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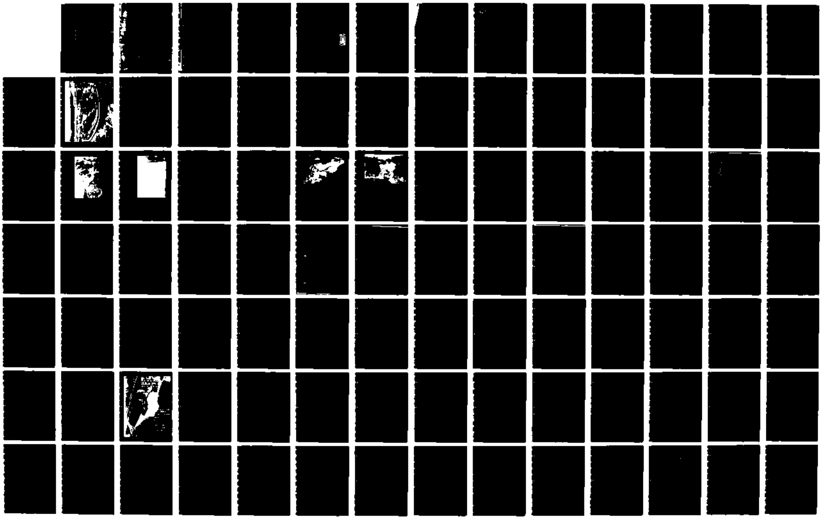

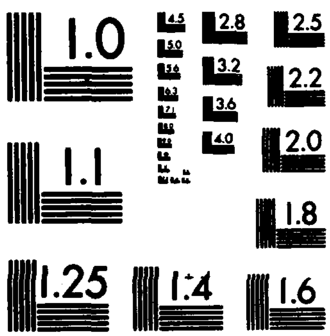

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1606 AIR BASE WING

Engineering, Construction, and Environmental Planning Branch

Kirtland Air Force Base, New Mexico

ENGINEERING STUDY for

KIRTLAND DESIGN CENTER

DTIC

JUL 20 84

EROSION CONTROL at TRESTLE

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MICHAEL W. KOCH, Capt., USAF

University of New Mexico

Submitted to

DR JOHN B. CARNEY, Jr

CE 552

23 April 1984

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AUTHOR: Michael W. Koch

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AT

TRESTLE

AN ENGINEERING STUDY

CE 551

SUBMITTED TO

DR. JOHN B. CARNEY, JR.

DEPARTMENT OF CIVIL ENGINEERING

UNIVERSITY OF NEW MEXICO

MICHAEL W. KOCH, CAPT, USAF

UNIVERSITY OF NEW MEXICO

MAY 1984

ACKNOWLEDGMENTS

No one ever does anything without assistance, advice, support, or inspiration from others. I certainly found that out during the writing of this report. My heartfelt thanks and gratitude goes to the men and women of the 1606th Engineering Design, Construction, and Environmental Planning Branch, especially to Airman First Class Tony Woodworth and Mr. Nick Mitchell of the drafting section; to Alice Petencin who somehow managed to read my primitive scribble and miraculously translated it to type written words; to Brenda, Kyra, and Michael who sacrificed valuable family time; to Dr. John B. Carney, Jr. who made learning an almost pleasurable experience; and finally, to Technical Sergeant James Pomasl, a very close friend, who made the funding for this entire academic adventure a reality.

DEPARTMENT OF THE AIR FORCE
AIR FORCE WEAPONS LABORATORY (AFSC)
KIRTLAND AIR FORCE BASE, NM 87117

REPLY TO ATTN OF: PA

SUBJECT: Security Review

TO: AFWL/NTEDE (Lt Kitch)

The attached Thesis titled: Erosion Control at Trestle, does not require security review. This decision was made in consultation between AFWL/PA and AFCMD/PA.

ROBERT F. MISKOWICZ, TSgt, USAF
NCOIC of Public Affairs

COURSE CONTRACT

Submitted to

Dr. John B. Carney, Jr., 6 February 1984

My proposal to satisfy CE 551 course and graduation requirements for a Master's Degree in Civil Engineering Soils and Foundations from the University of New Mexico is to perform an engineering study for the 1606th Air Base Wing Civil Engineers; Kirtland Air Force Base (KAFB), New Mexico. My problem will be Erosion Control, TRESTLE, Project Number KLD 58-1. The study will include theoretical and semiempirical treatments of the erosion process as well as a detailed discussion of methods of checking erosion (erosion control techniques). I intend to describe existing conditions at the TRESTLE, current maintenance costs, mission impact, what has been done so far, physical and monetary restrictions, design limitations, scope of work, etc. I will also be taking soil samples from the site to perform lab tests. As a minimum, I will run grain size distribution, moisture content, specific gravity, and consistency tests. I will make cost comparisons of the methods available for erosion control of this particular type soil and provide recommended solutions along with my own conclusions.

INTRODUCTION

The primary purpose of this report is to satisfy course requirements for CE 551 (CE 552) in lieu of a thesis for a Master's Degree in Civil Engineering Soils and Foundations for the University of New Mexico. The report addresses a real world problem that has been identified as a design project for the Kirtland Air Force Base (KAFB) Civil Engineer; Albuquerque, New Mexico. The design project is titled Erosion Control at TRESTLE, project number KLD 58-1.

The report is intended to provide a broad perspective of the erosion process and erosion control alternatives, then focus attention on the specific problem at the TRESTLE through field study and soil lab analyses. The first three chapters do not address the problem. They resulted from literature research and are included only to provide more detailed insight into the subject of erosion. Chapter 1 discusses the erosion process in general and how to compute soil losses from a given area or construction site. Chapter 2 pertains to conventional erosion control techniques. Chapter 3 explores soil stabilization methods. Those interested in the TRESTLE only may proceed directly to Chapters 4 and 5 for a description of the facility, discussion of the specific problem, and recommended solutions.

This report was accomplished in cooperation with KAFB 1606th Civil Engineers and KAFB Air Force Weapons

Laboratory TRESTLE personnel. All questions concerning this report may be addressed to the author at Headquarters, Air Force Engineering Service Center/DEMP; Tyndall AFB, Florida 32403.

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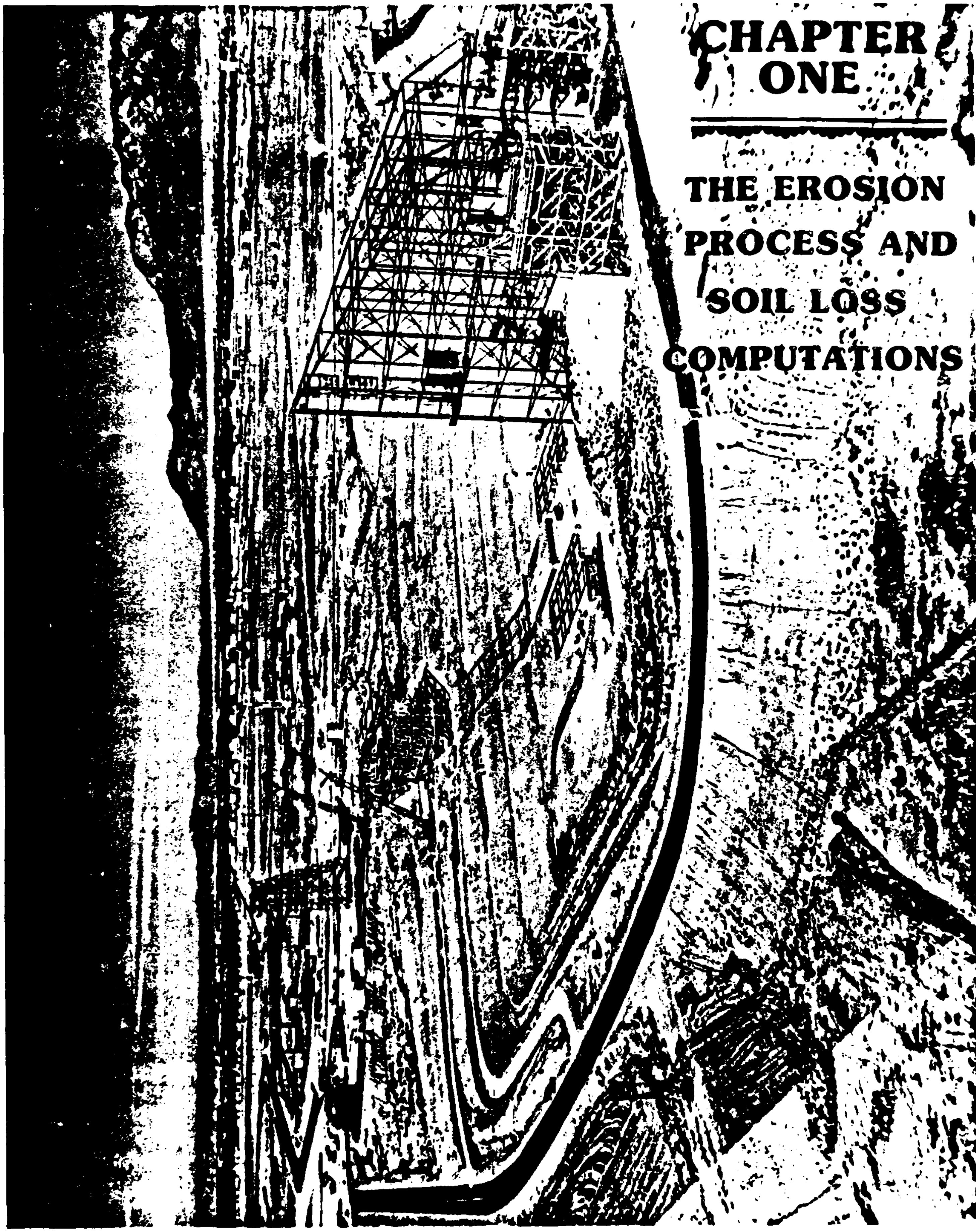


CHAPTER ONE

THE EROSION PROCESS AND SOIL LOSS COMPUTATIONS

INTRODUCTION

Highway engineers have realized since long before the Appian Way that erosion is one of the major problems in the maintenance of all roads in any environment. More recently, as the earth's resources have become more scarce, greater emphasis is being given to the control of erosion during the construction phases of all civil engineering projects. Today's engineers must not only protect the immediate site from loss of material, but must also prevent eroding soil from damaging the environment outside the construction area. Erosion control results from a conscientious effort by civil engineers, architects, planners, and builders alike. However, before any workable control methods can be realized or recognized, we must understand the causes and magnitude of the erosion process.

In 1972, Rapid City, South Dakota, situated at the foot of the Black Hills, was hit by a flash flood. The sudden cause of vast destruction and tragic loss of life shocked the nation. Rocks, mud, and silt could be seen everywhere in the devastated city. The flood, predictable enough once the rain began to fall so rapidly and continuously from a most unusual storm of long duration, was only the final transporting agent of the debris that invaded Rapid City. Much of that debris began its down hill trek a long time before, having been produced at a normal rate by chemical and mechanical weathering in the high parts of the central core of the Black Hills. The prevailing erosional process was suddenly augmented by the rapid downhill movement of water-soaked soil and rock into the flooding tributary streams of the Cheyenne River. The river rushed masses of debris onto the plains. Rock fragments and clay that had taken tens to hundreds of years to produce and build up on the slopes were caught up in fast downhill movements, provoked by a storm that might occur only once in a hundred years (Press and Siever, 1972).

Other examples of devastation caused by soil erosion exist throughout the U.S. and around the world. For instance, there have been several significant land slides in California that can be attributed to erosion effects. Fortunately, erosion catastrophies resulting in loss of life do not occur on a daily basis. However, highway maintenance and other kinds of property restoration made necessary due to soil erosion costs Americans millions every year—and the costs are steadily rising.

In New Mexico, the state expended more than a half million dollars during the period 1 July 1982 to 30 June 1983 on erosion repairs to its highways. The New Mexico State Highway Department Maintenance Division defines erosion repair as, "Repairing damage due to water or wind to include removing, hauling, placing, and compacting material to effect the necessary repairs." The state's erosion repair maintenance code does not include emergency repairs necessary due to land slides, repairs to culverts and other drainage channels, to bridges, guard rails, fences, signs, pavements, and the many other elements of highway maintenance that are definitely impacted by the erosion process. Consequently, I suspect that a half million dollars is an extremely conservative estimate of the state's repair costs due to erosion. As an example of the New Mexico State Highway Department's maintenance cost breakdown, an excerpt from district 3's report is provided in Figure 1.1 (District 3 includes the Albuquerque area west to the Arizona state line, south to Belen and north to Bernalillo).

*The cost of con*trolling water has to be a significant part of any state's total highway expense. According to John L. Sanborn, Research Assistant, Purdue University, approximately a quarter of the cost of new construction is for drainage or erosion control. Erosion is a particularly expensive factor because it costs money in two ways—soil lost must be replaced and soil deposited in drainage structures must be cleaned out. It is important, therefore, to plan and develop, carefully, adequate facilities for drainage, and measures for controlling erosion.

REPORT AMMS 0-01
DATE PREPARED 07/19/83

NEW MEXICO STATE H[illegible]Y DEPARTMENT
MAINTENANCE DIVISION
MAINTENANCE MANAGEMENT SYSTEM
MANAGEMENT EXCEPTION REPORT

OF 3

MR. WES HIBNER
SANTA FE OFFICE
983-0330

DISTRICT 3 SUMMARY

FROM 07/01/82 TO 06/30/83

MAINTENANCE WORK ACTIVITY CODE	NAME	PLAN OR ACTUAL	CREW DAYS ANNUAL	CREW DAYS TOT FOR PERIOD	ACCOMPLISHMENT ANNUAL	ACCOMPLISHMENT TOT FOR PERIOD	UNIT OF MEASURE	COST DISTRIBUTION ANNUAL	COST DISTRIBUTION TOT FOR PERIOD	UNIT COST	EXCPT	ACTION TAKEN TO CORRECT
1250	EROSION REPAIR	PLAN	55	55	3,520	3,520	MAN-HOUR	45,815	45,915	13.02	A	
		ACTUAL	68	83	5,656	5,656			73,471			
12[illegible]0	EROSION REPAIR	PLAN	17	17	1,088	1,088	MAN-HOUR	18,513	18,513	17.02	A	
		ACTUAL	7	7	464	464			7,950			
12[illegible]0	STR EXT/RIDE[illegible]ING	PLAN	34	34	1,904	1,904	MAN-HOUR	59,262	59,262	31.13	A	
		ACTUAL	15	15	836	836			15,255			
13[illegible]	LITTER PICKUP	PLAN	1,110	1,110	5,550	5,550	SHLD MI	190,920	190,920	34.40	A	
		ACTUAL	922	822	6,780	6,780			141,332			
137[illegible]	FENCE MAINTENANCE	PLAN	465	455	232,500	232,500	FT FENCE	114,390	114,390	.49	A	
		ACTUAL	[illegible]	583	150,442	150,442			144,722			
13[illegible]	[illegible]TCE OF [illegible] SIGNS	PLAN	418	413	8,360	8,360	NO.SIGNS	155,314	155,914	18.65	A	
		ACTUAL	4[illegible]0	430	14,11[illegible]	14,11[illegible]			263,319			
13[illegible]0	[illegible]TCE OF [illegible] SIGNS	PLAN	164	164	3,280	3,280	NO.SIGNS	71,663	71,663	21.85	C	
		ACTUAL	145	135	2,979	2,979			52,906			
13[illegible]0	[illegible]TCE OF [illegible] SIGNS	PLAN	47	47	245	245	NO.SIGNS	52,185	52,185	213.00	A	
		ACTUAL	56	56	421	421			89,673			
14[illegible]	GUARD RAIL MTCE	PLAN	146	146	43,600	43,600	FT RAIL	99,206	99,206	2.04	A	
		ACTUAL	243	243	24,542	24,542			50,056			
14[illegible]0	GUARD RAIL MTCE	PLAN	33	33	11,400	11,400	FT RAIL	25,650	25,650	2.25	A	
		ACTUAL	12	12	161	161			362			
14[illegible]0	CATTLEGUARD MTCE	PLAN	24	24	576	576	MAN-HOUR	26,856	26,856	45.63	A	
		ACTUAL	35	35	836	836			38,941			
[illegible]	[illegible] STALL PAVEMENT ARK	PLAN	50	50	1,200	1,200	MAN-HOUR	52,550	52,550	43.79	A	
		ACTUAL	0	0	0	0			0			
1[illegible]70	[illegible] PRECOATING	PLAN	10	10	9,000	9,000	CU YARDS	8,830	8,830	.98	A	
		ACTUAL	0	0	0	0			0			
[illegible]	[illegible]	PLAN	[illegible]	80	48,[illegible]	4[illegible]	CU YARDS	70,540	70,540	1.47	A	
		ACTUAL	29	[illegible]	[illegible]0,757	[illegible]0,757			24,253			
[illegible]	[illegible] MTCE	PLAN	570	570	4,550	4,550	MAN-HOUR	47,310	47,310	10.38	A	
		ACTUAL	905	[illegible]05	6,[illegible]43	6,[illegible]43			56,849			
[illegible]	[illegible]	PLAN	50	50	400	400	MAN-HOUR	10,450	10,450	26.13	A	
		ACTUAL	2[illegible]	2[illegible]	140	150			4,150			
[illegible]	[illegible]	PLAN	236	236	3,296	3,296	MAN-HOUR	37,904	37,904	11.50	A	

EROSION DEFINED

By definition, soil erosion is the removal of surface layers of soil by wind, water, or ice; it involves a process of particle detachment and transport by any or all of these agents. Erosion is initiated by drag, impact, or tractive forces acting on individual particles of soil at the surface.

The two most common types of erosion are water and wind erosion. The complexity of water erosion may be understood by considering the possible routing of a single raindrop. When a raindrop strikes the ground, it has impact energy which tends to loosen or detach particles from the soil surface--the beginning of erosion. If the raindrop runs down the surface, it introduces movement to the detached particles and the true definition of erosion is complete.

When the drops impact on bare or fallow ground, they can dislodge and move soil particles a surprising distance. At the beginning of run-off, water collects into small rivulets which may erode very small channels called rills. These rills may eventually become larger and deeper channels called gullies. Gullying is a complex and destructive process; once started gullies are difficult to stop.

As might be expected, the first erosion control measures are to reduce raindrop impact and to slow down soil particle movement. The simplest solution is to provide a vegetative cover on the soil. Unfortunately, vegetation does not occur instantaneously. Some temporary measure, usually mulch, must be used until the vegetation is established. The raindrop continues on its way, joined by other raindrops to collect in a rill, gully, ditch, or some other water course. The shape, roughness, and slope of a water course combined with the quantity of runoff determine the rate of flow. At low velocities, the flow will not dislodge or move the soil particles in the water course bed. As the velocity increases, vegetation may still prevent erosion if the flow is intermittent so the vegetation can survive. Higher velocities require a more positive protective lining or the use of small dams (ditch checks) to reduce the velocity. Linings may be concrete, which increases velocity, or stone, which decreases velocity.

Each type of lining has its drawbacks. For example, the added velocity of the concrete linings causes exit problems—overtopping or undermining of these linings causes complete failure. Rock linings, on the other hand, require larger channels to accommodate the slower flow rate; the rocks can move and the fines under the rocks can be sucked out causing settlement.

When the flow in the channel reaches a constriction, such as a pipe culvert, further erosion problems result. Whirlpools occur at the entrance loosening more soil. Downstream from the pipe, problems result from the added exit velocity due to constriction of the pipe, increased pipe slope, or reduced pipe friction. These problems may necessitate the use of head walls or other energy-dissipation structures at pipe outlets.

When the raindrop finally reaches a natural waterway such as a river, it continues to cause problems. In floods it can erode the toe of the slope of adjacent roadways. At bridges where flow is again constricted, flood water causes general scour (lowering of the channel bed by erosion). It can cause local scour around piers, abutments, and along embankments. Piping or spring sapping is yet another type of erosion caused by seepage and emergence of water from the face of an unprotected slope.

LANDSLIDES

Erosion, as you see, can cause some very serious engineering problmms--some even catastrophic. Another phenomenon often related to erosion is the movement of large soil masses. However, mass-movements, popularly known as landslides, may or may not be caused by soil erosion. The term is a descriptive name for the downward and outward movement of slope forming materials--natural rock, soils, artificial fills or a combination of these materials.

Unlike soil erosion, landslides involve the sliding, toppling, falling, spreading, or flowing of fairly large and often intact pieces of earth. The processes involved in slides comprise a continuous series of events from cause to effect. All slides involve the failure of earth materials under shear stress. Many of the same slope, soil, and hydrologic factors that control erosion also control landslides--for example, steepness of slope and soil shear strength. Precipitation, a key factor directly affecting rainfall erosion, only affects mass-movement indirectly. That is, precipitation may influence or change the ground water profile at the site of the potential landslide, or may assist in the removal of lateral support which leads to instability, or it may add its own weight as a surcharge. In contrast, geologic conditions such as orientation of joints and bedding planes in a slope have significant influence on mass stability but not on surface erosion. More to the point, the study of mass-movements requires individual in-depth consideration. Consequently, the subject will be left for more detailed attention in another report.

AGENTS OF EROSION

As previously defined, erosion is characterized by the detachment and transport of individual soil grains. The primary agents of erosion include water, wind and ice. These elements can scour and remove soil particles as a result of flowing past, impacting upon, or exiting from the surface of a soil. Frost action may act in concert with wind and rainfall to initiate and facilitate down slope movement of soil particles. The primary agents of erosion and their effect is summarized in Table 1.1.

Table 1.1. AGENTS AND TYPES OF EROSION

Agent	Type of Erosion or Degradational Process	
Water	1. Raindrop splash	
	2. Sheet erosion	
	3. Rilling	
	4. Gullying	
	t. Stream channel erosion	
	6. Wave action	
	7. Ground water piping	
Ice	1. Solifluction	
	2. Frost action	
	3. Glacial scour	
	4. Plucking	
Wind	Wind cannot be subclassified into types; instead it varies only by degree.	
Gravity	1. Creep	These are usually classified under mass-wasting, but they often act in conjunction with erosion.
	2. Earth flow	
	3. Avalanche	
	4. Rock fall	
	5. Debris slide	

Note: Many of the above processes operate jointly or in combination. This is particularly true of mass-wasting (gravity + water) and glacio-fluvial (ice + water) processes.

Reference: Biotechnical Slope Protection and Erosion Control by D. H. Gray and A. T. Leiser, 1982.

RAINFALL EROSION

Rainfall erosion is a function of four basic factors:

Climate - storm intensity and duration

Soil - inherent erodibility

Topography - length and steepness of slope

Vegetation - type and extent of cover

The most important climatic parameters controlling rainfall erosion are the intensity and duration of the rain. Rainfall erosion occurs in several forms beginning with the raindrop splash and ending with the gully. The mechanics of erosion differs considerably from one form to another.

Raindrop splash results from the impact of water drops falling directly on exposed soil particles or thin water surfaces covering the ground. The kinetic energy of the drops can splash soil particles into the air. On level ground the particles are distributed more or less uniformly in all directions, but on a slope there is a net transport down slope. Tremendous quantities of soil can be splashed into the air during a heavy storm. (Ellison, 1948, estimated as much as 100 tons per acre.) Splashed particles may move more than two feet vertically and five feet laterally on a level surface.

The impact of rain on bare ground also destroys the porous, open structure of soils, reducing their infiltration capacity. Increases of 15 percent in density in a one inch surface layer of soil have been attributed to raindrop impact. Of course, the erosion potential of raindrops is a function of the rainfall energy or momentum. Rainfall energy, in turn, is related to the intensity of the rain. Wischmeier and Smith, 1958, offers an approximate relationship between rainfall energy and intensity:

$$E = 916 + 331 \log i \qquad (1.1)$$

Where E = kinetic energy of rain (ft-tons/acre-in).

i = rainfall intensity (in/hr).

A summary of the kinetic energy and velocity of fall for various rainfall intensities and size of raindrops is given in Table 1.2.

Notice in Table 1.2, raindrops vary in diameter (d); also, terminal velocity (V) varies with the diameter from 0.01 to 30.5 ft/S. Since kinetic energy is proportional to d^3V^2, the erosive power of a raindrop from a cloud burst may be over seven billion times that from a fog. A more realistic comparison is a raindrop from a cloud burst has an erosive power of about two thousand times that from a drizzle. This conforms with the observation that a few intense storms account for most of the erosion. The effect is augmented by the fact that overland flow is more likely to occur during intense rains (Linsley, 1975).

SHEET EROSION

Sheet erosion is the removal of soil from sloping land in thin layers or sheets. From an energy standpoint,

Table 1.2. Kinetic Energy and Velocity of Raindrops for Various Rainfall Intensities., (From Lull, 1959)

Rainfall	Intensity, IN/HR	Median Diameter, Inch	Velocity of Fall FT/SEC	Drops per SF Per Sec	Kinetic Enegery, FT-LB/SF/HR
Fog	0.005	0.00039	0.01	6,264,000	$4.04\text{x}10^{-8}$
Mist	0.002	0.0039	0.7	2,510	$7.94\text{x}10^{-5}$
Drizzle	0.01	0.038	13.5	14	0.148
Light rain	0.04	0.049	15.7	26	0.797
Mod. rain	6.15	0.063	18.7	46	4.241
Heavy rain	0.60	0.081	22.0	46	23.47
Excessive rain	1.60	0.095	24.0	76	74.48
Cloud burst	4.00	0.11	25.9	113	216.9
Cloud burst	4.00	0.16	29.2	41	275.8
Cloud burst	4.00	0.24	30.5	12	300.7

raindrop erosion appears to be more important than sheet erosion because most raindrops have velocities of about 20 to 30 fps (Table 1.2), whereas overland flow velocities are about one to two fps. The eroding and transporting power of sheet flow are a function of the depth and velocity of runoff for a given size, shape, and density of soil particles.

Dry ravel and slope wash are forms of sheet erosion. Dry ravel occurs when surface layers of coarse-textured soil dry out and lose their apparent cohesion. Slope wash occurs when rainfall erodes without causing rilling or gullying. Frost heaving may cause more or less uniform loosening of surface layers which later erode from rain or wind action. Sheet erosion readily occurs in cut slopes of granitic and andesite soils. Highway cuts in these soils often give the impression of being very stable because rills and gullies are usually absent. However, the cuts will discharge tons of soil year after year to the roadside ditches through sheet erosion.

RILL EROSION

Rill erosion is the removal of soil by water from very small but well defined, visible channels or streamlets where there is concentration of overland flow. Some examples of rill erosion at the Kirtland Air Force Base (KAFB), New Mexico Trestle site are shown in Figures 1.2 and 1.3. Rilling is much more serious than sheet erosion because runoff velocities are higher in the rills or channels. Most rainfall erosion losses occur during rill erosion (Schwab, 1966).

Rill erosion is most serious where intense storms occur in water sheds or sites with high runoff-producing characteristics and loose, shallow top soil. Rills are sufficiently large and stable to be seen readily, but small enough to be removed easily by normal tillage and grading operations.

Fig. 1.2. Rill erosion at the TRESTLE site. This one runs about 18 inches deep and there are numerous such rills throughout the facility grounds.

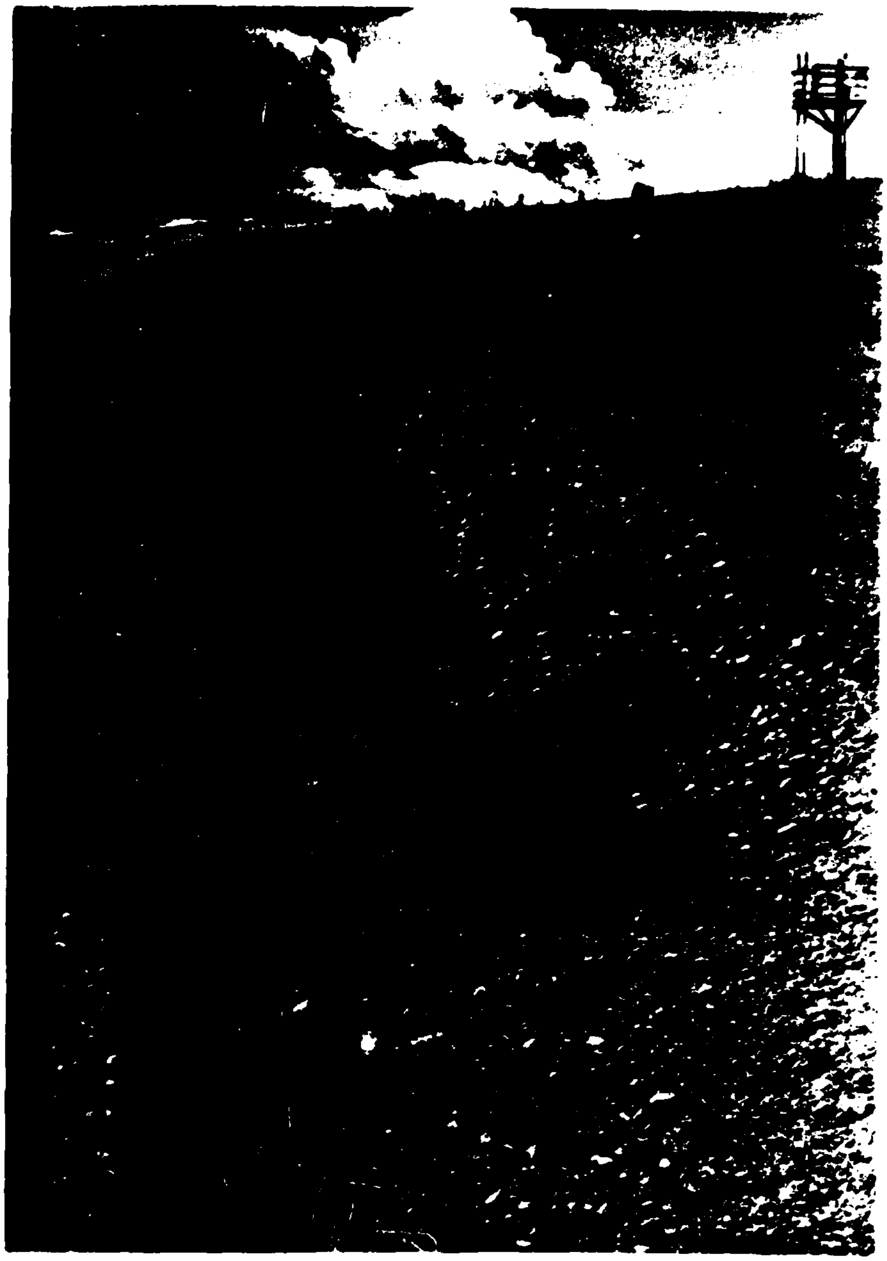


Fig. 1.3. Rill erosion at the TRESTLE site. East side of facility looking south. Although this is a natural drainage ditch alongside the access road, water is cutting deeper into the soil as it progresses down grade.

GULLY EROSION

Gullies, on the other hand, cannot be obliterated by normal tillage. Gullies are intermittent stream channels larger than rills. They carry water during and immediately after rains. The dynamics of gully formation are complex and not completely understood. Several statistical models for predicting gully growth and development have been proposed. Thompson (1964) chose linear advancement of gully heads, whereas Beer and Johnson (1963) selected changes in gully surface area as the dependent variable, in their respective models.

Four principal stages of gully development are generally recognized: downward cutting, headward erosion and enlargement, heading, and stabilization. Active gullies are those that continue to widen or enlarge; they may be recognized by the presence of bare soil exposed on their sides. Vegetation starts to grow in the gully channel during heading. The stabilization stage is characterized by an equilibrium gradient in the channel, stable gully sides, and vegetation sufficiently well established to protect the soil against any further erosion.

Gully-forming processes are diverse and pervasive. Studies by Piest (1975) in erodible loessial soils of western Iowa have shown that stream flow alone was not sufficient to cause gullying. Instead, mass-wasting of gully banks and headcuts were the prime processes. A whole array of processes can act individually or in concert to produce gullies, including:

1. Waterfall erosion at the gully head.
2. Piping and spring sapping at the head and sides.
3. Erosion and scour along the length of the gully bottom.
4. Raindrop splash and rilling on the sides.
5. Freezing and thaw erosion of the gully sides.
6. Mass wasting of gully sides and head.

Gullies may not be as significant as rills in terms of total quantities of soil eroded, but they are much more spectacular. Figures 1.4 and 1.5 illustrate examples of gully erosion at the KAFB Trestle site. Gullies are also much more difficult to control and arrest than rills. Effective gully control must stabilize both channel gradient and headcuts. Downcutting of gully bottoms leads to deepening and widening. Headcutting extends the channel into ungullied headwater areas, and increases the stream net and its density by developing tributaries.

STREAM CHANNEL EROSION

Stream channel erosion consists of soil removal from stream banks and sediment scour along the channel bottom. Where the erosion occurs is dependent upon the type of stream. "Youthful" or small streams generally exhibit bed erosion. "Mature" or large streams primarily exhibit bank erosion. In either case, there is usually

Fig. 1.4. Gulley erosion at the TRESTLE site. This is what remains of an open channel drainage chute designed to carry runoff from the northwest sector of the site. The access road is just above where the truck is parked. Directly across the road, there is evidence of ground water seepage; several rills, one about 2½ feet deep, have already formed. Unless something is done to arrest erosion, the road may eventually give way.

Fig. 1.5. Gulley erosion at the TRESTLE site.
See Fig. 1.4.

a balance between material eroded and material deposited along a particular reach of stream. Stream channel erosion should be considered separately from the rainfall-associated types of erosion previously discussed.

GROUNDWATER EROSION

Groundwater erosion is the removal of soil caused by ground water seepage or movement toward a free face. Such erosion is commonly referred to as piping. The phenomenon is also known as spring sapping—literally the detachment and movement of soil particles at the point of emergence of a spring or seep in the ground. Piping occurs when seepage forces exceed intergraunular stresses or forces of cohesion.

Pipes can form in the downstream side of earth dams (Sherard, 1972, and Carney, 1983), gully heads, stream banks, and slopes where water exits from the ground. Once a pipe or cavity forms, it enlarges quickly because flow lines are attracted to areas of lower flow resistance, and this in turn results in further concentration of flow lines or flow net density in a positive feedback cycle.

PREDICTING EROSION RATES

Predicting erosion rates or soil losses is not an exact science by any stretch of the imagination. The many variables involved make it an impossible task. However, a number of equations relating the factors controlling erosion have been proposed and are used to make such predictions. Although the equations are much better than an educated guess, bear in mind they are still, at best, approximations.

UNIVERSAL SOIL LOSS EQUATION

A semiempirical equation for predicting rainfall erosion was developed by the Agriculutral Research Service (Wischmeier and Smith, 1965). This equation was developed originally for predicting erosion losses from cropland east of the Rockies and is called the Universal Soil Loss Equation (USLE). The USLE was later modified and adapted to different regions of the U.S. (USDA Soil Conservation Service, 1972, 1977) and also for use at urban or highway construction sites (U. S. Environmental Protection Agency, 1973).

The USLE takes into account all the factors known to affect rainfall erosion--climate, soil, topography, and vegetation. It is based on a statistical analysis of erosion measured in the field on scores of test plots under natural and simulated rainfall. The annual soil loss from a site is predicted according to the following relationship:

$$X = RKSLCP \quad (1.2)$$

Where:

X = the computed soil loss in tons (dry weight) per acre from a given storm period.

R = the rainfall erosion index for the given storm period.

K = the soil erodibility factor.

L = the slope length factor.

S = the slope gradient factor.

C = cropping management (vegetation) factor.

P = erosion control practice factor.

In spite of limitations, the USLE provides a simple straightforward method of estimating soil losses and of evaluating the effectiveness of soil loss reduction measures. The USLE is particularly well suited for estimating rainfall erosion losses from construction sites. Since this is a prime concern to civil engineers, examination of the USLE will be from that perspective.

RAINFALL EROSION INDEX -R

The rainfall erosion index is also known as the rainfall factor. As previously noted, the single most important measure of the erosion producing power of a rainstorm is the product of the rainfall energy times the maximum 30-minute rainfall intensity or:

$$R = (0.01)(E)(I) \qquad (1.3)$$

Where: R = Rainfall erosion index

E = Total kinetic energy of a given storm in ft-tons per acre.

I = Maximum 30-minute rainfall in the area in inches per hour.

Since: $$E = 916 + 331 \log i \qquad (1.1)$$

Then: $$R = (916 + 331 \log i)(I)(.01)$$

$$R = I\,(9.16 + 3.31 \log i) \qquad (1.4)$$

The records of individual storms are summed over a given time to obtain cumulative R values for other periods of time (e.g., a month or a year). The annual R factors for approximately 2000 locations in the U.S. were summarized in the form of "iso erodent" maps by Wischmeier and Smith (1965). Annual R-factor values vary from a low of approximately 50 in the northern Great Plains to a high of 600 in the gulf coast region.

Studies by the USDA Soil Conservation Service (1972) have established a relationship between Type II, 2-year frequency, 6-hour duration rainfall and the average annual rainfall erosion index. This particular duration and frequency storm can be considered a typical "average" storm because it can be expected to occur 50 percent of the time; the 6-hour duration has been found by the Soil Conservation Service to be the most frequently occurring storm length. Type II refers to the rainfall characteristics within a specific region of the U.S. The continental U.S.

has been divided into two regions or zones, Type I and Type II. Type I rainfall is confined mainly to the Pacific Coast, North Cascades, and Central Sierra Nevada regions. Type II is the rest of the country. The relationship between the annual rainfall index and type is shown graphically in Fig. 1.6. The 2-year frequency, 6-hour duration rainfall depths for various parts of the U.S. as well as the zones for Type I and Type II rainfall can be obtained from a U.S. Weather Bureau map as shown in Fig. 1.7. With this information, the average annual erosion index can be determined from the curves in Fig. 1.6. If more precise estimates are required, the rainfall for a particular location can be determined from weather records published by the U.S. Weather Bureau.

In order to compare the effects of different erosion control measures at construction sites, the engineer may want to estimate potential soil loss values for an entire range of periods of times, ranging from individual storms to annually. Figs. 1.8 and 1.9 provide the rainfall erosion index for individual storms of different duration.

The following check list summarizes the procedures necessary to estimate the rainfall erosion index (R):

1. Locate the area under study in a rainfall atlas similar to Fig. 1.7.
2. Determine the value of the 2-year, 6-hour rainfall from the chart or atlas.

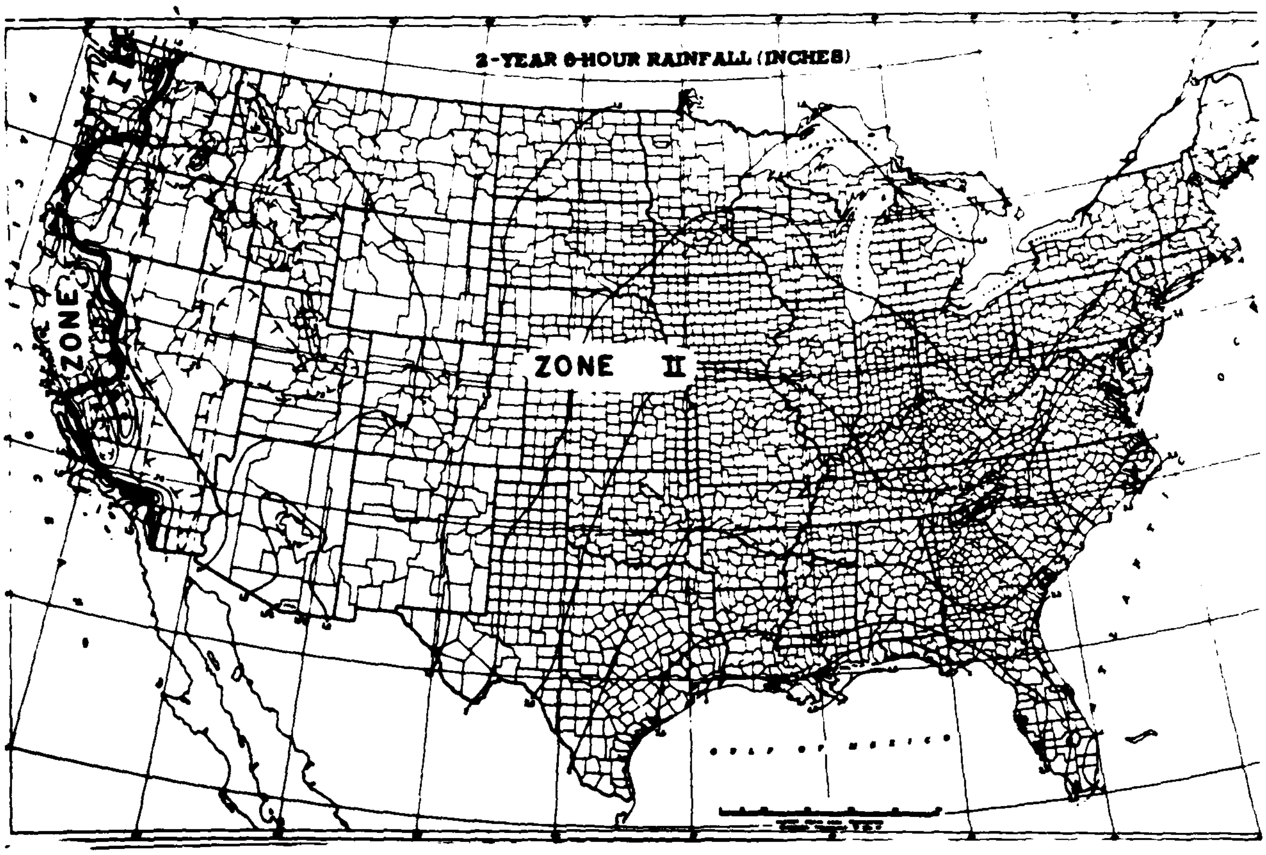


Fig. 1.7 Depths of the 2-year, 6-hour rainfall in various parts of the United States. Zones for Type I and Type II rainfall are also shown. *(Adapted from U. S. Weather Bureau, 1963)*

3. Check the zone (rainfall type) in which the area is located.
4. If for a "single storm" rainfall erosion index for a 2-year, 6-hour storm, use the graph in Fig. 1.8 or 1.9 to determine the erosion index, using the 6-hour duration.
5. If for an "average annual" rainfall erosion index, use Fig. 1.6 and the appropriate curve.

SOIL ERODIBILITY FACTOR - K

The susceptibility of a soil to erosion is known as its "erodibility." Some soils are inherently more erodible than others. In general, increasing the organic content and clay size fraction of a soil decreases erodibility. However, erodibility also depends on soil texture, gradation properties, natural moisture content, void ratio, PH, and composition or ionic strength of the eroding water. The dependence of soil erodibility is summed up in Table 1.3.

At present, a simple and universally accepted erodibility index for soils does not exist. Table 1.4 shows the relative erodibility of soils (from most erodible to least erodible) based on the Unified Soil Classification System and gradation and plasticity indices of remolded or disturbed samples.

From: Biotechnical slope protection and erosion control, Gray and Leiser, 1982.

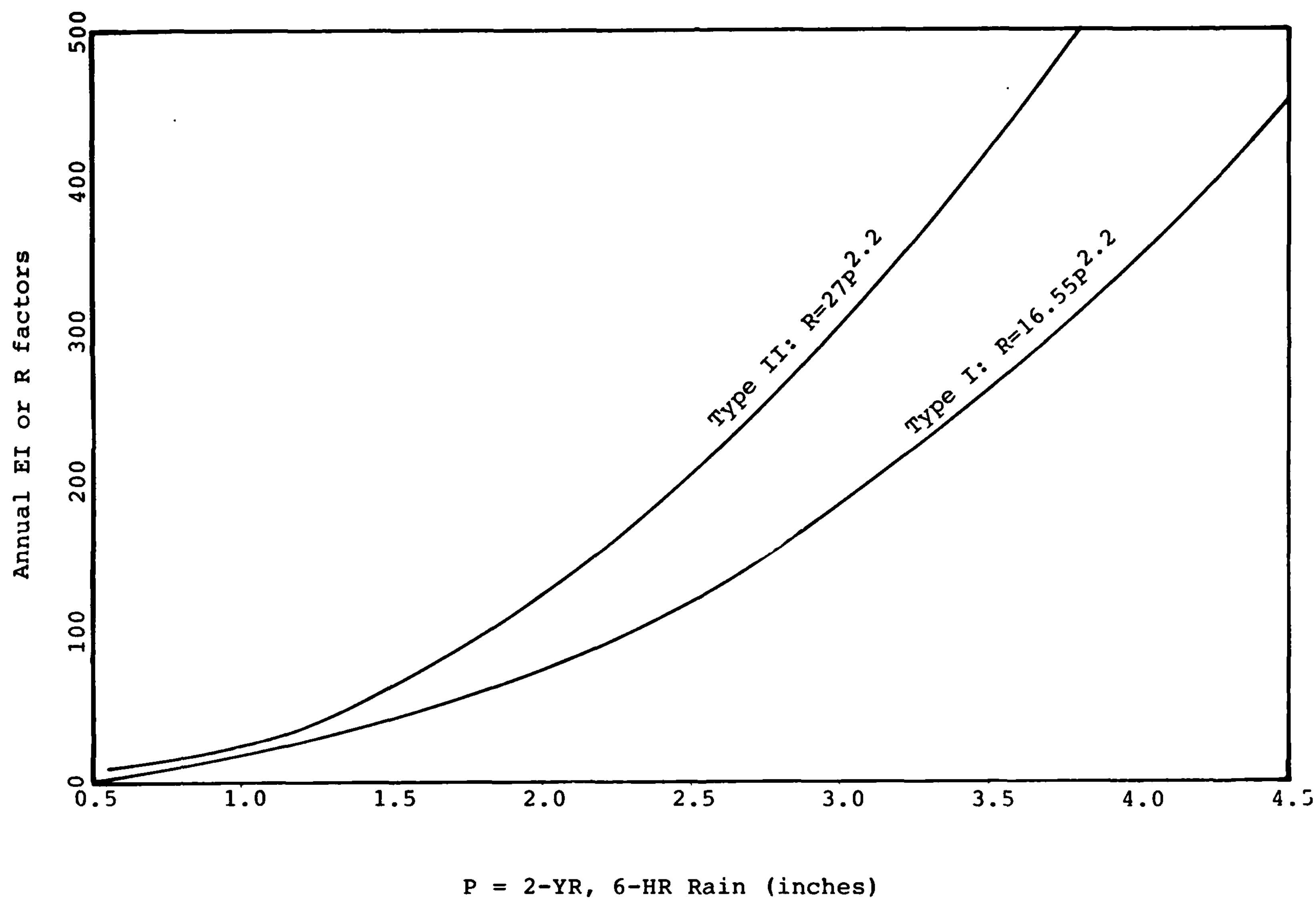


FIG. 1.6 Relation between annual R and 2-YR, 6-HR rainfall depth.

Table 1.3 SOIL ERODIBILITY FACTOR (K)

(From: Biotechnical Slope Protection and Erosion Control, Gray and Leiser, 1982)

- K is low in well-graded gravels.
- K is high in uniform silts and fine sands.
- K decreases with increasing clay and organic content.
- K decreases with low void ratios and high natural moisture content.
- K increases with increasing sodium absorption ratio and decreasing ionic strength of water.

The soil erodibility factor (K) represents the soil's *inherent susceptibility to erosion*; it is governed by textural and gradation properties of the soil. Erodibility factors for 23 bench mark soils, from which erosion has been experimentally measured since 1930, have been published by the Soil Conservation Service (Wischmeier and Smith, 1965). K-values that have been obtained experimentally range from 0.02 to 0.69.

Twelve K-value classes have been established by the Soil Conservation Service for ease of use. Soil series identified in Soil Conservation Service maps generally have a K-value assigned to them as part of the marginal information on each particular soil series.

Table 1.4 RELATIVE ERODIBILITY OF SOILS

From: Biotechnical Slope Protection and Erosion Control, Gray and Leiser, 1982.)

	Soil Group Symbol	Typical Names
Most Erodible	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
↓	SM	Silty sands and sand-silt mixtures.
	SC	Clayey sands, sand-clay mixtures.
	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
	OL	Organic silts and organic silty clays of low plasticity.
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
	CH	Inorganic clays of high plasticity, fat clays
	GM	Silty gravels, gravel-sand-silt mixtures.
	GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
Least Erodible	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.

NOTE: These soils could be divided into two large groups of 5 since the first 5 are significantly more erodible than the last 5 soil groups. Also, the comparisons fail to take into account the effects of soil structure, void ratio, and natural moisture content.

Wischmeier (1971) published a convenient nomograph based on easily measured soil properties which is valid for exposed subsoils at construction sites as well as farm lands. The nomograph is used to determine K-values; only fine soil parameters are required:

1. Percent silt and very fine sand (0.002-0.10 mm).
2. Percent sand (0.10-2.0 mm).
3. Percent organic matter.
4. Structure
5. Permeability.

The first three parameters will often suffice to provide a reasonable approximation of the erodibility which can be refined by including information on permeability and soil structure as indicated on the nomograph (Fig. 1.10).

SLOPE LENGTH AND STEEPNESS FACTORS - LS

The effects of slope length, L, and steepness, A, on soil loss were investigated separately but they are often combined in a single "topographic" factor, LS. This factor is the ratio of soil loss per unit area from a given site to that from a unit plot having a 9 percent slope and 72.6 ft. length. The combined LS factor can be computed from an empirical equation which is graphed in Fig. 1.11.

The topographic factor, LS, has been extended by the U.S. Soil Conservation Service (1972) to cover slope lengths up to 1600 ft. and for slope steepness up to 100 percent (1:1). Fig. 1.11 shows extension of the original chart beyond the 400-ft. length and 20 percent slope, the extent of physical data on which the USLE was based. Slopes commonly used along roads and highways have also been added to the chart shown in Fig. 1.11. These extensions and additions shown as dashed lines are extrapolations beyond confirmed data; therefore, use only as speculative estimates.

From the previous discussion, we know the topographic variables influencing rainfall erosion are slope angle, length of slope, and size and shape of watershed. The influence or importance of length tends to increase as slopes become steeper. For example, doubling the slope length from 100 to 200 ft. only increases soil losses by 29 percent in a 6 percent slope, whereas the same doubling of slope length in a 20 percent slope will result in a 49 percent increase in soil loss (Refer to Fig. 1.11). This is one of the reasons for benching or terracing and contour wattling long, steep slopes.

CROPPING MANAGEMENT (VEGETATION) FACTOR - C

Vegetation plays an extremely important role in controlling rainfall erosion. Removal or stripping of vegetation, whether by man or nature, often results in accelerated erosion. The cropping management factor C, is defined as the ratio of soil loss from land cropped under specific conditions as against the corresponding loss from tilled, continuous, fallow (bare) land. In physical terms it describes the protective effects of vegetation against erosion.

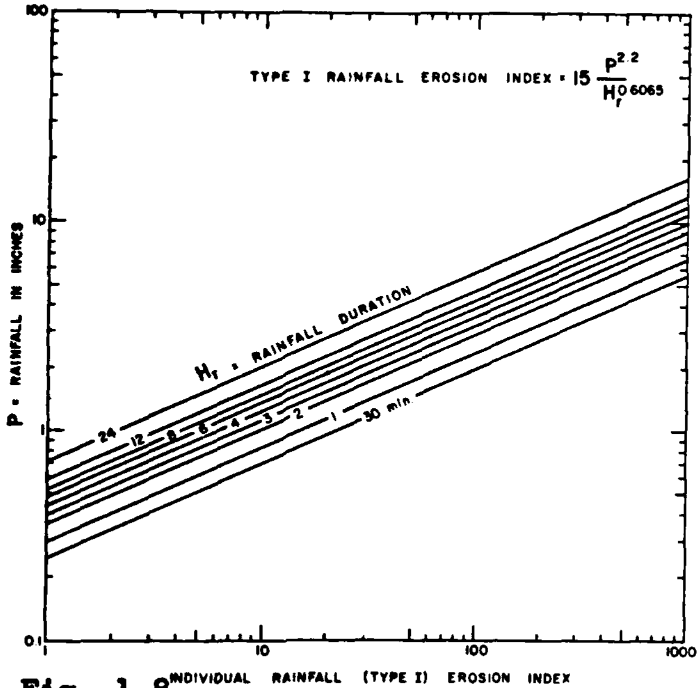


Fig. 1.8

Relation between depths and duration of Type I rainfall and single storm erosion index; 2-year frequency of occurrence.

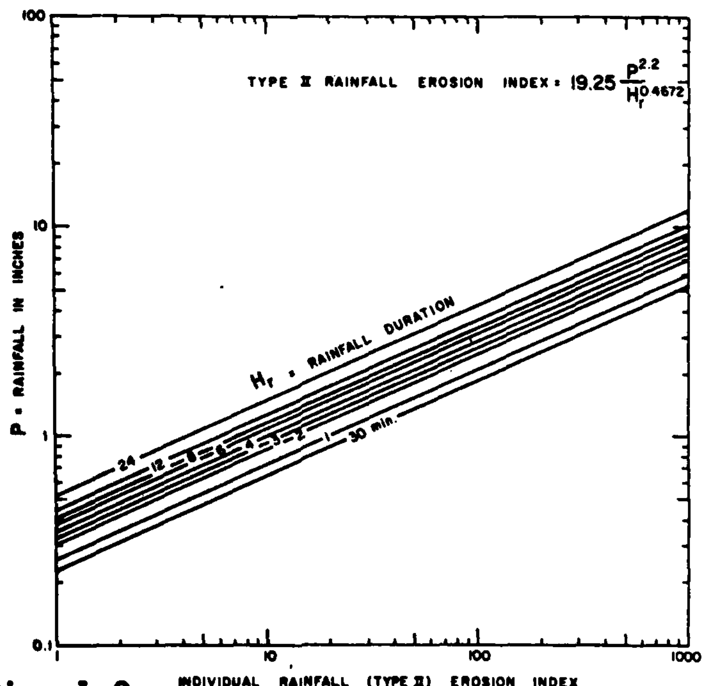


Fig. 1.9

Relation between depths and duration of Type II rainfall and single storm erosion index; 2-year frequency of occurrence.

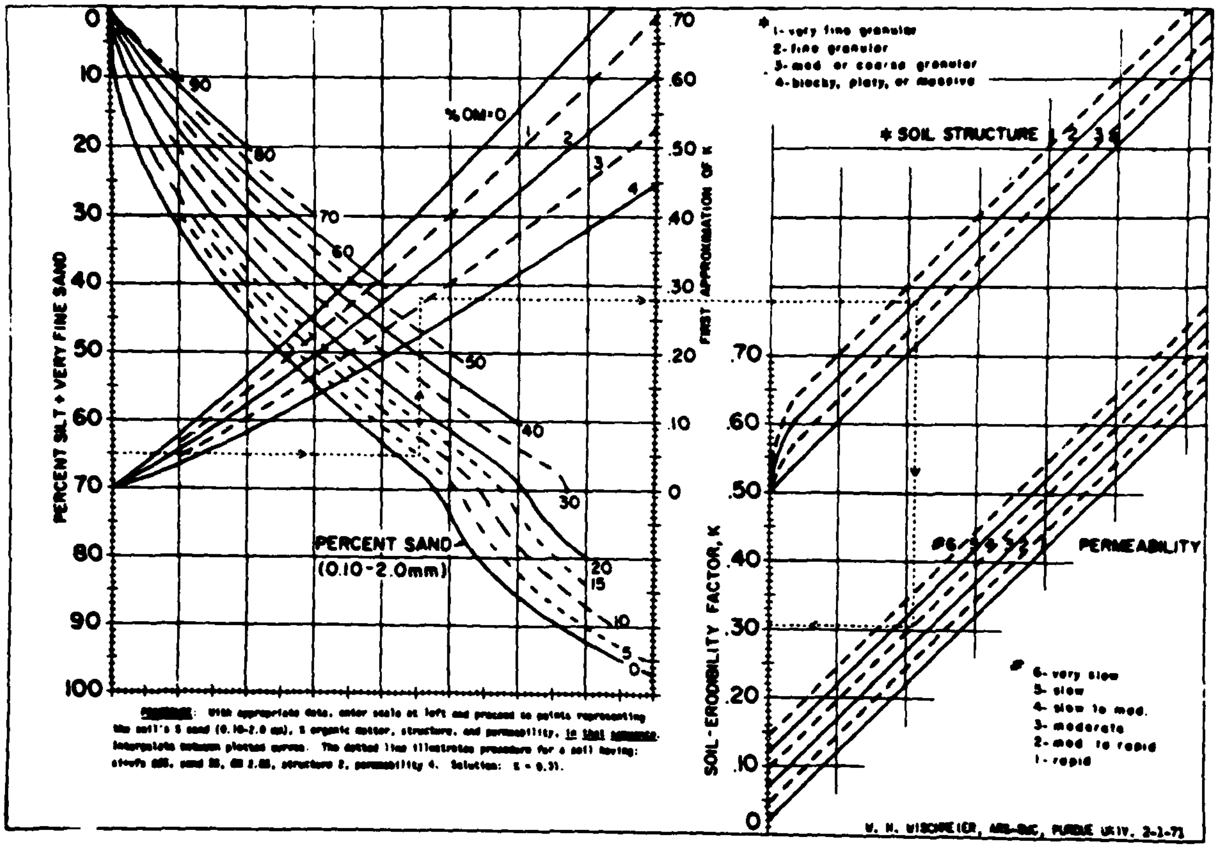


Fig. 1.10 Soil erodibility nomograph for determining *K*-values. (*From Wischmeier et al., 1971*). Reprinted from the *Jour. of Soil and Water Conservation,* Vol. 26, No. 5. Copyright © Soil Conservation Society of America.

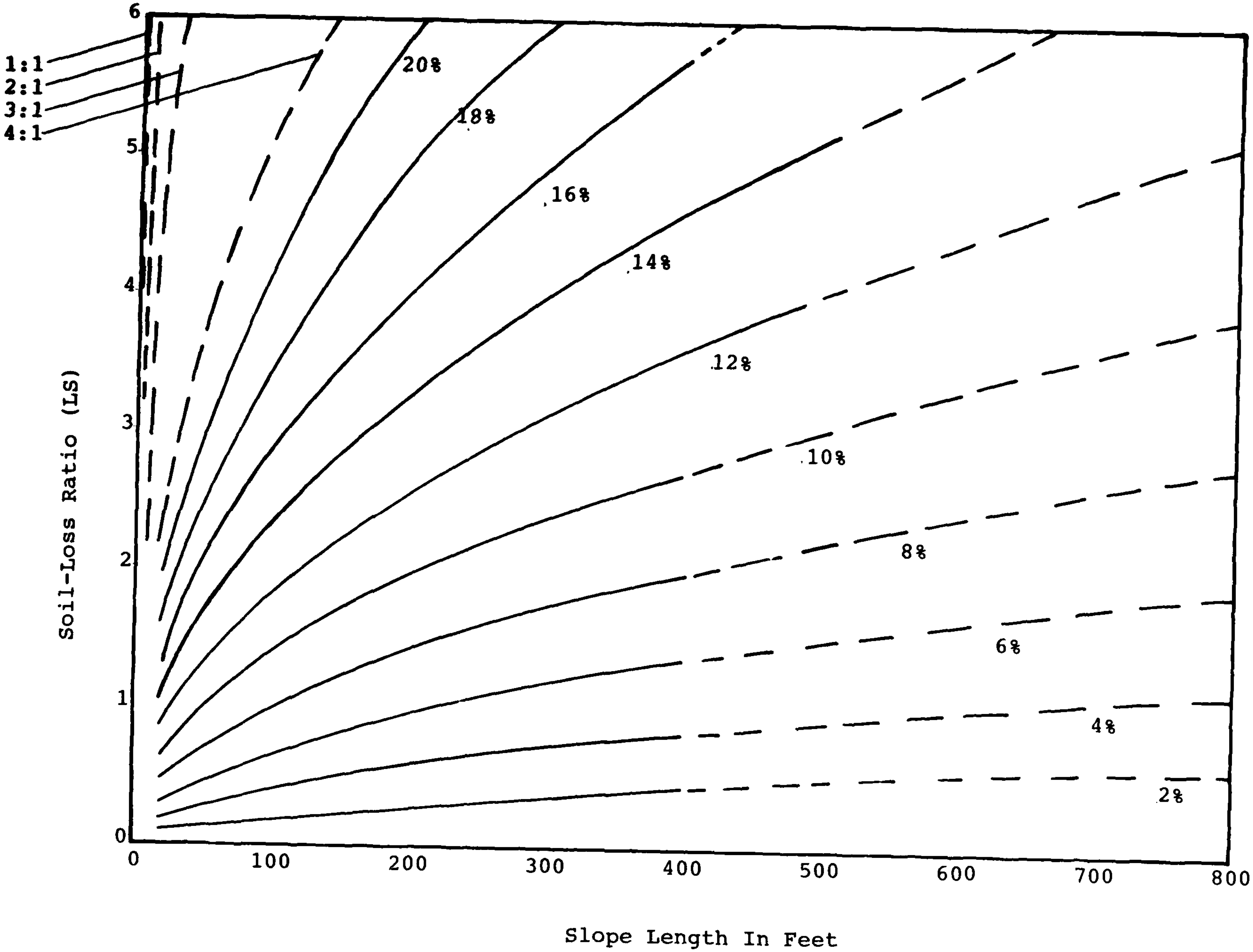


FIG. 1.11 Chart for determination of topographic factors, LS.

Vegetation or cropping management affects erosion through three separate and distinct but interrelated variables: canopy cover (weeds, brush, trees, etc.), vegetative cover in direct contact with the soil, and crop residue at or beneath the surface. The effects of these variables could be considered separately but for practical purposes are usually represented by a single value of the C factor.

For completely bare ground the C factor is unity. Factor C values for pasture, range, woodland, and idle land are tabulated in Tables 1.5 and 1.6. The influence of canopy, cover type, and percent ground cover are clearly indicated in Table 1.5. Information in Table 1.5 reveals the benefit of vegetation or plant cover for reducing erosion. Factor C values range as low as 0.003 for well-established plant cover. This corresponds to almost a thousand-fold reduction in erosion losses over the continuous-fallow or bare-ground case. Few other variables or factors in cropping management produces such dramatic reductions in erosion losses as this one. Mulching is also considered a form of cropping management. The influence of mulching with various types of organic mulches such as straw, hay, woodchips, etc. will be discussed with some detail in Chapter 2 of this report.

EROSION CONTROL PRACTICE FACTOR - P

The erosion control practice factor, P, is a parameter representing the reduction of soil loss resulting from soil conservation measures such as contour tillage, contour strip cropping, terracing, and stabilized waterways. Factor

Table 1.5 C factors for pasture, rangeland, and idle land.[1]

Vegetal Canopy			Cover That Contacts the Surface					
			Percent Ground cover					
Type and height of raised canopy[2]	Canopy cover[3] %	Type[4]	0	20	40	60	80	95-100
Column no.:	2	3	4	5	6	7	8	9
No appreciable canopy		G	.45	.20	.10	.042	.013	.003
		W	.45	.24	.15	.090	.043	.011
Canopy of tall weeds	25	G	.36	.17	.09	.038	.012	.003
or short brush		W	.36	.20	.13	.082	.041	.011
(0.5 m fall ht.)	50	G	.26	.13	.07	.035	.012	.003
		W	.26	.16	.11	.075	.039	.011
	75	G	.17	.10	.06	.031	.011	.003
		W	.17	.12	.09	.067	.038	.011
Appreciable brush	25	G	.40	.18	.09	.040	.013	.003
or bushes		W	.40	.22	.14	.085	.042	.011
(2 m fall ht.)	50	G	.34	.16	.085	.038	.012	.003
		W	.34	.19	.13	.081	.041	.011
	75	G	.28	.14	.08	.036	.012	.003
		W	.28	.17	.12	.077	.040	.011
Trees but no appre-	25	G	.42	.19	.10	.041	.013	.003
ciable low brush		W	.42	.23	.14	.087	.042	.011
(4 m fall ht.)	50	G	.39	.18	.09	.040	.013	.003
		W	.39	.21	.14	.085	.042	.011
	75	G	.36	.17	.09	.039	.012	.003
		W	.36	.20	.13	.083	.041	.011

[1]All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists.

[2]Average fall height of waterdrops from canopy to soil surface: m = meters.

[3]Portion of total-area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[4]G: Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 2 inches deep.

W: Cover at surface is mostly broadleaf herbaceous plants (as weeds) with little lateral-root network near the surface, and/or undecayed residue.

Source: USDA Soil Conservation Service, 1978

Table 1.6 C factors for woodland.

Tree canopy[1] % of area	Forest litter[2] % of area	Undergrowth[3]	C factor
100-75	100-90	Managed[4]	.001
		Unmanaged[4]	.003-.011
70-40	85-75	Managed	.002-.004
		Unmanaged	.01-.04
35-20	70-40	Managed	.003-.009
		Unmanaged	.02-.09[5]

[1]When tree canopy is less than 20%, the area will be considered as grassland, or cropland for estimating soil loss. See Table 2.5.

[2]Forest litter is assumed to be at least 2 inches deep over the percent ground surface area covered.

[3]Undergrowth is defined as shrubs, weeds, grasses, vines, etc., on the surface area not protected by forest litter. Usually found under canopy openings.

[4]Managed—grazing and fires are controlled. Unmanaged—stands are overgrazed or subjected to repeated burning.

[5]For unmanaged woodland with litter cover of less than 75%, C values should be derived by taking 0.7 of the appropriate values in Table 2.5. The factor of 0.7 adjusts for the much higher soil organic matter on permanent woodland.

(From USDA Soil Conservation Service, 1978)

P values for standard erosion control practices are tabulated in Table 1.7.

Table 1.7 P FACTORS FOR STANDARD EROSION CONTROL PRACTICES

Slope %	Up and Down Hill	Cross Slope Farming Without Strips	Contour Tillage	Cross Slope Farming With Strips	Contour Strip-cropping
2.0-7	1.0	.75	.50	.38	.25
7.1-12	1.0	.80	.60	.45	.30
12.1-18	1.0	.90	.80	.60	.40
18.1-24	1.0	.95	.90	.67	.45

(From: USDA Soil Conservation Service, 1978).

Note from Table 1.7, values of P range from 0.95 for contouring on steep slopes to 0.25 for contour strip cropping on gentle slopes. Terracing effectively reduces the length of slope from that of the entire site to the horizontal distance between terraces. The methods of determining P for a given conservation practice and, alternatively, the selection of a conservation practice, using the Universal Soil Loss Equation, have been described by Wischmeier and Smith (1965). The P factor is the last item required to compute soil loss due to rainfall erosion for a given site using the USLE. In chapter 4 of this report, we will use the USLE to estimate soil losses at the KAFB Trestle site.

ICE EROSION

As previously discussed, we know the primary elements of erosion are wind, water, and ice; we just finished exploring the effects of water erosion at some length; but, what effect do ice and wind have? Fortunately, at KAFB, New Mexico, ice presents very little impact where erosion is concerned. However, ice has played a tremendous role in the evolution of the earth's surface as we now see it. Further, in certain parts of the world, ice is still a major element of the erosion process. Low temperatures and precipitation of snow contribute to the formation of glaciers and snow fields in cold polar regions and in high mountains. As snow accumulates, it becomes compacted, gradually changing from snowflakes to granular to solid, massive ice. As glacial ice moves down valleys, it changes the topography by eroding rocks and transporting the debris to where melting takes place. Glaciers of continental size, like those in Greenland and Antarctica, produce a variety of erosional and sedimentary landforms. The glacial landforms of the recent past are evidence of the Pleistocene glacial epoch, during which huge areas of North American and Eurasia were covered by ice. Advances and retreats of the ice fronts caused large fluctuations in sea level, alternately flooding and exposing shallow ocean margins of the continents. The causes of past ice ages and any predictions of future ones are equally uncertain but I think we can be "reasonably" sure that the KAFB Trestle site will not be subjected to ice erosion at least for the life of the facility. Consequently, this will conclude my discussion of the ice aspect of the erosion process.

WIND EROSION

Give the wind enough sand-sized rock or mineral fragments of any variety and it will blow them to form dunes. The White Sands National Monument area in New Mexico is largely covered by drops of gypsum grains eroded from evaporite bedrock in a dry climate. Wind, though less powerful than water currents, can erode sand and silt effectively, particularly in arid regions. All of us have been caught, at one time or another, in a high wind so strong that it could have blown us over if we had not leaned into it or held on to something solid. A wind strong enough to move a person is easily capable of blowing sand grains into the air, as anyone who has ever been in a sandstorm can attest.

Wind is a turbulent stream of air. Its ability to erode, transport, and deposit sediment is much like that of water—the same general laws of fluid motion applies.

There are differences, of course, and they are traceable to two properties of wind: its low density and the fact that its flow is not restricted to channels. The low density of air limits its ability to move larger particles, and the fact that its flow is unrestricted enables it to spread over wide areas and high into the atmosphere. In contrast to rivers, whose discharge is dependent upon rainfall, it is the lack of rain that allows wind to work most effectively. Drought destroys the vegetative protection of soil against the driving forces of the wind. The soil structure formed by roots of the previous vegetation deteriorates and the dry soil is vulnerable to transportation by wind.

Until about 1940, research on wind erosion was almost nonexistent. Free studied the problem of soil movement by wind as early as 1911. He introduced the term "saltation" to denote the movement of soil by a series of short bounces along the surface of the ground, and "suspension movement" to designate the particles carried by the wind, more or less parallel to the soil surface. Bagnold (1941) made comprehensive studies on the movement of sand by wind and suggested the term "surface creep" to explain the rolling or sliding of particles along the surface through the impact of wind. Chepil and Milne (1939) were among the first to give special attention to the dynamics of wind erosion. They used both field and wind tunnel experiments to establish basic principles. Chepil and Woodruff (1963) summarized existing research, analyzed the mechanics of the wind erosion process, and discussed methods of control.

Wind erosion is controlled by the same basic factors that control rainfall erosion: climate (temperature, rainfall distribution, wind velocity, and direction); soil (texture, particle size, moisture content, and surface roughness); vegetation (type, height and density of cover and, seasonal distribution). Unlike rainfall erosion, topographic parameters such as length and steepness of slope are relatively unimportant in wind erosion. On the other hand, surface roughness and the presence of low barriers that act as wind breaks and sediment traps can be important (Gray, 1982).

Only relatively dry soils are susceptible to wind erosion. The climatic factors that must affect soil moisture are amount and distribution of rainfall, temperature, and humidity. The most important characteristics of the wind are its velocity, duration, direction and degree of turbulence. Winds are highly variable in direction and power. Though the average wind on a breezy day might be about 10 mph, gusts up to twice that speed occur intermittently and may spawn momentary blowing up of dust or sand clouds. Most people in temperate climates are used to winds that come mainly from one direction, the prevailing westerlies. Those in the tropics are familiar with the equatorial easterlies. Yet within these belts the winds will be variable in direction and power, depending on the movement of air masses and storms. Many of us live with wind fluctuations during the day, such as the sea breeze that blows during warm summer days and dies down in the evening, or the daytime valley breeze and nighttime mountain breeze in high-relief terrain. More constant are the steady, strong winds that blow for days without letup, like the dry Chinook winds of the eastern slopes of the Rocky Mountains (Press, 1972).

The distribution and intensity of winds in combination with climate has much to do with the location of wind erosion and wind blown deposits on earth. Most of the geologic work of the wind is done by the moderately infrequent strong winds of long duration, just as the major part of a river's geological work is done by floods. Every day mild winds and the rare tornado, whose winds may exceed 100 mph but which covers only a narrow strip of earth for a short time, are responsible for relatively little geologic change. Hurricanes and typhoons are important agents because of their frequency in certain regions, but, because of the rain they bring, they do their work by causing floods and stirring up waves rather than by blowing sand or dust; their rain washes dust particles out of the air and wets down the ground, which prevents pickup of more particles (Press, 1978). Again, this points to dryness of the wind as a crucial prerequisite for erosion. Winds need chemical and mechanical weathering coupled with dryness to be effective. Wet materials are cohesive; the water binds the particles together enough to resist the wind action. Alone, winds can do little to erode most solid rock exposed at the surface; but once given some fragmentation of mineral particles, the wind can act.

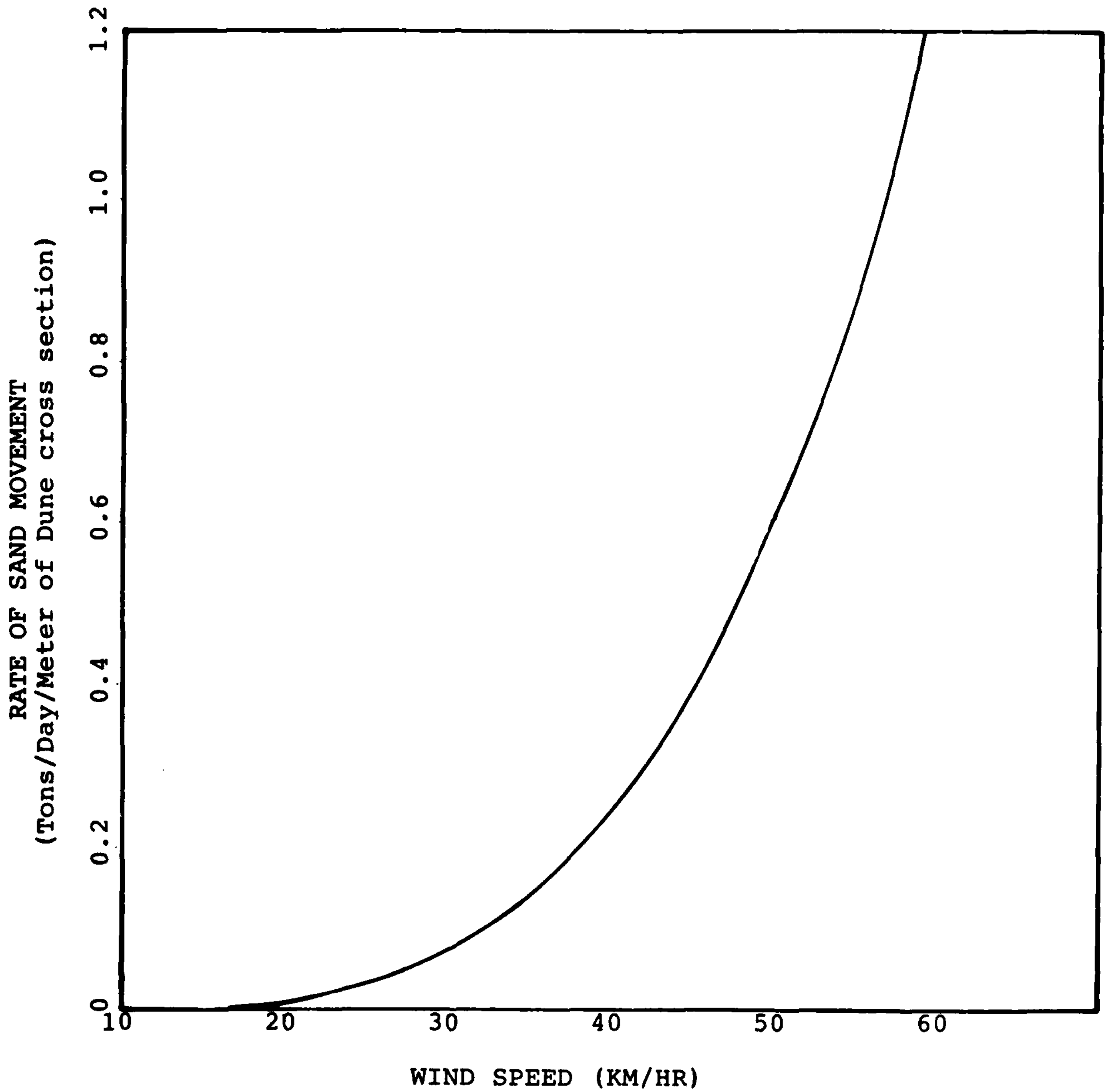
Sand grains carried by the wind may be of almost any mineral. But, most are quartz, which reflects the dominance of that mineral in most sands and sandstones. The grains are typically frosted in appearance, like the ground-glass focusing screens of many cameras. Much of this frosting results from the long continued action of dew. This moisture, even the tiny amount found in arid climates, is enough to dissolve away little pits and holes on the grain, creating a matte surface. This frosting, which is limited to the grains lying on the surface of the ground, is quickly smoothed and polished when the grains are blown into a river and transported by water. This experimental and observational evidence, which includes studies showing that wind alone could not produce the frosted surface, contradicts a long held inference that the matte appearance was the result of sand blasting, which is known to produce frosting on larger glass objects (Press, 1972).

Wind erosion consists of three distinct phases: initiation of movement (detachment), transportation of soil particles either along the ground or in the air, and the deposition of soil at a new location. Soil movement is initiated as a result of wind turbulence and velocity. The wind exerts the same kind of force on particles on the land surface as a river current exerts on its bed, the turbulence and forward motion combining to lift particles up into the windstream at least temporarily. Saltation, the bounding and jumping movement of grains mentioned earlier, operates in the air in the same way that it does under water, but in the air it is much more effective. Partly as a result of the lower frictional and retarding force of the air, saltating grains will frequently rise to heights of 20 inches over a sand bed and up to 6.5 feet over a pebbly surface. In a strong wind, there can be so many saltating grains that there will be a cloudy layer near the ground dense with blowing particles and capable of sand blasting any object in its path. The saltating grains falling back to the ground hit the surface with all the force of the wind plus gravity, hardly cushioned at all by the air. This strong impact induces saltation of some of the grains on the surface as they are struck. The struck grains too heavy to saltate tend to move enough to cause a general forward creep of sand particles along the surface as the rain of saltating grains falls on it. A sand grain striking the surface can move another grain up to six times its own diameter. Because saltation blows smaller grains more quickly and surface creep moves larger particles more slowly, the two will sometimes separate: the fine sand blows away leaving behind a pavement of coarser sand and gravel. The fine sand, up to 0.1 mm in diameter, accumulates in dunes and sheets downwind (Bauer, 1972).

The wind velocity required to start soil movement increases as the weight of particles increase. For many soils, this velocity is about 13 mph at a height of one foot above the ground. Of course, the velocity required to sustain movement is less than that required to initiate it. The amount of sand that can be moved by winds of various strengths is shown in Figure 1.13.

As can be seen from Figure 1.13, one-half ton of sand per day can be moved over a meter-wide strip of ground

FIG. 1.13 The amount of sand moved across each meter of width of a dune cross section in relation to wind speed. High speed winds blowing several days can move enormous quantities of sand.



[After R.A. Bagnold, "The physics of blown sand and desert dunes", London, Methuen, 1941] by a strong wind of 48 km/hr (30 mph). As wind increases to gale force, about 80 km/hr (50 mph), the rate of sand movement increases more rapidly. No wonder that a whole house can be buried in a long sand storm driven by strong winds.

Laboratory studies by Chepil (1945) established that soil particles are transported by wind in the manner shown in Table 1.8.

Table 1.8 MOVEMENT OF SOIL PARTICLES BY WIND (After Chepil, 1945)

Soil Type	Mechanics of Movement: Suspension	Saltation (Skipping & Bouncing)	Surface Creep (Rolling & Sliding)
	(percent)	(percent)	(percent)
Sceptre heavy clay	3	72	25
Haverhill loam	38	55	7
Hatton fine sandy loam	33	54	13
Fine driven sand	17	68	15
Size of particles mm	0.0-0.1	0.1-0.5	0.5-1.0

The major portion of soil particles transported by wind occurs near the ground surface at heights under three feet. Approximately 62-97 percent of the total wind-eroded soil is transported in this zone near the surface, a fact that suggests the utility of installing relatively low barriers or wind breaks to filter and impede the movement of wind borne soil. Vegetation partly serves this purpose in addition to its other control functions such as increasing surface roughness, slowing and deflecting the wind, and binding soil particles together.

As mentioned earlier, the severity of wind erosion increases with periods of drought and decreases with favorable moisture conditions. This is associated with changes in the protective influence of vegetative cover as well as the direct effect of soil moisture on decreasing the erodibility of the particles. Moisture films between individual particles provide the cohesive forces to hold them together. Wind velocities must create a force in excessof the film forces in order to cause soil movement. There are very few winds that have sufficient velocities to overcome the cohesive forces of moisture film (Chepil, 1956).

Soil structure effects on wind erosion are manifested primarily through the size and stability of the aggregates and clods. For example, high sand percentages do not form clods and generally undergo high erodibility. Silt and clay, on the other hand, seldom are found as primary particles since they serve as binding agents in the formation of nonerodible clods.

Wind tunnel experiments have shown that few units greater than 0.84 mm in actual diameter are moved by most erosive winds. Although the size of water-stable aggregates may be as small as 0.02 mm or less, those soils with a high percentage of aggregates greater than 1.0 mm offer considerable resistance to wind erosion. The smaller aggregates usually form larger structural units called clods. The amount of clods that are produced is highly correlated with the percentages of water-stable aggregates less than 0.02 mm and greater than 0.84 mm in diameter (Chepil, 1953). These larger particles tend to shield the erodible particles from the wind.

The structural units of the soil may be broken down by abrasion from wind-driven material (sand blasting), tillage operations, raindrops, alternate freezing and thawing, or wetting and drying. The first wind erosion on a field usually takes place after a surface crust has been formed by the impact of raindrops. This thin crust originally offers considerable resistance to wind and a higher drag velocity is required to initiate soil movement. The abrasive action of the first eroded particles cuts through the crust and exposes more erodible particles. Even nonerodible clods on the surface are disintegrated (weathered) by these abrasive impacts. The amount of abrasion varies directly with the square of wind velocity and inversely with the modulus of rupture (Chepil and Woodruff, 1963). The modulus of rupture is an index of the cohesive forces in the soil.

Tillage of dry soils tends to break up the clods to increase soil erodibility. If tillage brings subsurface clods to the surface, there will be a decrease in wind erosion.

Alternate wetting and drying and freezing and thawing also tends to break up clods reducing the mechanical stability of the surface clods by producing smaller granules susceptible to wind erosion. For example, freezing and thawing moist soils during the winter increases the soil's erodibility in the spring.

The surface conditions that affect wind erosion are the surface roughness, the degree of protection by surface cover, and the sheltering of surfaces from direct wind impact. The rate of erosion decreases with increasing surface roughness because of the diminishing wind velocity that hits the ground. Surface roughness can be produced through tillage operations that form ridges and furrows or that bring clods to the surface. These roughness features are effective only if they consist of nonerodible structural units. For example, a ridge of sand would soon be moved and flattened by the wind (Bauer, 1972).

Vegetation not only adds to surface roughness but also provides cover for the soil surface. Naturally, tall vegetation increases the roughness factor more than short plants. Plants that are flattened by the wind have a lower total surface roughness than the sturdier, more erect ones. Vegetation that is thick, such as grasses and small grain stubble, provide greater surface cover than coarse stubble such as corn or cotton.

Wind erosion control measures are based upon protecting the erodible soil fractions from the major erosive impacts of the wind and trapping the eroded particles either among the surface roughness barriers or on the leeward side of these barriers. Live vegetation or residues from previous plant cover constitute the major control effort because of their effectiveness, permanence, and economy.

Vegetative cover has proved to be the most effective and economical wind erosion control measure. It was only after the natural vegetation on the land was destroyed that wind erosion became a problem. The stabilization of soils under agricultural operations practically demands that vegetation serve as the key protective factor against wind erosion. The value of vegetation depends upon the density of the cover and the resistance to decomposition of the plant residue left on the surface. As previously mentioned, established grasses are the most effective for controlling wind erosion and row crops the least. Similarly, wheat residues give greater control than sorghum stubble. Chepil and Woodruff (1963) reported that wind erosion of 500 pounds per acre from standing wheat stubble was only 17.5 percent of that from fallow ground; the corresponding losses under the same weight of flat straw were 53 percent. This difference was due primarily to the greater surface sorghum of the erect stubble. The corresponding values for sorghum residues were about 81 and 90 percent, respectively. The differences between the sorghum and wheat residues were related to the higher density of the wheat whereas it took one ton of erect wheat stubble to reduce soil movement to a trace, three tons of standing sorghum residues were required. Stubble mulching and minimum tillage can be used to produce crops and keep the residues on the surface. Such practices can reduce the direct impact of the wind on the surface to almost zero in some cases (Bauer, 1972).

As previously mentioned, soil tillage may have harmful or beneficial effect on wind erosion. Tillage of dry fallow fields in semi-arid regions to control weeds breaks down soil structure and favors increased soil movement. Tillage that produces a rough, cloddy surface increases the surface roughness and the amount of nonerodible fractions on the surface. These effects tend to decrease wind erosion. Moldboard plows, listers and chisel cultivators are effective tools for generating rough, cloddy, surfaces. Deep plowing that brings clay subsoil to the surface will produce clods that are highly resistant to wind impact. All these tillage operations, just as vegetative barriers, should be perpendicular to the prevailing wind direction when possible. Any tillage operation should be considered a temporary, emergency measure and must be supplemented with other preventive practices if wind erosion is to be controlled (Bauer, 1972).

WIND EROSION EQUATION

Wind erosion soil loss from a site can be estimated using an equation similar to the Universal Soil Loss Equation discussed earlier. The wind erosion equation is an established method for predicting gross erosion from open fields. (Chepil and Woodruff, 1963; Woodruff and Siddoway, 1965; Skidmore and Woodruff, 1968):

$$E = ICKLV \tag{1.5}$$

Where E = annual soil loss in tons per acre

I = soil erodibility index

C = local wind climatic factor

K = soil surface roughness factor

L = field length factor

V = vegetative cover factor

The equation can be used to estimate the potential amount of wind erosion for a given open field under local climatic conditions and it can serve as a guide to reduce potential wind erosion to a minimum. Its utility for predicting wind erosion losses from steep slopes of limited extent such as highway cuts and fills is less certain. Therefore, the equation should be used with less assurance in these situations.

SOIL ERODIBILITY INDEX - I

The soil erodibility index (ton/acre/yr) is determined from the percentage of nonerodible soil fractions greater than 0.84 mm diameter as measured by dry sieve analysis. It depends on the soil's ability to clod. Although usually determined from sieve data, it's possible to use generalized relationships between erodibility and texture class. The Soil Conservation Service has established a series of wind erodibility ratings for all important texture classes based on the percentage of fine material present, percentage composition of clays, and whether or not the soil is calcareous.

CLIMATIC FACTOR - C

The local wind erosion climatic factor is measured as a percent and varies directly with the cube of the wind velocity and inversely with the cube of the soil moisture content. It can be calculated or read from maps prepared by the Agricultural Research Service (Skidmore and Woodruff, 1968).

SURFACE ROUGHNESS FACTOR - K

The soil surface roughness factor measured in inches is equal to the average height of the clods or ridges constituting the surface. Surface roughness inhibits erosion by absorbing or deflecting wind energy and by trapping some potentially abrasive materials. In many studies the factor is rated either 1.0 for smooth ground or 0.5 for rough ground, with 1.0 being used when no direct information is available.

FIELD LENGTH FACTOR - L

The field length factor measured in feet is the equivalent unsheltered field width across the field along the prevailing wind direction. It is determined by graphical procedures and relates to field or area size, windrose characteristics, and the presence or absence of a wind break (Skidmore and Woodruff, 1968). Use of the factor is based on observations that soil flow is zero at the upward side of a bare field and increases across the field, along the wind direction, until it reaches a maximum value equal to the product of I, C, and K. If the field is too small for the maximum to be reached, then the calculated erosion rate must be reduced accordingly.

VEGETATIVE COVER FACTOR - V

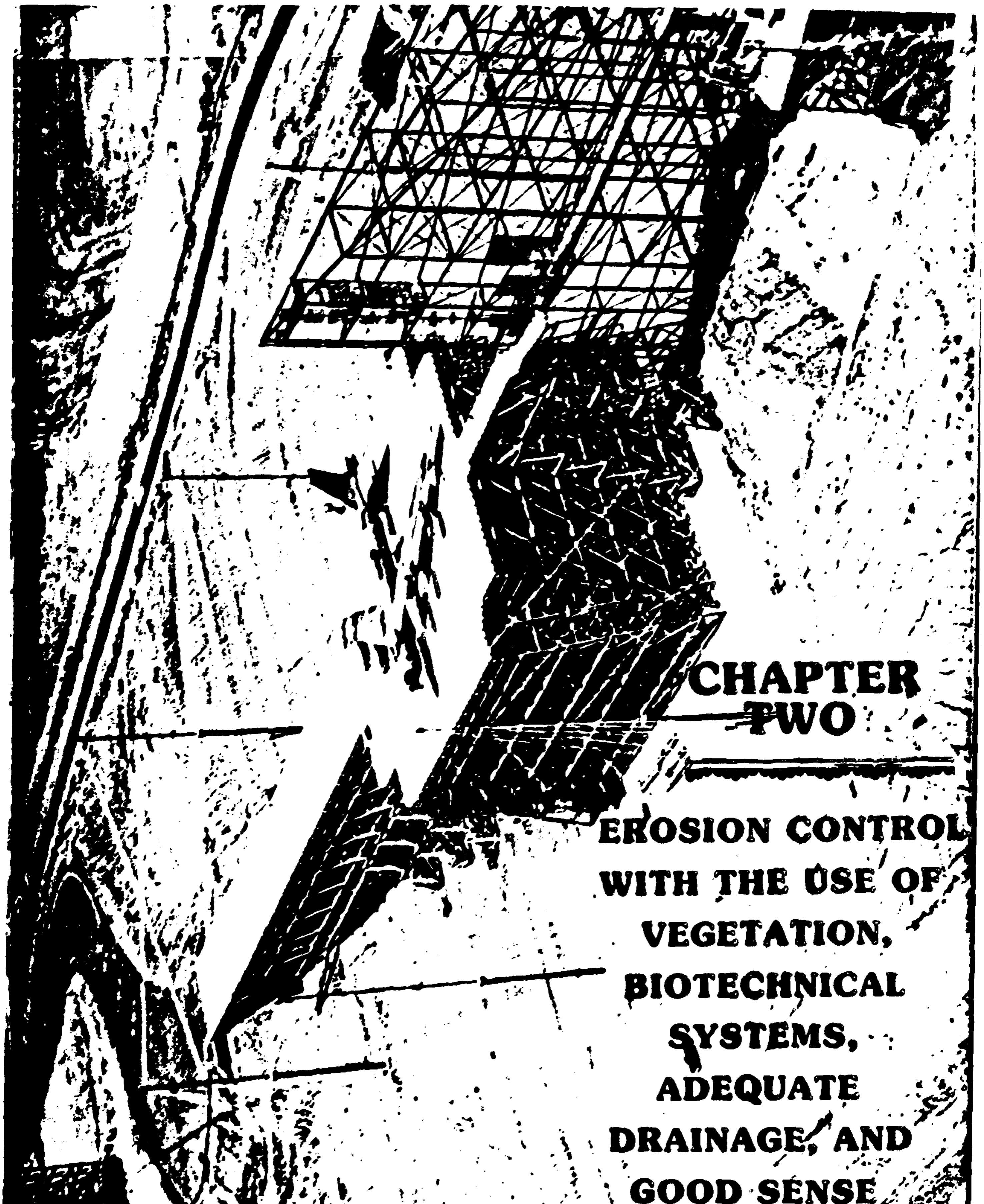
The vegetative cover factor is the equivalent quantity of vegetative cover which includes the quantity of vegetation above ground (pounds/acre), the kind of cover as experienced in its total cross-sectional area (obtainable from tables), and the orientation of the cover which includes the surface roughness factor (obtainable from charts). The factor reflects reduction in gross erosion resulting from protective effects of good cover. The effects of land clearing for construction, roads, and so on, potentially have a greater impact in humid regions than the same clearing might in arid areas. This is so because arid regions do not support as dense a plant cover. Therefore, arid regions have a higher baseline erosion rate.

SUMMARY

This chapter pointed out that with diminishing land resources for engineering work there is a continually growing concern to preserve our properties and our lands and to prevent the soil from eroding away. In order to prevent erosion, we must have some understanding of the erosion process. Erosion may be viewed as starting with the detachment of soil particles by the impact of raindrops. The kinetic energy of the drops can splash soil particles into the air and down slope. If overland flow occurs, the falling particles will be entrained in the flowing water and moved even farther down slope. Overland flow is predominantly laminar and cannot detach soil particles from the soil mass, but it can move loose particles already on the soil surface. The splash and overland flow processes are responsible for sheet erosion, the relatively uniform degradation of the soil surface. Overland flow may accumulate to form rills which may grow into gullies, the most damaging effect of raindrop erosion.

The elements of erosion are wind, water, and,ice and the process consists of particle detachment, transport, and deposit. Many factors affect the rate of erosion with the most important being the climate, vegetation, soil type, and land slope. Climate determines the primary element or elements of erosion. Vegetation provides significant protection from wind and water erosion. Well cemented soils resist erosion more readily than loose, granular soils. Generally, erosion increases with an increasing fraction of sand in the soil because of the loss of cohesion. Erosion decreases with an increasing percentage of water-stable aggregates. A soil whose individual grains do not tend to form aggregates will erode more readily than one in which aggregates are plentiful. Rates of erosion are greater on steep slopes than on flat slopes.

The Universal Soil Loss Equation and the Wind Erosion Equation relate the factors affecting erosion rates and were proposed as a basis for estimating soil losses. However, it was pointed out that it is difficult to properly define such factors as the rainfall or wind regime by a simple index number. Therefore, the equations are at best approximate and should be used with good engineering judgement and caution. Field measurements, of course, would take precedence over values obtained through the use of these equations.



CHAPTER TWO

EROSION CONTROL WITH THE USE OF VEGETATION, BIOTECHNICAL SYSTEMS, ADEQUATE DRAINAGE, AND GOOD SENSE

INTRODUCTION

I vividly recall my childhood in the hills of Fayette County, West Virginia, the southern part of the state. The mountains were my playground; I spent countless hours appreciating and exploring them. The mountains and hollows had been there forever; I assumed they always would be. Of course, West Virginia is, and has been for decades, one of our nation's leading producers of coal (especially that particular area of the state). It was in the fifties when they started "strip mining" the area. All the activity was exciting--blasting, bulldozing, loading, and hauling. Huge trucks travelled at top speed up and down the old dirt roads.

Then, tops of mountains started to almost miraculously disappear. From the valleys, you could see deep gashes in the surrounding hills. When the rains came, the little creeks flooded and swelled until sometimes they looked like rivers. The water was often black from the residue of the coal operations. I didn't know it at the time; but, that was "progress." Sure, the state had restoration laws and the miners were required to plant seedlings but more often than not the seeding was done half-heartedly, or not done at all. The last time I went back home, it wasn't there. The hollow is now a "gob" and slate dump almost completely full of waste from the mines.

I can't help but wonder how many other boyhood homes have been obliterated in the name of progress and how many could have been saved if good engineering practice in erosion control had been adhered to. Obviously, my home was subjected to one of the worst possible cases of erosion.

The erosion process, as we recall from chapter 1, is primarily caused by rainfall which displaces soil particles on inadequately protected areas and by water running over soil which carries some of the soil particles away. The rate of removal of the particles is proportional to the intensity and duration of the rainfalls, to the volume and characteristics of the water flow, as well as to the properties of the soil itself. (In some areas, significant erosion is caused by wind or ice.) Deposition of water-borne sediment occurs when the velocity is reduced and the transport capacity of the flowing water becomes insufficient to carry all of its sediment load.

Soil erosion can be natural or accelerated. Natural erosion is a geological process over which the engineer has little or no control and may be very slow or rapid depending upon many, various factors. Where man has disturbed land by construction, there may be a sudden large increase in erosion, producing accelerated erosion. Accelerated erosion is the type of erosion that should be controlled during engineering construction and after the project is completed. In areas of considerable natural erosion, the quantity of sediment that reaches a stream or lake, for example, before construction begins should be estimated and recorded. Seldom can erosion control measures be taken to reduce the natural erosion of a given area (AASHO, 1973).

Almost every facility, dam, road, and airport must rest upon soil; many of these structures also employ soil in their construction. To find a soil at a particular site which is satisfactory for a particular use as it exists is ideal, but unfortunately, most unusual. The causes of erosion suggest some basic principles for dealing with unsatisfactory soil and site conditions. Some of these principles are (AASHO, 1973):

1. Bypass "bad" soil if possible. Select a route or site where soil erosion will not be a serious problem.

2. Design slopes consistent with soil limitations.

3. Reduce the area of unprotected soil exposure.

4. Reduce the duration of unprotected exposure.

5. Protect the soil with vegetative cover, mulch, or erosion resistant material.

6. Control concentration of runoff.

7. Retard runoff with planned engineering works.

8. Trap sediment by temporary or permanent barriers, basins, or other measures.

9. Remove the bad soil, then replace it with good if necessary (to enhance vegetative growth, etc.).

10. Treat the soil to improve its properties (Soil Stabilization).

11. Properly maintain existing erosion control facilities.

12. Obtain easements for legal control where necessary.

EROSION CONTROL IN GENERAL

Effective erosion control begins in the planning and location of the engineering project. All possible construction sites have a base erosion potential which varies from place to place. Unless damage to the environment is considered early on, the cost of solving problems that might have been avoided can become astronomical. The initial cost of erosion control measures and the maintenance costs of such control over the anticipated life of a given project should be considered as a part of the economic analysis in site selection.

The natural drainage pattern, including subsurface flow should be examined for the sites considered. The drainage pattern beyond the perimeter of the construction site must also be studied to minimize and avoid damage to adjacent property or to anticipate preventive or corrective measures.

If the project is a highway, stream crossings should be made at stable reaches of the stream, avoiding meanders. For example, a highway constructed on the neck of a horseshoe bend that is subject to overflow is a poor location because the correct location of relief bridges sometimes varies with the flood stage. The amount and direction of flood flow at various stages must always be considered in the location of bridge openings to avoid undue scour and erosion which might result in a complete change in the river channel. Crossings should be made at a right angle to the direction of flow if possible with consideration for the direction of the flood flow where it is different from that of the low water. Stay away from streams if at all possible; the fewer the crossings the better (American Association of State Highway and Transportation Officials, AASHTO, 1973).

Check the geologic records of a site before construction. The ground conditions encountered in the field are usually the direct result of geologic processes operating on and within the earth. A knowledge of the geology of the area allows the engineer to detect potential problem areas and anticipate things like subsidence, landslides, and erosion problems. Avoid such areas if possible. Terrain features are the result of past geologic and climatic processes. A study of the terrain and the nature of natural erosion can aid in judging the complexity of the erosion and what erosion control measures are required.

It's a good idea to coordinate efforts with those agencies that might have knowledge of the soils and potential problems your selected construction site may have. The local offices of the Corps.of Engineers, Soil Conservation Service, and particularly natural or water resource agencies should be contacted in the early planning stages. Early recognition of potential problems and conflicts with your construction project may save you a lot of time and money in the long run and be of considerable public benefit as well.

Often these agencies are well informed on the pedological classification of the soil at your construction site. Pedology is the branch of the science of geology that deals with the outer four or five feet of the earth's crust. Pedological classification of a soil is determined principally on the basis of the geology of the parent material and certain inherent characteristics of the soil profile such as color, texture, structure, thickness, chemical composition, and the number and relative arrangement of the horizons. Pedology has been utilized extensively by agronomists as well as by highway and airport engineers. Because of correlations relating pedology to the behavior of shallow foundations, to the position of the water table, and to performance of septic tank disposal systems, it also enters into land use planning (Peck, Hansen, and Thornburn, 1974).

In the U.S., most pedological information is contained in county soil survey reports that have been published by the U.S. Department of Agriculture since the early part of the century. Such reports include detailed descriptions of the soil profile and geology of each soil series (groups of soils having similar profiles except for the texture of the surface horizon) in the county, and a map showing the boundaries of each series. Most reports today contain an engineering section which includes a soil map along with a tabulation of test data and engineering classifications, according to the AASHTO and unified systems, of samples taken from typical profiles of the more prominent soil series.

Another source of information the engineer may want to use to determine erosion characteristics of a particular site is aerial photography. Such photographs may yield information on the type of landform, soil color, erosion, surface drainage, vegetative cover, slope, land use, and relief. All these factors are indicative of the erodibility of an area. The principal elements of the photo pattern are often greatly influenced by soil texture and moisture conditions. Hence, each pattern must be correlated with the corresponding soil profile as determined by ground surveys, but after this has been done rapid and relatively inexpensive studies of very large areas can be carried out by air photo interpretation. For example, landscape characterized by sharp ridges and long, steep side slopes indicate a mature stage of geologic erosion. The landscape is at the height of its normal geological wearing away processes during this stage. Soils are shallow and slopes are critically steep. Any management planning activity on these kinds of soils should be given adequate study and consideration before the start of the project. Landscapes characterized by broad, flat ridges, gentle slopes, and wide, flat valleys with meandering streams indicate an old-stage of geologic erosion. The soil mantles are usually deep, finer textured, can retain high amounts of moisture, are less stony or rocky, and are frequently not as well drained as the soils characteristics of the mature stage of geologic eorison (Paine, 1981). Vertical aerial photographs may be obtained from the U.S. Department of Agriculture, U.S. Geological Survey, U.S. Forest Service, and numerous other services, both profit and nonprofit, throughout the U.S.

All too often, the design of a project is complete before erosion control measures are even considered. Many erosion problems that occur during and after construction can often be avoided by proper design and adequate specifications. Erosion control measures should be included in the original design package, and not left for subsequent contractors or for maintenance crews to provide after construction completion. When necessary, contour grading plans, coordinating grading, drainage, and geometry should be prepared. Geometry can be used to substantial advantage in minimizing soil erosion. For example, in highway construction, independent roadway grade lines which "fit" the terrain with a minimum of cuts and fills reduces exposed areas subject to erosion. Depressed roadways and underpasses require careful consideration to drainage design to avoid deposition of sediment and debris on the highway and in drainage facilities. Also, careful selection of alignment and grade of a highway is as important to successful erosion control as the general location. Alignment and grade, consistent with highway safety criteria, should be blended or fitted to the natural landscape to minimize cut and fill sections and reduce erosion and costly maintenance. These geometric features should be selected so that both ground and surface water can pass through the highway right-of-way or can be intercepted with minimum disturbance to streams or without causing serious erosion problems (AASHO, 1973).

Severe erosion of earth slopes is usually caused by a concentration of storm water flowing from the area at the top of cut or fill slopes. The concentration of storm water at the top of cuts should be avoided. A dike, preferably of borrow material to avoid disturbance of the natural ground, in conjunction with a grassed channel or paved ditch, can often be constructed at the top of the cut to prevent collecting water from running down the slope. Water can also be spread over the natural slope or carried to lower elevations in chutes, preferably closed pipes. Outlets for such high velocity chutes must be protected from scour. In some areas, serrated cut slopes aid in the establishment of vegetation, especially on decomposed rock or shale slopes. serrations may be constructed in any "rippable" earth or in earth that will hold a vertical face until vegetation becomes established. Where vegetation cannot be established or flow down the fill slope is objectionable, runoff should be collected at the top and directed to an adequate inlet and chute.

Construction practices and procedures are just as important to erosion control on a project as a good design. The plans, specifications and special provisions of a contract should be explicit in showing the location, scope, and manner of performing erosion control measures. Sufficient erosion control measures should be included as a part of the initial grading contract. Disposal areas for spoil should be considered early so that soil will not get into waterways. Proper planning and scheduling of construction operations are also major factors in controlling erosion. The schedule should consider the probable weather conditions and the occurrence of storms, particularly if the work is in or adjacent to a stream. Construction of drainage facilities as well as performance of other contract work which will contribute to the control of siltation should begin with the clearing and grubbing of the site and accomplished in conjunction with earthwork operations.

Adequate inspection during the construction phase of a project is essential for erosion control. The engineer should require strict adherence to the work schedule particularly in regard to the order in which operations should be performed. If deficiencies in the design or performance of erosion control measures are discovered during construction, the engineer should take immediate steps to correct the problem. Formal field reviews and inspections, involving design, construction, and maintenance engineers will help correct deficiencies and improve erosion control measures and procedures. The contractor should not be allowed to deface, injure, or destroy vegetation outside construction limits. Prior to suspension of construction for extended periods (weekends, holidays, work stoppages, etc.), the contractor should shape the top of earthwork in such a manner as to allow runoff without undue erosion. Temporary earth dikes may be required along the top edges of embankments to intercept runoff water. Temporary slope drains can be installed to carry runoff from construction areas in the immediate vicinty of rivers, lakes, etc. Cleaning operations should be scheduled and performed so that grading and permanent erosion control measures can immediately follow; otherwise, temporary erosion control measures should be taken between successive construction stages. The maximum allowable surface area of erodible soil exposed at any one time should be specified in the contract, subject to modification depending upon the construction progress and the erosion potential of the area. Temporary protection such as fiber mats, plastic, chemicals, compounds, straw, dust palliatives, and fast growing grasses may be required to prevent erosion of the exposed areas. Every effort should be taken to prevent erosion from developing; once started, it is often much more difficult to control or correct than it would have been to prevent. Additionally, a little effort in preventing erosion may save you a lot of time and money in the long run (AASHTO, 1973).

EROSION CONTROL FROM A MAINTENANCE PERSPECTIVE

In most engineering texts, a discussion of maintenance regarding engineering projects is usually the last chapter in the book (and that's where it usually stays in the engineer's mind). However, I consider the topic important enough to put it towards the front. It's not enough to design and construct erosion control measures and devices into a facility. Once construction is complete, a viable maintenance program is necessary to preserve the new investment and keep it in top condition. AASHTO defines highway maintenance as, "the preservation and keeping of each type of roadway, roadside structure, and facility as nearly as possible in its original condition as constructed or as subsequently improved, and the operation of highway facilities and services to provide satisfactory and safe transportation." The definition could be generalized to apply to all facilities and engineering projects. Without maintenance, things tend to return to natural conditions--to deteriorate. Early attention to routine maintenance items will save millions of dollars in maintenance repairs.

Routine maintenance activities and the efforts directed to each vary widely. For example, on our nation's highways, state highway agencies divided their $4 billion total maintenance expenditures as shown in Table 2.1.

Table 2.1 UNITED STATES HIGHWAY MAINTENANCE COSTS (Oglesby and Hicks, 1982)

Category	Expense in $ Billions	Percent of Total
Road & Roadside	2.5	63
Bridges	0.2	6
Snow Removal & Sanding	0.5	12
Operation of Toll Facilities	0.3	7
Traffic Control	0.5	12
	4.0	100

However, the figures in Table 2.1 are deceiving. Some agencies care for a large mileage of primarily low-volume rural roads; others have the responsibility for many miles of urban freeways and other high-volume urban arteries. There are many other variables; for example, Vermont, a rural state with a cold climate, spends 49 percent of its budget for snow removal and sanding while other states may not have this problem at all. At the other extreme, densely populated New Jersey allocated 18 percent of its budget to traffic ontrol and 23 percent to toll operations, levels far above the national average (Oglesby and Hicks, 1982).

Maintenance costs associated with erosion control measures usually pertains to the care and control of vegetation and to drainage and drainage structures. (Erosion repair is an altogether different category as discussed in chapter one.) The character of the facility determines what maintenance is required. Grass must be mowed, fertilized, and sometimes weeded with the use of herbicides. Mowing promotes better growth of the turf and aids in proper drainage; it should be started as soon as the grass or weeds are high enough to be cut and continued periodically throughout the growing season. Fertilization should be done as required.

If weeds are a problem, there are three major categories of herbicides commonly used (Ritter and Paquette, 1967):

1. Ester and amine forms of 2,4-D are commonly used to control and eliminate broad leaf weeds in

turf areas and in seldom mowed areas in which the presence of tall weeds is objectionable.

2. Various compounds are used to control and eliminate weeds beneath guard rails and other structures; they include sterilants, combinations of systemic herbicides such as 2,4-D and Dalapon, and contact herbicides which rapidly kill all exposed plant parts.

3. Retardants are used in some cases to inhibit the growth of grass to a certain height.

Seeding, sodding, and the planting of vegetation are important maintenance operations for the prevention of erosion. Seeding may be done on relatively flat areas, while sodding is necessary on steeper slopes. On very steep slopes where neither seeding or sodding is practical, the ground surface may be protected by the planting of vines or similar ground cover. The planting of dune grasses on sandy slopes to prevent wind erosion has proven very successful (Ritter and Paquette, 1967). Trees and shrubs work very well for erosion and control but their care and maintenance can become expensive. when you consider planting, trimming, fertilizing, spraying, and the construction of tree walls. Major tree surgery and the removal of broken limbs caused by storms are often necessary. Experts in soil conservation, agronomy, and drainage should be consulted to assist in the selection of erosion control measures.

Drainage maintenance involves keeping ditches, culverts, structures, and appurtenances such as drop inlets and catch basins clean and ready to carry the next flow of water. Sediments deposited during periods of heavy flow must be removed. Brush, branches, and other debris that collect in trash racks or at culvert and structure entrances must be disposed of. Badly eroded channels and dikes must be repaired, and paving, seeding, sodding, riprap, bank protection, or other means must be adopted to prevent recurrence (Oglesby and Hicks, 1982).

In highway design and construction, the roads are crowned and elevated to remove the water quickly so that it will not interfere with road use and to minimize the damaging effects of the water to the road structure. On high-use road surfaces, crowns are more permanent but on low-use roads, such as gravel, it is necessary to maintain a proper crown by blading.

Wide shallow ditches are preferable for maintenance and are less dangerous where automobiles are involved. Certain types of roadside ditches may be kept clean and the slope maintained by the use of a grader but care is necessary to preserve sodded areas and shrubs. The original line or grade of the ditch should also be maintained. When ditches become eroded owing to excessive grade, handling of large volumes of water, or both, it may be necessary to construct ditch checks to retard the flow or to line the ditches to prevent soil movement (Ritter and Paquette, 1967).

All drainage and erosion control devices should be checked immediately after construction so that deficiencies can be located and corrected before they develop into major problems and then checked periodically to ensure they are free from obstructions. Maintenance and inspection records should be kept and include sufficient detail to permit analysis of maintenance problems associated with erosion. Maintenance should be a pre-design consideration and not an afterthought. In my experience as the Chief of Maintenance for an Air Force Base Civil Engineering operation, I found this to be the exception rather than the rule. I was usually given the opportunity to review a design only after it was near completion. By then, time was "tight" and a maintainability review was, at best cursory. Needless to say, my maintenance shops spent an inordinate amount of time correcting design deficiencies.

EROSION CONTROL-DRAINAGE AND DRAINAGE STRUCTURES

Adequate drainage of the project site is a must for erosion control as well as for the protection of the project investment. Highway engineers recognize this and as a result spend one highway construction dollar in four on culverts, bridges, and other drainage structures. Substantial added expenditures are demanded on rural roads for ditches, dikes, channel, and erosion control installations. In urban and suburban locations, major capital investment goes into storm drains and their appurtenances (Oglesby and Hicks, 1982).

Drainage may be generally defined as the process of controlling and removing excess surface and underground water encountered within the limits of the project site. The flow of surface water the engineer is concerned with generally results from precipitation; a portion of the surface water evaporates or enters or "percolates" into the soil, while the remainder stays on the ground surface and must be carried on, beside, beneath, or away from the protected area. Measures taken to control the flow of surface water are generally termed "surface drainage" while those dealing with ground water, e.g. where the water table lies close to the surface, are called "subsurface drainage" or, more simply, "subdrainage." Our discussion will be limited to surface drainage.

Dealing with surface drainage and subsequent erosion control problems must begin with the location survey. Ideally, construction sites would be located between large drainage areas then all the flow is away from the site and the problem is reduced to caring for the water that falls on the facility and associated grounds. Also, ideal locations avoid steep grades and heavy cuts and fills, both of which raise difficult problems in erosion control. Once the construction site has been selected, the analysis of surface drainage problems follows three basic steps:

1. hydrology - estimating the peak rates of runoff
2. hydraulic design - selecting the kinds and sizes of drainage facilities to most economically accommodate the estimated flow.

3. review - ensure the design does not create erosion or other environmentally unacceptable conditions.

Estimating rates of rainfall runoff is relatively complex because there are so many unknown variables involved. Much of the rain which falls during the first part of a storm is stored on the vegetal cover as "interception" and in surface puddles as "depression storage." As rain continues, the soil surface becomes covered with a film of water, known as "surface detention," and flow begins toward an established surface channel. That part of storm precipitation which does not appear either as infiltration or as surface runoff during or immediately following the storm is "surface retention." In other words, surface retention includes interception, depression storage, and evaporation during the storm but does not include water temporarily stored enroute to the streams. Other factors that effect the amount of runoff are wind, temperature, vegetal cover, altitude, relief, and many more. Despite all this, the practice of estimating runoff as a fixed percentage of rainfall is the most commonly used method in design of urban storm drainage facilities, highway culverts, and many small water-control structures. The method can be reasonably correct only when dealing with a surface which is completely impervious so that the applicable runoff coefficient is 1.00 (Linsley, Kohler, and Paulhus, 1975). The uncertainties inherent in predicting runoff from rainfall records would be of no concern if long-time, nationwide records of the flow from all or at least a large number of representative small drainage areas were available. Unfortunately, only about 5,000 water sheds in the U.S. are gauged and the data reported by the participating states to the U.S. Geological Survey under a program titled National Small Streams Data Inventory (NSSDI). However, this coverage is too meager, the data too incomplete, and record duration too short to provide a reliable method for designing individual drainage facilities (Oglesby and Hicks, 1982).

Since 1852, more than 100 equations involving over 50 variables have been proposed to estimate peak runoff from small ungauged water sheds. A comparison of actual records of 493 water sheds with procedures used by state highway agencies indicate that two-thirds of the predictions were off by at least 25 percent and in one in five cases actual runoff was overestimated by a factor of three. In short, "presently used methods for estimating runoff on ungauged rural water sheds are unsatisfactory nationwide. Consequently, designers should make the best possible use of existing prediction methods, with full realization of the high probability of error, and giving careful consideration to the increased cost of overdesign versus the possible consequences of an underestimation of peak flow." (Oglesby and Hicks, 1982).

As stated before, there are numerous methods for estimating peak flows; I will discuss only one, The

Rational Method, recognizing that any prediction regarding future runoff is a calculated guess, that changes in land use alters runoff characteristics, and that, for small ungauged water sheds, any method of prediction may be subject to substantial error. The rational method, first proposed by Ireland in 1851, may be reduced to an expression similar to Darcy's Law:

$$Q = CiA \tag{2.1}$$

Where Q = runoff in CFS

C = a runoff coefficient expressing the ratio of rate of runoff to rate of rainfall

i = intensity of rainfall, inches per hour for a duration equal to the time of concentration

A = drainage area in acres

You will notice that $Q = CiA$ is not dimensionally correct. However, it yields numerically correct results because 1 inch/hour/acre and 1 CFS approximately represent the same amount of water per unit time (within 0.8 percent). Although the rational formula is often used to estimate flows from large drainage areas, some researchers have recommended that the limit be 200 acres; others say 500 maximum.

Suggested runoff coefficients are given in Table 2.2. Where ground cover is dissimilar, the drainage area can be subdivided and a composite coefficient obtained by weighting the coefficients for each section according to area.

Table 2.2 SUGGESTED VALUES OF COEFFICIENTS OF RUNOFF, C, FOR USE IN THE RATIONAL FORMULA (Ritter and Pacquertte, 1967).

Surface	C
Concrete or bituminous pavements	0.70-0.95
Gravel or macadam	0.40-0.70
Impervious soil*	0.40-0.65
Impervious soils with turf*	0.30-0.55
Slightly pervious soils*	0.15-0.40
Pervious soils*	0.05-0.10
Wooded areas (depending on slope and cover)	0.05-0.20

*For slopes from 1 to 2 percent.

Rainfall intensity is obtained from records of nearby stations of the U.S. Weather Bureau. These records are reduced to a graph showing rainfall intensity versus duration for various recurrence intervals as shown in Figure 2.1.

Actual selection of the value for rainfall intensity rests on estimates of the acceptable frequency of occurrence of the design flood and on the time of concentration for the area. The latter is the time period required for water to reach the outlet from the most remote point in the basin. For paved surfaces, the recommended time is five minutes; where the water is from grass, 10 minutes. For larger areas, this time can be considerably longer (Oglesby and Hicks, 1982).

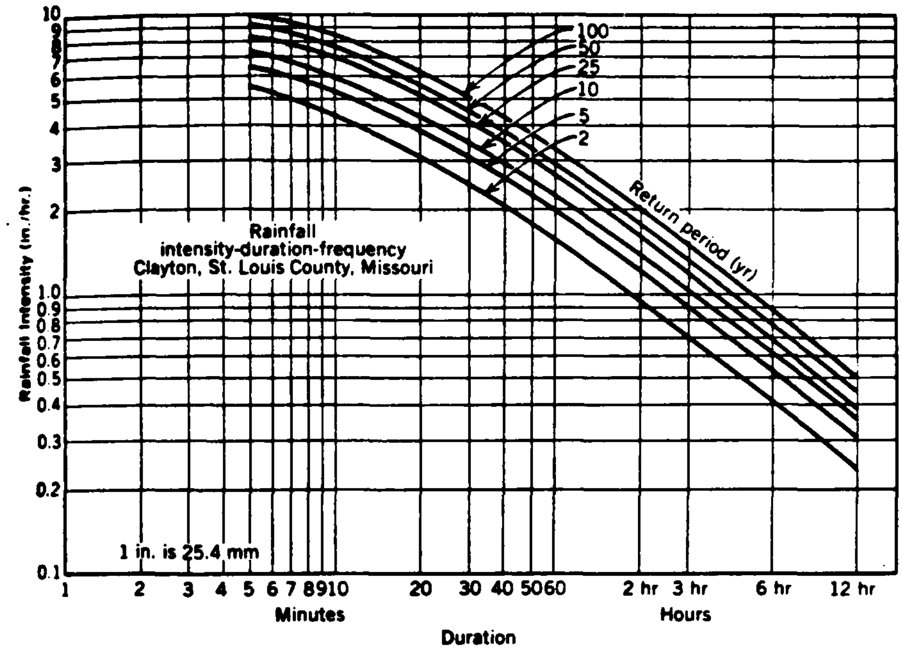


Fig. 2.1 A typical rainfall intensity, duration, frequency curve. (Source: *Design of Urban Highway Drainage*, FHWA.)

The drainage area is determined from topographic maps, aerial photographs, or rough field surveys comparable in accuracy to compass and pacing. Greater precision is not really justified and usually not required.

Once the engineer has made his best estimate of the amount of runoff, he must turn his attention to hydraulic design--selecting the kinds and types of drainage facilities to most economically accommodate the estimated flow. Hydraulic design includes the basic principles of fluid flow, particularly those relating to open channels and closed conduits. A cardinal rule of drainage design is that existing natural drainage patterns and soil cover be disrupted as little as possible. Necessary changes must not at any point bring velocities that will create new erosion problems. Disregard for this simple rule has created many serious maintenance problems and brought down the wrath of conservationists on many engineers.

Surface channels, natural or man-made, are usually the most economical means of draining runoff. A well-designed channel carries storm water without erosion and with the lowest overall cost, including maintenance. To minimize erosion, channels should have flat slopes and wide rounded bottoms and be lined with grass, rock, concrete, or other material depending on flow rates. The design of surface channels, as well as the design of conduits with a free water surface, is based upon established principles of flow in open channels. Manning's formula, which applies to conditions of steady flow in a uniform channel, is the most commonly used design criteria.

$$V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n} \qquad (2.2)$$

Where V = average velocity in fps

R = hydraulic radius in feet = area of the cross-section of flow in square feet divided by the wetted perimeter in feet

S = slope of the channel in feet per feet

n = Manning's roughness coefficient; some typical values are given in Table 2.3

Table 2.3 VALUES OF MANNING'S ROUGHNESS COEFFICIENT FOR OPEN CHANNELS
(Ritter and Paquette, 1967)

Type of Lining	n
Smooth concrete	0.013
Rough concrete	0.022
Riprap	0.030
Asphalt smooth texture	0.012
Good stand, any grass - depth of flow more than 6 inches	0.09-0.30
Good stand, any grass - depth of flow less than 6 inches	0.07-0.20
Earth, uniform section, clean	0.016
Earth, fairly uniform section, no vegetation	0.022
Channels not maintained, dense weeds	0.08

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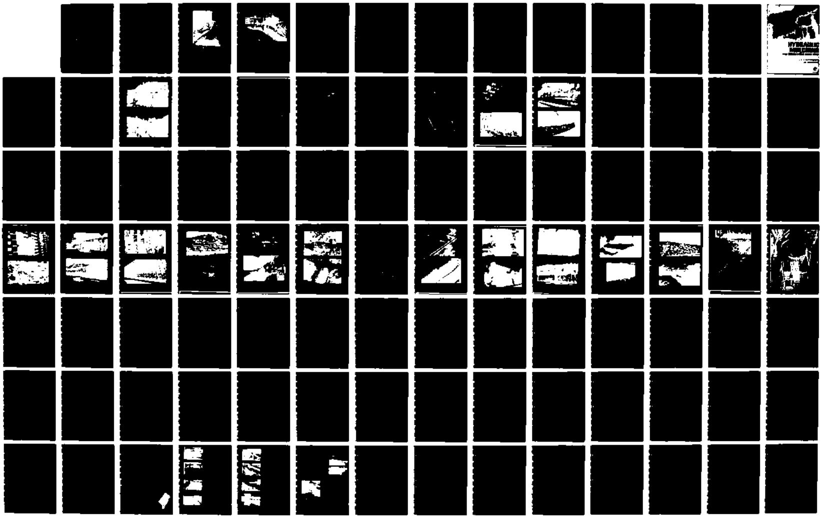
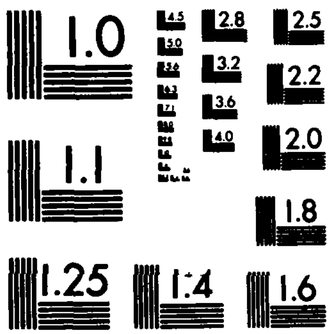




MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Another relation commonly used is the continuity equation,

$$Q = VA = \frac{1.486}{n} AR^{2/3}S^{1/2} \qquad (2.3)$$

Where Q = discharge in cfs

A = area of the flow cross-section.

There are also numerous charts available (developed by the Bureau of Public Roads) for the solution of Manning's equation for various common channel cross-sections.

Chutes generally have steep slopes and carry water at high velocities (See Fig. 2.2). Pipe chutes are preferable to open chutes because the water cannot jump out of the chute and erode the slope. Provision for dissipating the energy along the chute or at the outlet is usually necessary. In highly erodible soil, it may be necessary to provide water tight joints to prevent failure of the facility or embankment. In chute design, avoiding splash which causes erosion is always a prime concern (See Fig. 2.3).

Variations in channel alignment should be gradual, particularly if the channel causes flow at high velocity. Whenever practical, changes in alignment should be made on the flatter gradients to prevent erosion by overtopping the channel walls. Although usually more expensive, rectangular channel sections are preferred on curves of paved channels to give a more positive control of the flow. Highly erodible channels may require special lining. Protective linings for channels and streams can be very expensive and a considerable percentage of the highway dollar is spent on this item of work (AASHTO, 1973). Field

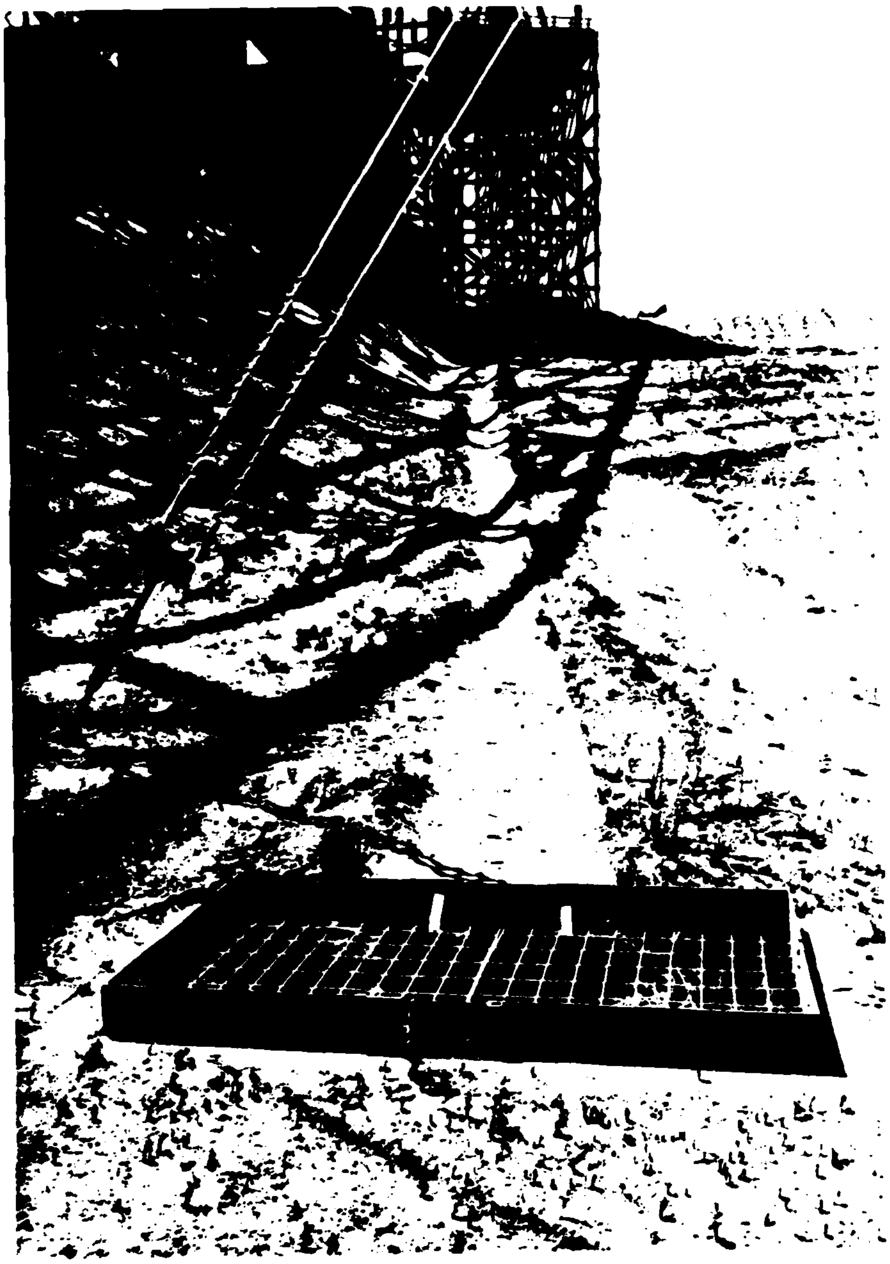


Fig. 2.2 Typical of the type chutes in use at the TRESTLE. This one, however, has a very moderate slope by comparison.

Fig. 2.3. A chute completely destroyed by excess flow and oversplash. A large gulley has formed down slope and appears to be advancing headward. (See Figs. 1.4 and 1.5).

manuals and publications of the Soil Conservation Service, Corp of Engineers, and Bureau of Reclamation contain additional channel design information which may be of considerable importance to the design engineer.

Culverts and bridges are also very important where drainage is concerned but their design will not be discussed here. However, their tendency to constrict flow and increase velocities to produce additional erosion problems warrants some attention. In many instances, erosion and scour at these locations damage the associated embankment, the structure itself, or the downstream channel. The energy at the outlet of culverts, as well as chutes, should be dissipated or the area subject to scour should be protected by riprap or other types of protection. The potential of scour at bridge piers and abutments must be considered and, if necessary, adequate embankment and foundation protection provided. The control of surface runoff is absolutely essential to minimize erosion effects and as such should not be regarded as a separate element of a project design. Rather, drainage considerations must accompany every step in project location and design so that the final design and resulting construction operations will provide for the best possible drainage and least erosion at reasonable cost.

EROSION CONTROL WITH VEGETATION

After all the inferences I've made so far about the use of vegetation and its role in erosion control, a section of this text devoted to the subject should come as no real surprise. Its economic advantage alone warrants its serious consideration as an erosion control measure. The use of vegetation for preventing surface erosion on slopes is fairly common and well understood. The U.S. Soil Conservation Service and other government agencies around the world have long advocated vegetation to control both rainfall and wind erosion. Vegetation may be herbaceous (pertaining to plants without woody tissue such as grasses) or woody (trees, shrubs, etc.). Each type has its advantages. Herbaceous, and to a lesser extent woody, vegetation controls erosion by (Gray & Leiser, 1982):

1. Interception - foliage and plant residues absorb rainfall energy and prevent soil compaction from raindrops.
2. Restraint - root system physically binds or restrains soil particles while above-ground residues filter sediment out of runoff.
3. Retardation - above ground residues increase surface roughness and slow velocity of runoff.
4. Infiltration - root and plant residues help maintain soil porosity and permeability.
5. Transpiration - depletion of soil moisture by plants delays onset of saturation and runoff.

Woody, and to a lesser extent herbaceous, vegetation helps to prevent mass-movement, particularly shallow sliding in slopes. The factors affecting slope stability have been grouped by Varnes (1958) into those tending to increase shear stress and those tending to reduce shear resistance. These groupings provide a basis for examining the likely influence of vegetation on slope stability. Gray and Leiser (1982) lists possible ways woody plants might affect the balance of forces in a slope.

1. Root reinforcement - roots mechanically reinforce a soil by transfer of shear stresses in the soil to tensile resistance in the roots.
2. Soil moisture modification - evapotranspiration and interception in the foliage limit buildup of soil moisture stress. Vegetation also affects rate of snow melt which in turn affects soil moisture regime.
3. Buttressing and arching - anchored and embedded stems can act as buttress piles or arch abutments in a slope, counteracting shear stresses.
4. Surcharge - weight of vegetation on a slope exerts both a downslope (destabilizing) stress and a stress component perpendicular to the slope which tends to increase resistance to sliding.
5. Root wedging - alleged tendency of roots to invade cracks, fissures, and channels in a soil or rock mass and thereby cause local instability by a wedging or prying action.

6. Windthrowing - destabilizing influence from turning moments exerted on a slope as a result of strong winds blowing downslope through trees.

The first three factors enhance slope stability, whereas the fourth may have beneficial or adverse impact depending on soil or slope conditions, and the last two are likely to affect stability adversely.

Perhaps the last two items, root wedging and windthrowing, are the reasons why so many engineers seem to shy away from vegetation, especially woody, as a means of soil-slope stabilization. However, there is no real evidence to support the claim that the adversities of woody vegetation on slopes outweigh the benefits. Root wedging was alleged to have contributed to the failure of Kelly Barnes Dam in Toccon, Georgia (Shaw, 1978) but the evidence is scant and unconvincing. One could argue that the roots actually helped hold parts of the earth dam together making the failure less severe. As for windthrowing, the total downslope force created by a wind blowing through a stand of trees, and hence its overall effect on slope stability, has never been evaluated. On the other hand, judging by the preponderance of evidence from published field and laboratory studies, the beneficial effects of root systems far outweigh any possible adverse effects (Gray and Lesier, 1982).

The biggest drawback in the use of vegetation as a soil stabilizer is getting it to grow and keep on growing. Over the continental U.S., soil, topography, temperature, and amount and character of rainfall range from one extreme to another. Plants that grow well in one area may not be suitable somewhere else. In addition, planting and maintaining vegetation can become costly; in arid and semi-arid regions irrigation is often necessary; in snow areas vegetation may be killed off by frost freeze or de-icing salts if adjacent to a highway.

Yet the use of vegetation as an erosion control measure has proven effective throughout the U.S. and around the world. It has been proven that no exposed slope will remain uniform unless protected from rain and wind by a healthy stand of some vegetation. If soil and moisture conditions are suitable, grass is usually the most economical cover for protecting slopes from erosion. For example, in Indiana an excellent turf is formed by a four to one mixture of tall fescue and bluegrass. The fescue is quick starting and will provide early protection. Bluegrass fills out the turf forming a firm uniform sod which, with proper maintenance, will provide a satisfactory permanent cover (Sanborn, 19).

Woody plants can help keep costs down in an area where shrubs are readily available near the site. Such ground hugging plants as juniper, low bush blueberry and sweet fern will serve very well where erosion conditions are not too severe, and if they can be acquired freely adjacent to the work.

In most areas of New Mexico, a very sturdy plant is required to withstand the semi-arid climate. The Soil Conservation Service has used indian ricegrass, sand dropseed, giant sand dropseed, and sideoats gramagrass with remarkable success in and around Albuquerque, New Mexico (Farmer, 1984). Also, it appears that "Corto" Australian Saltbush and "Cochise" atherstone lovegrass, although not proven yet, may be used in certain parts of the state. All these grasses have a high survivability rate in regions with very little annual rainfall.

Of course, proper soil preparation is essential to ensure rapid growth of cover. The soil surface should be loosened by scarifying, harrowing, or raking. Fertilizer and lime should be used if necessary. It's a good idea to get a soil test and follow the recommendations of the testing agency. A soil test should always be obtained before applying lime. In cases of exceptionally clean and well drained gravels, it is desirable to spread three to four inches of topsoil as a seed bed. For best results fertilizer, and lime if needed, should be well mixed into the soil before seeding.

Small areas are often seeded by hand. After the soil is loosened and fertilized, seed is spread either by broadcasting handfuls in a broad, sweeping action or with a small mechanical spreader of the lawn seeding variety. Follow manufacturers recommendation for quantity and mixture of seed.

When shrubs are used they should be delivered in such a manner as to avoid damage by drying out. If plants must be held more than a few hours before planting they should be placed in a shady area, the roots covered with earth if not balled and burlapped, and kept watered. Holes should be dug twice as large as the transplanted root system and the roots spread out naturally with no bending or crowding. Fill should be placed carefully to avoid breaking roots when it is firmed in place. Care should be taken to avoid placing fertilizer directly on roots or under plants. It's best to mix fertilizer with the backfill before placing, or to apply it to the surface, with mulch, after planting (Sanborn, 19).

Mulching, with or without topsoil, is a highly recommended practice. Straw, hay, roadside cuttings, or local materials such as pine branches, leaf litter, moss, sawdust, tobacco stems, cotton seed hulls, or threshed soybean plants are spread uniformly over the surface, often by passing them thru a special blower. The spread material is then worked into the previously loosened soil surface by means of disks, soil pulverizers, or with sheepsfoot or similar tamping rollers. On banks too steep for ordinary equipment, rollers can be propelled up and down the slope by winches mounted on trucks or tractors. Mulching retards washing of the soil, adds organic material, and holds moisture between rains. It is extremely effective for erosion control as it binds the soil together and provides favorable conditions for the growth of native plants. On high, steep slopes in rough country where rainfall is intense, mats of brush embedded in the slopes are sometimes used to supplement mulching. Mulching should immediately follow the seeding and planting of grass or woody plants. About one and one-half tons per acre or 70 lbs. per 1000 square feet, giving one to two inches of cover, is most effective (Sanborn, 19).

Hydraulic seeding is an alternate method of seeding especially large areas. In this process fertilizer and seed are mixed together in a water slurry and sprayed over the prepared soil surface in a single operation. Some of the commercially produced mulches can be added to the fertil r and seed slurry, and thus mulching is accomplished in th same operation (e.g., "Conwed" 1980 Advertisement, Fig ?.4-2.5). The equipment required is relatively expensive, so this method is usually employed only when large amounts of seeding are to be done, or for small areas, when done on a contract basis.

Seeding, sodding, and the planting of vegetation are important to the success of a viable erosion control program. Often it is the most economical and practical approach. However, care is necessary to select the right vegetation for the given site and geographic area. An expert's advice on what to plant will help ensure success. Whatever the species or method, the important principle is--"Fertilize and seed for a turf cover immediately after grading, and mulch to provide immediate protection" (Sanborn, 19).

EROSION CONTROL WITH BIOTECHNICAL SYSTEMS

Biotechnical systems combines the perspectives and techniques of engineering and horticulture to achieve a common goal. The concept entails the use of structures in combina-

tion with vegetation to arrest and prevent slope failures and erosions. Both biological and mechanical elements must function together in an integrated and complementary manner. The vegetative element should not be regarded as a cosmetic facelift of the structure because the vegetation has an important functional role in terms of preventing surface erosion and shallow mass movement. Principles of statics and mechanics and principles of horticulture and plant science are used to analyze and design biotechnical systems.

There are several advantages to using biotechnical (biotech) systems. Number one, of course, is that they do the job. Their success is evidenced mile after mile along our nation's highways. Also, biotech systems are usually cost effective. Actual field studies (White, 1979) have shown that in many instances biotech systems are more cost-effective than the use of either vegetation or structures alone. Vegetative treatments alone are naturally much less expensive than earth retaining structures or other constructed protective systems; however, their effectiveness in terms of erosion control under severe conditions may be inadequate. Another advantage of biotech systems is that they are esthetically pleasing--they blend into the landscape and do not visually intrude upon nature as much as conventional earth retaining structures. In addition, biotech systems emphasize the use of natural, locally available materials--earth, rock, timber, vegetation--in contrast to manmade materials such as steel and concrete. I do not imply that steel and concrete are not often used. In many instances an effective design may require their use but even in these cases biotech design can be incorporated. A good example is a porous or open-face crib retaining wall whose front face can be vegetated with a variety of plants and vines (Figure 2.6). Biotech systems tend to be more labor-skill-intensive than energy-capital-intensive. That is, the nature of biotech systems is such that well-supervised skilled labor can often be substituted for high-cost, energy-intensive materials. A good example would be slope protection by willow wattling (Figure 2.7) or brush layering (Gray and Leiser, 1982). There will, of course, be instances when vegetation alone will work satisfactorily and other instances where structural systems are all that is required. However, the benefits to be gained from biotech systems can be quite substantial and the prudent engineer would be wiser to give them very serious consideration in the design of a project.

There are probably as many different biotech designs as there are varieties of plants and elements of construction. I will not attempt to discuss every possible system but will present some of the more common designs in use today to provide some idea of how innovative the engineer can be with the typically preferred functional approach to design. If you haven't guessed it by now, biotech designs are almost always used as a slope protection measure. The structure is key to that purpose; a structure placed at the foot of a slope helps to stabilize the slope against mass-movement and

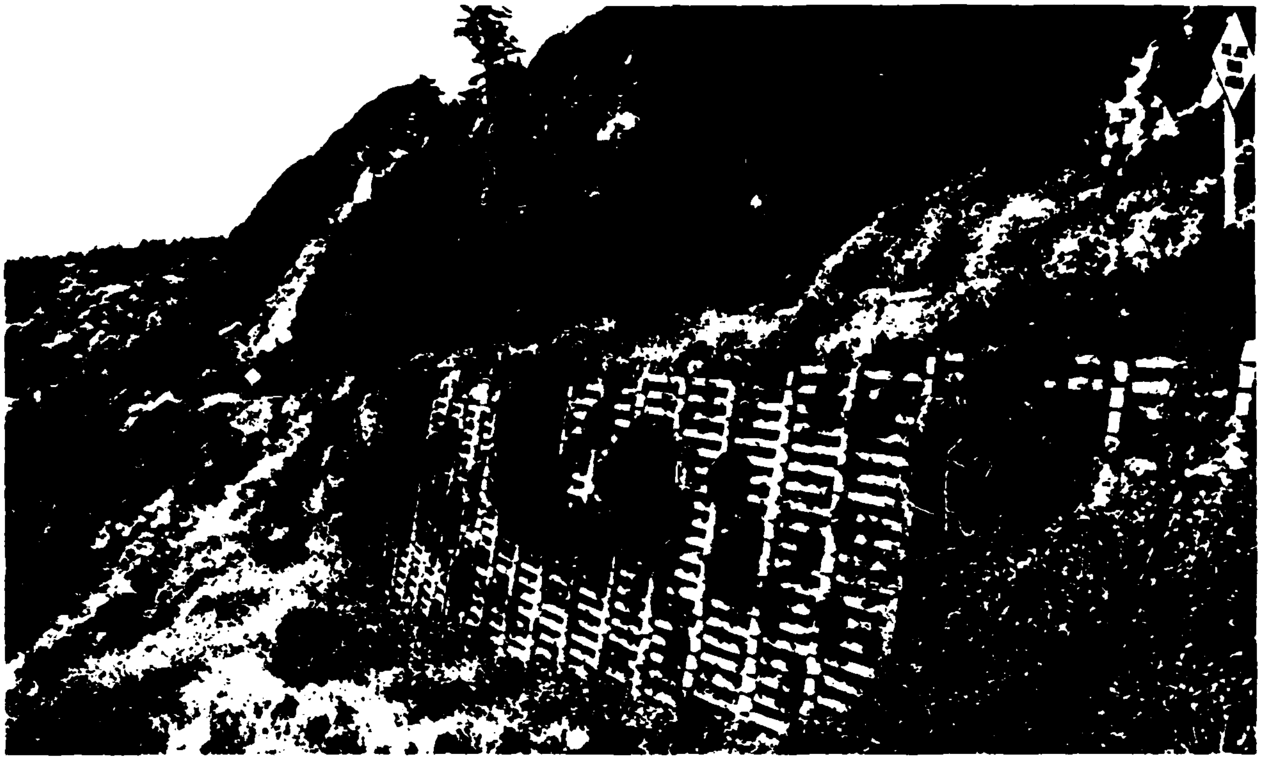


Fig. 2.6 Vegetated, open-front crib wall supporting a roadway. Colorful native shrubs and plants have become established in the openings between structural members at the face of the wall. Trinidad Beach, California.

Fig. 2.7 Slope protected by contour-wattling. Partially buried and staked willow wattles protect slope against erosion; wattles eventually root and sprout, thus further stabilizing the slope. Redwood National Park, California.

protects the toe and face against scour and erosion.

Selection of a suitable retaining structure entails a wide variety of choices. Several basic types are available, each with its particular advantages, requirements, and limitations. Selection depends upon such considerations as site constraints, availability of materials, appearance of wall, ease of construction, vegetal incorporation, and cost. With regard to basic types, retaining structures can be classified into one of the following categories (See Figs. 2.8 through 2.12):

1. Gravity walls (Fig. 2.8A).
2. Crib or bin walls (Fig. 2.8C and 2.9).
3. Reinforced earth (Figs. 2.8E, 2.11, and 2.12).
4. Cantilever and counterfort walls (Figs. 2.8B and 2.13).
5. Gabions and welded-wire walls (Figs. 2.8D, 2.8F, and 2.10).
6. Pile walls (Fig. 2.8G).
7. Tie-back walls (Fig. 2.8H).
8. Breast walls (Fig. 2.14).

Gravity walls (Fig. 2.8A) resist earth pressure by their own weight or mass. They are conventionally constructed from stone or concrete that can resist compression and shear but no appreciable tension. When constructed from masonry and cement, gravity walls are essentially monolithic. Gravity walls, like all earth retaining structures, must be capable of resisting external forces which cause overturning and sliding.

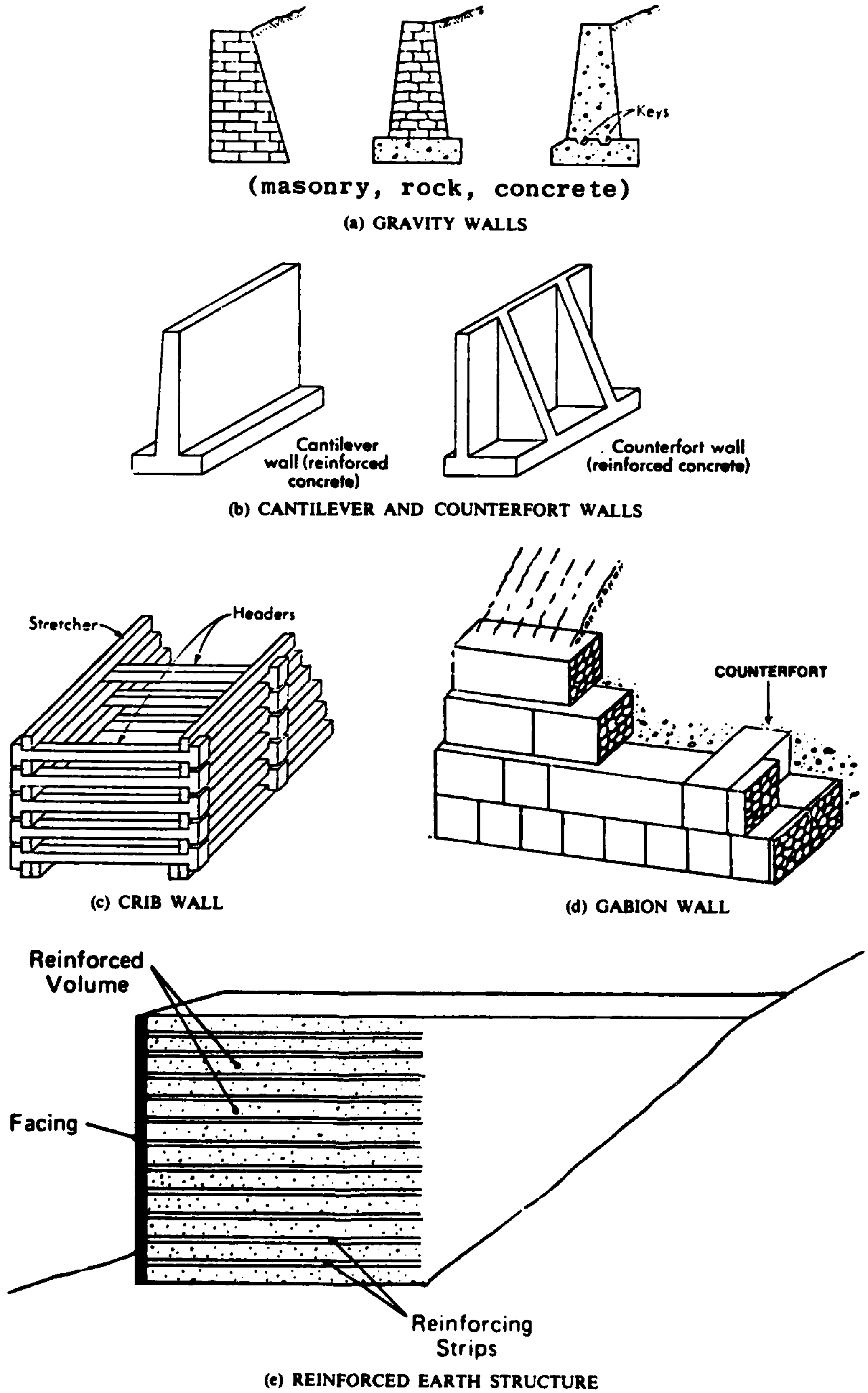
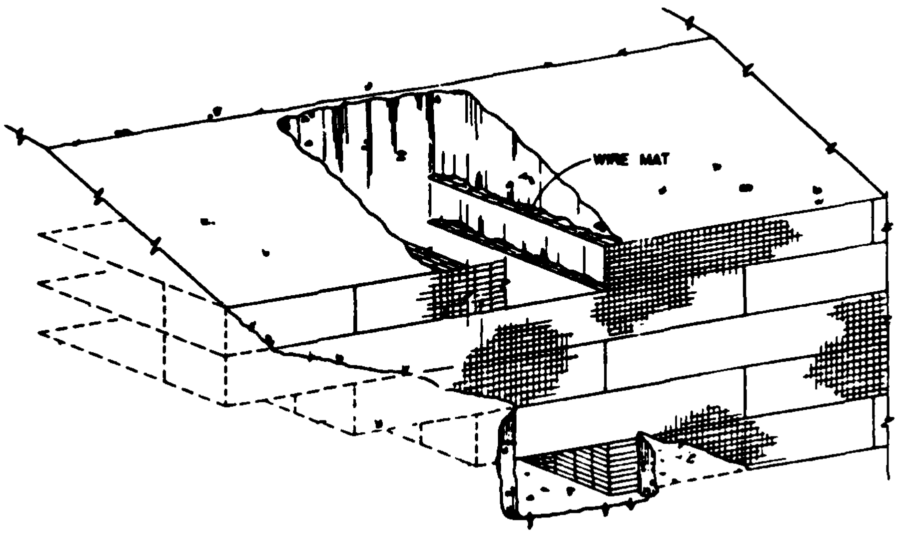
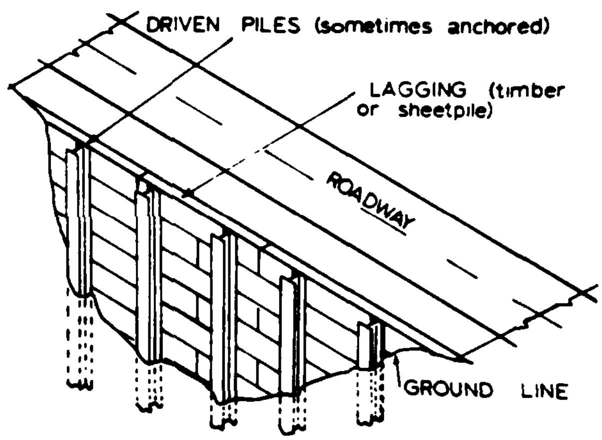


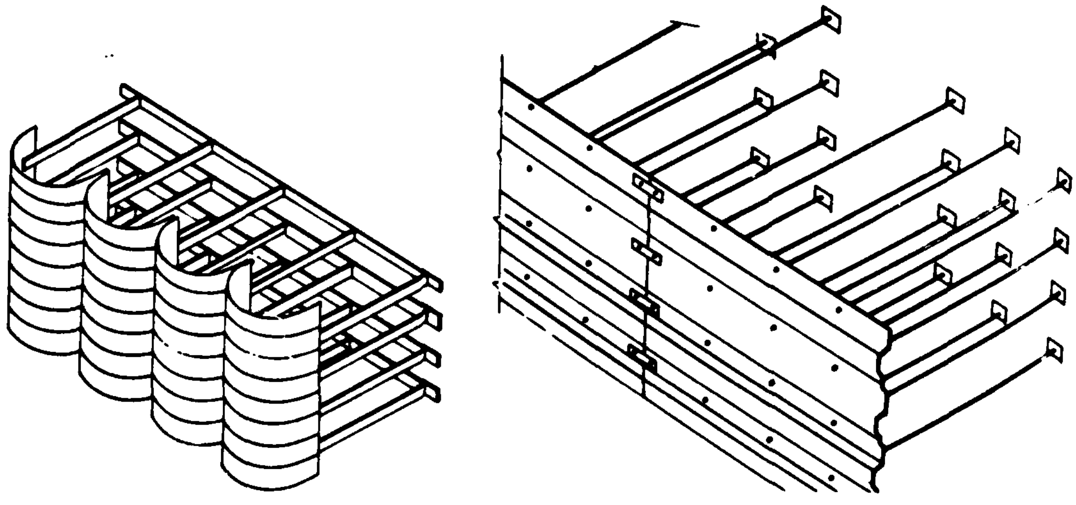
Fig. 2.8 Basic types of retaining structures. (a) Gravity walls. (b) Cantilever and counterfort walls. (c) Crib wall. (d) Gabion wall. (e) Reinforced earth wall. (f) Welded-wire walls. (g) Pile walls. (h) Tie-back or anchored walls.



(f) WELDED-WIRE WALL



(g)



(h)

Fig. 2.8 Contd.

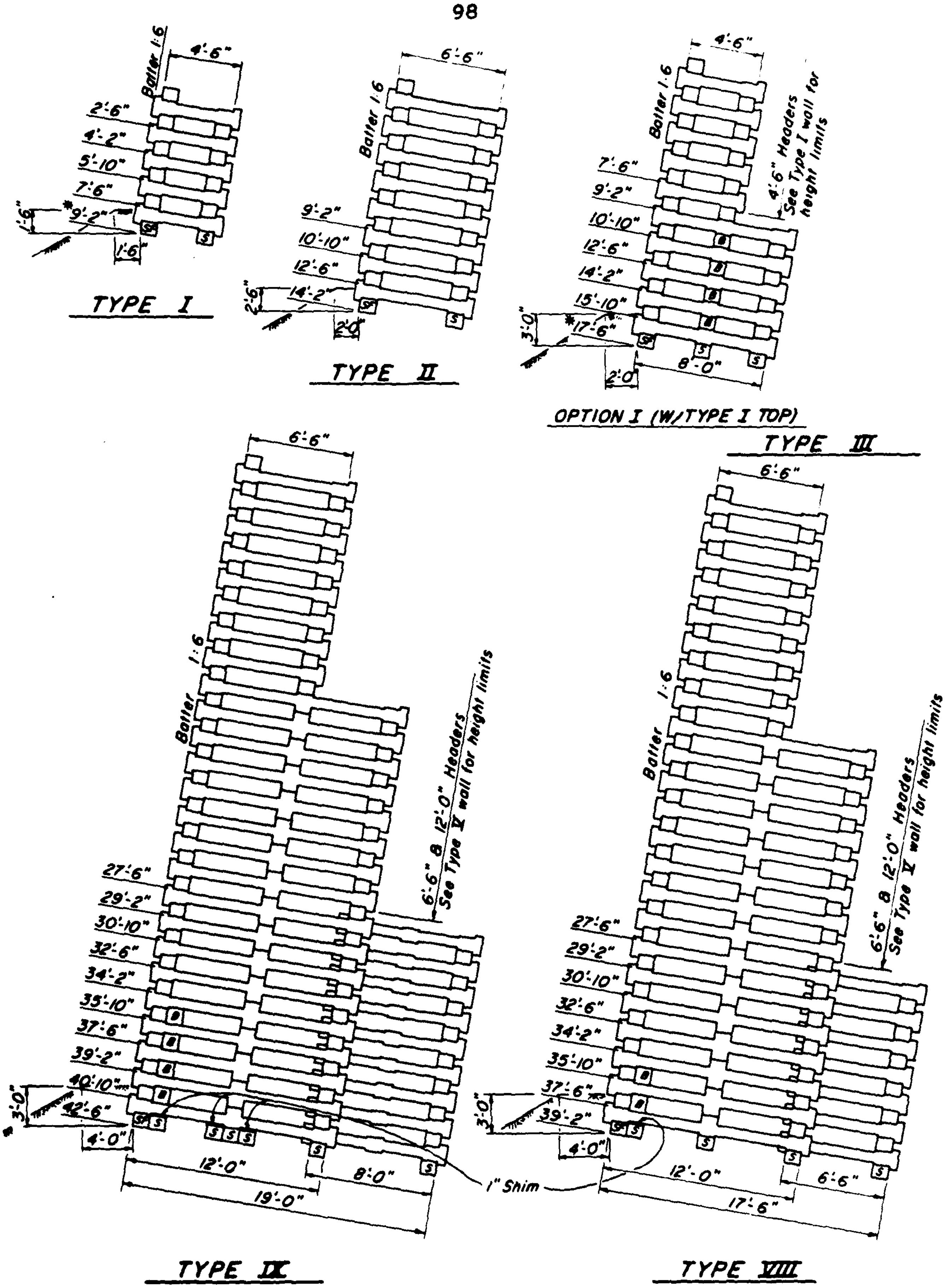


Fig. 2.9 Reinforced concrete crib Concrib walls showing examples of different arrangements of structural members for different wall heights. (*From Hilfiker, 1972*). Reproduced by permission of the Hilfiker Pipe Company, Eureka, California.

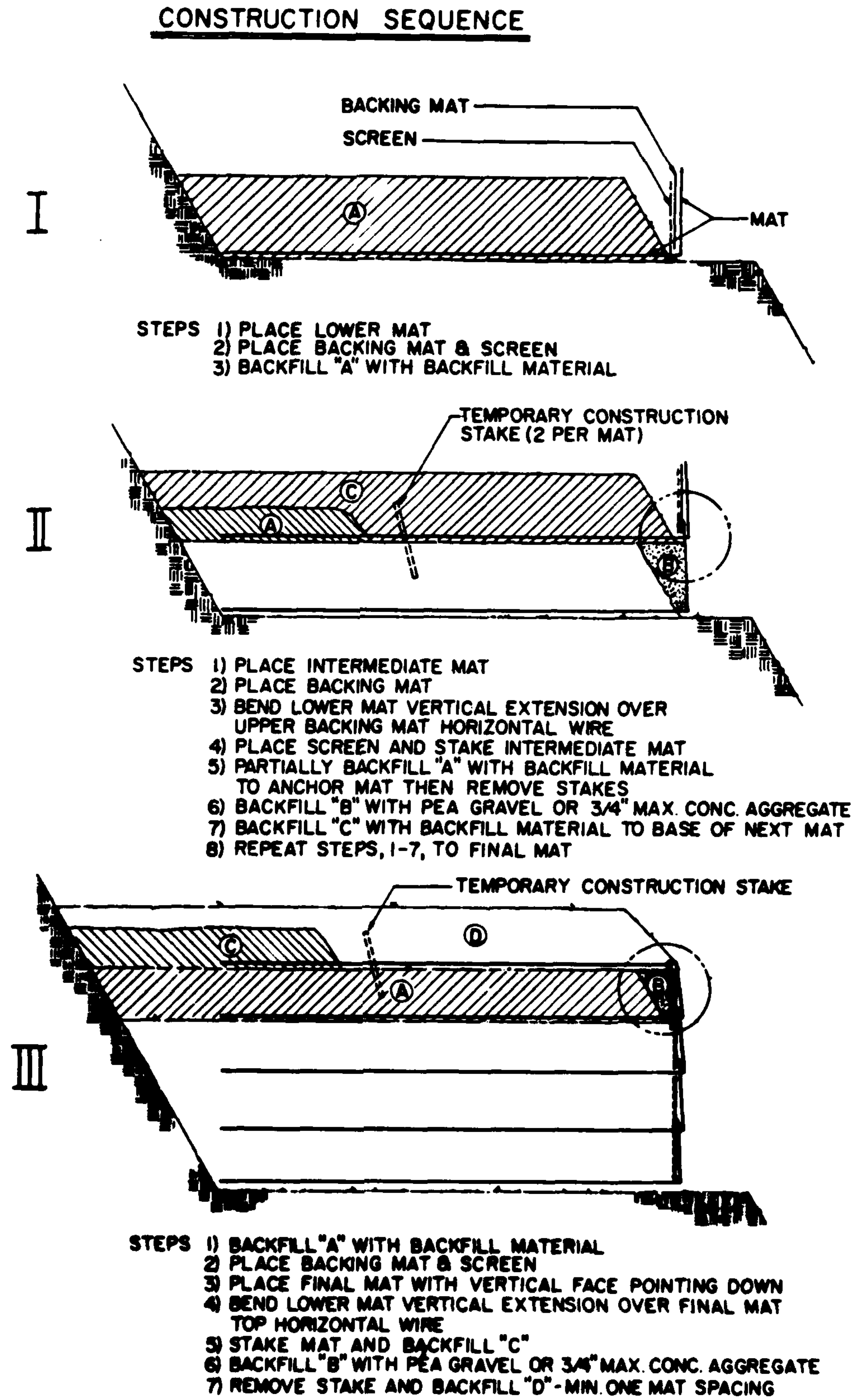


Fig. 2.10 Construction sequence for assembly and erection of a welded-wire wall. (*From Hilfiker, 1978*). Reproduced by permission of the Hilfiker Pipe Company, Eureka, California.

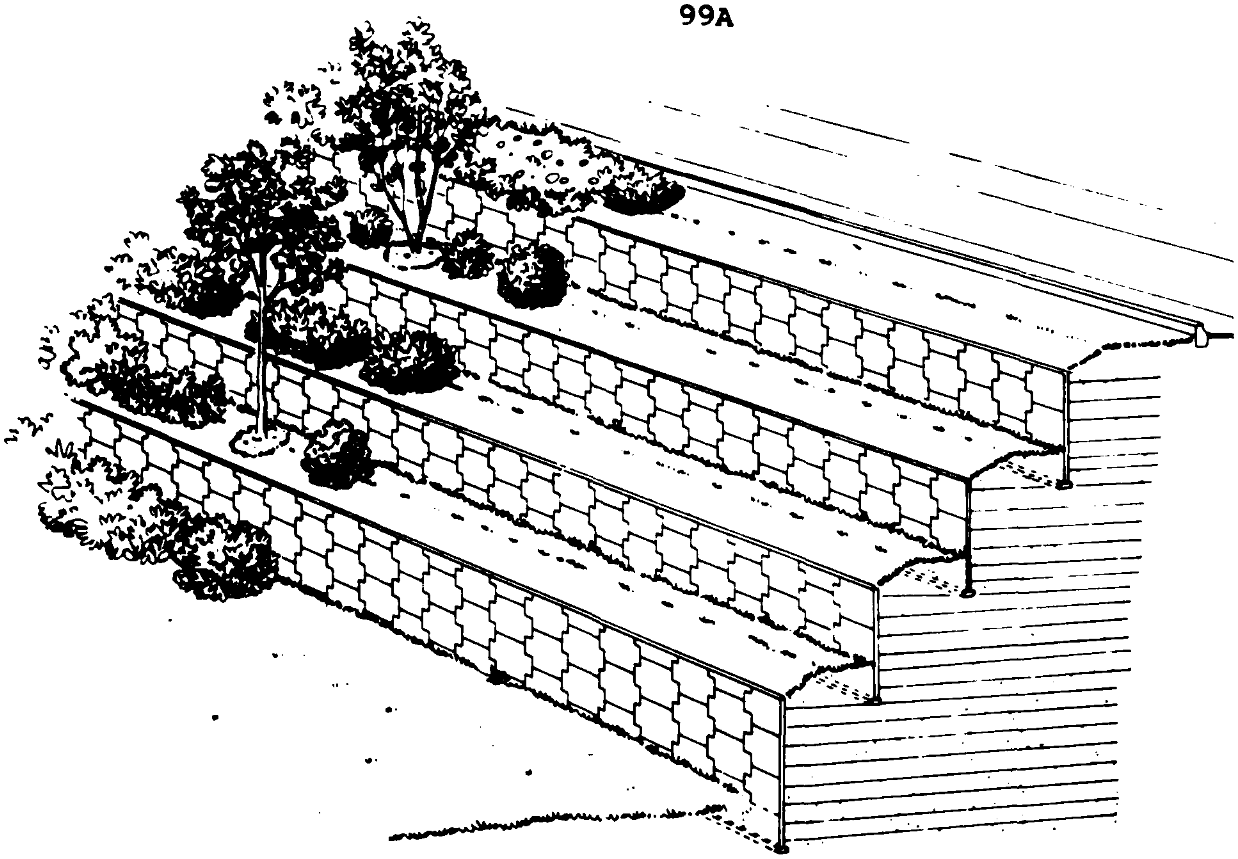


Fig. 2.11 Schematic illustration of stepped-back, Reinforced Earth wall with landscaped benches. (*From Walkinshaw, 1975*)

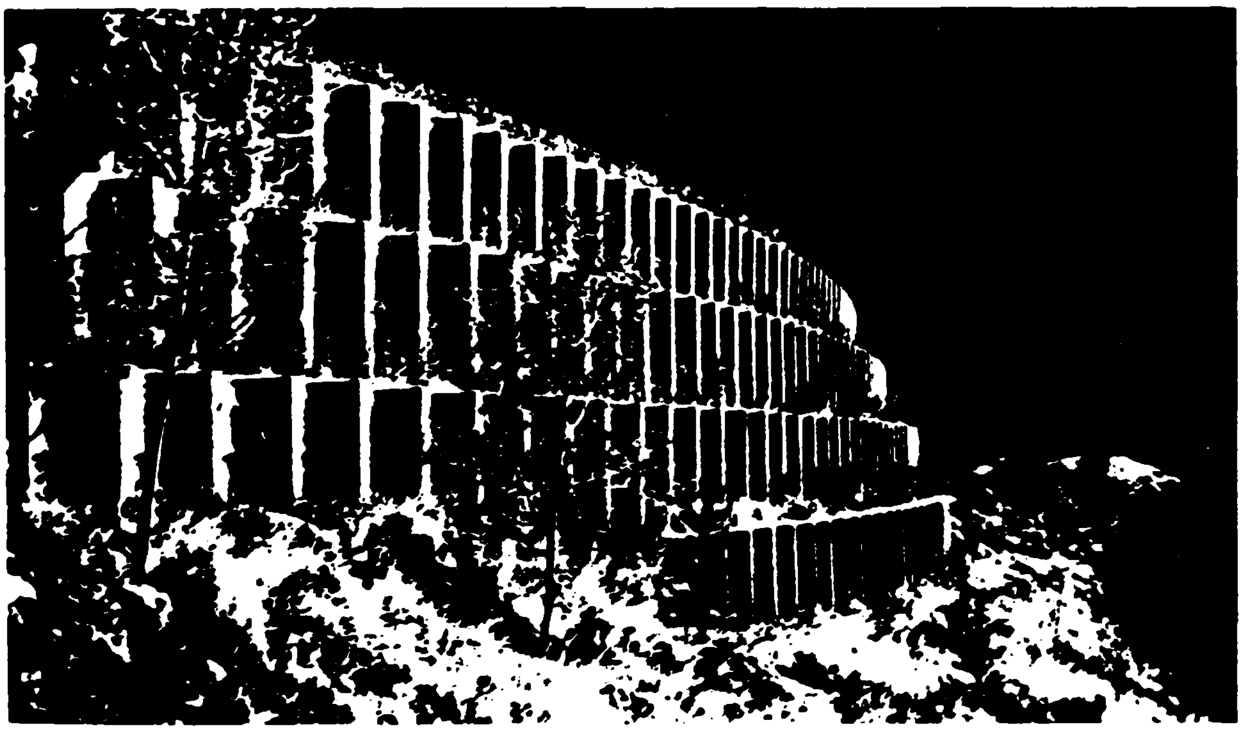


Fig. 2.12. Tiered, Reinforced Earth wall, Vail Pass, Colorado. (*Photo courtesy of the Reinforced Earth Company*)

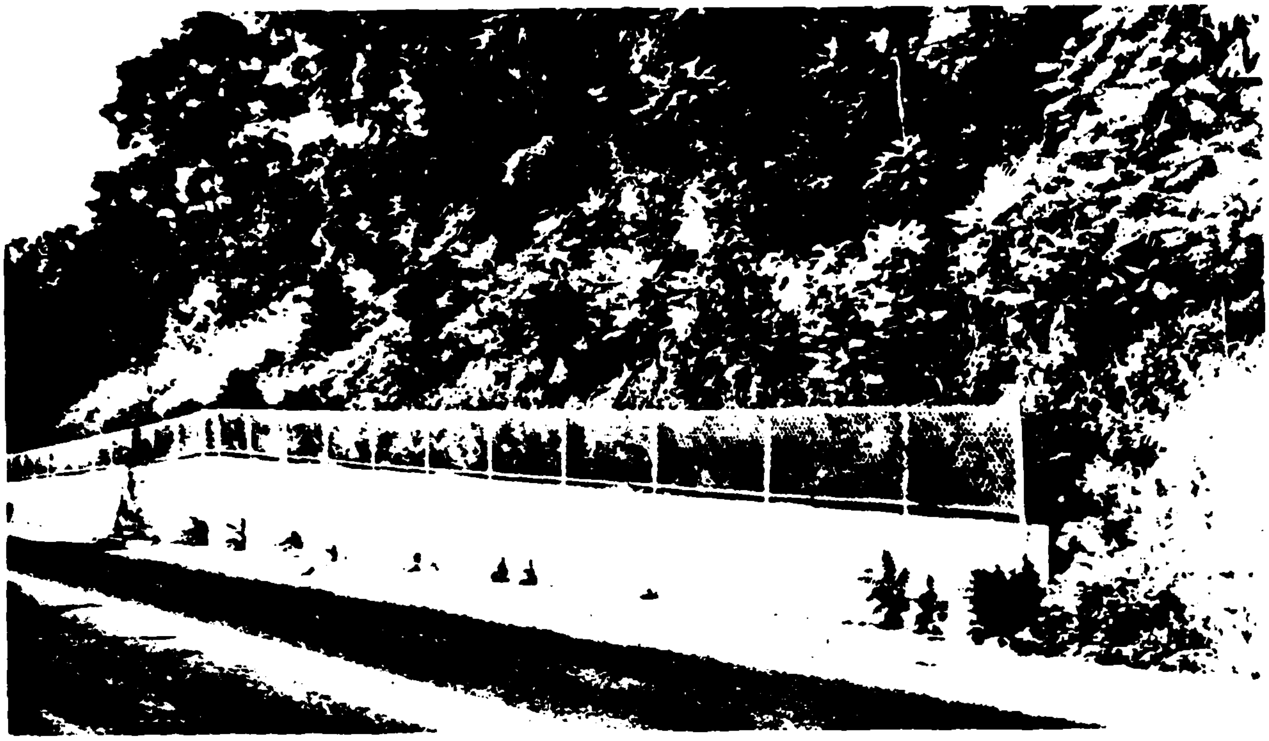


Fig. 2.13 Low toe wall at base of slope with plantings, grass and trees, on face of slope. Wall is reinforced concrete, cantilever design.

Fig. 2.14 Breast wall defense at toe of slope adjacent to roadway. Contour wattling and slope plantings are visible through holdown netting used on face of slope.

A crib retaining wall (Figs. 2.8C and 2.9) consists of a hollow, box like, interlocking arrangement of logs, timbers, reinforced concrete beams, or steel beams filled with soil or rock. A variation of this design known as a bin wall consists of steel boxes or bins that are bolted together and filled to form a wall. The cribwork can be vertical but is often tilted backward for greater stability. The crib members can be designed to have openings between them at the front face where plants can be established. Crib walls are relatively cheap and are usually flexible enough to tolerate some differential settlement. Structurally, cribs and bins are gravity walls and are designed accordingly. In addition, the crib itself must be analyzed for internal stability. That is, the structural members must be capable of resisting stresses caused by the crib fill and back fill.

Reinforced earth walls (Vidal, 1969) (Figs. 2.8E, 2.11, and 2.12) consist of a granular matrix or fill reinforced with successive layers of strips, usually of man-made materials. The strips are connected to facing elements that conventionally are either metal or concrete panels stacked atop one another. If lightweight, porous facings are used, vegetation can be established in the face of the structure. The reinforced volume can be regarded and analyzed as a coherent gravity structure (McKittrick, 1978). Internal stability requires in addition that the strips or ties be designed to resist tension failure or failing by pullout. The depth or length of reinforcement to prevent pullout typically ranges from 0.8 to 1.0 times the wall height. Reinforced earth offers several advantages in terms of flexibility, ease of construction, versatility and appearance.

Cantilever and counter-fort walls (Fig. 2.8B) are constructed from reinforced concrete and can be built to greater heights with a greater economy of materials than conventional gravity walls. Cantilever walls are used for heights up to 30 feet, and counter-fort walls are commonly used for heights greater than 25 feet. The cantilever wall is reinforced in the vertical direction to withstand bending moments (a maximum at the base of the stem) and in the horizontal direction to prevent cracking. The buttresses behind a counter-fort wall are are also heavily reinforced to resist tension. Both types are relatively expensive and require careful design and formwork.

Gabions (Fig. 2.8D) are wire baskets made of coarse wire mesh. These baskets are filled with stone and rock and stacked atop one another to form a gravity-type wall. Gabions depend mainly on the shear strength of the fill for internal stability and their mass, or weight, to resist lateral earth forces. Gabions are very flexible, easy to erect, relatively inexpensive and are a porous type structure that can be vegetated.

Welded-wire walls (Hilfiker, 1978) (Figs. 2.8F and 2.10) are a composite wire and granular soil structure. L-shaped, wire mesh sections are placed and connected between successive lifts of coarse granular back fill. The wire mesh provides both reinforcement in the back fill and containment at the face of the wall. Welded-wire walls are essentially gravity structures; they have features of both gabions and reinforced earth walls. They are relatively low-cost, easy to erect, flexible, and well adapted to vegetative treatment.

Pile walls (Fig. 2.8G) have occasionally been used as retaining structures. These may consist of a row of bored, cast-in-place concrete cylinder piles or, more typically, driven steel H-piles. Driven pile walls have been used to support low-volume roads (Schwarzhoff, 1975) where they traverse steep terrain characterized by weak but shallow residual soils underlain by a zone of weathered rock that increases in competency with depth. The use of driven piles in this case avoids excessive bench excavation that would be required for a bearing-type wall.

Tie-back walls (Fig. 2.8H) essentially consist of a relatively thin flexible facing connected to a dense network of anchored tie rods. They can consist of light-gauge steel sheeting held in place by horizontal 5/8-inch diameter steel rods installed perpendicular to the sheeting with 8 inch square anchors welded on the far end. They can also be made with U-shaped annular panels similar to half round culverts for facing with steel tie backs connected to a continuous strap anchor.

Breast walls (Fig. 2.14) can be considered as a type of gravity wall constructed with irregularly shaped rock. They are erected on firm ground and placed against a slope with only a small amount of fill behind them. Breast walls are not designed or intended to resist large lateral earth pressures; they are more like a revetment than a retaining wall. Breast walls are quite porous which provides opportunity for plant growth in the voids and interstices of the wall.

Although any one of the several types of retaining walls is often adequate, specific criteria will usually lead to the selection of one wall that is best suited for the job. The criteria may include environmental concerns, construction problems, management implications, site constraints, esthetics, and economics. In the case of biotech systems there are certain criteria that are especially important. The requirement that the structure blend in with the environment and its ability to incorporate vegetation are particularly important attributes. Vegetal incorporation, ease of construction, flexibility, and low cost limits the types of retaining structures for biotech systems to rock breast walls, gabions, crib walls, welded wire walls, and reinforced earth.

Although many retaining walls are constructed for specified slope-loading conditions and heights from "standard" designs, this can be a very dangerous and costly practice. Retaining wall design is often very complex. Standard designs can be used safely if, and only if, the stated conditions on which the designs are based also pertain at the site in question (which in reality never really happens). I do not intend to discuss the details of a good design but I do want to address two very important aspects of retaining wall design that are sometimes given too little thought during the design phase and are occasionally "overlooked" during the construction phase. Those two considerations are the type and compaction of backfill and the effect of ground water in the backfill.

If careful consideration is not given to the type and compaction of backfill, the work associated with design calculations could be for nothing and the wall might just as well be constructed by rule of thumb.

The compaction of a retaining wall backfill helps to reduce eventual subsidence of the fill. However, the effect of such compaction on lateral earth pressure must be considered. Compaction tends to crowd the soil against the wall, and in this respect, is equivalent to forcing the wall against the backfill. Consequently, compaction tends to increase the lateral earth pressure, at least while this operation is in progress. If compaction is completed without damage to the wall, i.e. too much pressure, the other effects of compaction depend largely on the nature of the backfill.

With a densely packed granular backfill, a slight yielding of the wall is sufficient to reduce the lateral pressure to the active value once compaction is complete. After that, there's no reason to expect any significant lateral expansion of the backfill and consequent increase in lateral pressure. (Except perhaps effects associated with freezing which will not be discussed.)

Compaction of a granular backfill is on the whole beneficial, assuming, of course, the wall is not damaged during compaction operations. The tendency for the lateral force to be increased by the increase of the unit weight of the soil is offset by the significant increase in internal friction of the granular material due to compaction.

Often a granular backfill may not be possible. Instead, expansive clay soils may be involved. If an expansive soil is placed behind the wall dry and later becomes wet and expands, the wall usually moves, sometimes enough to cause structural damage. Since the soil shear strength is no longer acting to hold the soil back from the wall, but pushed against the wall, the passive pressure condition applies. However, designing for passive pressure is not sufficient because in the expansion process the soil structure changes, increasing cohesion and decreasing the friction angle. It is impractical to design a wall to restrain expanding clay. If expansive clay must be used, ensure it is fully expanded when it is being backfilled, by designing for saturated conditions and "wetting" (not flooding) during back filling. Loose back filling of dry clay will not solve the problem since the entire soil skeleton made up of individual clay clods will expand and exert pressure on the wall. Heavy compaction or over-compaction of a retaining wall clay backfill is generally recognized to be undesirable. A heavily compacted clay tends to expand over a rather prolonged period and in so doing creates extreme lateral pressures.

The preceding discussion addresses the most desirable and least desirable types of backfill. In designing retaining walls on a semi-empirical basis, backfill material may fall into one of five categories:

<u>Excellent</u>

1. Coarse-gravel soil, high permeability (clean sand or gravel).
2. Coarse gravel soil, low permeability.
3. Residual soil with stones, fine silty sand, and granular material with clay content.
4. Soft clay, organic silts, or silty clays.
5. Medium or stiff clay, chunky but protected to prevent excess water entering spaces between chunks (if infiltration cannot be avoided, this material should not be used at all).

<u>Poor</u>

Even with the best possible specification of backfill material, a dishonest contractor or poor construction methods may negate the benefit of a carefully selected material. Modern practice calls for placement of the fill in layers, not to exceed 12 inches after compaction, and compaction by equipment suited to the type of soil. If possible, excavation for retaining walls should be made after compaction. Normally, however, walls must be constructed before the final lifts of fill are placed requiring care to avoid over-compaction. At higher water contents, the CBR-Water Content curves cross, and a lower strength will be obtained with higher compaction energies wet of optimum. This effect is known as over-compaction. Reference Fig. 2.15.

Over-compaction can occur in the field when wet of optimum soils are proof rolled with very heavy, smooth wheeled rollers or an excessive number of passes are applied to the lift--even good material can become weaker. Over-compaction can be field detected by observing the behavior of the soil immediately under the compactor or the wheel of a heavily loaded scaper. If the soil is too wet and the applied energy too great, pumping or weaving of the fill will result as the wheel shoves the wet weaker fill ahead of itself. Also, sheeps foot rollers won't be able to "walk out." As a rule, heavy compaction equipment should be avoided completely in retaining wall construction. Back fill materials are sometimes dumped into place loosely and then flooded in an attempt to compact them. This procedure should not be permitted. In cohesive backfills, it weakens and softens the soil leading to inadequate support and subsidence. In uniform or fine soils some benefit may be gained by causing collapse of the extremely loose unstable zones associated with bulking and leaving the sand at a density index close to zero. If the back fill is a well graded sand and gravel, bulking is negligible and no benefit at all is derived from flooding.

The only way to be reasonably certain you're getting what you paid for during back fill operations is to conduct

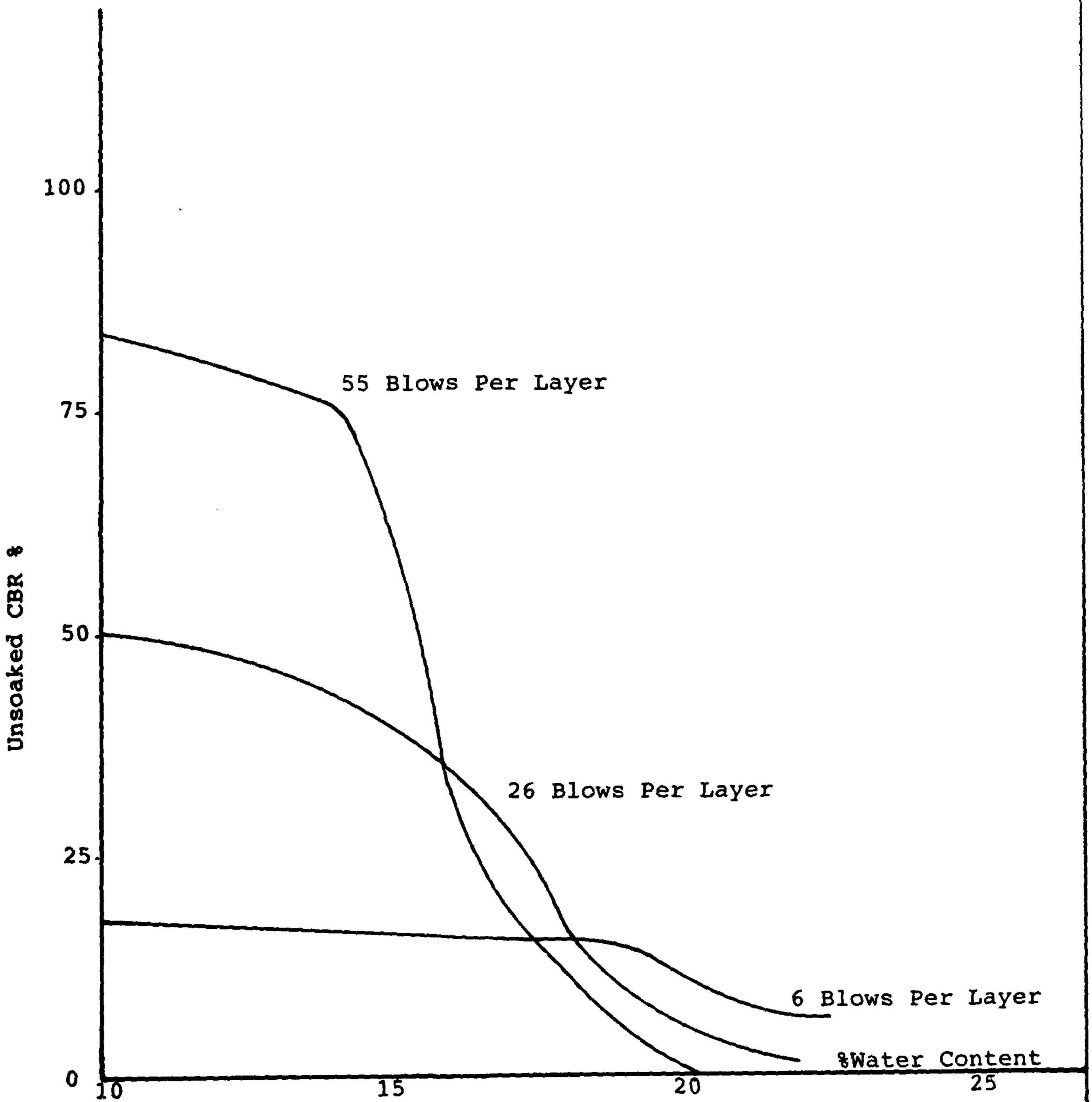


Fig. 2.15 Strength as measured by the CBR versus water content for lab impact compaction (Turnbull and Foster, 1956).

field tests. The test site should be typical of the compacted lift and borrow material. Typical specifications call for a new test for every 1000 to 3000 cubic meters or when the borrow material changes significantly. Field tests should be made at least one or two compacted lifts below the already compacted ground surface.

Field tests can be destructive (involves excavation and removal of some fill) or nondestructive (density and water content of the fill are determined indirectly). Both methods have good and bad points but time and cost generally dictates which one to use. Assuming backfill soil properties are the very best, the designer must still be careful to consider the effects of water and recognize the impact of a saturated backfill.

A retaining wall backfill is usually exposed and relatively vulnerable to saturation either by infiltration of surface water or seepage unless preventive measures are taken. When water is present in the backfill, it can change the problem considerably. In the case of flowing water, there will be a hydrostatic pressure plus differential pressure for which flow nets may need to be drawn to establish the water pressure. If the wall is impervious, there may be simply hydrostatic pressure. If it is capillary water, or disconnected soil pore water, the unit weight is increased. In the general case, if the water table rises behind an impervious wall, the lateral pressure increases.

From an analysis of the Rankine equation for a granular soil, it can be seen that, with other factors equal, water is far more serious as a contribution to lateral pressure than the backfill material. Consider the extreme case illustrated in Fig. 2.16.

Under these conditions, the water is capable of creating a pressure on the wall independent of the lateral earth pressure. In fact, the water pressure alone would be the same as though there were no backfill at all.

$$p_1 = p_{Water} = \gamma_w H \qquad (2.4)$$

and

$$p_2 = p_{soil} = \gamma_{sub} H\ Ka, \qquad (2.5)$$

Usual values of Ka range from 0.25 to 0.40. The unit dry weight of many soils ranges from 105 to 120 pcf. However, water has a constant unit weight of 62.4 pcf and no coefficient to diminish that value. Consequently, p_1 can be significantly greater than p_2.

Both the water pressure and the soil pressure will act horizontally with hydrostatic distribution as shown in Fig. 2.16. Therefore, the total pressure is the sum of the two: $p_T = p_1 + p_2$. The difference between this value and the value of the backfill alone is far greater than the difference between the unit weight of fully and partially saturated backfill material which may vary by only a few percent. The essential consideration is not the quantity of water in the backfill and its contribution to the total weight of the fill but the condition of the

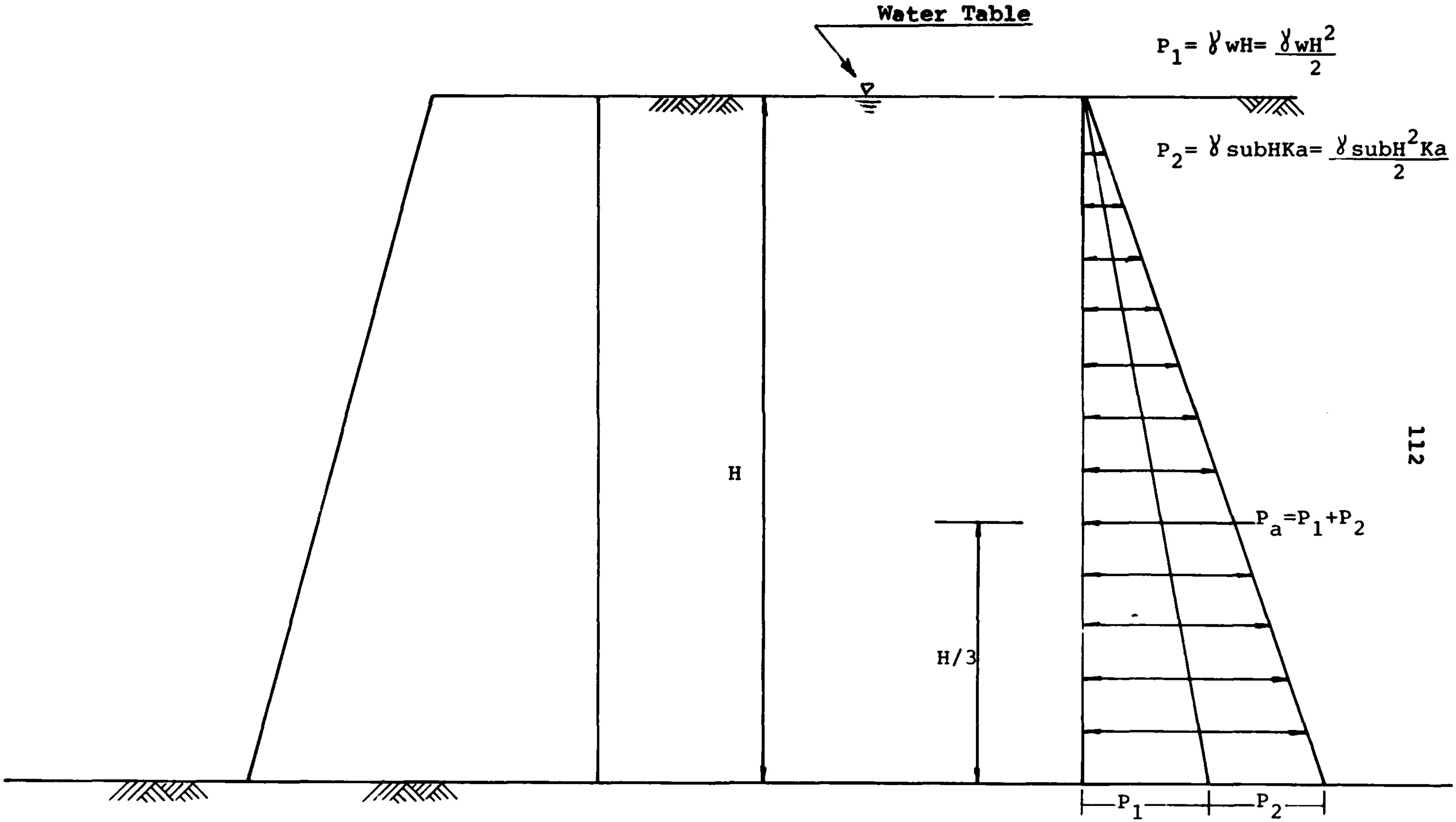


Fig. 2.16. The effect of water in the backfill.

water, i.e. whether it is held in the soil by capilliary or whether it is free water, the latter condition capable of exerting an independent lateral pressure. A change of only a few percent in the degree of saturation may change the condition and drastically increase the total lateral pressure against the wall. Similar reasoning may be made for a clay soil. The conclusion is the same. Backfill material should be free draining, with provisions incorporated into the wall to eliminate any accumulation of water behind the wall.

Since retaining walls are not expected to resist water pressure in addition to the earth pressure, well designed walls are provided with means for draining the water that would otherwise accumulate in the backfill. The drains commonly consist of pipes known as weepholes which have a diameter of 6 to 8 inches and which extend through the stem of the wall and are protected against clogging by pockets of gravel in the backfill. The drains are spaced at about ten feet intervals both vertically and horizontally.

Weepholes alone are not very efficient, especially in draining semipervious backfills. Unless the pockets of gravel satisfy the requirements for a filter they are likely to become clogged. In freezing weather the outlets may ice up. Consequently, total reliance on weepholes is somewhat risky; a continuous back drain (longitudinal drains along the back face) should be used if possible. The drain consists of a perforated pipe having a diameter not less than six inches. The pipe, which must be surrounded by a filter, usually terminates at an open ditch where it's accessible for cleaning.

Criteria for filter material to place at the entrance of the weepholes or over drain pipes are as follows:

For soils:

$$\frac{D_{15}\ \text{filter}}{D_{85}\ \text{protected soil}} \leq 5 \qquad \frac{D_{15}\ \text{filter}}{D_{15}\ \text{protected soil}} \geq 5$$

For slots:

$$\frac{D_{85}\ \text{filter}}{\text{slot width}} > 1.2$$

For circular openings:

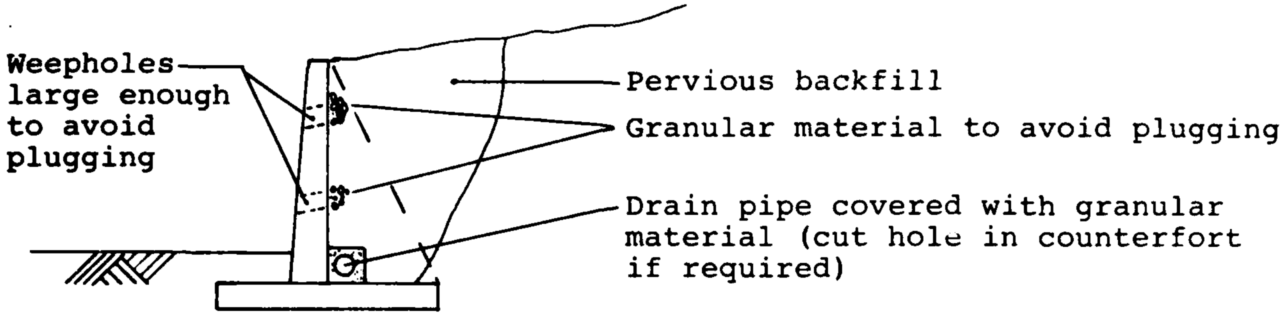
$$\frac{D_{85}\ \text{filter}}{\text{Hole diameter}} > 1.0$$

D_{15} and D_{85} are the grain diameters which are larger than 15 and 85 percent, respectively, of the material from the grain-size distribution curve.

Semipervious backfills require strips of filter material in addition to the drain pipes and/or weepholes. In fine grained backfill, a drainage blanket or double blankets are necessary. It is always good practice to place an impervious soil in the upper layer of backfill to reduce rain water infiltration. Common types of retaining wall draining systems just discussed are shown in Fig. 2.17.

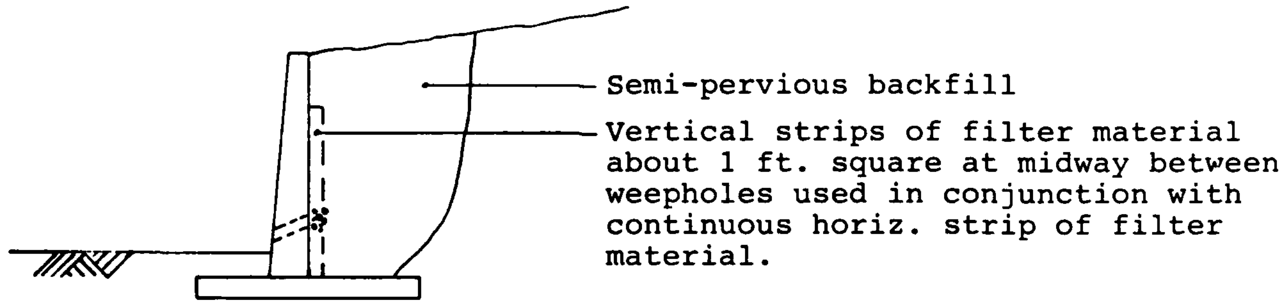
In summary, well designed retaining walls require the selection of properly compacted backfill material and the planning of adequate draining provisions. <u>These considera-</u>

Fig. 2.17. COMMON TYPES OF RETAINING WALL DRAINAGE

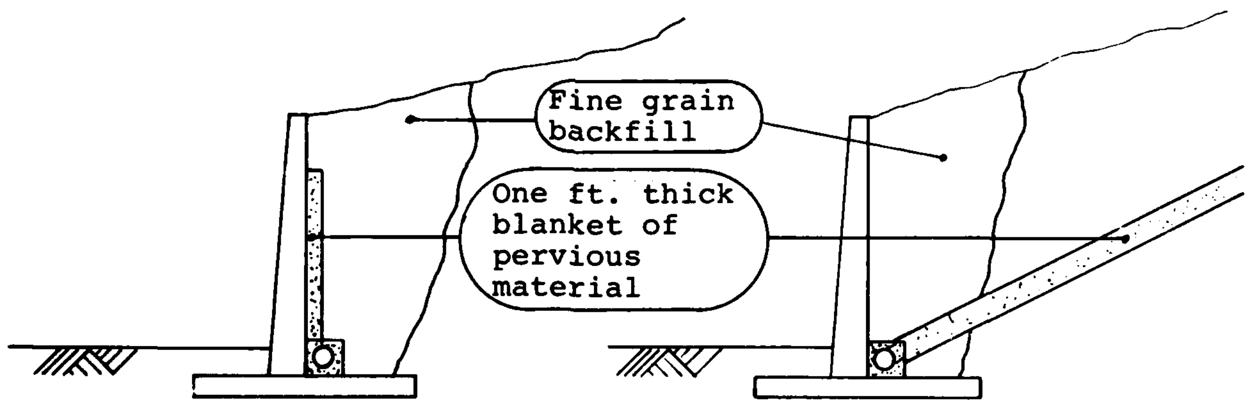


WEEP HOLES AND LONGITUDINAL DRAIN PIPES

Note: If weepholes are used with counterfort walls, at least one weephole should be located between counterforts.

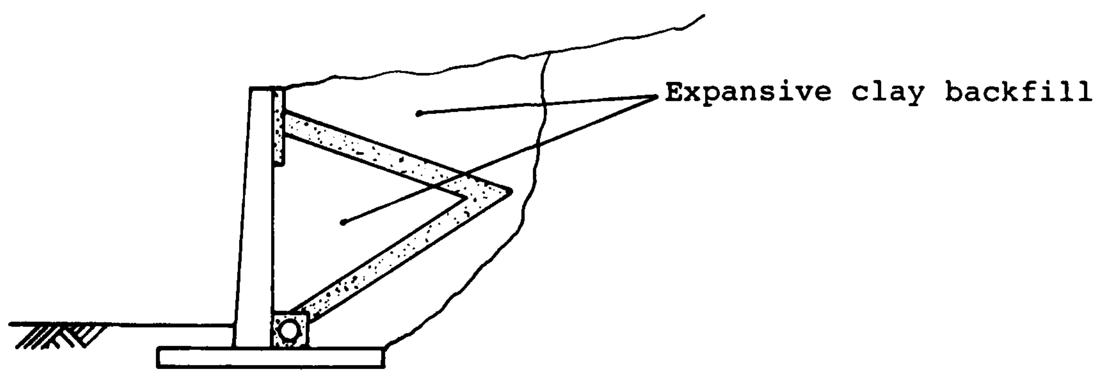


WEEPHOLES WITH FILTER STRIPS



BLANKET DRAIN

BLANKET DRAIN



DOUBLE BLANKET DRAIN

tions are more important than a correct evaluation of the earth pressure acting on the wall. Backfill material should be carefully selected. Ideal backfill is purely granular soil (clean sand, gravel, or soil and gravel) containing less than about five percent of very fine soil, silt, or clay particles. In addition, backfill should be compacted to prevent large ground subsidence due to consolidation under its own weight. The amount of compaction required for each job depends on the material used and on the nature of the job. More strict control is necessary where cohesive backfill is used. Caution should be exercised not to overcompact the backfill. Regardless of the type of material used, the backfill drainage system must be adequate to discharge infiltrated rain water. The amount of drainage work depends upon the permeability of the backfill materials.

The biotech systems we've discussed so far have been primarily concerned with the prevention of mass wasting or slope protection with secondary consideration given to preventing surface erosion. However, there are biotech systems whose primary function is to prevent surface erosion. One of those is a revetment.

A revetment is a facing placed on a slope to armor it and prevent scour and erosion. They are typically used along stream channels, waterways, or inland lakes to prevent or control bank erosion. A revetment offers some resistance against mass movement; however, its primary purpose is to prevent loss of bank material by wave action, ice scour, and fluvial erosion. Revetments are normally placed on slopes no steeper than 1½:1 (30 degrees or 57 percent). Steeper slopes usually require a retaining wall or other structure (Gray and Leiser, 1982).

Revetments have typically been constructed from hand-placed, dumped, or derrick-placed rock (riprap). Today many types of structural facings are in use which include gabion mattresses, rubber tire networks, sand-cement sacks, articulated precast concrete blocks, cellular grids, and riprap.

Riprap is one of the most common and effective methods of bank protection. It consists of a carefully placed layer of stones and boulders and is used under most conditions where bank erosion occurs. The limiting factors in the use of riprap are availability of suitable-size rocks; difficulty and expense of quarrying, transporting, and placing stone; and the large amount of material needed where streams are deep.

Riprap revetments are particularly effective at sharp bends in a stream channel with a less than 300 feet radius, constructions such as bridges and culverts where velocities are increased, drainage ditch openings and exits, etc. Most riprap is placed on a filter blanket of smaller size material to prevent washout of fines or bank material through the riprap. The area to be covered with a filter blanket should be smooth and an even thickness of filter material should be placed on the prepared surface. Care must be taken when placing the riprap to ensure the blanket is not ruptured or displaced. Natural vegetation will often invade and establish itself in riprap. Alternatively vegetation can be introduced by spot seeding and insertion of cuttings. The establishment of vegetation is key to the life and effectiveness of riprap or any other type revetment. Some examples of biotech systems including revetments are shown on the following pages (Gray and Leiser, 1982).

Fig. 2.18. Vegetated concrete, crib wall. Vegetation was planted in openings between headers at face of wall.

Fig. 2.19 Vegetated gabion wall. Green willow cuttings which were inserted through gabion into backfill have rooted and sprouted.

Fig. 2.22.

Highway cut stabilized by stepped-back, timber retaining wall. Woody shrubs (*Forsythia* spp.) have been planted on the benches.

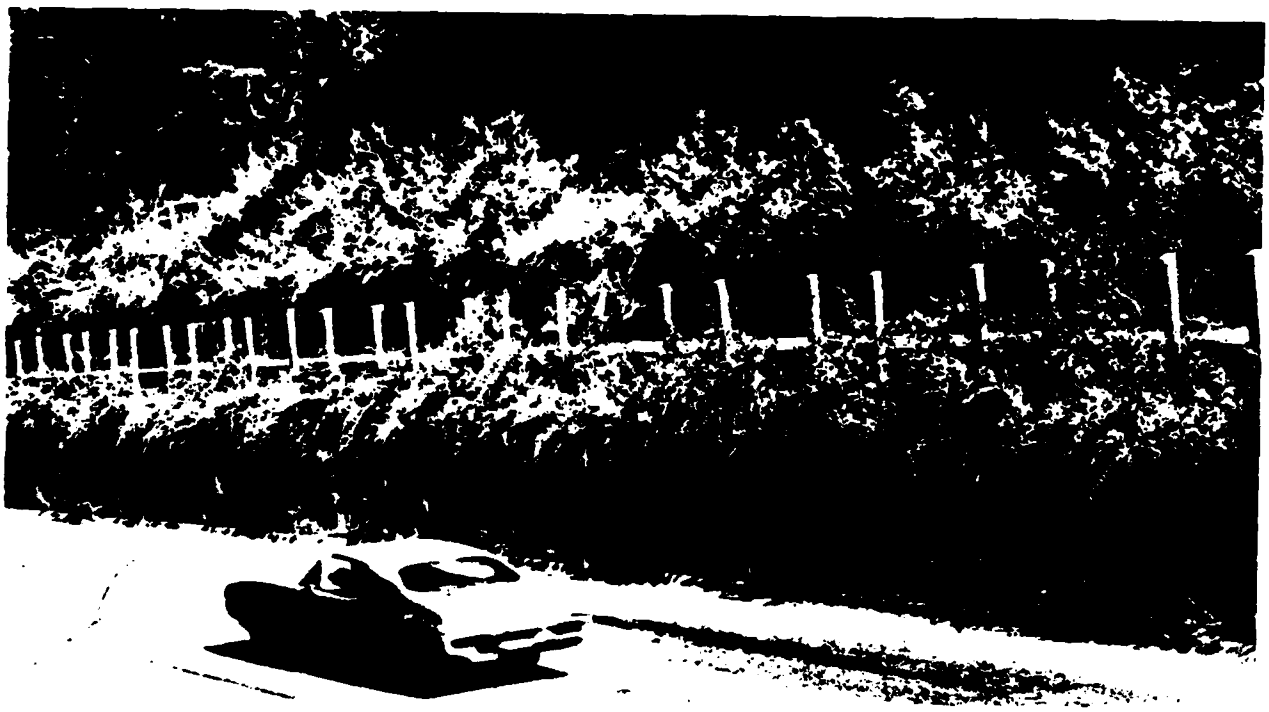


Fig.2.23. Tiered retaining wall with pedestrian path and vegetation on the horizontal benches.

Fig. 2.20. Vegetated concrete block revetment along drainage channel. Revetment consists of slotted, concrete blocks laid on the slope. Vegetation is volunteer or naturally occurring.

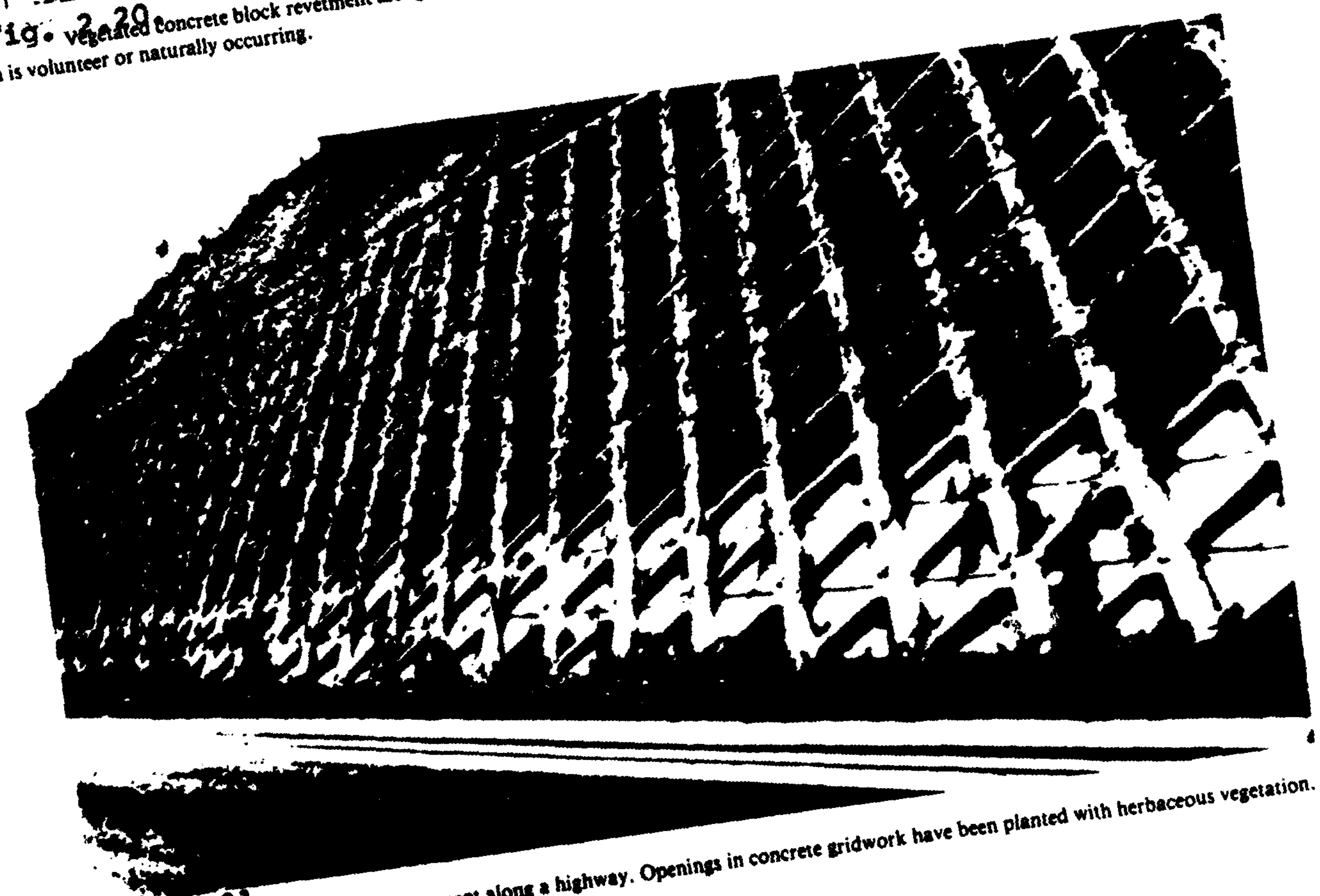


Fig. 2.21. Vegetated cellular grid revetment along a highway. Openings in concrete gridwork have been planted with herbaceous vegetation.

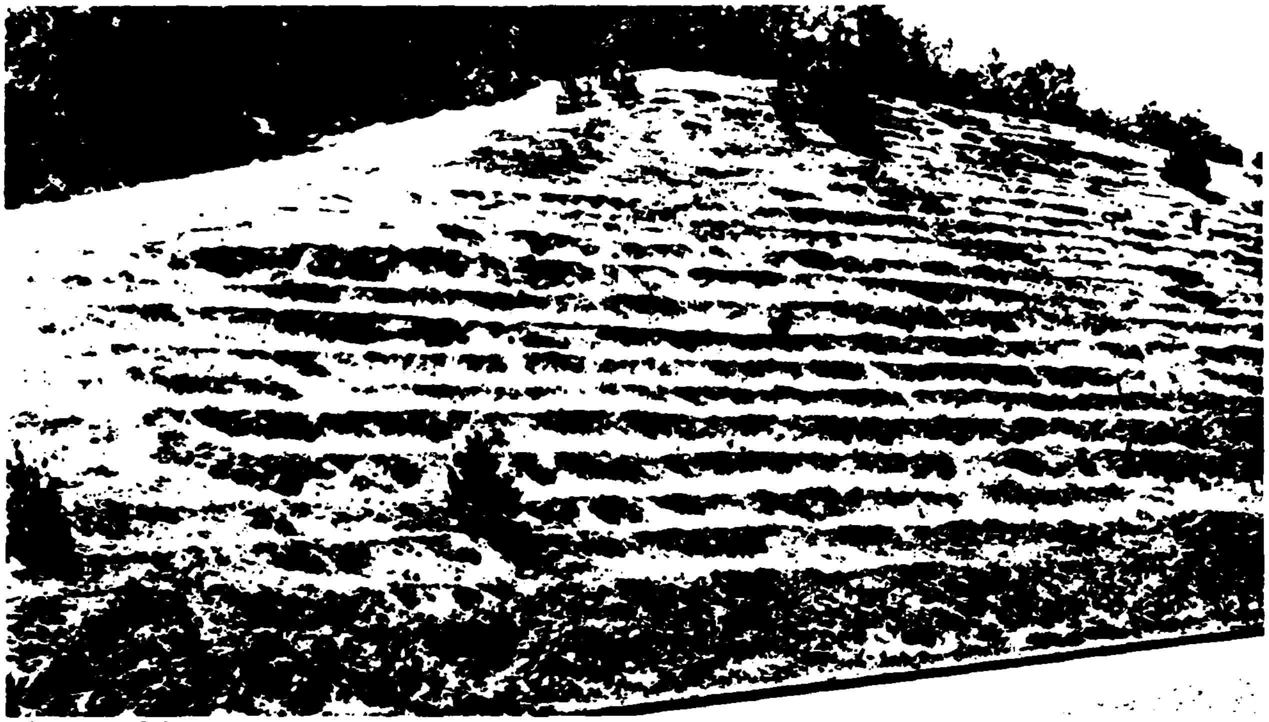


Fig. 2.24. Terraced, cut slope adjacent to highway aids in erosion control. Horizontal steps were seeded with native grasses. Interstate Highway, #80, near Colfax, California.

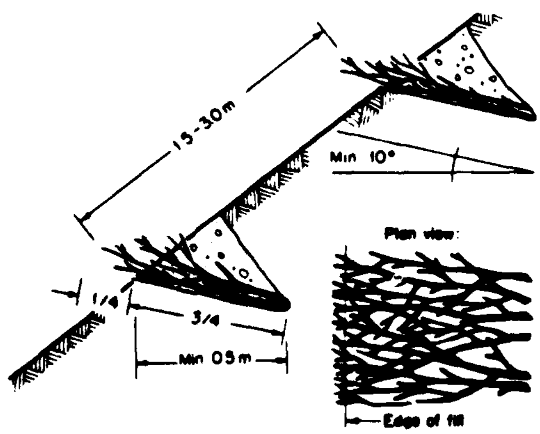


Fig. 2.25. Slope stabilization by brush-layering method. Cut brush or green branches of easy-to-root species such as willows are placed on contour benches across a slope as shown in the diagram.

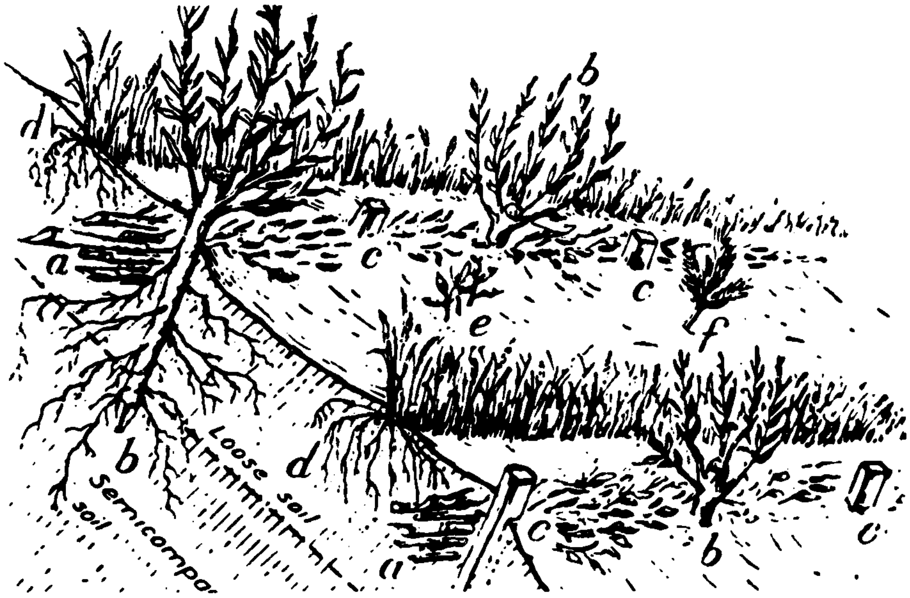
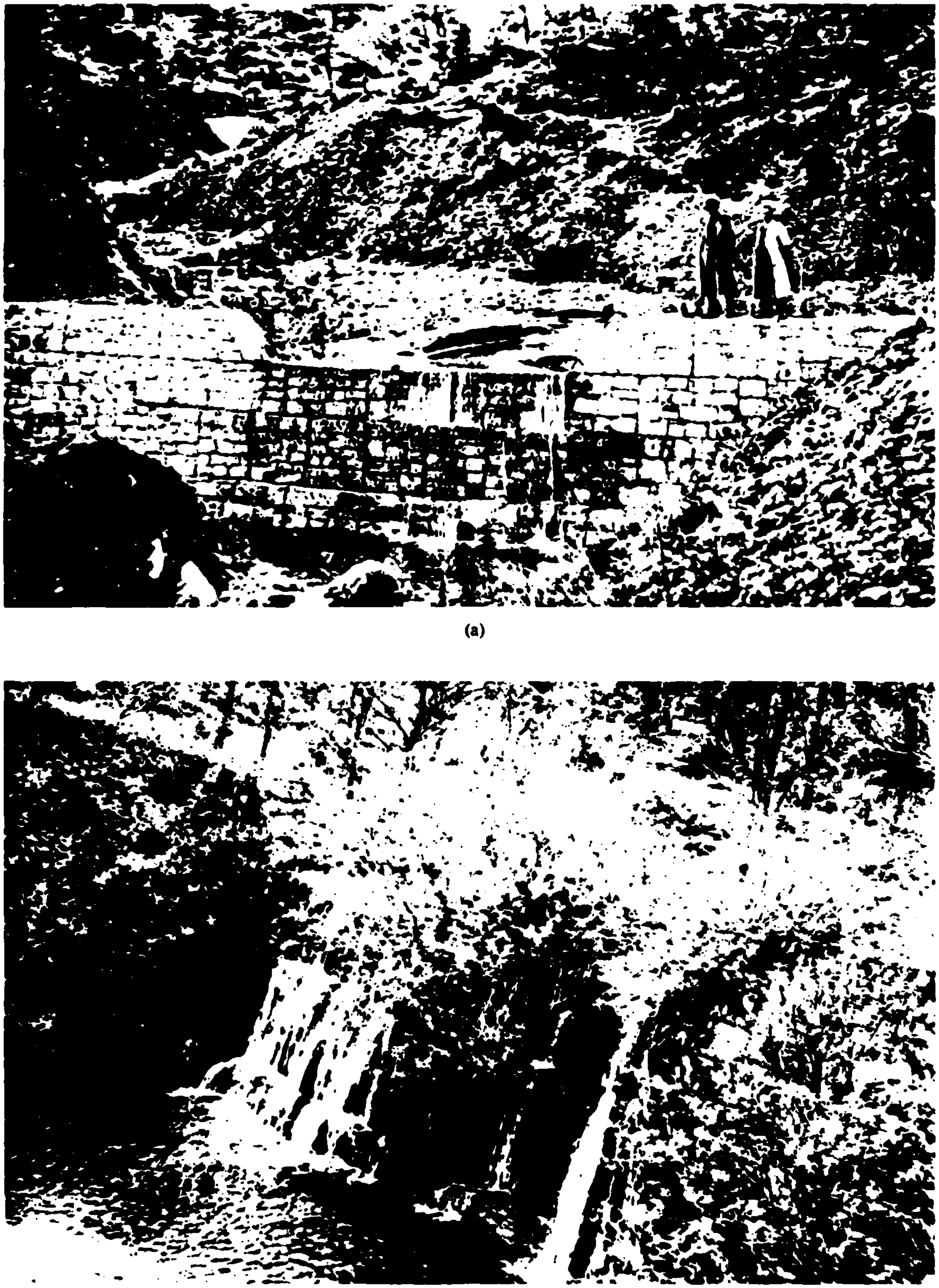


Fig. 2.26.

Schematic illustration of wattling installation. Shown are (a) stems of cut brush "wattles"; (b) live willow stakes that have rooted and sprouted; (c) inert construction stakes driven through wattles; (d), (e), and (f) vegetation (grasses, shrubs, and trees) established on benches between wattles.

Fig. 2.27.

Steep slope stabilized by willow staking and wattling. This previously denuded highway cut, about 1 acre in area, was producing over 100 cu yd of erosion per year. Unrooted willow cuttings were planted on 2-ft centers, and about 1100 lineal ft of willow wattling were also installed. The stuck cuttings and willow wattles have rooted and sprouted vigorously as shown. Erosion and bank sloughing problems have been virtually eliminated at this site. State Highway #89, near Luther Pass, California.



(a)

(b)

Fig. 2.28.

F... ? Gabion check dam used to control serious channel scour and erosion. Porretta Terme, Italy. (a) After construction in 1930; (b) after 29 years. Vegetation has become established in the channel and structure itself. (*Photos courtesy of Maccaferi Gabions, Inc.*)

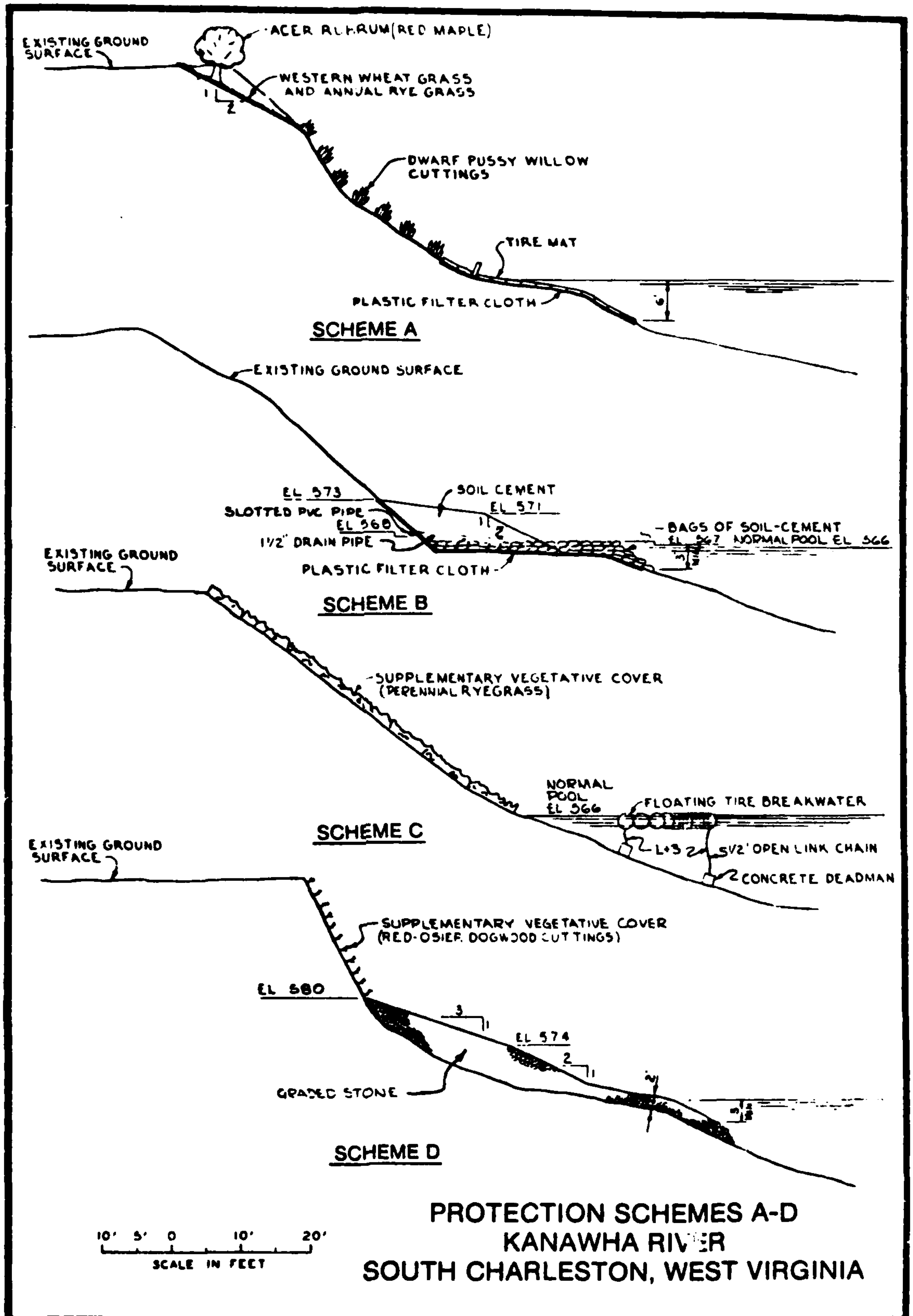
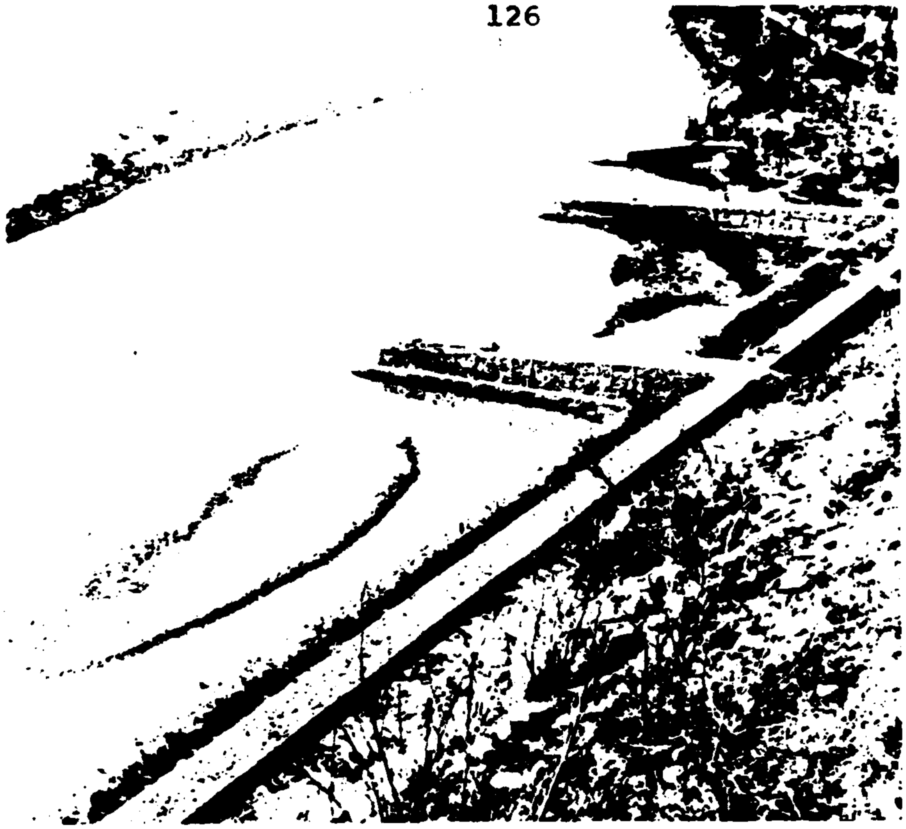


Fig.2.29. Use of slope plantings in conjunction with structural-mechanical measures to protect streambank along the Kanawha River. (*From U. S. Army Corps of Engineers, 1978*)



(a)

(b)

Fig. 2.30. Gabion groin and revetment system used to stabilize streambank. Sasso Marconi, Bologna, Italy. (a) After construction in 1939; (b) 20 years later, vegetation established in the porous defense works; (c) 30 years later, vegetation completely obscuring structures. (*Photos courtesy of Maccaferi Gabions, Inc.*)

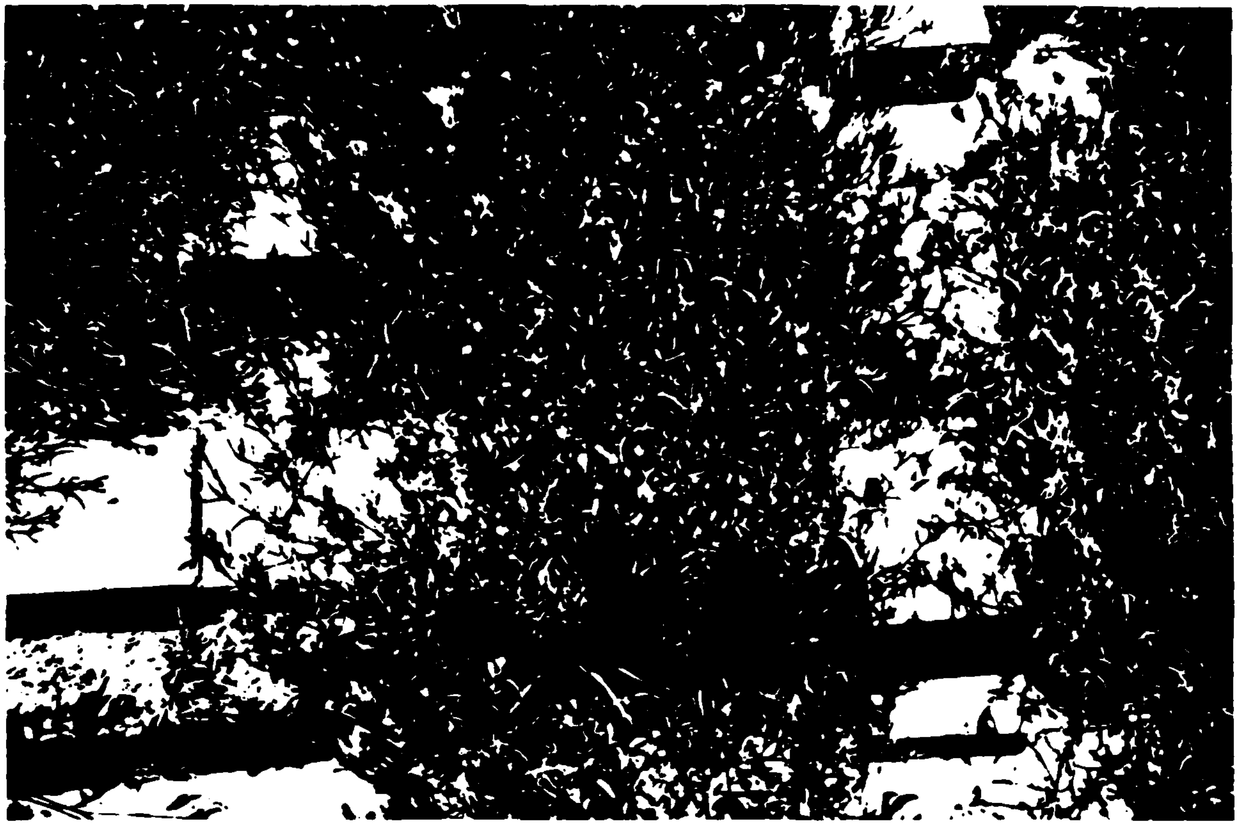


Fig. 2.31. Openings between headers in a Humes Minicrib wall planted with flowering plants.

Fig. 2.32.
Concrete crib wall landscaped with flowering shrubs and plants. (*Photo courtesy of Humes Ltd.*)

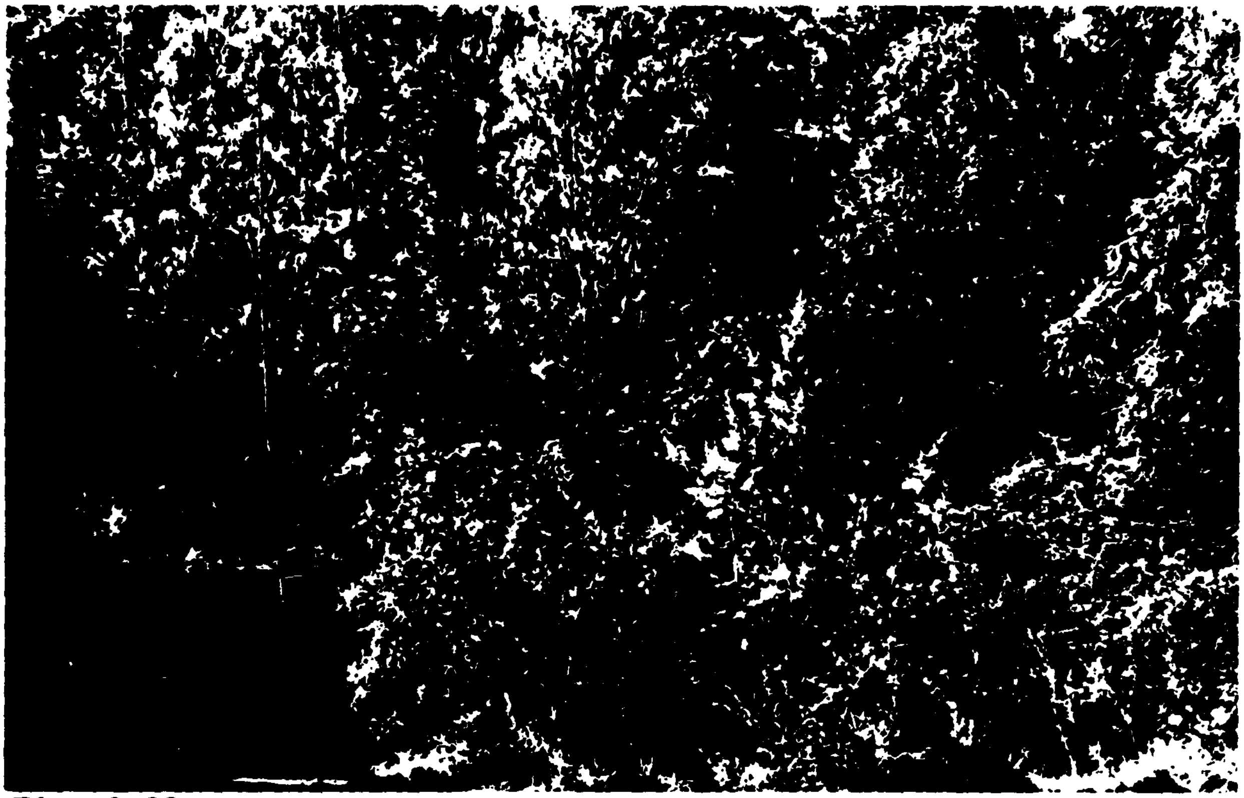


Fig. 2.33.
View of tiered, concrete crib Concrib retaining wall system. Benches between successive tiers have been planted and landscaped with native shrubs and trees. Oakland, California.

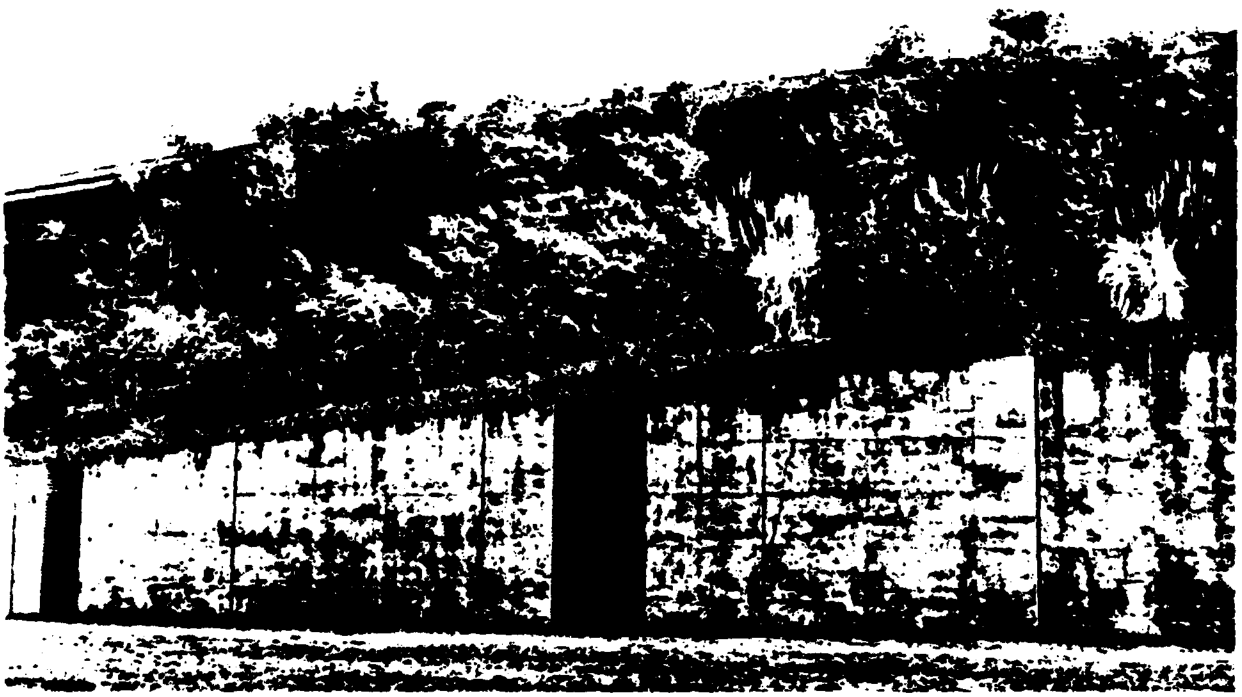


Fig. 2.34. Highway embankment supported by retaining walls. Concrete cantilever wall is visible in foreground with landscaped concrete crib wall above. Vegetation is growing both on the bench between walls and in the open bays of the cribbing. Oakland, California.

Fig. 2.35.
Tiered, Reinforced Earth wall, Heart O' The Hills Road, Olympic National Park, Washington. (*Photo courtesy of the Reinforced Earth Company*)

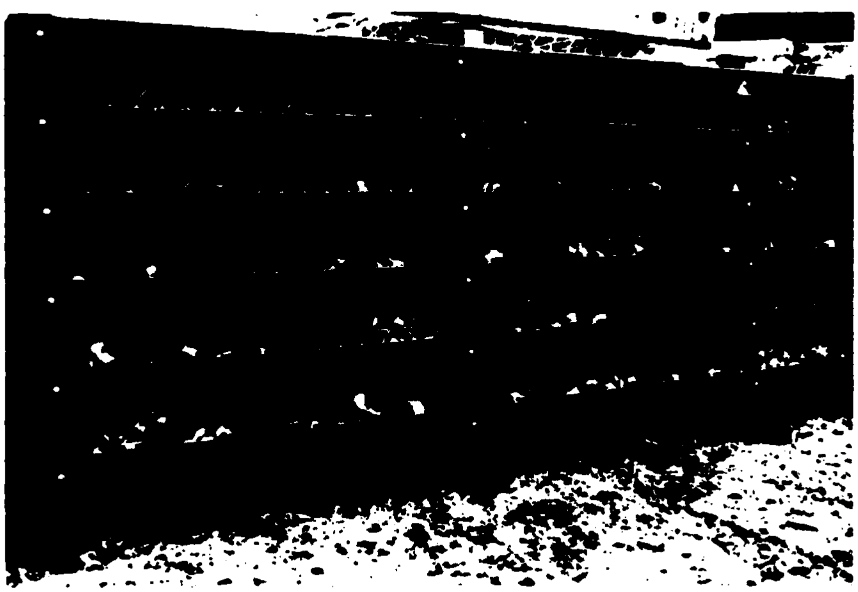
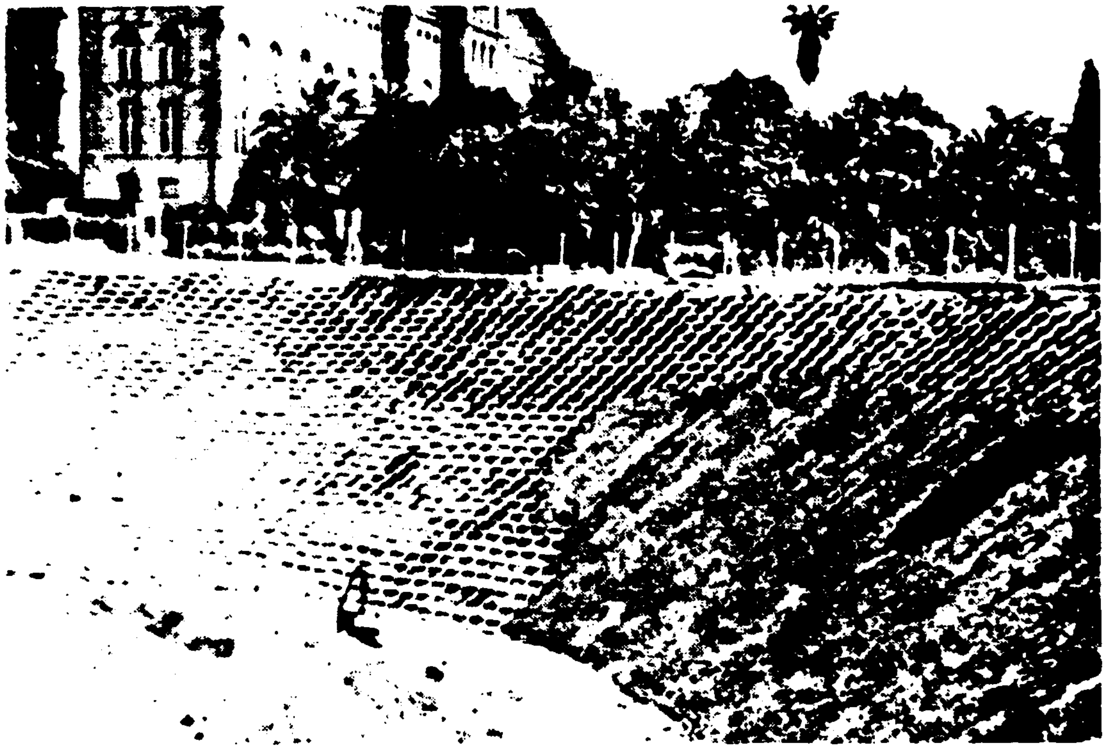


Fig. 2.36.
Experimental steel channel-wire fabric facing for a Reinforced Earth wall. Ties are connected to 4-inch, 10-gage channel modules or panels. Wire mesh retains coarse backfill placed immediately behind the panels.

Fig. 2.37. Stone armoring or revetment protecting cut slope along a highway. Vegetation has naturally invaded and established itself in this porous revetment. State Highway 27, Nevada.

Fig. 2.38. Gabion revetment protecting streambank along Carmel River, Monterey County, California. Picture taken January 8, 1979 during construction. (*Photo courtesy of Bekaert Gabions-Terra Aqua Conservation Co.*)



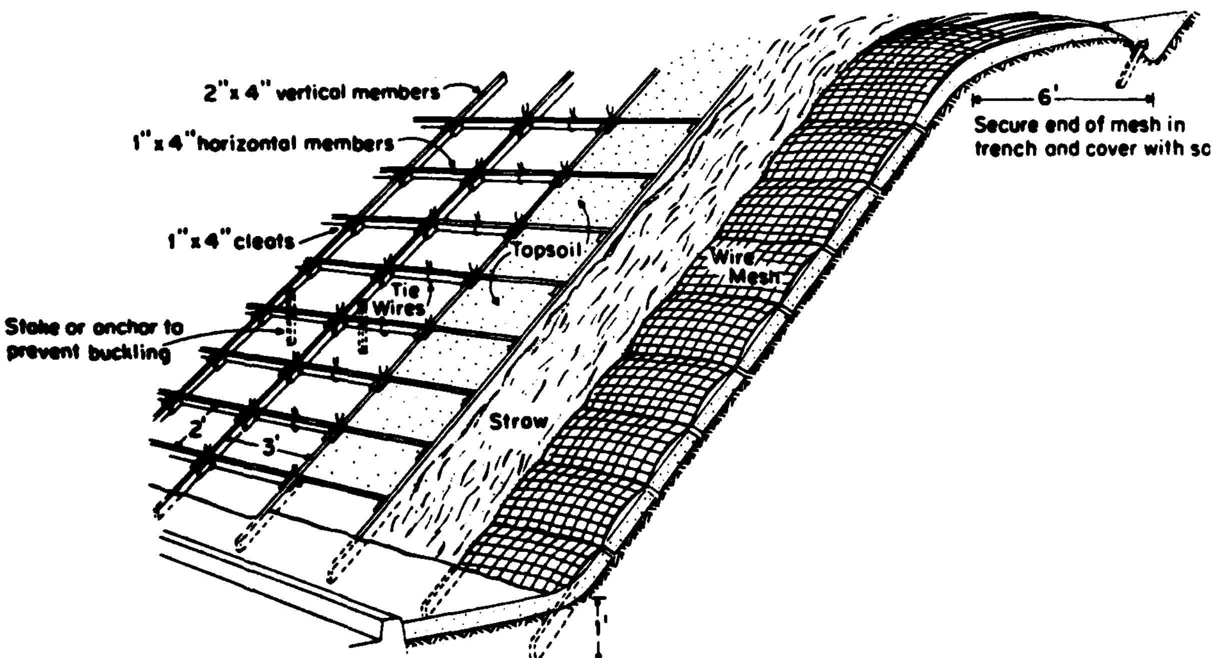


Fig. 2.39
Cut slope stabilization using anchored timber grid to hold topsoil and slope plantings in place. (*From Bowers, 1950*).

CHAPTER THREE

EROSION CONTROL AND SOIL STABILIZATION

INTRODUCTION

As discussed in chapter one, today's engineers are placing increased emphasis on soil erosion control measures because of the fast diminishing resource of "good" engineering, not to mention crop producing, soils. The civil engineer is faced with more and more situations involving construction on worse and worse soils. Consequently, there is added incentive for more effective erosion control measures. However, although some methods of erosion control are relatively new, the idea itself is far from young. Thousands of years ago, Neolithic man employed compacted soil in the building of his structures for the burial of his dead. The Romans used lime as a stabilizing agent during the construction of the Appian Way and many other Roman roads. Lime was also employed for this purpose in ancient Greece, India, and China. Some of the techniques used back then are still in use today.

We know erosion is a function of four basic factors--topography, vegetation, climate, and soil. We've spent a good deal of time talking about topography and vegetation, how they relate to the erosion process and what can be done to minimize their erosion effect. The third factor, climate, is a God given variable and there's not much we can do about it except to construct in such a way that climate has little or no effect, or simply avoid areas of severe climate altogether--an unrealistic approach in many cases. The only other factor left to discuss is the soil itself; how to alter the characteristics of the soil to improve its resistance to erosion, i.e. improve its engineering properties. This process is called soil stabilization.

Soil stabilization, in the broadest sense, is the alteration of any property of a soil to improve its engineering performance. Although the original objective of soil stabilization was to increase the strength or stability of soil, graduallly, techniques of soil treatment have been developed until soil stabilization efforts now encompass the alteration (increase or decrease) of almost every engineering property of a soil.

For example, soil stabilization techniques include methods which increase or decrease soil strength, reduce the sensitivity of strength to environmental changes (especially moisture changes), increase or decrease permeability, reduce compressibility, reduce frost susceptibility, decrease void ratio, alter the soil composition, as well as change the structure of the soil. Admittedly, although stabilization has been used in nearly every type of soil engineering problem, its most common application is the strengthening of soil components of highway and airfield pavements (Lambe, 1962).

A completely consistent classification of soil stabilization techniques is not available. Classifications have been based on the treatment given to soil (e.g., dewatering, compaction, etc.), process involved (e.g., thermal, electrical, etc.), and on additives employed (e.g. asphalt, cement, etc.). Since little if any confusion arises from this inconsistent system of classification, this chapter will generally employ the commonly used stabilization terms.

Before considering the various methods of stabilization; remember, there is no miracle cure. There is not now, or likely ever to be, any magic juice which can solve all soil problems. Soil stabilization offers many "medicines" for many problems. The choice of medicine, if any, depends on what the medicine will do in a particular case and its cost in comparison with other methods of solving the problem. The present use of soil stabilization is very large, and the potential use is tremendous. It is not, however, the best solution to all problems (Lambe, 1962). There are numerous approaches to soil stabilization; this chapter will include a number of them, many of which contain areas of controversy and ignorance. Much of the literature on soil stabilization is not as objective as the impartial engineer would like. Also, I do not intend to attempt to discuss all soil stabilization methods and techniques. In today's world of fast moving technology there are so many techniques it's doubtful whether there is any one person who is even aware of everything available. I do intend to touch upon a wide variety of methods to give the reader some insight as to what basic methods have been and can be used to improve soil properties. All of the methods discussed will not necessarily apply as erosion control measures but warrant at least a mention. Bear in mind too that where erosion control is the objective soil stabilization will almost always be more expensive than vegetation and many of the other methods and principles discussed in chapter 2. For erosion control, soil stabilization may be considered a last resort and is most often used because project restrictions prohibit the use of more conventional methods.

COMPACTION OR DENSIFICATION

Soil compaction is not an erosion control measure. Compaction, also called densification, is a mechanical soil stabilization method most commonly associated with foundation design and is usually intended to improve the strength characteristics of the soil. Compaction is the oldest and perhaps the most important method of soil stabilization. Compaction alone will often solve a particular soil problem and is usually the most economical of the techniques available. Compaction entails the rearrangement of soil particles and depends on three placement conditions--moisture content, amount of compaction, and type of compaction. The most desirable combination of these conditions depends on the type of soil involved and the desired results. A change in any of the three conditions will effect the soil's permeability, compressibility and swell-ability, and strength (stress-strain) characteristics. The properties of a fine-grained soil depend to a much greater extent on placement conditions than do those of a coarse-grained soil.

Thick deposits of loose cohesionless soils may require improvements in order to eliminate the subsequent development of excessive total and differential settlements and to minimize the possibility of liquefaction under dynamic loading. Suitable improvement can be achieved in many cases by densification; however, the needed densification cannot ordinarily be achieved using preload surcharge fills or compaction at the surface. Methods that are used for the in-situ deep densification of cohesionless soils include blasting, virbocompaction, and heavy tamping (Mitchell, 1981). Vibrocompaction encompasses all those techniques involving the insertion of vibrating probes into the ground with or without the addition of a backfill material including compaction pile techniques. The method of sand compaction piles was introduced in Japan, 1969 (Tanimoto, 1973) and since then it has attained for the stabilization of soft foundation ground an execution record of several tens of millions of meters in total pile length.

Compaction tends to reduce soil permeability since it both increases compacted density and rearranges the particles. This has a positive effect on erosion control because infiltration capacity has been reduced.

ADDITION-REMOVAL STABILIZATION

Changing the grain size distribution and the composition of the soil particles can significantly improve a soil's resistance to erosion as well as alter its other characteristics. The gradation of a soil can usually be changed by adding selected soil or by removing some selected fraction of the soil. The cost of this addition--removal type of mechanical stabilization can be very low; for example, a few percent of sandy clay from a convenient borrow area could be blended with gravel for the construction of a road subgrade at less than ten cents per square yard of one foot depth of treated gravel (Lambe, 1962). On the other hand, the addition of processed material to a soil can be relatively expensive; for example, the unit cost of processed and packaged bentonite (sodium montmorillonite) can cost more than such stabilizers as lime, cement, or asphalt.

The construction procedure, cost, and results obtained from the addition-removal stabilization depend almost entirely on the problem and soils at hand. A good example of the technique is the addition of fines (finer than 200 sieve) to serve as a binder. At the turn of the century, U.S. road builders found that certain requirements of soil gradation and binder must be met in order to get good service out of low-cost roads. In the 1930s theoretical work plus considerable lab and field testing resulted in standard specifications for the grading of soil-aggregate materials. The gradations were selected to give the densest mixture by supplying just the assortment of particles to minimize the amount of voids. The binder was to give cohesion to the mixture (Lambe, 1962). To ensure proper gradation and mix design, consult current AASHTO specifications. The engineer should be extremely cautious about adding fines to road bases and subgrades. The addition of fines may be changing a free draining, non-frost-susceptible material into a poor draining, frost-susceptible soil. Soil stabilization by the addition of fines represents a very cheap and powerful technique; however, it must be used with caution.

Another example of improving soil gradation is the addition of sodium montmorillonite ("bentonite") to reduce the permeability of a soil. However, a cheaper and superior reduction in permeability can be obtained by adding a locally available soil, if a satisfactory one can be found. Natural clays, being less sensitive to moisture and better graded, can be blended with pervious soils to result in a more nearly permanent blanket than can bentonite. One use for this type soil stabilization is the sealing of a reservoir. In one case, the natural floor of a reservoir consisted of highly pervious sands and silts. A one inch layer of locally available clay was trucked in, dumped, and mixed with approximately nine inches of the natural reservoir floor. The resulting mixture was brought to optimum moisture content and compacted. Then a four to six inch layer of gravel was placed on top and rolled to furnish protection. Numerous permeability tests on the field processed and compacted blanket material showed a permeability less than the permissable value of 10^{-7} cm. per sec. (Lambe, 1962).

Often, we want to increase the permeability of a soil, e.g. for pavement base courses and for filter courses. The presence of too large a percentage of fines may make a gravel soil unsatisfactory for these uses. An approximate upper limit of particles finer than 0.02 mm for a non-frost-susceptible gravel base course is three percent; for a gravel filter, the upper limit is seven percent passing the No. 200 sieve. An obvious treatment for a soil containing too many fines is to wash out the excess, simple in principle but difficult in practice. However, it can be done at reasonable cost ($0.50 to $1.00 per cy in 1963) if huge quantities are required at a site and there's plenty of water available. As selected base course materials get more scarce, washing becomes a more and more lucrative alternative.

It should be evident that improved gradation techniques could be used to help control erosion. Recall that the soil erodibility factor (K) is low in well graded-gravels. It also decreases with increasing clay and organic content.

SOIL REINFORCEMENT

Soil reinforcement has received more attention and made greater advances in application in the past several years than any other method of soil improvement and ground strengthening. Soil reinforcement is a mechanical stabilization technique which includes such methods as reinforced earth, soil nailing, root piles, and stone columns. Other quasi-forms of soil reinforcement include sand and gravel compaction piles; piles, press, and walls constructed by deep mixing methods, and thermal stabilization (Mitchell, 1981). Table 3.1 summarizes applications for the different reinforcing systems.

TABLE 3.1 APPLICATIONS OF SOIL REINFORCEMENT (Schlosser and Jenan, 1979)

	TYPE OF REINFORCEMENT			
Application	Reinforced Earth	Nailing	Root Piles	Stone Columns
Bearing Capacity	*		*	*
Stability	*	*	*	*
Settlement Magnitude	*		*	*
Settlement Rate				*

Reinforced earth stabilization was discussed as a biotechnical slope protection system in chapter two. It is unique among these four techniques in that reinforcements are used to carry tension only and it is a composite material in which the soil and reinforcements are built up in successive layers. Soil nailing consists of a series of reinforcing bars grouted into the ground to be supported. Root piles are small diameter piles (75 to 250 mm) of concrete cast in place, usually with a reinforcing bar in the center. They are installed in groups with individual piles both vertical and inclined. They can be used as structural supports as well as for stabilization of the included soil against movement and loss of stability. Stone columns are compacted columns of gravel or crushed rock installed into soft soils. Diameters usually range from 0.6 to 1.0 m. They provide vertical support for overlying structures or embankments and function as drains for the soft soil. They can be used also to resist shear in horizontal and inclined directions.

STABILIZATION BY DRAINAGE

The strength of a soil generally decreases with an increase in the amount of water in the soil pores and in the pressure existing in this pore water. The more water in a saturated soil, the farther apart are the particles; therefore, the weaker the soil. Addition of water to a clay causes a reduction of cohesion by increasing the electric repulsion between particles. Water is therefore an effective soil lubricant or weakener.

The strength of a saturated soil depends directly on the stress carried by the soil skeleton--effective or intergranular stress (p'). The effective stress is equal to the total stress (p) minus the pore water pressure (u). p' = p-u. Thus for a given total stress, soil strength increases with a decrease in the pore water pressure.

To increase soil strength, one of four methods of soil drainage may be employed:

1. Application of an external load to the soil mass to squeeze out pore water, adding surcharge or preloading prior to construction. Reference Mitchell, 1981 for detailed discussion.
2. Drainage of pore water by gravity, pumping, or a combination of both.
3. Application of an electrical gradient--electrical stabilization. Reference Casagrande, 1952 and Mitchell, 1981 for a detailed discussion.
4. Application of a thermogradient--thermal stabilization. Reference Lambe, 1962 and Mitchell, 1981 for a detailed discussion.

Various means have been used to expedite the escape of soil pore water. This increases the rate of strength build-up and the rate of settlement in the draining soil. One of the more common means is the vertical sand drain, similar to the stone columns previously discussed. State highway departments have made extensive use of sand drains or piles to expedite subdrainage. The usual installation consists of 18 to 20 inch diameter drains installed 6 to 10 feet apart, a drainage blanket placed on top of the soil stratum, and a surcharge fill placed on top of the drainage blanket.

Proper design involves determination of diameter and spacing of drains, thickness of drainage blanket, rate of fill placement, amount and duration of surcharge fill loading, amount of settlement during and after construction, and values of pore water pressures to be used for the control of construction operations. A proper design also includes measurements of soil strength as a function of consolidation and computation of embankment stability for various stages of construction.

Remember, sand drains per se give no significant strength to a soft soil; they merely increase the rate of consolidation by furnishing an escape route for the pore water. Sand drains have often been used where the increased rate of strength build-up was not needed or worth the cost. Particularly questionable is the use of sand drains in soils which exhibit a significant amount of secondary compression (Lambe, 1962). The most common cause of trouble with sand drain installation has been the failure to make stability analyses of the embankment at the various stages of construction.

SALT STABILIZATION

Salt stabilization is a chemical stabilization process involving the use of calcium chloride ($CaCl_2$), magnesium chloride ($MgCl_2$), or sodium chloride (NaCl, common table salt). Calcium chloride is the most often used as a soil stabilizer. Salts have the ability to readily absorb moisture from the air, dissolve, and become liquid. Additionally, they reduce the repulsive forces between particles and strengthen the bond of the water film. In solution, salts lower the freezing temperature of water.

It's been reported that if humidity is high enough, calcium chloride will absorb 4 to 10 times its own weight in water and retain one-third to two-thirds of it during the heat of the day. Therefore, in areas of fairly high relative humidity, it makes a dandy dust palliative since it will hold moisture to bind the road surface together, whereas without the salt treatment, the road would be dusty and ravel badly. Surfacing material losses are reported to be only half as great for treated as for untreated surfaces (Oglesby and Hicks, 1982).

Salt has served as a dust palliative in this country for more than 50 years. The usual application is 1½ to 2½ lb/sy/yr. Best results are obtained if, immediately after a rain, the surface is bladed and patched then the salt added in flake form. Rain washes a great part of the salt treatment away. Also, salt as a palliative is most effective on clayey soils. However, slick clay surfaces may become even slicker when treated.

Salt also assists in the compactive process, making it possible to obtain greater densities and greater strengths with normal compactive effort or to get usual densities with greatly decreased rolling. Sodium chloride has been used satisfactorily as a base stabilizer. It is reported to reduce the shrinkage, increase the strength, and reduce the moisture loss of certain clays. In some instances, it further improves lime-modified soils (Oglesby and Hicks, 1982).

In recent years, people complain that salt causes serious and costly corrosion of automobile bodies. Additionally, there is some concern that salt treatments will contaminate water supplies. For these reasons, there have been proposals that the use of salts as soil-stabilizing, dust-control, and de-icing agents be curtailed; salt stabilization may be on its way out except for limited use.

LIGNIN STABILIZATION

Lignin is one of the major constituents of wood, comprising 40 to 45 percent of its dry weight. It is a by-product in the manufacture of paper. Since lignin is water soluble, its chemical stabilizing effects are not permanent. So to improve it, the chrome-lignin process was developed. The addition of sodium bichromate or potassium bichromate to the sulfite liquor containing the lignin results in the formation of an insoluable gel.

Lignin has been used as a soil additive for years. Its biggest drawback is being water-soluble. Periodic applications are sometimes used. The effects of lignin depend on the form of lignin used and the nature of the soil. If the lignin is not neutralized, it is acid and acts as a soil aggregant. When neutralized it acts as a dispersant. (Reference the next sections on aggregents and dispersants.) Chrome-lignin acts as a cementing agent and increases soil strength. Lignin tends to reduce frost heave. In lab tests, treatments varying from 3 to 5 percent of the soil weight resulted in frost heaves less than 10 percent of those which occurred in untreated soils. Lab tests also showed that permeating water reduced most of the effectiveness of the lignin through leaching action (Lambe, 1962). Lignin has been used successfully as a dust palliative. It normally is mixed with materials from the loosened roadway, after which the combination is spread and compacted. About 0.2 to 0.5 gal/SY of lignin is applied in a 10 to 30 percent solution in water (Oglesby and Hicks, 1982).

AGGREGANTS

Aggregants are materials that make relatively modest changes in the properties of soils'containing some small particles, silt size and smaller. Aggregates alter electric forces between soil particles which are colloidal in behavior; they do not cement adjacent particles as for example, portland cement. The soil engineer has made very little use of aggregants in large stabilization works but this chemical treatment has promise in those applications where only modest changes in soil behavior is needed.

Aggregants increase the net electricl attraction between fine grained soil particles; therefore, the particles tend to aggregate or flocculate. Inorganic salts such as calcium chloride or ferris chloride and polymeric materials such as Krilium have been used as aggregants. The salts are more effective per unit cost of material.

Aggregants tend to reduce the maximum compacted dry density of a soil and increase the optimum molding water content. Also, by giving a soil a more random particle arrangement and a looser structure, aggregants increase permeability as much as 2 to 20 times. Altering permeability positively influences frost behavior. Soil strength is improved since electric repulsion between soil particles is reduced.

DISPERSANTS

Dispersants,like aggregants, also make modest changes in the properties of silt size and smaller soils by altering electric forces. Unlike aggregants, dispersants increase electric repulsion, reduces cohesion, and tends to cause the fine-grained soil particles to disperse.

Dispersants include phosphates, sulfonates, and versanates. The chemical agents, in breaking down aggregates, increases the fluidity of a soil-water system (this technique is used in the lab during grain size hydrometer analysis). Dispersed soil can be compacted to higher density and the water content for maximum density is lower than for untreated soil. Dispersants also reduces soil permeability on the order of 1/5 to 1/50 of the original value (Lambe, 1962). Dispersants can be blended or injected into a soil deposit as a water solution and have been proven to be effective in reducing the heave of compacted fine-grained soils upon freezing.

WATERPROOFERS

Waterproofers are chemical stabilizers which prevent or reduce the undesirable effects of water on soils. Waterproofers include alkyl chlorosilanes, siliconates, amines, and quaternary ammonium salts. They work by one end of the waterproofer molecule being attracted to and reacting with the soil surface; the other end is hydrophobic and repels waters, making the soil nonwettable by water. Waterproofers give soils no additional strength but do help them retain their natural strength when subjected to moisture. They have demonstrated their effectiveness in the lab but due to their high cost compared to other stabilization methods receive limited use in the field (Lambe, 1962).

LIME STABILIZATION

Lime stabilization dates back at least to the early Romans and the construction of the Appian Way. In the U.S., it goes back to the 1920's. Lime stabilization falls under the general heading of a chemical stabilizer and is now widely used to make clay-bearing soils suitable as sub-bases and to enhance the strength and other properties of potentially useful base coarse materials which contain clay. Lime is only effective when the natural material contains suitable amounts and types of clay--plasticity index (PI) should be greater than 10 (Oglesby and Hicks, 1982).

The lime stabilization process results because calcium hydroxide, commonly called lime, reacts favorably with some but not all clays. The reaction causes the clay's properties to change substantially. Lime generally increases the PI of low plasticity soils and decreases the PI of highly plastic soils. Reducing the plasticity of plastic soils makes the soil more workable and easier to handle. Lime generally causes a reduction in the maximum compacted density and an increase in the optimum molding water content. Also, lime generally increases the strength of almost all types of soil.

Because of the chemical complexity of clays, lime stabilization does not always work. Also, lime produced from different raw materials will react differently. It follows that careful lab study or field tests should precede any large-scale program of lime stabilization. Furthermore, there is evidence that lime stabilized base courses may break up or at least lose strength under freeze-thaw conditions. But even in areas subjected to ground freeze, the residual strength may be higher than that of untreated materials.

Lime is produced from natural limestone and the type of lime found depends on the parent material and production processes. There are five basic types of lime:

1. CaO	High-calcium quicklime
2. $CaO + MgO$	Dolomitic quicklime
3. $Ca(OH)_2$	Hydrated high-calcium lime
4. $Ca(OH)_2+MgO$	Normal hydrated dolomitic lime
5. $Ca(OH)_2+Mg(OH)_2$	Pressure-hydrated dolomitic lime

Lime is commonly delivered as a powder or slurry in slaked form, $Ca(OH)_2$, also called hydrated lime. At times, bulk unslaked "quicklime," CaO, is used but because it is caustic and also generates heat when slaked with water, care must be taken to protect workers from burns. In addition to being used alone as a soil stabilizer, lime is also used in admixtures. Lime-fly ash (hydrated lime, 4 to 8 percent of the soil weight, plus fly ash, 8 to 20 percent of the soil weight), lime-portland cement, and lime-bitumen are common examples.

As indicated, lime stabilization is intended to reduce plasticity and volume changes with moisture content and to increase soil strength. The PI test is used to measure plasticity reduction and volume change.

Strength increases are often measured by unconfined or triaxial compression, CBR, or flexure. Higher strengths result almost immediately with the increases almost in proportion to lime content up to six percent. Also, strengths usually increase over time.

The first step in the design of a lime-stabilized base is to select the required base thickness. This should be done on the basis of lab test results run on the soil with varying amounts of lime and molding water. The water and lime quantities required to give the desired CBR would be selected as values for field use. For granular soils lime-treatment levels of 2 to 5 percent are usually required. (clean sands usually higher). For plastic soils, 5 to 10 percent is normally required (Lambe, 1962).

Construction procedures are similar to those for soil-cement except for less stringent time requirements for placement operations. Since the lime-soil cementation reaction is slow, there's no need for strict limitations on the maximum time between the addition of lime and the completion of compaction. However, delays during the processing and compacting of lime-stabilized mixtures must be avoided. Otherwise, carbonation may occur--carbon dioxide from the air will react with the lime forming a weak calcium carbonate (Oglesby and Hicks, 1982). The normal construction sequence is as follows (Lambe, 1962):

1. Scarify the base.
2. Pulverize the soil.

3. Spread lime.
4. Mix lime and soil.
5. Add water to bring to optimum moisture content.
6. Compact mixture.
7. Shape the stabilized base.
8. Allow to cure--keep moist and traffic free for at least five days.
9. Add wearing surface.

PORTLAND CEMENT STABILIZATION

Portland cement is one of the most widely used and successful soil stabilizers. In 1917, J. H. Amies was issued a patent on soil and cement mixtures. The first controlled soil-cement construction was a road built in 1935 near Johnsonville, South Carolina, as a cooperative project of the Portland Cement Association (PCA), the Public Roads Administration, and the South Carolina State Highway Department. Since then, millions of square yards and many thousands of miles of soil cement roads and airfields have been constructed and many are still in use.

All mixtures of natural soil and portland cement are generally termed soil cement. (Other terms have been applied, e.g. cement stabilized soil, stabilized aggregate, cement modified soil, plastic soil-cement, etc.). Soil cement is a simple, highly compacted mixture of soil, portland cement, and water. (Tne soil comprises about 90 percent of the mixture.) As the cement hydrates, the mixture becomes a hard, durable paving material. It was developed in an effort to reduce road building and maintenance costs by using soils on or near the construction site.

The process of soil stabilization with cement is made possible through chemical reaction. A typical cement contains about 63 percent CaO, 21 percent SiO_2, 6 percent Al_2O_3, 3 percent Fe_2O_3, 3 percent MgO, plus other oxides. The cement reacts with fine grained soils in two ways. First, by surface chemical action it quickly produces flocculation and reduces the moisture affinity of clays. Second, and more slowly, it promotes cementation--producing a semirigid soil framework. Observations of cement-clay mixtures through the electron microscope indicate that, at first, the fabric is one of separate cement grains distributed throughout the clay. Then, as hydration of the cement proceeds, a gel forms along the edge of the clay particles. Eventually, the soil and cement can no longer be distinguished, indicating that clay and cement have chemically reacted (Oglesby and Hicks, 1982).

Almost any inorganic soil can be successfully stabilized with cement, although the treatment level of cement required and the properties of the resulting product vary considerably with the soil being stabilized. Organic matter tends to interfere with the hydration and weaken the treated soil. Experience has shown that best results are obtained with well-graded soils having less than 50 percent of its particles finer than 0.074 mm (No. 200 sieve) and a PI less than 20 percent. However, this soil does not have to be well graded to be hardened with portland cement because the stability results mainly from cement hydration and not from cohesion and internal friction of the materials. On the basis of gradation, soils for soil-cement construction may be divided into three groups: sandy and gravelly soils, sandy soils deficient in fines, silty and clayey soils.

Sandy and gravelly soils with about 10-35 percent silt and clay combined are best for soil-cement. Generally, they require the least cement. Almost all granular materials work well if they contain at least 55 percent material passing the No. 4 sieve. Included are glacial and wind-deposited sands and gravels, crusher-run limestone, caliche, and limerock. Well-graded materials may contain up to 65 percent gravel retained on the No. 4 sieve and still have enough fines for proper binding.

Sandy soils deficient in fines may require slightly more cement than the soils in the first group. This is true of certain beach, glacial and wind-blown sands. However, they still make excellent soil-cement.

Silty and clayey soils can make adequate soil-cement but those soils with a lot of clay are harder to pulverize. In general, the more clay a soil contains, the more cement it will need.

Table 3.2 shows typical compressive strengths for various soils treated with 10 percent cement.

TABLE 3.2 TYPICAL COMPRESSIVE STRENGTHS OF SOIL-CEMENT WITH DIFFERENT SOILS (Road Research Laboratory, 1952)

Type of Soil	Compressive Strength, psi
Plastic clay, organic soil	Less than 50
Silt, silty clay, very poorly graded sand, slightly organic soil	50-150
Silty clay, sandy clay, poorly graded sand and gravel	100-250
Silty sand, sandy clay, sand, gravel	250-500
Well-graded sand-clay, gravel- sand-clay mixture, sand, gravel	400-1500

Soil-cement is used mainly as a base for road, street, and airport paving. A bituminous wearing course is placed on top of the soil-cement base to complete the pavement. Soil-cement has been used for pavements without wearing courses, but not with much success, primarily because of its low resistance to abrasion from traffic.

During the last 30 to 40 years, research with soil-cement has led to its use on a wide variety of engineering projects such as slope protection for dams, levees, and other embankments, and linings for highway ditches, irrigation canals, open channels, reservoirs, and lagoons. Soil-cement's reasonable cost, ease of construction, and convenient utilization of in-place soil make such applications economical,practical, and esthetically pleasing. Soil-cement becomes hard, durable, impermeable, and resistant to freezing,thawing, wetting, and drying. Properly designed and constructed, it will not be eroded by the passage of water unless the flow carries enough of a bed load to be abrasive.

Plastic soil-cement is used to pave steep slopes or areas that are too small, irregular, or confined for the usual spreading and compacting equipment. In 1948, plastic soil-cement was used to line a short section of irrigation canal near Notus, Idaho; this lining has served well for more than 30 years. The primary difference between compacted and plastic soil-cement is the amount of water used. Compacted contains just enough water for maximum density when it is compacted by rollers or vibratory equipment. Plastic requires enough water to produce a wet, workable mixture with the consistency of plaster or mortar; consequently, it also requires a somewhat higher cement content than compacted soil-cement. Sandy soils are best for plastic soil-cement mixtures. Soils containing more than 30 percent combined silt and clay are seldom used since they are difficult to pulverize, require higher percentages of cement, and, because of their stickiness, are difficult to mix and place (PCA, 1978).

There are a number of factors which affect the strength and durability of soil-cement besides the type of soil being used. However, the nature of the soil is probably the overriding concern. We know that the higher the surface area per mass of a soil, the more cement required for stabilization. Clay can cause problems in pulverizing, mixing, and compacting, especially expansive clays. Organic matter weakens the soil-cement. Progress has been made with relating the response of a soil to cement stabilization and the soil's pedological classification. Use of these relationships can reduce the testing required for mix design. For example, a red soil probably contains iron; it would react well with cement. A black soil probably contains a large amount of organic material; it would react poorly. Soils having the same soil profile, wherever they are found, as identified by the U.S. Department of Agriculture are considred a soil series. Studies have shown that a definite soil series and horizon will always require about the same amount of cement.

Other factors affecting soil-cement are the amount and type of cement and the mixing procedure. The more cement, the stronger the resulting soil cement; also, high-early-strength cement is usually more effective than normal cement. As for mixing, the more thorough the mixing the stronger and more durable the soil-cement. However, continued mixing past an optimum can result in segregation of components. Also, mixing after hydration has begun has undesirable effects. Consequently, construction specifications usually limit the time between cement addition and completion of compaction (Lambe, 1962).

The properties of soil-cement also depend on the molding water content and the compacted density. The molding water influences the compaction characteristics and furnishes water for cement hydration. Sands should usually be compacted slightly dry, and clays slightly wet, of that water content which gives maximum density. Generally, the greater the compacted density, the better the resulting stabilized soil.

As with concrete, the strength of soil cement increases with age, and like concrete, it should be kept moist during the initial stages of cure. Finished soil-cement contains enough moisture for adequate cement hydration. However, a protective cover (bituminous material, water-proof paper, moist straw, etc.) should be immediately placed over the completed work to retain this moisture.

Finally, admixtures can be used to produce dramatic improvements in the strength of soil-cement. For many years, engineers have added lime or calcium chloride to accelerate the set and to improve the properties of the soil-cement. These and certain other chemicals permit a reduction in the amount of cement required to treat a soil responsive to cement and sometimes permit stabilization of those soils which are not normally responsive to cement alone (e.g. certain organic soils). PCA (1971) discusses the use of admixtures and recommended mixes.

Most soil-cement is built six inches thick. The thickness may be reduced if the subgrade is very stable and for very light traffic, four inches may be enough. On the other hand, high volumes of traffic may call for eight inches or more. The thickness of a soil-cement base is often taken equal to that of a granular base required for good quality subgrades and equal to 0.75 of the thickness of required granular base for a poor subgrade. This procedure amounts to determining the thickness of granular base required for a given subgrade and loading and then taking the thickness of soil-cement base as equal to 100 or 75 percent of this granular thickness. The CBR test is usually the controlling factor. Once the thickness is determined, soil-cement mix design consists of selecting the amount of cement, the amount of molding water, and the compaction density to be obtained in the field. Techniques for selecting these values are described in detail in the PCA Soil Cement Lab Handbook, 1971.

PCA (1958) describes in detail the field placement of soil-cement. Construction usually involves:

1. Shaping the soil to be treated.
2. Pulverizing the soil.
3. Adding the water and cement.
4. Mixing.
5. Compacting.
6. Finishing.
7. Curing.

The field control of soil-cement stabilization must be done mostly by visual inspection. Moisture content and density can be checked just as the compaction of untreated soils. Measures of cement content, degree of mixing, and strength are not, however, easy to obtain. PCA (1958) and PCA (1980) also provides detailed descriptions of the inspection and field control procedures.

The Waterways Experiment Station (1956) has summarized and evaluated performance data on more than 40 soil-cement roads and airfield jobs. The principal conclusions from this study are presented here in the way of review of soil-cement performance. Cement has proved an effective stabilizer for a wide range of soil types. Plastic soils (inorganic and organic) present the most serious difficulties.

Undersirable shrinkage cracking of soil-cement bases is to be expected; however these cracks are not signs of failure. A bituminous wearing surface is needed over soil-cement to protect it from abrasion and to keep water out of the shrinkage cracks. Construction joints and pavement edges are likely to be critical areas in soil-cement bases.

The thickness of a soil-cement base is usually about the same as required for a granular base on good quality subgrade. The thickness of a soil-cement base required on a subgrade of less than 3 CBR is less than that for a conventional granular base on the same subgrade, according to the Corps of Engineers CBR design curves.

The design of soil-cement based on the PCA durability method is apparently conservative. The design based on compressive strength is not foolproof; for example, a sand-cement pavement of strength 243 to 356 psi was failed, while in the same tests, a clay cement pavement of strength 198 to 315 psi was not failed (WES, 1956).

Care should be exercised in using soil-cement in frost areas. Frost susceptible soils treated with cement are not necessarily made non-frost-susceptible. Nearly all the soil-cement evaluated by WES (1956) was in nonfrost areas.

In summary, cement is a most successful soil stabilizer and will give excellent results if used properly. A few of the soil-cement projects in Albuquerque and thoughout New Mexico are presented by the PCA in Figures 3.1 through 3.4.

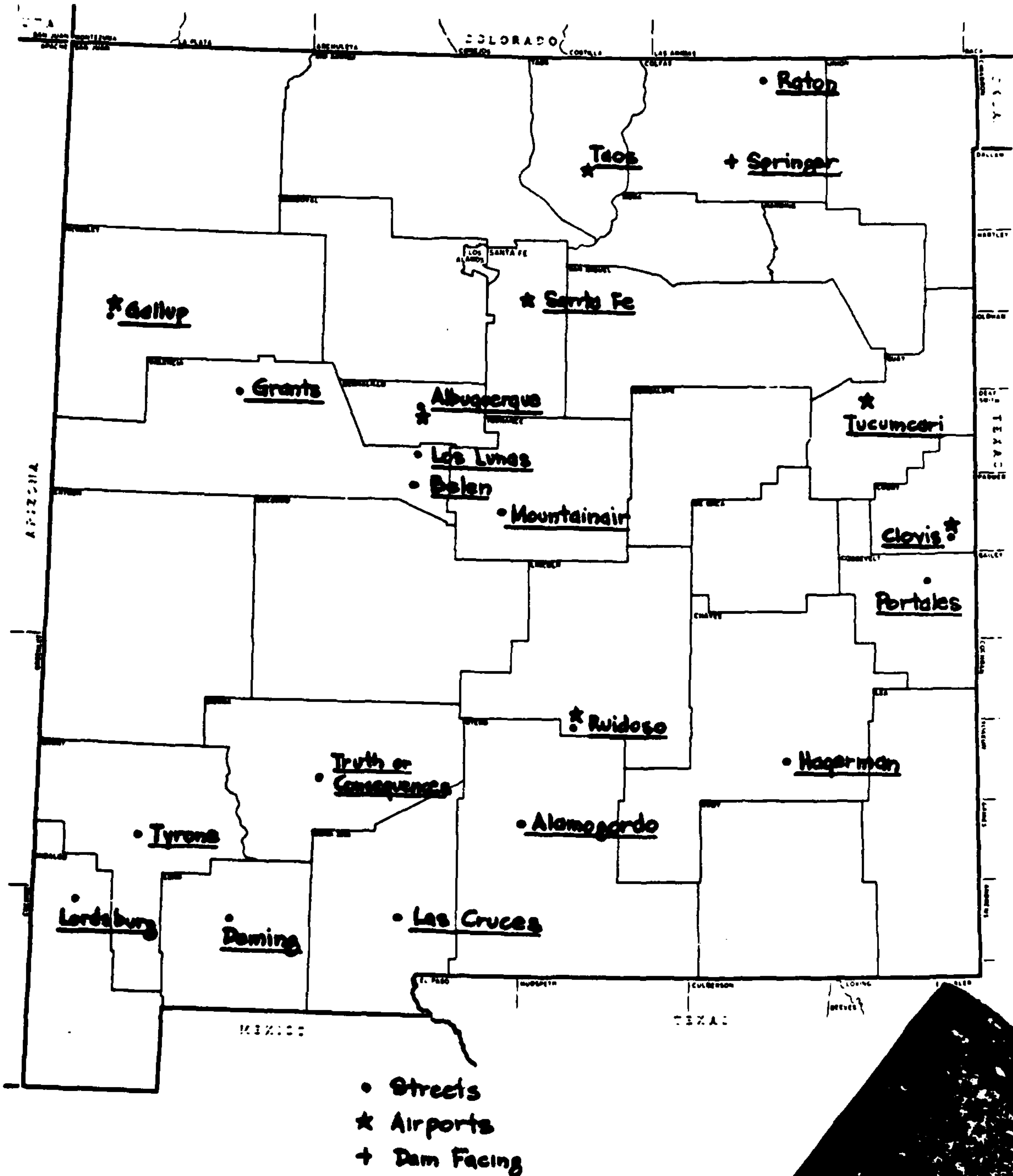
ASPHALT STABILIZATION

Bituminous materials are used in various consistencies to improve soil properties and it is a widely used and generally effective method. Mixed with cohesive soils, they improve bearing capacity and soil strength by waterproofing the soil and preventing high moisture content. Added to sand, they act as a cementing agent and produce a stronger, more coherent mass (Dunn, 1980). In discussing bituminous stabilization, three terms require definition: bitumens, asphalts, and tars. Bitumens are nonaqueous systems of hydrocarbons which are completely soluble in

CEMENT TREATED BASE

The favorite pavement base for New Mexico cities

MUNICIPALITIES WHERE YOU CAN FIND CEMENT TREATED BASE



Cement treated base (also called soil-cement) is a tightly compacted mixture of portland cement, water, and suitable soil or aggregate. With an asphalt wearing surface, it provides a pavement that is stronger inch-for-inch than any other low-cost pavement. More than 6 million square yards have been placed in streets and airports for New Mexico municipalities. In addition, over 30 million square yards of CTB pavement have been built by the New Mexico State Highway Department and other governmental agencies in the State.

Fig. 3.1

Work by City Forces — 1951

TRUTH or CONSEQUENCES

T or C was the first New Mexico city with cement treated base streets. This street built with city forces in 1951 using in place stabilization, has required little maintenance and continues to serve the community very well. All subsequent street paving in T or C has utilized cement treated base.

Paving District #1 — 1962
T. E. Scanlon & Assoc., Consulting Engineers

HAGERMAN

Town officials selected cement treated base using in place stabilization after constructing their own test street with borrowed equipment and much sweat.

Paving District #23 — 1967
Robert Lydick, Consulting Engineer

CLOVIS

City officials were so pleased with the low cost and performance of street Paving District #23, they specified it for construction of a new runway at the municipal airport.

Paving District #6621 — 1968
Maddox-Horne & Assoc., Consulting Engineers

LAS CRUCES

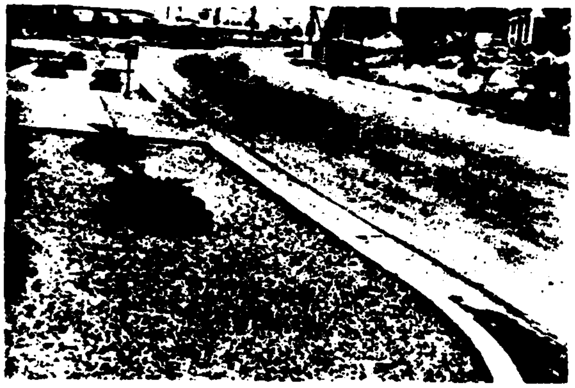
Cement treated base was bid 30% lower than an equal thickness of a competitive base material for Paving District ±6621. The cost of CTB is consistently lower than other materials of equivalent design.

Fig. 3.2.

Paving District #1 — 1961
Jack Kennady, Consulting Engineer

RUIDOSO

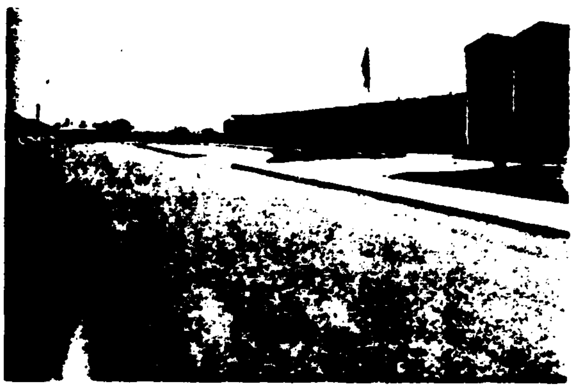
This resort town of 2,500 has more cement treated base per capita than any municipality in the country. Two large CTB paving districts and an airport have proven themselves under severe climatic conditions.



Paving District #14 — 1965
Gordon Herkenhoff & Assoc., Consulting Engineers

GALLUP

Cement treated base was found to be the economical solution to street paving in this Indian Capitol as suitable gravel base aggregates are not readily available in this part of the State. CTB was also used in reconstruction of the main runway at the Gallup-McKinley County Airport.



Paving District #11 — 1967
Quinton Daniel, Consulting Engineer

ALAMOGORDO

City officials were willing to pay a little more for cement treated base over untreated base. It should prove to be a wise long term investment.

Paving District #4 — 1969
D. F. Molzen & Associates, Consulting Engineers

BELEN

Newly placed cement treated base streets were subjected to severe flooding in June 1969. Hot mix surfacing peeled off in places and old gravel base streets suffered extensive damage and settlements, but the cement treated base remained intact. CTB effectively withstands water, from above or below, as from high water tables in valley areas.

Fig. 3.3.

- Low first cost
- High load carrying capacity
- Virtually eliminates pot holes and ruts
- Low maintenance
- Resistant to water and weather
- Gains strength with age

ALBUQUERQUE

Previously plagued by poor performance and deep rutting on its heavily traveled streets, Albuquerque has been using cement treated base on all arterials since 1963. Cement treated base arterials are designed for high traffic counts and heavy loads. CTB is also used for utility cut replacement and base maintenance on all paved streets.

BEFORE

AFTER

Paving District #168 — 1968
William Matotan & Assoc., Consulting Engineers

Cement treated base solved the problem of rebuilding failed flexible base streets in the Bel Air residential subdivision at a minimum cost. The existing base material and some of the worn out asphalt surface were salvaged, mixed with portland cement and water to provide new durable CTB streets.

Additional information and literature is available from:

PORTLAND CEMENT ASSOCIATION

An organization to improve and extend the uses of portland cement and concrete through scientific research and engineering field work

5301 Central, N.E. — Suite 1715
Albuquerque, New Mexico 87108
Phone (505) 268-6789

Fig. 3.4

carbon disulfide. Asphalts are materials in which the primary components are natural or refined petroleum bitumens or combinations thereof. Tars are bituminous condensates produced by the destructive distillation of organic materials such as coal, oil, lignite, peat, and wood.

Even though tars and other bitumens such as crude oil have been used to stabilize soil (especially as a dust palliative), most bitumen stabilization has been with asphalt. Therefore, this discussion is about soil-asphalt.

Soil-asphalts have been used for many applications but its greatest use has been for bases of highway and airfield pavements. There is over 35,000 miles of highway in the U.S. having soil asphalt bases. The asphalt levels in these bases range from approximately 2 to 10 percent by weight. Nearly every inorganic soil with which bitumen can be mixed can be stabilized. Best results, however, are obtained with soils meeting the following requirements (Lambe, 1962):

1. Maximum particle size less than 0.33 the compacted thicknnss of the treated soil layer.
2. Greater than 50 percent finer than No. 4 sieve (4.76 mm).
3. Thirty-five to 100 percent finer than No. 40 sieve (0.42 mm).
4. Greater than 10 percent but less than 50 percent finer than the No. 200 sieve (0.074 mm).
5. Liquid limit less than 40 percent.

6. Plasticity index less than 18 percent.

Bitumen stabilizes soil by one or both of two mechanisms. It binds soil particles together; it protects soil from the undesirable effects of water (waterproofs). The binding occurs in cohesionless soils and the waterproofing in the water sensitive cohesive soils. The water protection results from the asphalt's plugging voids in the soil and from coating soil particle surfaces.

As with soil-cement, there are several factors which affect the quality of soil-asphalt starting with the soil itself. Acid organic matter is detrimental to soil-asphalt; neutral and basic organic matter from arid and semiarid regions does not appear to be particularly detrimental. Fine grained soils from arid regions, being high in pH and dissolved salts, do not respond too well to asphalt stabilization. Plastic clays are difficult to treat because of the mixing problems and high level of asphalt required.

Within limits, the more asphalt used the better the results. Since the asphalt adds little, if any, strength to fine-grained soils, the influence of increased asphalt content shows up only in improved waterproofing. Too much asphalt results in a gooey mixture which cannot be properly compacted. Also, the more thorough the mixing of the asphalt and soil, the better the results.

The density of a mixture of soil and asphalt depends on the volatiles content and amount and type of compaction. The molding volatiles content which consists of the volatiles in the asphalts used plus the water in the soil plus the water to be added should be selected on the basis of lab tests. The stability of cured and then immersed samples should be determined for various volatiles content, and the optimum volatiles content taken as that which gives the highest strength.

A very valuable relationship to consider for curing time of soil-asphalt is that the strength of any given mixture is inversely related to the volatiles content at the time of test. In general, the longer the period of cure and the warmer the temperature of cure, the greater the volatiles lost. Also, the longer the period of immersion, the greater the water pickup.

There are numerous methods in use for the design of flexible pavements. Many of these methods are also used for selecting the thickness of soil-asphalt base for a pavement. The Corps of Engineers method employing the CBR test and the Asphalt Institute method are considered two of the best. The CBR method consists of selecting the required base thickness for the traffic and subgrade conditions using curves prepared by the Corps. The curves were developed to use with gravel bases; however, instead of gravel, the same thickness of soil-asphalt is used. The soil-asphalt must possess a soaked CBR value which meets Corps requirements (usually greater than 80 percent). The Asphalt Institute method consists of using curves and charts prepared by the Institute to determine the required base thickness for a given traffic and subgrade condition. The CBR method and the Asphalt Institute method give design thicknesses of the same order of magnitude. However, the CBR method is a bit more conservative in that it requires a thicker base. A 6 to 8 inch base of soil-asphalt is commonly used in the U.S. (Lambe, 1962).

The asphalt used to stabilize a particular soil should be as heavy and as warm as can be handled. The type, grade, and amount should be selected on the basis of lab tests which determine the effect of type, grade, and amount of asphalt on the stability of the soil-asphalt. Stability is measured in accordance with the CBR method or the Asphalt Institute method depending on which was used to determine soil-asphalt thickness.

The usual sequence of construction operations is as follows:

1. Pulverization of the soil to be treated.
2. Addition of the water necessary for proper mixing.
3. Adding and mixing the bitumen.
4. Aeration to the proper volatiles content for compaction.
5. Compaction.
6. Finishing.
7. Aerating and curing.
8. Application of surface cover.

To ensure proper stabilization requires close control over mixing, compacting, drying, and applying the surface cover. Field control tests comprise determination of water content before and during processing, bitumen content after mixing, and density after compaction (Lambe, 1962).

The Waterways Experiment Station (1956) has summarized and evaluated performance data on thirty airfields which had soil-asphalt pavement bases. The principal conclusions are presented here as a performance review. The asphalt content varies from 4.5 to 10.0 percent and the base soils were sands, gravels, or both. Of the thirty airfields, five were unsatisfactory from the standpoint of the stabilized material after an average of three years use. Of these five, four were stabilized with emulsified asphalt; however, five of the satisfactory base courses were stabilized with emulsified asphalt. Cutback asphalts and tars were used for the remainder of the satisfactory bases. In general, asphalt is a potentially cheap and very satisfactory soil stabilizer. However, care must be taken to properly design the mix and the construction must be carefully supervised.

LIME, CEMENT, OR ASPHALT?

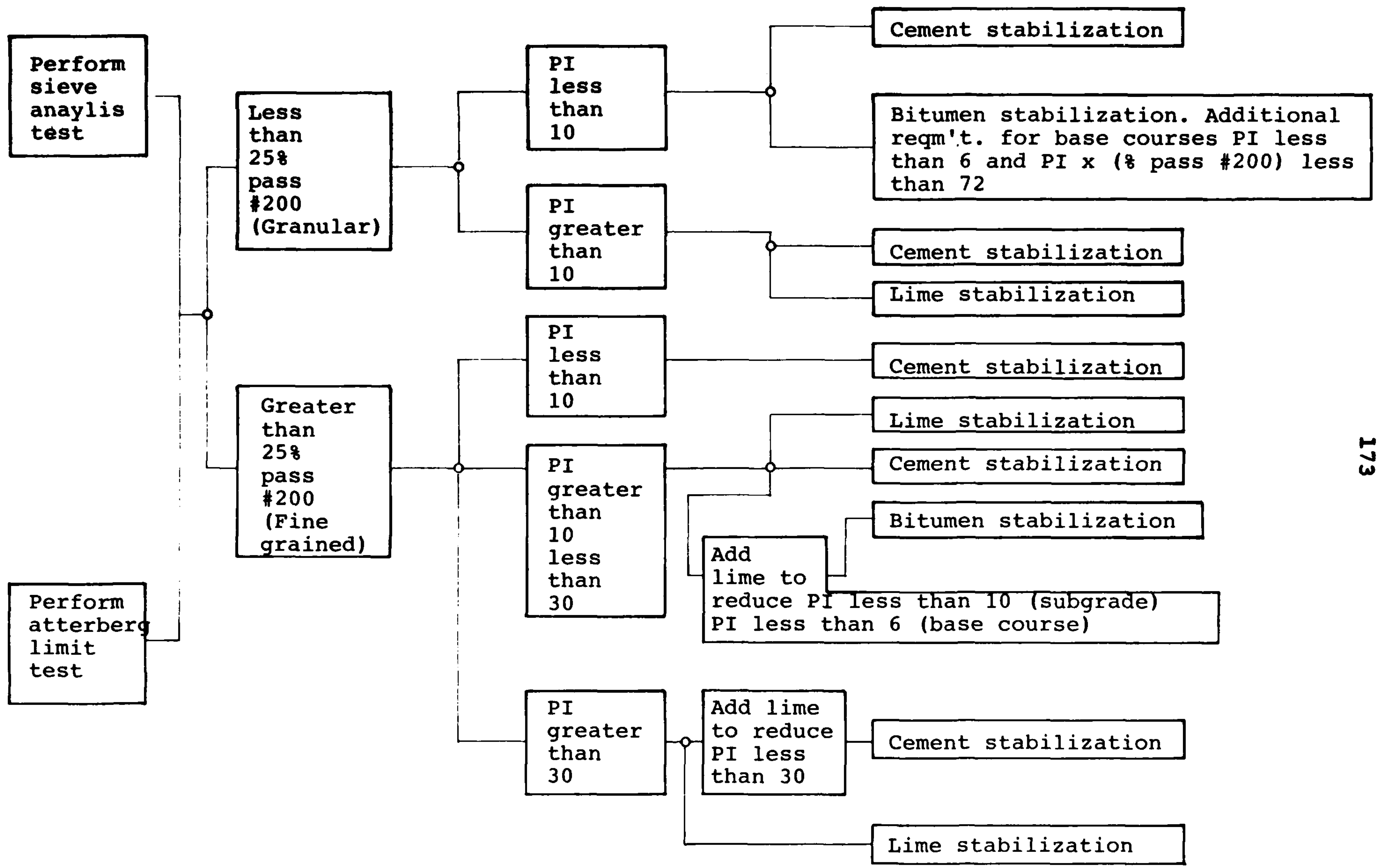
The last three soil stabilizers discussed (lime, cement, and asphalt) are the most common additives used for treated bases in road construction. The designer generally uses such treatment when he believes a satisfactory result can be obtained at a lower overall cost. If there is some confusion as to which stabilizer to use and when to use it, the Federal Highway Administration offers some helpful hints. Figure 3.5 is a useful guide for choosing among the three. It shows well proven combinations with acceptability of the soil defined by the plasticity index (PI) and percent passing the No. 200 sieve.

INJECTION AND GROUTING

Grouting has been extensively used primarily to control ground water flow. Since the process fills soil voids with some type of stabilizing material (chemicals, cement, soil, lime, etc.), grouting is also used to increase soil strength and prevent excessive settlement. Injection may be made in several ways. In one method a casing is driven and the injection is made under pressure to the soil at the bottom of the hole as the casing is withdrawn. Another method is to drill a grouting hole and at each level in which injection is required, withdraw the drill and place a collar at the top of the area to be grouted; grout is then forced into the soil under pressure. A third method is to perforate the casing in the area to be grouted and leave the casing permanently in the soil.

Grout may be designed to penetrate through the soil voids or to displace soil and improve the properties by densification or by restricting water flow. Penetration grouting may involve portland cement or fine-grained

FIG. 3.5 Suggested procedure for selecting a stabilizing agent for base courses.
(Source: FHWA Implementation package 80-2, Oglesby and Hicks, 1982)



soils such as bentonite. These materials penetrate only a short distance through most soils and are primarily useful in very coarse sands or gravels. Viscous fluids, such as sodium silicate, may be used to penetrate fine grained-soils. Displacement grouting usually consists of using a groutlike portland cement and sand, which when forced into the soil displaces and compacts the surrounding soil about a central core of grout. Injection of lime is sometimes used to produce lenses in the soil that will block the flow of water and reduce compressibility and expansion properties of the soil. Injection and grouting methods are generally expensive compared with other techniques and are used primarily in special situations (Dunn, 1980).

NEW PRODUCTS

There are a number of new products on the market that will assist the engineer with erosion control problems. Most of the products have registered trade marks and the majority I've seen are a sort of ready made riprap that can be very effective as part of a biotechnical system. They are also mostly aimed at arresting surface erosion. The American Enka Company, a part of Akzona Inc., and operating out of Enka, North Carolina has several promising products. Among them are products called "Enkamat," "Enkadrain," and "Stabilenka." Another product from Conwed Corp. out of St. Paul, Minnesota is the Conwed Erosion Control Net designed to hold mulch or soil in place and to

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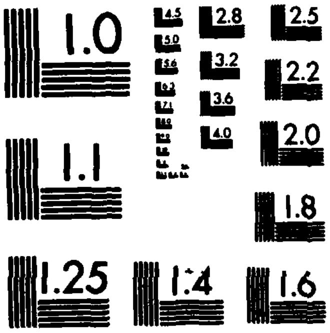

MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

reduce the incidence of erosion. Manufacturer's advertisements on these products are shown in Figures 3.6 through 3.20.

Another product, not shown, is called the "Excelsior Erosion Control Blankets" manufactured by PPS Packaging Company; Santa Fe Springs, California. These soil retention blankets consist of a uniform mat of wood excelsior fibers covered on top with extruded plastic netting (netting is similar to Conwed's, Fig. 3.19). They are designed to "prevent erosion, encourage germination, retain moisture, and protect seedlings under the most difficult erosion control conditions." The wood excelsior is made exclusively from Colorado high altitude aspen timber which is exceptionally absorbent and contains no harmful pitch or resin. The Soil Conservation Service has had considerable success with this type of retention blanket in the Albuquerque, New Mexico area. Approximately 600 acres of ground alongside the Black's Arroyoand Floodwater Diversion #2 which is just south of Rio Rancho and due west of Cibola High School was seeded with a mixture of Indian Ricegrass, Giant Sand Dropseed, Sand Dropseed, and Sideoats Gramagrass. The area, with slopes up to 25 percent, was then covered with a retention blanket. The erosion control project was completed in 1975 at a cost of approximately $117,000 ($195/acre). In 1984, the entire area showed good vegetation growth and very little erosion. Noteworthy is the fact that the area received no additional attention after construction in 1975. The concrete, open channel, vertical wall, 8000 cfs culvert, (approximately 25 feet wide and 10 feet high) was also constructed in 1975 at a cost of approximately one million dollars. The channel is still in excellent condition today and is well protected from the potentially damaging effects of wind and water erosion (Farmer, 1984). The $117,000 for erosion control appears to have been very well utilized.

These are just a few of the many, many erosion control products on today's market and are discussed here simply to attune the reader to what's available. I do not endorse any one product over the other, but merely suggest that all new applicable products be considered as alternatives during the project design process. These products come complete with installation guidelines and product specifications. Very often, they may be cost prohibitive; however, they at least merit serious consideration before deciding on a final design.

MISCELLANEOUS SOIL STABILIZERS

There are a great many other materials and methods which are in use, have been considered, or are being considered as a means of soil stabilization. Some of these materials are chemicals selected for study on the basis of theoretical principles, while others are cheap products whose study was based more on hope than on science. Rather than drag on this discussion of soil stabilization, I will

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March 9, 1984

Mr. Michael W. Koch
2123 Altex NE
Albuquerque, NC 87112

Dear Mr. Koch:

Thank you for your letter and for your interest in our Enkamat® Soil Erosion Matting. I have enclosed our full brochure for your review.

Please feel free to contact our distributor in Albuquerque, American Excelsior Company, 4019 Edith Blvd NE, Albuquerque, NM 87107. I am sure their Manager, Mr. Lynn Boyer, 505-345-7806 would be happy to give you information on various jobs they have done in your area. You might be interested to know that Aggie Stadium at New Mexico Stat contains our Enkamat material.

Good luck in your efforts. Please let me know if I can be of any further assistance.

Sincerely,

P L Skoglund Jr

P. L. Skoglund, Jr.
Director, Enkamat Systems

/e
Enclosure

PLANTS:
Enka, NC
Clemson, SC
Lowland, TN
SALES OFFICES:
Dalton, GA
Enka, NC
Greensboro, NC
New York, NY

ENKAMAT

NATURE'S EROSION CONTROL SYSTEM

Nature's own erosion control system is vegetation. Soil will remain stable under extremely adverse conditions if it is covered with a heavy growth of plant life having an extensive root system that serves as a reinforcing binder. However, it is difficult to establish and maintain vegetation on the steep slopes of embankments, streams, drainage ditches and sand dunes, particularly when these areas are subjected to severe water or wind erosion. Consequently, many of these problem areas have traditionally been surfaced with concrete or asphalt; such solutions are both costly and environmentally unappealing. *Enkamat* is the unique alternative.

WHAT IS ENKAMAT?

Enkamat is a tough, flexible soil reinforcement matting made from nylon monofilaments fused at their intersections. Its three-dimensional structure is a bulky mat of very open construction, leaving 90% of its volume to be filled with soil, gravel or other appropriate materials.

Fig. 3.6

Enkamat is easy to install in even the most critical situations. Once in place, it is highly resistant to environmental and chemical degradation. *Enkamat's* carbon black additive protects against the deterioration due to exposure to ultra-violet radiation.

HOW IS ENKAMAT USED?

Enkamat functions as a permanent turf reinforcement, providing a stable environment which encourages the growth of grass. It is ideal for virtually any public or private project which requires permanent surface erosion control at an economical price. This includes ditches, slopes, waterways and shorelines, among others. *Enkamat* is equally effective on steep or level surfaces, individual plots or acreage. Since grass grows through the material, and eventually covers it completly, *Enkamat* has particular application in those areas where an appealing, unspoiled landscape is desired.

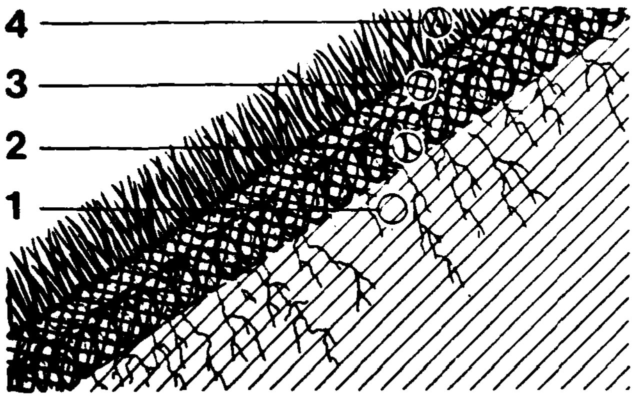
Enkamat is shipped in rolls and installed in strips approximately one meter wide. A full roll, weighing between 85 and 90 lbs., can be handled easily by one man. Prior to installation, the rolls can be stored near the job site. Delivery can be made several days before scheduled installation, thus eliminating costly construction delays.

Enkamat has been used successfully by highway departments in several states to stabilize both natural and artificial embankments, steep excavated slopes, bridge and viaduct aprons, and drainage ditches. In these cases, *Enkamat* has been specified as an economical and environmentally sound alternative to concrete, asphalt, and rip rap.

Roadside ditch

Below: Downslope ditch

Fig. 3.7

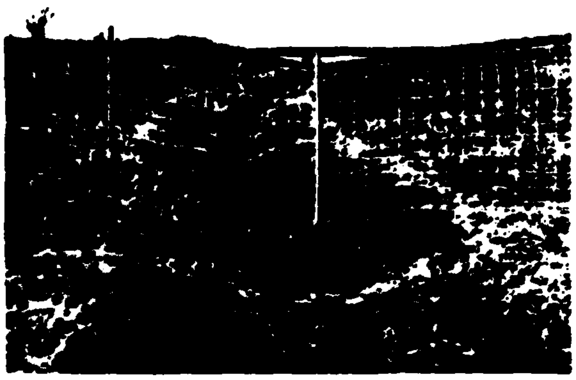


Enkamat Installed On Slope. 1) Subsoil, 2) Enkamat, 3) Sedimentation, 4) Grass area.

WHAT DOES ENKAMAT DO?

Enkamat provides a stable environment in which the growth of new vegetation is encouraged. Immediately after installation it functions as a mulch, by holding grass seed in place, slowing down the velocity of runoff water, and providing pockets for grass to establish itself securely. Because of its black color, the mat collects heat and works as an incubator to promote faster seed germination and to extend the growing season. This results in more rapid plant growth and deeper roots.

During the first few days after installation, when the root system is in its initial stage, *Enkamat* fills up with soil and sedimentation caused by wind and water erosion. Soil particles are trapped and held among its tangled filaments, thus securing *Enkamat* to the ground. The roots become entwined with the filaments making the vegetative cover extremely stable and difficult to dislodge or uproot.

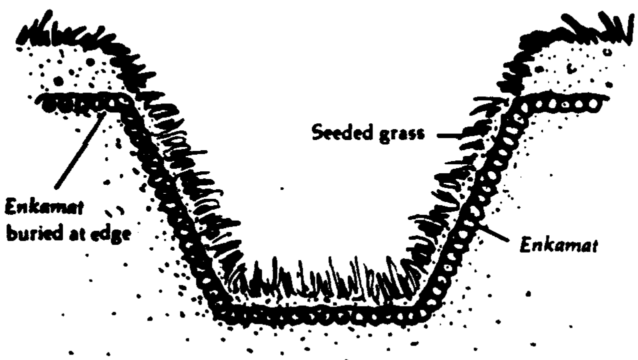


Throughout its life, *Enkamat* keeps the top layer of soil porous and, therefore, permeable to water. The presence of vegetation reduces water velocity and consequently its erosive effect on downstream areas. Thus *Enkamat* provides an erosion-proof surface at considerably lower cost than conventional permanent materials such as concrete, asphalt and rip rap.

Fig. 3.8

AMERICAN ENKA COMPANY · ENKA, NORTH CAROLINA 28728 · 704/667-7713

ENKA
AMERICAN ENKA COMPANY



Ditch Lined With Enkamat.

ENKAMAT REALLY WORKS!

Enkamat has been thoroughly tested and developed. It has proven itself effective on slopes up to 1:1 over a wide range of soil classifications. It has successfully withstood rainfall of 14″ in the first month after installation. Along highways, it has been used to stabilize natural soils as well as soils artificially compacted at 90% standard proctor. It is thoroughly compatible with all grass varieties and other types of vegetation.

Enkamat can be used successfully wherever a permanent channel lining or slope cover is needed. It is much more than just a mulching material: it is a product designed to provide years of effective turf reinforcement in critical areas. It is in harmony with its natural surroundings because it is so rapidly obscured by lush vegetation, as shown in the photographs at left.

Enkamat has definite cost advantages. It costs less to install and maintain than other proven permanent soil erosion control systems, and the potential savings in maintenance costs, both labor and material, can be even more significant. *Enkamat* will do the job right the first time.

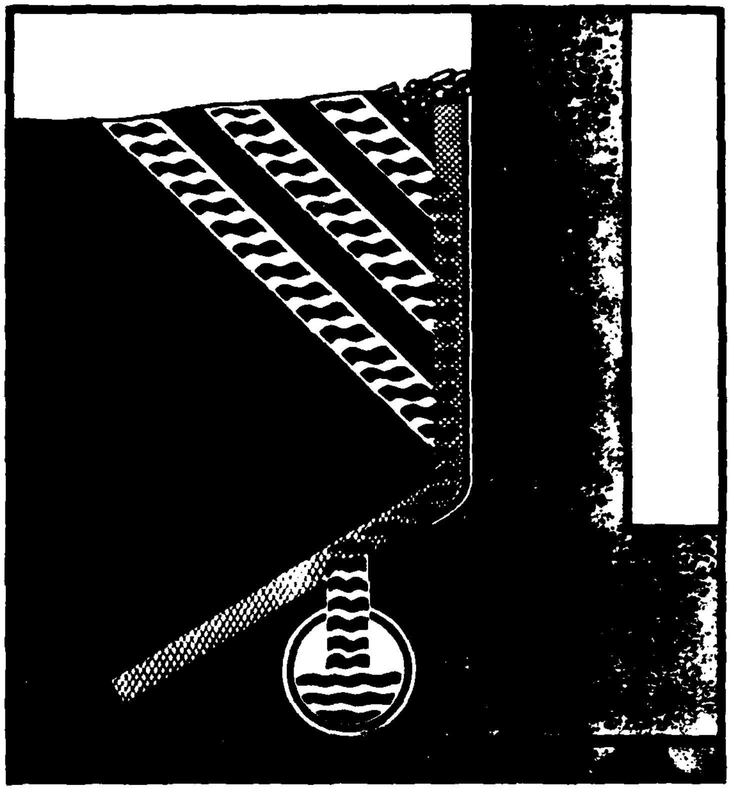
Enkamat is a trademark for a product of American Enka Company, Enka, North Carolina, a part of Akzona Incorporated, and is the subject of a number of United States patents.

Fig. 3.9

ENKADRAIN

A SOLUTION TO FOUNDATION DRAINAGE PROBLEMS

One of the primary elements in the design of basement and retaining walls is the provision of adequate drainage. This is because a buildup of moisture in the mass of earth resting against the wall increases the lateral pressure exerted against the wall by the weight of the earth. Where there is good drainage, this pressure is distributed evenly against the vertical surface of the wall and is imparted at a downward angle. A buildup of moisture, however, will concentrate and flatten the angle at which the pressure is applied, tending to rupture or overturn the wall. *Enkadrain* offers a solution to drainage problems where the water table is below the level of the foundation.



WHAT IS ENKADRAIN?

Enkadrain foundation drainage material is a two-layer composite consisting of a polyester nonwoven filter fabric heat-bonded to *Enkamat* matting, a compression-resistant nylon matting of open, three-dimensional construction. *Enkadrain* has been extensively tested and successfully marketed in Europe by American Enka's European associates.

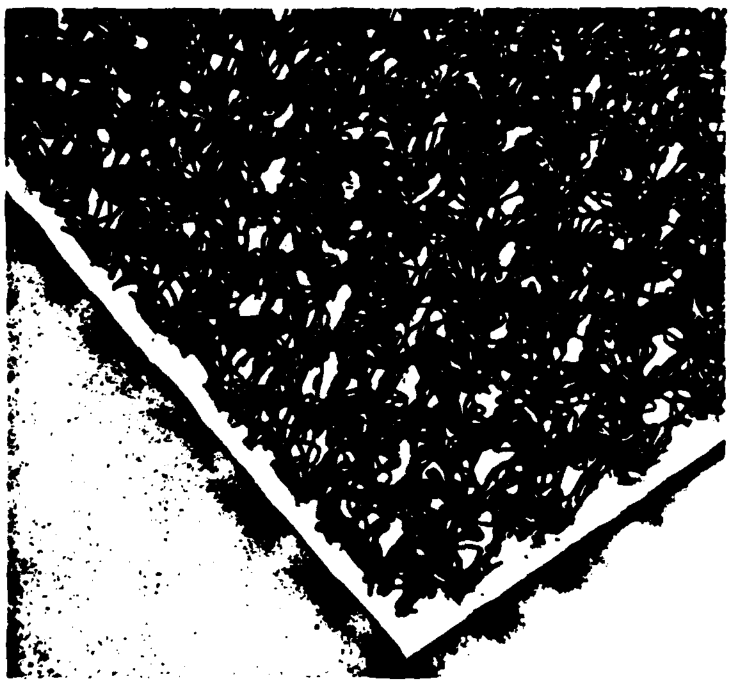


Fig. 3.10

WHAT DOES ENKADRAIN DO?

Enkadrain relieves hydrostatic pressure occurring in wet clay and silt soils adjacent to underground basement walls and retaining walls. Easy to install, it eliminates the problem of wet underground interiors and structural wall damage.

Enkadrain filters soil from water under pressure, providing a water escape route and hydrostatic relief drain which covers the *entire* area of the underground wall. *All* of the soil contacting the wall is thoroughly drained without clogging.

Quite often, depending on soil conditions, conventional water-proofing or dampproofing compounds eventually deteriorate and lose their ability to resist unyielding hydrostatic buildup. When this occurs, seepage or possible permanent damage follows.

Enkadrain relieves all of the pressure *before* it contacts the waterproofing membrane.

HOW IS ENKADRAIN USED?

Enkadrain is used in place of graded aggregates or gravel for vertical drainage purposes against underground walls, and to provide filtration for underground drainage pipes. Whereas gravel can, in time, clog and fail, *Enkadrain* will not.

Enkadrain is also used in place of protective boards or polyethylene film to protect waterproofing membranes from damage during backfilling.

Fig. 3.11

HOW IS ENKADRAIN INSTALLED?

Enkadrain can be easily, quickly and economically installed over the waterproofing membrane by one man, at the rate of 30 to 40 square meters per hour. There is a 3″ filter fabric overlap which allows adjacent strips of *Enkadrain* to be installed without open seams.

Above: *Enkadrain* laid over footing drain pipe

1. *Enkadrain* can be specified for unusual energy-efficient designs. 2. *Enkadrain* installed on vertical and horizontal parts of the structure. 3. Structure wrapped by *Enkadrain*. 4. & 5. *Enkadrain* conforms to creative design details. 6. Completed project

Fig. 3.13

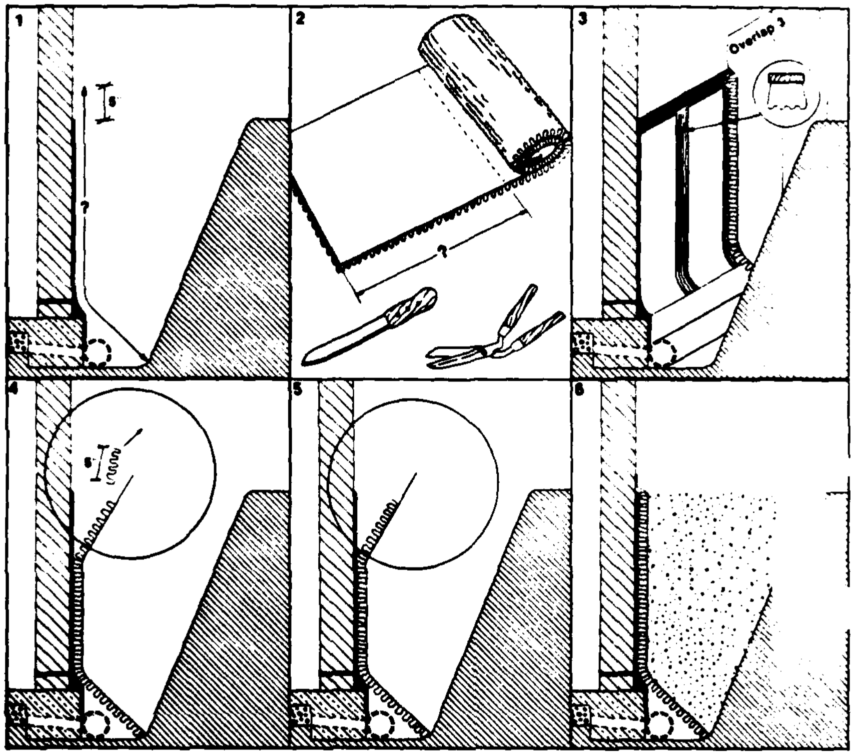
INSTALLATION INSTRUCTIONS 184

There are two recommended installation procedures:

1. **Gluing***

   Enkadrain is first cut to the desired length, ensuring that it extends far enough down to cover the footing drain. The top 4" to 6" of the black nylon mat is then cut away and the exposed filter fabric is glued to the foundation wall. Adjacent strips are installed by overlapping the 3" filter fabric excess over the previously installed strip.

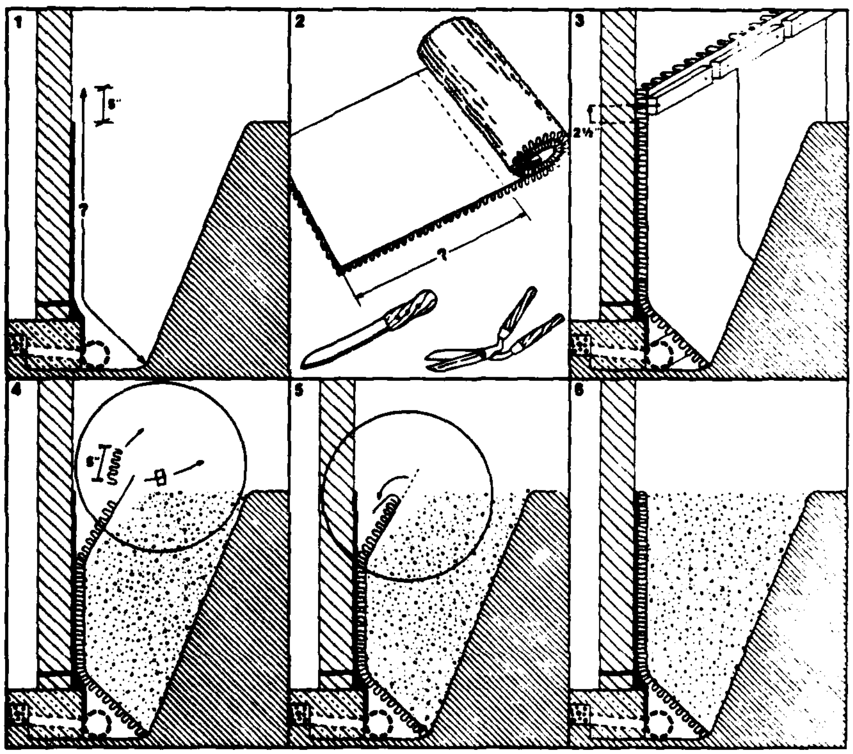
   1. Measuring 2. Cutting
   3. Applying Glue
   4. Shortening 5. Wrapping
   6. Pressing



2. **Nailing**

   This method is similar to the above, except that, instead of gluing the exposed filter fabric to the foundation wall, it is nailed to the wall above ground level by using furring strips and mortar nails. After backfilling, the furring strips are removed.

   1. Measuring 2. Cutting
   3. Batten Nailing
   4. Shortening 5. Wrapping
   6. Pressing



*Recommended glue: Goodyear *Plio-bond*™ or *Fastbond-10*™ contact cement by 3M.

***Enkadrain*, *Enkamat* and *Stabilenka* are trademarks for products of American Enka Company, Enka, North Carolina, a part of Akzona Incorporated, and are the subjects of a number of United States patents.**

AMERICAN ENKA COMPANY • ENKA, NORTH CAROLINA 28728 • 704/667-7713

ENKA
AMERICAN ENKA COMPANY

Fig. 3.13

STABILENKA™

A FAMILY OF FILTER FABRICS

Stabilenka is the brand name for a family of nonwoven polyester fabrics developed specifically for soil engineering purposes. Generally, the fabric functions as a soil/water filtration medium.

Available in different weights and strengths to suit a variety of soil engineering applications, *Stabilenka* filter fabrics offer significant performance advantages and cost savings over conventional filtration materials such as gravel and graded aggregates.

FUNCTIONS & CAPABILITIES

Stabilenka can provide a permanent filter **medium** for underground drainage systems by preventing soil fines from clogging water ducts. By performing soil/water filtration, *Stabilenka* can **prevent erosion** of soil around and under waterway lining materials such as riprap rock and sand bags. It can perform an **earth reinforcement** function when used to stabilize soils with low bearing capacities on steep slopes.

Stabilenka can perform efficiently as a **separation layer** between road base and subgrade materials on gravel and asphalt paved roadways. In this capacity, it prevents the blending of the base materials with unstable subgrade soils and helps distribute traffic loads over a greater area, thereby eliminating many expensive maintenance problems. *Stabilenka* can control **sedimentation runoff** originating from construction projects when installed as silt fencing or as a brush barrier filter. Preventive **measures** of this type are now required by many state and municipal regulations.

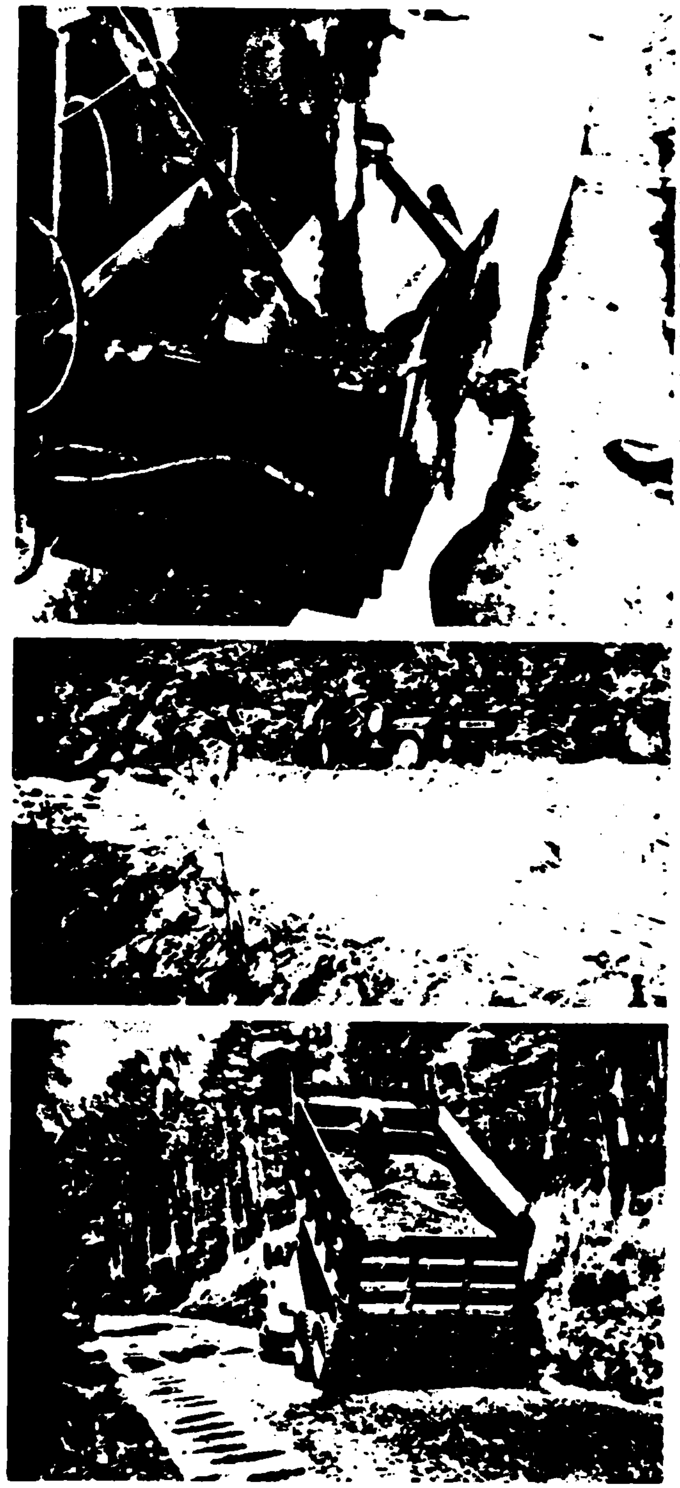
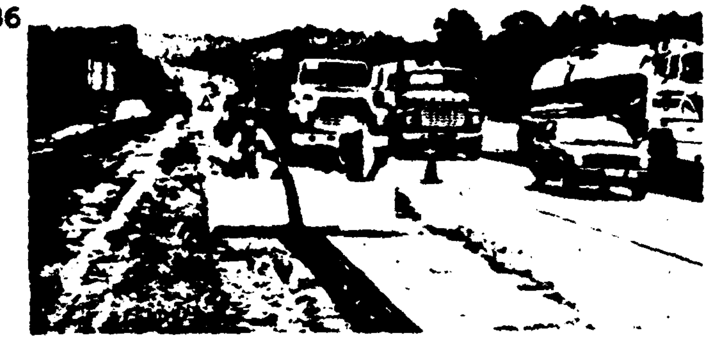


Fig. 3.14

APPLICATIONS

As A Filter Medium



Underground drainage trenches are being specified more frequently as a drainage technique, particularly in the area of highway construction. This is because efficient, low-cost filtration materials like *Stabilenka* are now available. *Stabilenka* prevents the clogging of drainage ducts — usually gravel or perforated plastic pipe — at the same time allowing the passage of underground water into the ducts.

Consequently, the subgrade materials maintain shear strength and are not subject to plastic flow or total shear failure. This preserves the ability of roadbeds to support heavy loads without cracking.

To Prevent Erosion

Stabilenka Type 100, 84" wide, is frequently specified as a filter underlay beneath rock or sand bags used for erosion control on river banks and lake and ocean shorelines. In the past, more expensive filtering agents such as graded aggregates or sand have been used.

In tests, *Stabilenka* has survived the impact of 400-pound riprap rocks dropped from a height of four feet directly onto the fabric, which was resting on a slope of 1.5:1.

An overlap 1' wide should be used when installing *Stabilenka* for this application.

Below: Shoreline erosion.
Bottom right: *Stabilenka* used in solving the problem.

Fig. 3.15

As Earth Reinforcement

Stabilenka has been used to construct fabric walls which perform an earth reinforcement function in places where rock and gravel have failed to provide a stable foundation. In this application, layers of *Stabilenka* and soil are alternately placed on top of each other until final grade is obtained. The *Stabilenka* fabric is folded back over the soil after each layer is deposited.

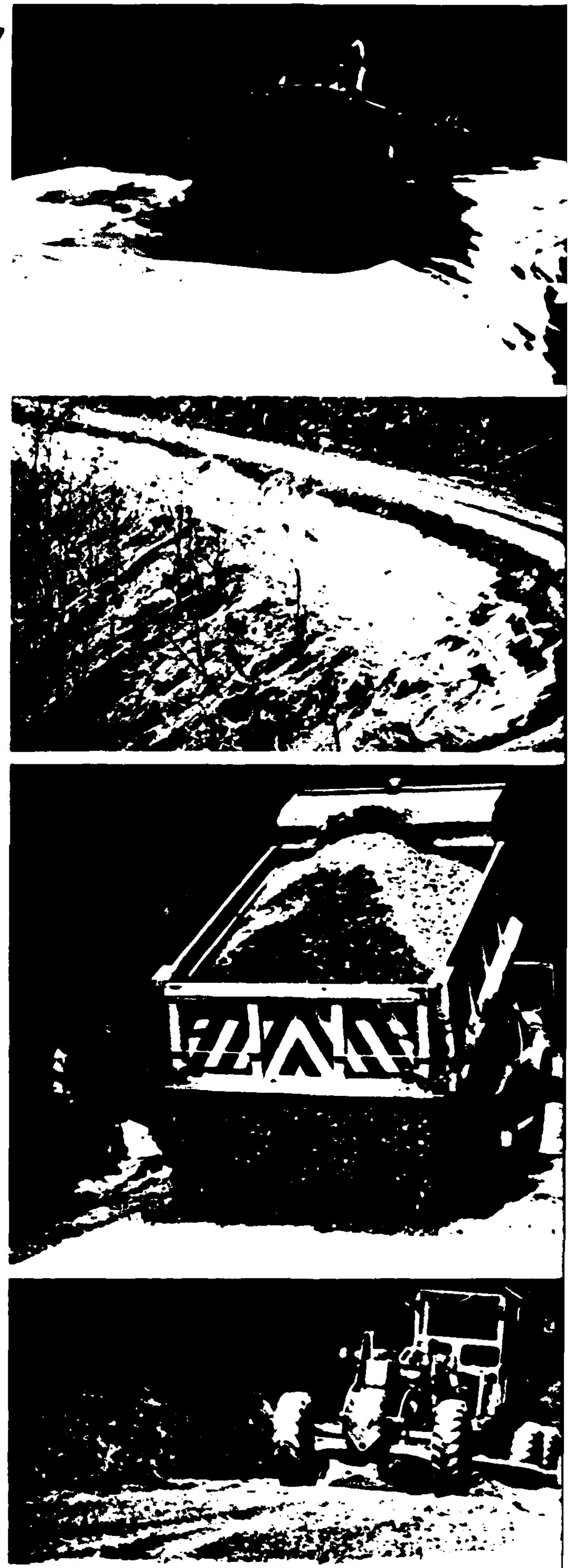
Stabilenka should be overlapped 1′ wide for this application. Also, the exposed fabric should be sprayed with asphalt emulsion after installation in order to prevent ultraviolet light degradation.

As A Separation Layer

Stabilenka can prevent the 'sinking' of gravel into unstable roadbed subsoil by providing a separation and filtration layer. When used in this way, *Stabilenka* can reduce the quantity of gravel required to provide trafficability on secondary roads, and also reduces *maintenance costs by significant amounts.*

On construction sites, *Stabilenka* can facilitate the movement of equipment and the maintenance of construction schedules by stabilizing mucky soils. It can also eliminate the need for the dredging and replacement of poor subgrade soils during highway resurfacing.

Again, for this application, *Stabilenka* should be overlapped 1′. When fabric and gravel are placed over wet soil, the gravel should be back-dumped to avoid damaging the fabric.



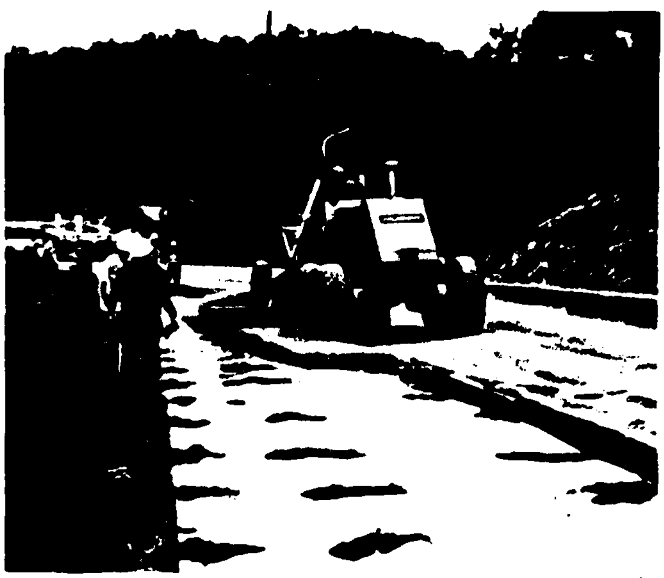
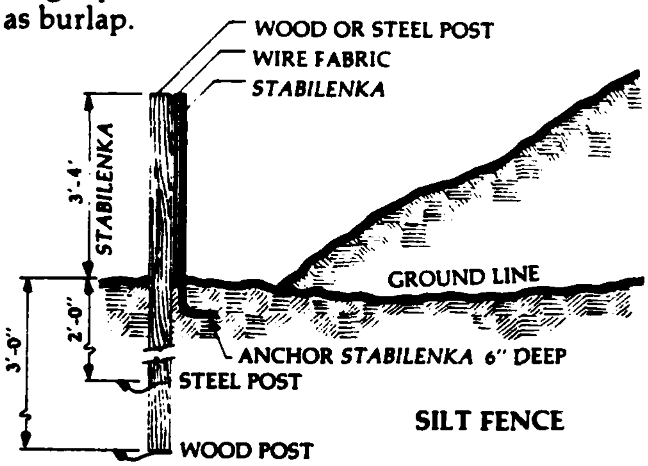


Fig. 3.16

To Control Sedimentation

Stabilenka Types 80 and 100 have been approved as filter media for silt fences and brush barriers. These temporary structures provide inexpensive and efficient control of sedimentation originating from construction sites. They have definite advantages over alternatives such as hay bales and sediment basins. *Stabilenka* can perform these functions without deterioration over a much longer period than conventional material such as burlap.

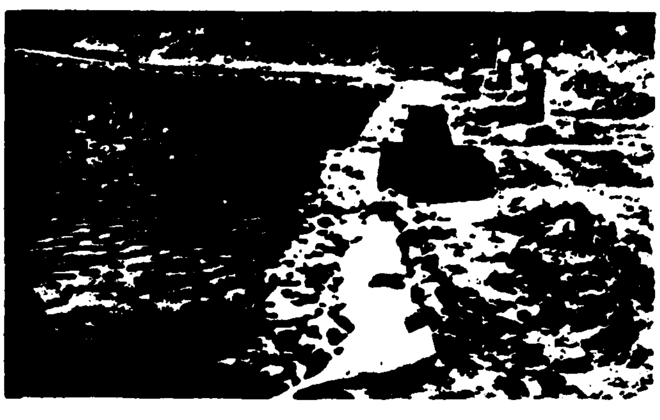


SILT FENCE

In Conjunction With *Enkamat*

Stabilenka can be used in conjunction with *Enkamat* soil reinforcement matting to provide a unique erosion control composite. *Stabilenka* is placed under *Enkamat* for maximum permanent turf reinforcement in those places where water velocities and quantities have the greatest erosive potential, for example on bridge aprons, ocean shorelines and culvert outputs.

In these situations, *Stabilenka* and *Enkamat* are anchored to the soil and seeded. As the vegetation becomes established, its roots will penetrate the *Stabilenka* fabric; thus providing additional anchoring power between the turf and its soil.



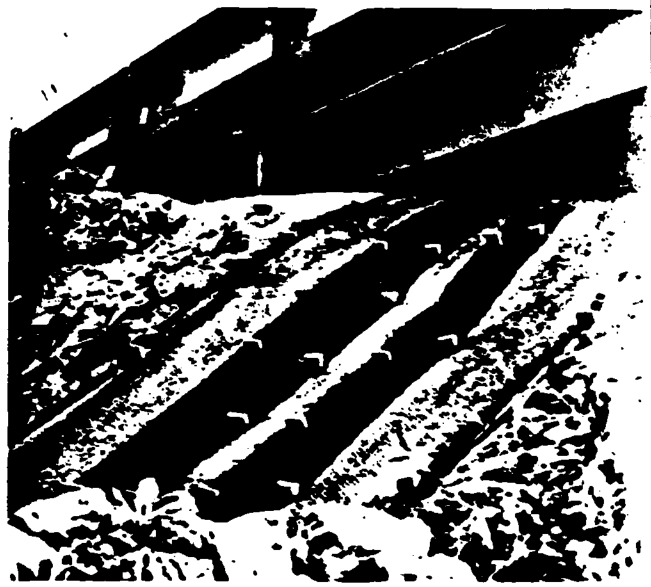


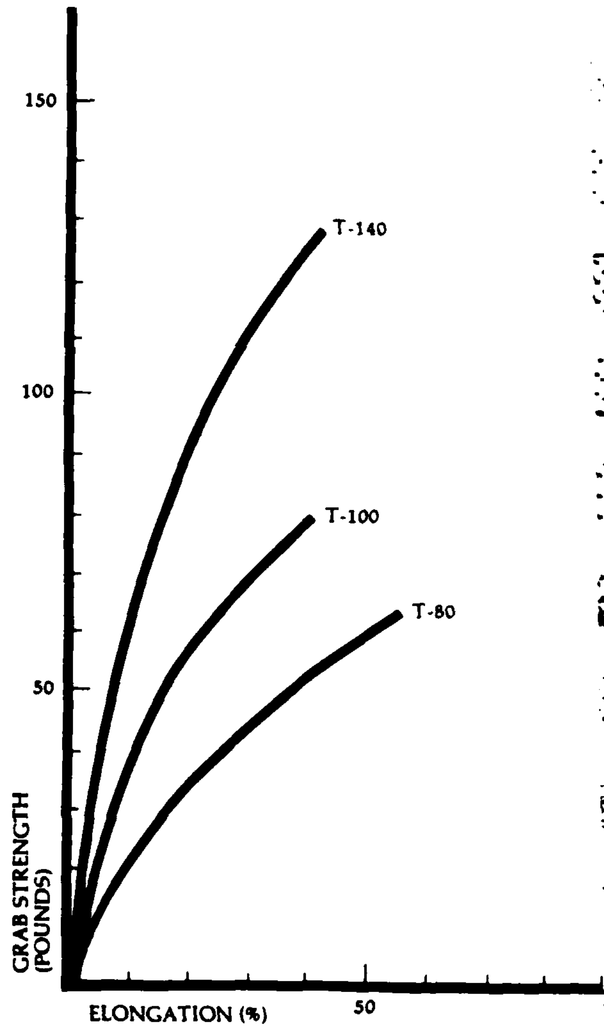
Fig. 3.17

NONWOVEN FOR STRENGTH & VERSATILITY

Stabilenka is a nonwoven fabric, and as such can be manufactured to satisfy a great variety of performance requirements, chief among which is uniform filterability. Nonwovens exhibit a high strength/weight ratio and excellent multi-axial strength. In addition, they generally offer superior performance at a much lower cost than fabrics made via other manufacturing methods.

POLYESTER FOR DURABILITY

Stabilenka is a polyester fabric. This polymer was specifically selected because its inherent fiber properties make it highly conducive to civil engineering applications. Polyester's high modulus and strength offer superior dimensional stability under heavier loads when compared to fabrics containing many other polymers. Polyester has a high specific gravity and this, for engineering purposes, reduces the bouyancy problems associated with underwater installations. In the many situations where the fabric is to be exposed to light, *Stabilenka's* polyester structure gives it a high degree of resistance to ultraviolet degradation. It is similarly resistant to biological and chemical attack.



STABILENKA LOAD–ELONGATION CURVES –LENGTH DIRECTION

Enkamat and *Stabilenka* are trademarks for products of American Enka Company, Enka, North Carolina, a part of Akzona Incorporated, and are the subject of a number of United States patents.

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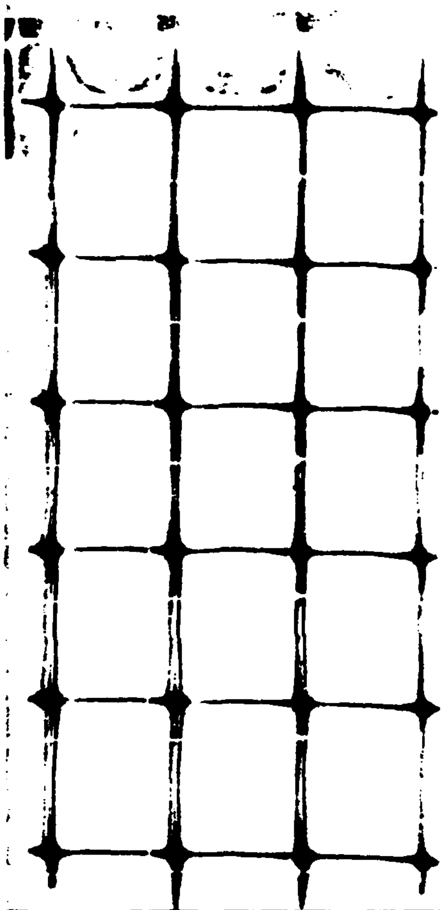
Fig. 3.18

Conwed
innovative products for better environments



ECONOMY EROSION CONTROL NETTING

"Holds Mulch In Place for Better Turf Results"

WHAT IS CONWED EROSION CONTROL NETTING?

This patented plastic netting is designed for maximum erosion control. It is strong and durable, yet lightweight and easy to handle. The net has a square mesh opening of approximately 3/4" x 3/4". Erosion control netting is available in the following rolls

Width	Length	Weight Per Roll	Coverage
3-3/4'	2,500'	35 lbs	1,041 sq. yds.
7-1/2'	2,500'	70 lbs	2,083 sq. yds.
15'	2,500'	140 lbs	4,166 sq. yds.

HOW IS CONWED EROSION CONTROL NET USED?

Conwed netting is designed for areas which have been seeded and mulched with straw, hay, shredded bark, wood chips, or other loose mulches. The netting is unrolled and stapled over locations susceptible to wind or water erosion. Typical applications are highway medians, ditch bottoms, and slopes. Erosion control netting can also effectively hold sod in place.

WHAT DOES CONWED EROSION CONTROL NET DO?

The netting physically holds the mulch or sod in place, reducing the incidence of erosion. By keeping the loose mulches intact over grass seed, germination is enhanced. The green colored net blends with the natural surroundings and is gradually disintegrated by sunlight as the turf is established.

WHY IS CONWED EROSION CONTROL NET SUPERIOR?

- A variety of widths are available so you can order the size needed to best suit the job.
- Greater coverage per roll-reduces application time.
- Little or no shrinkage after application.
- Strong and durable polypropylene plastic holds up during application and use.
- Non-irritating to the skin.
- Lightweight, easy to handle.
- Less expensive than blanket-type products.

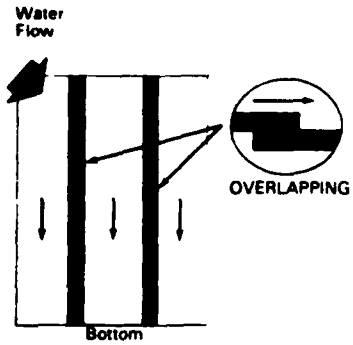
INSTALLATION INSTRUCTIONS

SEEDING AND MULCHING

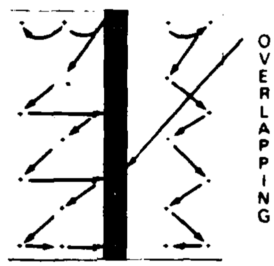
... No asphalt tack is needed with straw ...

INSTALLATION ON SLOPES

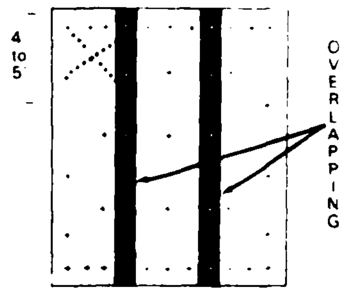
Unroll net from top to bottom of slope. Overlap adjacent net 2-4" with the net on the upstream side of any lateral water flow on the top.



Staple the top end first, starting in center of net and work to outside edges. Move down slope. Always work from center out.

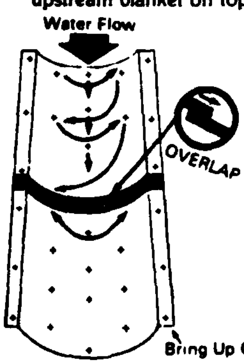


Using square top staples, insert one every 1-2 feet along the side, top and bottom ends. Remaining area should receive approximately one staple per square yard. These distances are suggested only and may be varied due to severity of conditions.

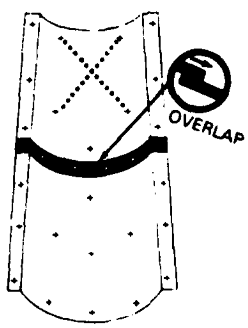


INSTALLATION IN DITCHES

Unroll netting in direction of water flow. It is essential and desirable to allow the netting to extend over top edge of ditch 4-6 inches (over highest waterline). Overlap ends of netting 2-4 inches with upstream blanket on top. Staple end of netting first starting in middle and continue down center of netting. Next, work toward outside edge applying interior staples first. Last, apply staples on outside edge.

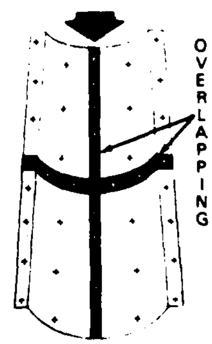


Using square top staples, insert one every 1-2 feet along end of netting and at every overlap strip. Remaining areas should receive approximately one staple per square yard. Distances are suggested and may be varied due to severity of conditions. Note: More staples should be used to tie netting down in ditches to insure maximum coverage of contour.



(When two netting strips must be used to cover ditch.)
Use procedure as before, but vary following steps:

1) Overlap two netting strips in middle of ditch
2) Staple all overlaps in 1-2 foot intervals



The performance data herein reflects Conwed's expectation based on tests conducted in accordance with recognized standard methods. The sale of these products shall be subject to the Terms and Conditions of Sale, **including those limiting warranties,** *as set forth in Conwed's sales forms.*

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Fig. 3.20

list the remaining items with cursory remarks and leave further investigation to the reader (The list is not considered complete.).

1. Tung oil
2. Linseed oil
3. Cottonseed oil
4. Castor oil
5. Rubber latex
6. Plasticized sulfer
7. Molasses
8. Mineral oil
9. Sodium carbonate
10. Calcium carbonate
11. Paraffins
12. Hydrofluoric acid
13. Ionic detergents

None of the above groups is now generally used for soil stabilization (Lambe, 1962).

14. Sodium silicate - most widely used in grouting and injection.
15. Phosphoric acid

Natural and synthetic polymers are long-chained molecules formed by the linking of certain organic chemicals called monomers. Following is a list of a few which have been tried for soil stabilization (Lambe, 1962):

Natural Resins

16. Vinsol resins - imparts almost no strength to soil, but appears to waterproof the finer-grained soils.
17. Rosin - when added to a soil and then reacted with certain metal salts forms insoluble gels which aid stabilization.

Natural
Resins (Cont'd)

18. Resin Stabilizer 231 - a rosin derivative; reduces the rate and amount of water absorption and furnishes sodium ions for ion exchange.
19. Stabinol - a rosin derivative mixed with portland cement; waterproofs the coarser soils.
20. NSP-121 and NSP-252: Materials made by National Southern Products Co.; have given some water repellency to certain soils.
21. NVX - a product of the Hercules Powder Co.; has given some water repelling to certain soils.
22. Shellac - Sand treated with shellac can have high strength but practically loses it all in water.

Synthetic
Resins

23. Aniline-furfural - waterproofs and strengthens soil.
24. Polyvinyl alcohol (PVA) - forms tough, flexible films when evaporated from aqueous solutions. Films are water-soluble however.
25. Polyvinyl acetate - Same results as shellac (item 22).
26. Resorcinol-formaldehyde - Gives some strength to sand but loses it in water.
27. Urea-furfural and phenol-furfural resins.
28. Phenol-formaldehyde combinations.
29. Urea-formaldehyde resins.
30. Calcium sulfamate-formaldehyde resins.

Synthetic
Resins (Cont'd)

31. Ethocel.
32. Methylol ureas and melamine.

The following are methods utilized for in-situ deep densification of cohesionless soils (Mitchell, 1981):

33. Blasting.
34. Vibrocompaction and compaction piles.
35. Heavy tamping

Other soil stabilization methods include (Mitchell, 1981):

36. Precompression - preloading a soil prior to construction is one of the oldest and most widely used methods to strengthen and preconsolidate weak and compressible soils.
37. Precompression by Electro-Osmosis
38. Deep mixing - The in-situ mixing of admixtures, usually lime or portland cement, with soft, fine-grained soils to form columns piers, and walls (Mitchell, 1981).
39. Thermal stabilization.

CONCLUSION AND SUMMARY

The essential aspects of a large number of soil improvement methods have been presented in this chapter. The majority of these methods are not suitable for erosion control, the subject at hand. But, I felt it important to convey that a great deal of research has been done and many technological advances have been made in the field of soil stabilization. However, for erosion control, vegetation and mother nature still appears to be the best overall solution to the problem. The selection of the most suitable stabilization method for a given project can only be made after careful consideration of all factors involved. Such factors include (Mitchell, 1981):

1. What the treated ground will be used for.
2. The area, depth, and volume of soil.
3. Soil type and in-situ properties.
4. Materials available.
5. Equipment and skills available.
6. Environmental factors.
7. Local experiences and preferences.
8. Time available.
9. Cost.

The engineer has the choice of accepting the limitations imposed by the in-situ soil properties, or improving the properties by stabilization as a means of fulfilling the design criteria. Stabilization is commonly undertaken to improve the strength and stiffness, but it may be necessary to adopt some procedure to improve such properties as permeability, durability, and volume stability. In general, the improvement is effected by controlling the void ratio, by introducing a cementing or waterproofing agent, or by injecting a substance to fill the pore volume. The stabilization technique primarily depends on the nature of the soil.

CHAPTER FOUR

THE TRESTLE IN-SITU CONDITIONS AND EROSION PROBLEMS

INTRODUCTION

(Ungvarsky, 1982) The TRESTLE, located on Kirtland AFB; Albuquerque, New Mexico, is an impressive sight, to say the least. Some idea of its mass and volume may be gained from the photos provided at the introduction of each chapter of this text. The facility receives its name from the large timber structures which are so predominant, somewhat resembling railroad trestles often seen spanning wide mountain gorges. The main timber structure is the Test Stand which is approximately 200 square feet towering 117 feet high--almost as tall as a 12 story building. The long narrow timber structure which joins the edge of the arroyo with the Test Stand is the Ramp which is 50 feet wide and almost 400 feet long. The last large timber structure in the complex is called the Terminator Stand.

Since these structures are dielectric and are fastened by wooden bolts, they enable scientists to test aircraft in a simulated flying mode without disturbing the electromagnetic (EM) fields generated in the Test Stand area. The EM field is a short duration pulse (roughly on the order of millionths of a second) electric field generated between the ground planes by quickly simultaneously discharging the two 5 million volt pulsers housed in the white gas boxes at the south end of the facility. The gas boxes are also supported by dielectric timber stands called the Pulser Stands. The transmission lines into which the pulsers are discharged terminate at the top of the Terminator Stand into a bank of resistors which absorb the electrical energy. When an aircraft is on the stand for testing, electromagnetic pulse (EMP) sensors are placed in strategic locations on and in the aircraft to detect the electric signals caused by the simulated EM field. The EM field is a wave of energy which simulates the field generated by a high altitude nuclear explosion. This field normally can radiate hundreds of miles in all directions following a nuclear burst.

The TRESTLE design began in 1973 with an Air Force contract to the McDonnell Douglas Astronautics Corporation (MDAC). MDAC was the integrating contractor which produced the design of the facility and started the construction. With the completion of the design, the excavation of the earthen bowl, the laying of utilities, and the construction of the steel wedge building, MDAC's work was complete. The Air Force Weapon Laboratory (AFWL) then performed the role of integrating contractor for construction and facility checkout. During this phase, the Allen M. Campbell Co. was the prime contractor for construction. This included the erection of all wood structures, the pulser installation, and the installation of the transmission line system with the terminator resistor bank. A separate effort was later identified and the Campbell Co. installed an aircraft checkout area consisting of a concrete pad, a nitrogen inerting system, and an air conditioning system for cooling the onboard aircraft electronics and checkout systems.

Construction of the wood structure began in 1976. The wood used is Glulam (Glue-laminated) which is an engineered lumber. The lumber was monitored very carefully from beginning to end to ensure quality results. The structure can hold the Air Force's heaviest aircraft, the C-5 Galaxy, which weighs 550,000 pounds. Loaded with the C-5, it can withstand a 40 mph wind. Without the aircraft, it's designed to take winds up to 100 mph. It contains more than six-million board feet of lumber, enough to build 4,000, 3-bedroom houses, and is held together by more than 250,000 wooden bolts. It's the largest Glulam structure in the world--that's impressive!

Figure 4.1 shows the TRESTLE and an Air Force B-52 aircraft. Reference 4.1 provides a cleared for public release synopsis of information concerning the TRESTLE.

CLIMATE

(USDA, 1977) The KAFB TRESTLE is located in the southeast sector of Albuquerque, New Mexico, a part of Bernalillo County, the central part of the state. The Rio Grande flows southward through the center of Bernallio

Fact Sheet
United States Air Force

OFFICE OF PUBLIC AFFAIRS, AIR FORCE CONTRACT MANAGEMENT DIVISION, KIRTLAND AFB NM 87117, (505) 844-6644

AIR FORCE WEAPONS LABORATORY

TRESTLE AND EMP TESTING

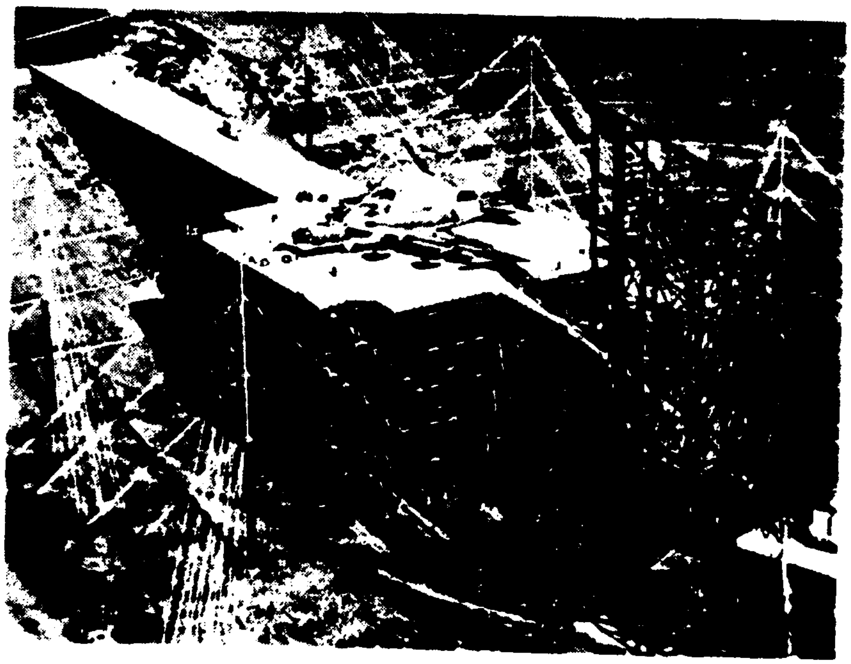

Fig. 4.1

An Air Force B-52 sits atop the 200-foot square platform of the TRESTLE Electromagnetic Pulse Simulator at Kirtland AFB, N.M. In the facility, the intense electromagnetic wave which is produced by a nuclear explosion is simulated and its effect on U.S. aircraft and their electrical components are measured. Scientists and engineers can thus develop measures to minimize these effects. (Official U.S. Air Force Photo)

(Current as of November 1983)

The Air Force's TRESTLE Electromagnetic Pulse (EMP) Simulator is a 12-story high, all-wood facility which enables scientists and engineers to simulate the in-flight EMP effects on aircraft electrical equipment.

An EMP is an electromagnetic wave created by a high altitude nuclear explosion, which travels hundreds of miles in all directions from the origin of the nuclear detonation. TRESTLE simulates this phenomenon by using two 5-million volt pulsers which discharge into wire antennae that surround the aircraft. Special electrical sensors measure aircraft EMP response signals, and fiber optic cables transmit them to computers inside a shielded enclosure for recording and later analysis by scientists and engineers.

The TRESTLE design began in 1973 when the Air Force designed a large aboveground wooden facility which was capable of supporting a large stationary aircraft. In 1976, construction of the wooden structure began; wood was chosen instead of steel and concrete because its nonconductive and nonmagnetic qualities would allow simulation of aircraft as if they were in flight.

Aircraft under test are parked on TRESTLE's 200-foot-square laminated wood deck, 118 feet aboveground. Access to the deck is across a 400-foot-long, 50-foot-wide wooden ramp. TRESTLE will accommodate aircraft the size of the giant C-5 Galaxy which weighs at least 550,000 pounds. It is built from wooden columns connected by wood crossmembers and held together with approximately 250,000 wooden bolts. It is the largest glued-laminated wood structure in the world. Over six-million board feet of lumber was used--enough to build 4,000 frame houses.

First tests occurred in March 1980 using a B-52 aircraft. The investment cost of the facility is approximately $58 million.

For additional information, contact AFCMD/PA, Kirtland AFB, NM 87117, (505) 844-6644 or AV 244-6644.

Reference 4.1. Synopsis of Information Concerning the TRESTLE.

County with the land rising on both sides to form mesas that have elevations of about 5,000 feet. To the east, the mesa is narrow and is just beyond the Sandia and Manzano Mountains. Tijeras Canyon, a main east-west highway pass, separates these two ranges. (The Tijeras Arroyo runs through KAFB, passing adjacent to the TRESTLE, and serves as the avenue for discharging surface drainage from the site.) Sandia Crest peaks at 10,678 feet, and forested mountain slopes decrease in elevation eastward in the central highlands. The valley and mesa areas are arid, having average annual precipitation near eight inches. In the mountains to the east, average annual precipitation ranges from 15 to 30 inches; the amount generally increases with increasing elevation.

Summer is the rainy season. Half the annual average falls during July to October; typically, as brief but often heavy thunderstorms. An average of 44 such storms occur each year, mostly during this period.

There is considerable variation in precipitation from year to year and month to month. The average maximum 24-hour precipitation amount is two or three inches, but an unofficial amount of four inches maximum has been reported. The average number of days having 0.10 inch or more precipitation ranges from 22 in the valley to 54 in the mountains. The average number of days having precipitation of 0.50 inch or more ranges from two in the valley to 17 at Sandia Crest.

Sunshine occurs more than 75 percent of the possible hours, or nearly 3400 hours a year, and is fairly evenly distributed in all seasons.

The average annual wind speed is nine (9) mph. Spring is the windy season and if weather is dry, soil blows occasionally. A brief period of soil blowing can also occur just before a thunderstorm. Winds blow most frequently from the north in winter, and from the south along the river valley in summer. In Tijeras Canyon, the heavy cold air held back by the Sandia and Manzano Mountains finds access to the basin and literally pours through the canyon, spreading out on the mesa and valley below in gusts of up to 50 mph.

Reference 4.2, from the National Oceanic and Atmospheric Administration (NOAA), provides an additional climatological summary for Albuquerque, New Mexico. Tables 4.1 to 4.8 provide climatological data for 1982, also from the NOAA.

SOIL PROFILE

USDA (1977) describes the soil where the TRESTLE is located as bordering on the Bluepoint soil series and the Wink soil series. The Bluepoint series consists of deep, somewhat excessively drained soils that formed in sandy alluvial and eolian sediments on alluival fans and terraces. Slopes are 1 to 15 percent. The native vegetation is principally mesa dropseed and Indian rice grass, with some giant dropseed and black grama. Eleva-

Local Climatological Data

Annual Summary With Comparative Data

1982

ALBUQUERQUE, NEW MEXICO

Narrative Climatological Summary

'Arid Continental' characterizes the climate of Albuquerque and vicinity in a minimum number of words. With an average annual rainfall of near eight inches there is generally insufficient natural moisture to maintain the growth of any but the most hardy desert vegetation. However, successful farming is carried on in the valley by irrigation and considerable fruit and produce are raised. In the mountains east of the City precipitation is considerably heavier. At Tijeras Ranger Station, about 15 miles east of Albuquerque, the average annual rainfall is around 15 inches. Some dryland farming is carried on in this mountain area and native vegetation shows the effect of the heavier rainfall with good native grass cover and timbered mountains. The average monthly precipitation at Albuquerque varies from less than one-half inch during the winter months, November through March, to over an inch and a quarter during the months of July and August. With normally less than two inches of moisture, the winters are generally very dry. A considerable portion of this meager winter precipitation falls in the form of snow, but the monthly fall exceeds 3 inches infrequently and there are normally only four days a year when as much as one inch of snow occurs. Snow rarely remains on the ground in the valley for more than 24 hours but in the nearby mountains, snow cover is normal from the middle of December until early spring and a modern ski resort operates during the winter months just 25 miles from the City. The July-September period furnishes almost half of the annual moisture with most of the rain falling in the form of brief but at times rather heavy thundershowers. Prolonged rainy spells are practically unknown. These summer showers do not materially interfere with outdoor activities but do have a considerable moderating effect on summer daytime temperatures.

Temperatures in Albuquerque are those characteristic of high altitude, dry, continental climates. The average daily range of temperature is relatively high but extreme temperatures are rare as testified by the fact that there is normally less than one day a year when the temperature reaches 100° or drops to zero. Daytime temperatures during the winter average near 50° with only a few days on which the temperature does not rise above the freezing mark. In the summer, daytime maxima average less than 90° except in July and with the large daily range, the nights normally are comfortably cool. The air is normally dry with an average annual relative humidity of approximately 43%. "Muggy" days are unknown and the usual humidity during the warmer part of the day is about 30%, dropping down to less than 20% in June, the least humid month of the year.

Another feature of the climate is the large number of clear days and the high percentage of sunshine. Sunshine is recorded during more than three-fourths of the hours from sunrise to sunset and this high percentage carries through the winter months when clear, sunny weather predominates. Wind movement throughout the year averages around nine miles per hour, but during the late winter and spring months the average is somewhat higher and occasional windy and dusty days occur. These occasional dust storms are the most discomforting part of Albuquerque's climate. However there are on an average only 46 days during the year when the maximum wind speed reaches 32 miles per hour. Tornadoes rarely occur in the vicinity of Albuquerque.

Reference 4.2. Narrative Climatological Summary for Albuquerque, New Mexico.

noaa NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION / NATIONAL ENVIRONMENTAL SATELLITE, DATA, AND INFORMATION SERVICE / NATIONAL CLIMATIC DATA CENTER ASHEVILLE, N.C.

Table 4.1. Meteorological Data For The Current Year

Station: ALBUQUERQUE, NEW MEXICO # 23050 INTL AIRPORT-KIRTLAND AFB Standard time used: MOUNTAIN Latitude: 35° 03' N Longitude: 106° 37' W Elevation (ground): 5311 feet Year 1982

Month	Temperature °F Average: Daily maximum	Daily minimum	Monthly	Extremes: Highest	Date	Lowest	Date	Degree days Base 65 °F: Heating	Cooling	Precipitation in inches, Water equivalent: Total	Greatest in 24 hrs.	Date	Snow, Ice pellets: Total	Greatest in 24 hrs.	Date	Relative humidity, pct. Hour 05	Hour 11	Hour 17	Hour 23 (Local time)	Wind, Resultant: Direction	Speed m.p.h.	Average speed m.p.h.	Fastest mile: Speed m.p.h.	Direction	Date	Percent of possible sunshine	Average sky cover, tenths, sunrise to sunset	Number of days, Sunrise to sunset: Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog, visibility ¼ mile or less	Temperature °F Maximum (b) 90° and above	32° and below	Minimum 32° and below	0° and below	Average station pressure mb. Elev. 5314 feet m.s.l.
JAN	49.0	22.8	35.9	63	25	10	4	895	0	0.32	0.19	12-13	3.6	2.1	12-13	71	48	31	60	28	1.8	9.2	40	W	21	75	4.9	14	8	9	6	1	0	1	0	0	30	0	837.1
FEB	52.5	26.2	39.4	70	23	8	6	709	0	0.20	0.10	11	1.2	1.0	1	79	50	39	58	01	4.2	9.9	36	E	25	61	6.0	8	2	18	4	1	0	2	0	0	22	0	839.1
MAR	62.0	32.8	47.4	73	18	19	6	538	0	0.84	0.41	13-14	0.7	0.7	5	62	36	29	46	25	3.9	10.3	46	SW	29	73	5.9	9	9	13	6	0	2	0	0	0	15	0	835.4
APR	71.5	40.6	56.1	83	29	28	8	268	6	0.05	0.05	23	T	T	22	46	24	18	32	21	1.0	13.3	46	E	30	81	5.9	7	13	10	1	0	0	0	0	0	2	0	836.0
MAY	79.3	46.6	63.0	90	26	32	7	94	38	0.52	0.39	22	0.0	0.0		53	30	23	37	23	3.1	10.0	38	S	22	82	4.0	17	7	7	4	0	3	1	1	0	1	0	835.1
JUN	91.2	58.4	74.8	99	28	48	3	8	301	0.09	0.05	20	0.0	0.0		44	21	12	29	15	2.7	10.6	42	E	19	91	3.4	16	11	3	2	0	2	0	18	0	0	0	836.4
JUL	93.9	64.2	79.1	100	21	55	2	0	441	1.32	0.31	30	0.0	0.0		54	30	25	43	24	1.5	8.8	42	E	11	79	4.5	12	17	2	8	0	11	0	26	0	0	0	840.2
AUG	90.9	63.9	77.4	96	16	57	27	0	394	1.09	0.47	10-11	0.0	0.0		74	43	39	60	12	0.9	7.6	38	E	12	77	5.1	8	19	4	11	0	15	0	20	0	0	0	842.5
SEP	82.2	56.7	69.5	97	2	42	29	23	163	1.34	0.55	17-18	0.0	0.0		71	43	38	50	15	2.3	9.3	38	S	16	73	5.3	11	8	11	10	0	6	1	5	0	0	0	839.8
OCT	69.7	39.9	54.8	83	5	27	29	314	4	0.26	0.24	12	0.0	0.0		57	34	23	43	35	1.1	7.9	37	E	19	85	4.0	15	9	7	7	0	0	1	0	0	4	0	840.2
NOV	54.8	30.9	42.9	66	7	24	15	650	0	0.60	0.23	17-18	0.9	0.9	26-27	74	50	45	64	17	1.3	7.8	42	E	23	63	6.4	6	10	14	7	0	1	2	0	0	19	0	838.5
DEC	45.1	23.7	34.4	58	10	8	29	941	0	0.78	0.29	10-11	3.3	2.9	30	80	58	54	74	06	1.9	8.7	41	?	7	61	5.5	12	7	12	6	1	2	0	0	2	30	0	837.8
YEAR	70.2	42.2	56.2	100	JUL 21	8	DEC 29	4440	1347	7.41	0.55	SEP 17-18	9.7	2.9	DEC 30	63	39	32	50	22	0.5	9.5	46	E	APR 30	76	5.1	135	120	110	65	3	42	8	70	2	123	0	838.2

Table 4.2. Normals, Means, And Extremes

Month	Temperatures °F Normal: Daily maximum	Daily minimum	Monthly	Extremes: Record highest	Year	Record lowest	Year	Normal Degree days Base 65 °F: Heating	Cooling	Precipitation in inches, Water equivalent: Normal	Maximum monthly	Year	Minimum monthly	Year	Maximum in 24 hrs.	Year	Snow, Ice pellets: Maximum monthly	Year	Maximum in 24 hrs.	Year	Relative humidity pct. Hour 05	Hour 11	Hour 17	Hour 23 (Local time)	Wind: Mean speed m.p.h.	Prevailing direction	Fastest mile: Speed m.p.h.	Direction	Year	Pct. of possible sunshine	Mean sky cover, tenths, sunrise to sunset	Mean number of days, Sunrise to sunset: Clear	Partly cloudy	Cloudy	Precipitation .01 inch or more	Snow, Ice pellets 1.0 inch or more	Thunderstorms	Heavy fog, visibility ¼ mile or less	Temperatures °F Max. (b) 90° and above	32° and below	Min. 32° and below	0° and below	Average station pressure mb. Elev. 5314 feet m.s.l.
(a)				43		43					43		43		43		43		43		22	22	22	22	43	15	43	43		43	43	43	43	43	43	43	43	43	22	22	22	22	10
J	46.9	23.5	35.2	69	1971	-17	1971	924	0	0.30	1.32	1978	T	1970	0.87	1962	9.5	1973	5.1	1973	70	50	39	60	8.1	N	61	E	1949	72	4.9	13	8	10	4	1	*	1	0	3	29	1	838.2
F	52.6	27.4	40.0	75	1972	-5	1951	700	0	0.39	1.42	1948	0.04	1959	0.51	1981	8.2	1969	4.2	1944	64	44	32	52	8.9	N	68	NW	1944	73	4.9	11	8	9	4	1	*	1	0	1	23	0	838.3
M	59.2	32.3	45.8	85	1971	8	1948	595	0	0.47	2.18	1973	T	1966	1.11	1973	13.9	1973	10.7	1973	56	35	24	43	10.1	SE	80	NW	1943	73	5.0	11	10	10	4	1	1	1	0	*	17	0	834.8
A	70.1	41.4	55.8	89	1965	19	1980	282	0	0.48	1.97	1942	T	1972	1.66	1969	8.1	1973	6.6	1973	47	25	18	34	11.0	S	72	S	1946	77	4.5	13	9	8	3	*	1	*	0	0	5	0	835.9
M	79.9	50.7	65.3	95	1974	28	1975	58	59	0.53	3.07	1941	T	1945	1.14	1969	1.0	1979	1.0	1979	47	24	18	33	10.5	S	72	W	1950	79	4.1	15	10	6	4	*	4	*	2	0	*	0	835.8
J	89.5	59.7	74.6	105	1980	40	1980	0	285	0.50	1.71	1967	T	1975	1.64	1952	0.0		0.0		44	23	16	31	10.0	S	82	SE	1946	83	3.2	18	8	4	4	0	5	*	18	0	0	0	838.1
J	92.2	65.2	78.7	105	1980	54	1979	0	428	1.39	3.33	1968	0.08	1980	1.77	1961	0.0		0.0		60	34	27	47	9.1	SE	68	E	1945	76	4.9	12	14	5	9	0	11	*	24	0	0	0	840.4
A	89.7	63.4	76.6	101	1979	52	1968	0	344	1.34	3.30	1967	T	1962	1.75	1980	0.0		0.0		65	39	30	52	8.2	SE	61	SE	1951	76	4.7	14	12	5	9	0	11	*	16	0	0	0	840.6
S	83.4	56.7	70.1	100	1979	37	1971	7	132	0.77	1.99	1940	T	1957	1.92	1955	T	1971	T	1971	61	40	31	52	8.6	SE	62	SE	1945	79	3.5	17	8	5	6	0	5	*	4	0	0	0	840.4
O	71.7	44.7	58.2	91	1979	25	1980	218	6	0.79	3.08	1972	0.00	1952	1.80	1969	0.9	1979	0.9	1979	59	37	28	48	8.3	SE	66	W	1959	80	3.4	18	7	6	5	0	2	*	*	0	2	0	840.1
N	57.1	31.8	44.5	77	1975	-7	1976	615	0	0.29	1.45	1940	0.00	1949	0.76	1940	9.3	1940	5.5	1946	65	42	35	54	7.8	N	57	NW	1948	77	4.0	15	8	7	3	*	1	1	0	*	17	*	839.4
D	47.5	24.9	36.2	72	1958	3	1974	893	0	0.52	1.85	1959	0.00	1981	1.35	1958	14.7	1959	14.2	1958	70	50	42	61	7.7	N	90	SE	1943	72	4.5	14	8	9	4	1	*	1	0	2	29	0	839.4
YR	70.0	43.5	56.8	105	JUL 1980	-17	JAN 1971	4292	1254	7.77	3.33	JUL 1968	0.00	DEC 1981	1.92	SEP 1955	14.7	DEC 1959	14.2	DEC 1958	59	37	28	47	9.0	SE	90	SE	DEC 1943	77	4.2	171	110	84	59	4	42	5	65	5	122	1	838.5

Means and extremes above are from existing and comparable exposures. Annual extremes have been exceeded at other sites in the locality as follows: Maximum monthly precipitation 8.15 in June 1852 (measured by Medical Officers of Army at Army Post near plaza).

(a) Length of record, years, through the current year unless otherwise noted, based on January data.
(b) 70° and above at Alaskan stations.
* Less than one half.
T Trace.

NORMALS - Based on record for the 1941-1970 period.
DATE OF AN EXTREME - The most recent in cases of multiple occurrence.
PREVAILING WIND DIRECTION - Record through 1963.
WIND DIRECTION - Numerals indicate tens of degrees clockwise from true north. 00 indicates calm.
FASTEST MILE WIND - Speed is fastest observed 1-minute value when the direction is in tens of degrees.

Table 4.3.

Average Temperature

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Annual
1943	30.6	43.6	48.2	60.9	65.8	74.0	79.0	78.4	70.2	56.8	44.3	32.1	57.7
1944	29.2	40.0	44.4	52.2	63.9	73.3	76.4	76.3	68.6	58.7	43.0	36.7	55.7
1945	36.8	43.0	45.0	52.3	66.2	77.6	78.4	78.2	70.6	58.6	45.2	33.4	56.7
1946	33.5	40.2	47.9	61.3	64.0	77.5	78.3	75.9	71.3	56.3	41.6	40.4	57.4
1947	33.8	42.2	47.4	54.5	67.0	77.8	80.2	76.0	72.6	60.4	40.4	34.6	56.8
1948	33.7	37.6	40.9	59.3	66.7	73.8	79.6	77.6	71.4	57.2	40.6	30.2	56.4
1949	30.3	37.2	47.9	55.5	64.8	73.7	77.7	76.3	71.8	56.2	50.7	34.1	56.4
1950	37.5	44.8	47.9	58.5	65.7	76.1	76.8	76.3	67.9	65.8	48.3	42.3	59.0
1951	36.6	39.8	46.3	54.8	66.5	74.7	82.6	76.6	71.9	58.8	42.2	34.2	57.2
1952	39.0	39.4	42.3	55.9	66.4	78.1	78.2	79.9	70.7	61.0	40.1	33.7	57.0
1953	42.2	38.7	48.8	55.1	61.5	79.1	80.0	77.7	73.0	59.2	46.3	32.8	57.8
1954	38.4	46.8	46.0	62.8	67.0	76.4	81.3	76.6	72.8	61.4	48.7	36.1	59.5
1955	33.8	34.8	47.4	55.1	63.9	73.9	77.0	76.9	72.5	61.1	45.1	40.0	56.8
1956	41.1	36.0	49.2	55.3	68.6	79.9	78.6	76.3	74.1	60.7	41.0	36.3	58.1
1957	39.9	48.1	47.3	54.2	61.9	74.8	79.1	76.0	70.3	56.4	40.3	38.5	57.3
#1958	35.3	43.5	42.8	53.0	68.6	76.5	79.6	78.9	69.2	56.9	46.0	41.5	57.8
1959	34.3	39.5	45.9	57.0	65.1	77.0	78.6	76.2	70.6	56.8	43.9	38.2	56.9
#1960	33.0	36.5	49.9	57.9	64.7	76.6	78.7	78.4	71.8	56.5	46.5	33.9	57.1
1961	33.9	40.6	47.0	54.5	65.9	75.8	76.7	75.2	65.6	56.8	40.3	34.1	54.5
1962	31.6	42.3	41.2	58.1	64.1	72.7	76.3	77.6	69.4	58.1	46.9	36.9	56.3
1963	29.4	40.5	45.2	57.7	68.0	74.6	81.4	75.9	72.5	61.5	45.7	34.8	57.3
1964	30.0	29.1	41.5	51.7	65.8	73.6	78.2	76.8	69.3	59.4	43.7	35.5	54.5
1965	38.8	39.4	44.6	54.8	61.7	69.4	77.9	75.4	66.6	58.0	48.4	35.8	55.9
1966	30.1	33.2	45.6	54.6	67.2	72.8	79.8	75.7	68.4	56.8	46.7	34.3	55.4
1967	33.2	40.5	52.0	57.8	63.8	71.5	79.2	74.5	68.4	58.2	46.1	32.4	56.5
1968	36.8	43.3	46.7	53.4	62.7	75.2	76.1	72.4	68.0	58.7	42.8	30.0	55.5
1969	38.0	38.5	41.1	57.4	66.2	73.6	83.7	79.0	70.0	53.8	41.4	39.1	56.6
1970	34.5	42.8	44.1	52.5	66.2	72.7	79.6	77.8	67.5	52.6	44.5	36.4	56.0
1971	33.6	38.9	47.7	53.3	61.7	73.8	78.1	73.9	66.4	53.8	45.2	31.9	54.8
1972	36.1	42.5	53.6	56.9	64.0	77.7	78.6	74.1	68.1	57.6	40.1	35.0	56.7
1973	31.8	35.9	45.1	50.2	62.7	73.5	78.4	78.0	67.5	56.4	44.6	34.0	54.8
1974	33.6	37.9	52.8	56.4	68.5	80.1	77.0	72.7	66.1	58.1	45.0	32.0	56.7
1975	30.8	38.0	45.0	49.9	61.0	73.0	76.8	76.1	66.3	56.5	42.6	35.6	54.3
1976	33.2	43.3	44.3	54.6	62.8	77.4	77.0	75.0	68.0	53.1	40.6	33.0	54.9
1977	29.8	40.7	43.2	56.5	64.2	75.5	78.6	77.4	69.4	58.9	46.4	40.4	56.8
1978	36.8	39.3	50.2	57.7	60.5	75.5	81.6	75.5	69.1	60.3	47.5	34.3	57.6
1979	32.9	41.1	48.4	56.9	63.7	73.3	80.6	77.1	72.3	61.5	41.0	37.7	57.2
1980	40.2	44.2	46.1	52.1	61.1	77.2	82.7	77.4	69.9	54.5	43.5	40.5	57.4
1981	38.0	42.9	46.2	59.0	64.5	77.0	79.8	76.4	69.7	55.7	47.0	40.5	58.0
1982	35.9	39.4	47.4	56.1	63.0	74.8	79.1	77.4	69.5	54.8	42.9	34.4	56.2
RECORD MEAN	34.6	39.7	46.4	54.9	63.8	73.5	77.4	75.3	68.5	56.8	43.9	35.3	55.8
MAX	47.1	53.0	60.8	70.0	79.0	89.1	91.2	88.8	82.3	71.1	57.5	47.6	69.8
MIN	22.0	26.3	31.9	39.7	48.5	57.9	63.5	61.7	54.6	42.4	30.2	22.9	41.8

Table 4.4

Heating Degree Days

ALBUQUERQUE, NM

Season	July	Aug	Sept	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	Total
1962-63	0	0	22	254	534	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	2	[illegible]
1963-64	0	0	0	124	573	931	[illegible]	[illegible]	722	[illegible]	[illegible]	3	[illegible]
1964-65	2	0	20	173	632	909	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1965-66	0	0	56	217	492	895	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1966-67	0	0	15	267	561	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1967-68	0	0	13	220	557	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1968-69	2	0	14	259	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1969-70	0	0	1	344	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1970-71	0	0	56	387	635	876	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1971-72	0	0	171	361	597	1022	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1972-73	0	3	34	244	741	921	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1973-74	0	0	43	257	606	955	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1974-75	0	2	68	217	593	1022	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1975-76	0	0	67	254	664	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1976-77	0	0	35	367	726	981	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1977-78	0	0	1	197	551	757	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1978-79	0	0	22	167	521	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1979-80	0	0	29	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1980-81	0	0	6	331	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1981-82	0	0	7	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1982-83	0	0	23	314	656	941							

Table 4.5

Cooling Degree Days

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Total
1969	0	0	0	[illegible]	127	267	478	[illegible]	156	[illegible]	[illegible]	0	[illegible]
1970	0	0	0	0	115	244	461	404	[illegible]	[illegible]	[illegible]	0	[illegible]
1971	0	0	5	0	26	277	414	282	145	[illegible]	0	0	1151
1972	0	0	0	5	52	267	428	394	[illegible]	[illegible]	0	0	[illegible]
1973	0	0	0	0	46	267	422	[illegible]	124	[illegible]	0	0	[illegible]
1974	0	0	0	[illegible]	144	464	380	[illegible]	107	[illegible]	0	0	[illegible]
1975	0	0	0	0	25	254	374	351	96	[illegible]	0	0	[illegible]
1976	0	0	0	0	38	267	382	319	137	5	0	0	[illegible]
1977	0	0	0	0	44	324	427	392	141	[illegible]	0	0	[illegible]
1978	0	0	0	[illegible]	41	324	521	[illegible]	151	27	0	0	1398
1979	0	0	0	[illegible]	67	264	491	382	249	[illegible]	0	0	[illegible]
1980	0	0	0	[illegible]	27	375	557	392	142	[illegible]	0	0	[illegible]
1981	0	0	0	28	51	367	470	[illegible]	152	1	0	0	[illegible]
1982	0	0	0	6	38	301	441	394	163	[illegible]	0	0	[illegible]

Table 4.6.

Precipitation

Year	Jan	Feb	Mar	Apr	May	June	July	Aug	Sept	Oct	Nov	Dec	Annual
1943	0.25	0.26	0.23	0.06	1.41	1.20	1.19	1.33	0.39	0.22	0.14	0.94	7.62
1944	0.46	0.42	0.49	0.91	0.57	0.85	1.58	1.44	0.65	0.86	0.56	0.76	9.55
1945	0.34	0.32	0.50	0.77	T	T	1.09	2.27	0.26	0.43	T	0.35	6.33
1946	0.25	0.33	1.03	0.26	0.31	0.07	2.28	1.49	0.57	1.07	0.54	0.12	8.27
1947	0.02	0.14	0.03	0.03	0.48	0.23	0.38	1.45	0.67	0.31	0.45	1.05	5.24
1948	0.18	1.42	0.41	0.33	0.94	0.57	0.46	0.51	0.80	0.60	0.11	0.11	6.44
1949	0.61	0.29	0.65	0.67	1.35	0.32	2.21	0.72	0.87	0.14	0.00	0.59	8.42
1950	0.02	0.38	0.04	0.27	0.06	0.23	2.00	0.08	1.01	0.01	T	0.00	4.10
1951	0.41	0.27	0.29	0.38	0.10	0.02	0.85	2.22	0.05	0.37	0.14	0.28	5.38
1952	0.20	0.17	0.59	0.76	0.65	1.64	1.91	1.10	0.34	0.00	0.53	0.20	8.09
1953	T	0.43	0.74	0.69	0.03	0.35	0.53	0.59	0.06	0.46	0.91	0.29	5.08
1954	0.20	0.07	0.24	T	0.51	0.01	1.45	0.65	0.77	0.25	0.22	0.14	4.51
1955	0.29	0.18	T	0.04	0.53	0.33	1.60	1.32	1.94	0.06	T	0.22	6.51
1956	0.46	0.49	T	T	0.18	0.43	1.49	0.62	0.02	0.34	0.03	0.00	4.06
1957	0.78	0.59	0.52	0.38	0.35	0.04	2.48	1.32	T	2.59	1.24	0.32	10.61
#1958	0.21	0.27	1.71	0.62	0.43	0.22	0.14	1.74	1.34	1.72	0.37	1.35	10.12
1959	0.17	0.04	0.42	0.43	0.80	0.78	0.77	2.79	0.36	1.70	0.07	1.85	10.14
1960	0.34	0.38	0.44	0.19	0.71	0.91	0.47	0.78	0.56	2.88	0.07	0.39	8.12
1961	0.23	0.18	0.61	0.73	0.01	0.11	2.70	1.69	1.09	0.47	0.48	0.65	8.87
1962	1.01	0.11	0.18	0.07	0.01	0.19	1.24	T	0.71	0.75	0.61	0.51	5.39
1963	0.29	0.24	0.55	0.14	0.03	0.11	1.47	3.00	0.63	0.76	0.29	T	7.47
1964	0.07	1.12	0.13	0.61	0.35	T	1.87	0.98	1.57	0.04	0.21	0.49	7.44
1965	0.47	0.60	0.49	0.49	0.19	0.99	1.65	0.61	1.18	0.89	0.33	1.42	9.31
1966	0.42	0.30	T	0.04	0.02	1.66	1.63	1.06	1.04	0.59	0.09	0.01	6.81
1967	0.01	0.44	0.25	T	0.04	1.71	0.61	3.30	0.79	0.18	0.15	0.56	8.04
1968	0.01	0.98	1.48	0.51	0.99	0.05	3.33	1.49	0.30	0.12	0.59	0.82	10.67
1969	0.08	0.34	0.41	1.76	1.31	0.59	0.94	0.95	1.08	2.37	0.01	0.72	10.56
1970	T	0.27	0.42	0.05	0.33	0.40	1.22	2.24	0.79	0.25	0.08	0.23	6.28
1971	0.27	0.21	0.03	0.78	0.16	0.02	1.05	0.87	1.44	1.15	0.67	1.40	8.05
1972	0.12	0.12	0.08	T	0.18	0.55	1.00	2.93	1.00	3.08	0.69	0.36	10.11
1973	0.85	0.33	2.18	0.91	0.66	1.37	1.80	1.19	1.13	0.35	0.08	0.03	10.88
1974	0.88	0.11	0.85	0.14	0.01	0.22	2.40	0.79	1.58	1.96	0.38	0.51	9.83
1975	0.26	0.99	0.95	0.10	0.66	T	1.43	1.40	1.66	T	0.28	0.28	8.01
1976	0.00	0.40	0.09	0.31	0.82	0.60	1.32	0.73	0.45	0.03	0.24	0.20	5.19
1977	0.88	0.13	0.63	1.07	0.10	0.04	0.69	2.28	0.78	0.76	0.42	0.13	7.91
1978	1.32	1.02	0.54	0.05	0.69	1.05	0.24	2.49	0.59	1.22	1.00	0.76	10.97
1979	1.07	0.62	0.14	0.24	2.48	1.02	0.80	1.53	0.40	0.27	0.91	0.87	10.35
1980	0.87	0.58	0.60	0.60	0.56	0.01	0.08	2.61	1.03	0.09	0.30	0.74	8.87
1981	0.05	0.67	0.80	0.30	0.53	0.35	1.07	1.68	0.41	1.43	0.37	0.00	7.66
1982	0.32	0.20	0.84	0.05	0.52	0.09	1.32	1.09	1.34	0.26	0.60	0.78	7.41
RECORD MEAN	0.39	0.37	0.45	0.52	0.63	0.58	1.40	1.36	0.92	0.81	0.43	0.45	9.32

Table 4.7.

Snowfall

Season	July	Aug	Sept	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	June	Total
1943-44	0.0	0.0	0.0	0.0	T	4.0	4.9	T	1.0	3.0	0.0	0.0	12.9
1944-45	0.0	0.0	0.0	0.0	T	1.0	1.0	[illegible]	T	[illegible]	0.0	0.0	[illegible]
1945-46	0.0	0.0	T	0.0	0.0	2.4	3.8	4.2	6.0	0.0	0.0	0.0	[illegible]
1946-47	0.0	0.0	0.0	0.0	6.5	T	0.3	T	T	T	0.0	0.0	[illegible]
1947-48	0.0	0.0	0.0	0.0	5.7	5.1	3.1	[illegible]	1.0	0.0	0.0	0.0	[illegible]
1948-49	0.0	0.0	0.0	0.0	T	2.2	2.8	2.4	3.0	[illegible]	0.0	0.0	[illegible]
1949-50	0.0	0.0	0.0	T	0.0	5.0	T	T	T	T	0.0	0.0	5.0
1950-51	0.0	0.0	0.0	0.0	T	0.0	6.0	1.0	T	T	T	0.0	[illegible]
1951-52	0.0	0.0	0.0	0.0	T	3.0	T	T	2.0	T	0.0	0.0	[illegible]
1952-53	0.0	0.0	0.0	0.0	2.7	0.9	T	3.8	T	T	0.0	0.0	[illegible]
1953-54	0.0	0.0	0.0	0.0	7.5	3.7	2.9	T	T	0.0	0.0	0.0	[illegible]
1954-55	0.0	0.0	0.0	0.0	0.0	1.4	3.0	1.2	T	0.0	0.0	0.0	[illegible]
1955-56	0.0	0.0	0.0	0.0	T	1.0	2.9	[illegible]	0.0	T	0.0	0.0	[illegible]
1956-57	0.0	0.0	0.0	0.0	T	T	1.5	T	T	T	0.0	0.0	[illegible]
1957-58	0.0	0.0	0.0	T	T	2.0	2.3	[illegible]	7.3	T	0.0	0.0	[illegible]
1958-59	0.0	0.0	0.0	T	2.2	14.2	1.7	[illegible]	4.8	T	0.0	0.0	[illegible]
1959-60	0.0	0.0	0.0	T	0.0	14.7	1.8	[illegible]	2.8	0.0	0.0	0.0	[illegible]
1960-61	0.0	0.0	0.0	0.0	0.0	2.5	0.6	[illegible]	3.0	T	0.0	0.0	[illegible]
1961-62	0.0	0.0	0.0	T	3.4	2.4	4.0	T	0.2	0.0	0.0	0.0	[illegible]
1962-63	0.0	0.0	0.0	0.0	T	1.0	2.5	[illegible]	2.5	0.0	0.0	0.0	[illegible]
1963-64	0.0	0.0	0.0	0.0	T	T	0.5	[illegible]	1.3	T	0.0	0.0	[illegible]
1964-65	0.0	0.0	0.0	[illegible]	T	[illegible]	1.4	[illegible]	T	T	0.0	0.0	[illegible]
1965-66	0.0	0.0	0.0	0.0	T	3.0	5.4	1.0	0.0	T	0.0	0.0	[illegible]
1966-67	0.0	0.0	0.0	0.0	T	T	T	1.0	1.1	T	0.0	0.0	[illegible]
1967-68	0.0	0.0	0.0	[illegible]	1.0	2.4	T	1.4	1.4	T	0.0	0.0	[illegible]
1968-69	0.0	0.0	0.0	0.0	T	7.4	T	1.5	5.5	T	0.0	0.0	[illegible]
1969-70	0.0	0.0	0.0	0.0	T	1.1	T	[illegible]	3.3	0.0	0.0	0.0	[illegible]
1970-71	0.0	0.0	0.0	0.0	T	T	3.0	[illegible]	0.5	T	0.0	0.0	[illegible]
1971-72	0.0	0.0	T	T	T	4.4	1.2	1.1	0.0	T	0.0	0.0	[illegible]
1972-73	0.0	0.0	0.0	T	2.9	1.2	9.5	1.8	13.9	[illegible]	0.0	0.0	[illegible]
1973-74	0.0	0.0	0.0	T	0.6	1.1	9.3	[illegible]	2.0	T	0.0	0.0	[illegible]
1974-75	0.0	0.0	0.0	0.0	T	[illegible]	0.9	[illegible]	3.8	[illegible]	0.0	0.0	[illegible]
1975-76	0.0	0.0	0.0	0.0	0.2	2.9	0.0	T	0.5	T	0.0	0.0	[illegible]
1976-77	0.0	0.0	0.0	T	2.4	1.2	8.4	[illegible]	2.3	[illegible]	0.0	0.0	[illegible]
1977-78	0.0	0.0	0.0	0.0	0.0	T	6.0	[illegible]	2.0	[illegible]	0.0	0.0	[illegible]
1978-79	0.0	0.0	0.0	0.0	T	1.0	2.6	[illegible]	T	[illegible]	0.0	0.0	[illegible]
1979-80	0.0	0.0	0.0	0.0	0.8	2.7	T	[illegible]	3.1	T	0.0	0.0	[illegible]
1980-81	0.0	0.0	0.0	0.0	2.8	7.4	0.5	[illegible]	0.9	T	0.0	0.0	[illegible]
1981-82	0.0	0.0	0.0	0.0	0.0	[illegible]	3.4	1.2	0.7	T	0.0	0.0	[illegible]
1982-83	0.0	0.0	0.0	0.0	0.9	3.7							
RECORD MEAN	0.0	0.0	T	T	1.1	2.7	2.4	1.9	2.2	[illegible]	T	0.0	[illegible]

# Indicates a station move or relocation of instruments. See Station Location table.

Record mean values above are means through the current year for the period beginning in 1893 for temperature and precipitation, 1940 for snowfall. Precipitation data are from Univ. of New Mexico location for 1/93-12/05, 7/06-9/06, 5/07-4/08, 4/08-8/08, 11/08, 1/09-2/09, 4/09-4/10, 9/18-3/31; 1216 W Central Avenue location for 1/06-6/06, 10/06-4/07, 8/07, 3/08, 9/08-10/08, 12/08, 3/09, 5/10-4/15, 7/15-9/15, 1/16, and Rio Grande Ind. School location for 5/15-6/15, 10/15-12/15 and 2/16-7/16. Temperature data are from Univ. of New Mexico location for 1/93-5/10, Rio Grande Ind. School location for 5-15-12/18, and Univ. of New Mexico location for 1/19-3/31.

Table 4.8

STATION LOCATION

ALBUQUERQUE, NEW MEXICO

Location	Occupied from	Occupied to	Airline distance and direction from previous location	Latitude North	Longitude West	Elevation above: Sea level: Ground at temperature site	Ground: Wind instruments	Extreme thermometers	Psychrometer	Sunshine Switch	Tipping bucket rain gage	Weighing rain gage	8" rain gage	Hygrothermometer	Automatic Observing Equipment *	Remarks
COOPERATIVE																
4th and W. Gold Avenue	1/1892	12/1892	NA	35° 05'	106° 39'	Unk							3			
University of New Mex.	1/1893	5/1910	1.5 mi. E	35° 05'	106° 37'	5150		Unk					39			
1216 W. Central Avenue	1/1906	1/1916	2 mi. W	35° 05'	106° 40'	4960							3			Precipitation only. Record intermittent
Rio Grande Industrial School	5/1915	12 31 18	5 mi. S	35° 01'	106 40'	4950		4					3			Temperature only after July 1916
5th and W. Central Ave.	8/1916	8/1918	5 mi. N	35 05'	106° 39'	4960							3			
University of New Mex.	9/1918	3/1931	1.5 mi. E	35° 05'	106° 37'	5150	59	48	48				39			
CITY																
Kimo Theatre Building 419 W. Central Avenue	4/1/31	1/23 33	1.5 mi. W	35° 05'	106° 39'	4960	66	52	51		45		45			Office moved 1000 feet SW to Federal Building 6 29 32, but instruments not moved
AIRPORT																
TWA Airport West of City	1 23 33	7.31.39	3.8 mi. W	35 05'	106° 43'	5100	39	6	5		15		15			
Administration Building Municipal Airport	7 31/39	6 23 58	6 mi.ESE	35° 03'	106 37'	5310	48	6	5	Unk	3	5	3			
Administration Building Municipal Airport	6 23 58	2/4/60	A			5310	48	16	15	Unk a31	13	15	13			A - Instrument relocation to roof 33 feet SSE of ground site.
Administration Building Municipal Airport	2/4/60	3.16 65	B	35° 03'	106° 37'	5311	48 b23	17	17	31	13	15	13	5		B - Instrument relocations and commissioning of hygrothermometer. a - Effective 9/16/59. b - Effective 3/1/60.
FAA/Weather Bureau Building † Albuquerque Sunport-Kirtland AFB ††	3 16 65	Present	350 ft. SW	35° 03'	106° 37'	5311	c23	16	16	16 d26	17	17	c13 e17	c5	NA	c - Not moved 3/16/65. d - Effective 1/22/66. e - Effective 4/16/66.
† FAA/Weather Service Building (Eff. 1971)																
†† Albuquerque Int'l AP (Effective 1981)																

* Type
M = AMOS
T = AUTOB

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L. Kay Hoxit
Acting Director
National Climatic Data Center

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tions range from 4850 to 6000 feet. The mean annual precipitation is 7 to 10 inches, the mean annual temperature is 58° to 60°F, and the frost free season is 165 to 195 days. Bluepoint soils are associated with Kokan, Latene, Madurez, and Wink soils.

In a representative profile, the surface layer of Bluepoint soils is pale brown loamy fine sand about 8 inches thick. The underlying material to a depth of 60 inches or more is pale brown and light yellowish brown loamy sand.

USDA (1977) uses the term loam extensively to describe soils that contain 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. However, the term also indicates that the soil is favorable for the growth of many varieties of plants or crops carrying with it the implication at least of a significant organic content. To the engineer, organic matter is usually an objectionable soil component. Thus the engineer would not classify a clean mixture of sand and silt as a loam for fear that there might be an unwarranted implication of organic content. In view of this situation, the term loam is rarely used in foundation engineering (Hough, 1957). Nonetheless, to be consistant with USDA description, the term loam will be used in the Soil Profile section of this text. The soil is slightly calcareous and mildly to moderately alkaline.

Permeability of Bluepoint soils is rapid. Available water capacity is 4 to 5.5 inches. Effective rooting depth is 60 inches or more. Uses are for range, irrigated crops, watershed, wildlife habitat, and community development.

See Table 4.9 for an indication of a representative profile of Bluepoint loamy fine sand with 1 to 9 percent slopes.

Table 4.9 - Representative Profile of Bluepoint Series (USDA, 1977)

Horizon	Depth (inches)	Description
A1	0-8	Pale brown loamy fine sand, brown moist, single grained; loose; many fine and very fine roots and intersticial pores; slightly calcerous; moderately alkaline; clear wavy boundary.
C1	8-20	Light yellowish brown loamy sand, brown moist; massive slightly hard, very friable; few very fine and fine roots; many very fine interstitial pores; slightly calcerous; moderately alkaline; clear wavy boundary.
C2	20-60	Light yellowish brown loamy sand, dark yellowish brown moist; massive, slightly hard, very friable; few fine and very fine roots,; many very fine interstitial pores; slightly calcerous in spots; mildly alkaline.

The Wink series consists of deep, well drained soils that formed in old unconsolidated alluvium modified by wind on piedmonts. Slopes are 0 to 1 percent. The native vegetation is principally blue grama, broom snakeweed, and sand dropseed. Elevations range from 5,000 to 6,000 feet. The mean annual precipitation is 7 to 10 inches, the mean annual air temperature is 58° to 60° F, the frost-free season is 170 to 195 days. Wink soils are associated with Madurez, Latene, Bluepoint, and Embudo soils. In a representative profile, the surface layer is brown fine sandy loam and sandy loam about 11 inches thick. The subsoil is light brown sandy loam about 16 inches thick. The substratum to a depth of 60 inches or more is pinkish gray and pinkish white sandy loam. The soil is calcareous and moderately alkaline.

Permeability of Wink soils is moderately rapid. Available water capacity is 5.5 to 8 inches. Effective rooting depth is 60 inches or more. Wink soils are good for range, watershed, wildlife habitat, and community development.

Further classification of the TRESTLE's surrounding soil is Wink fine sandy loam, 0 to 5 percent slopes (WaB) and Bluepoint-Kokan association, hilly (BKD). The latter is about 50 percent a Bluepoint loamy fine sand that has 5 to 15 percent slopes and 40 percent a Kokan gravelly sand that has 15 to 40 percent slopes. The gently rolling to rolling Bluepoint soil is on fans between gravelly ridges of the hilly to steep Kokan soil. The Kokan soil has the profile described as representative of the Kokan series (USDA, 1977). On about 10 percent of the acreage, however, it has a high lime layer in the substratum. On both soils (Bluepoint and Kokan), runoff is slow and the hazard of water erosion is moderate to severe. BKD is used for range, watershed, wildlife habitat, recreation, and community development. It is also a major source of sand and gravel.

WaB soil is on the East and West Mesas. The soil has a profile described as representative of the Wink series (USDA, 1977). Included with this soil in mapping are areas of Wink soil that has a thin surface layer of loamy sand (approximately 10 percent of the coverage). Runoff is medium and the hazard of water erosion is slight to moderate. Also, the hazard of wind erosion is moderate. WaB is used for range, watershed, wildlife habitat, and community development.

The engineering classification and estimated properties of the soils just discussed are summed up in Table 4.10.

Table 4.10 - Engineering Classification and Estimated Properties (USDA, 1977) Bernalillio County, New Mexico; KAFB TRESTLE Area

Soil Series	USDA Texture	Classification		Percent Passing						Permeability (inches/hr)	Available water capacity (inches/inch of soil	Shrink Swell Potential
		Unified	AASHTO	No. 4	No.10	No.40	No.200	LL	PI			
Blue-Point (BKD)	loamy fine sand and loamy sand	SP-SM or SM	A-3 or A-2	90-100	85-100	70-95	5-20	NP	NP	6.0-20.0	0.07-0.09	Low
Kokan (for Kokan part of BKD)	gravelly sand - very gravelly sand	GP GP-GM, or GM	A-1	45-60	40-55	20-45	0-20	NP	NP	Greater than 20.0	0.03-0.05	Low
Wink (WaB)	sandy loam	SM	A-2 or A-4	-	100	80-95	30-40	NP	NP	2.0-6.0	0.09-0.13	Low

(USDA, 1977) The soils around Albuquerque formed mainly in recent alluvium, old unconsolidated alluvium, alluvium modified by wind, alluvial fan and piedmont sediments, or material weathered from basalt, granite, schist, limestone, sandstone and shale. The influence of parent material in most soils is apparent in their texture, mineralogy, structure, reaction, and color.

Recent alluvium is deposited on the flood plain of the Rio Grande when the river overflows its channel and suddenly loses transporting power. The heavier sand is deposited first, then silt, and finally clay. The Rio Grande has changed its course many times, and the pattern of sediments, and therefore of soils, is complex. Although levees have protected the flood plain from major flooding since 1927, the irrigated cropland continues to receive annual small quantities of sediment from silty irrigation water diverted from the river.

Old unconsolidated alluvium, mostly from the ancestral Rio Grande and its tributaries, is the main parent material in the area. Madurez and Wink soils formed in sandy and loamy alluvium; Kokan soils formed in sandy and gravelly alluvium.

Sandy alluvium is often reworked or moved by the wind. Bluepoint soils formed in reworked sandy alluvium deposited on the sides of the Rio Grande and Rio Puerco Valleys and on piedmonts and mesas. Gravelly alluvial fans and piedmont sediments occur along the front of the

Manzano and Sandia Mountains.

The Manzano and Sandia Mountains are made up of folded igneous, metamorphic, and sedimentary rocks. Salas soils formed in residuum weathered from schist mixed with some gneiss and quartzite. Laporte and Escabosa soils found in material weathered from limestone.

Most of Albuquerque's soils contain several clay minerals, including montmorillonite, vermiculite, illite, kaolinite, and chlorite. All of the soils have varying amounts of carbonates received as part of the dust deposited by the wind.

Field Soil Tests

Field investigation of the soil at the TRESTLE was carried out in accordance with procedures outlined by Professor R. L. Sloane, University of Arizona, Reference 4.3. Field test results are summarized in Table 4.11.

Based on field results, the TRESTLE soil can be described as ROUNDED LIGHT BROWN SILTY SAND WITH SOME SCATTERED GRAVEL AND ORGANIC MATERIAL.

In addition to the tests performed in Table 4.11, two holes were dug to a depth of four feet at the base of the TRESTLE bowl; one in the southwest sector and the other in the southeast sector (See Drawing No. 5.1). Below 4-6 inches, the soil is well compacted silty sand, fairly moist throughout.

Table 4.11 Field Test Results; KAFB TRESTLE Sample, March, 1984

Type of Test	Indicated Results
Visual inspection	Rounded, light brown sand, inorganic
Angle of Repose (Holtz and Kovacs, 1981)	35°
Crushing	Silt
Dusting	Silt
Shaking	Silt
Molding	Non-plastic, silt
Modified settling	Silty Sand

Ref. 4.3 FIELD IDENTIFICATION OF SOILS

Professor R. L. Sloane
University of Arizona

Introduction

Accurate field identification of soil types and accurate, complete description of soils encountered in the field are a necessary and very helpful part of any soils investigation. This information forms the basis for preliminary screening of field samples for laboratory testing, reduces the amount of classification testing needed to prepare final boring or test pit logs, and aids in the extrapolation of boring profiles to soil profiles over the investigated area. Incomplete or erroneous field identification always multiplies the amount of laboratory work required and may lead to erroneous interpretation of the soil profile at the site.

Systems of Field Classification and Soil Description

Almost every organization engaged in soil site investigation has its own system of soil type description or soil nomenclature; however, all have a great deal in common and most base the nomenclature on the same arbitrary grain-size definitions of soil components. It would require a book to cover all the systems in use; therefore, the following discussion is intended to show principles of soil identification and description rather than details of any one system.

The most widely used grain-size definition of soil components is the following:

Boulders - larger than 6 inches
Cobbles - 2 inches to 6 inches
Gravel - 4.76 millimeters to 2 inches
Sand - 0.074 millimeters to 4.76 millimeters
Silt - 0.005 millimeters (5 microns) to 0.074 millimeters
Clay - smaller than 5 microns

In general, the use of the term "rock" should be avoided in soil description because it is ambiguous. It is not clear whether this term means particles larger than gravel-size, or ledge rock, or bedrock. If "rock" is used in description it should always indicate specifically what kind of rock is meant; e.g. bedrock, ledge rock, rock fragments, etc. It should also be noted that the above definitions apply only to the minimum particle dimensions and do not imply anything about the mineralogical composition, geological origin, or genesis of the soil. As an example, particles of clay-size may be true clay minerals, other layer silicates, like mica, or rock flour having almost any mineral composition. Therefore, in a strict sense, individual particles or groups of particles in the fine-size range should always be referred to as silt-size or clay-size and not as silt or clay. In practice, however, written soil descriptions use these terms and a table of grain-size definitions, such as the foregoing, to indicate the fact that the nomenclature is based only on grain size.

Some of the older classifications, having derived from the U.S. Department of Agriculture Bureau of Soil classification, make wide use of the term "loam." In more modern practice this term is not used; primarily because it carries the connotation to almost everyone that the soil contains humus or other organic matter. A trend away from three-term soil descriptions, such as "sandy silty clay" or "sandy silt with some clay," has also become noticeable during the past few years because this kind of description is inherently somewhat ambiguous. Almost all natural soils contain sand-size, silt-size, and clay-size particles in some proportions; however, usually only two are in sufficient quantity to be dominant in determining soil properties. Therefore, the practice is becoming more widespread to use a two-term soil description, such as "sandy clay" or "clayey silt,"

Ref. 4.3 Cont'd

in which it is implicit that the third component is present but in such small quantity that the effect on soil properties is very small or even negligible. The basic two-term description is in use in this laboratory.

For reasons which will become apparent to the student as he becomes more familiar with soil engineering, complete soil descriptions normally consist of three parts: 1) the color of the soil, 2) the basic soil type, and 3) necessary modifying terms. Taking these in order:

Color. The simplest and least ambiguous descriptive terms are those of the primary colors and some binary colors (such as purple and green) modified by the adjectives "light" or "dark." Medium shades require no modifying adjective. Terms like "mauve," "beige," "orchid," "tan," and the like should be avoided--they would seem to be more fitted to interior decoration than to engineering.

Basic Soil Type. In two-term descriptions the noun is always the soil component which is dominant and the adjective is always the component which is next in quantity. Thus, there are clayey sands, sandy clays, silty sands, clayey silts, etc.

Modifying Terms. Modifying terms fall into two classes: 1) modifying clauses which follow the basic soil type, and 2) adjective modifiers which precede the color and basic soil type. Examples of the first class are--silty clay with some fine gravel, sandy gravel with scattered cobbles. Examples of the second class are--angular coarse brown sand, varved blue silty clay.

Some examples of correct and complete soil descriptions are the following: rounded light brown medium sand with some fine gravel; black organic muck containing some fibrous material; varved blue silty clay with some fine sand partings; and subangular gray sandy gravel with some cobbles and scattered boulders.

Simple Field Identification Tests

The following simple methods will be found most useful to determining the proper descriptive terms:

1. Visual examination of grain size and grain shape (coarse-grained soils only).
2. Estimation of plasticity by molding with fingers. "Molding Test."
3. Cohesion in the dry state. "Crushing Test" and "Dusting Test."
4. Change in consistency and strength during remolding with fingers. "Molding Test."
5. Pore water mobility and dilatancy observed by shaking in the hand, then crushing. "Shaking Test."
6. Rate of sedimentation and character of sediment and suspension in a suspension of a small amount of soil in water in a test tube. "Settling Test."

Most of these simple tests and examinations are in the nature of indicator tests. In many cases, no single test or examination will be completely diagnostic but several will usually identify the basic soil type. There are always some "borderline" soils which will be difficult to identify; in these cases the classification which is the more coarse-grained will usually be right. The coarse-grained soils are rarely identified incorrectly because all of the grains can be discerned with the unaided eye. The lower limit in grain size for sand (.074 mm) is about the limit of resolution for the average person. Below this size, individual grains cannot be distinguished, hence other means than visual examination must be used. The application of the simple field tests and examinations to identification of soils is described in the following section.

Ref. 4.3 Cont'd

Identification of Soils

1. Coarse-grained soils. Identification of all coarse-grained soils in the wet or dry state is accomplished by visual inspection of grain size, grain shape, and uniformity of grain size.

2. Fine-grained Soils. In the identification of fine-grained soils, the simple tests are designed to help distinguish between silt and clay or, in the case of mixed soils, to determine the relative percentages of silt, clay, and sand.

As a starting point in the identification procedure, it is usually helpful to determine first whether the soil is organic or inorganic. Organic content is usually indicated by dark gray to black color (sometimes brownish-black), sometimes showing banding, and often by a faint to strong odor of decay. When molded (molding test), organic silts and clays have a noticeably softer feel to them than corresponding inorganic materials. Organic content increases the plasticity and sometimes the stickiness (molding test) but decreases the dry strength (crushing test).

The next step is to determine whether the soil is predominantly silty or clayey. In this determination the most reliable indicators are the crushing and dusting tests although the others are useful in confirming the diagnosis. In the crushing test, a small piece of the soil (undisturbed state preferred) is air-dried and then crushed between the fingers. Silts will crumble and powder under light to moderate pressure whereas clays will fracture or break but not crumble or powder and considerable pressure is required. Feebly-plastic (lean) clays will break under moderate to strong pressure; highly-plastic (fat) clays require strong pressure and may often be strong enough to resist fracture entirely.

The dusting test consists in making a thin smear of the wet soil on the heel of the hand, allowing it to dry, and attempting to brush off the dried material with the other hand. If the soil is mostly silt, it will dust off, leaving the hand reasonably clean; if mostly clay, it will resist dusting off and scales rather than dusts as it comes off the hand. Usually, clays strongly resist removal in this manner.

The shaking test will readily and immediately identify the coarser silts and rock flours. In this test, a small amount of the soil is thoroughly mixed with water to make a fairly stiff saturated slurry, the mixture shaken rapidly in the hand from side to side, and observation made as to whether or not pore water is brought to the surface (pore water mobility). Silts will become quite shiny, clays will show little to no change in appearance. The wet soil is then pinched between the fingers and both feel and water intake (dull appearance) observed (dilatancy). If the soil shows a noticeable increase in strength and the shiny appearance disappears, it is a silt.

The molding test can be used to aid in distinguishing between silts and clays by rolling it out into threads and observing whether the thread has wet strength or not. If silt, the thread will break into pieces when it is picked up by one end; if clay, the thread will remain intact when so picked up and, in addition, will show some tensile strength when pulled apart. The relative amount of tensile strength will determine whether the clay is lean or fat.

In soils where silt and clay (also very fine sand) may be present in about equal amounts, the settling test is most useful for identification. A small amount of the dry powdered soil is shaken with distilled water (to which a

Ref. 4.3 Cont'd

dispersant has been added to prevent flocculation), for about a minute in a 4-inch test tube and set aside in a rack to sediment out. All sand-size particles will settle out in 30 seconds or less. At the end of about 20-25 minutes all silt-size particles will have settled out. By the end of 2 hours all but the finest colloidal clay will be settled out. The sediments at the bottom of the tube will be observed to have reasonably well- defined layers of sand, silt, and clay (in that order from the bottom). The relative percentages of sand, silt, and clay can be estimated from the thicknesses of the layers. (Note: because the silt will be unconsolidated and almost liquid, the height of the silt layer should be divided by 2 for this comparison).

Miscellaneous Special Soil Types

Certain soil types have sufficiently distinctive characteristics that they may be immediately identified by their unique properties. Some of these soils predominate in certain parts of the United States; for instance, caliche is widespread throughout the arid West and Southwest, and marl is common in areas bordering the central and eastern parts of the Gulf of Mexico. Others, while widespread over the United States, may occur in relatively small local deposits as is characteristic of peats and organic mucks.

Peat can be distinguished by its color (brown to black), the presence of considerable amounts of partially-carbonized plant fiber, its odor (a swampy smell), and its very high compressibility. When dried, it is brittle, can be ignited (usually), and will float on water (usually). Peat may contain some very fine sand, silt, or clay.

Organic muck has all the characteristics of peat except it has little or no fibrous material, usually has more silt and/or clay content, and cannot be ignited when dry.

Caliche is a fairly well-graded mixture of sand, silt, and clay (occasionally some gravel, also) which has been lightly to completely cemented with water-deposited calcite or lime. When dry, it may vary from crumbly and chalky to very hard and rocklike. It has a characteristic very light brown to white appearance and will effervesce strongly when a drop or two of silute hydrochloric acid is placed on the surface.

Marl is a very fine clayey sand having a high calcium carbonate content. It can be identified by its appearance and by strong effervescence when treated dilute hydrochloric acid.

Bentonite is a very fine-grained colloidal clay which possesses extreme swelling properties when wetted and which is extremely hard and difficult to fracture when air-dry. When very stiff to stiff, it has a soapy or waxy appearance and feel. When fully saturated and slaked, it has a gelatinous appearance and feel. In the plastic state, bentonite is very sticky.

Topsoil generally can be distinguished by the presence of rootlets or root holes (also worm holes), by the color (yellow, red, or brown), and by the organic content. The color usually becomes progressively lighter with depth.

Fill is the name given to all man-made deposits of natural or waste materials. It may consist of almost any conceivable material or combinations thereof. When the fill is composed of soil materials, identification may be very difficult but usually can be accomplished by noting the lack of regular

Ref. 4.3 Cont'd

bedding and by the presence of topsoil underlying the fill. Fills of non-soil materials or mixtures of non-soil and soil materials usually show man-made objects or artifacts such as brick or concrete fragments, plaster, pieces of wood, or metal objects.

Loess is an aeolian material which, because it has been deposited by wind action, is usually almost uniform in grain size (coarse silt and ultra-fine sand) and particles are angular to subangular forming a strong interlock when the material is dry, which strength is lost completely when disturbed or remolded and saturated.

Terminology & Abbreviations for Soil Descriptions

Color			Adjectives:
	White (wh)	brown (br)	light (lt)
	black (blk)	red (r)	dark (dk)
	gray (gr)	purple (p)	
	blue (bl)	green (grn)	
	yellow (y)		

Basic Soil Type

bedrock (bdrk)	adjectives: large (lge), medium (med), small (sm)
ledgerock (ldgrk)	
boulder(s) (bldr)	coarse (co), medium (med), fine (fi)
cobbles (cobs)	
gravel (ly) (grav)	inorganic (inorg), organic (org)
sand(y) (sa)	
silt(y) (si)	
clay(ey) (cl)	

Modifying Terms

with with (w/)
- with some (w/so)
- with scattered (s/sc)
- rock fragments (rk.frags)
- varved (v)
- partings (ptgs)
- lens(es) (lns)
- alternate (alt)

fine-grained soils:
- soft (s)
- medium (med)
- stiff (st)
- very stiff (v.st)
- hard (hd)

fine grained soils:
- compressible (comp)
- plastic (pl)
- non-plastic (npl)

coarse-grained soils:
- angular (ang)
- subangular (sbang)
- subrounded (sbrd)
- rounded (rd)
- well-graded (w.g.)
- poorly-graded (p.g)
- uniform (uni)

Source: Professor R. L. Sloane
University of Arizona

LAB SOIL TESTS

An approximate 100 pound sample was taken from the southwest sector of the TRESTLE bowl indicated on Drawing No. 5.1. This particular location was selected because of proximity to the turbine pumps also shown on the drawing. Historically, the pumps have been damaged when the discharge tanks became heavily loaded with eroded soil. A sample taken from this particular location provides a good indication of soil composite getting into the pump lines.

The sample taken is top soil, approximately 4-6 inches deep. The soil was tested at the University of New Mexico, Civil Engineering Soils Lab with the exception of gradation tests performed by Albuquerque Testing Lab, Inc. (See Reference 4.4 and Figure 4.7). Table 4.12 summarizes lab test results. The remainder of this section describes test procedures and provides lab data sheets that display given test results.

Table 4.12: Lab Test Results; KAFB TRESTLE Sample; March 1984 (Tests performed at University of New Mexico, Civil Engineering Dept.)

			Ref. pp.
1.	Hygroscopic Moisture Content, w	1.72%	228-231
2.	Specific Gravity, G	2.65	232-233
3.	Liquid Limit, LL	20	234-235
4.	Plastic Limit, PL	20	234-235
5.	Plasticity Index, PI=LL-PL	NP	234-235
6.	Uniformity Coefficient, $C_u = \frac{D_{60}}{P_{10}}$	100	236
7.	Coefficient of Curvature, $C_c = \frac{D^2_{30}}{D_{10} \times D_{60}}$	17	236
8.	USDA Classification Triangular Classification Chart	Sandy Loam	
9.	AASHO Classification	A-4(0); Silty Soil; Fair to Poor as Subgrade	
10.	Unified Classification	SM;Silty Sand	

(Items 8-10, ref. Peck, Hanson and Thornburn, 1974.)

			Ref. pp.
11.	U.S. Army Corps. of Engineers. Triangular Classification Chart	Silty Sand	-248
12.	Unit Weight of Solids = $\frac{W_s}{V_s}$	165.6 pcf	253A
13.	Unit Dry Weight = $\frac{w_s}{V_t}$	102.4 pcf	253A
14.	Unit Wet Weight (stabilized at room temp.) = $\frac{W_t}{V_t}$	119.6 pcf	253A
15.	Void Ratio, e = $\frac{V_v}{V_s}$	0.65	253A

Table 4.12 (Continued)

16.	Porosity, $n = \frac{V_v}{V_t}$	0.39	253A
17.	*Permeability, k (Head, 1982)		
	a. HAZEN: $k = C_1(D_{10})^2 = 100(.0012)^2$	1×10^{-4} cm/s	
	b. Lambe and Whitman: $k=16(.0012)^2$	2×10^{-5} cm/s	
	c. Degree of permeability (Terzaghi and Peck, 1948)	Medium	
18.	Degree of saturation (s)	7%	253A
19.	Maximum dry density	117.5 pcf	253A
20.	Optimum moisture content	12.3%	253A

* The results of laboratory permeability tests on cohensionless soils such as sands are of limited value for determining the true permeability of these soils in their natural state. There are two main reasons for this: (1) Without specialized equipment, it is very difficult to measure the density, and hence the void ratio, of graunular soils in-situ—especially below the water table. Therefore, the void ratio at which to set up samples for test can only be surmised. (2) Even if the void ratio is approximately assessed the features of the soil fabric cannot be reproduced when a sample is recompacted in the lab (Head, 1982).

Permeability is not a fundamental property of soil but depends upon a number of factors (Head, 1982):

(1) Particle size distribution

(2) Particle shape and texture

(3) Mineralogical composition

(4) Void ratio

(5) Degree of saturation

(6) Soil fabric

(7) Nature of fluid

(8) Type of flow

(9) Temperature

Fig. 4.2
Flow Diagram
Sample Preparation
Disturbed Sample

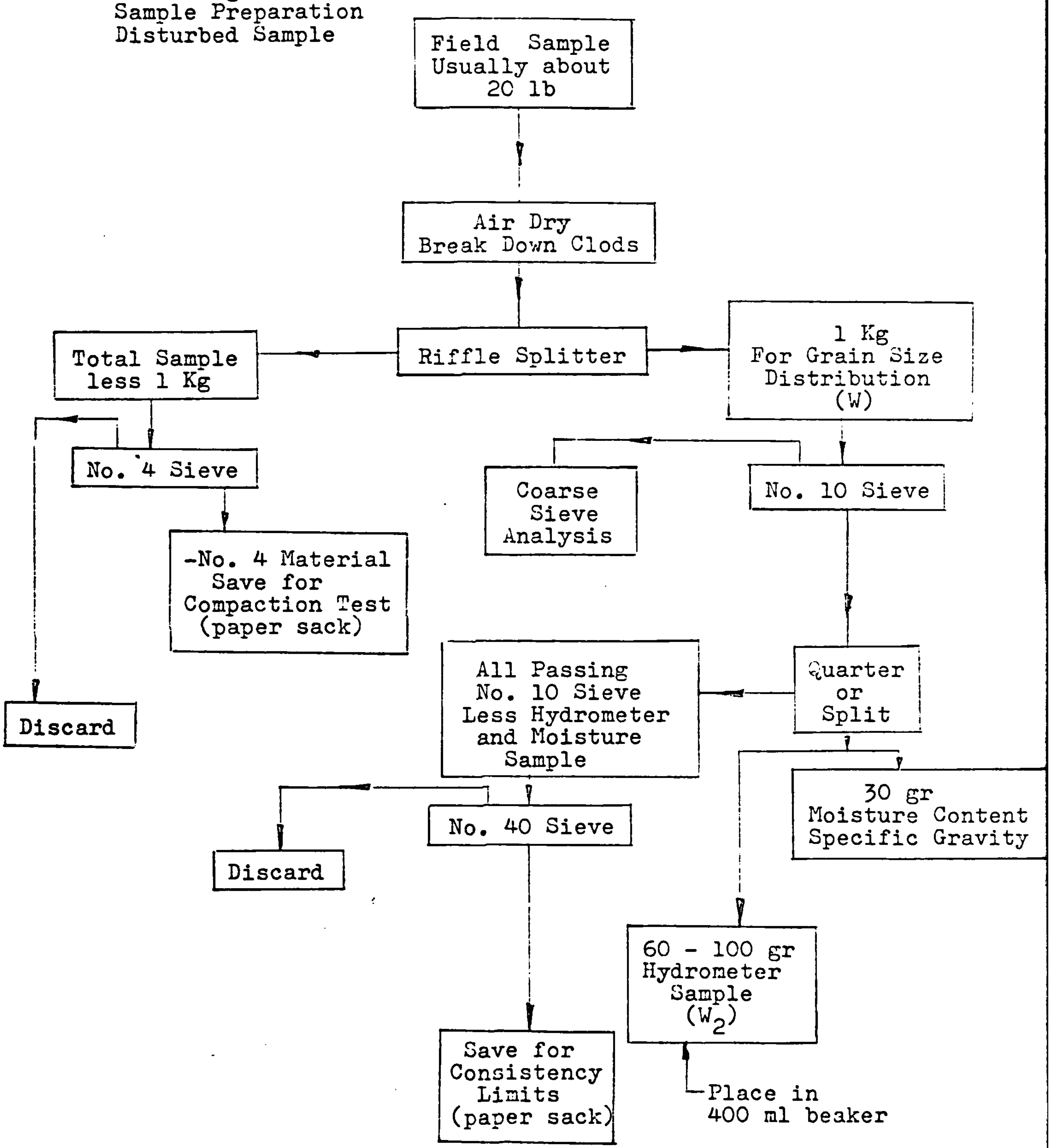
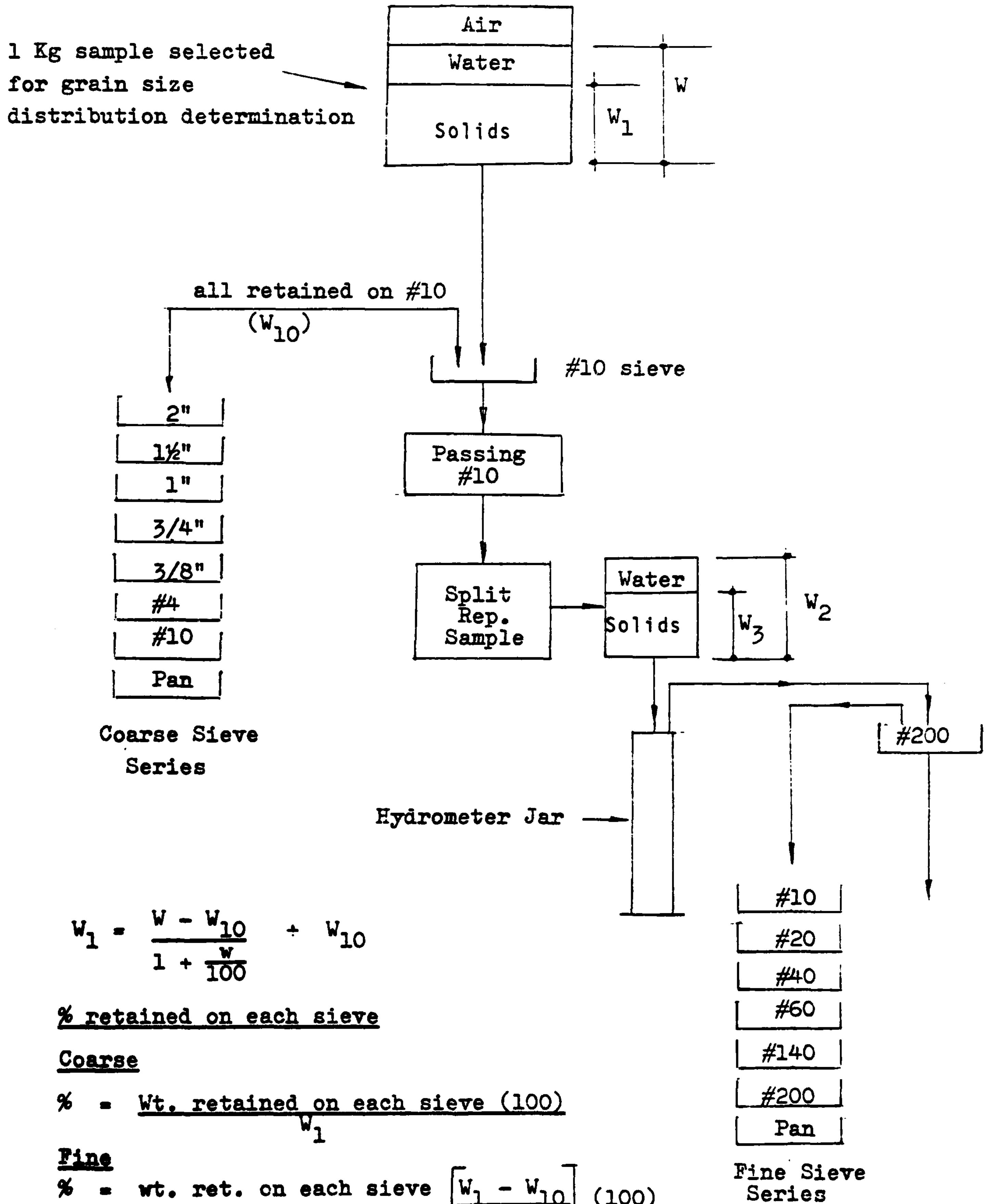


Fig. 4.3



$$W_1 = \frac{W - W_{10}}{1 + \frac{w}{100}} + W_{10}$$

<u>% retained on each sieve</u>

<u>Coarse</u>

$$\% = \frac{\text{Wt. retained on each sieve (100)}}{W_1}$$

<u>Fine</u>

$$\% = \frac{\text{wt. ret. on each sieve}}{W_3} \left[\frac{W_1 - W_{10}}{W_1}\right] (100)$$

Table 4.13

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Hygroscopic Moisture

Test No. 1

Tested By Koch   Party

Sample KAFB Trestle ~~Albuquerque, N.M.~~   Date 15-3-84

Remarks W = 1.72% was used for the sieve and hydrometer analysis computations because it more closely approximates natural field conditions. The second test was performed several days after the sample was brought into the lab.

Trial	1	2
Can no.	83-1	7-68
Weight of damp soil + can	51.04	50.56
Weight of dry soil + can	50.56	50.03
Weight of moisture	0.48	0.53
Weight of dry soil + can	50.56	50.03
Weight of can	21.08	20.81
Weight of dry soil	29.48	29.22
Hygroscopic moisture (w) %	1.63	1.81

Note: All weights in grams

$$W_1 = \frac{W_w}{W_s} = \frac{0.48}{29.48} = 1.63\%$$

$$W_2 = \frac{W_w}{W_s} = \frac{0.53}{29.22} = 1.81\%$$

$$W_{AVE} = \frac{1.63 + 1.81}{2} = 1.72\%$$

Table 4.14

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Hygroscopic Moisture

Test No. 2

Tested By Koch
Party
Sample KAFB Trestle Albuquerque, N.M.
Date 19-3-84
Remarks Use test 1 results for sieve and hydrometer analysis.

Trial	1	2
Can no.	7-68	7-08
Weight of damp soil + can	50.80	50.90
Weight of dry soil + can	50.41	50.50
Weight of moisture	0.39	0.40
Weight of dry soil + can	50.41	50.50
Weight of can	20.80	20.90
Weight of dry soil	29.61	29.60
Hygroscopic moisture (w) %	1.32	1.35

Note: All weights in grams

$$w_{AVE} = \frac{1.32 + 1.35}{2} = 1.33\%$$

Table 4.15

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Specific Gravity Determination

Test No. 1

Name Koch

Party

Sample KAFB Trestle Albuquerque, N.M.

Date 16 Mar. 84

	Trial #1
Weight of dry soil (A)	29.48
Weight of pycnometer + water (B)	151.08
Weight of pycnometer + soil + water (C) 51.41	169.37
A + B - C 51.41	11.19
Temperature of water (T)	23.5°C
Specific gravity of water at temperature T, (G_w)	0.99745

$$G = \frac{A\ G_w}{A + B - C}$$

$G =$ 2.63 Note: All weights in grams

$$G - \frac{2.63 + 267 + 2.62 + 2.68}{4} = 2.65$$

TABLE A-2. SPECIFIC GRAVITY OF WATER *

°C	0	1	2	3	4	5	6	7	8	9
0	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

* Also the density or unit weight of water in grams per milliliter.
From *International Critical Tables*, Vol. III, McGraw-Hill Book Co., 1928.

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Table 4.16

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Specific Gravity Determination

Test No. 1

Name Koch

Party

Sample KAFB Trestle Albuquerque, N.M.

Date 16 Mar 84

	Trial #2
Weight of dry soil (A)	29.22
Weight of pycnometer + water (B)	162.39
Weight of pycnometer + soil + water (C)	180.68
A + B - C	10.93
Temperature of water (T)	23.5°
Specific gravity of water at temperature T, (G_w)	0.99745

$$G = \frac{A\ G_w}{A + B - C}$$

G = 2.67

Note: All weights in grams

TABLE A-2. SPECIFIC GRAVITY OF WATER *

°C	0	1	2	3	4	5	6	7	8	9
0	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

* Also the density or unit weight of water in grams per milliliter.
From *International Critical Tables*, Vol. III, McGraw-Hill Book Co., 1928.

Table 4.17

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Specific Gravity Determination

Test No. 2

Name Koch

Party

Sample KAFB Trestle Albuquerque, N.M.

Date 22-3-84

	Trial #1
Weight of dry soil (A)	29.61
Weight of pycnometer + water (B)	151.16
Weight of pycnometer + soil + water (C)	169.51
A + B - C	11.26
Temperature of water (T)	25°C
Specific gravity of water at temperature T, (G_w)	.9971

$$G = \frac{A\,G_w}{A + B - C}$$

G = 2.62

Note: All weights in grams

TABLE A-2. SPECIFIC GRAVITY OF WATER *

°C	0	1	2	3	4	5	6	7	8	9
0	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

* Also the density or unit weight of water in grams per milliliter.
From *International Critical Tables*, Vol. III, McGraw-Hill Book Co., 1928.

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Table 4.18

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Specific Gravity Determination

Test No. 2

Name Koch

Party

Sample KAFB Trestle Albuquerque, N.M.

Date 22-3-84

	Trial #2
Weight of dry soil (A)	29.60
Weight of pycnometer + water (B)	162.42
Weight of pycnometer + soil + water (C)	181.00
A + B - C	11.02
Temperature of water (T)	23°C
Specific gravity of water at temperature T, (G_w)	.9971

$$G = \frac{A\,G_w}{A + B - C}$$

G = 2.68

Note: All weights in grams

TABLE A-2. SPECIFIC GRAVITY OF WATER *

°C	0	1	2	3	4	5	6	7	8	9
0	0.9999	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	0.9999	0.9999	0.9998
10	0.9997	0.9996	0.9995	0.9994	0.9993	0.9991	0.9990	0.9988	0.9986	0.9984
20	0.9982	0.9980	0.9978	0.9976	0.9973	0.9971	0.9968	0.9965	0.9963	0.9960
30	0.9957	0.9954	0.9951	0.9947	0.9944	0.9941	0.9937	0.9934	0.9930	0.9926
40	0.9922	0.9919	0.9915	0.9911	0.9907	0.9902	0.9898	0.9894	0.9890	0.9885
50	0.9881	0.9876	0.9872	0.9867	0.9862	0.9857	0.9852	0.9848	0.9842	0.9838
60	0.9832	0.9827	0.9822	0.9817	0.9811	0.9806	0.9800	0.9795	0.9789	0.9784
70	0.9778	0.9772	0.9767	0.9761	0.9755	0.9749	0.9743	0.9737	0.9731	0.9724
80	0.9718	0.9712	0.9706	0.9699	0.9693	0.9686	0.9680	0.9673	0.9667	0.9660
90	0.9653	0.9647	0.9640	0.9633	0.9626	0.9619	0.9612	0.9605	0.9598	0.9591

* Also the density or unit weight of water in grams per milliliter.
From *International Critical Tables*, Vol. III, McGraw-Hill Book Co., 1928.

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Table 4.19

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Department of Civil Engineering
Soil Mechanics Laboratory

Consistency Limits

Sample KAFB Trestle Albuquerque, N.M. Date 23-3-84 Test No. 1

Party ______ Name Koch

Liquid Limit (TEST PERFORMED IAW ASTM DESIGNATION: D423-61T)

Trial No.	1	2	3	4	5
No. of blows	45	18	21	8	10
Can No.	7-64	7-40	31	37	16
Wt. can + wet soil	33.36	33.83	37.07	35.51	36.30
Wt. can + dry soil	31.35	31.60	34.51	32.82	33.37
Wt. can	20.89	20.93	21.99	20.89	20.87
Wt. water	2.01	2.23	2.56	2.69	2.93
Wt. dry soil	10.46	10.67	12.52	11.93	12.50
% water	19.2	20.9	20.5	22.6	23.4

Plastic Limit (TEST PERFORMED IAW ASTM DESIGNATION: D424-59)

Trial No.	1	2	3
Can No.	7-52	7-25	17
Wt. can + wet soil	21.76	23.73	23.41
Wt. can + dry soil	21.52	23.19	23.13
Wt. can	20.38	20.44	21.54
Wt. water	0.24	0.54	0.28
Wt. dry soil	1.14	2.15	1.59
% water	21	20	18

Summary	
Liquid Limit	20
Plastic Limit	20
Plasticity Index	0

PI = LL - PL

WHEN THE PLASTIC LIMIT IS EQUAL TO, OR GREATER THAN, THE LIQUID LIMIT, REPORT THE PLASTICITY INDEX AS NON-PLASTIC, NP. (ASTM D424-59)

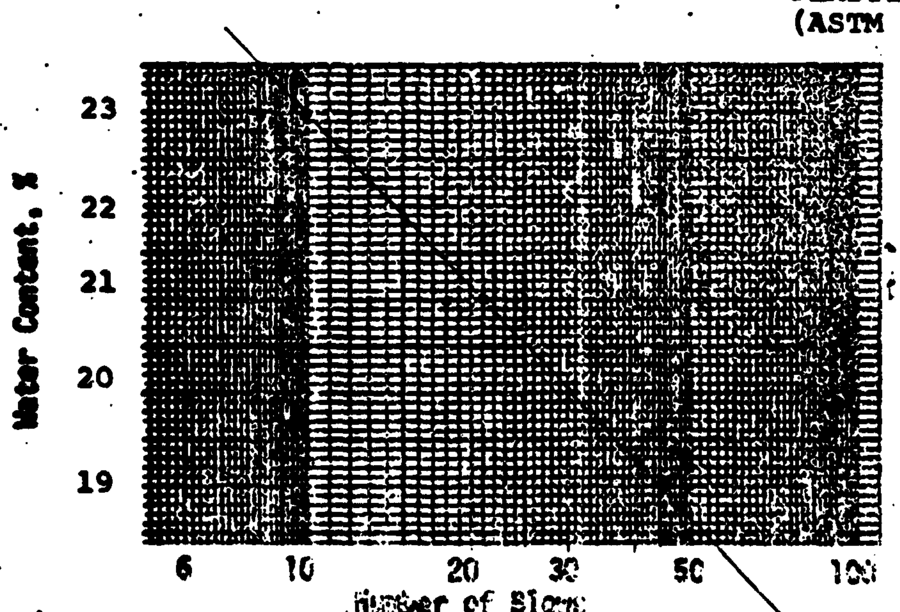

Table 4.20

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Soil Mechanics Laboratory

Consistency Limits

Sample KAFB Trestle Albuquerque, N.M. Date 27-3-84 Test No. 2

Party Name Koch

Liquid Limit (TEST PERFORMED IAW ASTM DESIGNATION: D423-617)

Trial No.	1	2	3	4	5
No. of blows	33	25	23	13	13
Can No.	7-63	57	17	45	7-39
Wt. can + wet soil	33.39	32.71	32.88	32.22	32.65
Wt. can + dry soil	31.41	30.77	30.97	30.01	30.54
Wt. can	21.01	21.07	21.56	19.83	20.90
Wt. water	1.98	1.94	1.91	2.21	2.11
Wt. dry soil	10.40	9.70	9.41	10.18	9.64
% water	19	20	20	22	22

Plastic Limit (TEST PERFORMED IAW ASTM DESIGNATION: D424-59)

Trial No.	1	2	3
Can No.	7-68	7-44	83-1
Wt. can + wet soil	25.65	24.87	23.66
Wt. can + dry soil	24.86	24.17	23.22
Wt. can	20.78	20.64	21.07
Wt. water	0.79	0.70	0.44
Wt. dry soil	4.08	3.53	2.15
% water	19	20	21

Summary	
Liquid Limit	20
Plastic Limit	20
Plasticity Index	0

PI = LL - PL

WHEN THE PLASTIC LIMIT IS EQUAL TO, OR GREATER THAN, THE LIQUID LIMIT, REPORT THE PLASTICITY INDEX AS NON-PLASTIC, NP. (ASTM D424-59)

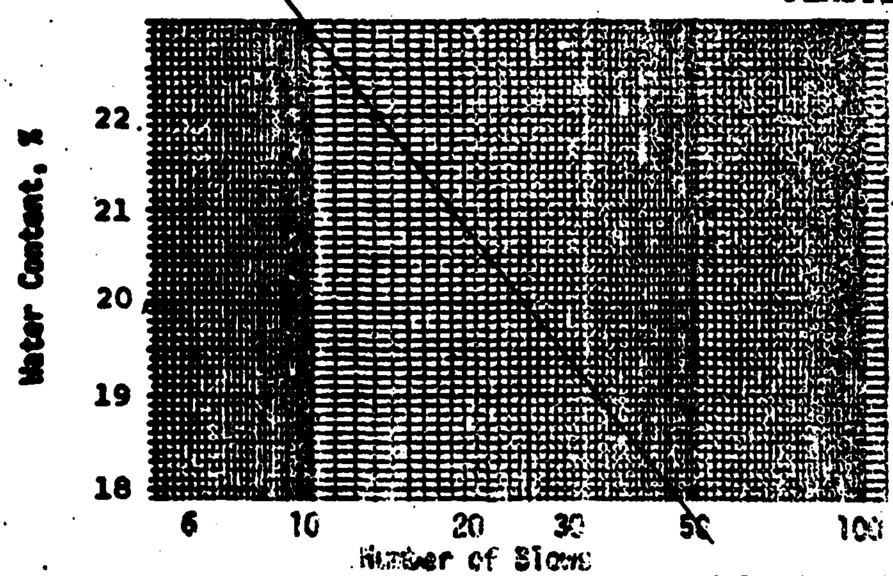

Figure 4.4

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University of New Mexico

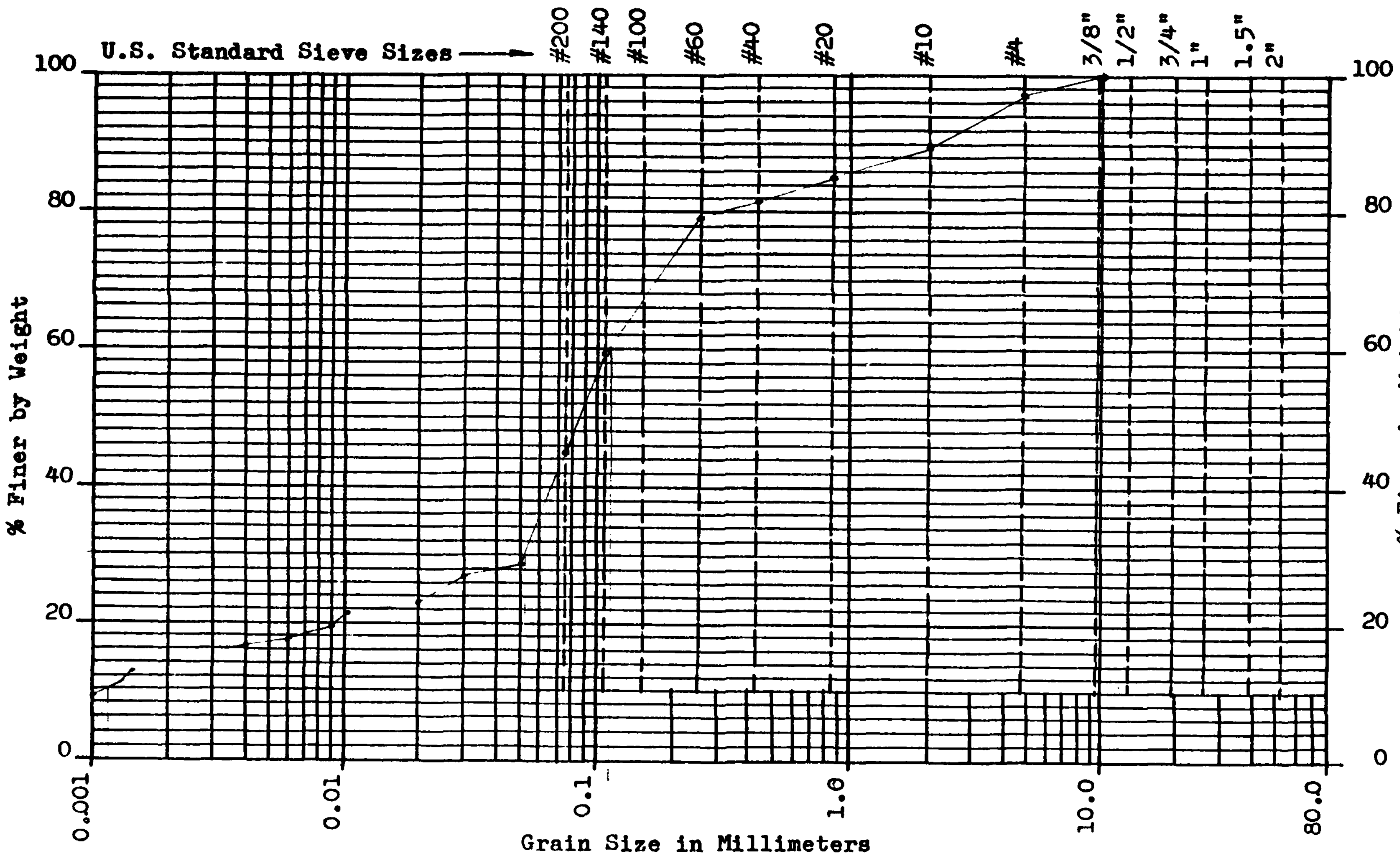
Grain Size Distribution Curve

Test No. 2

Sample KAFB TRESTLE Albuquerque, N M

Tested By Koch Party ____ Date 26-3-84

D_{10} 0.0012 D_{60} 0.12 U_c 100



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DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

Computation of Weights used in Grain Size Distribution

Test No. 2

Sample KAFB Trestle Albuquerque, N.M. Name Koch Date 19-3-84

Party ______ Hygroscopic Moisture Content (w) 1.72 %

Item		
A	Weight of sample selected for grain size analysis. This sample has not been corrected for moisture content and is called (W). (W)	1000.00
B	Amount of total sample (W) retained on Number 10 sieve. (Dry weight of coarse material)	100.25
C	A less B. This is the amount of (W) passing the Number 10 sieve.	899.75
D	$\frac{C}{1 + \frac{w}{100}}$ This is the material passing the Number 10 sieve and corrected for hygroscopic moisture content.	884.54
E	B + D This is the dry weight of the total sample. This weight is called (W_1). The grain size distribution is based upon this weight. (W_1)	984.79
F	The sample selected from C for the hydrometer test (60 - 100 gr) is called (W_2). This weight has not been corrected for moisture content. This sample is assumed to be representative of all material passing the Number 10 sieve. (W_2)	60.00
G	$\frac{F}{1 + \frac{w}{100}}$ This weight is (W_2) corrected for moisture content. This weight is called (W_3). (W_3)	58.98

The grain size distribution is based upon W_1. Percentages based upon W_3 can be converted to a basis of W_1 by multiplying the percentages based upon W_3 by the factor

$$\frac{W_1 - B}{W_1} = \frac{984.79-100.25}{984.79}$$ 0.90

Comments All weights in grams

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Hydrometer Analysis
Example
Test No. 4.1

You Will Determine   From Table I

Tested By ____________ Party ____________ Date ____________

Sample ____________ G ____ a ____ Hydrometer Type 152H W_2 ____ W_3 ____

Dispersing agent = 5g Sodium Metaphosphate

Clock Time	Elapsed Time min	Temp °C	Orig R	Men Corr	Temp Corr	ΣR	% Passing W_3	% Passing W_1	Orig R + Men Corr	L cm	K	Particle Size mm
Determine Before Start of Test. Elapsed time is time from Start of test. Test starts at 0 time.		Record for each reading	This is the original reading at the top of the meniscus	This is taken at start and is Constant for entire test.	This is the correction for your hydrometer. It includes the correction for dispersing agent, temperature + meniscus.	Cols. 4 + 6	$\frac{\text{Col. 7} \times a}{W3} \times 100$	Col. 8 $\times \left[\frac{W1 - W10}{W1}\right]$	Col. 4 + Col. 5	From Table II using Col. 10 as actual Hydrometer Reading	From Table III	$dmm = K\sqrt{\frac{L}{T}}$ Col. 12; L = Col 11, T = Col 2
	1/2											
	1											
	2											
	5											
	10											
	15											
	30											
	60											
	120											
	→											
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)

Table 4.22
Tables for Use in Hydrometer Analysis

Source: Procedures For Testing Soils ASTM 1964

TABLE I.—VALUES OF CORRECTION FACTOR, a, FOR DIFFERENT SPECIFIC GRAVITIES OF SOIL PARTICLES.*

Specific Gravity	Correction Factor, a
2.95	0.94
2.90	0.95
2.85	0.96
2.80	0.97
2.75	0.98
2.70	0.99
2.65	1.00
2.60	1.01
2.55	1.02
2.50	1.03
2.45	1.05

* For use in formula for percentage of soil remaining in suspension when using Hydrometer 152H.

Temperature Correction

Temp °C	R
21	+0.7
22	+1.0
23	+1.3
24	+1.6
25	+1.9
26	+2.2
27	+2.5

TABLE II.—VALUES OF EFFECTIVE DEPTH BASED ON HYDROMETER AND SEDIMENTATION CYLINDER OF SPECIFIED SIZES.*

Hydrometer 151H		Hydrometer 152H			
Actual Hydrometer Reading	Effective Depth, L, cm	Actual Hydrometer Reading	Effective Depth, L, cm	Actual Hydrometer Reading	Effective Depth, L, cm
1.000	16.3	0	16.3		
1.001	16.0	1	16.1	31	11.2
1.002	15.8	2	16.0	32	11.1
1.003	15.5	3	15.8	33	10.9
1.004	15.2	4	15.6	34	10.7
1.005	15.0	5	15.5	35	10.6
1.006	14.7	6	15.3	36	10.4
1.007	14.4	7	15.2	37	10.2
1.008	14.2	8	15.0	38	10.1
1.009	13.9	9	14.8	39	9.9
1.010	13.7	10	14.7	40	9.7
1.011	13.4	11	14.5	41	9.
1.012	13.1	12	14.3	42	9.
1.013	12.9	13	14.2	43	9.
1.014	12.6	14	14.0	44	9.
1.015	12.3	15	13.8	45	8.
1.016	12.1	16	13.7	46	8.
1.017	11.8	17	13.5	47	8.
1.018	11.5	18	13.3	48	8.
1.019	11.3	19	13.2	49	8.
1.020	11.0	20	13.0	50	8.
1.021	10.7	21	12.9	51	7.
1.022	10.5	22	12.7	52	7.
1.023	10.2	23	12.5	53	7.
1.024	10.0	24	12.4	54	7.
1.025	9.7	25	12.2	55	7.
1.026	9.4	26	12.0	56	7.
1.027	9.2	27	11.9	57	7.
1.028	8.9	28	11.7	58	6.
1.029	8.6	29	11.5	59	6.
1.030	8.4	30	11.4	60	6.
1.031	8.1				
1.032	7.8				
1.033	7.6				
1.034	7.3				
1.035	7.0				
1.036	6.8				
1.037	6.5				
1.038	6.2				

TABLE III.—VALUES OF K FOR USE IN FORMULA FOR COMPUTING DIAMETER O[F] PARTICLE IN HYDROMETER ANALYSIS.

Temperature, deg Cent	Specific Gravity of Soil Particles								
	2.45	2.50	2.55	2.60	2.65	2.70	2.75	2.80	2.85
16	0.01530	0.01505	0.01481	0.01457	0.01435	0.01414	0.01394	0.01374	0.013
17	0.01531	0.01486	0.01462	0.01439	0.01417	0.01396	0.01376	0.01356	0.013
18	0.01492	0.01467	0.01443	0.01421	0.01399	0.01378	0.01359	0.01339	0.013
19	0.01474	0.01449	0.01425	0.01403	0.01382	0.01361	0.01342	0.01323	0.013
20	0.01456	0.01431	0.01408	0.01386	0.01365	0.01344	0.01325	0.01307	0.012
21	0.01438	0.01414	0.01391	0.01369	0.01348	0.01328	0.01309	0.01291	0.012
22	0.01421	0.01397	0.01374	0.01353	0.01332	0.01312	0.01294	0.01276	0.012
23	0.01404	0.01381	0.01358	0.01337	0.01317	0.01297	0.01279	0.01261	0.012
24	0.01388	0.01365	0.01342	0.01321	0.01301	0.01282	0.01264	0.01246	0.012
25	0.01372	0.01349	0.01327	0.01306	0.01286	0.01267	0.01249	0.01232	0.012
26	0.01357	0.01334	0.01312	0.01291	0.01272	0.01253	0.01235	0.01218	0.012
27	0.01342	0.01319	0.01297	0.01277	0.01258	0.01239	0.01221	0.01204	0.011
28	0.01327	0.01304	0.01283	0.01264	0.01244	0.01225	0.01208	0.01191	0.011
29	0.01312	0.01290	0.01269	0.01249	0.01230	0.01212	0.01195	0.01178	0.011
30	0.01298	0.01276	0.01256	0.01236	0.01217	0.01199	0.01182	0.01165	0.011

Table 4.23

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Soil Mechanics Laboratory

Hydrometer alysis

Test No. 1

DISPERSANT = 5g SODIUM METAPHOSPHATE

Tested By Koch Party ____ Date 22-3-84

Sample KAFB TRESTLE Albuquerque, N M G 2.65 a 1.00 Hydrometer Type 152H #25589 W_2 59.84 W_3 58.83

	Clock Time	Elapsed Time min	Temp °C	Orig R	Men Corr	Temp Corr	ΣR	% Passing W_3	% Passing W_1	Orig R + Men Corr	L cm	K	Particle Size mm
22-3	1238	0											
	1234	1	25	22.0	0.5	-3.8	18.2	30.9	27.8	22.5	12.6	0.01286	0.0457
	1235	2	25	21.0	0.5	-3.8	17.2	29.2	26.3	21.5	12.8	0.01286	0.0325
	1240	7	25	20.0	0.5	-3.8	16.2	27.5	24.8	20.5	12.95	0.01286	0.0175
	1248	15	25	17.5	0.5	.3.8	13.7	23.3	21.0	18.0	13.3	0.01286	0.0121
	1303	30	25	16.0	0.5	-3.8	12.2	20.7	18.6	16.5	13.6	0.01286	0.0058
	1333	60	25	15.5	0.5	-3.8	11.7	19.9	17.9	16.0	13.7	0.01286	0.0062
	1430	117	25.5	14.0	0.5	-3.7	10.3	17.5	15.8	14.5	13.9	0.01279	0.0044
	1700	267	25.5	13.0	0.5	-3.7	9.3	15.8	14.2	13.5	14.1	0.01279	0.0029
23-3	0912	1239	24.0	12.0	0.5	-3.9	8.1	13.8	12.4	12.5	14.25	0.01301	0.0014
	1156	1403	25.0	11.0	0.5	-3.8	7.2	12.2	11.0	11.5	14.4	0.01286	0.0013

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Soil Mechanics Laboratory

Table 4.23

Hydrometer .alysis

Test No. 1

DISPERSANT = 5g SODIUM METAPHOSPHATE (Continued)

Tested By Koch Party ______ Date 22-3-84

Sample KAFB TRESTLE Albuquerque, N M G 2.65 a 1.00 Hydrometer Type 152H #25589 W_2 59.84 W_3 58.83

	Clock Time	Elapsed Time min	Temp °C	Orig R	Men Corr	Temp Corr	ΣR	% Passing		Orig R + Men Corr	L cm	K	Particle Size mm
								W_3	W_1				
	1414	1541	25.5	10.0	0.5	-3.7	6.3	10.7	9.6	10.5	14.6	0.01279	0.0012
	2100	1747	25.0	9.5	0.5	-3.8	5.7	9.7	8.7	10.0	14.7	0.01286	0.0011
5-3	1600	4527	24.5	9.0	0.5	-3.85	5.15	8.8	7.9	9.5	14.75	0.012935	0.0007
6-3	1200	5727	24.0	9.0	0.5	-3.9	5.1	8.7	7.8	9.5	14.75	0.01301	0.0007

Table 4.24

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Soil Mechanics Laboratory

Hydrometer Analysis

DISPERSING AGENT = 5g SODIUM METAPHOSPHATE

Test No. 2

Tested By Koch   Party ____   Date 26-3-84

Sample KAFB TRESTLE Albuquerque, N M   G 2.65   a 1.00   Hydrometer Type 152H #25589 W_2 60 W_3 58.98

29-3

Clock Time	Elapsed Time min	Temp °C	Orig R	Men Corr	Temp Corr	ΣR	% Passing		Orig R + Men Corr	L cm	K	Particle Size mm
							W_3	W_1				
1150	2841	24.0	10.0	0.5	-3.9	6.10	10.3	9.27	10.5	14.6	0.01301	0.0009
0930	4141	24.0	10.0	0.5	-3.9	6.10	10.3	9.27	10.5	14.6	0.01301	0.0008
1130	4361	25.0	9.0	0.5	-3.8	5.20	8.8	7.92	9.5	14.75	0.01286	0.0008
1550	4521	26.0	8.0	0.5	-3.6	4.40	7.5	6.75	8.5	14.9	0.01272	0.0007

Table 4.25

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Coarse & Fine Sieve Analysis

Test No. Example 4

Tested By ______________ Party ______________

Sample ______________ Date ______________

____

Coarse Sieve Analysis W_1 ______

Sieve Size	Opening Size	Retained Weight	Retained % of W_1	Cumulative % of W_1 Retained	Cumulative % of W_1 Passing
	mm	grams	%	%	%
2"	50.8	This is the Wt retained on ea. Sieve.			
1.5"	38.1		$\frac{\text{Col (3)}}{W_1} \times 100$	Σ Col (4)	100 - Col (5)
1"	25.4				
3/4"	19.1				
3/8"	9.52				
No. 4	4.76				
No. 10	2.00			•	•
Total		W_{10}			
(1)	(2)	(3)	(4)	(5)	(6)

Same Entry

Same

Fine Sieve Analysis W_3 ______

Sieve No.	Opening Size	Retained Weight	Retained % of W_3	Retained % of W_1	Cumulative % of W_1 Retained	Cumulative % of W_1 Passing
	mm	grams	%	%	%	%
10	2.00	—	—	—	•	•
20	0.840	0 This is the WT RETAINED ON EACH SIEVE				
40	0.420		$\frac{\text{Col (3)}}{W_3} \times 100$	$\text{Col (4)} \left[\frac{W_1 - W_{10}}{W_1}\right]$	Σ Col (5)	100 - Col (6)
60	0.250					
140	0.105					
200	0.074					
Total						
(1)	(2)	(3)	(4)	(5)	(6)	(7)

Table 4.25 A

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Coarse & Fine Sieve Analysis

Test No. 1

Tested By Koch

Party

Sample KAFB Trestle

Albuquerque, N.M.

Date 27-3-84

Coarse Sieve Analysis

W_1

Sieve Size	Opening Size	Retained Weight	Retained % of W_1	Cumulative % of W_1	
				Retained	Passing
	mm	grams	%	%	%
2"	50.8				
1.5"	38.1				
1"	25.4				
3/4"	19.1				
3/8"	9.52				
No. 4	4.76				
No. 10	2.00				
Total					

Fine Sieve Analysis

W_3 58.83

Sieve No.	Opening Size	Retained Weight	Retained % of W_3	Retained % of W_1	Cumulative % of W_1	
					Retained	Passing
	mm	grams	%	%	%	%
10	2.00	0.32	0.54			
20	0.840	3.47	5.90			
40	0.420	1.98	3.37			
60	0.250	2.03	3.45			
140	0.105	12.63	21.47			
200	0.074	10.00	17.00			
Total	Pan	1.62				

Table 4.26

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Coarse & Fine Sieve Analysis

Test No. 2

Tested By Koch

Party

Sample KAFB Trestle
Albuquerque, N. M.

Date 30-3-84

Coarse Sieve Analysis

W_1 984.79

Sieve Size	Opening Size	Retained Weight	Retained % of W_1	Cumulative % of W_1	
				Retained	Passing
	mm	grams	%	%	%
2"	50.8	0	0	0	100
1.5"	38.1	0	0	0	100
1"	25.4	0	0	0	100
3/4"	19.1	0	0	0	100
3/8"	9.52	0	0	0	100
No. 4	4.76	31.50	3.20	3.20	96.80
No. 10	2.00	68.75	6.98	10.18	89.82
Total		100.25			

WT. OF PAN + DRY WT. RETAINED ON # 200 = 598.8
WT. OF PAN = 567.2
WT. OF DRY SOIL = 31.6

Fine Sieve Analysis

W_3 58.98

Sieve No.	Opening Size	Retained Weight	Retained % of W_3	Retained % of W_1	Cumulative % of W_1	
					Retained	Passing
	mm	grams	%	%	%	%
10	2.00	0.61	1.03	0.92	10.18	89.82
20	0.840	3.23	5.48	4.93	15.11	84.89
40	0.420	1.97	3.34	3.01	18.12	81.88
60	0.250	2.01	3.41	3.07	21.19	78.81
140	0.105	12.56	21.30	19.17	40.36	59.64
200	0.074	10.03	17.00	15.30	55.66	44.34
Total		30.41	51.56	46.4	55.66	44.34

TEXTURAL CLASSIFICATION

SAMPLE: KAFB TRESTLE

DATE: 9 April 84

Table 4 - 27

Textural Classification (IAW U.S. Public Roads Administration	% of Total	% of -2mm fraction
Gravel (+2mm) = 100 - 89.8 =	10.2	
Sand (3 to 0.05mm) = 89.8 - 35.0 =	54.8	$\frac{54.8}{89.8}$ = 61
Silt (0.05 to 0.005mm) = 35.0 = 17.0 =	18.0	$\frac{18.0}{89.8}$ = 20
Clay (- 0.005mm) = 17.0 - 0 =	17.0	$\frac{17}{89.8}$ = 19
Total	100	100

SAMPLE IS SILTY SAND

(Also refer to triangular classification chart, Fig. 4.5).

Fig. 4.5

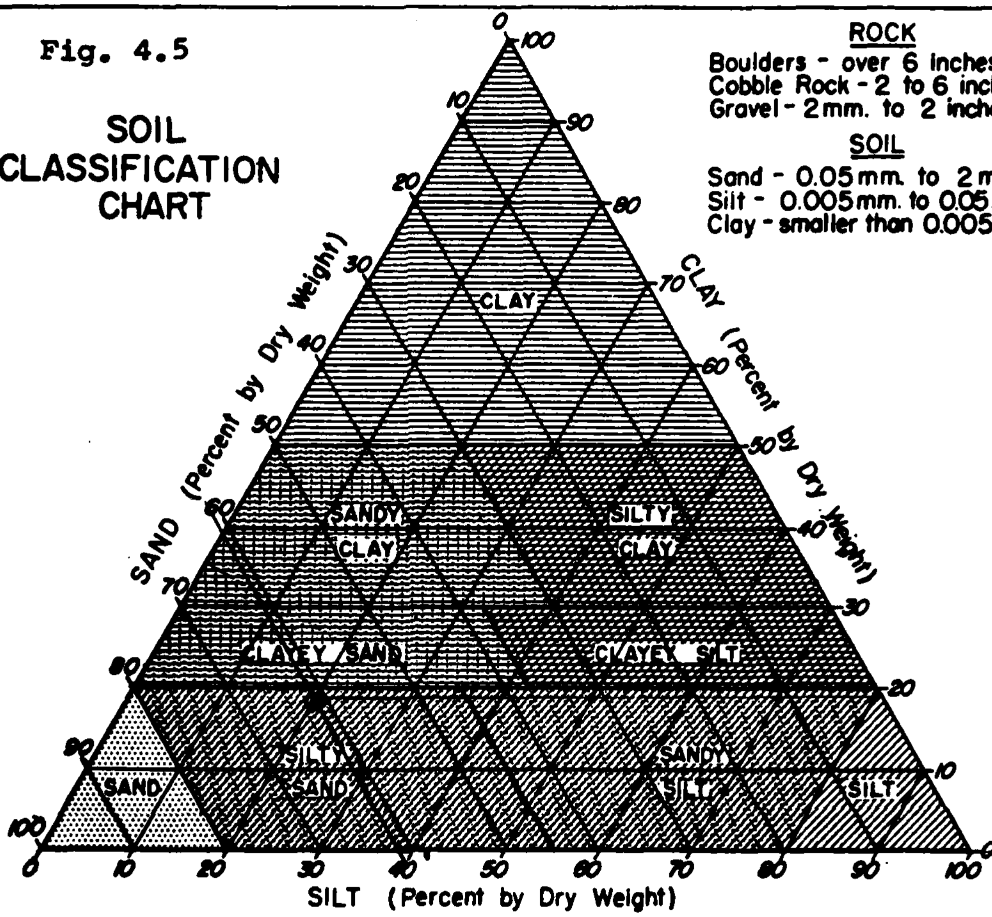
SOIL CLASSIFICATION CHART

ROCK

Boulders - over 6 inches
Cobble Rock - 2 to 6 inches
Gravel - 2 mm. to 2 inches

SOIL

Sand - 0.05 mm. to 2 mm.
Silt - 0.005 mm. to 0.05 mm.
Clay - smaller than 0.005 mm.



KEY TO BORING LOGS

TOPSOIL (or FILL)

CLAY

GRAVEL

SANDY CLAY (or CLAYEY SAND)

SAND

SILTY SAND (or SANDY SILT)

SILT

CLAYEY SILT (or SILTY CLAY)

UNDISTURBED SAMPLE Taken at this level

Indicates GROUND WATER LEVEL

37 Indicates number of blows of a 140 lb. weight falling 30 in. req'd. to drive sampler one foot.

SAND & SILT (Relative Density)

VERY LOOSE - LESS THAN 4 BLOWS/FOOT.
LOOSE - 4 TO 10 BLOWS/FOOT.
MEDIUM - 10 TO 30 BLOWS/FOOT.
DENSE - 30 TO 50 BLOWS/FOOT.
VERY DENSE - MORE THAN 50 BLOWS/FOOT.

CLAY (Consistency)

VERY SOFT - LESS THAN 2 BLOWS/FOOT.
SOFT - 2 TO 4 BLOWS/FOOT.
MEDIUM - 4 TO 8 BLOWS/FOOT.
STIFF - 8 TO 15 BLOWS/FOOT.
VERY STIFF - 15 TO 30 BLOWS/FOOT.
HARD - MORE THAN 30 BLOWS/FOOT.

Sample: KAFB TRESTLE

Consistency Limit Test Results → LL=20, PL=20, PI=0

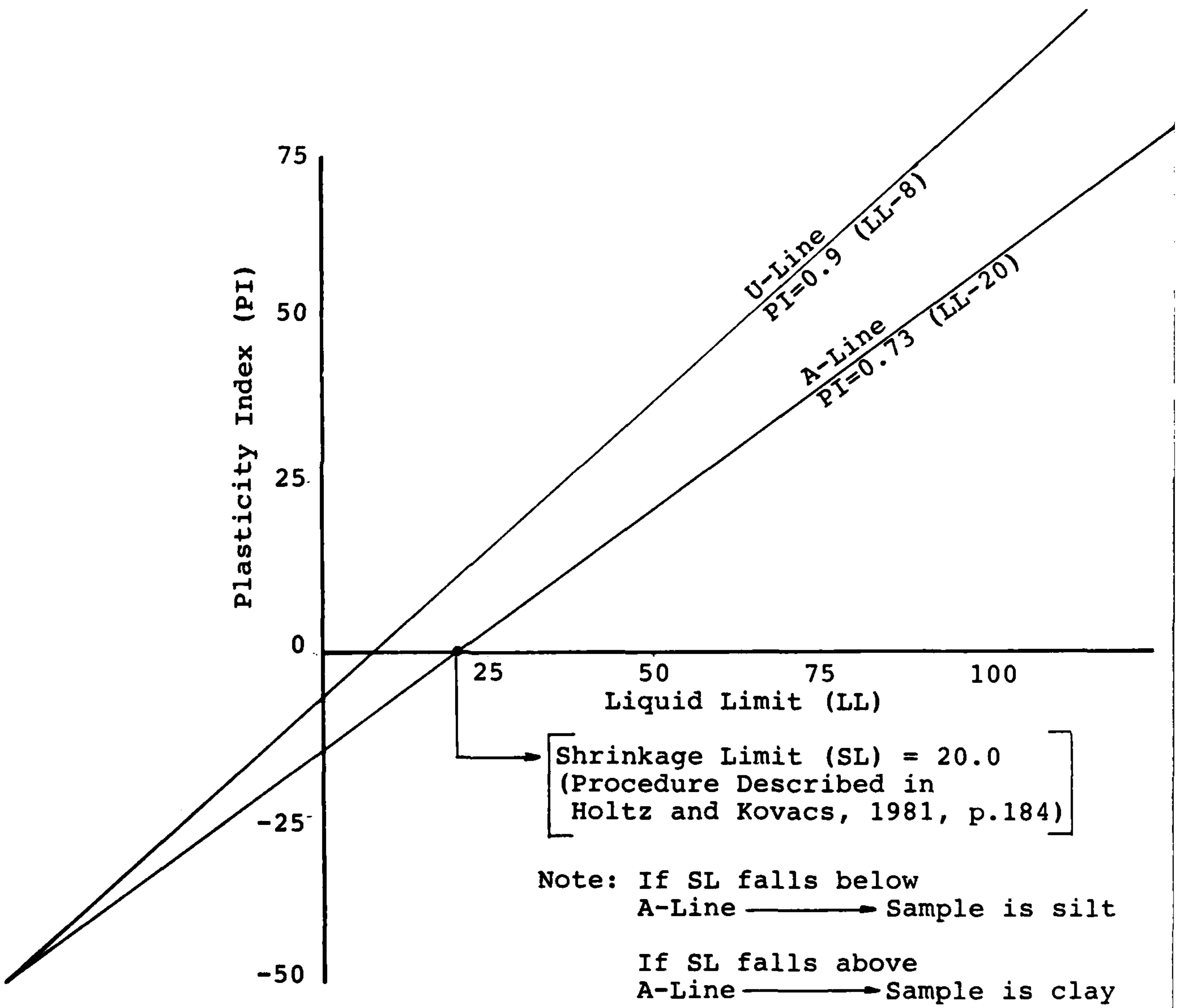


Fig. 4.6 Casagrande's Plasticity Chart (Developed From Casagrande, 1948 and Howard, 1977).

ATL ENGINEERING SERVICES

Reference 4.4
Albuquerque Testing Laboratory, Inc.
532 Jefferson N.E. (87108)
P. O. Box 4101 (87106)
Albuquerque, New Mexico
(505) 268-4537

Scanlon & Associates
7911 Mountain Road, N.E.
Albuquerque, New Mexico 87110

ATL Lab No. 3512-82

Report Date: September 8, 1982

Attention: Mr. Ross E. Schmidt

TEST RESULTS

Project: Trestle Erosion Control - Kirtland AFB

Source of Material: Two samples of soil delivered to our laboratory on August 28, 1982.

SIEVE ANALYSIS TEST: (ASTM C-117 & C-136)

Sieve Size	West Side Sample 1	East Side Sample 2
3/4"	100	
1/2"	98	100
3/8"	96	99
No. 4	85	93
No. 10	72	85
No. 40	59	75
No. 80	53	69
No. 200	29.1	31.0
Material Finer Than No. 200 Sieve by Washing	25.3	27.0

Respectfully Submitted,

ATL ENGINEERING SERVICES

Ray Mondragon

Ray Mondragon, P.E.
Project Engineer

LHO/co

Fig. 4.7

University of New Mexico
Department of Civil Engineering
University of New Mexico

Grain Size Distribution Curve

Test No. ______

Sample KAFB TRESTLE, Albuquerque, N. M. — Tested By Albuquerque Testing Lab. Inc. — Party ______ — Date 8 Sept. 1982

D_{10} ______ D_{60} ______ U_c ______

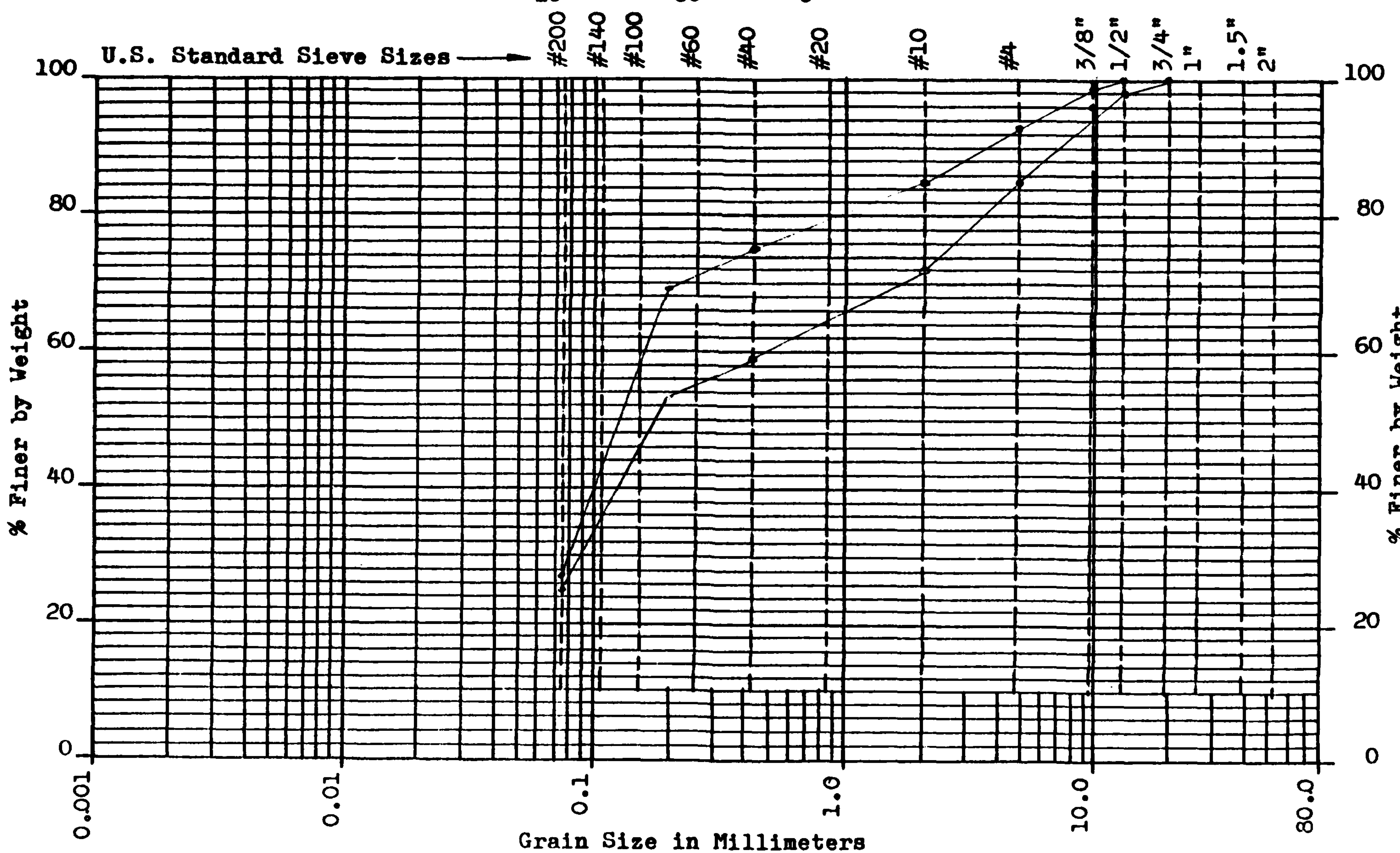


Table 4.28 Weight-Volume Relationships of Soil Aggregate

Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State

Description	Porosity (n)	Void Ratio (e)	Water Content (w)[a]	Unit Weight g/cu cm γd[b]	Unit Weight g/cu cm γsat[e]	Unit Weight lb/cu ft γd	Unit Weight lb/cu ft γsat
1. Uniform sand, loose	0.46	0.85	32	1.43	1.89	90	118
2. Uniform sand, dense	0.34	0.51	19	1.75	2.09	109	130
●3. Mixed-grained sand, loose	0.40	0.67	25	1.59	1.99	99	124
4. Mixed-grained sand, dense	0.30	0.43	16	1.86	2.16	116	135
5. Windblown silt (loess)	0.50	0.99	21	1.36	1.86	85	116
6. Glacial till, very mixed-grained	0.20	0.25	9	2.12	2.32	132	145
7. Soft glacial clay	0.55	1.2	45	1.22	1.77	76	110
8. Stiff glacial clay	0.37	0.6	22	1.70	2.07	106	129
9. Soft slightly organic clay	0.66	1.9	70	0.93	1.58	58	98
10. Soft very organic clay	0.75	3.0	110	0.68	1.43	43	89
11. Soft montmorillonitic clay (calcium bentonite)	0.84	5.2	194	0.43	1.27	27	80

[a] W=water content when saturated, in per cent of dry weight.

[b] γd=dry weight.

[e] γsat=saturated unit weight.

Table 4.29 Permeability of Soil

Coefficient of Permeability of Various Soils

k (cm/sec)	Drainage	Soil Type	Determination of k				
10^{2} 10^{1}	Good Good	Clean gravels Clean gravels	Pumping tests. Reliable if properly conducted	Constant head permeameter-reliable			Computation from grain size
1.0	Good	Clean sands			Falling-head permeameter	Reliable	
10^{-1} 10^{-2} 10^{-3} 10^{-4}	Good Good Good Good	Clean sand and gravel mixtures					
10^{-5} 10^{-6}	Poor Poor	Very fine sands Organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits.				Unstable. Much experience required	
10^{-7} 10^{-8} 10^{-9}	Practically impervious	Impervious soils, for example, homogeneous clays below zone of weathering.				Fairly reliable	Computation from consolidation data (reliable)

After Casagrande and Fadum (1940).

UNIVERSITY OF NEW MEXICO
DEPARTMENT OF CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

253A

COMPACTION CHARACTERISTICS

Table 4.30

Test No. 1

Name Koch ~~Party~~/ 453.6 g = 1 lb. Date 19 April 84

Sample KAFB TRESTLE Albuquerque, N. M. (-4 material) Method of Test Standard Proctor

G 2.65 Mold Volume 4.31 in^3

Trial	1	2	3	4	5	6	7
Wt of Mold + Wet Soil	296.17	310.95	311.98	315.49	311.43	325.50	329.82
Wt of Mold	179.26	179.26	179.26	179.26	179.26	179.26	179.26
Wt of Wet Soil	116.91	131.69	132.72	136.23	137.67	146.24	150.56
Wet Density (PCF)	103.3	116.4	117.3	120.4	121.7	129.3	133.1
Moisture Can No.	Pan	8-28	8-16	3	44	8-27	5
Wt of Can + Wet Soil	220.79	170.98	172.87	173.75	175.79	182.69	188.19
Wt of Can + Dry Soil	219.74	162.53	164.10	164.05	164.96	168.64	168.36
Wt of Water	1.05	8.45	8.77	9.70	10.83	14.05	19.83
Wt of Can	103.88	39.41	40.20	37.71	37.66	36.53	37.77
Wt of Dry Soil	115.86	123.12	123.90	136.34	127.30	132.11	130.59
Moisture Content (%)	0.9	6.9	7.1	7.7	8.5	10.6	15.2
Dry Density (PCF)	102.4	108.8	109.5	111.7	112.5	116.8	115.4

S = 100%	
w	γ_d
%	PCF
14	120.6
16	116.1
20	108.1

$\gamma_d \rightarrow \dfrac{(62.4 \times G)}{(1 + e)}$

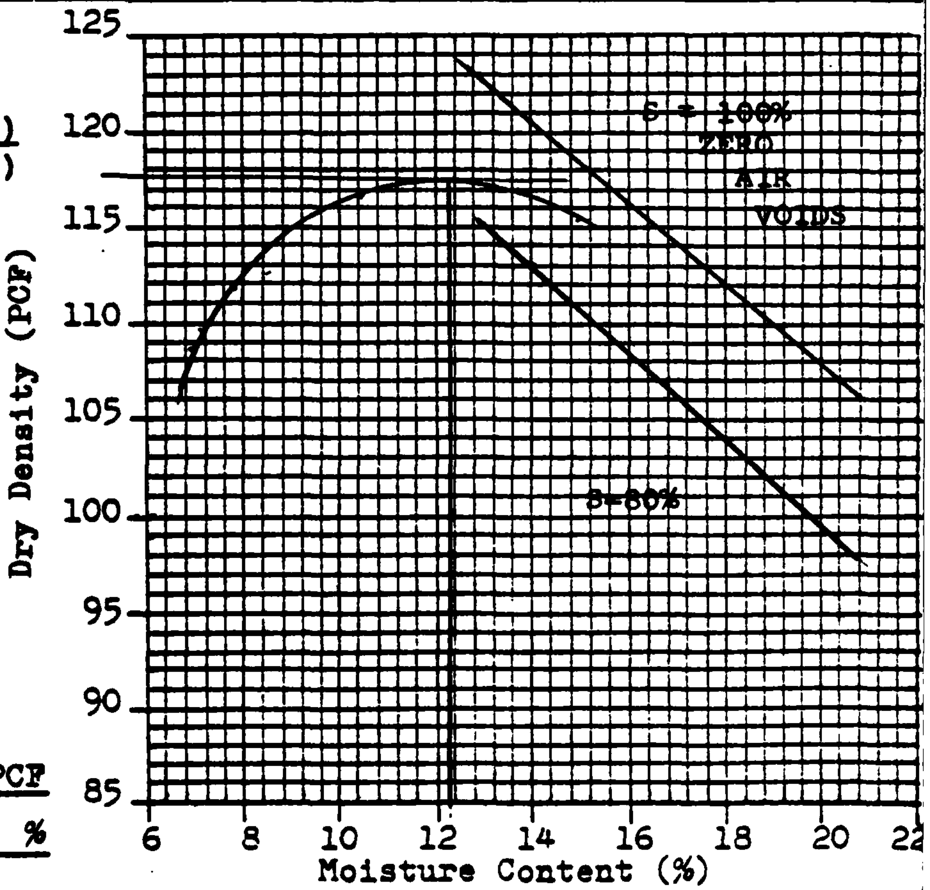
$Se = wG$

S = 80%	
w	γ_d
%	PCF
14	113.0
16	108.1
20	99.5

$\gamma_d \rightarrow \dfrac{(165.36)}{\left[1 + \dfrac{wG}{S}\right]}$

Max Dry Density 117.5 PCF

Optimum Moisture 12.3 %



EXISTING DRAINAGE CONDITIONS

Recall from chapter two in the section on drainage and drainage structures, p. 71, that "adequate drainage of the project site is a must for erosion control as well as for the protection of the project investment." (Considering the TRESTLE cost about 60 million dollars, I'd say that's a worthwhile investment to protect.) Chapter two also generally defined drainage as the process of controlling and removing excess surface and underground water encountered within the limits of the project site.

A brief look at Figure 4.8 and Drawing No. 5.1 indicates that someone gave a good deal of thought about the direction of drainage, where to discharge the excess, flow avenues and channels; ditch, pipe, and chute carrying capacities, and drainage in general. The drainage system appears to be a good design--except for one thing. The effects of accelerated erosion were apparently not given due consideration. I feel certain that the system worked very well perhaps through the first rainy season and maybe longer. However, the TRESTLE soil is highly erodible and I'm just as certain that it didn't take very many "heavy" rains (greater than one inch) before erosion completely destroyed the initial drainage design. In the interim, nature has "re-designed" the drainage system to suit her. At present, water is flowing pretty much where it wants to flow (if it flows at all). The new direction of flow is undesirable and is creating more havoc with increased accelerated erosion in the bowl area, levying a greater performance demand on the pumps at the base of the bowl.

Referring again to Drawing No. 5.1, runoff from the northwest section just below the TOW-WAY is supposed to drain to the west side of the access road through the two pipe culverts then run alongside the road through two more pipe culverts into a drainage chute to be discharged into the arroyo. However, at present that drainage path does not exist. The riprap ditch checks along both sides of the access road are no longer there. They were completely covered with eroded soil and severely deteriorated over the years by erosion and maintenance attempts to reshape the ditch lines.

Additionally, all four pipe culverts are almost full with eroded soil. Practically zero flow is getting through. Also, the three concrete open drainage channels which are supposed to drain to the west side of the road contain five to eight inches of eroded soil. Finally, the open channel drainage chute completely failed and is presently nonexistant, see Figures 1.4 and 1.5, page 17. In short, the runoff in this sector cannot be flowing into the arroyo, most of it must be draining into the TRESTLE bowl which accounts for the

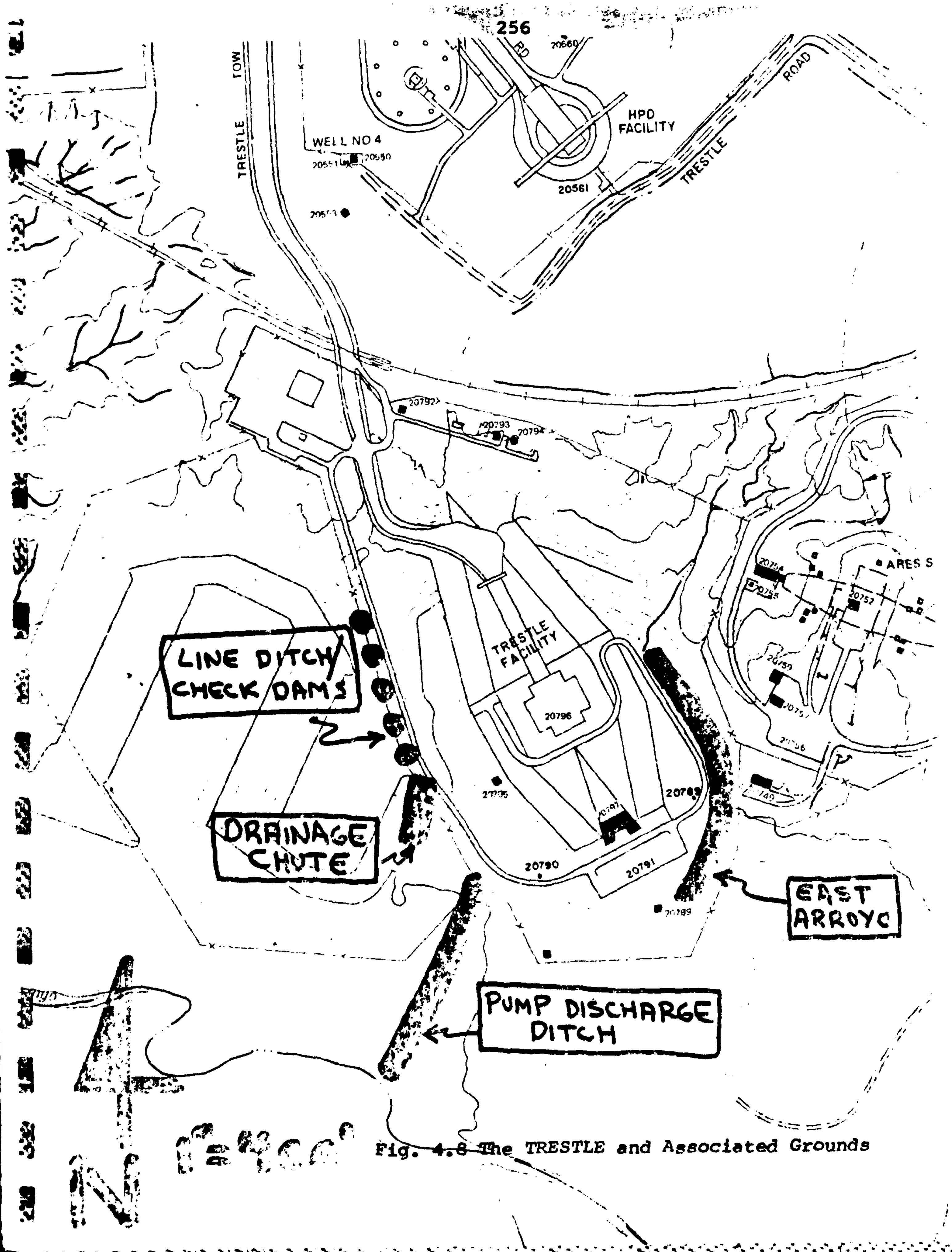


Fig. 4.8 The TRESTLE and Associated Grounds

heavily soil laden drainage channels.

On the northeast side of the TRESTLE, drainage in the top four concrete ditches is intended to flow directly into the arroyo snaking around the outside perimeter of the facility and eventually joins other runoff in the larger arroyo to the west of the site. There were also rip-rap ditch checks in the arroyo on the northeast side of the TRESTLE (not shown on drawings), but those two have been destroyed. After an unusually heavy rain, erosion caused the east arroyo to dam up. Consequently, flow in the top four ditches reversed, the pumps and catch basins could not handle the increase and the depth of water rose to about 18 inches in the TRESTLE bowl.

The northwest and northeast sectors are the two worst drainage areas at the site with the northwest sector being most critical. Both these areas require immediate attention to help alleviate the erosion problems. The open channel gunite rundowns on the southern exterior of the TRESTLE bowl shown on Drawing No. 5.1 will eventually create additional problems. As stated earlier, the one in the southwest sector (G-1) has already been washed out and G-6 in the southeast sector is about ready to collapse. Soil is being washed out from under and along the sides of the rundown. Also, a large structural crack has developed at the top allowing water free access to the bearing strata beneath. All the rundowns appear to be much too steep and too shallow permitting fast moving water to jump out of the chutes and erode the slopes. Also, there is nothing to dissipate the energy along the chute or at the outlets. Chute number G-7 on the inside east slope of the bowl is another one which shows signs of accelerated deterioration.

The intended main flow in the TRESTLE bowl can be surmised with a quick glance at Drawing 5.1. Were it not for the erosion problems, the design would be more than adequate to handle the necessary flow. The following photographs provide a much better understanding of the drainage pattern and associated erosion problems than mere words can do.

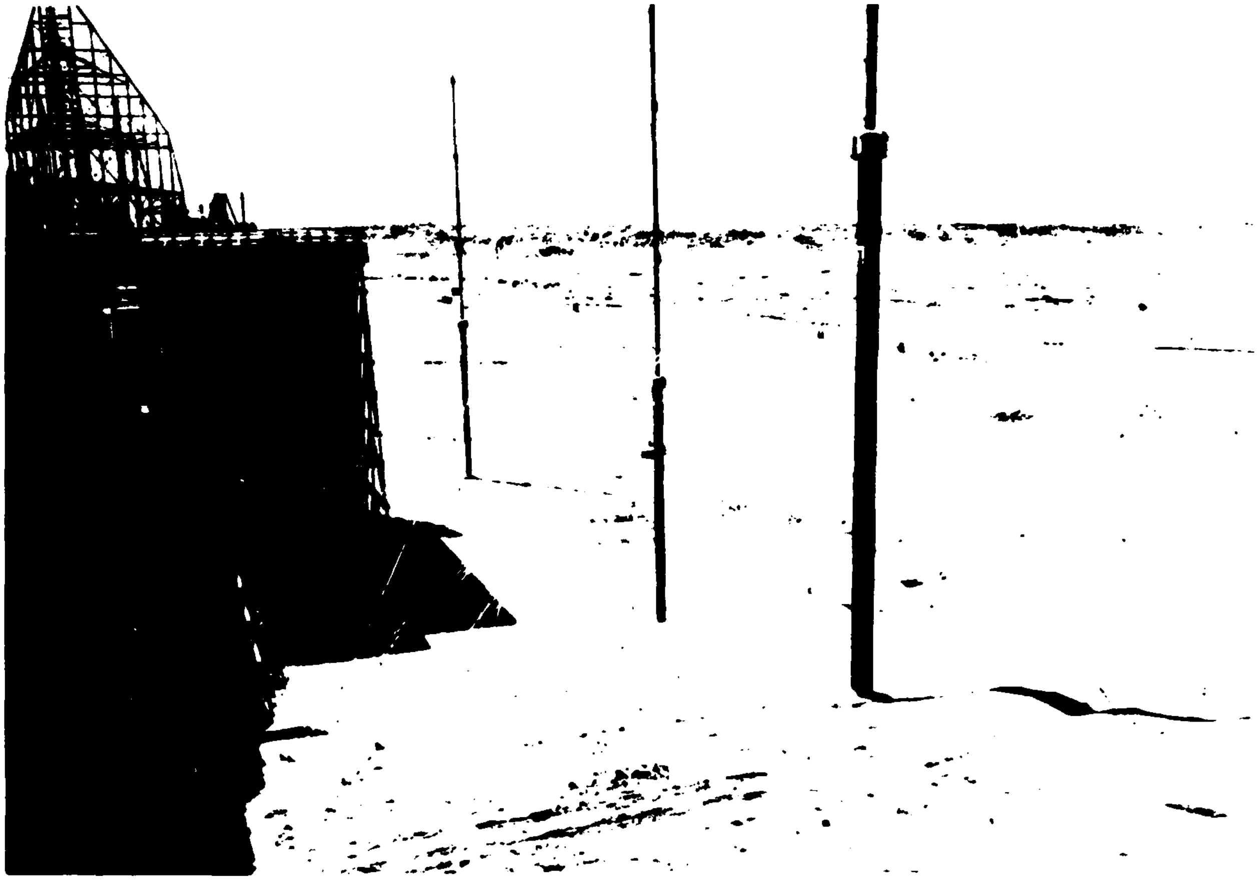


Fig. 4.9 The west side of the TRESTLE and bowl, looking south. Water flows to the right and south to be discharged through the pumps shown in the photo on the south side of the bottom of the Bowl. Note the access road on the high elevation. Also, rill erosion is very evident.

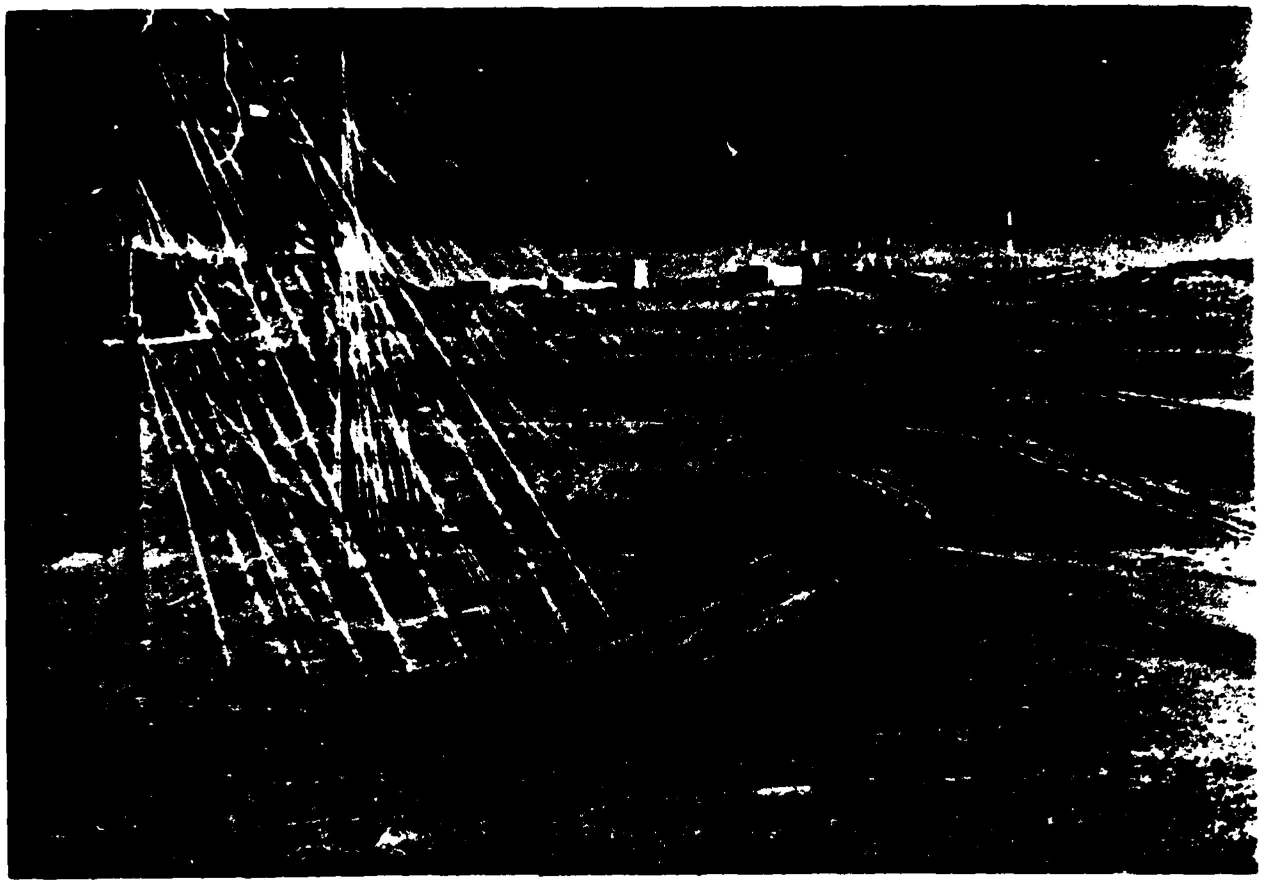


Fig. 4.10. The east side of the TRESTLE and bowl. looking north. Maintenance personnel have done some tillage on the upper slopes at the bottom of the photo. According to site personnel this helped alleviate some of the problems. Catch basins can be seen in the base of the bowl. These drain to the west flowing into the pump station.

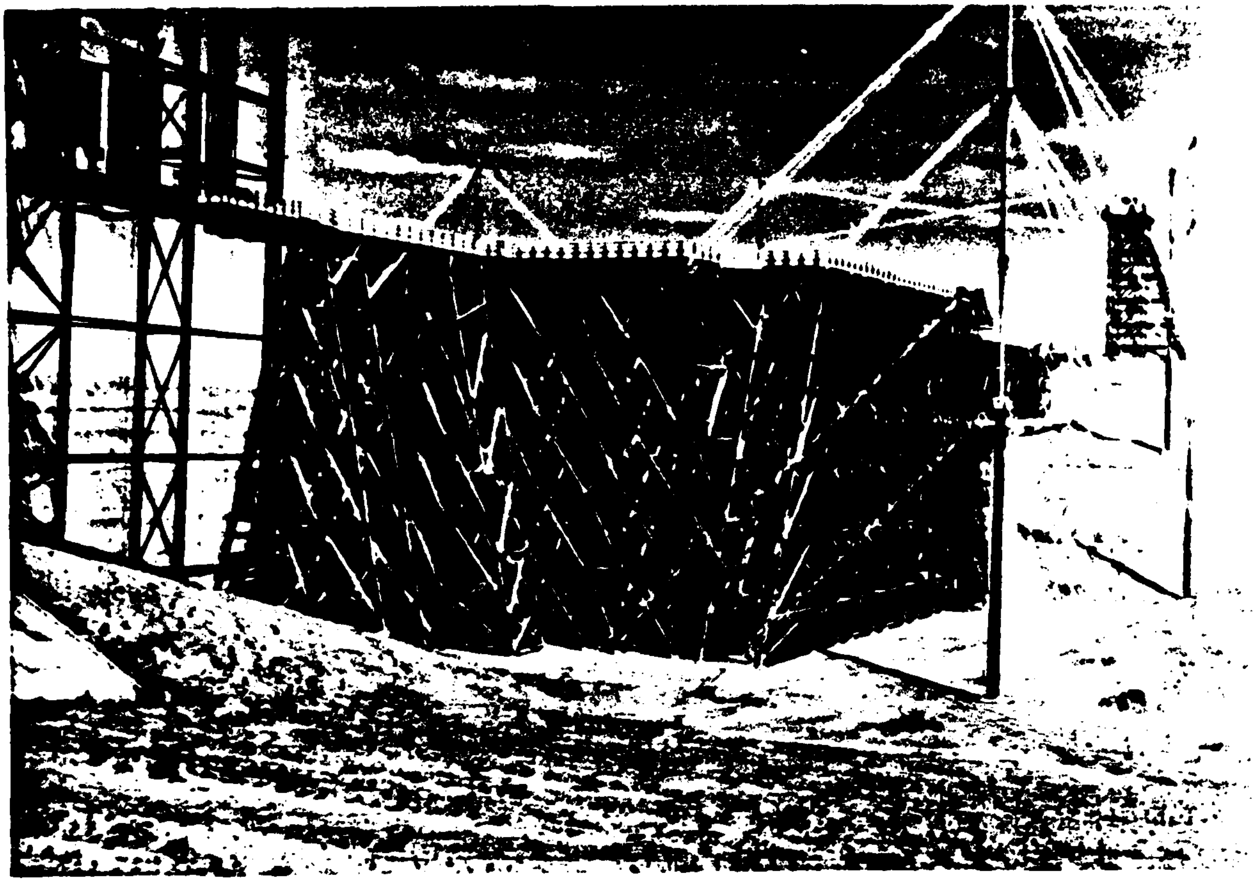


Fig. 4.11. The east side of the TRESTLE. Looking northwest..

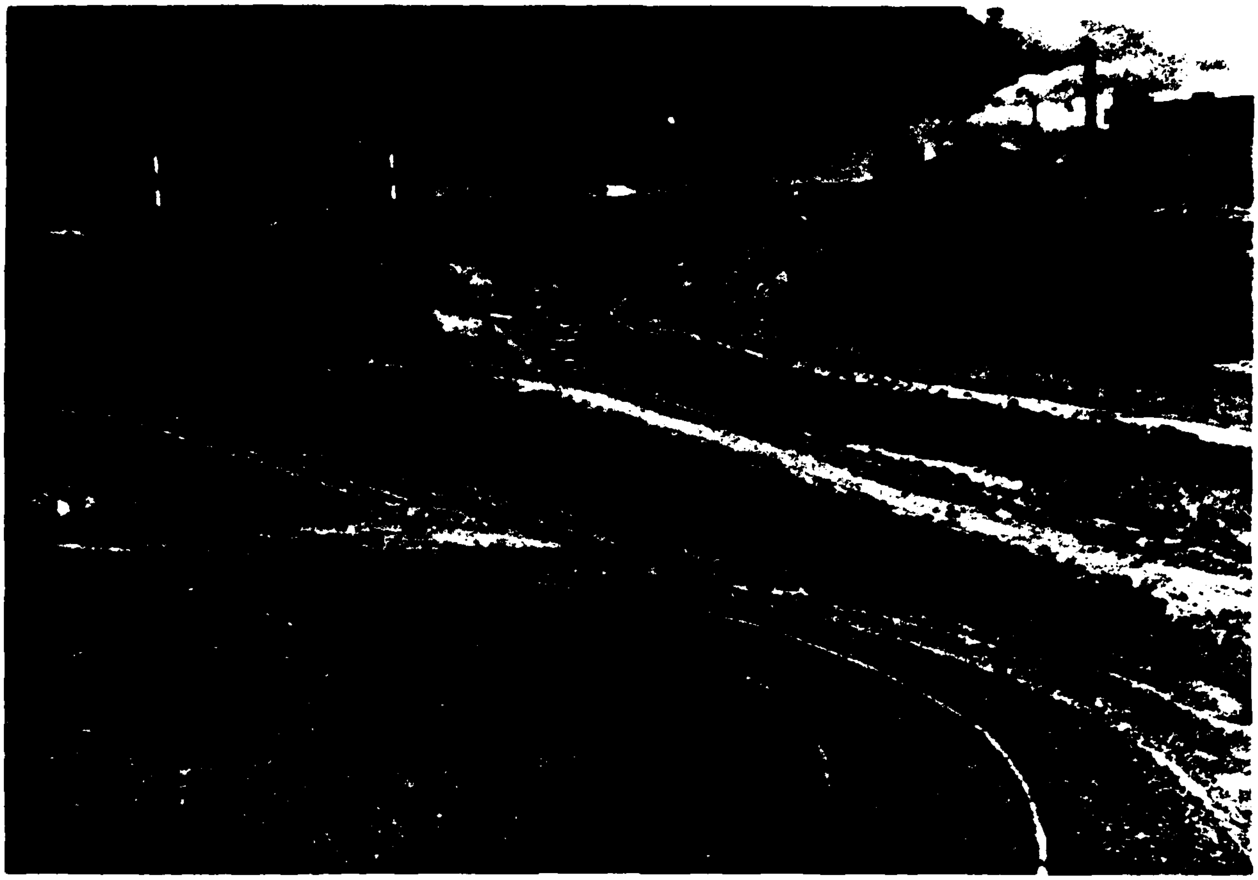


Fig. 4.12. The access road leading to the TRESTLE Bowl and the five concrete drainage ditches on the slopes in the background. The top four are designed to flow into the east arroyo. The bottom one flows into a catch basin in the bowl and finally to the pump station. You can see evidence of where the riprap ditch checks were located in the earth channel to the right.

Fig. 4.13. Outside the TRESTLE bowl. East side looking east. Most of the flow on the east slopes is intended to flow through here. At one time, the channel walls eroded and formed a dam. The water then backed up into the TRESTLE bowl. Although the ditch was regraded, there is still evidence of accelerated erosion. See. Fig. 4.14.

Fig. 4.14. Outside the TRESTLE bowl. Looking southeast. This is where flow reversed and dumped into the bowl area. See Fig. 4.15.

Fig. 4.15. East side of the TRESTLE looking north. Provides a better picture of how flow reversed and dumped into the bowl. See Fig. 4.14.

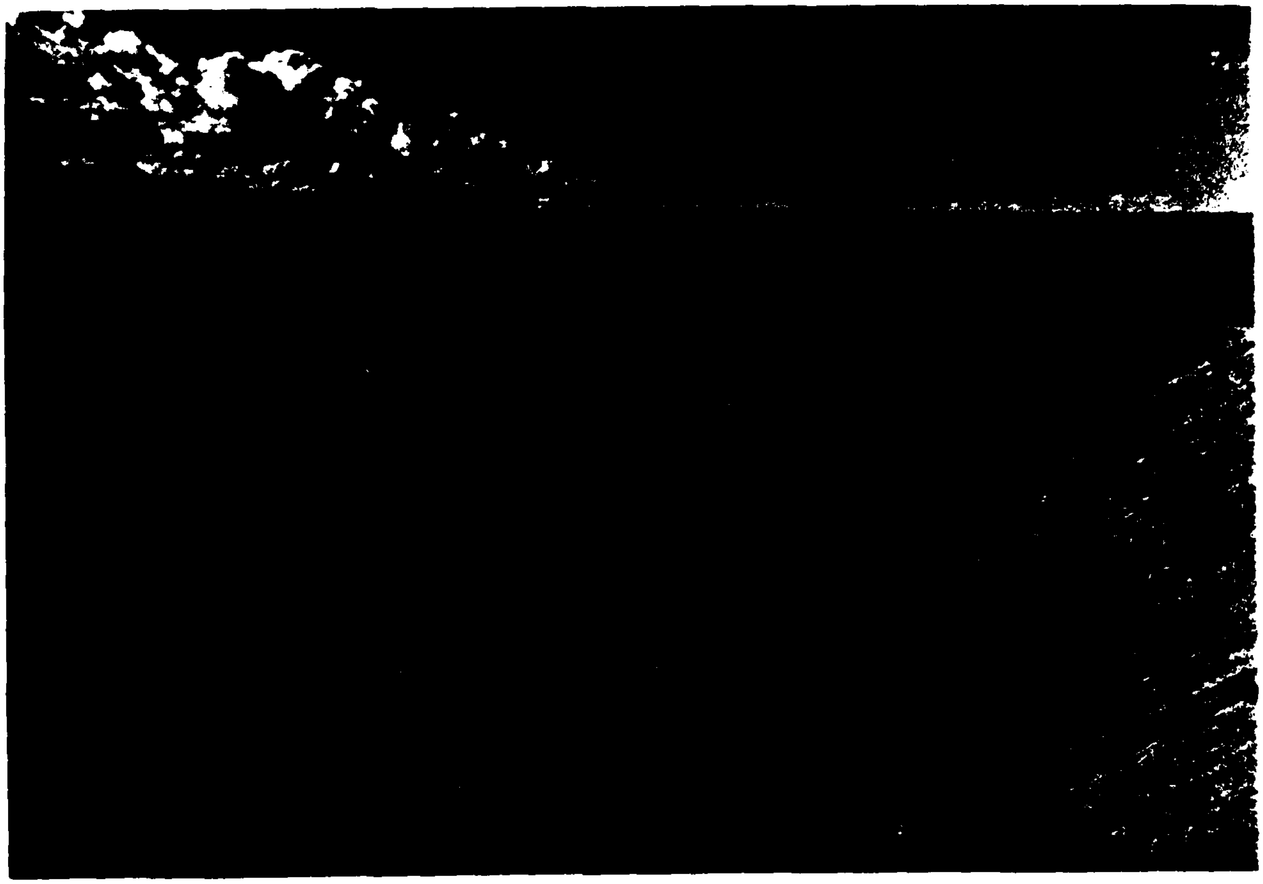


Fig. 4.16. Outside the TRESTLE bowl. East side looking south. Continuation of ditch line shown in Fig. 4.13 and 4.14.

Fig. 4.17. East arroyo. Outside the bowl looking southwest. As you can see, once outside the TRESTLE site, water flow assumes a natural course.

SOIL LOSS COMPUTATIONS

Since efficient operation of the sump pumps is critical to adequate drainage of the site, the amount of soil migrating toward the pumps is a prime concern. Most of the eroding soil above 5250 feet elevation gets trapped in one of the concrete drainage channels. The area that feeds runoff to the pumps, associated catch basins and drain lines is predominantly below 5250 feet (Reference drawing 5.1). This encompasses the basin of the TRESTLE bowl measuring approximately 440' x 660' = 290,400 SF or 6.7 acres. If the flat area of the bowl is discounted that leaves about three acres of sloped land feeding into the drainage system. The average slope of this area is approximately 35° and the length about 40'. Therefore the topographic factor, LS, from Fig. 1.11 is assumed to be 6. The erosion index R, is 13 for a single storm and 60 for the average annual, Figures 1.8 and 1.6 respectively. The soil erodibility factor K, is 0.25 from Figure 1.10 and the cropping management factor, C, is 1.0 for fallow ground. The erosion control practice factor, P, is assumed to be 0.95 from Table 1.7.

Reference Chapter One, Predicting Erosion Rates, page 19, the estimated soil loss, X, in tons per acre from a given storm period is computed as follows:

X = RKSLCP = tons/acre/year

R = 13

K = 0.25

SL = 6

C = 1.0

P = 0.95

X = 13 x 0.25 x 6 x 1 x 0.95

X = 18.5 tons/acre/year

Multiplying by the three acres to account for the lower most embankment area gives an estimated 55.5 tons of soil per year migrating towards the basin's drainage system and sump pumps. Recognizing that this number may be a gross approximation and only a small portion actually ends up in the drainage system, the number is still large enough to cause concern.

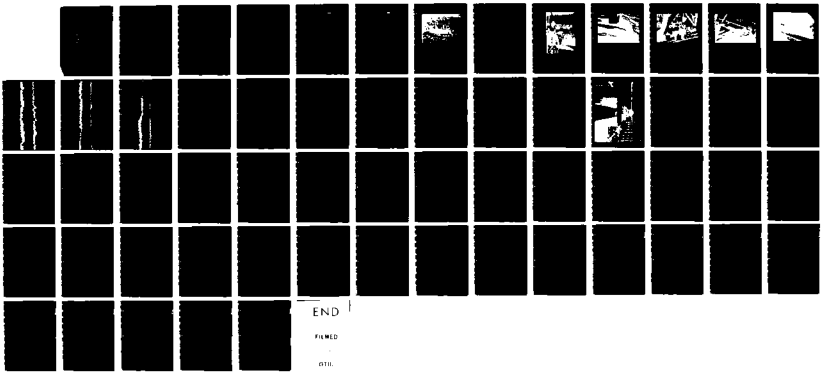
Consider, also, that according to Base Civil Engineering (equipment shop) in Oct. 83, they hauled about fifty trucks of soil to the TRESTLE. If the truck capacity is 3 cy., that equates to about 170 tons of soil being hauled; soil used to replace eroded soil. My question is where did all that eroded soil go? One look at the TRESTLE tells you the only avenue is down.

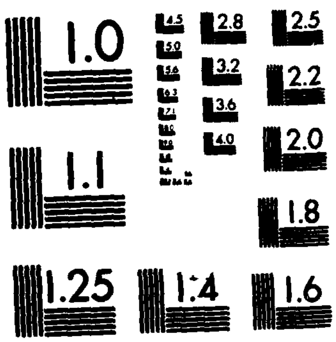
In 1981, the sump pumps became clogged and had to be completely removed and rebuilt. Eroded soil was the primary cause of the pump damage according to TRESTLE maintenance personnel.

I only bring these points up for the benefit of the sceptics who find it difficult to believe that the TRESTLE is losing that much soil. The following

AD-A143 236   EROSION CONTROL AT TRESTLE AN ENGINEERING STUDY CE 551   4/4
(U) AIR FORCE INST OF TECH WRIGHT-PATTERSON AFB OH
M W KOCH MAY 84 AFIT/CI/NR-84-20T

UNCLASSIFIED   F/G 13/2   NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

photographs provides a better indication of the effects of rainfall erosion on the facility. (Figs. 4.19-4.25)

The impact wind erosion has on the TRESTLE is much more difficult to quantify than rainfall erosion effects. As explained in Chapter One, the wind erosion equation serves questionable purpose when applied to areas with steep slopes. Additionally, the shape of the bowl itself and the erratic behavior of the wind makes the computation even more difficult. Figure 4.26 gives some idea of wind variation during a 10 hour period in April 1984. In early evening, there were 30 mph winds with gusts as high as 50 mph; by 2 A.M. the next morning, winds were almost zero. Suffice it to say the wind does have some impact on soil erosion at the TRESTLE. In a crude attempt to measure to what degree, I fashioned a field test using a 0.25 SF area in the southeast and southwest sectors of the TRESTLE basin.

These test locations are indicated on Drawing No. 5.1. I dug a 3 ft. hole at each location then inserted a plastic bag in each hole. A 6 x6 inch wood plate with a 4 inch diameter hole in its center held the bag in place. The wood plates were placed so that only wind deposited soil could get into the bags. The bags were left in place for 29 days.

When retrieved on the 29th day, there was no evidence of the adjacent ground being disturbed by animal or person. Everything looked exactly as I recalled.

During the 29 days, it rained several times so each bag contained an appreciable amount of water. Luckily, they did not leak. The samples were taken to the lab, the exterior of the bags carefully cleaned, and the contents were washed into sample containers. The samples were oven dried then weighed. The east side test yielded 164.65 grams of soil and the west, 11.06 grams. Results of an "unwashed" fine sieve analysis are provided in Tables 4.30 and 4.31. Notice most of the material was less than 0.250mm diameter.

Assuming all the soil collected resulted from wind erosion, then the estimated soil loss on the east side:

$$X = \frac{164.65g}{0.25\ SF} = 658.6\ \frac{g}{SF} = 1.45\ \frac{lbs.}{SF}$$

$$1.45\ \frac{lbs.}{SF} \times \frac{43,560SF}{acre} \times \frac{TON}{2000\ lbs.} = 31.6\ \frac{Ton}{acre}$$

On the west side:

$$X = \frac{11.06}{0.25} \times \frac{1}{453.6} \times 43,560 \times \frac{1}{2000} = 2.1\ \frac{Ton}{acre}$$

These losses are for the 29 day test period. Wind losses for the year would, of course, be much higher. Also, the test period ran from 14 March to 13 April--the windy season. Consequently, greater wind erosion losses should be expected. The reason for the large disparity between the two tests may be due to the difference in topography. The east side test location was in a much more open area; the west side was protected by steep embankments. Of course, these results cannot be considered valid without additional testing but it does give some idea of the magnitude of soil loss from wind erosion. Also, much of the of soil collected was less than 0.1 mm size--suspension category (reference Table 1.8, page 43). Most of the remainder falls in the saltation category (0.1 to 0.5 mm). Wind erosion by surface creep was not accounted for in these tests. Soil in this size range (0.5-1.0 mm) would have simply fell into the hole outside the bag.

Table 4.30

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Coarse & Fine Sieve Analysis of Wind Deposited Material
Test No. ______

Tested By: Koch

[redacted] Wind Erosion West Side of TRESTLE

Sample: KAFB TRESTLE Albuquerque, N. M.

Date: 20 April 84

Coarse Sieve Analysis

W_1 ______

Sieve Size	Opening Size	Retained Weight	Retained % of W_1	Cumulative % of W_1 Retained	Cumulative % of W_1 Passing
	mm	grams	%	%	%
2"	50.8				
1.5"	38.1				
1"	25.4				
3/4"	19.1				
3/8"	9.52				
No. 4	4.76				
No. 10	2.00				
Total					

Fine Sieve Analysis
(Sample not washed through No. 200 sieve)

W_3 11.06 g

Sieve No.	Opening Size	Retained Weight	Retained % of W_3	Retained % of W_1	Cumulative % of W_1 Retained	Cumulative % of W_1 Passing
	mm	grams	%	%	%	%
10	2.00	0.02	0.2	Mostly		
20	0.840	0.12	1.1	plant		
40	0.420	0.58	5.2	residue		
60	0.250	1.16	10.5			
140	0.105	5.57	50.4			
200	0.074	1.01	9.1			
-200	Pan	2.60	23.5			

Table 4.31 271D

University of New Mexico
Department of Civil Engineering
Soil Mechanics Laboratory

Coarse & Fine Sieve Analysis
of Wind Deposited Material
Test No. ______

Tested By Koch

Wind Erosion
[redacted] East Side of TRESTLE

Sample KAFB TRESTLE
Albuquerque, N. M.

Date 20 April 84

Coarse Sieve Analysis

W_1 ______

Sieve Size	Opening Size	Retained Weight	Retained % of W_1	Cumulative % of W_1 Retained	Cumulative % of W_1 Passing
	mm	grams	%	%	%
2"	50.8				
1.5"	38.1				
1"	25.4				
3/4"	19.1				
3/8"	9.52				
No. 4	4.76				
No. 10	2.00				
Total					

Fine Sieve Analysis
(Sample not washed through No. 200 sieve)

W_3 164.65

Sieve No.	Opening Size	Retained Weight	Retained % of W_3	Retained % of W_1	Cumulative % of W_1 Retained	Cumulative % of W_1 Passing
	mm	grams	%	%	%	%
10	2.00	0.83	0.5	Mostly		
20	0.840	8.46	5.1	plant		
40	0.420	8.03	4.9	residue		
60	0.250	8.60	5.2			
140	0.105	77.39	47.0			
200	0.074	25.44	15.5			
-200	Pan	35.90	21.8			

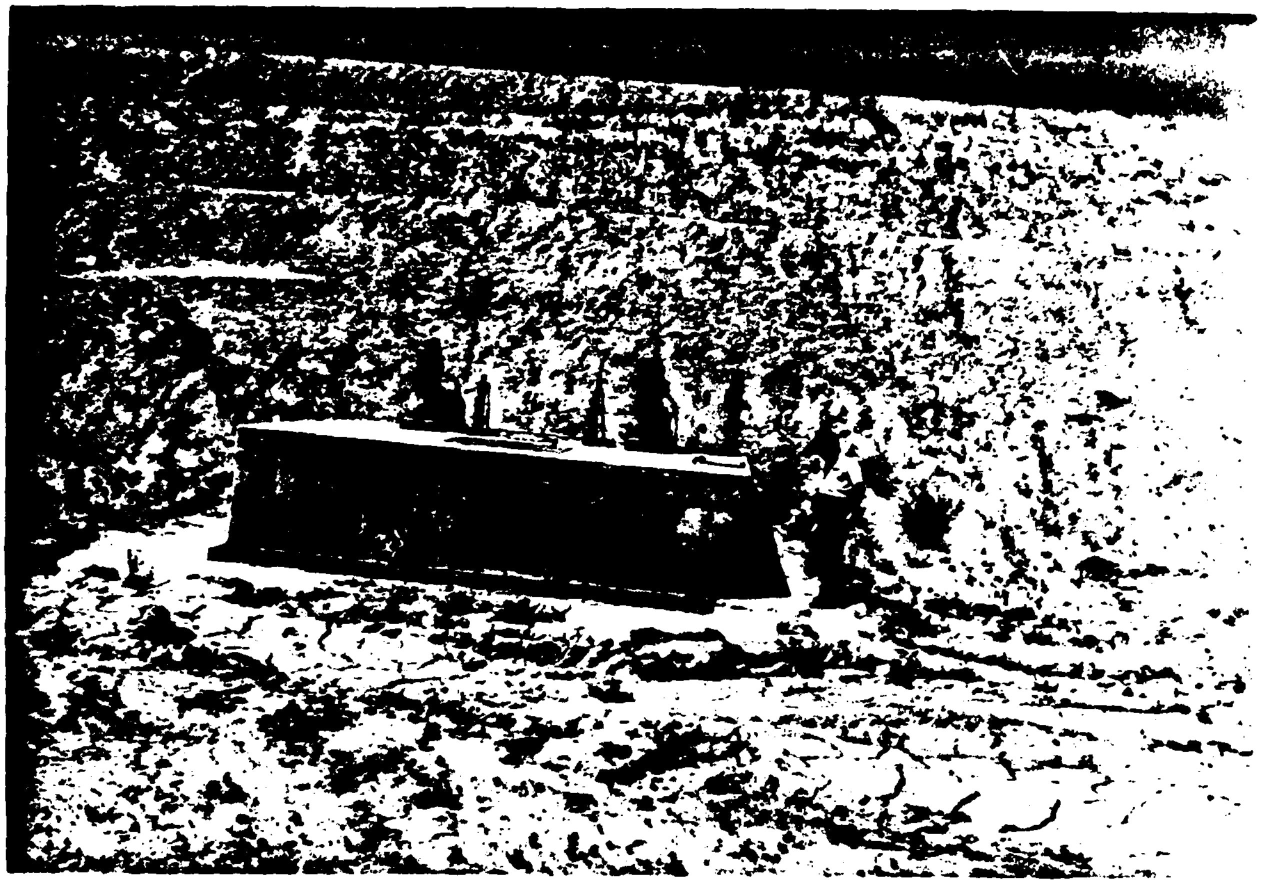


Fig. 4.19. The TRESTLE basin, southwest sector. This gives some idea of the magnitude of the volume of soil migrating towards the sump pumps. More than 50 tons per year is estimated using the Universal Soil Loss Equation. Note the metal dam checks surrounding the grill work over the settling tank in front of the pumps. Also, a sizeable access port was cut into the top of the pump well housing to facilitate cleaning the well. (See sketch, Fig. 4.20).

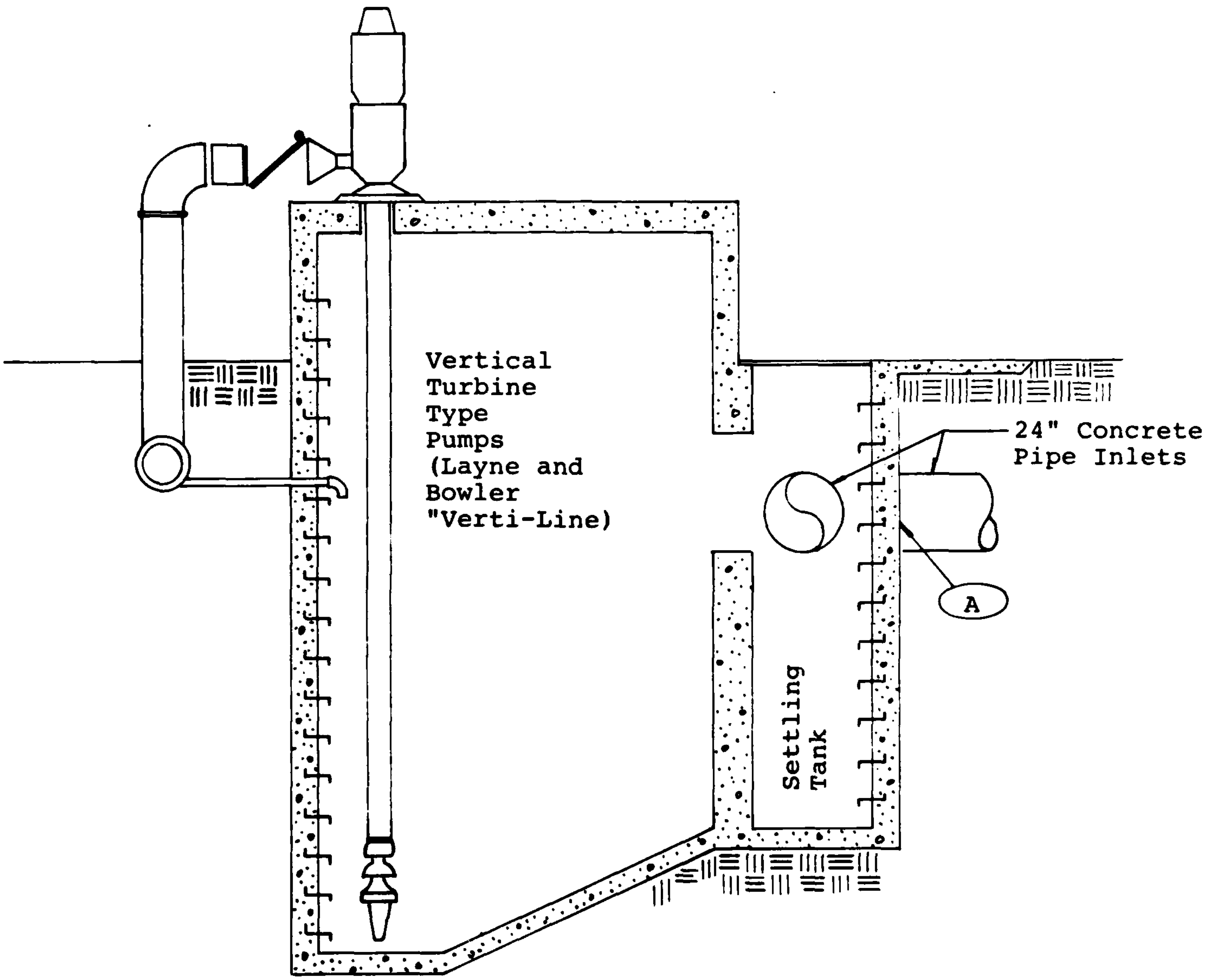


Fig. 4.20 STORM DRAINAGE PUMP & TANK SECTION
Scale: ¼"=1'-0"

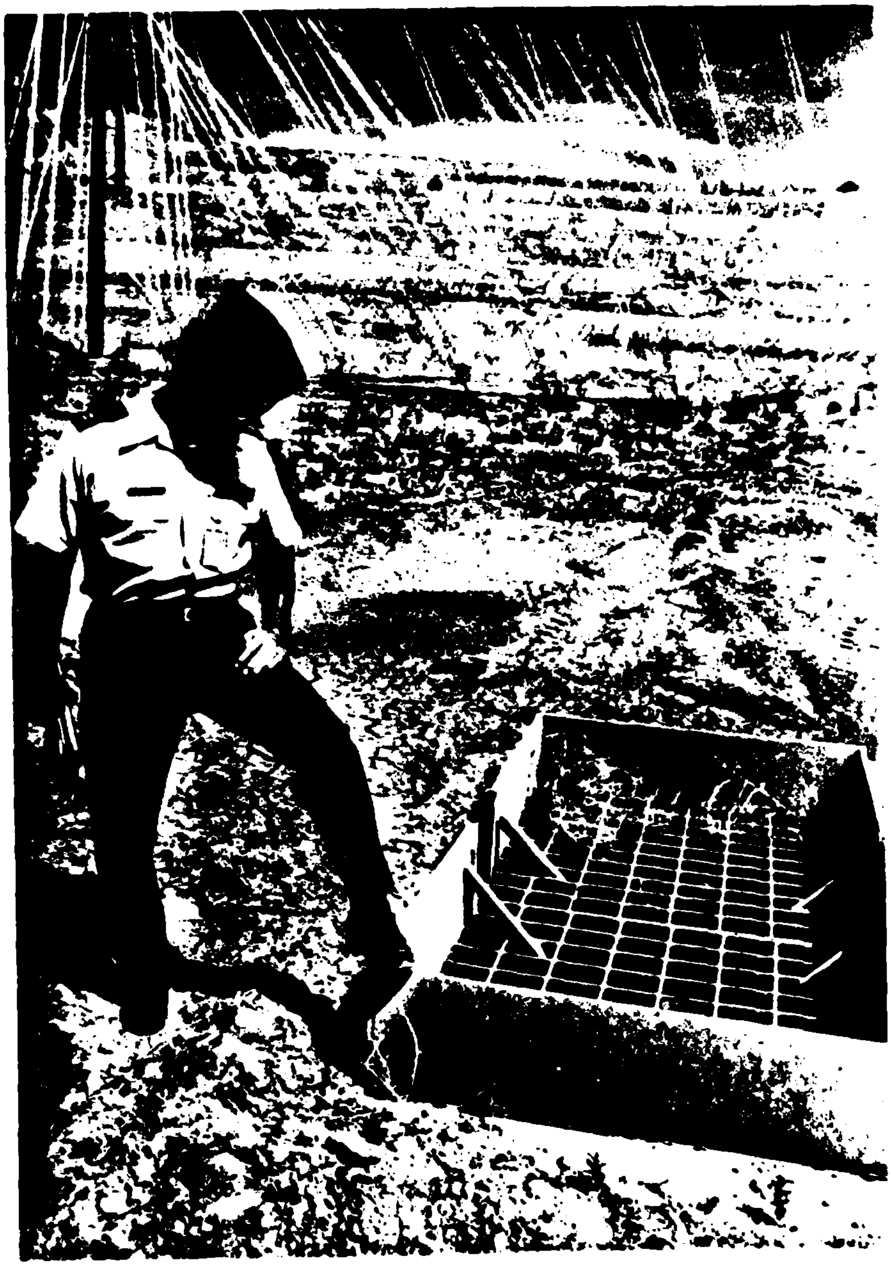


Fig. 4.21. Catch basin typical of the eight located in the TRESTLE bowl. The metal sides were installed to help check the amount of soil getting into the drainage system.

Fig. 4.23. Erosion beginning about the TRESTLE base supports (typical).

Fig. 4.22. Rill erosion at the base of the structure. This particular one is about 18 inches deep (The Captain's left leg is standing in it.). These sire rills can be found throughout the facility.

Fig. 4.24. Erosion under the North side of the structure. Sandbags were used in an attempt to help arrest the erosion process (Also see Fig. 4.25).

Fig. 4.25. Another perspective of erosion under the TRESTLE (See Fig. 4.24).

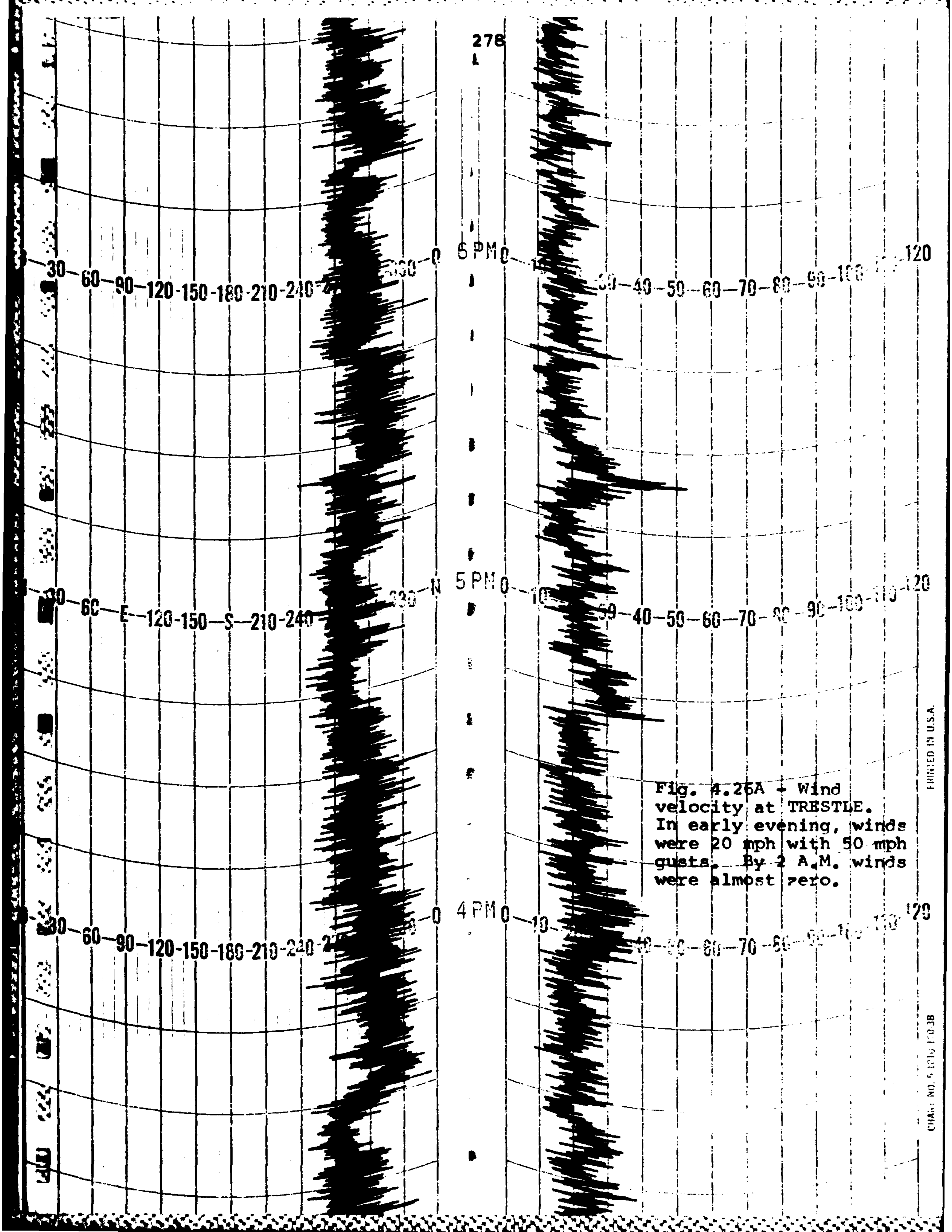


Fig. 4.26A - Wind velocity at TRESTLE. In early evening, winds were 20 mph with 50 mph gusts. By 2 A.M. winds were almost zero.

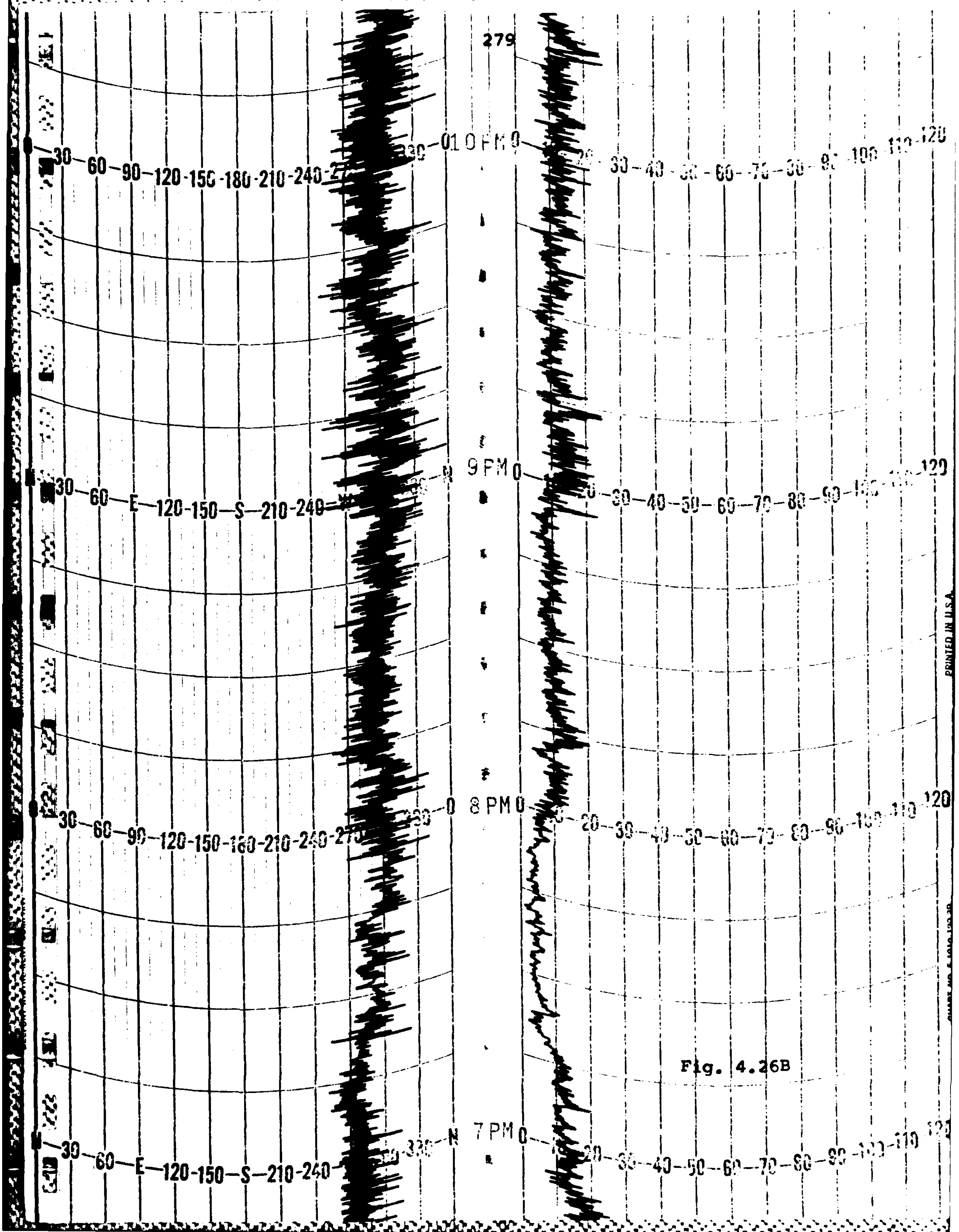

10 PM
9 PM
8 PM
7 PM
30—60—E—120-150—S—210-240
30-60-90-120-150-180-210-240-270
0-10-20-30-40-50-60-70-80-90-100-110-120
PRINTED IN U.S.A.

Fig. 4.26B

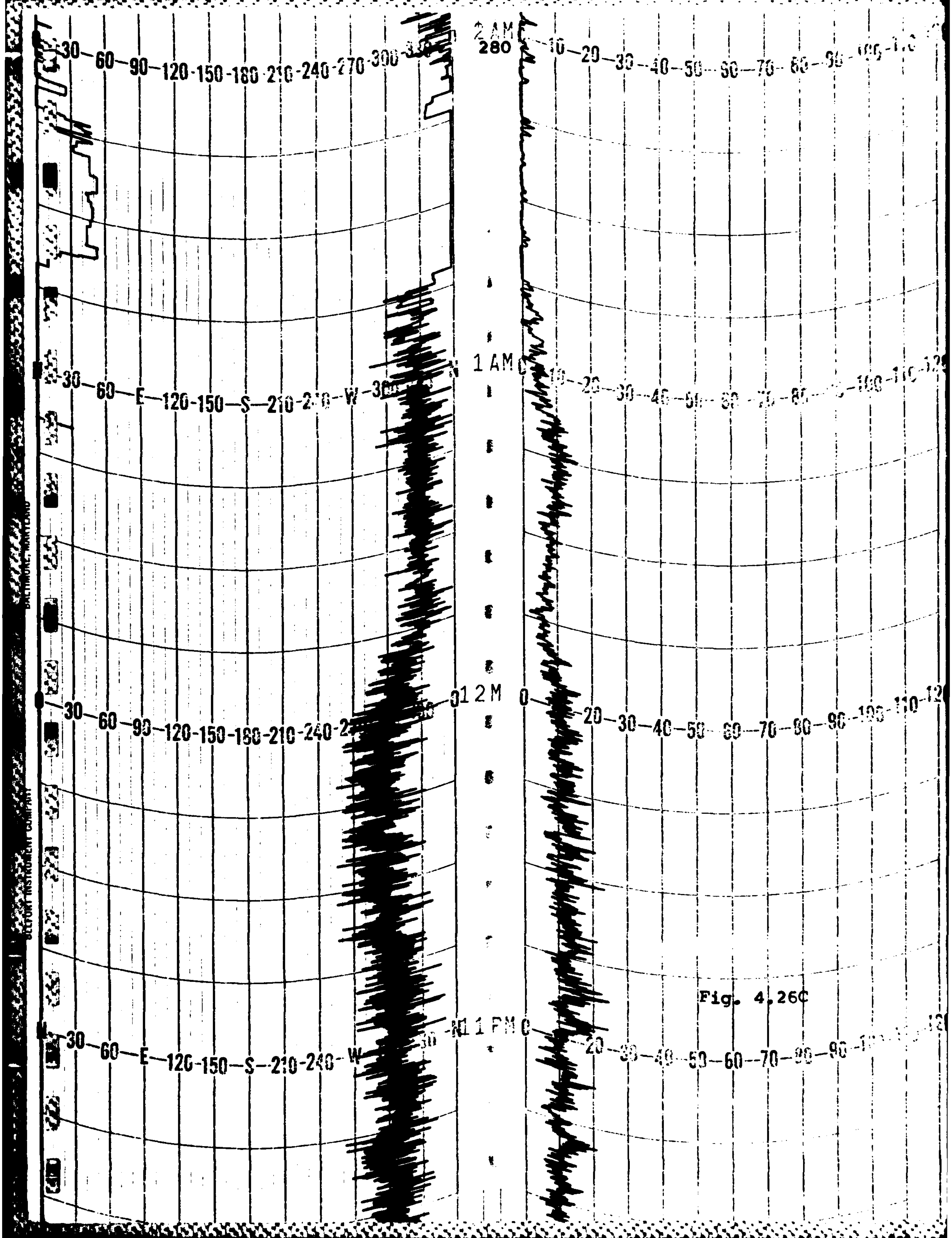

2 AM
280
1 AM
12 M
1 PM
BELFORT INSTRUMENT COMPANY
BALTIMORE, MARYLAND

Fig. 4.26C

EROSION CONTROL INITIATIVES - PAST AND PRESENT

As explained in the introduction to Chapter 4, TRESTLE design began in 1973 and construction of the wood structure started in 1976. Completion of the earth work was somewhere in between, about 1974-1975. I was first introduced to the facility in 1977 when assigned to the Kirtland Base Civil Engineer. I was told in no uncertain terms to "go fix it." Obviously, it never got "fixed" and the problem has been kicked around, passed on down the line, forgotten, and then remembered after the first heavy seasonal rain. It seems that everyone has recognized the TRESTLE has erosion problems but it's one of those kind that always nags at you but never gets big enough to get too concerned about. However, with each passing rainy season, the problem gets bigger and bigger, and maintenance gets heavier.

As early as April 1975, the Waterways Experiment Station (WES), Corps of Engineers from Vicksburg, Miss. were applying test patches of asphalt emulsion products to the TRESTLE slopes in an attempt to pinpoint an effective soil stabilizer.

In August 1976, WES informed the TRESTLE of the test results, "All five of the test sections except the one treated with DCA-1295 and fiber glass appeared to have sustained a considerable amount of rain runoff. A lip about 9 inches high across the top of the DCA-1295 and fiber glass section, prevented any runoff over this treated area. However, prior treatment with this system at another facility indicated that the DCA-1295 and fiber glass would be effective in controlling erosion. Observation of the other four sections over which runoff occurred indicated that the SS1h asphalt emulsion and chopped fiber glass treatment was the most effective." WES also at that time provided cost estimates to treat 21 acres at the site. Using SS1h and fiber glass, the estimate given was $117,000; DCA-1295 and fiber glass - $231,000.

In November 1977, Kirtland's Air Force Weapons Lab (AFWL) authorized $8,000 for WES to apply a 3000 SF SS1h asphaltic emulsion test patch over the TRESTLE site. The patch was installed in November 1977 and was "obliterated" by a heavy rainstorm in Summer 1978. Suspected reason for failure, aside from the rain, was that the clay content of the soil prevented a good bond between asphalt and soil.

In August 1978, after the test failure, WES made the following observations and recommendations:

1. SS1h should still be regarded as an experimental means of controlling erosion, but one of the most economical ones available <u>and</u> one with a high potential. It is suggested that another test area of uncompacted soil be

prepared and the time and amount of placement of prewet water be varied up to approximately 0.1 gal. per sq. yd. The prewet water should be applied before the fiber glass reinforcing in one section and after in another. One other section should be prepared to test placing the SS1h in two applications (0.3 gsy the first application and 0.2 gsy the second application) with the fiber glass reinforcing placed between applications. All these test sections should be photographed monthly to determine the performance history.

2. Where clay areas are encountered, they should be covered with a thin layer of sand, prewet thoroughly, and treated. The sand will give the asphalt something to cling to with enough asphalt left to fasten the entire mass to the clay surface. Also, in areas where considerable foot traffic is expected, the SS1h surface may be blotted with sand as required to form a cling-free walk. In fact, this action should be encouraged because local observation reveals it helps increase the expected design life of the asphalt.

3. Finally, while it is doubtful the pelletized Pramitol harmed the erosion control films, it is suggested that this or similar materials be placed three days or more before application of the erosion control materials and the surface thoroughly wet several times. This will allow the pellets to dissolve and percolate into the soil and thereby eliminate this variable.

4. The SS1h asphalt used in this project cost $0.305 per gal. or approximately $0.15 per sq. yd. The fiber glass cost $0.52 per lb. (November 1974) or approximately $0.20 per sq. yd. The optimum crew would probably consist of one asphalt distributor operator/driver, two laborers, and one foreman/engineering technician. The technician would check application rates as necessary and point out areas for retreatment. This crew could probably treat one acre per 8 hour work day for nonreinforced, or one acre each two work days for reinforced.

The WES experiment was apparently a "last ditch" effort as far as attempts to stabilize the TRESTLE soil. To my knowledge, there was no follow-up to WES on anyone's part. I tend to agree with the reasons given for the test failures, i.e. steep slopes and clay content of the soil (lab results indicate 14 percent clay; another 6 percent and, the soil could be classed as sandy clay. Recall too from Chapter three, that fine grained soils from arid regions, being high in pH and dissolved salts,do not respond too well to asphalt stabilization. Also, asphalt adds very little strength to fine grained soils. Consequently, the test patch added its own additional weight to the steep slope but no appreciable increase in soil strength.

Since 1978, TRESTLE erosion control initiatives have been maintenance oriented; but, considering the accelerated rate of erosion, there's no way I can see that either the TRESTLE maintenance crew or the Base Civil Engineer's shops could ever keep up with it. As one TRESTLE crewman put it, "We clean it up one day and it comes right back with the next good rain." (Which might be the next day.) The same gentleman estimates that it would take two full-time workmen to maintain the TRESTLE drainage system and associated grounds. I agree wholeheartedly. At the present rate, routine maintenance is daily maintenance. Unfortunately, the manpower is not there to do it. Consequently, the site is not being well maintained in this regard. (As evidenced by the description of drainage conditions earlier in this chapter.)

That's not saying maintenance isn't being done. It's just that the present staff cannot keep up with it. For example, from January to November 1978, the Base Civil Engineer (BCE) expended approximately 6,000 man hours of plumbing, pavement, and equipment hours (all estimated man hours are unofficial). As recent as October 1983, the BCE equipment shop spent 80 hours just for earth hauling to the site. In 1982 TRESTLE maintenance people logged 800 man hours during a four week effort to clean the drainage channels, culverts, and associated grounds. Additionally, they had to rent a backhoe at $140 per day to facilitate the clean-up. In 1981, the sump pumps became completely clogged, had to be removed, rebuilt, and reinstalled.

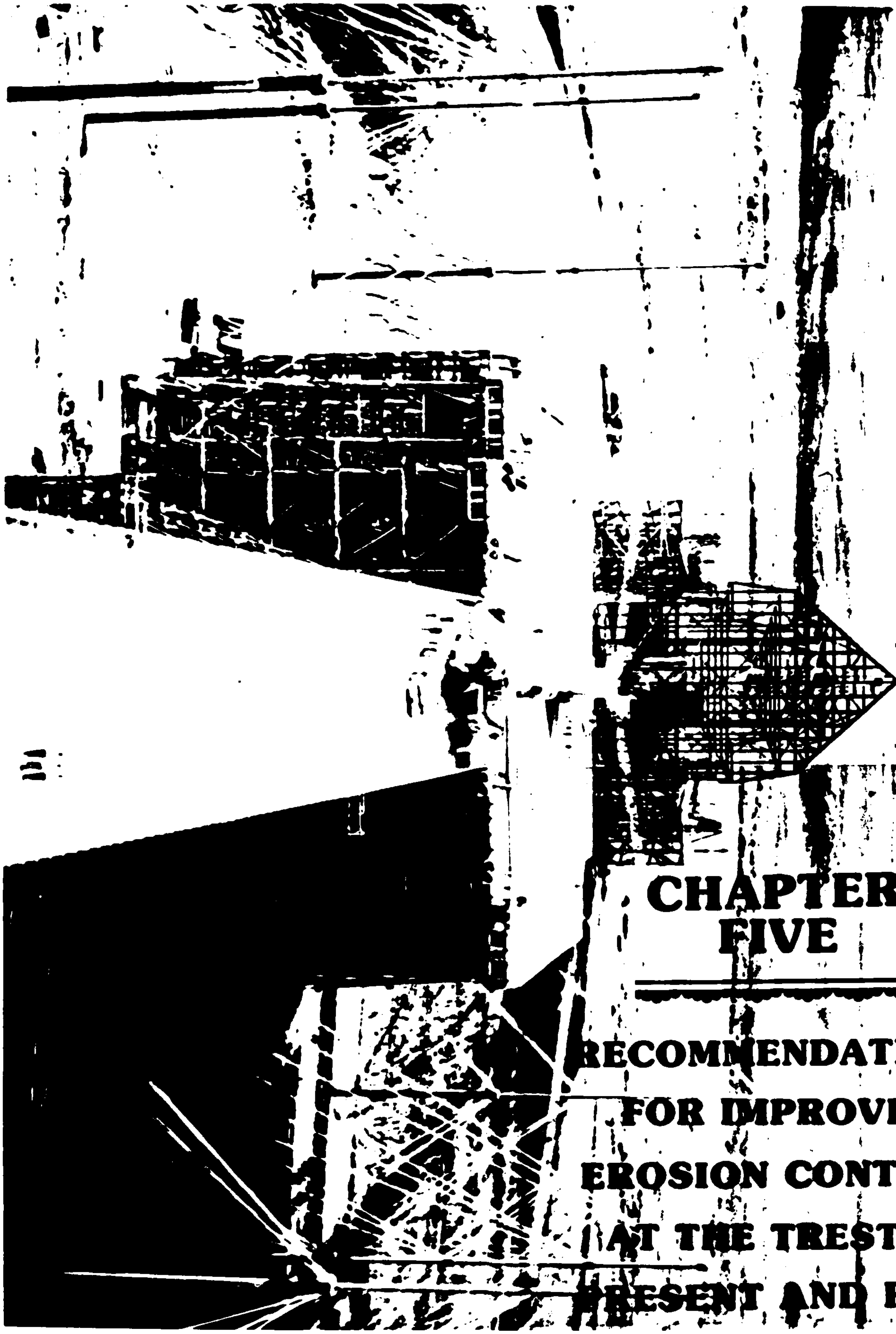
In addition to maintenance time, the BCE design section has put a considerable amount of design time as well as drafting and surveying effort into the TRESTLE erosion problems. The BCE site developers put 72 hours into a survey in October 1981. That was followed up with a survey by Scanlon and Associates of Albuquerque to the tune of $9,000. The base engineers even designed a new settling tank to handle the increased erosion. I suppose the idea was that if the ground couldn't be stabilized, then at least have something there big enough to handle ever increasing loads.

In reviewing the Base Civil Engineer's project folder "Erosion Control of TRESTLE" which dates back to the WES experiment, I have found many ideas, suggestions, recommendations, etc. on ways to correct the problem-- some good ideas and some not so good. Among them are:

1. Hydromulch
2. Hay mulch
3. Different types of grasses
4. Gravel
5. Soil cement
6. Soil asphalt
7. Fiber glass mixtures
8. Increased maintenance
9. Increase pipe size to decrease velocity
10. Increase size of settling tank
11. Provide lined ditches with check dams.

12. Line east arroyo with milled asphalt
13. Water jet the drain lines
14. Construct storm water collecting ponds within TRESTLE bowl
15. Rip-rap all erosion problem areas
16. Don't do anything
17. Use gunite
18. Install energy dissipaters

As you can see, the problem has not lacked for ideas to solve it. However, it appears we've had a difficult time putting anything into action. The reason for reviewing all these initiatives and focusing on the time and effort spent on the problem is hopefully to convince someone that now is the time to act. However, restoring the site to its original condition is not the answer, only part of it. Attention needs to be refocused on how to arrest the erosion. Once that is stopped you'll have time to go back and improve on or restore other areas. WES was on the right track; if you don't have the answer then experiment until you find one. Some of these recommended solutions are sound ones, and frankly, the list covers a lot of alternatives. In the next and final chapter, I hope to pull some of these suggestions together and recommend some viable and economically feasible means and methods to help solve the problem.



CHAPTER FIVE

RECOMMENDATIONS FOR IMPROVED EROSION CONTROL AT THE TRESTLE- PRESENT AND FUTURE

CHAPTER V

RECOMMENDATIONS FOR IMPROVED EROSION CONTROL AT THE TRESTLE - PRESENT AND FUTURE

INTRODUCTION

Chapter IV addressed the in-situ conditions at the TRESTLE focusing on the problems of accelerated erosion and existing drainage deficiencies. Those are the two main problems that must be corrected to decrease the present rate of erosion and bring it within acceptable bounds. After all the studying, reading, and research I've put into this problem, I wish I could be the one to blurt out the answer that would solve the "mystery"; an answer satisfactory to all concerned including the engineers, administrators, financial managers, scientists, "maintainers," and technicians. However, to my own disappointment, I cannot be the one. There appears to be no easy answer and, frankly, most of the recommendations I intend to make have already been made in some form or fashion at some point in time.

In my search for a solution to the problem, I have talked with numerous agencies, product manufacturers (All of which,by the way, have the product to "do the job"), engineers, and soil specialists. I have also contacted as many people I possibly could that are presently associated with, or were at one time associated with, the construction, maintenance, or operation of the TRESTLE, personnel and agencies contacted include:

1. Baum, N. P. Ph.D, KAFB Civil Engineering Research Facility, Chief Scientist.
2. Ungvarsky, J. J., KAFB Air Force Weapons Lab/ PRP.
3. Plammondon, M.A., KAFB Air Force Weapons Lab/ NTE, Chief Engineer.
4. Case, R. S. Lt. Col., Air Force Weapons Lab/ NTXP.
5. Slater, B., Former USAF Captain associated with initial construction. Now belongs to local firm (268-9920).
6. Kline, B., KAFB Air Force Weapons Lab/NTMF. (Mr. Kline was my primary contact at the TRESTLE. I coordinated efforts directly associated with the facility through him.)
7. Otto, B., TRESTLE's Facility Maintenance Lead Technician.
8. Johnson, A., TRESTLE Facility Maintenance Technician.
9. Bohannon, H. C., KAFB Civil Engineer's Chief Design, Construction, and Environmental Planning, 1606 AEW/DEE.
10. Sotelo, R., KAFB Civil Engineer's Chief of Design, 1606 AEW/DEEE.

11. U.S. Army Corps. of Engineers, Albuquerque, New Mexico.
12. U.S. Army Corps. of Engineers, Waterways Experiment Station, Vicksburg, Miss.
13. Soil Conservation Service (Farmer, R.), Albuquerque, New Mexico.
14. American Enka Company, Enka, North Carolina.
15. U.S. Department of Commerce, National Oceanic and Atmospheric Administration.
16. Soil Conservation Society of America, Ankeny, Iowa.
17. Portland Cement Association, Skokie, Illinois.
18. KAFB Office of Public Affairs, AF Contract Management Division.
19. USAF Civil Engineering Research Center, Tyndall AFB, Florida.
20. American Excelsior Company, Albuquerque, New Mexico.

There were numerous others, but these were my primary contacts for acquiring information and also to ensure coordination with appropriate offices. In my coordination efforts, I was given two project constraints. Dr. Baum emphasized there should be very little, if any, metal placed directly beneath the TRESTLE structure; Mr. Kline explained that due to current fiscal restraints, the project scope should not exceed $50,000.

Money is the key limitation (as is nearly always the case where maintenance is concerned). Consequently, I divided the TRESTLE site into seven zones which contribute the most to accelerated erosion and then prioritized the zones based on relative degree of erosion impact. That is, the zone which contributes most to erosion is zone one, and so on through zone seven. Table 5.1 lists the zones and recommended treatments; discussion of each treatment is referenced by page number. Drawing No. 5.1 delineates the zones and correlating priorities identified in Table 5.1.

Table 5.1. TRESTLE Erosion Control Zones and Recommended Treatment

Zone	Description	Approx. Quantity	Recommended Treatment	Reference Page
1	Inside bowl embankments on north, west, and east side. Primarily the sloped areas adjacent to the open channel concrete drainage ditches.	8.6 acre or 375,000 SF	Scarify, fertilize, seed, and install soil retention blanket	293
2	Ditches adjacent to access road and ditch (not shown) northeast side of TRESTLE.	1840LF	Clean with auger four pipe culverts under roadway shown, clean and reshape ditch lines, install ditch checks, line ditches, seed.	297

Table 5. 1 (Continued

Zone	Description	Approx. Quantity	Recommended Treatment	Reference Page
3	Entrances and/ or exits at drainage ditches, pipe culverts, drainage chutes, and ditch checks.	40 ea.	Riprap	300
4	Gunite rundowns	10 ea.	Replace with corrugated pipe as they fail.	301
5	Inside bowl embankments on southwest and south-east sides.	2.7 acre or 118,000 SF	Scarify, fertilize, seed, and mulch, install RR ties as wind rows, water interceptors, and soil retainers (two rows on west side and three rows on east side).	302
6	Basin of bowl-excludes area under TRESTLE. Basically all flat areas on south side of access road.	2.1 acre or 91,000 SF	Regrade to level and install four inch compacted gravel surface course.	303
7	Under TRESTLE access ramp (north side) and under steel wedge shape structure (south side).	1.1 acre or 47,000 SF	Soil cement	304

Zone One

As described in Table 5.1 and shown on drawing 5.1, this area lies mostly on either side of the open channel concrete ditches. The reason this zone is designated number one priority is due to acreage and steepness of slopes. It alone probably contributes more eroded soil to the drainage system within the bowl than all the others combined. It also is the biggest headache for the maintenance crews. Consequently, it should be given priority attention. Alleviate problems here and crews will have more time to devote to other problem areas.

You may be wondering why not make drainage the number one priority; especially, considering its' very poor existing condition. If drainage is attended to first, that would essentially put the TRESTLE in the same good shape it was in before the first hard rain. However, if you get a very hard rain the day after you correct the drainage problem, you'd be well on your way back to where you are right now. I would like to emphasize that before considering techniques of soil improvement (especially where improved soil strength is the objective), it is important to realize that the simple procedure of drainage can lead to major improvements in certain soil properties. Adequate "improvement" of silts, particulary, may be achieved by drainage.

Also, if soil stabilization, as discussed in chapter three, is an alternative, recall that the stabilization technique depends on the nature of the soil. Mitchell (1976) categorizes techniques as those applicable to cohesionless soils and to cohesive soils. This is clearly illustrated in Figure 5.1. Note that bitumen stabilization is way over on the left side of the chart. The TRESTLE soil however tends toward the right. This is another indication as to why the WES experiment (chapter four) was unsuccessful. Recall too that soil stabilization techniques are, as a rule, much more expensive than vegetative or biotech systems, which is why I recommend vegetation in Zone 1. Most people are skeptical about getting anything to grow in this climate without irrigation. This was certainly the concensus shared by those who entered comments in the Base Civil Engineers' "Erosion Control - TRESTLE" design folder. However, the success that the Soil Conservation Service had with proper seeding in Black's Arroyo (reference chapter three, page 175) leads me to believe a similar approach would be successful at the TRESTLE. Recall also from chapter four that native grasses for the TRESTLE area include mesa dropseed, Indian ricegrass, giant sand dropseed, black gramma, and blue gramma. Table 5.2 provides recommended seed mix for the TRESTLE soil.

If the recommended mix in Table 5.2 cannot be obtained then increase the proportions of Indian ricegrass and giant sand dropseed accordingly. Also, a good

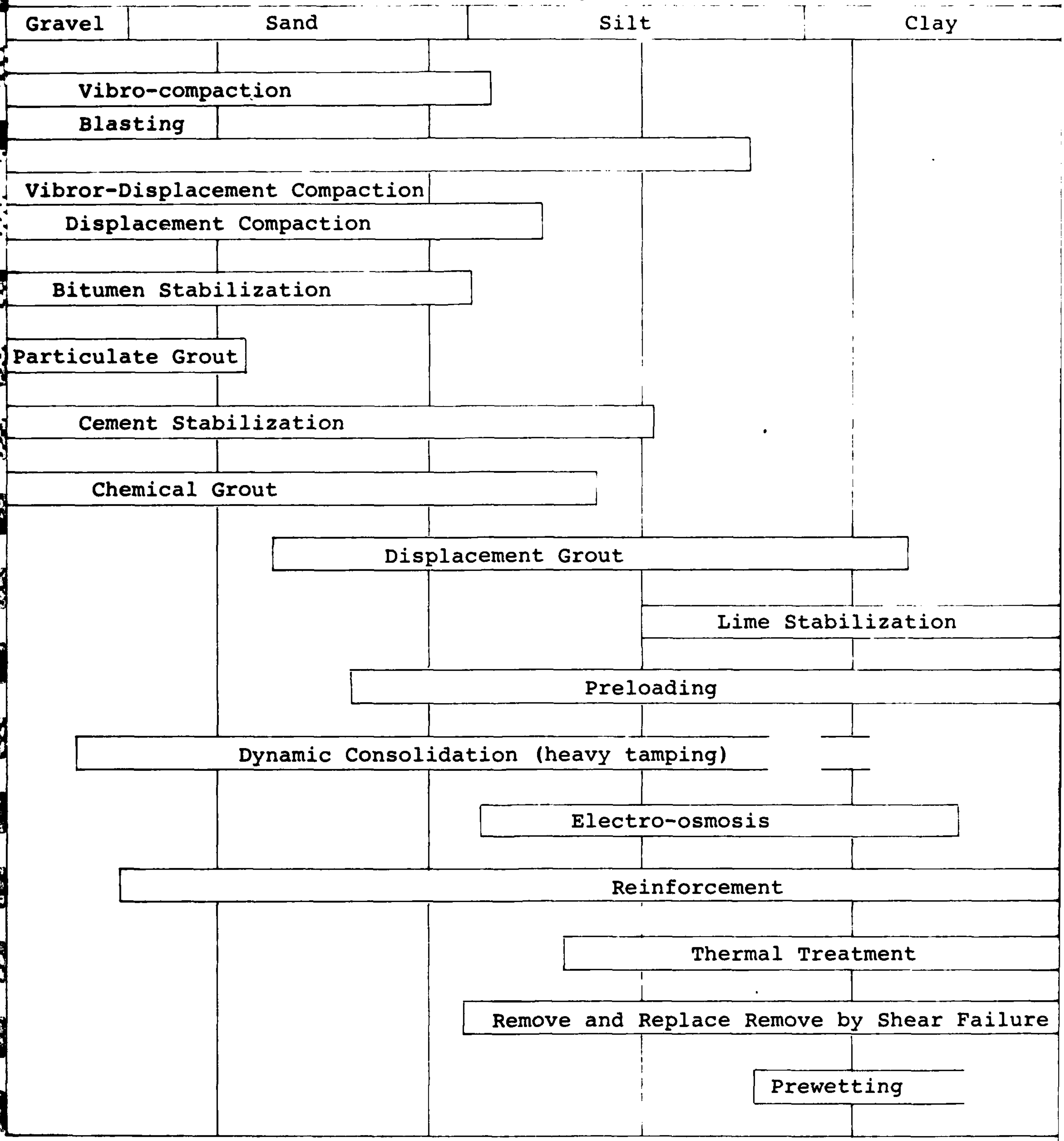


Fig. 5.1 Feasibility of Stabilization Techniques (Source: Based on Mitchell, 1976. Courtesy American Society of Civil Engineers.)

Table 5.2. Recommended Seed Mix for TRESTLE Soil (Source: Farmer, 1984)

Seed	Pounds/ Acre	*$/PLS	$/acre
Indian ricegrass	6	9.50	57.00
Side oats gramma	4	9.00	36.00
Sand dropseed	1	3.20	3.20
Giant sand dropseed	2	3.20	6.40
Galleta Viva	6	28.00	168.00
Total	19	52.90	270.60

PLS = Pure Live Seed = $\frac{\text{\% Germination x \% Purity}}{100}$

(costs subject to change)

rule of thumb when buying seed is to buy from a source within 250 miles of the area to be seeded; the seed will stand a better chance of survival (Farmer, 1984).

Once the slopes in Zone 1 have been worked and seeded, install excelsior erosion control blankets, or an approved equal product. Zone 1 will need these blankets. Otherwise, the seed will not take. The blankets tend to hold the seed in place, provide necessary moisture through increased retention, and provides a mulch to protect the seeded area and hold the soil in place. The blankets are designed such that they naturally decay after about two years. This gives the planted seed plenty of time to germinate and spread. Remember, Zone 1 is steeply sloped--retention blankets are a must if seeding recommendations are followed.

It takes about 62 blankets an acre so 8.6 x 62 = 533.2 blankets, say 550 @ estimated $48.50 (based on local current price) = $26,675.00.

Estimated time for installation is 5 men, 10 days, 8 hours per day or 400 hours. Assuming labor wages run about $13.35 per hour, 400 x $13.35 = $5,340.00. Total cost for treating Zone 1 (Farmer, 1984):

Seed @ $270.60/acre..............	$2,327
Retention blankets...............	26,675
Labor............................	5,340
Total	$34,342

Once the blankets are installed, do not allow traffic on it. The steep slopes shouldn't present a real problem (only goats would go there) but on the upper side of each drainage ditch is a 7-10 feet flat area (wide enough for a tractor). You may want to leave these "paths" open to permit maintenance accessibility to the ditches. Doing so would reduce costs. However, I still suggest seeding and mulching these paths if the blankets aren't used.

Zone Two

Restoring adequate drainage should be the second priority objective. The primary area of concern is the northwest sector of the site, refer to Drawing 5.1. The objective basically entails "cleanup" of existing, intended flow channels then concentrate on erosion control initiatives toprevent a return to the present state of deterioration. First, clean out the four pipe culverts. Two run under the main access road about 600 feet apart, another is under the first dirt road off the west side of the access road, and the other goes under the second dirt road where the drainage chute failed and the large gulley formed. Look close for these last two culverts; they're so well covered they're easy to miss. These culverts all contain very fine soils; a sewer auger and water truck should do the trick.

Second, clean and dress all exits from the concrete channels leading into the access road culverts and do the same at the entrance and exit of each culvert. One reason why so much soil has collected in the concrete channels is probably due to reversed flow caused by the fact that water has not been able to go through the culverts.

Third, regrade the entire ditch line along both sides of the access road. Tear out what's left of the riprap that was installed but save the material if possible. Once the ditch line is clean, turn attention to erosion control measures in the same area. The riprap was a good idea as far as intent; ditch flow must be decreased. However, due to the way the riprap was installed, eorsion overwhelmed it and pretty soon the water didn't know it was even there. I suggest using a modified crib wall (Fig. 2.9) sloped toward the higher elevation. Sections of railroad ties, about four feet long, would make excellent structural members and you wouldn't have to go too high to create a gradual 2-3 percent slope. On the downstream side of each ditch check, place construction gravel with heavier rock on top (use that saved from the original riprap). Eventually the soil behind each crib wall should level out and the velocity of water would be dramatically reduced. Experiment with these. I don't think it would take very many, but since the riprap was spaced at 15 feet intervals I'd space the ditch checks at 75 feet intervals.

Fourth, install riprap at the entrance and exit of each of the four culverts and finally at the exits of each drainage ditch coming from inside the bowl. Use broken concrete from sidewalk and street repairs, broken block and brick from masonry jobs, etc.

The final touch would be to seed and mulch (no retention blanket) the ditch line with the same mixture used for priority one zone.

After the northwest sector is finished,do the same thing on the northeast side where the TRESTLE drainage ditches feed into the earth ditch leading into the east arroyo.

Total estimated cost for Zone 2 treatment. (Means, 1982):

RR ties, 2.5 per ditch check, spaced 75

feet within 1840 LF = 24,

2.5 x 24 = 60 @ $20.00 ea.................$1,200

Gravel, 1 cy per ditch check,

60 cy @ $6.90.............................. 414

Riprap, 3 cy per location, 3 x 14 =

42 cy @ $19.45 (machine placed)........... 817

Labor, 5 men, 15 days,

15 x 8 = 120 hrs. per man

Foreman @ $19.25

Equipment operator @ $18.45

3 laborers @ $13.55 = $40.65

Total labor costs = $78.35 x 120......... 9,402

*Equipment - grader, 3 days rental

@ $325 per day........................... 975

Backhoe, 15 days @ $147 2,205

Auger, 1 day @ $560 560

$15,573

*Assumes equipment will be rented which is very unlikely. Consequently, actual costs will be considerably lower.

Zone Three

The majority of this work was considered in the discussion of priority 2. It all entails riprap. The only riprap remaining to be discussed is that required at exits of the gunite rundowns. These, however, in comparison to the other erosion control requirements are priority last. The only reason they are in priority three is that they were lumped with the riprap discussed in priority two.

Costs for these ten additional riprap areas is approximately 10 x 3 cy x $19.45 = $583.50 for material plus 3 men, 10 days @ $13.55 or 3 x 10 x 8 x $13.55 = $3,252.00. Total for riprap placed at ten drainage chutes approximately $3,835.5.

Zone Four

The gunite rundowns are designated Zone 4. Eventually, all these should be replaced with corrugated closed conduit at more gradual slopes than the existing. As they are now, oversplash will continue to erode the sides of the chutes. The real problem area where these chutes are concerned is G-1. Figures 1.4 and 1.5, page 17 shows the condition of this one--it simply no longer exists. G-1 should actually be considered a part of priority 2 treatment. Otherwise, once the drainage is restored and flow returns to normal the growth of the gulley depicted will tend to accelerate. Flow rates should be estimated to design the proper size of pipe required but I think a 12 inch one would do (Flow appears to be on the order of 10 cfs).

Corrugated metal pipe, galvanized, 12 inch diameter, 16 gage costs about $7.65 per LF. I suggest a 100 feet run to allow a more gradual drop and to place the exit far enough away from the embankment to reduce possibility of scour. Therefore, the cost of pipe is

$765. Earthwork and backfill will be required prior to installation. Approximately 3,000 cy of fill is required.

Total estimated cost to repair G-1 (Means, 1982):

Pipe (includes installation		$ 765
Fill (in place) $1.50 x 3000 cy		4,500
Labor 4 men, 3 days		
Operator @ $18.45	= $18.45	
2 Laborers @ $13.55	= 27.10	
1 Foreman @ $19.25	= 19.25	
	$64.80/hr.	
Time = 3 x 8 = 24 hours		
24 x $64.80		1,555
Equipment, 2 days rental		
Tractor Dozer @ $375 x 2		750
(It's highly unlikely equipment would be rented so cost would be lower).		$7,570

Very little earthwork would be necessary to replace the remaining rundowns. Therefore, costs would be significantly less. However, I do not suggest replacement until at such time they are no longer effective or have structurally failed. At present G-6 and G-7 merit serious consideration for near future replacement.

Zone Five

This area is on the inside of the bowl, south side of the facility. The slopes are gradual enough here to seed and mulch without the use of retention blankets. It's general topography and proximity to the access road lends itself very well to hydromulching which could save some money. However, if you're going to have a crew out to do priority 1, they could do priority 5 at the same time and I'm sure at less cost than contracting for hydromulch. This area also appears to be subjected more to wind erosion than the other areas. That's one reaon I suggest installing rows of railroad ties perpendicular to the slope. On the southwest side, align the ties with the two drainage ditches so that water will be diverted to the ditches. On the southeast side, use three rows at 5250, 5260, and 5220 elevations.

Estimated cost for treating zone five (labor included with Zone 1) (Means, 1983):

Seed @ $270.60/acre x 2.7........................	$ 731
860 LF of ties @ $16.00/8.5 ft................	1,619
Mulch @ $0.56/sy x 13,111 sy..................	7,342
Total	$9,692

Zone Six

This area is that portion of the TRESTLE basin on the south side of the access road. The primary reason for treatment here is to cut down wind erosion and to keep soil out of the drainage system during sheet flow. Treatment is simple. Regrade the area to uniform slope, install four inches of gravel, and compact. This will prevent the occasional but necessary traffic from tearing up the ground and will also serve to help bind the soil. An asphalt wearing course could be added later.

Estimated cost for this work (Means, 1982):

Material, gravel @ $0.90/sy x 13,000 sy........	$11,700
Labor and Equipment	
Dozer and operator, 5 days @ $674/day........	3,120
Compactor and operator, 1 day @ $79/day.....	79
Total Cost Zone 6	$14,899

Zone Seven

Use plastic soil-cement under the structure itself due to the steep slopes, confined and irregular space, and the undesirable effects of metal on the TRESTLE test program. The primary reason erosion control is necessary in these areas is to protect the base supports of the structure. Although necessary, this treatment is not an immediate concern whereas treatments in other zones are.

Estimated costs are based on an average of two soil-cement projects in New Mexico:

Ute Dam; Logan, New Mexico

Soil-cement cost per sy of slope (3:1).......$5.68

Municipal Reservoir #2, Springer, N. M.

Soil-cement cost per sy of slope (2.75:1)....$7.21

Average is $6.45 per sy of slopes

Cost for Zone 7:

$6.45/sy x 5222 sy........................$33,683

The area under the access ramp needs the treatment more. If this area alone is treated, the estimated cost becomes........................$10,750

Conclusion and Summary

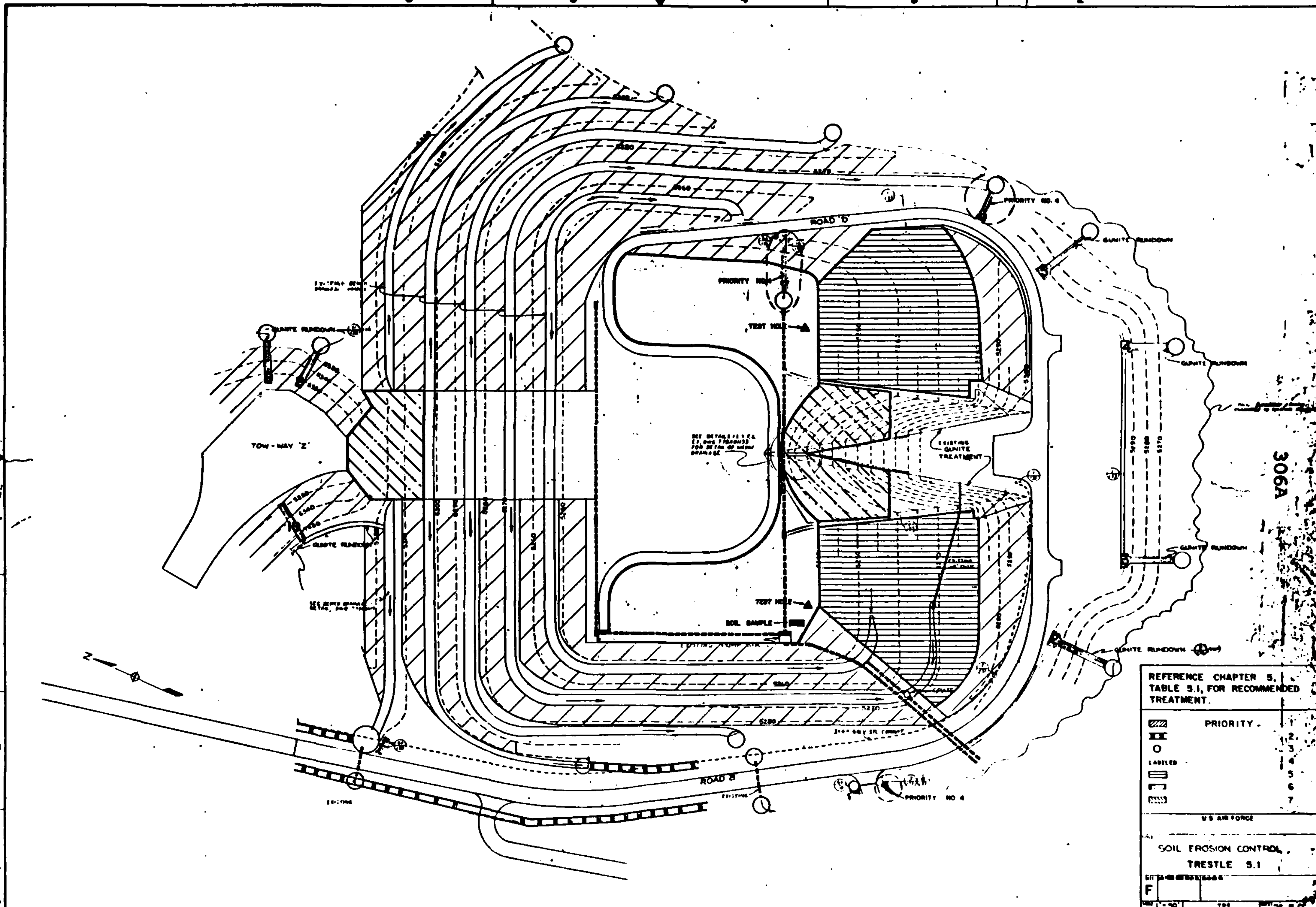
Table 5.3 summarizes the costs necessary to arrest erosion at the TRESTLE. All costs are estimated and are based on 1982 or later figures. Costs were only provided to give a better perspective of magnitude and should not be used for programming. Locally acquired costs would provide a more accurate basis for budget.

Table 5.3. Summary of Costs for Erosion Control and Repair at TRESTLE (Refer to Table 5.1 and Drawing 5.1 for zone locations and description of work).

Zone	Priority	Total Cost
1	1	$34,000
2	2	16,000
3	3	3,800
4	4	7,600
5	5	9,700
6	6	14,900
7	7	10,800
	Total Cost	$96,800

So, $100,000.00 would correct the major erosion problems at the TRESTLE. Half of that would solve immediate concerns. Of course, there are many other measures that should be taken but fall into a less urgent category. All the slopes on the exterior of the bowl need to be stabilized; closed drainage channels should be installed, especially along the access road and in the northeast drainage ditch; all ditch lines should be regraded and stabilized, etc. However, if the recommended initiatives are taken, most of the grounds maintenance headaches will be over. If these initiatives or some other measures are not taken, then be prepared to pay the price for increased manhours or suffer the consequences.

All questions concerning this report may be referred to the author at Headquarters, Air Force Engineering Service Center/DEMP; Tyndall AFB, Florida 32403.


ROAD 'D'
ROAD 'B'
TOW - WAY 'Z'
PRIORITY NO. 4
PRIORITY NO. 4
GUNITE RUNDOWN
GUNITE RUNDOWN
GUNITE RUNDOWN
GUNITE RUNDOWN
GUNITE RUNDOWN
GUNITE RUNDOWN
TEST HOLE
TEST HOLE
SOIL SAMPLE
EXISTING GUNITE TREATMENT
EXISTING
EXISTING
REFERENCE CHAPTER 5, TABLE 5.1, FOR RECOMMENDED TREATMENT.
PRIORITY.
1
2
3
4
5
6
7
LABELED
U S AIR FORCE
SOIL EROSION CONTROL
TRESTLE 5.1
F
N

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0999243 AD-A130 355/1
Mississippi River Headwaters Lakes in Minnesota. Feasibility Study. Appendices
(Final rept)
Corps of Engineers, St. Paul, MN. St. Paul District.
Corp. Source Codes: 008647003; 314050
Sep 82 687p
Appendices to AD-A130 354. Original color plates: All DTIC and NTIS reproductions will be in black and white.
Languages: English
NTIS Prices: PC A99/MF A01 Journal Announcement: GRAI8322
Country of Publication: United States
This investigation of six Mississippi River Headwaters Lakes: Gull Lake, Leech Lake, Pine River, Pokegama, Sandy and Winnibigoshish, centers on all water resource problems involving the operation of Corps' dams. The study includes a review of lake operation plans for flood control, water supply, recreation, navigation, power, and conservation; review of erosion control problems downstream of Pokegama dam; review of bank erosion; a feasibility study of stabilized White Oak Lake water levels on the Mississippi River upstream of Pokegama and Leech Lake River; a review of flood problems near Riverton, Minnesota; a hydrologic review of the existing headwaters lakes perimeter diking system; a feasibility study of removing channel obstructions on the Whitefish Lakes Chain; a study of the adequacy of the Leech Lake inlet channel; a review of a possible subimpoundment in Leech Lake and marsh restoration; and a review of the adequacy and effectiveness of the existing flood control project for Aitkin, Pine Knoll and Cedar Brook. The appendixes provide greater detail than the main report on the development of the alternatives and specific study aspects and include pertinent correspondence relating to study coordination activities among federal, state and local interests. (Author)
Descriptors: *Lakes; Minnesota; Mississippi River; Reservoirs; Flood control; Soil erosion; Banks(Waterways); Channels(Waterways); Modification; Feasibility studies
Identifiers: NTISDODXA
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8H (Earth Sciences and Oceanography--Hydrology and Limnology); 50B (Civil Engineering--Civil Engineering)

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Mississippi River Headwaters Lakes in Minnesota. Feasibility Study. Main Report
(Final rept)
Corps of Engineers, St. Paul, MN. St. Paul District.
Corp. Source Codes: 008647003; 314050
Sep 82 281p
Original contains color plates: All DTIC and NTIS reproductions will be in black and white. See also AD-A130 355.
Languages: English
NTIS Prices: PC A13/MF A01 Journal Announcement: GRAI8322
Country of Publication: United States
The investigation of six Mississippi River Headwaters Lakes: Gull Lake, Leech Lake, Pine River, Pokegama, Sandy, and Winnibigoshish centers on all water resource problems involving the operation plans for flood control, water supply, recreation, navigation, power, and conservation; review of erosion control problems downstream of Pokegama dam; review of bank erosion; a feasibility study of stabilized White Oak Lake water levels on the Mississippi River upstream of Pokegama and Leech Lake River; a review of flood problems near Riverton, Minnesota; a hydrologic review of the existing headwaters lakes perimeter diking system; a feasibility study of removing channel obstructions on the Whitefish Lakes Chain; a study of the adequacy of the Leech Lake inlet channel; a review of a possible subimpoundment in Leech Lake and march restoration; and a review of the adequacy and effectiveness of the existing flood control project for Aitkin, Pine Knoll and Cedar Brook. The main report is a general, nontechnical presentation which evaluates selected problems, summarizes costs and benefits and other impacts of each alternative, and makes recommendations for further study.
Descriptors: *Lakes; Minnesota; Mississippi River; Reservoirs; Flood control; Soil erosion; Banks(Waterways); Water supplies; Water quality; Recreation; Feasibility studies
Identifiers: NTISDODXA
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8H (Earth Sciences and Oceanography--Hydrology and Limnology); 50B (Civil Engineering--Civil Engineering)

0985330 PB83-196410
Efficiency of Erosion Control Practives of the Virginia Department of Highways and Transportation
(Final rept. May 79-Jul 82)
Wyant, David C.
Virginia Highway and Transportation Research Council, Charlottesville.
Corp. Source Codes: 057487000
Sponsor: Federal Highway Administration, Richmond, VA. Virginia Div.
Report No.: VHTRC-83-R3; FHWA/VA-83/R3
Jul 82 158p
See also PB81-245417.
Languages: English
NTIS Prices: PC A08/MF A01 Journal Announcement: GRAI8316
Country of Publication: United States
Twelve streams affected by highway construction were monitored to ascertain the effectiveness of the Virginia Department of Highways and Transportation's erosion and sediment control practices. The streams were located throughout Virginia in areas having different types of soils; viz, silts, sands, and clays. The results of the study indicate that the proper installation of erosion and sediment control measures prior to undertaking earthwork is very important.
Descriptors: *Stream erosion; *Erosion control; *Highways; Sediments; Clay soils; Silts; Monitoring; Vegetation; Dams; Virginia
Identifiers: NTISDOTFHA
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8M (Earth Sciences and Oceanography--Soil Mechanics); 8H (Earth Sciences and Oceanography--Hydrology and Limnology); 68D (Environmental Pollution and Control--Water Pollution and Control); 50A (Civil Engineering--Highway Engineering); 50D (Civil Engineering--Soil and Rock Mechanics); 48G (Natural Resources and Earth Sciences--Hydrology and Limnology)

939858 PB83-108282
Soil Characterization and Alternate Shoreline Erosion Control Measures for Constructed Lakes in Iowa
(Project completion rept.)
Lohnes, R. A. ; Berg, B. M.
Iowa State Water Resources Research Inst., Ames.
Corp. Source Codes: 063930000
Sponsor: Office of Water Research and Technology, Washington, DC.
Report No.: ISWRRI-111; W83-00218; OWRT-A-074-IA(1)
Oct 81 66p
Also pub. as Iowa State Univ., Ames. Engineering Research Inst. rept. no. ESU-ERI-AMES-82204.
Languages: English
NTIS Prices: PC A04/MF A01 Journal Announcement: GRAI8301
Country of Publication: United States
Contract No.: DI-14-34-0001-0117; OWRT-A-074-IA
Field observations of eroded lake shoreline profiles and field and laboratory measurements of geotechnical properties of shoreline soils led to development of a shoreline erosion model. The model includes equations which can be used to estimate the potential maximum height of wave-cut cliff. Predictions obtained from the model were compared with measured values in the field, and were found to show reasonable agreement. This model may be the first stop for developing a design method whereby shorelines of artificial lakes can be graded to an equilibrium configuration, thereby eliminating/minimizing erosion by wave action. Soil characterizations included soil composition analysis (e.g., sand, clay, gravel, silt), dry weight, moisture, and shear strengths. Samples were taken from shorelines of two man-made lakes to compare/contrast shoreline erosion. The lakes examined were Big Creek Lake in Polk County and Prarie Rose Lake in Shelby County.
Descriptors: *Soil erosion; *Erosion control; *Shore protection; *Lakes; Water waves; Mathematical models; Design criteria; Height; Cliffs; Soil properties; Soil classification ; Soil water; Shear strength; Big Creek Lake; Prairie Rose Lake; Field tests; Slope; Iowa
Identifiers: Artificial lakes; NTISDIOWRT
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8M (Earth Sciences and Oceanography--Soil Mechanics); 50D (Civil Engineering--Soil and Rock Mechanics); 48E (Natural Resources and Earth Sciences--Soil Sciences)

930768 PB82-209099
Polymer Stabilization of Sandy Soils for Erosion Control
Siddiqi, Razi A. ; Moore, John C.
Transportation Research Board, Washington, DC.
Corp. Source Codes: 044780000
1981 5p
Included in General Soils Problem, p30-34 1981.
Languages: English
NTIS Prices: (Order as PB82-209032, PC A04/MF A01) Journal Announcement: GRAI8218
Country of Publication: United States
No abstract available.
Identifiers: NTISNASTRB
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 13C (Mechanical, Industrial, Civil, and Marine Engineering--Construction Equipment Materials and Supplies); 8M (Earth Sciences and Oceanography--Soil Mechanics); 11I (Materials--Plastics); 50A (Civil Engineering--Highway Engineering); 71O (Materials Sciences--Plastics); 50D (Civil Engineering--Soil and Rock Mechanics)

930762 PB82-209032

General Soils Problems

Transportation Research Board, Washington, DC.

Corp. Source Codes: 044780000

Report No.: TRB/TRR-827; ISBN-0-309-03269-5

1981 53p

See also PB82-209040 through PB82-209115. Also pub. as ISSN-0361-1981. Library of Congress catalog card no. 82-3545.

Paper copy also available from Transportation Research Board, 2101 Constitution Ave., NW, Washington, DC 20418.

Languages: English

NTIS Prices: PC A04/MF A01 Journal Announcement: GRAI8218

Country of Publication: United States

The eight (8) papers in this report deal with the following areas: permeability testing of geotextiles; performance of soil-aggregate-fabric systems in frost-susceptible roads, Linn County, Iowa; pavement cracking; pavement design of unsurfaced roads; lime-soil mixtures for low-volume road construction in Egypt; polymer stabilization of sandy soils for erosion control; pore-size distribution and its relation to durability and strength of shales; eccentrically loaded surface footing on sand layer resting on rough rigid base.

Descriptors: *Pavements; *Subgrades; *Soil properties; Soil stabilization; Fabrics; Fluid filters; Cracking(Fracturing); Construction; Egypt; Erosion control; Sands; Shales; Pavement bases; Footings; Frost action; Iowa; Bearing capacity

Identifiers: Linn County(Iowa); Pore size distribution; Unpaved roads; NTISNASTRB; NTISNASNRC

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 13C (Mechanical, Industrial, Civil, and Marine Engineering--Construction Equipment Materials and Supplies); 8M (Earth Sciences and Oceanography--Soil Mechanics); 11I (Materials--Plastics); 11E (Materials--Fibers and Textiles); 50A (Civil Engineering--Highway Engineering); 50D (Civil Engineering--Soil and Rock Mechanics); 71I (Materials Sciences--Fibers and Textiles); 71O (Materials Sciences--Plastics)

930541 PB82-202102

The North Carolina Sedimentation Control Program - An Assessment

Howells, David H. ; Britt, Harlan K.

North Carolina Water Resources Research Inst., Raleigh.

Corp. Source Codes: 058521000

Sponsor: Office of Water Research and Technology, Washington, DC.

Report No.: W82-04160; OWRT-A-999-NC(55)

Jul 81 58p

Color illustrations reproduced in black and white.

Languages: English

NTIS Prices: PC A04/MF A01 Journal Announcement: GRAI8218

Country of Publication: United States

Contract No.: OWRT-A-999-NC

The history, legislation, and experience associated with a state-wide program for the control of erosion and sedimentation from urban, highway, utility, and government land-disturbing activities are reviewed. The initial principles of prevention, local involvement, flexibility, and acceptance continue to guide the program. Continued high public acceptance is reported. Work loads at state and local levels are cited. A review of technology includes: stormwater management; ground cover; channel alterations; temporary structures and drains; sediment basins; and silt check barriers, fences, and dams.

Descriptors: *Soil erosion; *Soil conservation; *Sedimentation; Erosion control; Urban areas; Highways; Land use; State government; Local government; Runoff; Earth fills; Assessments; Cost analysis; Recommendations

Identifiers: *Storm water runoff; NTISDIOWRT

Section Headings: 8M (Earth Sciences and Oceanography--Soil Mechanics); 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 48E (Natural Resources and Earth Sciences--Soil Sciences); 48B (Natural Resources and Earth Sciences--Natural Resource Management); 91A (Urban and Regional Technology and Development--Environmental Management and Planning); 43GE (Problem Solving Information for State and Local Governments--General)

876154 AD-A105 285/1

National Dam Safety Program. Goose Creek Lake Dam (MO 30297), Mississippi - Kaskaskia - St. Louis Basin, St. Francois County, Missouri. Phase I Inspection Report

(Final rept.)

Perez, Jean-Yves ; Berggreen, Richard G.

Woodward-Clyde Consultants, Chicago, IL.

Corp. Source Codes: 059768000; 411445

Jun 81 78p

Original contains color plates: All DTIC and NTIS reproductions will be in black and white.

Languages: English

NTIS Prices: PC A05/MF A01 Journal Announcement: GRAI8203

Country of Publication: United States

Contract No.: DACW43-81-C-0039

Goose Creek Lake Dam is an earth embankment impounding a lake for recreational purposes. The height of the dam at the maximum section is approximately 52 ft. The crest of the dam varies from 10 to 20 ft in width. The embankment was constructed of gravelly to bouldery silt and clay residual soil and is covered for the most part with grass and weed vegetation. A number of small trees are also growing on the downstream slope. There is no riprap or other erosion control measure on the upstream slope. The results of the visual inspection indicate the embankment is in fair to good condition. No evidence of cracking of the embankment was noted, although two areas which might be old slumps were identified on the downstream slope. No animal burrows or sinkholes were found. Motorcycle traffic has worn a path on the downstream slope and may contribute to accelerated erosion in this area. The horizontal alignment of the crest appears undeformed; settlement appears to have lowered the center of the dam crest 18 to 24 in. below the ends of the dam. Drainage from the right abutment flows in a channel along the junction of the embankment and right abutment. Erosion in this channel has steepened the toe of the slope and may have contributed to shallow slumping in this area. No erosion protection has been installed on the upstream slope, but as of the visual inspection, wave erosion was relatively minor. Hydraulic/hydrologic analyses indicate the dam will be overtopped by a flood which produces greater than 16 percent of the Probable Maximum Flood (PMF).

Descriptors: *Earth dams; Missouri; Reservoirs; Visual inspection; Safety; Stability; Hydrology; Flooding; Deficiencies

Identifiers: *Goose Creek Lake Dam; National Dam Inspection Program; NTISDODXA

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 50B (Civil Engineering--Civil Engineering)

869608 PB81-245417

Efficiency of Erosion Control Practices of the Virginia Department of Highways and Transportation

(Interim rept.)

Wyant, D. C.

Virginia Highway and Transportation Research Council, Charlottesville.

Corp. Source Codes: 057487000

Sponsor: Federal Highway Administration, Richmon, VA. Virginia Div.; Virginia Dept. of Highways and Transportation, Richmond.

Report No.: VHTRC-81-R32; FHWA/VA-81/R32

Feb 81 95p

Sponsored in part by Virginia Dept. of Highways and Transportation, Richmond.

Languages: English

NTIS Prices: PC A05/MF A01 Journal Announcement: GRAI8126

Country of Publication: United States

Stream monitoring stations have been installed on eight construction projects under phase I of this study. Monitoring on four of the projects is complete and monitoring on the remaining four is continuing. It appears that the amount of suspended sediment transported from a construction project in the Valley and Ridge region, a predominantly clayey soil area, is quite large as compared to that from a project in the Piedmont region, a silty soil area. This trend is especially evident when no erosion control measures are used. It also has been noted that relatively large amounts of sediment are generated during the spring and fall, when the soil generally is most susceptible to erosion.

Descriptors: *Stream erosion; *Erosion control; *Highways; Suspended sediments; Construction; Monitoring; Clay soils; Silts; Spring season; Autumn; Tolerances(Physiology); Virginia

Identifiers: NTISDOTFHA

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8M (Earth Sciences and Oceanography--Soil Mechanics); 8H (Earth Sciences and Oceanography--Hydrology and Limnology); 68D (Environmental Pollution and Control--Water Pollution and Control); 50A (Civil Engineering--Highway Engineering); 50D (Civil Engineering--Soil and Rock Mechanics); 48G (Natural Resources and Earth Sciences--Hydrology and Limnology)

786672 PB80-197338
Effective Methods of Establishing and Maintaining Vegetative Ground Covers for Highways in West Central Oahu
(Final rept. Jul 72-Jun 75)
Choy, Gary C. P. ; Imada, Leslie H. ; Hironaga, Patrick I.
Hawaii State Dept. of Transportation, Honolulu. Materials Testing and Research Branch.
Corp. Source Codes: 028561002
Sponsor: Federal Highway Administration, Honolulu, HI. Hawaii Div.
Report No.: FHWA/HI-79/2
Feb 79 31p
Color illustrations reproduced in black and white.
Languages: English
NTIS Prices: PC A03/MF A01 Journal Announcement: GRAI8021
Country of Publication: United States
A study to test plant materials for use in highway landscaping was initiated in 1972. Various species were screened initially and nine selected for further study. Growth characteristics and nutrient requirements were observed. Results indicate that trailing gazania may be an acceptable alternative to the currently extensive use of wedelia.
Descriptors: *Vegetation; *Highways; Soil fertility; Landscaping; Plant growth; Nutrients; Sodium; Erosion control; Slopes; Soil stabilization; Clay soils; Loams; Silts; Oahu Island; Photographs; Soil tests; Hawaii
Identifiers: Gazania uniflora; Trailing gazania; NTISDOTFHA
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 2D (Agriculture--Agronomy and Horticulture); 50A (Civil Engineering--Highway Engineering); 98D (Agriculture and Food--Agronomy, Horticulture, and Plant Pathology)

771840 PB80-175342
Soil-Water Retention, Transmission, and Quality of Leachate from Polymer-Treated Soils, Effects of the In Situ Crosslinking
(Technical completion rept. Oct 77-Sep 79)
Richardson, J. L. ; Pappas, S. P. ; Deibert, E. J. ; Wicks, Jr, Z. W.
North Dakota Water Resources Research Inst., Fargo.
Corp. Source Codes: 100435000
Sponsor: Office of Water Research and Technology, Washington, DC.
Report No.: W80-04609; OWRT-A-059-NDAK(1)
Oct 79 15p
Prepared in cooperation with North Dakota State Univ., Fargo., and North Dakota Univ., Grand Forks.
Languages: English
NTIS Prices: PC A02/MF A01 Journal Announcement: GRAI8016
Country of Publication: United States
Contract No.: DI-14-34-0001-8036; OWRT-A-059-NDAK
Clay, silt loam, and fine sandy loam were treated with various two-phase crosslinkable polymer systems and with polyvinyl alcohol (PVA) to study their effects on selected soil characteristics. Surface samples of the three soil types were taken in North Dakota and were analyzed for primary particle size distribution, natural water stable aggregate size distribution, percent organic carbon, and percent calcium carbonate equivalent. Samples were then treated with a variety of types and concentrations of amines and aldehydes of the two-phase crosslinkable polymer system. Some samples were treated with PVA and water to compare treatment effectiveness and to serve as a control. Following treatment four basic tests to determine the quality of soil aggregates on natural soils were performed: (1) plant available water holding capacity; (2) saturated hydraulic conductivity; (3) water stable aggregate size distribution; and (4) water drop penetration tests to measure water repellency.
Descriptors: *Erosion control; *Soil properties; Clay soils; Loams; Fluid infiltration; Soil stabilization; Hydraulic conductivity; Soil texture; Permeability; Agglomeration; Polyvinyl alcohol
Identifiers: NTISDIOWRT
Section Headings: 8M (Earth Sciences and Oceanography--Soil Mechanics); 48E (Natural Resources and Earth Sciences--Soil Sciences)

631076 PB-276 634/3
Re-vegetating Silt Cutbanks for Erosion Control
(Final rept. 1971-75)
Londagin, Herman E. ; Harding, Frederick H.
Alaska Dept. of Transportation and Public Facilities, Valdez. South Central Region.
Sponsor: Alyeska Pipeline Service Co., Juneau, Alaska.; Alaska Univ., Palmer. Inst. of Agricultural Sciences.; Federal Highway Administration, Juneau, Alaska. Alaska Div.
Report No.: FHWA/AK-77/2
Aug 77 55p
Prepared in cooperation with Alyeska Pipeline Service Co., Juneau, Alaska and Alaska Univ., Palmer. Inst. of Agricultural Sciences.
NTIS Prices: PC A04/MF A01 Journal Announcement: GRAI7809
Field experiments were conducted at various locations in the state of Alaska, South Central Region of the Department of Transportation in order to evaluate the adaptability of plant species, methods, materials and equipment for establishing a grass cover on silted back slope soils. The field experiments consisted of replicated tests of fertilizer rates, seeding rates, species adaptation, asphalt emulsion, and mulches. The tests showed the importance of refertilization and the use of mulch materials. Wood excelsior mulch plus asphalt emulsion proved to be the most effective mulch for fast sod formation by providing the best combination micro environment, and at the same time, slope protection from erosion.
Descriptors: *Embankments; *Slope protection; *Vegetation; *Erosion control; *Highways; Planting; Grasses; Field tests; Silts; Fertilizers; Mixtures; Plant growth; Effectiveness; Wood products; Excelsior; pH; Mulches; Asphalts; Emulsions; Alaska
Identifiers: NTISDOTFHA
Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 2C (Agriculture--Agricultural Engineering); 50A (Civil Engineering--Highway Engineering); 98C (Agriculture and Food--Agricultural Equipment, Facilities, and Operations)

/591919 PB-268 458/7
Chemical and Vegetative Stabilization of Soils: Laboratory and Field Investigations of New Materials and Methods for Soil Stabilization and Erosion Control
Morrison, W. R. ; Simmons, L. R.
Bureau of Reclamation, Denver, Colo. Engineering and Research Center.
Report No.: REC-ERC-76-13
Jan 77 167p
NTIS Prices: PC A08/MF A01 Journal Announcement: GRAI7719
A study was conducted on various chemical and vegetative methods of soil stabilization for possible use in the USBR's construction and O&M programs. This report summarizes three main items of work accomplished under the study: (1) Section A. Laboratory Studies. - (a) Results of screening tests conducted on 30 liquid soil stabilizing materials. Based on the results of these tests, tentative performance requirements were developed for water-based soil stabilizers. (b) Special studies conducted on materials and methods to waterproof problem soils, to arrest erosion occurring on slopes, and to bind gravel together for riprap applications. (2) Section B. Field Applications. - Discussions on where various chemical and vegetative methods have been used in new construction, rehabilitation, or O&M work. (3) Section C. State-of-the-Art Survey. - A survey on chemical stabilization of soils, and revegetation methods and materials for erosion control. Results of this study indicate that through chemical and vegetative stabilization of disturbed soils, sediment production can be reduced, fertile top soil preserved, and a more environmentally acceptable condition achieved after construction is completed.
Descriptors: *Erosion control; *Soil stabilization; Reviews; Laboratories; Compression tests; Gravel; Sands; Silts; Roads; Slopes; Weathering; Wind erosion; Moisture proofing; Latex; Emulsifying agents; Polyethylene; Polyvinyl chloride; Asphalts ; Mulches; Field tests; Vegetation
Identifiers: NTISDIBR
Section Headings: 8M (Earth Sciences and Oceanography--Soil Mechanics); 50D* (Civil Engineering--Soil and Rock Mechanics); 50A (Civil Engineering--Highway Engineering)

410327 PB-234 554/4

Practices in Detention of Urban Stormwater Runoff

Poertner, Herbert G.

Poertner (Herbert G.), Bolingbrook, Ill.

Sponsor: Office of Water Resources Research, Washington, D.C.; American Public Works Association, Chicago, Ill. Research Foundation.

Report No.: W74-10696; OWRR-C-3380(3722)(1)

Jun 74 206p

Also pub. as American Public Works Association, Ill. Research Foundation, Special report no. 43.

NTIS Prices: PC A10/MF A01 Journal Announcement: GRAI7422

Contract No.: DI-14-31-0001-3722; OWRR-C-3380(3722)

On-site detention of runoff was investigated as an alternative to other methods of urban stormwater runoff management. It was found that this method, which involves collecting excess runoff before it enters the sewer system, can often be applied as an effective and economical means of reducing peak runoff flow rates to lessen or eliminate problems of flooding, pollution, soil erosion and siltation. The captured runoff sometimes can be used to augment water supplies for potable or non-potable uses, and the detention facilities can be designed to serve multiple-purpose uses, especially recreation. The use of on-site detention facilities has been given emphasis in those urban areas where flooding is a frequent problem. A 1972 survey of selected local jurisdictions in urban areas of the United States and Canada revealed that more than 1,400 facilities were operational in about 100 local jurisdictions reporting applications.

Descriptors: *Surface water runoff; *Reservoirs; *Water storage; *Urban areas; Surface drainage; Flood control; Storm sewers; Design standards; Storms; Hydrology; Erosion control; Water pollution control; Silts; Surveys

Identifiers: *Storm water runoff; NTISDIOWRR

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8H (Earth Sciences and Oceanography--Hydrology and Limnology); 48G (Natural Resources and Earth Sciences--Hydrology and Limnology); 68D (Environmental Pollution and Control--Water Pollution and Control); 91A (Urban and Regional Technology and Development--Environmental Management and Planning)

325948 PB-218 854/8

Depositional Behavior of Cohesive Sediments

(Final technical rept.)

Partheniades, Emmanuel ; Mehta, Ashish J.

Florida Univ., Gainesville. Coastal and Oceanographic Engineering Lab.

Report No.: TR-16

Mar 73 299p

NTIS Prices: PC A13/MF A01 Journal Announcement: GRAI7311

Contract No.: NSF-GK-31259

The need to control erosion and deposition in waterways prompted this investigation of the fundamental laws governing transport of fine cohesive sediment. Shoaling is particularly a problem in estuaries where high salinity results in the settling of flocculated sediments. Experiments were conducted using a special testing apparatus free of secondary currents. A law describing the rates of deposition has been derived. This law gives the deposited fraction of the depositable portion of the sediment as a logarithmic-normal function of time. Using a sediment continuity principle, it was further shown that the apparent settling velocities of the flocs also depend on the same parameters. Data on the deposition of sediments from the San Francisco Bay estuary indicates a good agreement with the results derived from measurements in the testing apparatus. (Author)

Descriptors: *Sediment transport; Reviews ; *Estuaries; Sediment transport ; *Channel stabilization; *Erosion control ; Sedimentology; Cohesive soils; Water erosion; Flocculating; Suspended sediments; Waterways(Watercourses); Clays; Silts; Agglomeration; San Francisco Bay; California

Identifiers: NTISNSF

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 8H (Earth Sciences and Oceanography--Hydrology and Limnology); 60B (Civil, Structural, and Marine Engineering--Civil Engineering); 64H (Earth Sciences--Hydrology and Limnology)

309948 EIS-MS-73-0031-D

Moorhead Bayou Watershed, Sunflower County, Mississippi

(Draft environmental impact statement)

Soil Conservation Service, Washington, D.C.

Corp. Source Codes: 388099

Report No.: ELR-0031; USDA-SCS-ES-WS(Adm)-73-25-(D)

Jan 73 17p

Prepared in cooperation with Moorehead Drainage District, Indianola, Miss., and Sunflower County Soil and Water Conservation District, Indianola, Miss.

NTIS Prices: PC A02 Journal Announcement: GRAI7304

A watershed project is to be carried out in Sunflower County, Mississippi. It includes conservation land treatment and stream channel modifications on 40.4 miles of existing multiple purpose channels. Erosion and sediment production will be reduced. Floodwater damages will be reduced. Wildlife food and habitat will be enhanced. The economy of the watershed will be improved by providing additional employment. There will be a temporary loss of vegetation and associated wildlife habitat and minor channel silting during construction.

Descriptors: *Environmental surveys; *Watersheds ; *Mississippi; Environmental surveys ; Land development; Soil conservation; Channel improvements; Flood control; Erosion control; Ecology

Identifiers: *Environmental impact statements; Moorhead Bayou watershed; Sunflower County(Mississippi); NTISEIS

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 68GE (Environmental Pollution and Control--General)

305413 PB-213 207/4

Erosion Prevention During Highway Construction by the Use of Sprayed on Chemicals

(Final rept. 29 Jun 71-31 Jul 72)

Wyant, David C. ; Sherwood, W. Cullen ; Walker, Hollis N.

Virginia Highway Research Council, Charlottesville.

Report No.: VHRC-72-R1

31 Jul 72 40p

Sponsored in part by Federal Highway Administration.

NTIS Prices: PC A03/MF A01 Journal Announcement: GRAI7302

Nine commercial spray on plastic chemicals were evaluated as erosion inhibitors. All the chemicals were compared with straw tacked with asphalt emulsion and with untreated soil. The cost of each chemical was also obtained to determine if the most effective chemicals were economically feasible. In Virginia there are three principal physiographic provinces. The soils in each province are significantly different in composition and erodibility. Two highway slopes with a vertical height of 15 to 20 feet and a slope of approximately 2:1 were selected to be tested in each province. It was found that the same chemicals were not the most effective in the different areas. Also, no chemical performed better or cost less than the straw tacked with asphalt emulsion in any of the three areas. However, some chemicals did show significant erosion protection and are recommended in cases where the conventional straw and asphalt are undesirable or unavailable. (Author)

Descriptors: *Soil erosion; *Inhibitors ; *Highways; Construction ; Chemical compounds; Plastics; Feasibility; Soils; Effectiveness; Tests; Virginia

Identifiers: *Highway erosion control; Siltation; *Erosion inhibitors; NTISFHAPR

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 60F (Civil, Structural, and Marine Engineering--Highway Engineering)

282628 PB-210 316

Crownvetch, An Adaptable Ground Cover for Massachusetts Roadsides

Zak, John M. ; Kaskeski, Peter A.

Massachusetts Univ., Amherst. Dept. of Plant and Soil Sciences.

Report No.: RR-17

Mar 72 10p

Pub. as Massachusetts Univ., Amherst. Cooperative Extension Service Pub-74.

NTIS Prices: PC A02/MF A01 Journal Announcement: GRAI7216

The legume, Crownvetch (Coronilla varia), was seeded and planted on a number of roadside sites in the several soil/climatic regions in Massachusetts. Except on sandy, droughty soils, seeding produced satisfactory stands under certain conditions. It was determined that Crownvetch, with proper lime and fertilizer supplied to the planting site, is a valuable plant to revegetate slope areas with deteriorating existing vegetation. (Author)

Descriptors: *Leguminous plants; Soil stabilization ; *Highways; Erosion control ; *Ground cover; Highways ; Massachusetts; Environmental engineering; Horticulture; Methodology; Reviews

Identifiers: *Crownvetch; NTISFHAPR

Section Headings: 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 60F (Civil, Structural, and Marine Engineering--Highway Engineering)

264334 PB-207 418

Soil-Cement Slope Protection on Bureau of Reclamation Features

DeGroot, Glenn

Bureau of Reclamation, Denver, Colo. Engineering and Research Center.

Report No.: REC-ERC-71-20

May 71 111p

NTIS Prices: PC A06/MF A01 Journal Announcement: GRAI7209

A summary of Bureau of Reclamation experience with soil-cement slope protection is presented. Compacted soil-cement has been used as a riprap substitute on 7 major Bureau structures. Preconstruction testing, construction equipment and procedures, construction control testing for soil-cement, and performance of soil-cement facings are discussed. Successful performance of a soil-cement test section at Bonny Reservoir in eastern Colorado was used as the basis for the design of the facings, and durability and compressive strength test results limits established by the test section have been generally followed. Most soils used by the Bureau have been fine, silty sands; a summary of test results is presented. (Author)

Descriptors: *Soil cements; *Erosion control ; *Earth dams; Soil cements ; Slopes; Soil compacting; Durability; Compressive properties; Mixtures; Performance; Mixing; Facings ; Construction; Test methods

Identifiers: Bonny Reservoir(Colorado); Cheney Dam(Kansas); NTISBR

Section Headings: 8M (Earth Sciences and Oceanography--Soil Mechanics); 13B (Mechanical, Industrial, Civil, and Marine Engineering--Civil Engineering); 64L (Earth Sciences--Soil and Rock Mechanics); 60B (Civil, Structural, and Marine Engineering--Civil Engineering)

254942 PB-205 800

Chemical Stabilization of Soils. Laboratory and Field Evaluation of Several Petrochemical Liquids for Soil Stabilization

Morrison, W. R.

Bureau of Reclamation, Denver, Colo. Engineering and Research Center.

Report No.: REC-ERC-71-30

Jun 71 49p

NTIS Prices: PC A03/MF A01 Journal Announcement: GRAI7205

Laboratory and field evaluations of several petrochemical, liquid soil stabilizers were conducted. Laboratory tests indicated that a sprayable liquid vinyl polymer has excellent properties for stabilizing sandy soil. Initial observations showed that a deep penetrating liquid cutback asphalt was performing satisfactorily in stabilizing dune sand around transmission tower sites. A water-base acrylic copolymer is providing satisfactory erosion control on test sections of spoil banks. However, the high cost would limit the use of the material to minimum wind and water erosion control. None of 5 protective coatings applied to shale seams, were effective in reducing air-slaking. (Author)

Descriptors: *Soil stabilization; *Plastic coatings ; *Erosion control; Soil stabilization ; *Bituminous coatings; Soil stabilization ; Vinyl resins; Acrylic copolymers; Asphalts; Tailings; Emulsions; Weathering; Bonding

Identifiers: NTISBR

Section Headings: 8M (Earth Sciences and Oceanography--Soil Mechanics); 52J (Agriculture and Food--Soil Sciences)