed for areas in the Western U.S., and elsewhere. The following references provide a good starting point for matching your particular situation with the proper type of channel protection measure.

1. Heede, Burchard H., 1966, Design, Construction and Cost of Rock Check Dams, U.S.D.A. Forest Service, Research Paper RM-20, Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colorado.

2. Heede, Burchard H., 1965, Multipurpose Prefabricated Concrete Check Dam, U.S.D.A. Forest Service, Research Paper RM-12, Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colorado.

3. Heede, Burchard H., 1968, Conversion of Gullies to Vegetation Lined Waterways, U.S.D.A. Forest Service, Research Paper RM-40, Rocky Mountain Forest and Range Experiment Station, Fort Collins, Colorado.

(Dr. Heede is now located at the Forest Sciences Laboratory, Arizona State University, Tempe, AZ.)

4. U.S.D.A. Forest Service, 1974, Forest Service Handbook, FSH2509:12-Watershed Structural Measures Handbook, Amendments 1-3, July 1969, 103 pages.

5. High Sierra Resource Conservation and Development Council, 1981, Erosion and Sediment Control Guidelines for Developing Areas of the Sierras, California Water Resources Control Board - Central Valley Region, 170 pages.

XVIII. SUBMERGED SPILLWAYS

A. Definition of job.

A submerged spillway is nothing more than a submerged check dam placed with the spillway at streambed level. Like check dams, they can stabilize the adjacent channel bank by preventing downcutting and lateral cutting if runoff is directed through the spillway, and the submerged spillway is not undermined by channel downcutting from below the structure. Submerged spillway construction is most applicable in broad channels with shallow, poorly defined channel banks and rock bottoms.

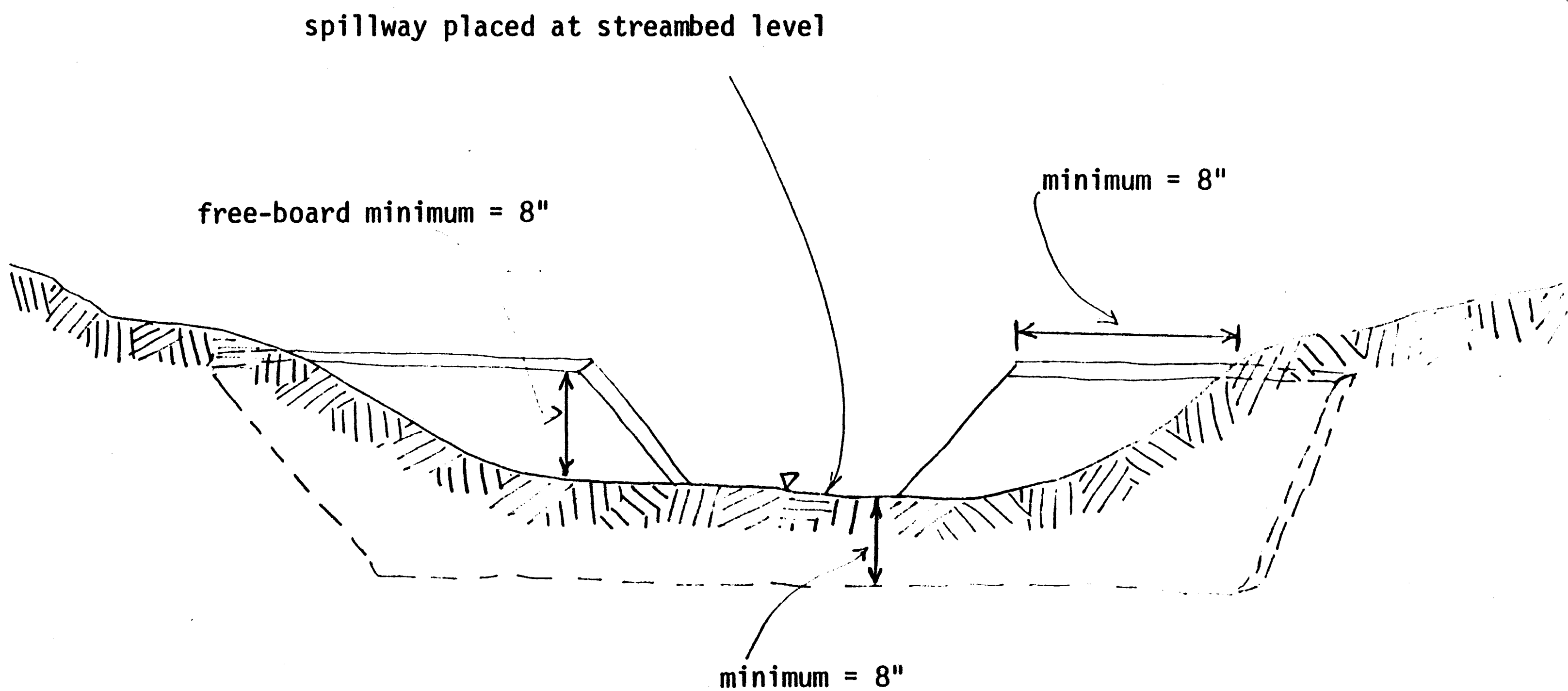
B. Specifications.

1. Thickness and length of submerged spillways. Submerged spillways are to be constructed from redwood or cedar boards and shall be keyed into adjacent banks (see B.6.). Board thickness shall never be less than 1 inch and shall be at least 1 1/4 inches thick if submerged spillways are greater than 6 feet long.

2. Free-board height. (see sketch) Free-board height is the vertical distance between the spillway level and the lowest point of the top of the submerged spillway. Free-board height shall be at least 8 inches.

3. Total height. (see sketch) The total height of a submerged spillway is the sum of the free-board height and that portion of the structure which is keyed (buried) into the channel bottom. Total height shall never be less than 14 inches; i.e., 6 inches of board surface keyed into the channel below the spillway level, plus 8 inches of free-board.

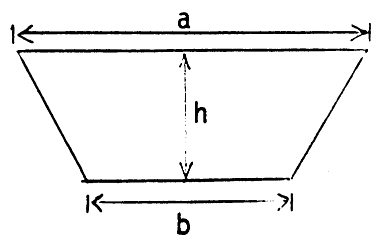
4. Spillway area. The S.O.W. shall specify the spillway area for submerged spillways to be constructed within a particular reach. A spillway can be cut into the board prior to installing the submerged spillway into the channel.



SCHEMATIC DRAWING OF SUBMERGED SPILLWAY

XVIII. B. (Con't)

5. Spillway design. Construct the spillway in a form of a traperzoid. The formula to compute area of a trapezoid is: $A = 1/2\ (a+b)\ h$



6. Excavation into channel banks and bottom. Boards shall be keyed (inset) into channel banks to provide strength and prevent lateral breaching of the submerged spillway. Banks shall be neatly excavated (notched) only enough to key the boards at least 8 inches into the channel banks. It will also be necessary to excavate the channel bottom 6 inches deep to receive the submerged spillway. Once a spillway has been placed into the channel, clean fill material (i.e., no large rocks or organics) shall be packed into the channel bank and bottom where the spillway is inset to create a seal.

7. Anchoring submerged spillways. Submerged spillways shall be securely anchored to the channel by either wooden stakes (1 1/2 inches diameter) or metal rebar. Stakes shall be driven at least 2 feet deep into channel bank and/or bottom, and still have sufficient length to span the free-board height. A minimum of 4 stakes shall be driven: 2 on each bank with 1 against the upstream and 1 against the downstream side of the spillway. When submerged spillways exceed 6 feet in length, 2 additional stakes shall be driven against the downstream side of the spillway spaced evenly across its length. Stakes shall not extend into the spillway area.

8. Submerged spillway placement. Submerged spillways are always installed with the spillway at streambed level, and perpendicular to the channel. No energy dissipation is required downstream from the spillway. The S.O.W. shall specify the distance between each submerged spill-way. Begin at the bottom of the channel to be treated. Excavate channel banks and bottom to receive the lower-most spillway, and stake into place. Measure channel distance to next submerged spillway as specified in S.O.W., and install the next structure. Channel areas between structures can be rock amored for added protection.

XVIII (Con'd)

C. Comments.

Submerged spillways described here have only been tested on very small streams (drainage area = 10-20 acres). They essentially act to control local base levels and keep streamflow near the center of the channel (away from the banks).

XIX. WATER LADDERS

A. Definition of job.

Water ladders are wooden structures, similar in appearance to ladders, which serve to convey water across a steep slope while preventing channel downcutting. They serve the same purpose as half-round culverts that conduct ditched or culverted water over steep road fills onto vegetated and/or slash-covered slopes. Essentially, water ladders are energy dissipation devices that can effectively handle concentrated runoff, and which work well in conjunction with strategically placed slash and planting of stem cuttings.

Water ladders can be used in combination with check dams or at the downstream end of cross road drains. Alternatively, water ladders may be used in lieu of check dams where dam installation is difficult because of unstable banks or difficult excavation of channel beds.

B. Specifications.

1. <u>Construction of Ladder.</u> Ladder construction will be left up to the discretion of the contractor, but the following criteria must be met in construction:

    a. Each ladder must be large enough to carry design storm-flows. Each ladder must be at least 18 inches wide unless it is to be placed in a well-defined gully which is less than 18 inches in width. In all cases, the ladder must be as wide as the bottom of the ditch or drainage channel directly above the ladder.

    b. Ladder treads must overlap and dip slightly downhill once the ladder is installed. Grooves (about one-half inch deep) may be cut into the top side of the treads to help direct flow towards the center. Bevelling of the leading edge of the treads and nailing of slats under the treads may also be used to help prevent backflow under the treads (see illustrations).

    c. Where necessary, wing walls should be installed at the top of a ladder to insure that all runoff is directed into the ladder. This may be especially important on wide or poorly defined channels.

    d. Outlet areas below ladders should be defended with adequate energy dissipation measures (rocks, slash, etc.).

XIX.B. (Con't)

2. Placement of ladder. Ladders must be sufficiently inset into the slope so that runoff will course over the ladder treads and not run under or around the ladder. Adequate excavation and especially careful placement of the top of the ladder relative to the ditch or drain are crucial. Improperly placed ladders that do not successfully convey runoff over them (during the first winter season) must be re-installed on request of the Contracting Officer.

3. Type of ladder. Type and composition of water ladders will largely depend on availability of materials and equipment at the site. Boards of rot resistant wood cut on site with an Alaska mill or similar equipment are preferable. Hand split and hand sawed boards can be used but they may pose problems because of their uneven surfaces. These permit water to leak through cracks thus causing possible undercutting of the structure. Dry wood shims should be hammered into all cracks and seams to seal them off. The following are examples of water ladders that can be constructed:

   Type 1 - Hand split wood ladder with log supports.

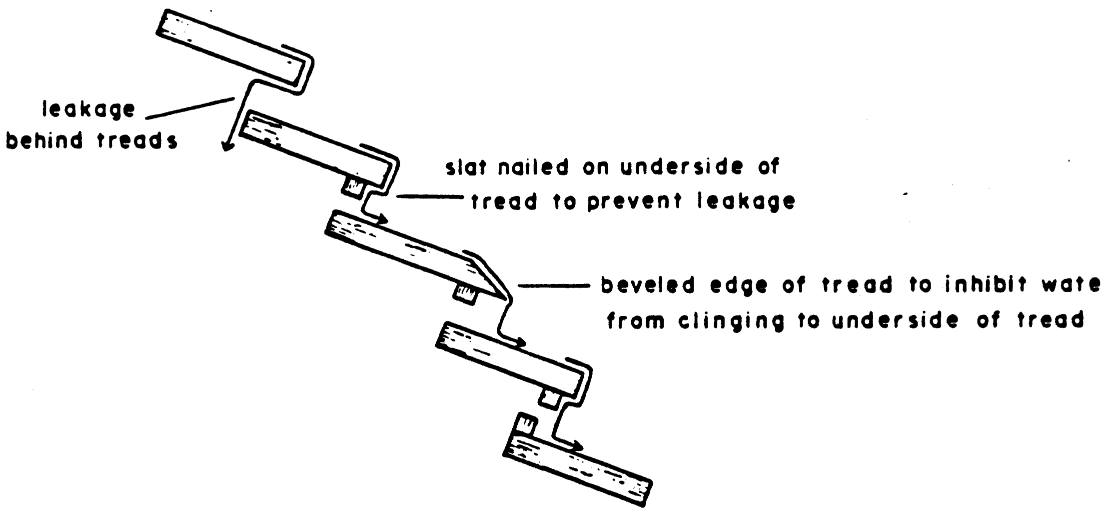
   Ladder is built in two overlapping sections, conforming to channel gradient. Top tread of ladder is keyed into a partially buried log (see illustrations).

   Type 2 - Milled wood ladder.

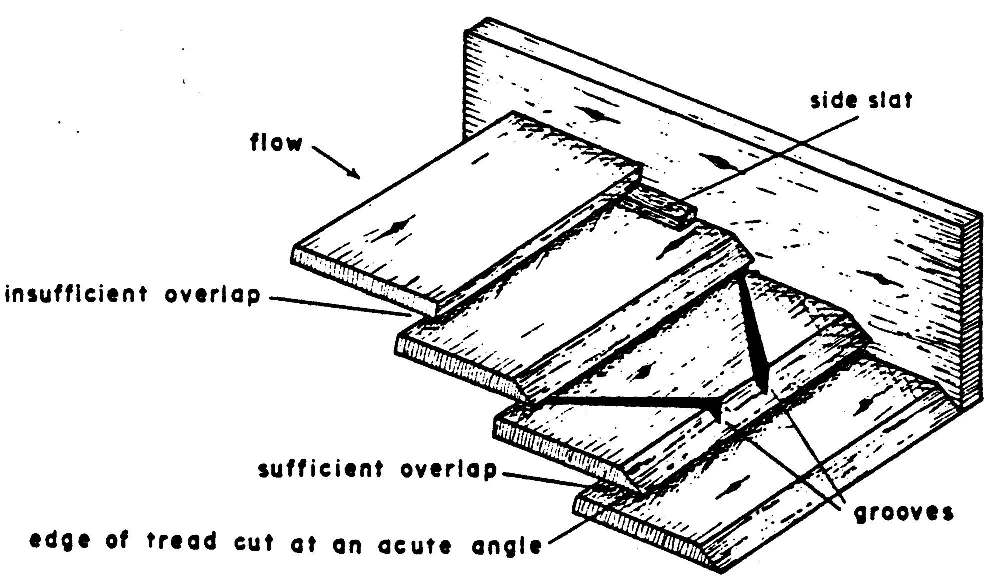
   Ladder is constructed from milled slabs and boards. Treads are supported on stair-stepped slats (see illustration).

C. Comments.

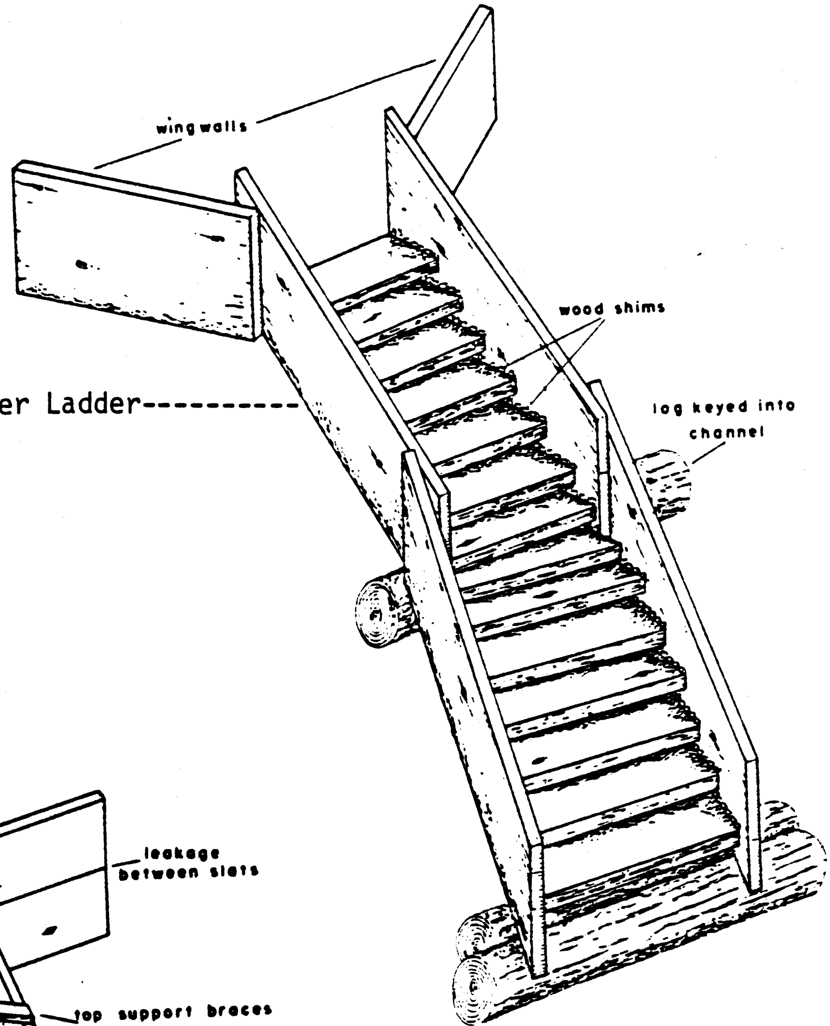
By nature, water ladders are relatively expensive to construct. The construction of structures that would be large enough to contain peak flows of streams with even a moderately high discharge (over 5 cfs) may also be physically and logistically impossible in many remote locations. If access for heavy equipment is available, it may be more cost-effective to excavate a channel which can then be protected with rock armor or check dams. All-in-all, over the life span of the structure, water ladders can very successfully prevent soil erosion. Their use is justified in remote, sensitive areas.



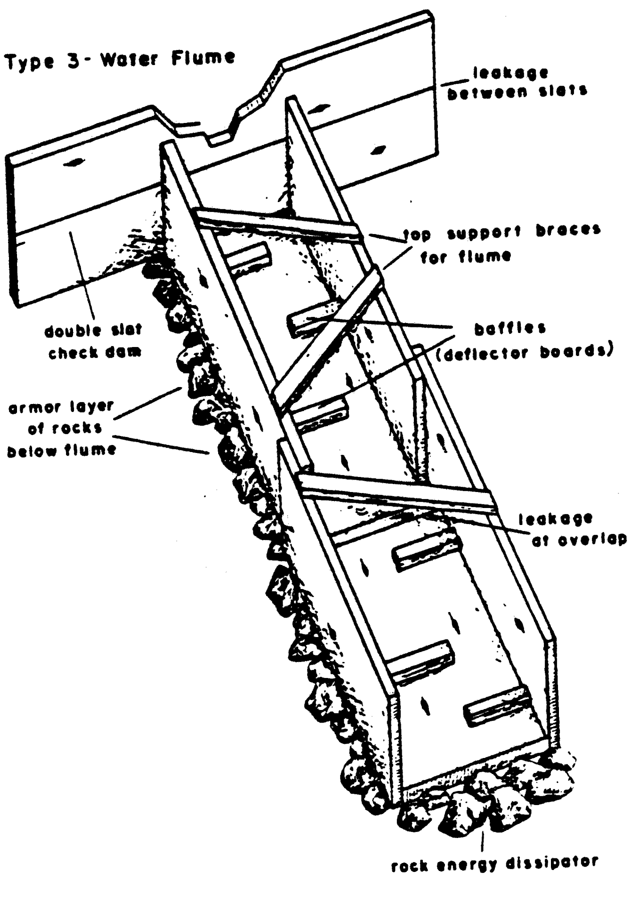
Side View of Water Ladder Treads



Plan View of Water Ladder Treads

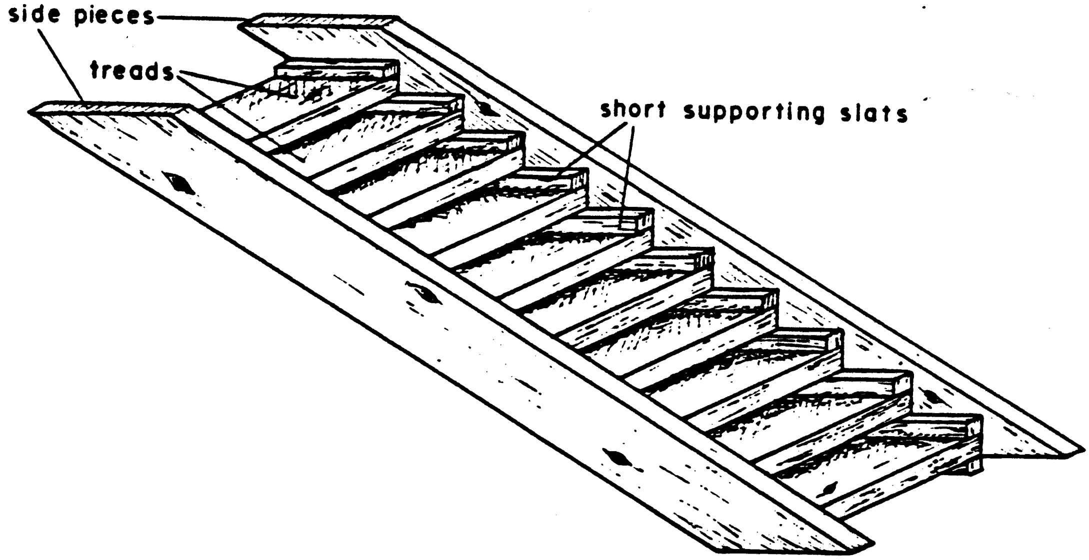


Type 2 Water Ladder



Type 3- Water Flume

Type 4 - Water Ladder



BIBLIOGRAPHY OF REDWOOD NATIONAL PARK PUBLICATIONS

September 1983

Redwood National Park Technical Report Series

1. Madej, M. A., H. Kelsey, and W. Weaver. 1980. An Evaluation of 1978 Rehabilitation Sites and Erosion Control Techniques in Redwood National Park. Redwood National Park Technical Report Number 1. National Park Service, Redwood National Park. Arcata, California. 113 pp.

2. Kelsey, H. and P. Stroud. 1981. Watershed Rehabilitation in the Airstrip Creek Basin. Redwood National Park Technical Report Number 2. National Park Service, Redwood National Park. Arcata, California. 45 pp.

3. Kelsey, H., M. A. Madej, J. Pitlick, M. Coghlan, D. Best, R. Belding and P. Stroud. 1981. Sediment Sources and Sediment Transport in the Redwood Creek Basin: A Progress Report. Redwood National Park Technical Report Number 3. National Park Service, Redwood National Park. Arcata, California. 114 pp.

4. Sacklin, John A. 1982. Wolf Creek Compost Facility, Operation and Maintenance Manual. Redwood National Park Technical Report Number 4. Second Edition. National Park Service, Redwood National Park. Arcata, California. 61 pp.

5. Reed, Lois J. and M. A. Hektner. 1981. Evaluation of 1978 Revegetation Techniques. Redwood National Park Technical Report Number 5. National Park Service, Redwood National Park. Arcata, California. 70 pp.

6. Muldavin, Esteban H., J. M. Lenihan, W. S. Lennox and S. D. Veirs, Jr. 1981. Vegetation Succession in the First Ten Years Following Logging of Coast Redwood Forests. Redwood National Park Technical Report Number 6. National Park Service, Redwood National Park. Arcata, California. 69 pp.

7. Lenihan, James M., W. S. Lennox, E. H. Muldavin, and S. D. Veirs, Jr. 1982. A Handbook for Classifying Early Post-Logging Vegetation in the Lower Redwood Creek Basin. Redwood National Park Technical Report Number 7. National Park Service, Redwood National Park. Arcata, California. 40 pp.

8. Pitlick, John. 1982. Sediment Routing in Tributaries of the Redwood Creek Basin: Northwestern California. Redwood National Park Technical Report Number 8. Naitonal Park Service, Redwood National Park. Arcata, California. 67 pp.

Technical Reports in Progress (1983 - 1984 Publishing Date)

1. Walter, Tom. Gully Erosion on Prairies of the Redwood Creek Basin, Northwestern California.

2. Varnum, Nicholas. Significance of Channel Changes at Cross Sections in the Mainstem of Redwood Creek, California, During the 1982 Water Year.

3. Madej, Mary Ann. Recent Changes in Channel-Stored Sediment in Redwood Creek, California.

4. Best, David. Recent Land Use History in Redwood Creek, California.

5. Best, David. Contribution of Fluvial Erosion to the Sediment Load of Redwood Creek, California.

6. Kelsey, H. M., M. Coghlan, J. Pitlick, and D. Best. Geomorphic Analysis of Streamside Landsliding in the Redwood Creek Basin.

7. Best, D. W., H. M. Kelsey, D. K. Hagans, and M. Alpert. Role of Fluvial Hillslope Erosion and Road Construction in the Sediment Budget of Garrett Creek, Humboldt County, California.

Selected Publications on Watershed Rehabilitation by Redwood National Park Staff

1. Bundros, Gregory J., T. Spreiter, K. Utley and E. Wosika. 1981. Erosion Control in Redwood National Park, 1980. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 273-282.

2. Coghlan, M. and M. A. Madej. 1981. Main Channel Response to Increased Sediment Supply, Upper Redwood Creek, California. Transactions American Geological Union, Abstracts, Vol. 62, Number 45, p. 858.

3. Hagans, D. K., W. E. Weaver and M. Alpert. (In preparation) Land Use as an Independent Variable Affecting Fluvial Erosion in the Redwood Creek Basin, Northern California. Abstract in: Proceedings, Symposium on Effects of Forest Land Use on Erosion and Slope Stability, May 7-11, 1984, Honolulu, Hawaii. Union of Forestry Research Organizations. Published by the Forest Research Institute, New Zealand Forest Service, Christchurch, New Zealand.

Selected Publications on Watershed Rehabilitation (Continued)

4. Hektner, Mary, L. Reed, J. H. Popenoe, R. J. Mastrogiuseppe, D. Vezie, N. G. Sugihara and S. D. Veirs, Jr. Review of Revegetation Treatments Used in Redwood National Park: 1977 to Present. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 70-77.

5. Hofstra, Terry. 1981. Aquatic Resources Rehabilitation Program, Redwood National Park. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 56-58.

6. Kelsey, H. M. and W. Weaver. 1979. Watershed Rehabilitation for Erosion Control on Logged Lands in Redwood National Park. Guidebook for Geological Society of America Field Trip, April 12-14, 1979: A Field Trip to Observe Natural and Management-Related Erosion in Franciscan Terrain of Northern California. pp. XII-1 to XII-14.

7. Kelsey, H. M., W. Weaver and M. A. Madej. 1979. Geology, Geomorphic Processes, Land Use and Watershed Rehabilitation in Redwood National Park and Vicinity, Lower Redwood Creek Basin. Guidebook for Geological Society of America Field Trip, April 12-14, 1979: A Field Trip to Observe Natural and Management-Related Erosion in Franciscan Terrain of Northern California. pp. XIII-1 to XIII-18.

8. Kelsey, H. M., W. E. Weaver and G. Bundros. 1979. An Evaluation of Erosion Control Devices Used in Gullies Within Redwood National Park. Geological Society of America, Abstracts With Programs, Vol. 11, Number 3. February, 1979.

9. Kelsey, H., M. A. Madej, J. Pitlick, P. Stroud, and M. Coghlan. 1981. Major Sediment Sources and Limits to the Effectiveness of Erosion Control Techniques in the Highly Erosive Watersheds of North Coastal California. In: Proceedings of a Symposium on Erosion and Sediment Transport in Pacific Rim Steeplands. January 25-31, 1981. Christchurch, New Zealand. IAHS-AISH Publication Number 132. International Association of Hydrological Sciences. Washington, D. C. pp. 493-510.

10. Klein, R. D. (In preparation) Channel Adjustments Following Logging Road Removal in Small Steepland Drainages. Abstract in: Proceedings, Symposium on Effects of Forest Land Use on Erosion and Slope Stability, May 7-11, 1984, Honolulu, Hawaii. Union of Forestry Research Organizations. Published by the Forest Research Institute, New Zealand Forest Service, Christchurch, New Zealand.

Selected Publications on Watershed Rehabilitation (Continued)

11. LaHusen, R. G. (In preparation) Characteristics of Management Related Debris Flows, Northwestern California. Abstract in: Proceedings, Symposium on Effects of Forest Land Use on Erosion and Slope Stability, May 7-11, 1984, Honolulu, Hawaii. Union of Forestry Research Organizations. Published by the Forest Research Institute, New Zealand Forest Service, Christchurch, New Zealand.

12. Larson, James P., C. L. Ricks, and T. J. Salamunovich. 1981. Alternatives for Restoration of Estuarine Habitat at the Mouth of Redwood Creek, Humboldt County, California. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 236-245.

13. Lenihan, James M., W. S. Lennox, E. H. Muldavin and S. D. Veirs, Jr. 1981. Redwood Forests in Their Initial Stages of Secondary Succession Following Logging and the Application to Forest Rehabilitation. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 56-68.

14. Madej, M. A. and H. Kelsey. 1981. Sediment Routing in Stream Channels: Its Implications for Watershed Rehabilitation. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 17-25.

15. Pitlick, John. 1981. Organic Debris in Tributary Stream Channels of the Redwood Creek Basin. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studeis of the John Muir Institute. Berkeley, California. pp. 177-190.

16. Pitlick, John. 1981. Sediment Routing in Tributaries of the Redwood Creek Basin, Northern California. Transactions, American Geophysical Union, Abstracts. Vol. 62, Number 45, p. 858.

17. Popenoe, James H. 1981. Effects of Grass Seedings, Fertilizer and Mulches on Vegetation and Soils of the Copper Creek Watershed Rehabilitation Unit: The First Two Years. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 87-95.

Selected Publications on Watershed Rehabilitation (Continued)

18. Sonnevil, R. A., and W. E. Weaver. 1981. The Evolution of Approaches and Techniques to Control Erosion on Logged Lands, Redwood National Park, 1977-1981. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 258-272.

19. Teti, Patrick. 1981. Rehabilitation of a 290 Hectare Site in Redwood National Park, 1980. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 283-297.

20. United States Department of the Interior. 1978. Proceedings of a Workshop on Techniques of Rehabilitation and Erosion Control in Recently Roaded and Logged Watersheds, With Emphasis to North Coastal California. March 13-14, 1977. National Park Service, Redwood National Park, Resources Management Division. Arcata, California. 89 pp.

21. Veirs, Stephen D., Jr., and W. Lennox. 1981. Rehabilitation and Long-Term Park Managment of Cutover Redwood Forests: Problems of Natural Succession. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 50-55.

22. Weaver, W. E., H. M. Kelsey and M. A. Madej. 1979. General History of Redwood National Park: Guidebook for Geological Society of America Field Trip, April 12-14, 1979: A Field Trip to Observe Natural and Management-Related Erosion in Franciscan Terrain of Northern California. pp. IX-1 to IX-3.

23. Weaver, W. E., H. M. Kelsey, M. A. Madej, D. Hagans and G. Bundros. 1979. Minimizing Concentrated Runoff, Surface Erosion, and Mass Slope Movement in Redwood National Park by Removing Former Logging Haul Roads. Geological Society of America. Abstracts with Programs, Vol. 11, Number 3, February, 1979.

24. Weaver, W. and M. Seltenrich. 1980. Summary Results Concerning the Effectiveness and Cost-Effectiveness of Labor-Intensive Erosion Control Practices Used in Redwood National Park, 1978-1979. Unpublished memorandum report, on file, Redwood National Park. 20 pp.

Selected Publications on Watershed Rehabilitation (Continued)

25. Weaver, W. and M. A. Madej. 1981. Erosion Control Techniques Used in Redwood National Park, Northern California, 1978-1979. In: Proceedings of a Symposium on Erosion and Sediment Transport in Pacific Rim Steeplands, January 25-31, 1981. Christchurch, New Zealand. IAHS-AISH Publication Number 132. International Association of Hydrological Sciences. Washington, D. C. pp. 640-645.

26. Weaver, W. E., A. V. Choquette, D. K. Hagans and J. Schlosser. 1981. The Effects of Intensive Forest Land Use and Subsequent Landscape Rehabilitation on Erosion Rates and Sediment Yield in the Copper Creek Drainage Basin, Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 298-312.

27. Weaver, W. E., M. S. Seltenrich, R. A. Sonnevil, and E. M. Babcock. 1981. The Use of Cost-Effectiveness as a Technique to Evaluate and Improve Watershed Rehabilitation for Erosion Control, Redwood National Park. Proceedings, Symposium on Watershed Rehabilitation in Redwood National Park and Other Coastal Areas. August 24-28, 1981. Arcata, California. Center for Natural Resource Studies of the John Muir Institute. Berkeley, California. pp. 341-360.

28. Weaver, W. E. and R. A. Sonnevil. (In preparation) Relative Cost-Effectiveness of Forest Land Rehabilitation, Redwood National Park, Northern California. Abstract in: Proceedings, Symposium on Effects of Forest Land Use on Erosion and Slope Stability, May 7-11, 1984, Honolulu, Hawaii. Union o fForestry Research Organizations. Published by the Forest Research Institute, New Zealand Forest Service, Christchurch, New Zealand.

Abstracts and Papers Submitted By Redwood National Park Employees for a Conference on Research in California's National Parks September 9-10, 1982

Geology/Hydrology

1. STATUS OF THE EMERALD CREEK LANDSLIDE, REDWOOD NATIONAL PARK
   E. M. Babcock, R. G. LaHusen, R. D. Klein, R. A. Sonnevil, W. E. Weaver, D. K. Hagans

Geology/Hydrology (Continued)

2. COMPARISON OF SLOPE TREATMENTS FOR REDUCING SURFACE EROSION ON DISTURBED SITES AT REDWOOD NATIONAL PARK
   K. J. Kveton, K. A. Considine, E. M. Babcock, R. G. LaHusen, M. S. Seltenrich, R. A. Sonnevil, W. E. Weaver

3. CONSTRUCTION OF A SEDIMENT BUDGET FOR REDWOOD CREEK WATERSHED, NORTHERN CALIFORNIA
   M. A. Madej, J. P. Pitlick, D. W. Best

4. AMOUNTS OF SEDIMENT DERIVED FROM SURFACE EROSION AND CHANNEL ADJUSTMENTS ON EXCAVATED STREAM CROSSINGS AT REDWOOD NATIONAL PARK
   R. A. Sonevil, E. M. Babcock, R. D. Klein, R. G. LaHusen, W. E. Weaver, M. S. Seltenrich, K. A. Considine, K. J. Kveton

Vegetation

5. THE BALD HILLS PRAIRIES OF REDWOOD NATIONAL PARK, CALIFORNIA
   M. M. Hektner, R. W. Martin, D. R. Davenport

6. THE FOREST ASSOCIATIONS OF THE LITTLE LOST MAN CREEK RESEARCH NATURAL AREA, REDWOOD NATIONAL PARK
   J. M. Lenihan

7. GUIDELINES FOR CLASSIFYING EARLY, POST-LOGGING VEGETATION IN THE LOWER REDWOOD CREEK BASIN OF REDWOOD NATIONAL PARK
   J. M. Lenihan, W. S. Lennox, E. H. Muldavin, S. D. Veirs, Jr.

8. STAND COMPOSITION AND DIAMETER DISTRIBUTION IN SIXTY-YEAR-OLD SECOND-GROWTH COAST REDWOOD FORESTS
   W. S. Lennox

9. ARTIFICIAL AND BIOLOGICAL CONTROL OF TANSY RAGWORT, *Senecio jacobaea L.,* IN REDWOOD NATIONAL PARK, CALIFORNIA
   R. J. Mastrogiuseppe, N. T. Blair, B. C. Griffith

10. VEGETATION SUCCESSION IN THE FIRST TEN YEARS FOLLOWING LOGGING OF COAST REDWOOD FORESTS
    E. H. Muldavin, J. M. Lenihan, W. S. Lennox, S. D. Veirs, Jr.

11. *Whipplea modesta Torr.:* PROMISING NATIVE FOR EROSION CONTROL IN THE REDWOOD REGION
    J. Popenoe, L. Reed, R. Martin

12. EFFECTS OF SEED, FERTILIZER AND MULCH APPLICATION ON VEGETATION RE-ESTABLISHMENT ON REDWOOD NATIONAL PARK REHABILITATION UNITS
    L. J. Reed, M. M. Hektner

Vegetation (Continued)

13. THE ROLE OF SYMBIOTIC MICRO-ORGANISMS IN THE POST-DISTURBANCE ECOSYSTEMS OF REDWOOD NATIONAL PARK
    N. G. Sugihara

14. OREGON WHITE OAK WOODLANDS OF REDWOOD NATIONAL PARK: DESCRIPTION AND MANAGEMENT CONSIDERATIONS
    N. G. Sugihara, M. M. Hektner, L. J. Reed, J. M. Lenihan

15. STAND DYNAMICS IN OLD-GROWTH REDWOOD FOREST VEGETATION
    S. D. Veirs, Jr.

16. FOREST-PRAIRIE VEGETATION DYNAMICS AND THE ROLE OF MAN: GANN'S PRAIRIE, REDWOOD NATIONAL PARK
    S. D. Veirs, Jr.

Aquatics/Wildlife

17. ANADROMOUS SALMONID NURSERY HABITAT IN THE REDWOOD CREEK WATERSHED
    D. G. Anderson, R. A. Brown

18. AN EVALUATION OF TECHNIQUES FOR COLLECTION AND ANALYSIS OF BENTHIC INVERTEBRATE COMMUNITIES IN SECOND-ORDER STREAMS IN REDWOOD NATIONAL PARK
    J. M. Harrington

19. AQUATIC RESOURCES REHABILITATION, REDWOOD NATIONAL PARK
    T. D. Hofstra, J. M. Harrington

20. BLACK BEAR RESEARCH, REDWOOD NATIONAL PARK
    M. T. Schroeder, T. D. Hofstra

21. SUMMER "COLD POOLS" IN REDWOOD CREEK NEAR ORICK, CALIFORNIA AND THEIR IMPORTANCE AS HABITAT FOR ANADROMOUS SALMONIDS
    E. Keller, T. D. Hofstra

22. WATER QUALITY AND PRODUCTIVITY OF THE REDWOOD CREEK ESTUARY
    J. Larson, J. McKeon, T. Salamunovich, T. D. Hofstra

23. REDWOOD CREEK ESTUARY: FLOOD HISTORY, SEDIMENTATION AND IMPLICATIONS FOR AQUATIC HABITAT
    C. Ricks

24. DETERMINING THE ECONOMIC VALUE OF AQUATIC RESOURCES WITHIN THE IMPACT AREA OF PROPOSED HIGHWAY CONSTRUCTION
    R. Wood, California Department of Fish and Game, Eureka, California
    T. Hofstra, Redwood National Park, Arcata, California
    D. McLeod, California Department of Fish and Game, Eureka, California

Papers Now Being Prepared by National Park Service Staff for Inclusion in U.S. Geological Survey Professional Paper on Research in Redwood National Park

1. SUMMARY OF SCIENTIFIC INVOLVEMENT IN THE REDWOOD CREEK BASIN
   S. Veirs, Jr. and R. J. Janda

2. GEOLOGY AND DESCRIPTIVE GEOMORPHOLOGY OF THE REDWOOD CREEK BASIN
   H. M. Kelsey and D. R. Harden

3. A CLIMATOLOGICAL ANALYSIS OF THE RESPONSE OF REDWOOD CREEK TO EXTREME STORMS
   M. Coghlan

4. RECENT LAND USE HISTORY IN THE REDWOOD CREEK BASIN
   D. Best

5. MAGNITUDE AND CAUSES OF GULLY EROSION IN THE LOWER REDWOOD CREEK DRAINAGE BASIN
   W. E. Weaver and D. K. Hagans

6. ROLE OF FLUVIAL HILLSLOPE EROSION AND ROAD CONSTRUCTION IN THE SEDIMENT BUDGET OF GARRETT CREEK, HUMBOLDT COUNTY, CALIFORNIA
   H. Kelsey, D. Best, D. K. Hagans and M. Alpert

7. SEDIMENT ROUTING IN TRIBUTARIES OF THE REDWOOD CREEK BASIN: NORTHWESTERN CALIFORNIA
   J. Pitlick

8. RECENT CHANGES IN CHANNEL STORED SEDIMENT IN REDWOOD CREEK
   M. A. Madej

9. GEOMORPHIC ANALYSIS OF STREAMSIDE LANDSLIDES IN THE REDWOOD CREEK BASIN
   H. M. Kelsey, M. Coghlan, J. Pitlick and D. Best

10. SUMMER COOL POOLS IN REDWOOD CREEK NEAR ORICK, CALIFORNIA
    E. Keller, T. Hofstra and C. Moses

11. THE ESTUARY OF REDWOOD CREEK: IMPACTS OF RECENT DRAINAGE BASIN CHANGES ON AQUATIC HABITAT
    C. Ricks

Management Reports

1. U.S. Department of the Interior. 1975. Environmental Assessment, Management Options for Redwood Creek, Redwood National Park. National Park Service, Western Region, San Francisco. 31 pp.

2. U.S. Department of the Interior. 1979. Draft Environmental Statement, General Management Plan, Redwood National Park. Volume 1 of 3, NPS 1433. National Park Service, Denver Service Center, Denver, Colorado. 247 pp.

3. U.S. Department of the Interior. 1980. General Management Plan, Redwood National Park. NPS 1661. National Park Service, Denver Service Center. Denver, Colorado. 51 pp.

4. U.S. Department of the Interior. 1981. Environmental Assessment, K & K Road Relocation, Redwood National Park. National Park Service, Denver Service Center. Denver, Colorado. 51 pp.

5. U.S. Department of the Interior. 1981. Watershed Rehabilitation Plan, Redwood National Park, Del Norte and Humboldt Counties, California. National Park Service, Denver Service Center. Denver, Colorado. 92 pp.

6. U.S. Department of the Interior. 1982. Redwood National Park Resources Management Plan and Environmental Assessment. National Park Service, Redwood National Park. Crescent City, California. 275 pp.

7. U.S. Department of the Interior. 1982. Redwood National Park Resources Management Plan. National Park Service, Redwood National Park. Crescent City, California. 72 pp.

RESERVOIR SHORELINE REVEGETATION

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INTRODUCTION

Components for Successful Revegetation

Successful revegetations for reservoir and streambank sites involves a number of interacting factors or components: biological, environmental, physical, political, and economic.

The biological component includes: the selection of suitable plant materials, the choice between native and introduced (or "exotic") plants, their propagation, culture, planting, and after-care.

The environmental component includes: rainfall (amount and seasonal distribution), temperature (heat and cold, and time, duration, and intensity), humidity, day length, etc.

The physical component includes: site stability, aspect, i.e. compass facing, (which influences the environmental component), adjacent terrain (e.g. off-site water erosion), wave action, etc.

The political component includes restraints on use of plant materials imposed by governmental jurisdictions or by public pressures (e.g. limited to use of native materials), lack of grazing controls, limitations on the use of chemicals for rodent, insect, disease, or weed control.

The economic component invariably affects final decisions on revegetation methods, planting densities, and the opportunities for advance planning or experimentation.

All of these components must be considered in the planning of any revegetation project.

Revegetation Project Steps

The revegetation project consists of three major steps.

The first is the planning stage. 'This step includes site surveys, vegetation selection, selecting appropriate timetables, and in general, answering questions raised under the various components of the project. Portions of the second step, the procurement step, may run concurrently with the planning stage.

The procurement stage includes collection of seeds and plant materials, production of plant materials, getting growing contracts if needed, location of materials that will be used on-site for stuck cuttings, wattling, and the writing of specifications and contracts.

The final stage is the implementation of the project. A portion of this stage will be discussed in the second talk. Some of it will be covered in the field exercise.

PLANNING

Site Analysis

A number of factors are considered in the site analysis.

Climate and Microclimate: The overall climate of the site must be considered. The selection of plant materials, planting season, site preparation. and planting methods are very dependent on this factor. Summer and winter temperatures (intensities, time, and duration), rainfall amounts and seasonal distribution, season and duration of flooding will all enter into the final revegetation decisions. Fall rains and winter temperature information will affect the choice of planting season in part. Rainfall amounts and distribution will affect choice of plant materials, choice of planting season, and the decisions of whether or not to irrigate. All of the factors will affect the ultimate choice of plant materials.

Vegetation Spectrum: The vegetation existing on and near the site and on similar areas nearby which have revegetated naturally will help in the selection of the plant species to be used, whether these plants are native or introduced ("exotic"). The choice must be made between natives or introduced species. Native species are often mandatory on regional, state or national lands. Native species have known adaptation to the climate and soils of the area. For wattling, brush layering and sticking of unrooted cuttings, materials are often growing nearby. However, in many cases there may be a limited choice of native materials which will be adapted to the rigorous conditions of the site. For example, on reservoir sites in the Central Valley and foothills of California, the extremes between late spring and early summer flooding and late summer and fall drought impose stresses which limit the number of native species available for use.

The use of introduced species allows the potential for selecting plants from many other parts of the world, some of which have similar situations to that found in these difficult areas. The commercial availability of potential plant species usually will be increased when introduced species may be used as well as natives.

Finally, the availability of plant materials will enter into the final choices for the revegetation plan. This topic will be discussed more at length in the second talk. Some other aspects of plant selection, such as the importance of ecotypes or provenances, will also be covered then.

Soils and Fertility: The nature and fertility of soils will affect both erosion potential and plant growth. Information on soil erodibility can be gained from observation and from such sources as the U.S.D.A. Soil Conservation Districts.

The need for fertilizers may be obtained from soil tests, pot tests, and field trials. Soil analyses do not always indicate

the plant response on the site to fertilizers. Pot tests done in the greenhouse or out-of-doors may give a better indication of plant response. The ultimate decision of whether or not to fertilize is best made by field trials but lead time for a project may not allow these trials. Pot tests are best run by the "subtractive method". A complete fertilizer and a non-fertilized treatment serve as the controls and one element is subtracted at a time. Growth comparisons are used to determine fertility needs.

Types of fertilizers may be placed into several categories. Conventional fertilizers are soluble and may be primarily of one or several of the essential plant nutrients. Slow release fertilizers may be subdivided into two general types, those with the nutrient elements in the mineral form and those in which the nitrogen is in an organic form. The latter may be of little value on very sterile soils such as sands because there is insufficient biological activity in the soil to convert the organic nitrogen compounds to the inorganic or mineral form.

Many woody plants, especially natives, may not really need fertilizers even though grasses may respond in the pot tests. Too much fertilizer may impair plant establishment. In colder areas over-fertilization may produce too much growth late in the season and result in excessive winter killing. If water is the limiting factor for plant growth, additional fertilizer will be of little value.

Grass species usually used in revegetation work often perform better or even require additional fertilizers for survival.

The water holding capacity of the soils, season of flooding, and seasonal rainfall distribution will affect the plant selection and the possible need for irrigation for the first season or two.

Special Problems: Many other problems must be considered in developing the final revegetation plan. Among these problems are wave action from wind or boating, site terrain and the need for site

preparation, the occurrence of seeps which may cause slope instability (or conversely aid plant survival), and other erosion factors including off-site run-off water. In stream, river and delta situations the current flow at high water may be a severely limiting factor for revegetation without the use of artificial protection such as gabions, groins, wire fencing, etc.

Developing the Revegetation Plan

When all the information on the site analysis, including the consideration of political and economic contraints, are in hand, a revegetation plan may be developed.

Plant species, total spectrum and quantities, can be determined. Planning for procurement of the plant materials can be done. Season of planting can be determined based on season of flooding, climate and rainfall, availability of irrigation, the size of the project, etc. In large projects, planting may require more than one season and may have to be integrated with the construction of engineering features.

Site preparation planning includes consideration of need for supplemental protection. Severe erosion problems may have to be corrected and minor erosion may be remedied as the planting progresses.

Aesthetics should be considered in the project design but must be considered in conjunction with other factors. A detailed landscape plan as used in urban situations usually is not needed. The project layout may be generalized according to erosion control needs. The natural grouping of plants may be illustrated by a sample of a small section of the area. Usually it is best to avoid straight rows, except in the placement of willow wattling, brush layering, or other special circumstances. Randowm placement of plants, within and among species will help control surficial erosion as well as result in a more pleasing design. Loss of some plants is a foregone conclusion in this type of planting and this will add to the random nature of the finished planting.

Specifications must be written rigorously and in considerable detail. Some seemingly trivial details of planting techniques can be all-important for a successful project. The enforcement of specifications must be equally rigorous.

Procurement of the vegetation materials will be discussed in detail in the later talk but must often be planned well in advance of the actual implementation of the project.

IMPLEMENTATION

Planting Methods--cuttings and transplants

Tools: The tools required will depend upon the revegetation plan, the size of plant materials, soils and size of the project. We will see some of these this afternoon. They include picks, mattocks and shovels for site preparation, shovels, spades or tile spades for planting larger plants and trenching for wattling and brush layering, and dibbles for planting smaller plants and cuttings. Star drills and hammers may be needed to plant cuttings in cemented soils. Power augers may be useful on many sites for larger jobs. Heavy hammers and sledges are needed for staking the job, driving stakes for fencing or cages for plant protection, and driving stakes in the installation of wattling which will be discussed later. Chain saws, lopping and hand pruning shears, and hatchets may be needed for preparation of cuttings and materials for wattling, brush layering and brush matting.

Other materials may include fertilizers, fencing, wire for plant protection cages, and stakes for holding plant protection cages in place and for wattling, etc. Each job will have its own requirements.

Planting Holes and Methods: Size of the planting holes depend on the size of material to be planted and sometimes on the soils. When soils are friable the hold may not need to be much larger than the plant material. In heavy or compacted soils, a larger hold to allow backfilling of looser material may allow better initial root

penetration. The size also depends on whether fertilizer or soil amendments are used.

When fertilizers are used the holes should be deeper than needed for the plant size, the fertilizer mixed thoroughly in the bottom and cover with several inches of backfill to avoid burning of the plant roots. Use only the quantities of fertilizers recommended by the manufacturer or as needed, determined by soil or pot tests.

The use of soil amendments is currently in debate. Older recommendations often called for amendments to loosen the soil and increase water holding capacity. Some research indicates that root growth may be increased in the amended soil but not into the native, undisturbed soil. Amendments may increase planting costs substantially.

Holes on slopes should be constructed as in the following diagram to accumulate moisture and reduce burying of the plant by loose soil.

Planting should be done immediately after digging the planting holes to reduce drying of the back fill. This is especially important where supplemental irrigation is not available. Plants should be removed from the containers unless containers are bio-degradeable. With bio-degradeable pots, especially peat pots, the pot rim should be removed well below soil level. If the roots have not thoroughly penetrated, bio-degradeable pots the material should be removed entirely.

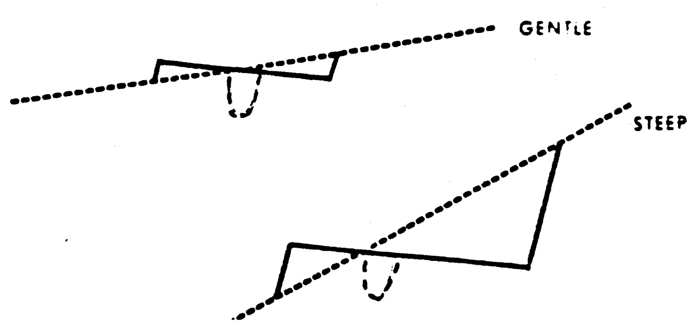

Figure 1. Planting pocket on a slope. Adjust scale according to the size of the container.

In revegetation projects it is usually desireable to set the plants just below the level in which they were grown to conserve soil moisture. This is contrary to the usual level planting recommended for irrigated landscapes. Backfill should be thoroughly tamped to insure good soil-rool contact and to eliminate any air pockets. If irrigation is available, the plants should be watered in to aid this compaction and supply supplementary water.

The use of berms around the planting hole are often useful to concentrate rainfall or irrigation. These should be two to four inches high and of sufficient diameter to perform this function. On sloping ground it is desireable to leave the berm open on the uphill side to trap more run-off. The inside of the berm should be tapered toward the plant to concentrate water near the root system.

The use of mulches is of questionable value on sites subject to flooding. Plastic mulches may reduce aeration when the plants are flooded. Organic mulches will float away. However, if a growing season will elapse between planting and flooding time, mulches may increase plant survival. If plastic is used is should be removed before flooding.

Planting Methods--seeding:

Hydroseeding: Grasses and forbs may be established by hydroseeding where flooding does not coincide with the germination stage. Woody species usually cannot be established this way because seeds are not placed beneath the soil surface. Costs vary with the choice of species, rates of seeding, choice of mulch as binders, size of job (and equipment), accessibility, etc. These costs may run to $700-1,000 per acre.

Range Drills: Grasses and forbs may be economically seeded on gentle sites with modified range drills.

Direct or Spot Seeding: Wood species may be established on

some sites by direct seeding. This technique requires fairly intensive management, but where successful, it is more economical than using transplants.

Plant Protection

On many sites it is essential to protect plantings from damage by animals. Rodents will often girdle plants at ground level. Rabbits, deer, and domestic animals can kill plants by browsing and the larger animals can cause considerable damage by trampling.

Wire caging is usually used for this purpose. The mesh size and height of the cage will depend on the predator. Hardware cloth is needed for mice and other small animals. Larger mesh sizes are suitable for larger animals. The height will depend upon the kind or kinds or predators. They may be fairly low for rodents, of medium height for rabbits and as much as three or four feet for larger animals. The larger animals may browse the branches extending beyond the cage and even the tops but the plants will usually out-grow the need for protection. These cages may be anchored with wooden stakes or large "hairpins" made of number 9 wire.

Fencing an entire site may be necessary in areas where deer populations are heavy or where domestic animals graze.

Maintenance and After Care

Irrigation: The use of irrigation will improve growth and survival of plantings. The decision must be made on economics contrasting overplanting and increased plant mortality vs. the cost of irrigation. On many sites, irrigation may not be necessary because of summer rainfall. On other sites the increase in survival may be worth the cost. Some species which are both flood and drought tolerant may be irrigated for one or two years and then can survive without further irrigation.

Temporary irrigation systems may be used where soils and

choice of plant materials permit (e.g. Eucalyptus species on soils of adequate moisture holding capacity) or where plant roots can eventually reach moist soil near the water's edge. These systems may be moveable such as irrigation pipe or inexpensive plastic which can be abandoned.

Permanent systems may be needed where choice of drought-flood tolerant plants is very limited or where recreational use or sensitivity of the site justify the costs.

Many types of systems are available. Underground systems which are perforated have been used. Above-ground systems may be conventional sprinklers or some of the newer types using drip, trickle or emitter distribution.

Weed Control: The control of weeds is desireable in any revegetation planting. On riparian sites the use of chemicals often is not possible. Persistent chemicals such as pre-emergence herbicides should never be used. Only those chemicals which degrade rapidly into harmless compounds should be used and their use should be limited to some distance from the water. The problems with mulches has already been discussed. In many cases only manual weed control is acceptable.

Fertilization: Additional fertilizer should not be needed for most woody plants if the correct species have been selected. On some soils, grass vegetation cover may need to be fertilized to maintain adequate stands.

Planting Methods--Wattling, Brush Layering and Brush Matting

Wattling: The word wattle is derived from an Anglo-Saxon word watel, meaning interwoven twigs and hence a framework or hurdle made of such. The word was adopted by Dr. Kraebel in the 1930's to describe a process of erosion control where bundles of willow or other materials were tied and placed in trenches, overlapped and staked and partially covered with soil.

For steep cuts and fills and other areas subject to downhill

movement, wattling must be placed on contour. On riparian sites subject to wave action they may be placed diagonally to the wave action. This may be done in one direction if the waves come from varied directions. The length of the "runs" will need to be determined by local site conditions including such factors as the water level fluctuation. We have only used it to a limited extent to help establish fish spawn habitat at Lake Oroville. I have not had an opportunity to observe the long term effectiveness of this use.

The methodology is best summarized by the following diagram and set of specifications:

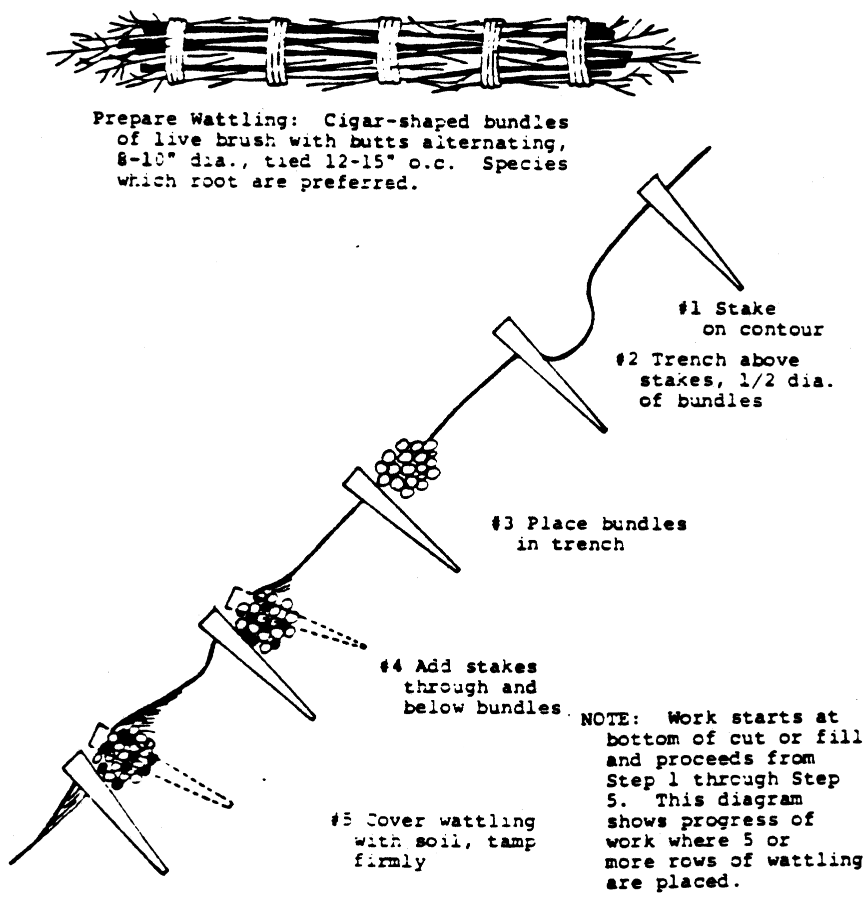

Figure 2. Wattling installation - schematic diagram.

Specifications are as follows:

1. Wattling bundles shall be prepared from live, shrubby material, preferably of species which will root, such as Salix spp. (willow), Baccharis spp. (Coyote bush and Mulefat), etc.

2. Wattling bundles may vary in length, depending on material available. Bundles shall taper at the ends and shall be 1 - 1 1/2 ft. (max. 2 ft.) longer than the average length of stems to achieve this taper. Butts shall not be more than $\pm$1 1/2 in. in diameter.

3. Stems shall be placed alternately (randomly) in each bundle so that approximately one-half the butt ends are at each end of the bundle.

4. When compressed firmly and tied, each bundle shall be $\pm$8 in. in diameter ($\pm$2 in.).

5. Bundles shall be tied on not more than 15 in. centers with two wraps of binder twine or heavier tying material with a non-slipping knot.

6. Bundles shall be prepared not more than two days in advance of placement except that if kept covered and wet they may be prepared up to seven days in advance of placement.

7. Grade for the wattling trenches shall be staked with an Abney level, or similar device, and shall follow slope contours (horizontal).

8. Trenches shall be 3 ft. vertical spacing (or such other spacing specified. Economics may dictate wider placement).

9. Bundles shall be laid in trenches dug to approximately one-half the diameter of the bundles, with ends of bundles overlapping at least 12 in. The overlap shall be as long as necessary to permit staking as specified below.

10. Bundles shall be staked firmly in place with vertical stakes on the down-hill side of the wattling not more than 18 in. on center and diagonal stakes through the bundles on not more than 30 in. centers (see Fig. III-1). Where bundle overlap occurs between previously set bottom or guide stakes, an additional bottom stake shall be used at the midpoint of the overlap. Bundle overlaps shall be "tied" with a diagonal stake through the ends of both bundles.

11. Stakes may be made of live wattling material greater than 1 1/2 in. in diameter or they may be construction stakes (2" x 4" x 24" or 2" x 4" x 36", diagonal cut). Reinforcing bar may be substituted only as specified below.

12. All stakes shall be driven to a firm hold and a minimum of 18 in. deep. Where soils are soft and 24 in. stakes are not solid (i.e. if they can be moved by hand), 36 in. stakes shall be used. Where soils are so compacted that 24 in. stakes cannot be driven 18 in. deep, 3/8 - 1/2 in. steel reinforcing bar shall be used for staking.

13. Work shall progress from the bottom of the cut or fill toward the top and each row shall be covered with soil and packed firmly behind and on the uphill side of the wattling by tamping or by walking on the wattling as the work progresses or by a combination of these methods.

14. The downhill "lip" of the wattling bundle shall be left exposed when staking and covering are completed. However, the preceding specification must be rigorously adhered to.

Wattling has several advantages: energy dissapation, temporary stabilization to allow establishment of other vegetation, sediment entrapment, and, if easy-to-root species are used, it becomes part of the vegetation component. It may ultimately be crowded out by more dominant species.

Costs and labor breakdown of wattling and sticking unrooted willow cuttings on a small job (± 1 Acre) at Lake Tahoe in 1973 are as follows:

1. **Prepared and Install Wattling (1,140 lf)**

a. **Labor**

		Man Hours
1)	Scaling (1/2 total)	2
2)	Cutting	27
3)	Prepare (stack, tie, load)	28
4)	Layout	9
5)	Install	75
6)	Down time (rain, 1/2 total)	10
7)	Travel (from Sacramento, Marysville, 1/2 total)	42
		193 @ $9.00* = $1,737

b. **Material**

** 1)	840 Con Stakes (2x4x24") @ 25¢ ea.	$210
2)	Misc. (twine, gas, etc.)	50
3)	Willows (obtained from Forest Service)	0

c. **Equipment**

1)	Chain saw	25
2)	Transportation and trucking	200
3)	Misc. (shears, mattock, shovel, hammer, etc.)	25
	Total	$2,247

Unit Cost: $2,247 ÷ 1,140 = $1.97/lf, say $2.00/lf for Wattling.

2. **Prepare and Plant Willow Cuttings (8,000 Cuttings)**

a. **Labor**

		Man Hours
1)	Scaling (1/2 total)	2
2)	Cutting	9
3)	Prepare	34
4)	Plant	76
5)	Down time (rain, 1/2 total)	10
6)	Travel (from Sacramento, Marysville, 1/2 total)	42
		173 @ $9.00* = $1,557

b. **Material**

1)	Willows (obtained from Forest Service)	$ 0
2)	Misc. (twine, auxin solution, etc.)	50

c. **Equipment**

1)	Transportation and trucking	200
2)	Misc. (shears, drills, hammers, etc.)	25
	Total	$1,832

Unit Cost: $1,832 ÷ 8,000 = $0.229¢/ea., say 23¢/ea. for Willow Cuttings

Or

6¢/sq. f. (based on planting willows at about 2' centers).

* $7.00/hr. + $2.00/hr. subsistence.
** 1.36' o.c. for stakes (except doubled at overlap, so probably 1.5' o.c. average).

Brush Layering: This is a technique used in Europe for years and to a limited extent in the United States. This technique may be installed at the time of construction of fills but for most riparian work it would be installed in a trenching operation. The general principles are illustrated in the following figure.

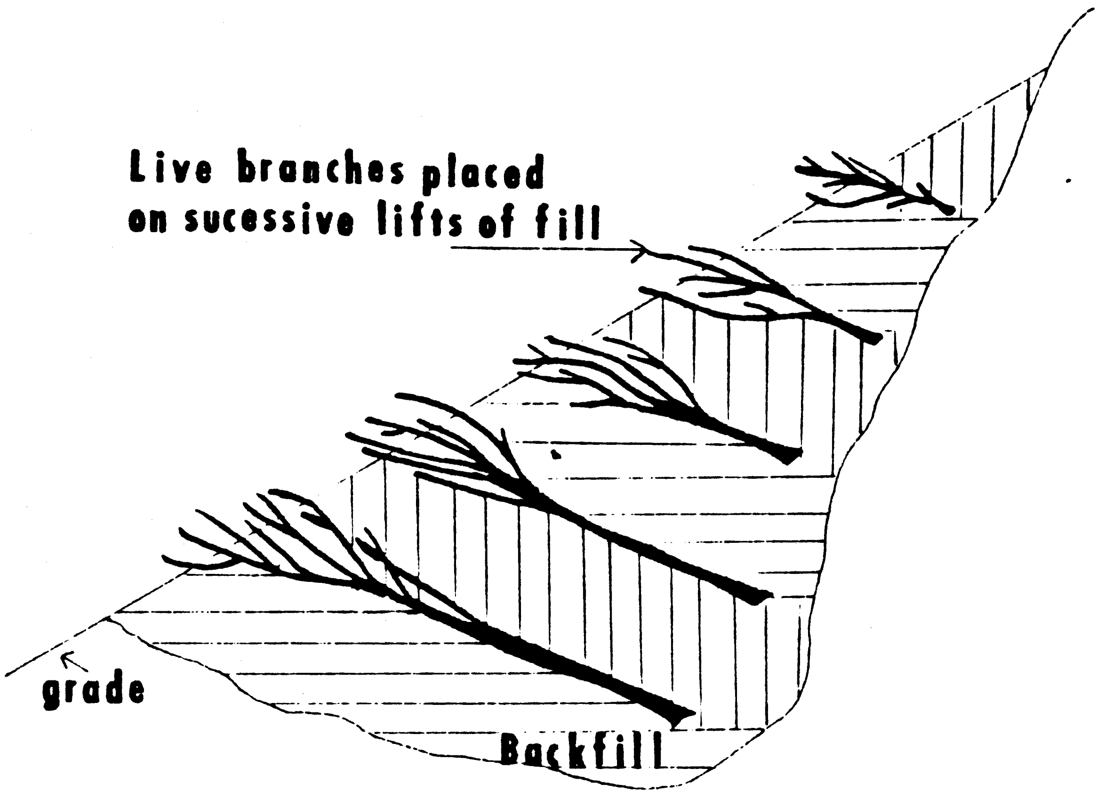

Figure 3. Schematic diagram of brush layering. Brush may be laid on lifts of fill or in trenches cut in existing slope.

Cut, live material, preferably live plants of a species which will root are laid either on the successive "lifts" of a fill or in trenches cut successively from the bottom to the top as in wattling. Soil removed from each successively higher cut is used to fill the cut below. The cut material will vary in length depending on whether the use is in fill construction or on existing banks. For fills, brush may be up to six feet or more long. For trenching, lengths of two to three feet long are more suitable. Cut branches should be laid in a criss-cross pattern for greater stability. Avoid excessive lengths of protruding branch ends to avoid excessive sediment build-up which can ultimately cause increased erosion.

Brush layering has the similar advantages of wattling: energy dissapation, entrapment of sediments, temporary erosion control and, if of rootable species, it adds to the vegetation component.

Brush Mattling: This procedure is the laying or brush secured with stakes and wire. The brush may be laid on site and covered with wire netting or a network of wires, or placed between two layers of wire netting which are then tied together by wire to make a mattress. A disadvantage of brush matting is that other plants, cuttings or transplants are difficult or impossible to plant through the matting. Matting must be throughly anchored or it will float or wash away and it must be protected from under-cutting.

SUMMARY

No vegetation techniques will resist severe erosion until established. Auxiliary methods such as rip-rap gabions, bulkheads, and groins often need to be used in riparian situations for protection until establishment of vegetative cover. In areas of severe stream or river flow they may be needed in addition to vegetation as a permanent part of the control measures.

I have tried to describe a <u>strategy</u> for the solution of some riparian revegetation problems. This <u>strategy</u> has worked well in the solution of a wide variety of situations in addition to those

of reservoir shoreline revegetation. The steps or components of the overall revegetation plan are:

1) thorough analysis of the problem, considering the various components and constraints, and biological, environmental, physical, political and economic factors;
2) thorough planning including all of the information obtained from the first step; and
3) careful implementation of the revegetation plan.