FM 5-430-00-1
AFJPAM 32-8013, Vol I
MCRP 3-17.7A

PLANNING AND DESIGN
OF
ROADS, AIRFIELDS, AND HELIPORTS
IN THE
THEATER OF OPERATIONS—
ROAD DESIGN

HEADQUARTERS,
DEPARTMENT OF THE ARMY
DEPARTMENT OF THE AIR FORCE

Field Manual
No. 5-430-00-1
Air Force Joint Pamphlet
No. 32-8013, Vol I

Headquarters
Department of the Army
Department of the Air Force
Washington, DC, 26 August 1994

PLANNING AND DESIGN OF ROADS, AIRFIELDS, AND HELIPORTS IN THE THEATER OF OPERATIONS—ROAD DESIGN

TABLE OF CONTENTS

Volume I

Page

*This publication together with FM 5-430-00-2/AFJPAM 32-8013, Vol II, 29 September 1994 supersedes FM 5-165/AFR 86-13, 29 August 1975, FM 5-335, 2 December 1985 and TM 5-330/AFM 83-6, Vol 1, 8 September 1968.

Volume II

PREFACE

Field Manual (FM) 5-430 is intended for use as a training guide and reference text for engineer personnel responsible for planning, designing, and constructing roads, airfields, and heliports in the theater of operations (TO).

FM 5-430 is divided into two separate volumes to make it more *user-friendly*. FM 5-430-00-1/AFPAM 32-8013, Vol 1, *Road Design*, encompasses Chapters 1 through 9 and Appendices A through H. FM 5-430-00-2/AFJPAM 32-8013, Vol II, *Airfield and Heliport Design*, encompasses Chapters 10 through 14 and Appendices I through P.

FM 5-430-00-l/AFPAM 32-8013, Vol 1 is a *stand-alone* volume for the design of TO roads. This volume also serves as a detailed description of information common to both roads and airfields, such as site selection, survey and earthwork, clearing and grubbing, base and subbase courses, and drainage.

FM 5-430-00-2 /AFJPAM 32-8013, Vol II serves as the basis for airfield and heliport design. It discusses the complete process of airfield and heliport construction from the preliminary investigations, through design criteria, to the final project layout and construction techniques. It is not a *stand-alone* volume. FM 5-430-00-1/AFPAM 32-8013, Vol 1 contains much of the information required to design the substructure of an airfield or a heliport.

The material in this manual applies to all levels of engineer involvement in the TO. The manual is intended to be used by United States (US) Army Corps of Engineers personnel.

The provisions of this publication are the subject of the following international agreements:

- Quadripartite Standardization Agreement [QSTAG) 306, American-British-Canadian-Australian Armies Standardization Program, *Fortification for Parked Aircraft.*
- North Atlantic Treaty Organization (NATO) Standardization Agreement (STANAG) 3158 Airfield Marking and Lighting (AML) (Edition 4), *Day Marking of Airfield Runways and Taxiways.*
- STANAG 2929, *Airfield Damage Repair.*
- STANAG 3346 AML (Edition 4), *Marking and Lighting of Airfield Obstructions.*
- STANAG 3601 Air Transport (TN) (Edition 3), *Criteria for Selection and Marking of Landing Zones for Fixed Wing Transport Aircraft.*
- STANAG 3619 AML (Edition 2) (Amendment 2), *Helipad Marking.*
- STANAG 3652 AML (Amendment 3), *Helipad Lighting, Visual Meteorological Conditions (VMC).*
- STANAG 3685 AML, *Airfield Portable Marking.*

This publication applies to the Air National Guard (ANG) when published in the National Guard Regulation (NGR) (AF) 0-2.

This publication, together with FM 5-430-00-2/AFJPAM 32-8013, Vol H: Airfield and Heliport Design (to be published), will supersede TM 5-330/AFM 86-3, Volume II, 8 September 1968 and FM 5-165/AFP 86-13, 29 August 1975.

The proponent for this publication is the US Army Engineer School (USAES). Send comments and recommendations on Department of the Army (DA) Form 2028 (Recommended Changes to Publications and Blank Forms) directly to—

Commandant
US Army Engineer School
ATSE-TDM
Fort Leonard Wood, MO 65473-5000.

Unless this publication states otherwise, masculine nouns and pronouns do not refer exclusively to men.

GENERAL INFORMATION

CHAPTER 1

Army engineers plan, design, and construct airfields, heliports, and roads in the TO. To ensure these facilities meet proposed requirements, the responsible engineer officer must coordinate closely with all appropriate ground and air commanders. The engineer depends on the appropriate commanders for information on the weight and traffic frequency of using aircraft, facility life, geographic boundaries governing site selection, and the time available for construction as dictated by the operation plan.

Detailed planning, reconnaissance, and site investigations are often limited by lack of time and by the tactical situation. However when time and security permit, the engineer should conduct normal ground reconnaissance and on-site investigations. If this is not possible, the engineer should obtain photographs of the area.

BASIC PLANNING CONSIDERATIONS IN THE THEATER OF OPERATIONS

Army engineers should use the following guides in the TO:

- Keep designs simple. Simple designs require minimum skilled labor and specialized materials.
- Use local materials whenever possible. This helps eliminate construction delays associated with a long communications and logistics line.
- Use existing facilities whenever possible. This helps avoid unnecessary construction.
- Remember that safety factors in design are drastically reduced in the TO because of time constraints and the inherent risks of war.
- Build one of two types of structures in the TO: initial or temporary. Initial design life is up to six months; temporary design life is up to two years.
- Whenever possible, phase construction to permit the early usc of the facility while further construction and improvements continue.
- Generally avoid sites with dense brush, timberland, and rolling terrain that require heavy clearing or grading.
- Take care to prevent destruction of natural drainage channels, culverts, and roads. Repairs require time and labor far exceeding that needed to prevent damage.

AIRFIELD CONSTRUCTION

The planning and construction of Air Force bases in the TO is a joint responsibility of Army and Air Force personnel as outlined in Army Regulation (AR) 415-30/Air Force Regulation (AFR) 93-10. A summary of each service's responsibilities follows:

AIR FORCE RESPONSIBILITIES

The Air Force provides the following support:

- Emergency repair of war-damaged air bases.
- Force bed down of Air Force units and weapon systems, excluding Army base-development responsibilities.
- Construction management of emergency repair of war damage and force bed-down.
- Operation and maintenance of Air Force facilities and installations.
- Crash rescue and fire suppression.
- Supply of material and equipment to perform Air Force engineering missions.

ARMY RESPONSIBILITIES

The Army will provide the following troop construction support to the Air Force:

- Development of engineering designs, standard plans, and material to meet Air Force requirements.
- Reconnaissance, survey, design, construction, or improvement of airfields, roads, utilities, and structures.
- Rehabilitation of Air Force bases and facilities beyond the immediate emergency recovery requirements of the Air Force.
- Supply of materials and equipment to perform Army engineering missions.
- Construction of temporary standard air base facilities.
- Repair management of war damage and base development, including supervision of Army personnel. The Air Force base commander will set the work priorities.
- Road and airfield construction.

ROAD CONSTRUCTION

Engineer construction units, under the appropriate Army command, have the following responsibilities:

- Reconnoiter roads and bridges.
- Recommend traffic-control procedures.
- Construct and install signs and other route-marking materials.
- Regulate traffic at locations where engineer work is being performed.
- Assist vehicles to keep traffic moving on main supply routes regardless of weather, enemy activity, or other difficulties.

ENGINEERING STUDY

After the specific requirements for roads, air fields. and heliports have been determined engineers should prepare the facilities for use as soon as possible. In most cases, the he need is critical because the accomplishment of a mission depends on using certain airfields and roads. To obtain these facilities quickly, an adequate investigation of each site and a careful study of the design details are essential. This is explained in greater detail in Chapter 2 of this manual.

SITE SELECTION AND RECONNAISSANCE

CHAPTER 2

This chapter outlines the location, layout, and design of military roads and airfields. The first steps in constructing a road or airfield are determining the best location for the facility and formulating the essential areas and construction features. Throughout the preconstruction phase, problems can *be avoided by a well-planned site selection.*

LOCATION FACTORS

Construction of a road or airfield initially consists of providing a prepared subgrade and base course according to design criteria. Airfield runways require more transverse areas than roads. Although the governing criteria and dimensions for roads and airfields differ, the basic approach to their location and layout is the same. Engineers should use the factors listed below to locate and lay out all construction projects.

MISSION

The most important factor in selecting a site is to ensure it will fulfill mission requirements. Lines of communication (LOC) must be build to accomplish a specific mission in the most direct and efficient manner possible. All location factors must be evaluated to support the mission.

EXISTING FACILITIES

Use all existing facilities. The wartime missions of engineer troops arc so extensive and the demand for their services so great that new construction should be avoided. Extensive roadnets of varying quality and capacity already exist in most areas of the world, Where possible, use these roadnets to the fullest extent. In many cases, expansion and rehabilitation of existing facilities is adequate for mission accomplishment.

Except in highly developed areas, existing airfields are seldom adequate to handle modern, high-performance aircraft. However, with minimum rehabilitation these airfields can usually be made adequate to accommodate them. They may serve as the nucleus for larger fields that meet the requirements of high-performance aircraft. Helicopters and light planes can often operate from existing roads, pastures, or athletic fields.

LOCATION AND DESIGN

To the greatest extent possible, the location and design for a facility must provide the best response to all requirements. Alternative road and airfield plans can be evaluated, from the standpoint of total earthwork and drainage structure requirements, to reduce construction effort.

Try to construct airfields in an area that will serve existing and future requirements. Consider the future needs of military units and facilities, such as depots and hospitals, when locating roads. Soil type and incumbent pavement structure requirements, rock formations, and vegetation should also be considered in locating roads. A given road segment to be constructed or improved should be considered in view of its contribution to the overall network. Similarly, an airfield should be evaluated for its ability to enhance an arifield network.

SOIL CHARACTERISTICS

Locate all roads and airfields on terrain having the best possible subgrade soil conditions. This will decrease construction effort and result in a better facility. The subgrade should be compacted under conditions allowing it to support the design loads. Conduct a basic soils investigation prior to construction to provide data needed to ensure good construction decisions. Refer to FM 5-410 for soils information and FM 5-530 for soil survey procedures.

DRAINAGE

Locate roads in areas that are easily drained and where drainage structures are minimized. Drainage is a more critical factor in locating airfields than roads. Because of the wide areas involved in airfield installations, water must be diverted completely around the field or long drainage structures that are difficult to maintain must be constructed. This topic is further discussed in Chapter 6 of this manual.

Avoid the low points of valleys or other depressed areas because they are focal points for water collection. Many airfields are constructed across long, gentle slopes because of the relative ease of diverting water around the finished installation. Avoid construction on unprotected floodplains and alluvial fans, if possible, due to the flood hazard. Alluvial terraces are often ideal locations for air fields. They offer flat expanses that are above the river floodplain and are normally protected from flooding.

Avoid constructing facilities in areas of high water tables. Although it is possible to construct subsurface structures that will remove part of this moisture, maintaining routes through these areas presents a continual problem. If it is impossible to avoid constructing a road or airfield in this type of terrain, the water tables must be lowered during construction to reduce the adverse effect of water on the strength of the supporting subgrade and base course.

GEOLOGY

Before locating any lines of communication, carefully analyze the geology of the area. Sizeable quantities of rock anywhere along a construction project will cause a large removal problem, slow construction, and increase the construction effort. Engineer troop units require special equipment and training to excavate rock.

Rock outcropping are more common in hilly terrain than in flat or rolling country. In areas where the preliminary design indicates that cutting is required to reach final grade, take enough borings to determine the location of the rock.

identify the type of rock material for evaluation as a suitable construction aggregate. Determine the structural orientation of the rock mass to properly design road cuts and ensure rock-slope stability. In sedimentary rocks it is best to align road cuts perpendicular to the strike. If this is not possible, use the safe-slope ratios shown in Figure 2-1.

TOPOGRAPHY

Construct all roads and airfields within maximum grade specifications. The specifications depend upon the facility's construction standard. Thus, avoid excessive grades and steep hills when locating these routes. If steep hills must be negotiated, the route should run along the side of the hill rather than going directly over it. This may result in a longer route, but it is generally more economical and avoids excessive grades.

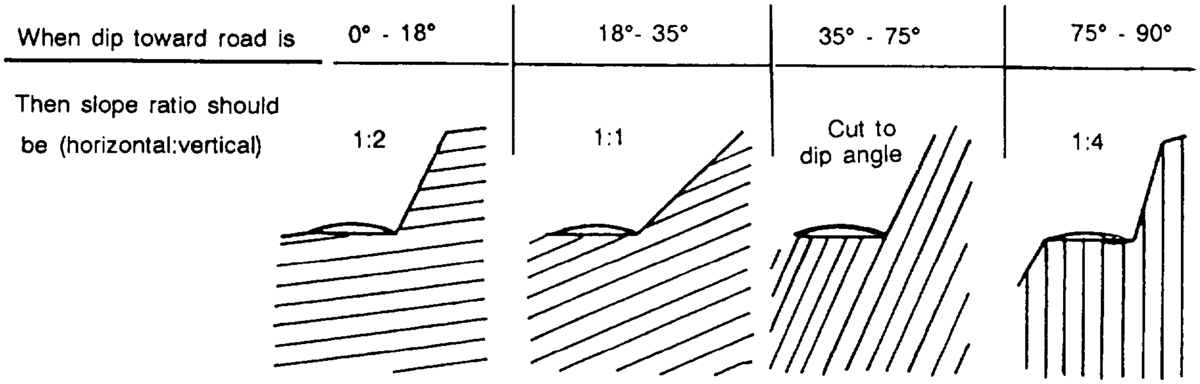


Figure 2-1. Safe-slope ratio

EARTHWORK

The largest single work item during construction of LOC is earthwork operations. Any step that simplifies earthwork operations will decrease required work and increase job efficiency. Generally, when cutting and filling on a project, earth handling is reduced by using the material excavated to construct required embankments. This balancing must be within the haul capabilities of the available equipment. Even though earthwork should be balanced throughout a project, if the haul distance becomes excessive, it may be more practical to open a nearby borrow pit or establish spoil areas. Balancing cannot be done where the excavated material is not acceptable for usc in an embankment.

ALIGNMENT

Keep the number of curves and grades at a minimum for efficient traffic flow over roads. All vehiceles have difficulty in negotiating sharp curves; even gentle curves decrease traffic capacity. Lay all routes with minimum curves by making the tangent lines as long as possible. Locating long tangents is influenced primarily by the terrain and limited by the following principles of efficient location: minimizing earthwork, avoiding excessive grades, and obtaining suitable fill material. Align runways in the direction of the prevailing wind because aircraft usually land and take off into the wind.

OBSTACLE CROSSINGS

Whenever a route crosses a major obstacle, such as a river, a ravine, or a canal, bridges or other structures arc required. Construction is time-consuming and requires materials that may be in short supply. Avoid these obstacles whenever possible. It will be advantageous to forego many of the other location principles to decrease the number of obstacle crossings. Use existing structures to decrease total work requirements, This may require only the strengthening of an existing bridge or no bridging work at all. When possible, the road should not cross a particular obstacle more than once.

BRIDGE APPROACHES

When locating rou tes, carefully evaluate construction requirements for approaches to obstacles. Construction of approaches over marshes or floodplain areas can cause greater requirements than the obstacle crossing itself. Approach conditions may be the prime factor in obstacle crossing and may dictate route location. Consider the approach with the obstacle when establishing the optimum route.

GROUND COVER

All routes should avoid heavily wooded areas that require extensive clearing, lf this is not possible, the route should pass through areas having the least vegetation. Precede all earthwork by stripping unsuitable material.

REQUIRED AREAS

Airfields need large areas of relatively flat land to efficiently accomplish their mission. This usually restricts the number of sites that can be considered for airfield construction. Advance location and layout will avoid cramping necessary facilities. Frequently, the airfield must be spread over a large section to obtain the required area. This results in the construction of a complex network of taxiways and service roads. When this is the case, keep in mind the ability to construct this connecting network to appropriate specifications.

Roads built on rolling or flat terrain seldom require large, lateral areas. Roads constructed in deep cuts or fills require proportionately greater lateral areas to account for slopes.

ACCESSIBILITY TO MATERIALS AND UTILITIES

The efficient operation of airfields requires the use of electricity, water, gas, and sewer systems. Locating new airfields near existing utility systems can avoid the construction of new facilities or long transmission lines. A nearby railhead will help the construction effort.

Consider the quality and availability of construction materials when locating a facility. Obtain suitable base-course materials from existing pits and quarries whenever possible because much planning and effort are required to open a new quarry. The quality and available quantity of materials must meet the construction requirements. The proximity of a suitable base-course-material source is a critical planning factor.

MARGINAL MATERIAL

When planning the location of a project, consider using marginal materials nearby. Using marginal materials for subbase and base construction and, as an aggregate in pavements is sometimes possible by using geotextiles, mechanical stabilization, or admixtures. Often the use of marginal material is unavoidable. Where possible, poor -quality material should be excavated and replaced with more suitable material, or the project should be relocated.

FLIGHT-WAY OBSTRUCTIONS

The safe operation of fixed- or rotary-wing aircraft requires the removal of all obstacles above lines specified by design criteria. These criteria depend upon the operating characteristics of the aircraft to be serviced. For example, most heliports require an approach zone with a 10:1 glide angle (8:1 for short-duration operations), whereas heavy-cargo aircraft in the rear area require a glide angle as flat as 50:1. To achieve this glide angle, it is often necessary to remove vegetation and hills and perform extensive earthwork operations far from the airfield proper. Thus, avoid locations that require extensive work to achieve the necessary glide angle.

A similar clearance is required on the sides of runways. An area of specified width must be cleared of all obstacles and graded according to specifications.

SUNLIT SLOPES

lf tactical concealment is not required, locate roads on the sunny, exposed sides of valleys or hills, particularly in wet or cold areas. This permits the road surface and subgrade to dry rapidly, minimizes icy conditions, and makes maintenance easier. Prevailing winds should also be considered when locating roads. Prevailing winds will carry snow, rain, and sand onto the road - way, if the orientation of the road is undesirable. Protective snow or sand fences should be oriented to take into account the prevailing winds.

TACTICAL CONSIDERATIONS

Frequently, it is necessary to construct temporary roads or heliports or to improve landing strips to move personnel and materials. When this is the case, consider the following tactical factors:

Defilade. Locate all roads in a defilade position on the reverse side of a hill or ravine to avoid enemy observation and to provide cover from direct artillery or mortar fire.

Camouflage. When constructing a road or airfield in an exposed area, take advantage of all natural camouflage and concealment.

Defense. Air fields in forward areas are prime targets for enemy air and ground attacks. When designing the airfield, disperse the facilities to minimize the effects of bombing or strafing attacks. It may be necessary to use ground troops in defensive positions against enemy ground action.

FUTURE EXPANSION

Due to the unpredictability of military operations, engineer troops are often required to modify and expand previously completed construction. The road that is adequate for today's maneuvers may be inadequate for tomorrow's operations. Airfields built for small aircraft with a limited evacuation mission may have to be modified to meet more stringent design criteria for accommodation of high-performance aircraft, Improvement and expansion are a continuing job on all military construction.

Try not to construct a road or airfield in a restricted area where there is no possibility of expansion. Design basic facilities so that they can be used as part of the expanded facilities. The ability to expand an existing route or facility will conserve personnel and material and permit rapid completion of future projects.

RECONNAISSANCE

Reconnaissance operations vary with the operational environment; the assigned mission: and the size, type, and composition of the reconnaissance element. An aerial, map, or ground reconnaissance is necessary to determine the best existing or best possible location for a future road or airfield.

MISSION

The primary mission of a reconnaissance party is to find a site meeting most requirements, to recommend a general layout and construction plan, to estimate the work required to construct the facility, and to obtain the data needed to determine a completion date and detailed construction schedules. When the reconnaissance mission is complete, the reconnaissance report serves as the basis for tactical plans and construction schedules.

The final construction plans and schedules are made with regard to the tactical and logistical situation and the construction time available. The reconnaissance report, submitted by personnel conducting the investigation, must be complete, comprehensive, and sufficiently detailed to permit careful analysis,

RECONNAISSANCE-PARTY CAPABILITIES

Thorough reconnaissance requires qualified, trained, and experienced personnel. The quality of the reconnaissance is directly related to the abilities of the party accomplishing it. This is especially true in airfield reconnaissance, which requires broader engineering judgment than any other engineer reconnaissance, Even a qualified civil engineer with civilian or military experience requires special training for this activity. It is unusual for one person to be proficient in all the items a thorough reconnaissance must include. Therefore, the assignment of personnel to the party must provide for its overall efficiency as a unit. The party must be selected with regard to the conditions it may confront.

Factors to be considered include the road - net, the general nature of the terrain, the weather, the prevalence of land mines, the attitude of the civilian population, and the amount of enemy resistance the party may expect. These factors also influence the equipment assigned to the party. The equipment should include all items necessary for soil and topographic surveys, mobility, security, and good communication. The success of the mission depends on proper personnel and equipment. One without the other will not accomplish the needed results, If available, a soils or terrain analyst is a valuable member of the reconnaissance party, If an analyst is not available, obtain soil samples for later analysis.

STEPS IN RECONNAISSANCE

Reconnaissance involves the steps that follow.

Planning

Planning is concerned with the formation of a reconnaissance mission. It involves the coordination of reconnaissance efforts by appropriate headquarters, the estimation of needs, and the assignment of a reconnaissance mission. Both ground and aerial methods should be integrated. This is a responsibility of the engineer brigade, the group, or the battalion, not the individual reconnaissance party. Reconnaissance missions are based on user requirements as governed by ground forces. Maintain close liaison with all headquarters to achieve proper coordination, Improper coordination results in duplication of effort in some areas and inadequate reconnaissance in other areas.

Briefing

The briefing tells the reconnaissance party exactly which site or area is to be reconnoitered, what is already known about the area or site, and what information the party is expected to obtain, Details concerning the time or methods of reporting the information will be included in the briefing. The party must also know the type of facility for which it is reconnoitering. If a site has been tentatively selected or if some information has already been determined from a preliminary study, the party must be informed. Otherwise, time and effort will be wasted. A soils or terrain analyst should brief the reconnaissance party, if such an expert is not able to accompany the party, If available, aerial photos should be used in the briefing.

The following information is necessary for a full understanding of a particular reconnaissance mission and should be covered in the briefing:

- The general area to be covered, if an area reconnaissance is to be conducted; or the exact location of the site or facility to be investigated, if a specific reconnaissance is to be done.
- The nature of the proposed facility; the types of vehicles or aircraft scheduled to use it; the length of time such use is anticipated; and the minimum requirements concerning dimensions, grades, and clearances. (These items are covered by reference to the applicable standard layout and specifications published by the joint force commander in the theater. They are usually familiar to the reconnaissance officer but should be kept for reference.)
- The anticipated vehicle traffic and number of aircraft and personnel to be initially accommodated at the proposed facility. (When dealing with airfields, figures are often given in terms of the number and type of aviation units to be assigned to the installation. Strength and equipment figures should also be available for reference.)

- The minimum amount of aircraft service, repair facilities, and special requirements needed.
- The expected future expansion of the new facility.
- The expected construction time available for building support facilities.
- Information previously obtained about the proposed project.
- The essential details concerning the report and how, when, and to whom the report should be made,

When the reconnaissance party is to be away from its parent unit for a lengthy, continued reconnaissance, the following additional instructions must be covered in the briefing:

- The location where rations, clothing, and equipment replacements can be drawn.
- The source from which petroleum, oils, and lubricants (POL) supplies can be drawn.
- The service facility where vehicle maintenance can be obtained.
- The form of communications to be arranged; for example, radio, messenger, or telephone.

When ground reconnaissance is ordered ahead of forward ground-force elements, the following additional instructions are necessary:

- Friendly-force situation.
- Known enemy-force situation,
- Location of adjacent friendly units.

The following instructions are applicable only to parties engaged in air reconnaissance:

- Alternative and emergency-landing instructions.
- Location of available aviation petroleum supplies.
- Location of the forward flying line.

Preliminary Study

The preliminary study consists of studying the information obtained during the briefing, conducting a map reconnaissance of the area involved, studying aerial photos, delineating soil boundaries, assembling other available preliminary information, and planning and preparing for the actual reconnaissance.

Sources of information that may be useful in the preliminary planning of reconnaissance missions and in the preliminary study of a specific mission are discussed below. Such information must be verified by ground reconnaissance.

- Intelligence dossiers that provide planning data and other information on a particular airfield site or route that may already exist. These dossiers are the result of previous reconnaissance or reconnaissance plans and can usually be obtained if adequate coordination is maintained with higher headquarters and other units engaged in reconnaissance. Similarly, reports of aerial reconnaissance that were conducted in anticipation of later ground reconnaissance may be available from adjacent or higher units.
- Strategic and technical reports, studies, and summaries on specific areas of actual or potential military importance are prepared by the Office of the Chief of Engineers and subordinate agencies. These reports provide the best data available at the time they were printed. Topographic, geologic, and soil maps, as well as data on the climate and groundwater tables, are usually included. These reports may contain information on water supply, construction

materials, vegetation, and special physical phenomena.

- Army and Air Force periodic intelligence reports are important, reliable sources of information. Intelligence reports are usually prepared in the interior zone, but periodic intelligence reports are field-prepared reports of all-around force elements. They include facts learned by prisoner-of-war interrogations, tactical data, reports, records, and interrogation of local inhabitants. Intelligence reports are used to prepare strategic and technical reports.

- Road, topographic, soil, vegetation, and geologic maps published by friendly or enemy govern ments and agencies are sources of information. Maps showing the suitability of terrain for various military purposes may be of considerable value in planning roads and airfields.

- Aerial photographs show the approximate amount of grading and excavation required, the total area and extent of promising sites, the extent of necessary clearing, the presence of flying hazards (for airfields), and the area and local drainage conditions.

- If time and facilities are available, topographic maps should be prepared from aerial photographs.

- Weather reports published by governmental agencies and the Air Force Air Weather Service are used to determine critical factors for runoff determination, prevailing winds, and cloud cover which will affect construction and future operations.

- Aeronautical reports and charts provide an overview to help plan aerial reconnaissance.

- Indigenous governmental agencies may provide valuable information on a great diversity of subjects.

Air Reconnaissance

Air reconnaissance involves a general study of the topography, drainage, and vegetation of the area. The construction problems, camouflage possibilities, and access routes should be visualized. Usually the specific ground-reconnaissance procedure is planned by selecting, from the air, areas that need investigating and by determining what questions need answering. Air reconnaissance can provide valuable negative information by eliminating unsuitable sites, but it cannot be solely relied on for positive information.

Ground Reconnaissance

While air reconnaissance can effectively reduce the amount of ground reconnaissance, it cannot replace ground reconnaissance. It is on the ground that most questions are answered or that questions tentatively answered from the air are verified. Often ground and air reconnaissance are not separate missions, A continuing air reconnaissance may be interspersed with specific ground reconnaissance.

REPORTING

The reconnaissance party must always submit its report on time. Reports are submitted for all sites investigated, even if the reconnaissance party considers the site unsuitable.

Full details on the method, place, and time of submitting reconnaissance reports should be included in the instructions given to the reconnaissance party. Reconnaissance reports can be submitted in writing or by radio. A radio report should be followed by a detailed written report. Standard reconnaissance reports are preferred. They ensure full coverage of needed information and allow a comparative evaluation of two or more sites. Standard formats are helpful in comparing sites which have been reconnoitered by different parties. They simplify each party's work in preparing reports.

Military roads and road networks are defined according to location and use. They are classified according to width, surface, and obstructions. Terms and formulas approved by the member nations of the NATO, the Southeast Asia Treaty Organization (SEATO), the United States, the United Kingdom, the Canadian and Australian Armies Nonmaterial Standardization Program, and other treaty nations are covered in FM 5-36.

Abbreviations, symbols, and notations used in route reconnaissance (described in FM 5-36) may also be used in airfield reconnaissance. Information given in road reconnaissance reports is useful in reporting on access roads to airfield and heliport sites.

AIR RECONNAISSANCE

An air-reconnaissance team generally consists of only two members: the pilot and the engineer observer. Having the officer in charge of the ground-reconnaissance party act as the engineer observer is advantageous and should be arranged when possible. Time is saved and errors of omission are minimized when a report from the engineer observer to the officer in charge of the ground-reconnaissance party is not necessary except as a matter of record, The pilot can also assess the site and make the appropriate recommendations.

Two -place, fixed -wing aircraft or two-place helicopters are suitable for most air-reconnaissance missions, Reconnaissance of enemy-occupied airfields is best accomplished with modified tactical aircraft.

Effective air reconnaissance should provide the following information:

- Determination of terrain features.
- Description of obstacles.
- Evaluation of LOC.
- Assessment of suitability of the area for various types of construction.
- Identification of available sources of water.
- Supply evaluation of construction materials in the area of operations.
- Discussion of cover and concealment.

GROUND RECONNAISSANCE

The composition of the ground-reconnaissance party depends on the scope and extent of the mission and the nature of the terrain it must traverse. The composition depends upon the probability of contact with the enemy, the attitude of the civilian population, and the prevalence of mines in the area to be reconnoitered. Table 2-1, page 2-10, provides a list of personnel suitable for an airfield reconnaissance. The list can be modified to meet the particular needs of the situation.

All personnel involved should be trained in ground reconnaissance. It is important that the person in charge and the assistant be well versed in all aspects of reconnaissance.

All equipment needed to carry out the assigned tasks should be taken. The equipment varies as the composition of the party varies. A typical list of equipment suitable for the party is listed in Table 2-2, page 2-10.

Map and air studies are not substitutes for ground reconnaissance; they only reduce the amount of ground effort required. Ground reconnaissance should determine the following information:

- Estimated grades to be encountered.
- Estimated amount of clearing involved. This includes trees, tree stumps, and boulders. Sometimes objects such as buildings and concrete foundations are included.
- Consideration of debris generated during clearing operations. In some

Table 2-1. Typical airfield ground-reconnaissance party

	Grade	Primary Duty	Secondary Duty
1	Officer	Command Party	General reconnaissance
2	Sr NCO	Second in command	
3	EM	Technical Engineer	Machine gunner
4	EM	Plane-table man	Machine gunner
5	EM	Terrain intelligence analyst	Rodman
6	EM	Airphoto interpreter	Soils analyst
7	EM	Driver/RTO	Assistant machine gunner
8	EM	Driver/RTO	Wheel-vehicle mechanic

Table 2-2. Suggested equipment list for airfield ground-reconnaissance party

Item	Quantity	Item	Quantity
Truck, 1 1/4 ton	2	Clinometer	1
*Carrier, personnel, armored	2	Panel marking sets	2
Machine gun, 7.62 mm	2	Pioneer tools	1 set/vehicle
Pedestal, 7.62 mm machine gun mounted	2	Towing chain	2
Binocular, 7 x 50	2	Material for marking, fording, and swimming sites	As required
Goggles, sun, plastic	6	Improvised means of measuring water depths	1
Radiacmeter, IM-93/UD	1	Measuring tape	2
Radiacmeter, IM-174/PD	2	Three-man pneumatic reconnaissance boat	1
Detector kit, chemical agent, AN-M256	1	Vehicular first-aid kit	2
Paper, chemical agent detector, M8	1 book	FM 5-34	1
MOPP gear	As required	Reconnaissance report forms and formats	As required
Radio set, mounted in truck	1	Adequate map and aerial photo coverage	As required
Flashlight	4	Tracing tape (tape, textile)	As required
Camera (Polaroid) with film	1	Lensatic compass	2

*Desirable when operating in support of mechanized forces or in northern areas.

cases, the trees removed may be used in the construction operation. Details of clearing operations are discussed in Chapter 4 of this manual.

- Nature of soil encountered, field determination of gradation, percentage of fine-gradient materials, and plasticity characteristics.
- Conditions of streams at crossing sites; width, depth, and velocity of the stream; condition of the banks and streambed; and indications of high water levels.
- Presence or absence of local construction materials, including possible sources of sand, gravel, cement, tar, asphalt culvert pipe, and lumber, Local construction capabilities and labor conditions are included.
- Estimated amount of earthwork necessary, the approximate balance between cut and fill, and the necessity for long hauls of earth material.
- Errors or discrepancies on the maps from which the site was tentatively selected and the effects of such errors on the selection.
- Local rainfall data and other pertinent information about seasons and weather obtained through local inhabitants or other sources,
- Information or observations which affect the final facility location.
- Relationship with the local population.

ROUTE AND ROAD RECONNAISSANCE

Thorough reconnaissance is essential in the selection of roads. It starts with a study of available maps and aerial photographs. Aerial reconnaissance provides valuable information. Detailed information, however, can be obtained only by ground reconnaissance. Reconnaissance performed in connection with military LOC is route reconnaissance. Reconnaissance to check existing roads is road reconnaissance. Reconnaissance to determine the location for a new road is location reconnaissance.

ROUTE RECONNAISSANCE

Route reconnaissance includes gathering information about roads, bridges, tunnels, fords, waterways, and natural terrain features that may affect the movement of troops, equipment, and supplies in military operations. Route reconnaissance may be hasty or deliberate. A hasty route reconnaissance is conducted to determine the immediate trafficability of a specified route and is limited to critical terrain data. It may be adequately recorded on a map overlay or sketch and be supplemented by reports about various aspects of the terrain, A deliberate route reconnaissance is detailed. It provides the data necessary for a thorough analysis and classification of significant features along a route, including repair or demolition procedures, if required, An overlay is used to point out exact map locations, and enclosures are attached to the overlay. The enclosures are DA Reconnaissance Report forms that provide a permanent record and ensure enough detail is recorded, The use of these forms is explained in FM 5-36.

ROAD RECONNAISSANCE

Road reconnaissance is conducted to determine the traffic capabilities of existing roads and to provide more detailed information than is needed for route classification. It may include enough information to develop work estimates for improving the road to certain standards of trafficability DA Form 1248, shown in Figure 2-2, pages 2-12 and 2-13, is used to record this information. Maps, overlays, and sketches are used as necessary.

ROAD RECONNAISSANCE REPORT
For use of this form, see FM 5-36; proponent agency is TRADOC

DATE: 29 AUG 88

TO (Headquarters ordering reconnaissance): CDR, ATTN: S-2, 21ST ENGR. BN.

FROM (Name, grade and unit of officer or NCO making reconnaissance): D. MOONEY, SFC, CO. A, 21ST ENGR. BN.

1. MAPS	a. COUNTRY	b. SCALE	c. SHEET NUMBER OF MAPS	2. DATE/TIME GROUP (Of signature)
	FT. LEONARD WOOD SPECIAL	1:50,000	AMS V733 SHEET 5561 IV	29 1430 AUG 84

SECTION I — GENERAL ROAD INFORMATION

3. ROAD GRID REFERENCE FROM	TO	4. ROAD MARKING (Civilian or Military number of road)	5. LENGTH OF ROAD (Miles or kilometers, specify)
UT 122864	UT 097999	VIRGINIA ROUTE 617	16 KM.

6. WIDTH OF ROADWAY (Feet or meters, specify): 6.7 m to 9.3 m

7. RECONNAISSANCE — DATE: 29 AUG 88 — TIME: 0615

8. WEATHER DURING RECONNAISSANCE (Include last rainfall, if known): FAIR - TEMP 79° LAST RAIN FALL - 15 AUG 88

SECTION II — DETAILED ROAD INFORMATION (When circumstances permit more detailed information will be shown in an overlay or on the mileage chart on the reverse side of this form. Standard symbols will be used.)

9. ALINEMENT (Check one ONLY)		10. DRAINAGE (Check one ONLY)	
	(1) FLAT GRADIENTS AND EASY CURVES		(1) ADEQUATE DITCHES, CROWN/CAMBER WITH ADEQUATE CULVERTS IN GOOD CONDITION
	(2) STEEP GRADIENTS (Excess of 7 in 100)		
	(3) SHARP CURVES (Radius less than 100 ft (30m))	✓	(2) INADEQUATE DITCHES, CROWN/CAMBER OR CULVERTS, ITS CULVERTS OR DITCHES ARE BLOCKED OR OTHERWISE IN POOR CONDITION
✓	(4) STEEP GRADIENTS AND SHARP CURVES		

11. FOUNDATION (Check one ONLY)			
✓	(1) STABILIZED COMPACT MATERIAL OF GOOD QUALITY		(2) UNSTABLE, LOOSE OR EASILY DISPLACED MATERIAL

12. SURFACE DESCRIPTION (Complete items 12a and b)

a. THE SURFACE IS (Check one ONLY)			
✓	(1) FREE OF POTHOLES, BUMPS, OR RUTS LIKELY TO REDUCE CONVOY SPEED		(2) BUMPY, RUTTED OR POTHOLED TO AN EXTENT LIKELY TO REDUCE CONVOY SPEED

b. TYPE OF SURFACE (Check one ONLY)			
	(1) CONCRETE		(6) WATERBOUND MACADAM
✓	(2) BITUMINOUS (Specify type where known): ASPHALT		(7) GRAVEL
			(8) LIGHTLY METALLED
			(9) NATURAL OR STABILIZED SOIL, SAND CLAY, SHELL, CINDERS, DISINTEGRATED GRANITE, OR OTHER SELECTED MATERIAL
	(3) BRICK (Pave)		(10) OTHER (Describe):
	(4) STONE (Pave)		
	(5) CRUSHED ROCK OR CORAL		

SAMPLE

SECTION III — OBSTRUCTIONS (List in the columns below particulars of the following obstructions which affect the traffic capacity of a road. If information of any factor cannot be ascertained, insert "NOT KNOWN")
(a) Overhead obstructions, less than 14 feet or 4.25 meters, such as tunnels, bridges, overhead wires and overhanging buildings.
(b) Reductions in road widths which limit the traffic capacity, such as craters, narrow bridges, archways, and buildings.
(c) Excessive gradients (Above 7 in 100)
(d) Curves less than 100 feet (30 meters) in radius
(e) Fords

SERIAL NUMBER a	PARTICULARS b	GRID REFERENCE c	REMARKS d
	STEEP GRADE - 8%	UT 119872	200 m LONG
	SHARP CURVE	UT 112877	RADIUS 21 m
	CONSTRICTION	UT 112878	6.7 m WIDE, 300 m LONG.
	CONSTRICTION	UT 105896	7 m WIDE, 100 m LONG.

DA FORM 1248, 1 JUL 60 — PREVIOUS EDITION IS OBSOLETE

Figure 2-2. Sample Road Reconnaissance Report, DA Form 1248

SECTION IV - MILEAGE CHART

ROUTE		SCALE	DATE
FROM UT 122864	TO UT 097999	2 units = 1 km	29 Aug 88

ROAD INFORMATION	DISTANCE		ROAD INFORMATION
	MILES	KILOMETERS	
Shirley Highway	10		16.0 Km
	9		
(OB) Built-up area (wostfeld)			Bd 7.3/9.3 m kb (OB)
	8		
	7		11.0
	6		A 7.0/9.0m Kb (OB)
	5		
(OB) Constriction			
	4		6.0
	3		
			BCGD (fp) 6.7/8.7m Kb (OB)
	2		
(OB) Constriction			
Sharp Curve			
	1		
Steep Grade			
	0		

REMARKS ALL MEASUREMENTS IN METERS

Shoulders very soft/NOT STABLE

REVERSE OF DA FORM 1248, 1 JUL 60

SAMPLE

Figure 2-2. Sample Road Reconnaissance Report, DA Form 1248 (continued)

The most important factor in planning military roads is making maximum use of the existing roadnet. Subject to the requirements of the tactical plan, the existing roadnet must be adapted to military use before undertaking new construction. Existing roads should be surveyed at the earliest opportunity to determine their condition and capacity. Time is saved by improving an existing road rather than constructing a new one.

Periodic road reconnaissance is conducted to obtain information about the road situation in a specific area. A situation map is prepared and kept current to show the condition of roads, the density of traffic, the need for maintenance work, and the results of maintenance. Periodic reconnaissance is important during wet or unusually dry weather to determine the effects of these conditions. Maintenance requirements based on periodic reconnaissance must be coordinated with the agencies using the roads to ensure proper standards of maintenance and to avoid work on roads no longer needed.

LOCATION RECONNAISSANCE

When a new road is necessary, the first step is the location reconnaissance. This requires reconnaissance of all possible routes to ensure selection of the best route. The main objective of a location reconnaissance is to locate a new road that will withstand anticipated traffic and provide the best possible operating conditions.

ENGINEER RECONNAISSANCE

Engineer reconnaissance is often conducted in conjunction with deliberate route reconnaissance to determine route conditions (including work estimates) and to locate construction materials to improve or maintain the route. It is either a general or special reconnaissance. General engineering reconnaissance gathers engineering information of a broad nature within the operational area to locate and evaluate construction materials, resources, terrain features, and facilities that have engineer implications. Special engineer reconnaissance obtains detailed information regarding an investigation of a specific site or evaluates the potential use of an undeveloped facility such as an airport or heliport. DA Form 1711-R is a required enclosure to the route reconnaissance, as specified in FM 5-36.

AIRFIELD RECONNAISSANCE

Airfield reconnaissance differs from road-location reconnaissance, described in FM 5-36, in the scope of information. An airfield project involves more personnel, machine-hours, and material than a road project. Air traffic imposes more severe limitations on its traffic facilities than vehicular traffic. Consequently, the site selected must be the best site available.

PLANNING AIRFIELD RECONNAISSANCE

Tentative airfield sites are selected within enemy territory using map and aerial photograph reconnaissance, supplementing data from reports of aerial observers or intelligence sources. These sites may be undeveloped potential sites or operating enemy installations. Reconnaissance should begin as soon as possible.

For an undeveloped potential site, the object of the reconnaissance is to verify or amend tentative selections and layouts and to estimate the material, equipment, and troop requirements for the construction planned. If it is a captured enemy airfield, a decision is needed on whether to use the captured field or develop a completely new site. Estimates of the engineering effort necessary to restore the airfield may also be required.

New airfields added to an area in which our aircraft are already operating can be developed in the following manner:

- Select the best available map of the area in which the new airfields are to be located. Draw a 5-mile-diameter circle around existing airfields and shade them. Note all high-tension, electric transmission lines and shade a 2-mile-wide strip centered on these lines. Locate and shade all similar obstructions on the map. Assault or hasty airfield selection is discussed in Chapter 10 of FM 5-430-00-2/Air Force Pamphlet (AFPAM) 32-8013, Vol 2.
- Confine the study for potential airfields to the unshaded parts of the map. Look for sites of sufficient area, preferably flat with good natural drainage, unobstructed air approaches, and accessibility to routes of communication. Assign the most likely sites to reconnaissance parties for appropriate air and ground investigation.

SELECTING RUNWAY LOCATION

A convenient way of selecting a runway location at a site that meets glide-angle requirements is to prepare and use the ah-field-siting template illustrated in Figure 2-3, page 2-16. This template can be drawn on acetate or heavy cellophane for use on any map to meet specifications for flight way, horizontal approach, and glide angle. When placed on the map, the template shows land forms and natural or manufactured obstacles that are in or above the plane of the glide angle.

In Figure 2-3, any hill within the approach zone at a distance of 8,000 feet from the end of the overrun and having an elevation of more than 160 feet above that of the end of the proposed runway, is in a 50:1 glide angle. This runway location is unsuitable according to the specifications. The template is useful to the reconnaissance officer and to the preliminary planning group. Prepared templates can measure distances in feet, yards, miles, and kilometers by placing gradations along their edges.

PROCEDURES FOR AIRFIELD AIR RECONNAISSANCE

The general procedure for an air reconnaissance follows:

En route to a particular site or a general area, the engineer notes open borrow pits, large stockpiles of construction material, rail and road accesses to the site, and errors on maps that have been studied. The pilot plays an important role on the reconnaissance team. Besides chauffeuring the engineer officer, the pilot considers approaches, mental hazards, and physical obstructions related to tactical aircraft that may use the proposed installation. A pilot who is familiar with operational requirements and the performance characteristics of tactical aircraft is more valuable than one who is not.

The engineer observer assesses possible construction problems at a potential site. The engineer selects tentative sites and directs questions to the ground reconnaissance party. The engineer receives the pilot's suggestions concerning the flying-related characteristics of the sites investigated and modifies estimates according to these recommendations.

To be effective as an engineer observer the officer should possess the following qualifications:

- Knowledge of road and airfield requirements and construction procedures and experience in airfield work.
- Immunity to airsickness. An airsick officer cannot effectively accomplish air reconnaissance. Any tendency of the engineer observer to become airsick is greatly enhanced by the continual concentration on a particular site and by

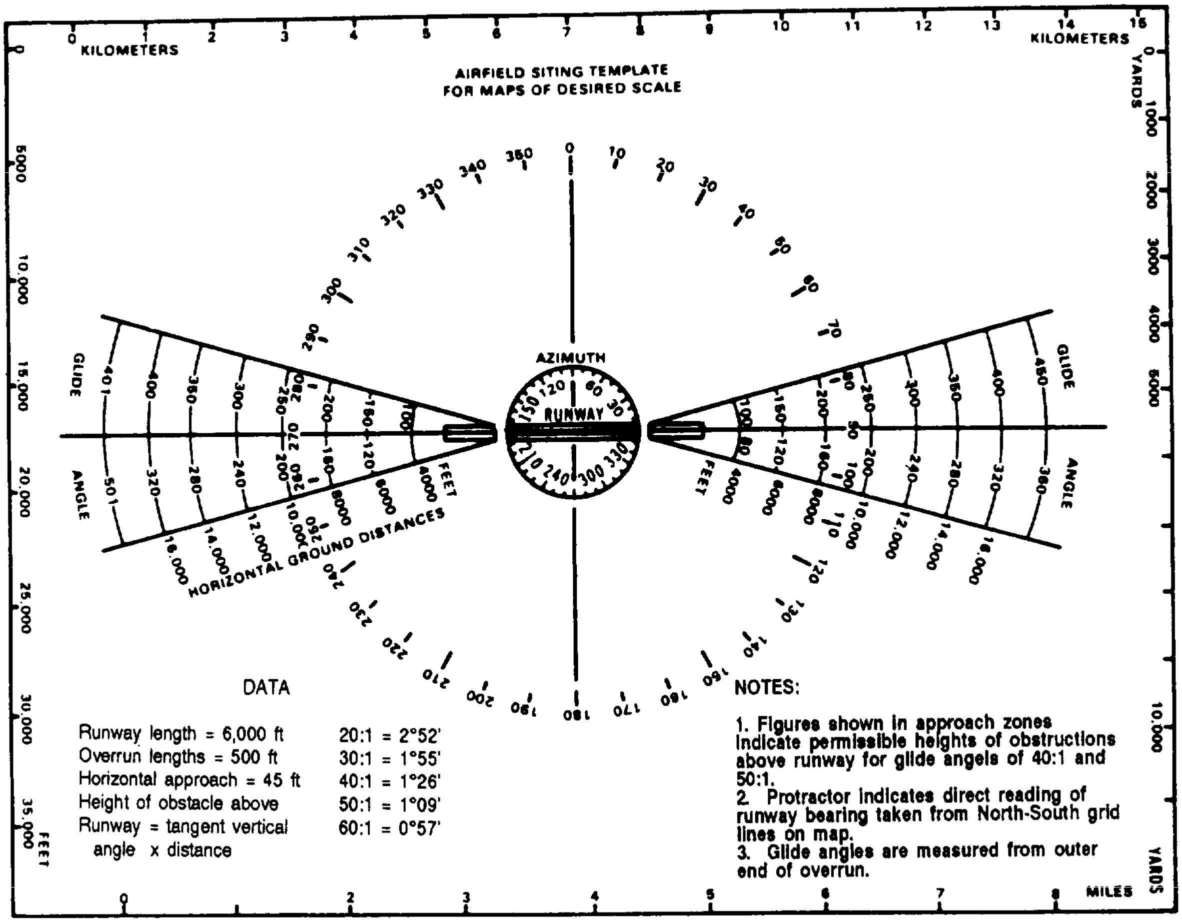


Figure 2-3. Sample airfield-siting template

the steep turns and maneuvers essential to continued observation.

- Proficiency in map reading. Upon approaching a designated or tentatively selected site for reconnaissance, the normal altitude for the first circuit is approximately 300 feet. Nothing more than orientation can be accomplished in this circuit. Sometimes a site tentatively selected during an area search can be eliminated during this circuit or the next few passes.

Similar second and third passes are flown. During these circuits, obstructions, main slopes, and general features are noted. The pilot begins to formulate an estimate of the flying-related characteristics of the field. Pinpoints for the ends of the runway are made on the map, but additional trips should be flown across the area, if necessary.

After the runway has been selected, an initial low pass is made at about 50 yards to one side of the proposed centerline. A second pass in the opposite direction is flown on the other side of the centerline. Both of these flights should be made at a constant air speed so the runway length can be estimated by multiplying the air speed by the average flight time.

NOTE: The length usually is overestimated when flying at low air speeds if a strong wind is blowing along the centerline. This can be decreased if the distances obtained by two passes in opposite

directions along the centerline are averaged (assuming the wind is constant).

A final circuit is then flown at approximately 200 feet. During this trip, the ends and centerline of the runway are given a final check, and the pilot completes the appraisal of the field's flying suitability.

In departing, the observer reviews dispersal areas and again checks access roads. Additional passes over the site are made if questions arise as a result of this last check. An air reconnaissance report similar to Figure 2-4, page 2-18, may be used.

An area reconnaissance then proceeds by similar inspection of other possible sites, Complete notes must be kept to avoid reviewing sites already checked however, a reinvestigation of the final site selected and any selected alternative sites may sometimes be necessary.

PROCEDURES FOR AIRFIELD GROUND RECONNAISSANCE

The general procedure for ground reconnaissance follows:

The ground-reconnaissance phase is preceded by map and air reconnaissance to discover what specific sites and questions warrant ground investigation.

En route to the site or sites to be investigated on the ground, the reconnaissance party should properly record the general condition of roads and bridges, the location of usable or repairable railheads, the locally available materials and equipment, and the potential water points. When reconnaissance of a definite site is involved, a more detailed observation of the access route should be made. A check must be made of bridge capacities, overhead clearances, and features that might hinder the movement of construction equipment to the site, as well as the suitability of railheads and sidings for use in construction. A detailed report of the quantity and quality of materials available at quarries, pits, and stockpiles must be prepared.

When the site to be surveyed is reached, the most likely locations for a runway must be investigated. If the terrain is open enough to permit good observation, these locations may be quickly determined. Locations for runways are traversed by vehicle or on foot. A rough survey of each selected runway is carried out immediately. Lengths are paced, critical slopes are measured with a clinometer, and directions are determined with a magnetic compass. The type of soil is noted and observations of a few samples are made. A preliminary check of a possible runway can be made in 15 minutes, if the country is reasonably clear and open.

If the country is rough and is not sufficiently open to permit a quick selection of runway locations, a detailed search must be made on foot. The reconnaissance officer, accompanied by necessary personnel, follows the centerline of the area for the runway and dispersal areas. The reconnaissance officer notes on a large-scale map or sketch all obstacles that cannot readily be eliminated, such as gullies, rock outcrops, and swampy areas. Examination of the results discloses the possible runway locations.

The best runway location is selected by considering these centerline investigations with prevailing wind direction, air approaches, glide angles, groundwater conditions, discharge areas for collected runoff, clearing, grubbing, and earthwork. If a suitable runway does not exist, a negative report on the site is submitted.

Once the selection of a potential runway is made, a careful and detailed walk of the centerline of each runway is made to recheck its suitability. Stakes are driven at each end of the runway and prominent features are properly referenced to later expedite the location of the selected runway by construction unit surveyors.

The survey sergeant of the reconnaissance party stakes out the centerline of the runway and runs a ground profile of it at the centerline and at each shoulder line. Levels are taken at 500-foot intervals and

AIR RECONNAISSANCE REPORT

DATE 05 JUL 92 NO. 4

1. To CO 327 ENGR BN 3. Sheet JOHANNASVILLE QUADRANGLE

2. From CO. C 4. 10 MILES NORTH OF JOHANNASVILLE
(Nearest main road center)

5. (a) Coordinates of EAST end of runway N3 765, E1 900

(b) Length (feet) 5000 FT. BUT MIGHT BE EXTENDED 2000 FT. (SEE ITEM 12)

6. Classification of Site (overall):

Excellent ____ Good ✓ Fair ____ Poor ____ Reject* ____

7. Natural Surface Drainage:

Excellent ____ Good ✓ Fair ____ Poor ____

8. Flying Approaches:

Excellent ____ Average ✓ Poor ____

9. Clearing:

Light ✓ Moderate ____ Excessive ____

10. Aircraft Dispersal:

Unlimited ✓ Adequate ____ Inadequate ____

11. Access Roads:

Good ____ Adequate ✓ Inadequate ____

12. Remarks:
Extension mentioned in 5(b) above must be checked as there may be a swamp area in that suggested extension. Cannot be certain from air observation.

CPT MARK KUEHL
(Signature)

1400
(Time)

*If "Reject" classification is indicated, reason(s) for same will be given under remarks.

Figure 2-4. Air Reconnaissance Report

at intermediate breaks or slope changes. In flat country, this interval may be increased to as much as 1,000 feet. If an alternative runway is selected, a similar survey is conducted for that runway, if time permits.

The soils analyst conducts a field investigation of the soil conditions at the site, Refer to Chapter 7 of this manual for more information about soil conditions.

Previously acquired information is checked at the site for accuracy. Errors, including discrepancies on maps and mistakes in aerial photograph interpretations, are included in the report. A suggested report format is shown in Figure 2-5:

When possible, local inhabitants are interviewed to check information already obtained and to obtain more information. Several opinions should be obtained. Questions should be phrased to provide the best comparison of answers. Information must be weighed carefully with regard for the credibility of each person questioned.

The reconnaissance of a designated site should be accomplished in one day, unless

RECONNAISSANCE REPORT
UNDEVELOPED AIRFIELD SITE

TO Commanding Officer 327 ENGR BN

FROM CO C CP ______ DATE 29 SEPT 93

Note — The reconnaissance party must be furnished with the following information

a Location of airfield general or specific

b Type of aircraft that will occupy the airfield

c Number of groups expected to occupy the airfield

1 DESIGNATION Name CHELTENHAM AIRFIELD Number 1

2 LOCATION BAYS VILLAGE OF MARYLAND MD DEPT

a Map reference OF PUBLIC IMPROVEMENTS SCALE 1"=200' (NAME and SCALE) b Elevation 225 FT

c Latitude & longitude 38° 44'N LAT 76° 40' WEST LONG Map coordinates N328018 E844109

d Nearby towns CHELTENHAM (POP 100) 1.5 MI DUE EAST FROM SITE
(Size distance and direction from site)

3 ROADS US301 AT CHELTENHAM EXCELLENT BITUM GOOD GRAVEL ROAD FROM SITE WEST TO LOCAL ROAD NET
(Type condition bridges fords etc)

4 RAILROADS PENNSYLVANIA RR SIDING AT TOWN, 2 MILES EAST OF SITE.
(Gage condition distance from site siding capacity tunnels bridges)

5 GENERAL DESCRIPTION OF LANDING AREA AND SURROUNDING COUNTRY AGRICULTURAL AREA US NAVAL RADIO STATION SITE NOW DETENTION HOME FOR DELINQUENT BOYS.

6 GLIDE ANGLES NEARLY UNLIMITED IN MOST DIRECTIONS SEE ITEM 7
(Direction slope distance)

7 FLIGHT OBSTRUCTIONS AND MENTAL HAZARDS ANTENNA, FARM AT N.R.S IS 1.5 MI N OF SITE; ELEVATED WATER TOWER AT 3 V MI E OF SITE.
(Ravines cliffs mountains timber steeples chimneys towers power lines etc)

8 METEOROLOGICAL CONDITIONS WESTERLY WINDS - LITTLE FOG - NORMAL MD PRECIPITATION AND RAIN
(Prevailing winds storms frost precipitation temperature visibility)

9 HYDROLOGICAL CONDITIONS STREAMS ON EACH SIDE OF SITE RUN TO PISCATAWAY RIVER
(Streams ground water flood conditions tidal variations)

10 DRAINAGE GOOD NATURAL DRAINAGE
(Flat sloping direction number of culvert plan)

11 SOIL TYPES AND GEOLOGICAL DATA CLAY AND GRAVEL
(Sand gravel clay silt or combinations rock outcrops coral tuff caliche)

12 CLEARING NO EXTENSIVE CLEARING, FEW ISOLATED TREES - 4 ABANDONED FRAME BUILDINGS - WOULD HAVE TO BE REMOVED.
(Area size and density of timber rocks)

13 PROPOSED LAYOUT NE-SW RUNWAY USE EXISTING BLDGS 2500 FT RUNWAYS SUITABLE FOR C-130s AND C-123s
(Location and direction of runway and dispersal system (attach sketch))

14 RECOMMENDED SURFACING AM2 OR SIMILAR PORTABLE SURFACING
(PSP SMT PRS mechanical stabilization coral tuff)

15 CAMOUFLAGE LITTLE NATURAL CONCEALMENT AFFORDED, ADEQUATE DISPERSAL POSSIBLE.
(Cover concealment dispersion deception)

16 BIVOUAC AREAS USE EXISTING DORMITORIES - WILL ACCOMMODATE 450 MEN.
(Location size cover previously occupied)

17 WATER SUPPLY PUMPS, ELEVATED TANK AND DISTRIBUTION SYSTEM IN OPERATION, SMALL STREAMS AFFORD ADDED FIRE PROTECTION.
(Source location quantity quality)

18 EXISTING FACILITIES SEWAGE TREATMENT NEEDS REPAIR, POWER PLANT OPERABLE; ELECTRIC WIRING SUITABLE GENERATORS MUST BE INSTALLED.
(Buildings storage power water sewerage)

19 MATERIALS AVAILABLE GRAVEL PIT 4 MI S. MUCH TIMBER NEAR PIT. CINDER PILE AT POWER PLANT. WAREHOUSE SPACE IN VILLAGE.
(Equipment borrow pits gravel banks quarries mine dumps timber)

20 WORK ESTIMATE QUANTITIES

a Clearing NEGLIGIBLE
(Acres size of timber density)

b Drainage OPEN DITCHES AROUND PERIMETER OF RUNWAY AND TO STREAM
(Linear feet of open ditching number of culverts amount of pipe and approximate diameter)

c Earth moving ABOUT 4000 CUBIC YDS NO BORROW, SPOIL OR LONG HAUL.
(Estimate for R W T W H S)

d Surfacing LAY ABOUT 120,000 SQ FT AM2
(Quantity for R W T W H S recommended type)

e Roads access and service ACCESS EXIST SOME SERVICE - NEED SUPPLEMENTING
(Miles condition)

f Buildings PRESENT BLDGS SUITABLE BUILD CONTROL TOWER.
(Number and size suitability for operations quartering)

21 TIME ESTIMATE FOR COMPLETION 4 CO - DAYS
(Bn days or Co days)

22 ADDITIONAL INFORMATION 1"=40' TOPOGRAPHIC MAPS AVAILABLE FROM MD DEPT OF PUBLIC IMPROVEMENTS, CIVILIAN PERSONNEL IN AREA COULD BE USED FOR CONSTRUCTION NATIVES APPEAR FRIENDLY. PART OF PROPOSED RUNWAY UNDER CONSTRUCTION. DAIRY HERD AND MILK PROCESSING PLANT AT SCHOOL

23 ANNEXES SEE ATTACHED MAP AND OVERLAY
(Maps photographs sketches estimates soil samples)

24 SIGNATURE [signature]
(In charge of reconnaissance party)

Figure 2-5. Ground Reconnaissance Report - Undeveloped Airfield Site hostile forces delay the work. A specific reconnaissance of a captured enemy airfield is somewhat different from that outlined above. Detailed information about the existing facilities and their condition is desired. The specific information needed is indicated on the suggested form for reconnaissance reports of captured enemy airfields shown in Figure 2-6.

When the reconnaissance parties are operating at a considerable distance from the headquarters directing the reconnaissance, it is imperative that an initial report reach headquarters without delay. Use organic radio equipment and the suggested message format in Figure 2-7, page 2-22. The tactical situation may dictate the amount of information transmitted. Unit standing operating procedures (SOPS) should indicate what information is critical for radio reports. A complete, written report should follow the radio report.

The formats illustrated in Figure 2-4, page 2-18; Figure 2-5, page 2-19; and Figure 2-6, are suggested for written reports. The reports should include the same items of information shown on these forms. Suitable sketches should be attached to all written reconnaissance reports. Figure 2-8, page 2-23, is a typical sketch.

RECONNAISSANCE REPORT
CAPTURED ENEMY AIRFIELD

TO Commanding Officer 327 ENGR BN

FROM CO C CP ______ DATE 29 SEPT 93

1 DESIGNATION Name BONGO BONGO AIRFIELD Number 1

2 LOCATION PRINCIPALITY OF BONGO

a Map reference MAP #3 SCALE 1:5000 Elevation 7500 Ft
(Name and Scale) (MSL)

c Latitude and longitude 40 53N, 37 23E Map coordinates NS 750, E3 235

d Nearby towns BONGO BONGO IS 28 MILES N E OF SITE POPULATION ABOUT 4000
(Size distance and direction from site)

3 RUNWAYS

	Length	Width	Grid az	Surface	Possible extension
No 1 EW RUNWAY	4700	80	85°-265°	CONCRETE	+ 2300 Ft
No 2 N.S. RUNWAY	2700	60	350°-170°	CONCRETE	NONE
No 3 NONE					

4 HARDSTANDS Number 2 Type 1-300 FT x 300 FT N OF MAIN RUNWAY (CONCRETE) 1-300 FT x 300 FT E OF AUXIL RUNWAY (CONCRETE)

5 CONDITION OF AIRFIELD FACILITIES

a Runways MAIN RUNWAY EXCELLENT BUT EAST SECONDARY RUNWAY FAIR BUT NEEDS MINOR REPAIRS MOST 1000 FT NEEDS REBUILDING

b Taxiways GOOD

c Hardstands ABOUT HALF OF EACH USEABLE, ADJACENT TURF IN EXCELLENT SHAPE FOR LIGHT CRAFT

d Roads (Classify using symbols)

(1) Access FROM EAST A23/29 FT Kb, FROM NORTH A24/30 FT Kb

(2) Service A 11/17 Kb

e Gasoline storage EMBANKMENT-PROTECTED DRUM STORAGE AREA; NO TANKS ON PIPE LINE.
(Number of tanks capacity U S gals)

f Bomb and ammunition dumps SIMILAR TO ITEM E

g Serviceable hangars NONE IN SERVICEABLE OR REPAIRABLE CONDITION BURNED BUT CAN BE
(Size type condition)

h Office space available ONE 3 STORY BRICK BLDG 80 FT x 50 FT - EASILY RESTORED
(Number of buildings sq ft)

i Shop space ONE OPEN ROOFED AREA 300 FT x 20 FT W/WOOD BLOCK FLOOR FORMERLY USED AS SHOP AREA, ALMOST NO EQUIPMENT REMAINING.
(Number of buildings sq ft)

j Storage space Covered 2-FRAME BLDG, 20 [illegible] x 30 FT Open 1

k Drainage system STORM SEWER SYSTEM SLIGHTLY DAMAGED-NEEDS MINOR REPAIR, OUT FALL OF SYSTEM AT Open BOBANGBASS RIVER [illegible] SOUTH SIDE OF INSTALLATION
(Type extent and location of damages location of outfall)

l Utilities

(1) Water supply RIVER IS SOURCE NO PURIFICATION SYSTEM- FOR FIRE PROTECTION PUMPS AND PIPES ADEQUATE

(2) Sewerage system OUTDOOR LATRINE SYSTEM NEEDS CLEANING + REPAIR OR REBUILDING.

(3) Electricity WIRING ADEQUATE-GENERATOR SYSTEM DAMAGED BEYOND REPAIR

(4) Heating systems (type) NONE NEEDED

m Other facilities CONTROL TOWER, ADEQUATE-20 TARGET RIFLE RANGE 20 MILES N.
(Telephone control tower shooting ranges butts siding capacity)

n Railroads DOUBLE TRACK RR BONGO BONGO TO PORT SINGLE TRACK NARROW GAGE RR BONGO BONGO TO AIRFIELD, 28 MILES.
(Distance from airfield siding capacity condition)

6 PERMANENT TROOP HOUSING ON OR ADJACENT TO AIRFIELD
Capacity Officers 2-10 MAN BARRACKS EM 4-50 MAN BARRACKS

7 BIVOUAC AREAS PUBLIC PARK AREA ADJOINING AIRFIELD SUITABLE FOR BIVOUAC AREA. ABOUT 3 WOODED ACRES.
(Location area cover previously occupied access road net)

8 UNDERGROUND SHELTER AVAILABLE NONE
(Number rooms height size)

9 DAMAGE CAUSED BY BOMBING AND DEMOLITION NONE EXCEPT AS PREVIOUSLY NOTED.

10 SOIL TYPE & GEOLOGICAL DATA SANDY
(Sand gravel clay silt or combinations rock outcrops coral tuff caliche)

11 HYDROLOGICAL CONDITION BOBANGBASS RIVER ABOUT ½ MILE WIDE AT AIRFIELD FLOWS TO PORT [illegible] BONGO NAVIGABLE BY RIVER
(Streams ground water flood conditions)

12 METEOROLOGICAL CONDITIONS PREVAILING WINDS SOUTHERLY BUT EXTREMELY LIGHT. PRECIPITATION ABOUT 20 IN PER YEAR, NO OUTSTANDING WET SEASON.
(Prevailing winds storms precipitation temperature visibility)

13 CONSTRUCTION MATERIALS AND EQUIPMENT 20 CARLOADS OF CEMENT ON SIDING. GRAVEL PIT ON OTHER SIDE OF RIVER FROM AIRFIELD
(Gravel sand cement asphalt lumber glass tile hardware quarries timber machinery borrow pits mine dumps)

14 ADDITIONAL INFORMATION CHIEF OF VILLAGE LIVES NEAR AIRFIELD. NATIVES UNFRIENDLY. CHIEF SAYS PROBABLE SABOTAGE BY NATIVES OF VILLAGE

15 ANNEXES SEE ATTACHED SKETCH
(Maps photographs sketches overlays soil samples)

16 SIGNATURE [illegible]
(In charge of reconnaissance party)

Figure 2-6. Ground Reconnaissance Report - Captured Enemy Airfield

AIR LANDING AREA REPORT

Air Landing Sites Letter designation	Explanation
A	Map sheet(s)
B	Date and time of collection of information
C	Location (grid references)
D	Runway (1) Bearing (2) Length and width (3) Gradients exceeding standards (4) Rough appraisal of earth work (5) Feasibility of runway extension
E	Drainage
F	Major obstacles to flying (1) Within the approach zone (2) Outside the approach zone but within 5 miles
G	Type of soil
H	Whether suitable area for dispersals can be found
I	Local resources
J	Approach roads

Airstrips (Runways) Letter designation	Explanation
A	Map sheet(s)
B	Date and time of collection of information
C	Location (grid references)
D	Dimensions
E	Type and condition of the airstrip
F	Access by road
G	Feasibility of runway extension
H	Any other information such as work required, in man-hours, to make the airstrip serviceable for sustained or limited operations

Report air landing areas by serial number. The appropriate letter designation must precede each category of information reported.

Figure 2-7. Air Landing Area Report

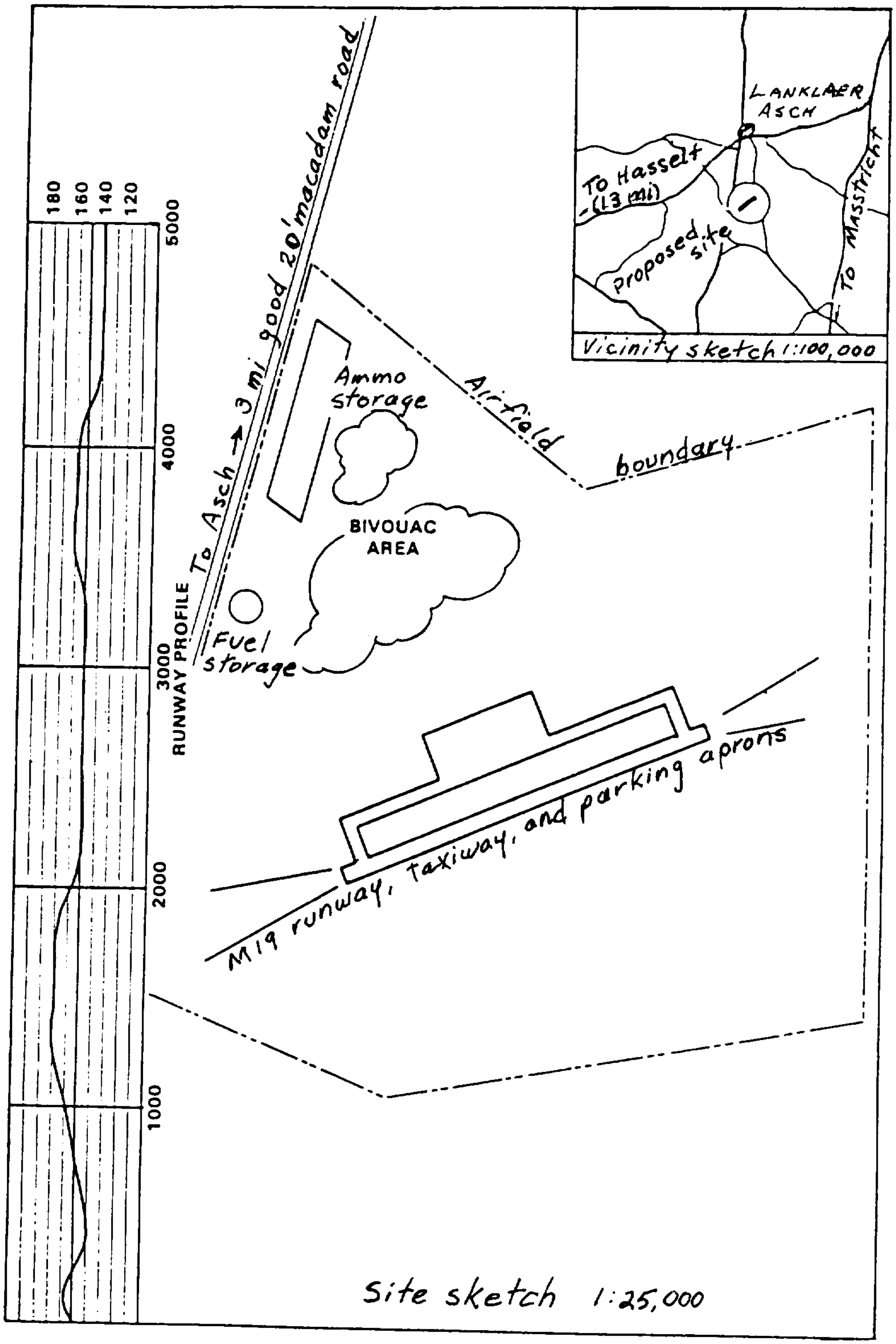


Figure 2-8. Typical sketch to accompany airfield reconnaissance report

SURVEYS AND EARTHWORK OPERATIONS

CHAPTER 3

Construction surveys are initiated when new construction is necessary. These surveys reveal the kinds of stakes to be used: provide data for earthwork estimation, including which method of estimation to use; and provide information for use on the mass diagram. The finished survey books should be filed with the construction project records of the Operations and Training Officer (US Army) (S3).

Earthwork operations are one of the most important construction aspects in road and airfield construction. Earthwork reqires the greatest amount of engineering effort from the standpoint of personnel and equipment. Therefore, the planning, scheduling, and supervision of earthwork operations are important in obtaining an efficiently operated construction project.

CONSTRUCTION SURVEYS

Construction surveying is the orderly process of obtaining data for various phases of construction activity. It includes the following surveys: reconnaissance, preliminary, final location, and construction layout. The reconnaissance and preliminary surveys are used to determine the best location. The remaining surveys are conducted after a location has been established.

The purpose of construction surveys is to control construction activities. The number and extent of surveys conducted is governed by the time available, the standard of construction desired, and the

ducted for a deliberate project in the communications zone. The quality and efficiency of construction is directly proportional to the number and extent of surveys and other preplanning activities. The principles and techniques of field surveying are discussed in detail in technical manual (TM) 5-232 and FM 5-233.

After completing a thorough construction survey, transfer the design information from paper to the field by construction stakes. These stakes are the guides and reference markers for earthwork operations.

RECONNAISSANCE SURVEY

The reconnaissance survey provides the basis for selecting acceptable sites and routes and furnishes information for use on subsequent surveys. If the location cannot be selected on the basis of this work, it must be determined by the preliminary survey.

PRELIMINARY SURVEY

The preliminary survey is a detailed study of a location tentatively selected on the basis of reconnaissance, survey information, and recommendations. It consists of running a traverse along a proposed route, recording topography, and plotting results. For roads, it may be necessary to conduct several preliminary surveys if the reconnaissance party has investigated more than one suitable route. Establish, station, and profile the route centerline with horizontal and vertical control points set. Take cross-section readings to allow rough calculations of the earthwork involved. (Sometimes cross sections may be taken during the reconnaissance survey if the conditions warrant.) If the best available route has not been chosen, select it at this time.

The airfield survey consists of establishing controls, noting terrain features, measuring glide-angle clearance, making soil profiles, and investigating drainage patterns and approaches. Accurately establish the final centerline during the survey.

FINAL LOCATION SURVEY

When time permits, conduct a final location survey. Establish permanent bench marks for vertical control and well-marked points for horizon tal control. These points arc called hubs because of the short, square stake used. On most surveys, the hub is driven flush with the ground, and a tack in its top marks the exact point for angular and linear measurements. The hub location is indicated by a flat guard stake extended above the ground and driven at a slope so its top is over the hub. Hubs are 2 inches by 2 inches and the guards are flat stakes, about 3/4 inch by 3 inches.

Horizontal Control

The purpose of horizontal control is to accurately determine points for the various facilities of an engineering project. Establish permanent, well-marked points for horizontal control and reference them at the site before construction begins. On a large facility, establish a grid network and use it for this control. Tie the network into the military grid system in the particular area, if such a system has been established. On an airfield, place control points beyond the clear zone. These points define the centerline of the runway and other important sections of the airfield.

As the taxiways and other facilities are laid out, establish and reference new control points. In laying out the centerline, place target boards at each end of the runway so the instrument person can make frequent checks on alignment while the line is being staked out. Target boards may be set up on any line that requires precision alignment. Reference control stakes to ensure replacement, if they are disturbed or lost. Locate the target board just beyond the outermost control-point stake.

Vertical Control

Vertical control methods determine the difference in elevation between points. If available, establish a level reference surface or datum from a known bench mark. Differences in elevation, with corrections, are subtracted from or added to this assigned value, resulting in the elevation of the points. Take the datum of the bench mark system from a known elevation or barometer reading or make an arbitrary assumption.

CONSTRUCTION LAYOUT SURVEY

The construction layout survey is the final preconstruction operation. It provides alignments, grades, and locations that guide construction operations. The survey includes determining exact placement of the centerline; laying out curves; setting all remaining stakes, grades, and shoulders; staking out necessary structures; laying out culvert sites; and performing other work required to begin construction. Continue this survey until construction is completed.

CONSTRUCTION STAKES

Use construction stakes for centerline, slope, offset, shoulder, grade, reference, ditch, culvert, and intermediate slakes and for temporary bench marks. The stakes should be approximately 1 inch by 3 inches by 2 feet. Use finished lumber when possible. If it is not possible to usc finished lumber, usc small trees or branches blazed on both sides and cut to length. Finished grade stakes and temporary bench marks are 2 inches by *2* inches by 12 inches. Place stakes using a three- to five-person crew equipped wtth transit, level, rod, tape, ax, sledgehammer, and machete.

The primary functions of construction stakes are to indicate facility alignment control elevations, guide equipment operators, and eliminate unnecessary work. They also determine the width of clearing required by indicating the limits of the cut and fill at right angles to the centerline of a road.

Mark and place construction stakes to conform to the planned line and grade of the proposed facility. Use colored marking crayons to mark the stakes. Use a uniform system so the information on the stakes can be properly interpreted by the construction crew.

Construction stakes indicate-

- The stationing or location of any part of the facility in relation to its starting point. If the stake is located at a critical point such as a point of curvature (PC), point of intersection (PI), or point of tangency (PT) of a curve, note this on the stake.
- The height of cut or fill from the existing ground surface to the top of the subgrade for centerline stakes or to the shoulder grade for shoulder or slope stakes.
- The horizontal distance from the centerline to the stake location.
- The side-slope ratio used on slope stakes.

The number and location of stakes used differ between roads and airfields. A typical set of construction stakes consists of a centerline stake and two slope stakes and is referred to as a three-point system. Point one is the centerline of the facility. Points two and three are the construction limits of the cut and fill at right angles to the centerline.

CENTERLINE OR ALIGNMENT STAKES

The centerline or alignment (hub) stakes, shown in Figure 3-1, are placed on the centerline of a road or air field and indicate its alignment, location, and direction. They are the first stakes placed and must be located accurately. These stakes are used as reference points in locating the remaining stakes. Centerline stakes are placed at 100-foot (or 30-meter) intervals. On rough ground or sharp horizontal and vertical

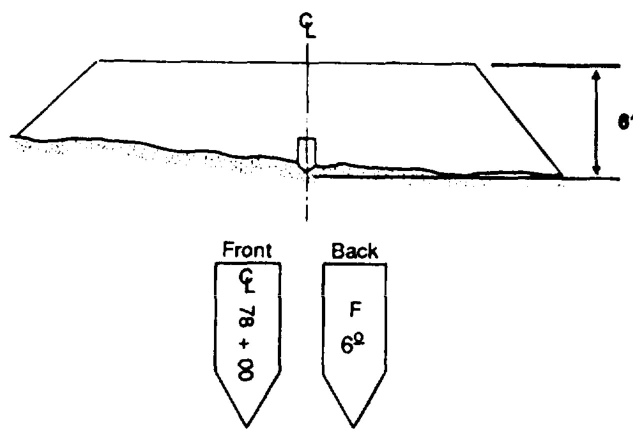


Figure 3-1. Centerline stakes

curves, place the stakes closer together. On horizontal curves, also stake the PC, PI, and PT. On vertical curves, also stake the point of vertical curvature (PVC), the point of vertical intersection (PVI), the point of vertical tangency (PVT), and the low point (LP) or high point (HP) of the curve.

Place centerline stakes with the broad sides perpendicular to the centerline. The side of the stake that faces the starting point is the front. Mark the front of the stake with a ℄ for centerline and, if applicable, PC, PI, or PT. Also mark on the front the distance from zero or the starting point in 1 00-foot stations and the fractional part of a station, if used, For example, $6 + 54^{\underline{22}}$ marked on a stake indicates it is 654.22 feet from the origin of the facility and is known as the station of this point. Stations are used in locating sections of construction and in preparing reports.

PIace the amount of cut or fill required at the station on the reverse side of the stake. A cut is marked C; a fill, F. A centerline stake, placed at station 78 + 00 and requiring a fill of 6.0 feet to bring this station up to the final grade line, would be placed and shown as indicated in Figure 3-1, page 3-3.

The amount of cut or fill indicates the difference between the final grade line and the ground line where the stake is emplaced. A point on the stake is seldom used as the line of reference to the final grade.

To prevent misinterpretation of the amount of cut or fill, mark decimal parts of a foot, as shown in Figure 3-1. The decimal part is written smaller, raised, and underlined. Facing the direction of increasing stations, the centerline forms the dividing line between the right and left sides of the area to be graded. When facing either side of the centerline, it is customary to refer to the areas as the right or left side.

SLOPE STAKES

Slope stakes, shown in Figure 3-2, define the limits of grading work. When used in read work, they can be used as guides in

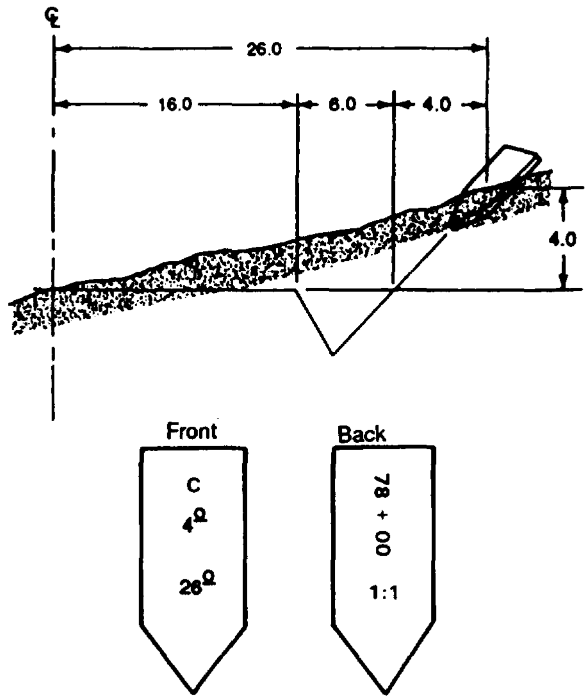


Figure 3-2. Marking and placement of slope stakes

determining the width of clearing necessary. The area to be cleared usually extends 6 feet beyond the slope stakes. Set slope stakes on lines perpendicular to the centerline (one on each side), at points where the cut and fill slopes intersect the natural ground surface. Stakes at points of zero cut or fill are placed sloping outward from the centerline.

Sloping the stakes outward allows the equipment to work to the stake without removing it. The slope indicates the direction of the centerline of the road and enables the equipment operators to read the stakes more easily. Place slope stakes at 100-foot intervals on tangents and at 50-foot intervals on horizontal or vertical curves, Whenever a sharp break in the original ground profile occurs, it should be staked.

The front of a slope stake is the side facing the centerline. On this side of the stake, mark the difference in elevation between the natural ground elevation at this point and the finished grade at the edge of the shoulders. Under this figure, place another figure that indicates the horizontal distance from the centerline of the road to the slope stake. Place the station number on the other side of this stake. Below the station number, indicate the appropriate slope ratio. Figure 3-2 shows the proper markings for a slope stake in a typical situation.

OFFSET STAKES

Equipment used on a cut or fill section may destroy or remove many of the grade (centerline, shoulder, or slope) stakes. To prevent loss of man-hours and repetition of survey work, caution construction crews to protect grade stakes whenever possible. Place offset stakes beyond construction limits to avoid resurveying portions of the road to relocate these stakes. Figure 3-3 shows offset stakes used to relocate the original stakes.

Place offset stakes on a line at right angles to the centerline of the facility. From these, the slope stakes can easily be located. After relocating a slope stake, relocate the centerline stake by measuring toward the centerline of the road the horizontal distance indicated on the slope stake and placing the new centerline stake there.

An offset stake contains all the information given on the original slope stake plus the difference in elevation and horizontal distance from the original slope stake to the offset stake. Mark the offset distance on the front of the stake and circle it to indicate it is an offset reference. If the offset stake is at a different elevation from the slope stake, the cut or fill value must be increased or decreased by the difference in elevation. An offset stake placed a horizontal distance of 10 feet from and 1 foot above the right slope stake would be placed and marked as shown in Figure 3-3. Coordination between the surveyor and grade supervisor concerning the meaning of the markings is most important regardless of the type of marking used.

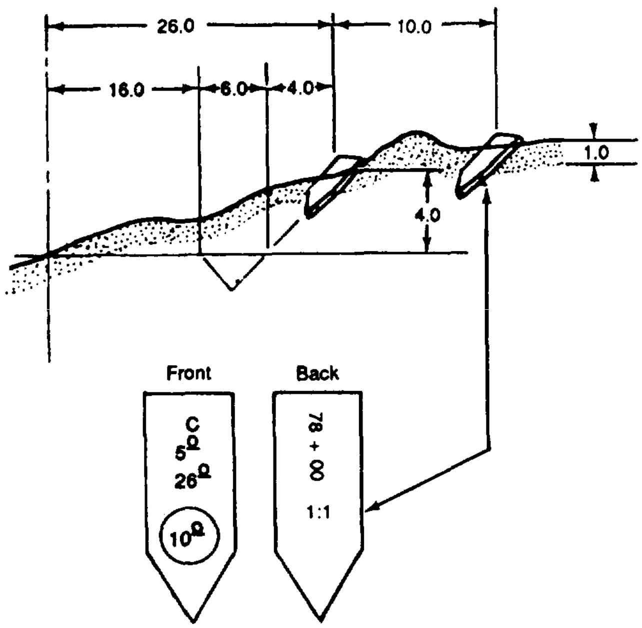


Figure 3-3. Marking and placement of offset stakes

FINISH-GRADE STAKES

Use wooden stakes, 2 inches by 2 inches, with tops colored red or blue, for finish-grade stakes. Blue or red tops, as they are called, indicate the actual finished elevation of the final grade to which the completed facility is to be constructed. They are used when the grade is within a short distance of the final elevation. Do not use these stakes in combat road construction except in areas with steep slopes. This type of stake normally requires a guard stake to protect it and indicate its location. On large projects, it may be impractical to use guards with each stake.

There are no markings on finish-grade stakes other than the color on the top. These stakes may be set for use with the top of the stake exactly at the finished grade or with the top of the stake above the finished grade, as decided upon by the surveyor and construction foreman.

With the stakes set and marked at a predetermined distance above the finished grade, stretch a string between two stakes across the work and use a graduated ruler or stick to check the elevation. On an airfield layout, place these stakes along the centerline, edge of pavement, intermediate lines, shoulder lines, and ditch slopes. For road work, place stakes along the centerline and the edge of the shoulder; they may or may not be placed on the slopes.

REFERENCE STAKES

Many hubs marking the location of highways and airfields are uprooted or covered during construction. They must be replaced, often more than once, before construction is completed. As an aid in relocating a point which may become hidden by vegetation, or as a means of replacing points which may have been destroyed, measurements are made to nearby permanent or semipermanent objects. This process is known as referenceing or witnessing a point. On many surveys, permanent objects may not be available as witnesses. In such cases, additional stakes may be driven. These stakes usually are approximately 2 inches by 2 inches by 18 inches.

There are no markings on a reference stake. A point can be referenced by a known distance and a known angle or by two known distances. A transit must be used in the first case and may be used to advantage in the second, The method of using two known distances can be used, however, when a transit is not available. Place two points at measured distances from the point to be referenced. Use two tapes to relocate the original point or stake. Hold the zero end of one tape on one reference point and the zero end of the other tape on the other reference point. The point of intersection of the two tapes at the respective distances gives the location of the point in question.

To be of most value in replacing a missing station or point, the reference stakes or witnesses will be less than 100 feet from the point and, if possible, the arcs should intersect at approximately right angles. Place them outside the construction limits, and indicate their location by blazing trees or additional stakes. Normally, the location of the reference stakes can be obtained from the surveyor's notebook.

CULVERT STAKES

Culvert stakes are located on a line parallel to and offset a few feet from the centerline. The information required on the culvert stakes includes the distance from the stake to the centerline, the vertical distance to the invert, and the station number. Once the survey crew has finished staking out the culvert, the construction supervisor can place the pipe accurately by using batter boards.

BENCH MARKS

Vertical control of a road or airfield must be maintained during construction. To do this, points of known elevation must be established. Obtain elevations from permanent monuments, known as bench marks, established by geodetic surveys. From these bench marks, run a line of levels and set temporary bench marks (TBMs). On small projects the TBMs frequently are set by running the levels from a point of assumed elevation. This is especially true of construction in combat areas.

Usually, TBMs are placed at 500- to 1,000-foot (or 150- to 300-meter) intervals and are placed off the limits of construction. Stakes 2 inches by 2 inches, solidly emplaced in the ground, may be used for this purpose. However, a nail driven into a tree, a manhole cover, or a pipe driven into the ground may also be used. Frequently, reference points serve as TBMs. The TBMs are set before setting the centerline stakes because vertical control must be established before construction begins.

EARTHWORK ESTIMATION

Earthwork computations involve the calculation of earthwork volumes, the determination of final grades, the balancing of cuts and fills, and the planning of the most economical haul of material. The exactness with which earthwork computations are made depends upon the extent and accuracy of field measurements, which in turn arc controlled by the time available and the type of construction involved. To plan a schedule, the quantity of earthwork and the soil and haul conditions must be known so the most eficient type and quantity of earthmoving equipment can be chosen and the appropriate time allotted.

When time is critical, the earthwork quantities are estimated either very roughly or not at all. When time is not critical, higher construction standards are possible and earthwork quantities are estimated and con trolled by more precise methods.

FUNDAMENTAL VOLUME DETERMINATION

The volume of a rectangular object may be determined by multiplying the area of one end by the length of the object, This relationship can be applied to the determination of earthwork by considering road cross sections at the stations along the road as the end areas and the horizontal distance between cross sections as the lengths. The end areas of the cross sections must be computed before volumes can be calculated.

METHODS OF END-AREA DETERMINATION

When the centerline of the construction has been located, measurements are taken in the field from which the required quantities of cut or fill can be computed. A cross-sectional view of the land is plotted from these measurements. The cross sections are taken on vertical planes at right angles to the centerline. Where the ground surface is regular, cross sections are taken at every full station (100 feet), Where the ground is irregular, they must be taken at intermediate points as determined by the surveyor. A typical cross section is shown in Figure 3-4.

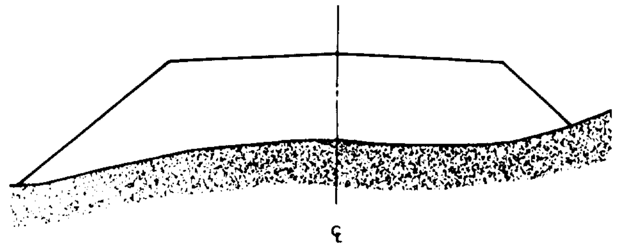


Figure 3-4. Typical fill cross section

Plot ground elevations from the surveyor's notes. Make a sectional template of the subgrade that shows the finished subgrade and slopes plotted to the same scale as the cross sections. Superimpose the template on the cross section and adjust it to the correct centerline elevation. Trace the template and extend the side slopes to intersect the original ground. If the section involves both cut and fill, draw only the appropriate lines of each template. When the sections are completed, begin the end-area measurements, then determine the volume. Of the several satisfactory methods of measuring the end areas, only the trapezoidal, stripper, double-meridian (triangular), and planimeter methods will be described in this manual. The method chosen will depend upon the time available, the accuracy desired, the aids at hand, and the engineer's preference.

Trapezoidal Method

The trapezoidal method is widely used to determine end areas. The computations are tedious, but the results are accurate. In using the trapezoidal method, the area of any cross section is obtained by dividing the cross section into triangles and trapezoids, computing the area of each part separately, and taking the total area of the verticals to the ground line (Figure 3-5, page 3-8) in order to divide the cross section into two triangles and two trapezoids. Make the assumption that the ground is perfectly straight between these selected points on the ground line. While this is not usually correct, the assumption is within the accuracy normally required.

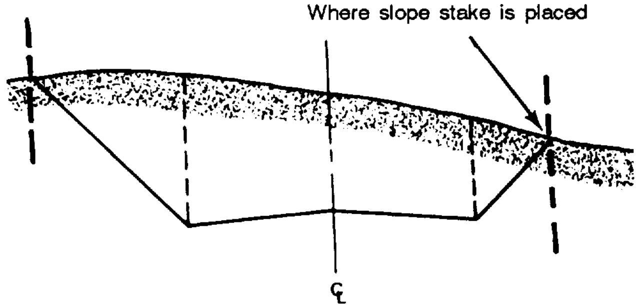


Figure 3-5. Cross section in cut with verticals drawn at critical points

Basic Formulas. Before the area of the cross section can be computed, the basic formulas for the computation of the areas of triangles and trapezoids must be understood. If a line is drawn, as shown in Figure 3-6, from one of the vertices of a triangle perpendicular to the side or base (b) opposite this vertex, the line formed represents the altitude (h) of the triangle. The area of any triangle can be expressed as the product of one-half the base multiplied by the altitude. This relationship is expressed by the formula:

$$A = \tfrac{1}{2}bh$$

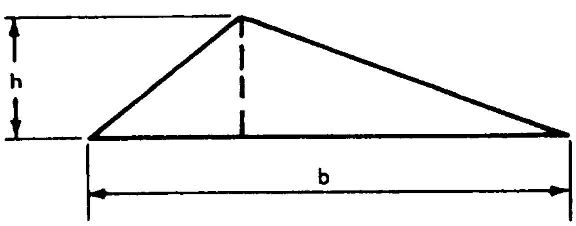


Figure 3-6. Triangle base and height dimension locations

A trapezoid is a four-sided figure having two sides parallel but not equal in length, as shown in Figure 3-7. If the two parallel sides of the bases (b_1 and b_2) are crossed by a line perpendicular to each, the distance between the two bases along this perpendicular line is the altitude (h) of the trapezoid. The area of any trapezoid can be expressed as the average length of the bases multiplied by the altitude. This relationship can be expressed by the formula:

$$A = \frac{(b_1 + b_2)\,h}{2}$$

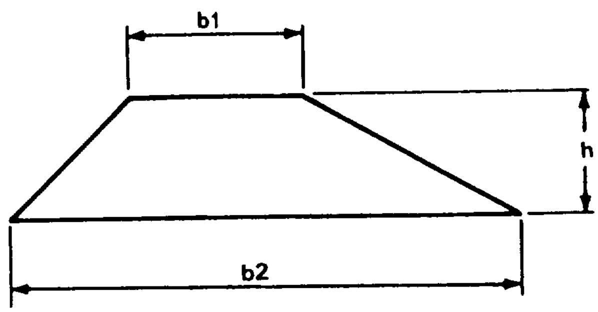


Figure 3-7. Trapezoid base and height dimension locations

Computation of Areas. The first step in computing areas by the trapezoidal method is to break the cross-sectional area into triangles and trapezoids by drawing verticals, as shown in Figure 3-5. Then determine the area of these small figures by the appropriate formula.

To determine the appropriate dimensions, the notes taken by the surveyors must be known. The cross-section notes taken in the field are in fractional form. The figure below the line indicates the horizontal distance from the centerline to that point on the ground. The figure above the line indicates the ground elevation of that point. Points on the grade line of the proposed road are written in a similar manner and are obtained by computations from the final grade line to be established, as shown in Figure 3-8. Thus, the note 32.0/21 indicates a point that is at elevation 32.0 and 21 feet from the centerline of the road. If the cross section is divided into triangles and trapezoids by erecting verticals, obtain notes for the centerline, shoulders, and end of slopes to solve for the area.

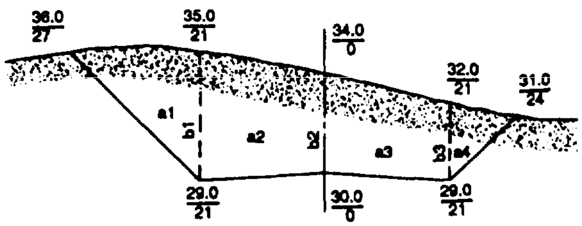


Figure 3-8. Cross-section cut showing distances and elevations

To solve the triangles and trapezoids formed, consider the bases of these figures to be vertical and the altitudes to be horizontal. All vertical bases are found by subtracting elevations, and all horizontal altitudes are found by subtracting horizontal distances from the closest vertical in the direction of the centerline.

Examples:

Referring to Figure 3-8, area a], and substituting in the formula for the area of a triangle:

$$a_1 = \tfrac{1}{2}bh = (35.0 - 29.0)(27-21)$$
$$= \tfrac{1}{2}(6.0)(6) = 18.0 \text{ square feet}$$

Referring to Figure 3-8, area a_2, and substituting in the formula for the area of a trapezoid:

$$a_2 = \tfrac{1}{2}(b_1 + b_2)h$$
$$= \frac{(35.0 - 29.0) + (34.0 - 30.0)\ (21-0)}{2}$$
$$= \tfrac{1}{2}(6.0 + 4.0)(21) = 105.0 \text{ square feet}$$

Find the areas of the remaining trapezoid and triangle in the same way.

Stripper Method

The stripper method is a variation of the trapezoidal method. To use this method, consider a section such as that shown in Figure 3-9.

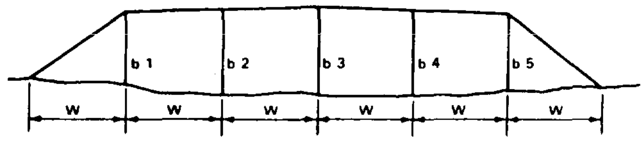


Figure 3-9. Fill cross section arranged to show the stripper method

Example:

If vertical lines are drawn at equal distances apart, then by the trapezoidal formula, the end area, A, will be given by the following computation:

$$A = \tfrac{1}{2}b_1w + \tfrac{1}{2}(b_1 + b_2)w + \tfrac{1}{2}(b_2 + b_3)w$$
$$+ \tfrac{1}{2}(b_3 + b_4)w$$
$$+ \tfrac{1}{2}(b_4 + b_5)w + \tfrac{1}{2}b_5w$$

Factor in and combine terms:

$$A = \tfrac{1}{2}w(2b_1 + 2b_2 + 2b_3 + 2b_4 + 2b_5)$$
$$A = w(\textstyle\sum b)$$

First, measure (graphically) each length (b) and multiply the sum by the width (w) (constant). The distance between vertical lines, w, may be any value, but it must be constant throughout the cross-section area. In rough terrain the vertical lines should be closer together to ensure greater accuracy.

One of the easiest and most convenient ways to measure the vertical lines (b) is with a strip of paper or plastic. Lay the strip along each vertical line in such a manner as to add each in turn to the total. The strip will show the sum of all vertical lines in the same scale that the cross section is plotted. This figure, multiplied by the value of w, will give the area of the cross section.

Inaccuracies result when either a triangle or trapezoid falls within the limits of w or when the area is curved. However, the method is rapid, and the accuracy is adequate under urgent conditions. Figure 3-10 shows a typical cross section with a stripper marked to show the total length of all vertical lines and the value of w. The stripper indicates that the sum of all vertical lines is 21.7 feet: w is given as 10 feet. Applying these figures to the formula, then-

$$A = (\sum b)w$$
$$= 21.7 \times 10 = 217 \text{ square feet (sq ft)}$$

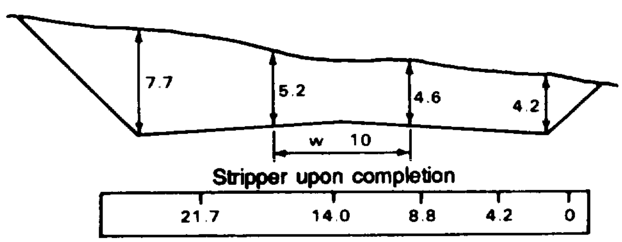


Figure 3-10. Cross section with the sum of all vertical lines added on the stripper

Double-Meridian Triangle Method

The double-meridian method explained in Chapter 13 of TM 5-232 gives a more precise value for a cross-section area than the stripper method. However, it involves more time.

With this method, shown in Figure 3-11, the area is subdivided into two series of trapezoids using the elevations of adjacent points and their projections on the centerline (the distances). These trapezoids have bases equal to the horizontal distance of the respective points from the centerline, and heights equal to their differences in elevation. Where the difference in elevation is plus, the area of the trapezoid is plus; where the difference is minus, the area of the trapezoid is minus. The component areas are added algebraically. Because this procedure uses the sum of the bases of the trapezoid, the area obtained is double the true area and must be divided by 2. The computation is simple arithmetic: subtract adjoining elevations, multiply by the distance from the centerline, add the multiplied results and list plus and minus quantities, add these quantities, and divide by 2.

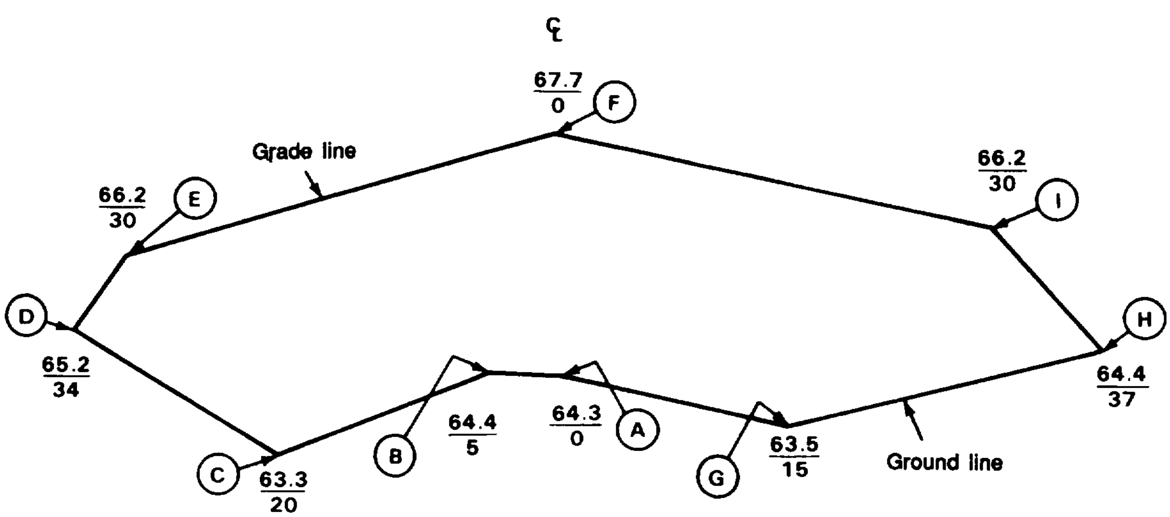


Figure 3-11. Cross-section area by the double-meridian method

The steps for completing the procedure for the double-meridian triangle method follow (refer to Figure 3-11).

1. Start at the centerline ground or grade elevation, whichever is lower (A). Work from the centerline in a clockwise direction to the left (A), (B), (C), (D), (E), (F); and counterclockwise to the right (A), (G), (H), (I), (F), to the centerline ground or grade elevation, whichever is higher (F).

2. Working from point to point, multiply the difference in elevation between each adjacent pair of points by the sum of their distance from the centerline. Point (F) to point (A) is not considered because the sum of their distances from the centerline is zero. Going from a lower to a higher elevation gives a plus quantity, while going from a higher to a lower elevation gives a minus quantity. Place plus quantities in one column and minus quantities in another.

3. Divide the algebraic sum of the plus and minus quantities by 2 to obtain the area of the cross section in square feet (sq ft). In sections having both cut and fill, treat each part as a separate section.

Example:

The area of the cross section shown below is computed as follows:

Plus quantities:

(A) to (B) (64.4 - 64.3) x (5 + 0) = 0.1 x 5 = 0.5
(C) to (D) (65.2 - 63.3) x (34 + 20) = 1.9 x 54 = 102.6
(D) to (E) (66.2 - 65.2) x (30 + 34) = 1.0 x 64 = 64.0
(E) to (F) (67.7 - 66.2) x (0 + 30) = 1.5 x 30 = 45.0
(G) to (H) (64.4 - 63.5) x (37 + 15) = 0.9 x 52 = 46.8
(H) to (I) (66.2 - 64.4) x (30 + 37) = 1.8 x 67 = 120.6
(I) to (F) (67.7 - 66.2) x (0 + 30) = 1.5 x 30 = 45.0

Total of plus quantities = 424.5

Minus quantities:

(B) to (C) (63.3 - 64.4) x (20 + 5) = 1.1 x 25 = -27.5
(A) to (G) (63.5 - 64.3) x (15 + 0) = 0.8 x 15 = -12.0

Total of minus quantities = -39.5

Algebraic sum = 424.5 - 39.5 = 385.0

Area of section = 385.0 divided by 2 = 192.5 sq ft

Planimeter Method

A polar planimeter is an instrument used to measure the area of a plotted figure by tracing its perimeter. The planimeter, shown in Figure 3-12, touches the paper at three points: the anchor point, P; the tracing point, T; and the roller, R. The adjustable arm, A, is graduated to permit adjustment to the scale of the plot. This adjustment provides a direct ratio between the area traced by the tracing point and the revolutions of the roller. As the tracing point is moved over the paper, the drum, D, and the disk, F, revolve. The disk records the revolutions of the roller in units of tenths; the drum, in hundredths; and the vernier, V, in thousandths.

NOTE: Always measure cut and fill areas separately.

Check the accuracy of the planimeter as a measuring device to avoid errors from temperature changes and other noncompensating factors. A simple method of testing its consistency is to trace an area of 1 square inch with the arm set for a 1:1 ratio. The disk, drum, and vernier combined should read 1.000 for this area.

Before measuring a specific area, determine the scale of the plot and set the adjustable arm of the planimeter according to the chart in the planimeter case. Check the setting by carefully tracing a known area, such as five large squares on the cross-section paper, and verifying the reading on the disk, drum, and vernier. If the reading is inconsistent with the known area, readjust the arm settings until a satisfactory reading is obtained.

To measure an area, set the anchor point of the adjusted planimeter at a convenient position outside the plotted area. Place the tracing point on a selected point on the perimeter of the cross section. Take an initial reading from the disk, drum, and vernier. Continue by tracing the perimeter clockwise, keeping the tracing point carefully on the lines being followed. When the tracing point closes on the initial point, take a reading again from the disk, drum, and vernier. The difference between the initial reading and the final reading gives a value proportional to the area being measured.

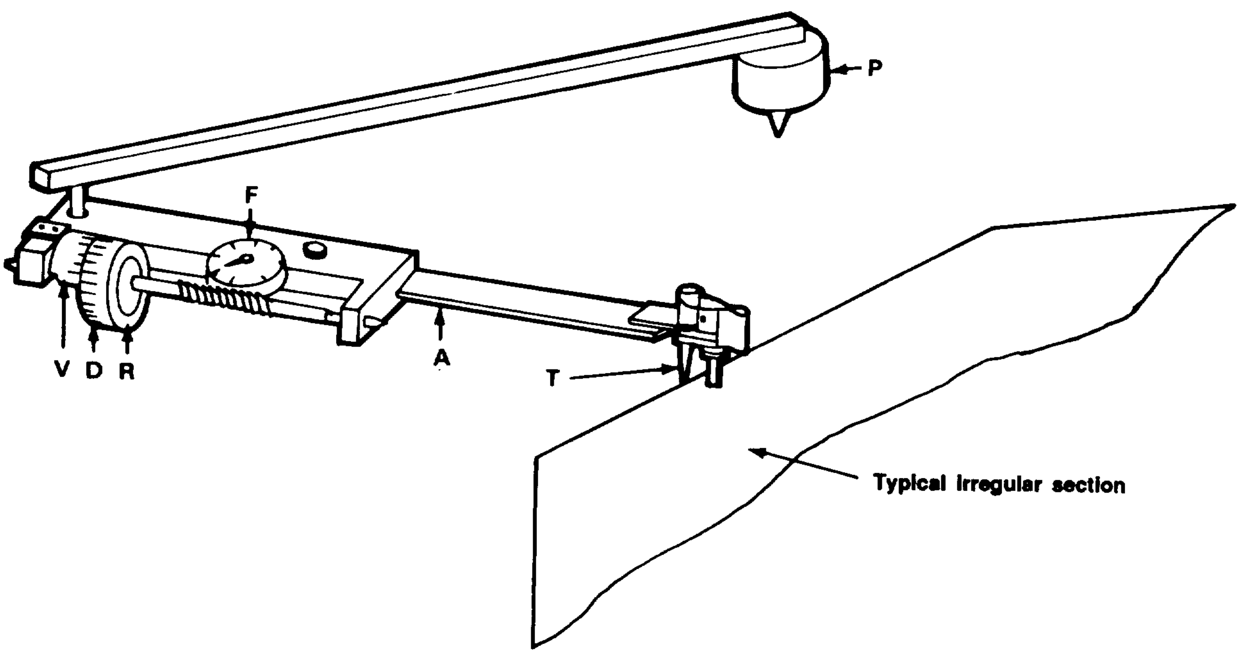


Figure 3-12. Polar planimeter in use

Make two independent measurements to ensure accurate results. The first is performed as discussed above. The second measurement is made with the anchor point again placed outside the area being measured but on the opposite side of the area from its position in the first measurement. This procedure gives two compensating readings the mean of which is more accurate than either.

To measure plotted areas larger than the capacity of the planimeter, divide the area into sections and measure each section separately, as outlined above.

Computer-Aided Design (CAD)

Very accurate measurements can be made if cross sections arc digitized using CAD. Cross sections can be placed on a digitizing pad, points plotted into the computer and, with one command, the area calculated,

METHODS OF VOLUME DETERMINATION

An engineer can accomplish the necessary earthwork computations by using the following methods: average- end-area, prismoidal formula, average-depth -of-cut-or-fill, grid, or contour.

Average-End-Area Method

The average-cnd-area method is most commonly used to determine the volume bounding two cross sections or end areas. use the formula:

$$V = \frac{(A_1 + A_2)}{2} \times \frac{L}{27}$$

where—

V is the volume, in cubic yards (cy) (1 cy = 27 cubic feet (cf)), of the prismoid between cross sections having areas in square feet of A1 and A2, separated by a distance of L feet .

If cross sections are taken at full 100-foot stations, the volume in cubic yards between successive cross sections $A_1 + A_2$, in square feet, may be found by the formula:

$$V = 1.85\ (A_1 + A_2)$$

In either form, the formula is only accurate when A1 and A2 are approximately the same shape. The greater the difference in shape between the two end sections, the greater the possibility of error. However, the method is consistent with field methods in general. In most cases, the time required for a more accurate method is not justified.

Prismoidal-Formula Method

The prismoidal method is used where either the end areas differ widely in shape or a more exact method of computing volume is needed. Its use is very limited because it requires more time than the average-end-area method and gives greater accuracy than is required for most road and airfield construction.

The prismoidal formula is—

$$V = \frac{1}{6}\ (A_1 + 4A_m + A_2)\ \frac{L}{27}$$

where—

V = volume (cy)
L = distance between end sections A_1 and A_2
A_m = area of section midway between A_1 and A_2

Determine A_m by averaging the corresponding linear dimensions of A_1 and A_2 and then determining its area, rather than averaging the areas of A_1 and A_2.

Average-Depth-of-Cut-or-Fill Method

With only the centerline profile and final grade established, earthwork can be estimated with the average-depth-of-cut-or-fill method. Estimate the average depth of cut or fill between 100-foot stations and obtain the volume of material from Table 3-1, page 3-14, The accuracy of this method depends on the care given to establishing the centerline profile, the instruments used, and the accuracy of field reconnaissance.

Table 3-1. Earthwork average cut or fill

This table shows the number of cubic yards of earthwork that are in a 100-foot-long section of cut or fill having a known average depth. To use this table you must know the following:

1. Width.

   a. Cut section - the width of the base of the cut, including ditches.

   b. Fill section - the width of the top of the fill.

2. Average amount of cut or fill.

3. Slope ratio. Column 2 gives the correct amount of earthwork when the side slopes are 1:1. When the slope ratio is other than 1:1, an adjustment must be made (see column 4).

NOTE: The final answer obtained from the table is for a section 100 feet long. If the actual length of the cut or fill is not 100 feet, an adjustment must be made. (For an 85-foot section, multiply by 0.85; for a 50-foot section, multiply by 0.50, and so on.)

Column 1	Column 2 Width of the base of the cut (or top of the fill) in feet:										Column 3	Column 4	
Average amount of cut or fill in feet	26	28	30	32	34	36	38	40	42	44	For each additional foot of width, add:	If the slope is 1.5:1, add:	If the slope is 2:1, add:
1	100	107	115	122	130	137	144	152	159	167	3.5	2	4
2	208	222	237	252	267	281	296	311	326	341	7.5	7	15
3	323	344	367	389	411	433	455	478	500	522	11.0	16	33
4	444	474	504	533	563	593	622	652	681	711	15.0	30	59
5	574	611	648	685	722	759	796	833	870	907	18.5	46	93
6	710	756	800	844	889	933	978	1,022	1,066	1,111	22.0	67	133
7	855	907	959	1,011	1,063	1,115	1,167	1,219	1,271	1,323	26.0	91	181
8	1,010	1,067	1,126	1,185	1,245	1,304	1,363	1,433	1,482	1,541	29.5	118	237
9	1,167	1,233	1,300	1,367	1,433	1,500	1,567	1,634	1,700	1,767	33.5	150	300
10	1,333	1,407	1,482	1,556	1,630	1,704	1,778	1,852	1,926	2,000	37.0	185	370
11	1,507	1,589	1,670	1,752	1,833	1,915	1,996	2,078	2,159	2,241	41.0	224	448
12	1,688	1,778	1,867	1,956	2,045	2,133	2,222	2,311	2,400	2,489	44.5	267	534
13	1,877	1,974	2,070	2,167	2,263	2,359	2,456	2,552	2,648	2,745	48.0	313	626
14	2,074	2,178	2,282	2,385	2,489	2,593	2,696	2,800	2,904	3,007	52.0	363	725
15	2,268	2,389	2,500	2,611	2,722	2,833	2,944	3,056	3,167	3,278	55.5	426	852

However, the volumes obtained by this method are generally adequate for most military construction.

The centerline profile of a road is typical of the entire transverse section because of the narrow widths. Because of the greater width required on an airfield runway, the centerline profile may be misleading as to the typical conditions across the entire transverse width at that point. Therefore, earthwork quantities for airfields should be estimated mainly from cross sections. However, in the absence of sufficient time, the average-cut-or -fill method is better than none at all.

Determine the following before using Table 3-1:

- Average amount of cut or fill.

• Width of the base of the cut or the top of the fill in 2-foot increments between 26 feet and 44 feet.

• Quantity to be added to the figure in column 2, if the width of the base or top of the fill is an odd number of feet.

Ž Quantity to be added if the slope ratio on both sides is 1.5:1 or 2:1.

NOTE: The table is based upon a length of 100 feet between cross sections and a slope ratio of 1:1.

Follow these steps to use the table:

1. Enter column 1 and read down to the average amount of cut or fill for the length concerned.

2. Read horizontally to the right and obtain the figure under the appropriate base of the cut or top of the fill in column 2.

3. Make corrections to this figure from columns 3 and 4, if they apply.

4. If the length is not 100 feet between the points considered, adjust the answer proportionately,

Grid Method

When the quantity of material within the limits of the cut sections is not enough to balance the fill sections, material must be borrowed. The most convenient method is to widen the cuts adjacent to the fills where the material is needed. Compute the volume by extending the cross sections. However, where this is not possible, locate borrow pits at some other area. The grid method is a convenient method of computing the borrow material available in a given borrow pit.

In this method, first stake out over the area a system of squares referenced to points outside the limits of work. The dimensions of these squares depend on the roughness of the original terrain, the anticipated roughness of the final surface, and the accuracy desired. Rougher terrain requires smaller dimensions to get accurate results. The squares must be of such size that no significant breaks, either in the original ground surface or in the pit floor, exist between the corners of the square or between the edges of the excavation and the nearest interior corner.

By taking elevation readings at the stakes before and after excavation, data is obtained to compute the volume of borrow taken from the pit. Figure 3-13 shows a borrow pit over which 25 squares were staked, To identify the various intersecting points, label lines in one direction by numbers and in ‘the other direction by letters. Thus the intersection of lines C and 3 would be labeled C3.

Outline squares falling completely within the excavation with a heavy line, Within that line, determine the volume of excavation for each square in the following manner:

1. Label the points on one square, as shown in Figure 3-14, page 3-16,

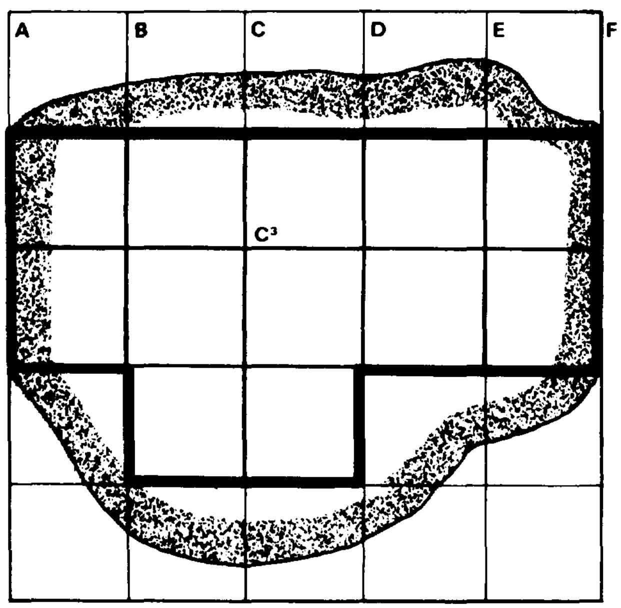


Figure 3-13. Computation grid system for a borrow pit

2. Points a, b, c, and d are on the original ground line, while a^1, b^1, c and d^1 are on the final ground line. The volume of the

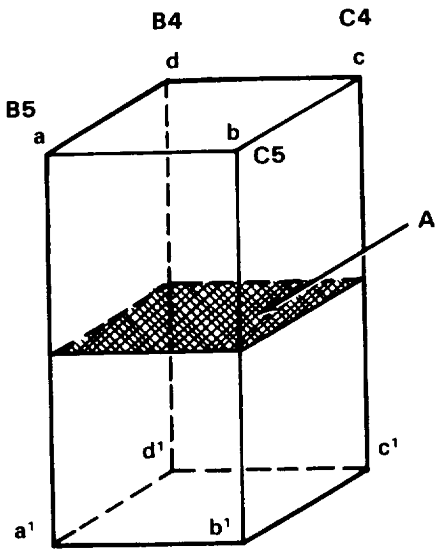


Figure 3-14. Excavation volume for one square

resulting form is the product of the right cross-sectional area A and the average of the four corner heights aa^1, bb^1, cc^1, and dd^1, in cubic yards.

$$V = \frac{A}{27}\,\frac{(aa^1 + bb^1 + cc^1 + dd^1)}{4}$$

3. The volume represented by each square might be computed by the preceding method and all volumes added. However, when a number of such volumes adjoin one another, it is quicker to use the following relation which gives the total volume, represented by all complete squares:

$$V = \frac{A}{4 \times 27}\left(\sum h_1 + 2\sum h_2 + 3\sum h_3 + 4\sum h_4\right)$$
(corner cuts)

This could be approximated by adding all corner cuts and multiplying by A, or

$$V = \frac{A}{27}\sum h$$

In the preceding formula, A is the right cross-sectional area of one rectangular solid, h_1 is a corner height found in one solid, h_2 is a corner height common to two solids, h_3 is a corner height common to three solids, and h_4 is a corner height common to four solids. As an example, aa^1 is an h_1, bb^1 is an h_2, dd^1 is an h_3 and cc^1 is an hq. (Refer to Figure 3-13, page 3-15 and Figure 3-14.) The total borrow-pit quantity also includes the wedge-shaped volumes lying between the complete solids and the limits of excavation. For these volumes, use proportional surface areas, Use the formula:

$$V = \frac{A}{27}\sum h$$

A grid is illustrated in Figure 3-15. The length of the sides of each square is 50 feet. Therefore, given—

$$V = \frac{A\,(h_1 + 2h_2 + 3h_3 + 4h_4)}{4 \times 27},$$

$$V = \frac{50 \times 50 \times 492}{4 \times 27} = 11{,}388 \text{ cy}$$

An alternative method is to compute the total of all cuts at each corner (123 feet), compute the average cut across all squares (123/25 = 4.92), and then multiply by the length of the sides of the figure.

$$V = \frac{4.92 \times 250 \times 250}{4 \times 27} = 11{,}388 \text{ cy}$$

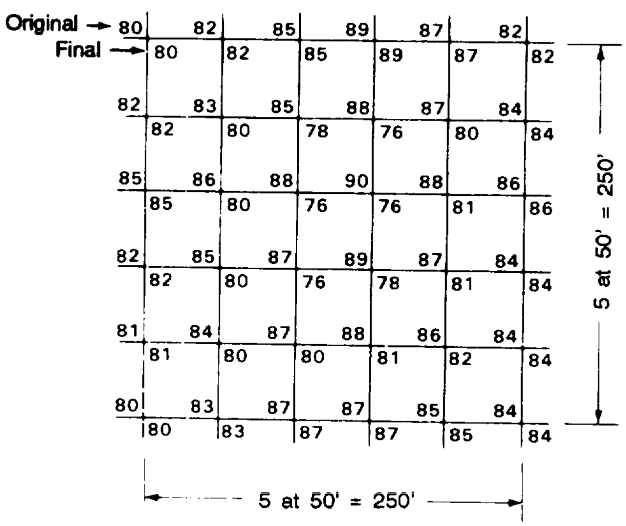


Figure 3-15. Sample grid-system work sheet

FACTORS INFLUENCING EARTHWORK CALCULATIONS

On many projects, one objective of the paper location study is to design the grade line so the total cut within the limits of the work equals the total fill. The uncertain change of volume of the material make this difficult. It is usually more economical to haul excavated material to the embankment sections, thereby eliminating borrow and waste.

Shrinkage

Shrinkage has occurred when 1 cubic yard of earth, as measured in place before excavation, occupies less than 1 cubic yard of space when excavated, hauled to an embankment, and compacted. This difference is due to the combined effects of the loss of material during hauling and compaction to a greater-than-original density by the heavy equipment used in making the embankment.

Shrinkage is small in granular materials such as sand and gravel, and is large in ordinary earth containing appreciable percentages of silt, loam, or clay.

Shrinkage is very high (possibly 70 percent) for shallow cuts containing humus, which is discarded as unsuitable for embankments. These shallow cuts (usually 4 to 8 inches deep) arc called *stripping*.

Loose and swell refer to a condition which is the reverse of shrinkage. The earth assumes a larger volume than its natural state when stockpiled or loaded into a truck. This factor ranges from 10 to 40 percent swell and is usually uniform for a given material.

Shrinkage, however, varies with changes in the soil constituents and with changes in moisture content and the type of equipment used. Consequently, a percentage allowance assumed in design may eventually prove to be 5 percent or more in error. A common shrinkage allowance is 10 to 30 percent for ordinary earth.

Settlement refers to subsidence of the completed embankment. It is due to slow additional compaction under traffic and to gradual plastic flow of the foundation material beneath the embankment.

Net Volume Calculation

Compute the volume of cut and fill and the net volume between any two points on the construction project. The net volume is the difference between the volume of cut and the volume of fill between any two specified stations. The net volume may apply to the entire project or to a few stations. Net volume may be described in a compacted, in-place, or loose state. Table 3-2 provides conversion factors used to find the net volume. All calculations are recorded on

Table 3-2. Soil conversion factors

Soil type	Conversion factors for earth-volume change			
	Present soil condition	Converted to --		
		In place	Loose	Compacted
Sand	In place		1.11	0.95
	Loose	0.90		0.86
	Compacted	1.05	1.17	
Common earth	In place		1.25	0.90
	Loose	0.80		0.72

the earthwork volume sheet shown in Table 3-3.

Earthwork Volume Sheet

The earthwork volume sheets allow you to systematically record this information and make the neccessary calculations. They provide a means of tabulating earthwork quantities for use in the mass diagram discussed later in this chapter. The earthwork volume calculation sheet, shown in Table 3-3, is divided into columns for recording and calculating information.

Stations (column 1). List in column 1 all stations at which cross-sectional areas have been plotted. Normally, these areas are taken at all full stations and at intermediate stations that are required to fully represent the actual ground conditions and earthwork involved.

Area of Cut (column 2). Record in column 2 the computed cross-sectional areas of cut at each station. These areas may be computed by one of the commonly used methods, depending on the degree of accuracy required.

Area of Fill (column 3), Complete column 3 in the same manner as column 2, except show cross-sectional areas of fill.

Volume of Cut (column 4). Complete the volume of cut material between adjacent stations and record it in column 4. The most common method for computing volumes is the average-end-area method (or the earthwork table based on this method).

This volume represents only the volume of cut between the stations and the volumes reflected as in-place yardage.

Volume of Fill (column 5). Complete column 5 in the same manner as column 4, except show fill volumes. Fill volumes reflect com-

This layer varies in depth but is usually 4 to 6 inches deep. This material must be wasted because it is not satisfactory to place in an embankment, Indicate in column 6 the volume between stations of this humus material over sections of cut.

Stripping Volume in Fill (column 7). Before an embankment can be constructed, the same layer of humus must be removed and the volume replaced with satisfactory material. Indicate in column 7 the volume of this material between stations over sections of fill.

Net Volume of Cut (column 8). Indicate in column 8 the volume of cut material between stations that is available for embankment. Column 8 is column 4 minus column 6, because the total cut must be decreased by the amount of material wasted in stripping, including the organic material.

Adjusted Volume of Cut (column 9). One cubic yard of material in its natural, undisturbed state occupies approximately 1.25 cubic yards when removed and placed in a truck or stockpile. The same 1 cubic yard, when placed in an embankment section and compacted, occupies a volume of approximately 0.9 cubic yards. In planning operations, convert these various volumes to the same state so the comparisons can be made, Changes in volume of earthwork are discussed in this chapter, and Table 3-1, page 3-14, provides the necessary conversion factors. Column 9 is column 8 multiplied by the appropriate conversion factor (in this case, 0.9) to convert it from in-place yardage to compacted yardage.

Total Volume of Fill (column 10). Indicate in column 10 the amount of compacted material required between stations to complete needed embankments, Column 10 is column 5 plus column 7, plus the amount necessary to replace the quantity removed by stripping. This figure represents the fill

Table 3-3. Earthwork volume calculation sheet

Station	End area cut (sq ft)	End area fill (sq ft)	Volume of cut (BCY)	Volume of fill (BCY)	Stripping cut cy (BCY)	Stripping fill (BCY)	Net cut (BCY)	Adj cut cy (CCY)	Total fill cy (CCY)	Algebraic sum (CCY)	Mass ordinate (CCY)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
0 + 00											
				71		18			89	-89	
0 + 50		115									-89
				210		30			240	-240	
1 + 00		112									-329
				307		44			351	-351	
2 + 00		54									-680
			40	78		22	40	36	100	-64	
2 + 50	64	30									-744
			170	19	26		144	130	19	+111	
3 + 00	120										-633
			519		76		443	399		+399	
4 + 00	160										-234
			883		74		809	728		+728	
5 + 00	317										+494
			681		60		621	559		+559	
6 + 00	51										+1,053
			90	4	21		69	62	4	+58	
6 + 50	46	6									+1,111
			29	121		25	29	26	146	-120	
7 + 00		125									+991
				576		81			657	-657	
8 + 00		186									+334
				480		69			549	-549	
8 + 50		332									-215

Notes:

1. BCY: Banked cubic yardage
2. CCY: Compacted cubic yardage

material that is available (plus) or required (negative) within the station increment after the intrastation balancing has been done.

Mass Ordinate (column 12). Column 12 indicates the total of column 11 starting at station 0 + 00. While passing through a stretch where cutting predominates, this column increases in value. While passing through a stretch where embankment is required, this column decreases.

THE MASS DIAGRAM

The first step in planning earthmoving operations is the estimation of earthwork quantities involved in a project. This can be done accurately by one of several methods, depending upon the standard of construction preferred. With these estimates, the engineer can prepare detailed plans for economical and efficient completion of the earthmoving mission.

The mass diagram is one method of analyzing earthmoving operations. This diagram can tell the engineer where to use certain types of equipment, the quantities of materials needed, the average haul distanccs and, when combined with a ground profile, the average slope for each operation. This permits the preparation of detailed management plans for the entire project. The mass diagram is not the complete answer to job planning, and it has limitations that restrict its effectiveness for certain typres of projects. However, it is one of the most effective engineer tools and is easily and rapidly prepared.

CONSTRUCTION OF THE MASS DIAGRAM

Using column 1 (station) and column 12 (mass ordinate, cumulative total) of a completed earthwork volume sheet, a mass diagram can be plotted as shown in Figure 3-16.

Plot the mass diagram on scaled graph paper with the stations indicated horizontally and the mass indices (column 12) denoted vertically. Connect all plotted points to complete the mass diagram as shown in Figure 3-16. Positive numbers arc plotted above the zero datum line, negative numbers below.

PROPERTIES OF THE MASS DIAGRAM

Figure 3-17 shows a typical mass diagram with the actual ground profile and final grade line of the project plotted. Note that both usc the same horizontal axis (stations). The ground profile is placed above the mass diagram to facilitate the calculation of the average grade over which equipment will work. The horizontal axis is the only thing these graphs have in common.

The mass diagram is a running total of the quantity of earth that is in surplus or deficient along the construction profile. If at one station more material is being cut than filled, you have a cut operation at that station. The quantity or *volume* of surplus material will be increasing as cutting operations continue through the station, producing an ascending mass diagram curve line. Cutting is occurring from stations A to B and stations D to E in Figure 3-17. The total volume for the cut at station A to B is obtained by projecting the points on the curve line at stations A and B to the vertical axis and reading the volume (Q).

Conversely, if at one station more material is being filled than cut, you have a *fill* operation at that station. The quantity or *volume* of deficient material will be increasing as filling operations continue through the station, producing a descending mass-diagram curve line. Filling is occurring from stations B to D in Figure 3-17. The total volume for the fill at stations B to D is obtained by projecting the points on the curve line at stations B and C to the vertical axis and reading and adding the volumes above and below the zero datum line.

The maximum or minimum point on the mass diagram, where the curve changes from rising to falling or vice versa, indicates a change from cut to fill or vice versa. This point is referred to as a transition point (TP). On the ground profile, the grade line crosses the ground line at the TP, as illustrated at stations B and D.

When the mass diagram crosses the datum line or zero volume, as at station C, there is exactly as much material filled as there is material cut, or zero volume excess or deficit at that point. The section of the mass diagram, from the start of the project at station A to a point of crossing the zero volume line, is known as a *node.* Each crossing point on the zero volume line indicates another node. The last node may or may not return to the zero datum line. Nodes are numbered from left to right.

The final position of the mass diagram line, above or below the datum line, indicates whether the project was predominately cut or fill. In Figure 3-17, where the mass diagram ends at station E, the operation was cutting; that is, surplus material was generated by cutting and must be hauled away (waste operation). Borrow operations occur when the final position of the mass diagram is below the zero volume line.

Earthwork volume sheet

Station (1)	Mass ordinate (CCY) (12)
0+00	0
0+75	675
1+50	975
2+25	960
3+00	850
3+75	575
4+50	0
5+25	-300
6+00	-400
6+75	-380
7+50	-200

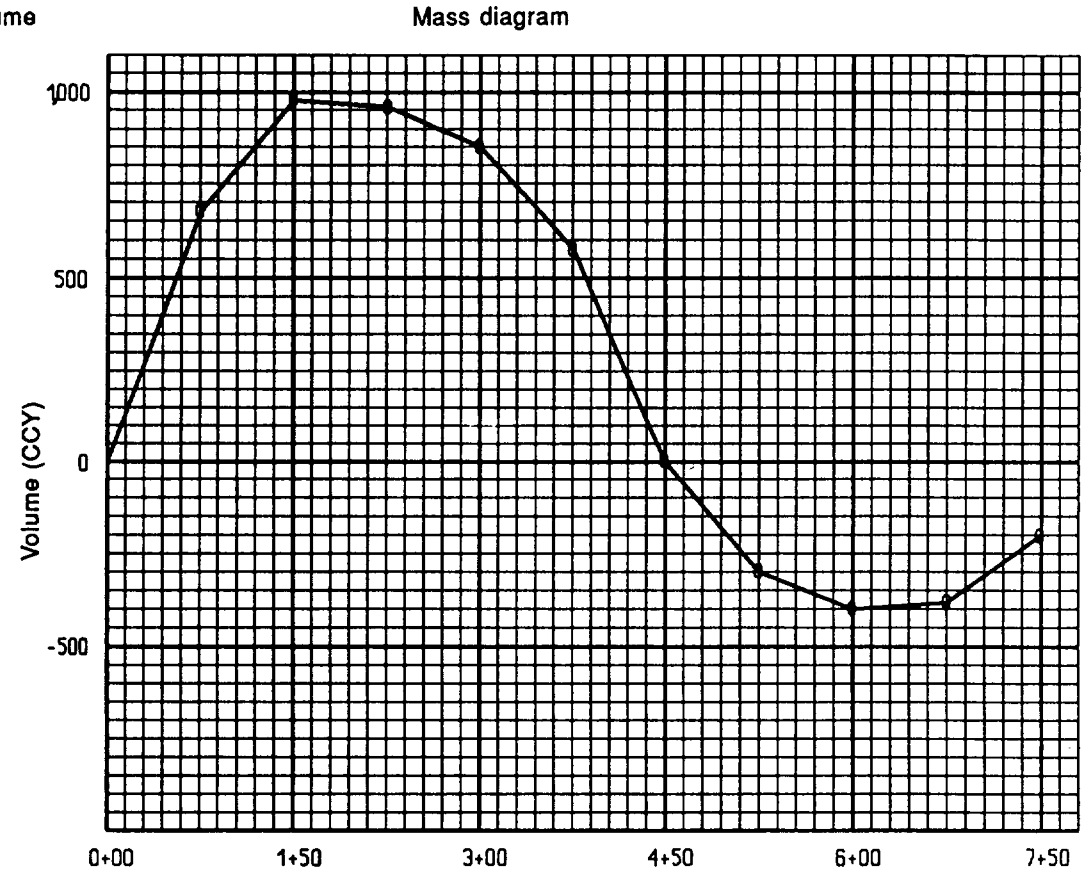


Figure 3-16. Plotting the mass diagram

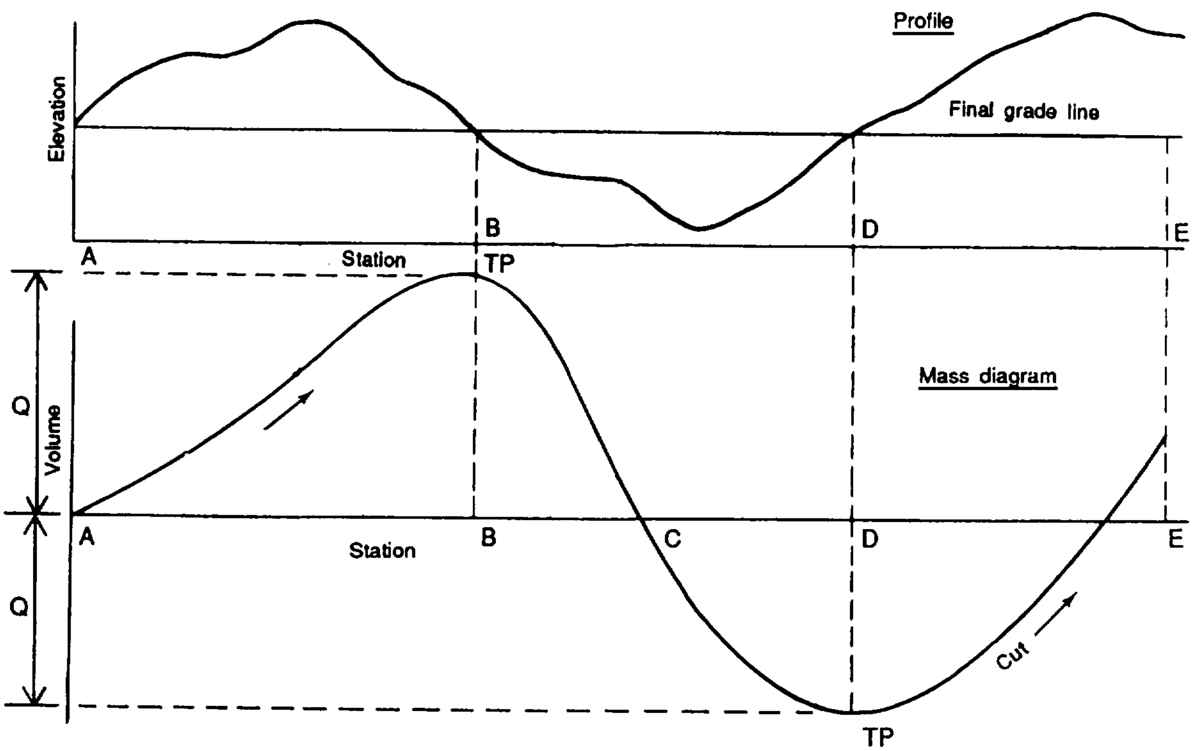


Figure 3-17. Properties of a mass diagram

PROJECT ANALYSIS

Once the basic properties of the mass diagram arc understood, the engineer can conduct a detailed analysis 10 determine where dozers, scrapers, and clump trucks will operate This is accomplished by using *balance lines*. A balance line is a line of specific length drawn horizontally, intersecting the mass diagram in two places. The specific length of the balance line is the recommended working or maximum haul distance for different pieces of equipment. The term *maximum haul distance* is used because opera ting the equipment beyond this point would not be efficient. The maximum haul distances (balance-line lengths) are—

Equipment	Maximum Haul Distance
Dozer	Up to 300 feet
Scraper	301 to 5,000 feet
Dump Truck	5,001 feet to several miles

These lengths are measured using the horizontal scale (stations measured in hundreds of feet). Always use the maximum haul distance (length) of each piece of equipment, provided the project or node is at least that length. Start excavating each node with the dozer followed by the scraper and dump truck.

Figure 3-18 shows a balance line drawn on a portion of a mass diagram. If this was a dozer balance line, the distance between stations A and C would be 300 feet.

Cut equals fill between the ends of a balance line. The mass line returns to exactly the same level, indicating that the input and the expenditures of earth have been equal. In Figure 3-18, this occurs between stations A and C. There has been an exact balance of earthwork.

In Figure 3-18, the amount of material made available by cutting between stations A and B is measured by the vertical line marked Q. This is also the amount of embankment material required between sta-

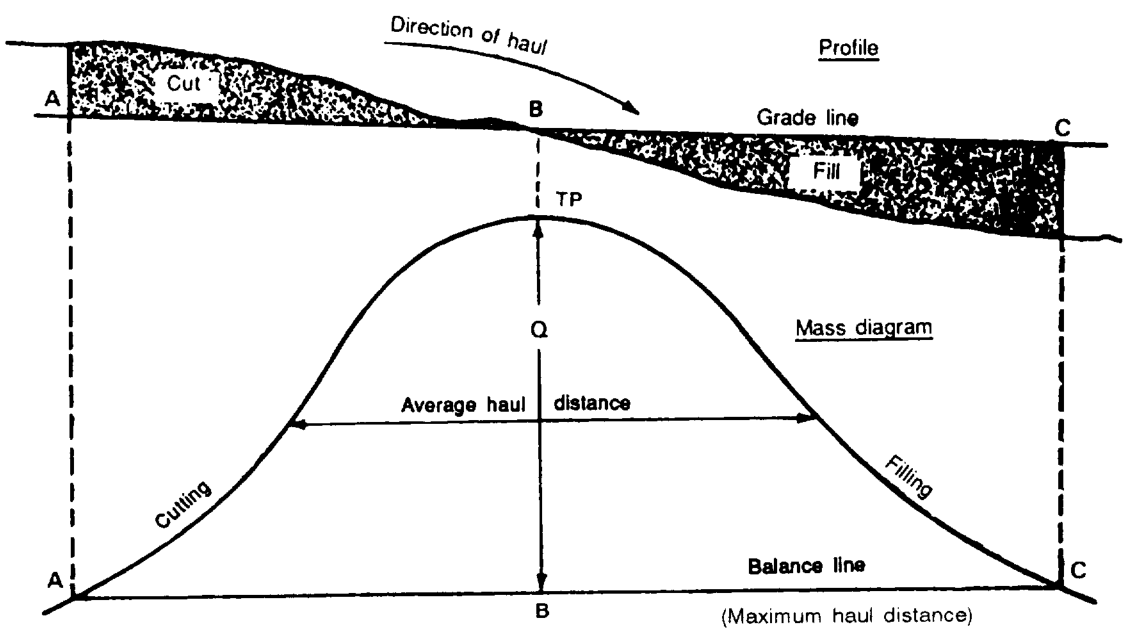


Figure 3-18. Mass diagram with a balance line

tions B and C. This is described as the balanced quantity of earthwork.

If equipment was used to do the balanced earthwork between stations A and C, the maximum distance that earth would have to be moved would be the length of the balance line AC.

In accomplishing balanced earthwork operation between stations A and C, some of the haul distance would be short, while some would approach the maximum haul distance. The *average haul distance* (AHD) is the length of the horizontal line placed midway between the balance line and the top or bottom point (transition point) of the curve (Figure 3-18) and is found by dividing the vertical distance of Q in half.

If the curve is above a balance line, the direction of haul is from left to right. The converse is true when the curve is below a balance line.

Figure 3-19 shows a part of a mass diagram on which two balance lines have been drawn. The same principles apply for the area between the lines as with only one balance line. The quantity balanced is the vertical distance between the balance line, while the horizontal bisector is the average haul distance. The longer balance line is the maximum haul distance, and the shorter balance line is the minimum haul distance. The haul distance depends upon the position of the curve with respect to the balance lines.

The mass diagram is a useful indicator of the amount of work expended on a project. By definition, *work* is the energy expended in moving a specified weight a given distance. It is the product of weight times distance. Because the ordinate of the mass diagram is in cubic yards (which represents weight) and the abscissa is in stations or distance, an area on the mass diagram represents work. In Figure 3-20, page 3-24, if equipment is used to do the balanced earthwork between the ends of the balance lines as drawn, the work expended is equal to the area between the mass line and the balance line.

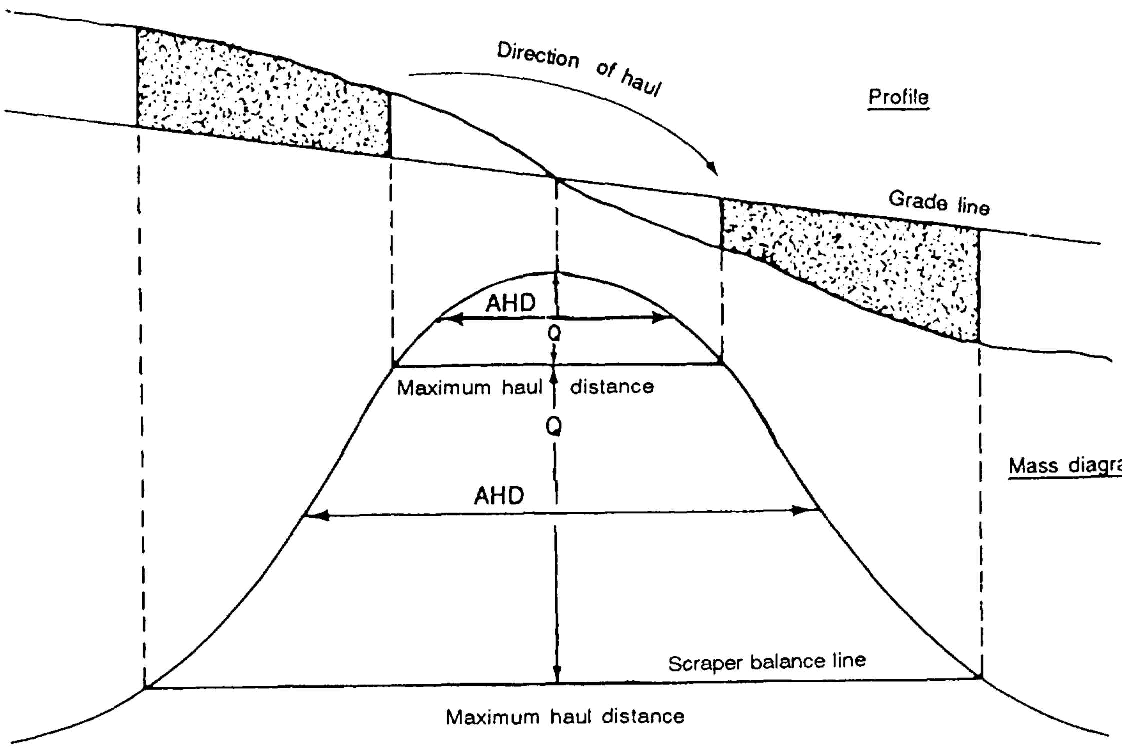


Figure 3-19. Mass diagram with two balance lines

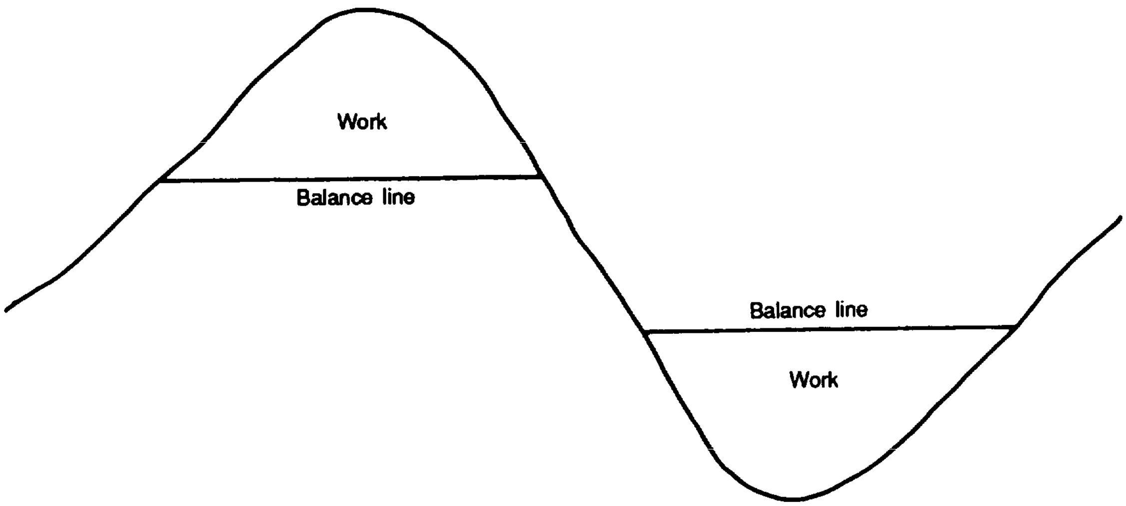


Figure 3-20. "Work" in earthmoving operations

Another item calculated from the mass diagram is average grade. This value is used when computing equipment scheduling and utilization. Figure 3-21 and Figure 3-22, page 3-26, illustrate a portion of the mass diagram on which the average grade has been determined.

USE OF THE MASS DIAGRAM

The mass diagram is used to find the cost of a project in terms of haul distance and yardage, to locate the areas for operation for various types of equipment, to establish the requirements for borrow pits and waste areas, and to provide an overall control of required earthmoving operations. However, the means used to analyze the mass diagram will follow the same principles regardless of the end result desired. The analysis of the mass diagram is based upon the proper location of balance lines.

Because the lengths of balance lines on a mass diagram are equal to the maximum or minimum haul distances for the balanced earthmoving operation between their end points, they should be drawn to conform to the capabilities of the available equipment. Equipment planned accordingly will operate at haul distances that are within its best range of efficiency. Figure 3-21 illustrates a portion of a mass diagram on which two balance lines have been drawn: 300 feet to conform to dozer capabilities and 5,000 feet for the scraper.

The following job analysis can be made from the diagram in Figure 3-21:

Use dozers between stations C and E. The maximum haul distance is 300 feet; the average haul distance is the horizontal bisector shown. The amount that will be cut between stations C and D and filled between stations D and E is the length of the indicated vertical lines.

Use scrapers for cutting from stations A to C and filling from stations E to G. The minimum and maximum haul distances are 301 and 5,000 feet, respectively. The average haul distance is the horizontal line midway between the balance lines. The amount of earthwork is indicated by the vertical line.

To determine the average grade for either the scraper or dozer work area, use the following procedure:

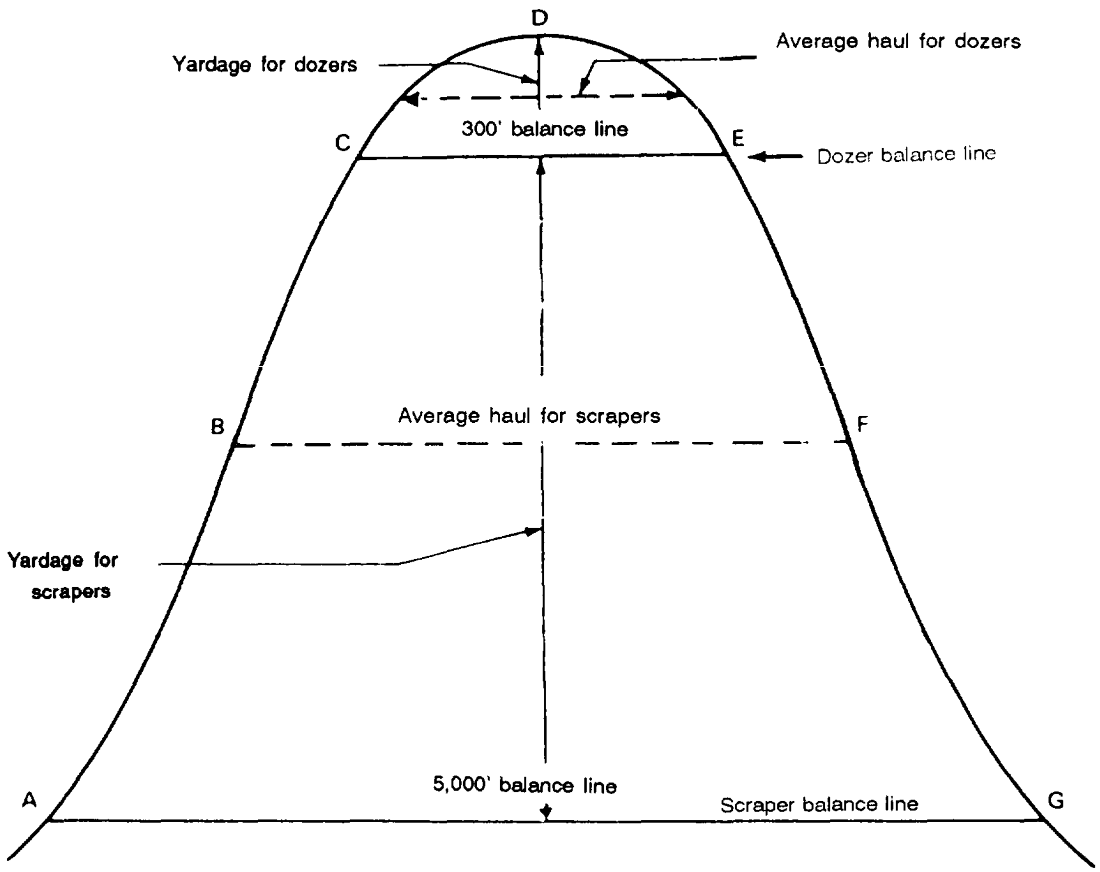


Figure 3-21. Balance lines for equipment efficiency

1. Draw on the profile a horizontal line through the work area that roughly divides the area in half. This is a rough estimation. (See Figure 3-22, page 3-26.)

2. Extend a vertical line from the end points of the previously determined average haul line up through the project profile. These lines are referred to as the average haul vertical. (See Figure 3-23, page 3-26.)

3. Draw a final line connecting the intersecting points of the lines drawn in steps 1 and 2. This line represents the average grade.

4. Determine the average change in elevation (the vertical distance between the cut and fill).

5. Calculate the average grade as follows:

$$\textit{Average Grade } \% = \frac{\textit{Average change in elevation}}{\textit{Average haul distance}} \times 100$$

In the example shown in Figure 3-23, the average grade for the dozer would be—

$$\text{Average Grade } \% = \frac{18'}{203'} \times 100 = -8.87\%$$

Since this is an operation which moves earth *downhill,* the grade would be negative, or -8.87% An uphill cut would have a positive grade.

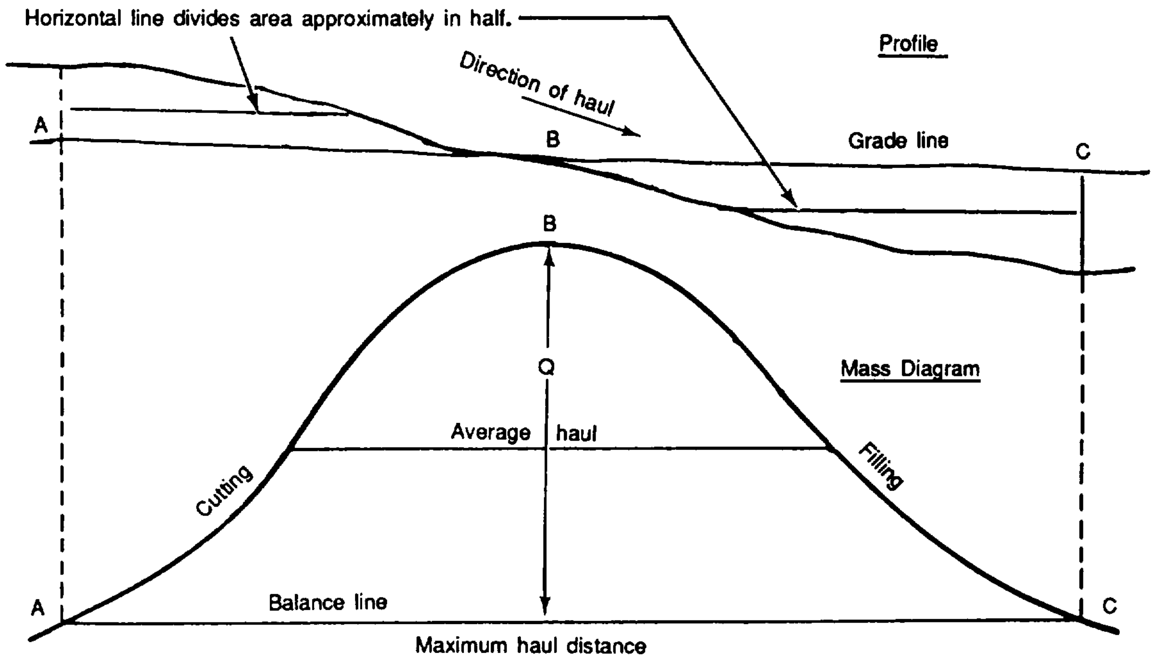


Figure 3-22. Determining average grade, step 1

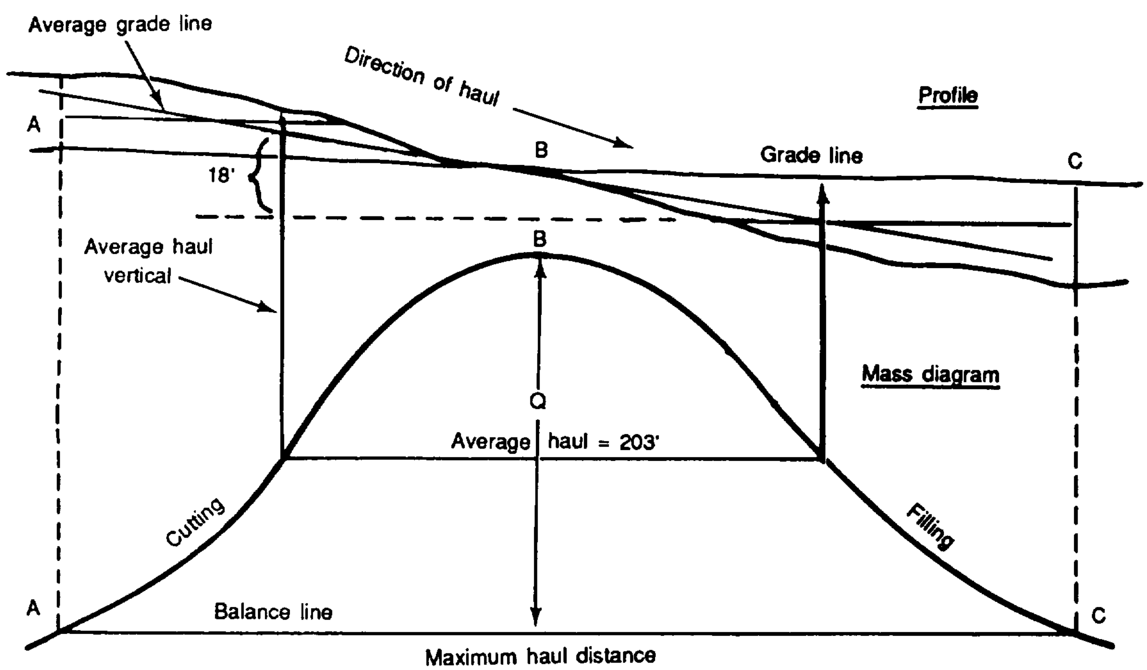


Figure 3-23. Determining average grade, step 2

Placement of Balance Lines to Minimize Work

Because the area between the apex of the mass diagram and the balance line is a measure of the work involved in the balancing operation, the size of these areas should be decreased whenever possible, However, the method used to minimize the area depends upon the shape of the mass diagram and the number of adjacent nodes that can be used.

If two nodes are adjacent, work is minimized when two balance lines are drawn as one continuous line, with the balance lines equal in length. Each balance line must be within the maximum efficient haul distances for the equipment. The best placement of balance lines on the portion of a mass diagram shown in Figure 3-24 would be lines AE and EF, with AE = EF.

Only one balance line, CH, may be needed if it is within efficient haul distance specifications. The quantity involved would be Q yards and the work involved would be the area above CH.

If this one balance line was replaced by two balance lines, BD and DG, with BD less than DG. the quantity of earthwork balanced would remain the same. The work would be decreased by the size of the area between CH and DG and increased by the size of the area between BD and C. This would result in a savings because the increase is less than the decrease for the same amount of earthwork balanced. This decreasing process will continue by raising the lines to the point where one equals the other, or until AE = EF is reached.

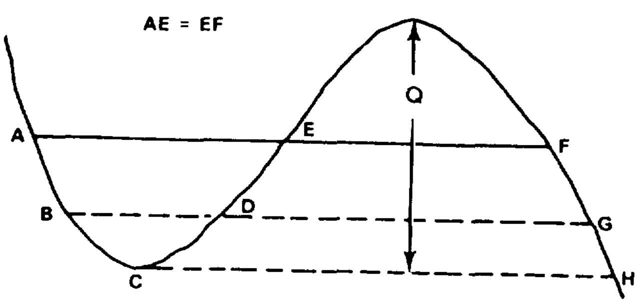


Figure 3-24. Minimizing work with two nodes

If there is an even number of adjacent nodes, as shown in Figure 3-25, work is reduced when the balance lines are one continuous line and AB + CD + EF = BC + DE + FG. The length of each balance line must be within equipment maximum haul capabilities as defined earlier.

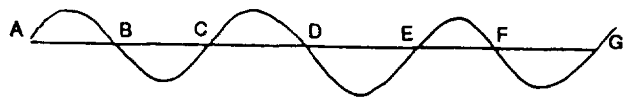


Figure 3-25. Minimizing work with an even number of nodes

If there is an odd number of adjacent nodes, as shown in Figure 3-26, work is decreased when the balance lines are one continuous line and AB + CD + EF - (BC + DE) equals the limit of efficient haul, or approximately 1,000 feet. All balance lines must be within equipment limits.

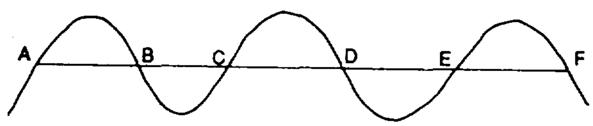


Figure 3-26. Minimizing work with an odd number of nodes

Calculation of Earthwork not Within Balance Lines

It is usually impossible to place balance lines so that the entire amount of earthwork on a project can be balanced. Some part of the mass line will be outside the balance lines. This material must be wasted or borrowed. If the portion not within balance lines is ascending (cutting), there is waste; if it is descending (filling), there is borrow, This is shown in Figure 3-27, page 3-28. Concentrate all necessary borrow and waste operations in one general area.

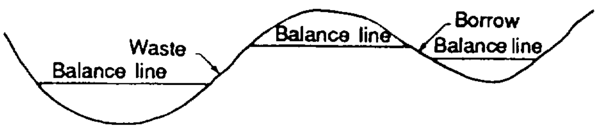


Figure 3-27. Waste and borrow on a mass diagram

Format for Analysis

The simplest and most practical method of Tabulating the results of a mass diagram is to write all quantities and distances on the diagram, as shown in Figure 3-28. It is also possible to extract information from the mass diagram and put it in a format that effectively controls the operation. One method is to prepare a mass diagram analysis sheet as shown in Figure 3-29.

LIMITATIONS OF THE MASS DIAGRAM

The mass diagram has many limitations that preclude its use in all earthmoving operations. At best, it is merely a guide indicating the general manner in which the operations should be controlled. Any attempt to get exact quantities and distances from it may be misleading. However, it is a good starting point.

The mass diagram is most effective when used to plan operations along an elongated project similar to a road, an airfield runway, or a taxiway. The haul distances are along the centerline or parallel to it. However, if the project becomes relatively wide compared to its length, movement of earth may be transverse as longitudinal, resulting in longer, transverse haul distances and invalidating the mass diagram analysis.

The mass diagram is used to analyze only the potentiality of balancing within one

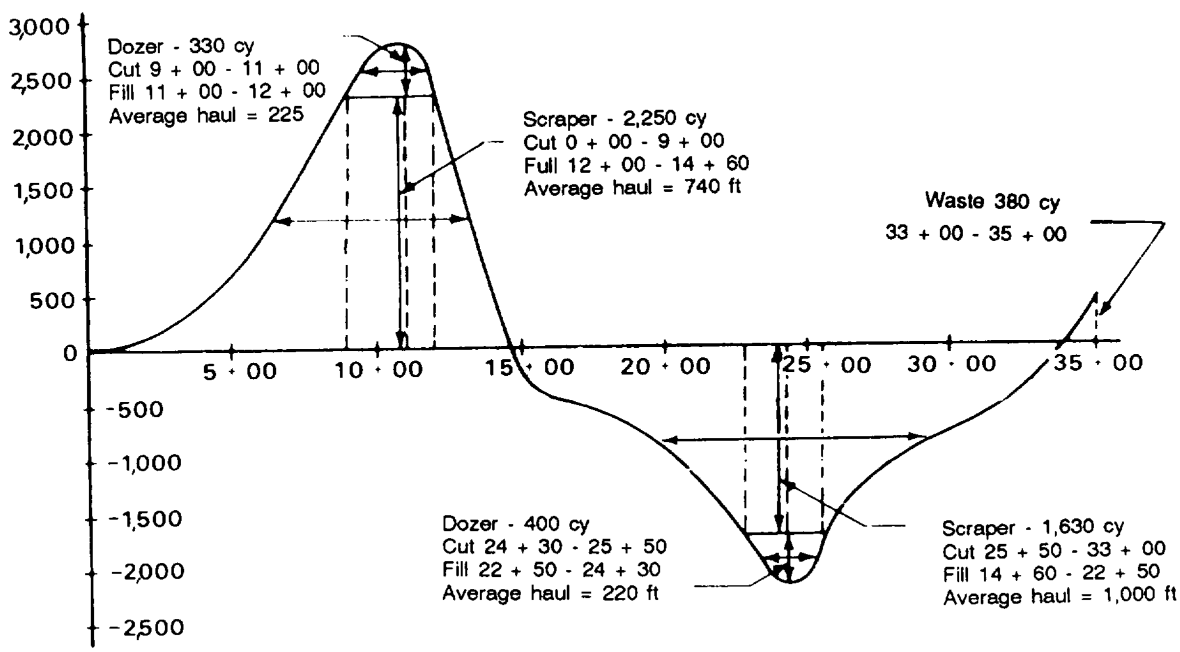


Figure 3-28. Mass diagram showing analysis results

phase of a project. For instance, the mass diagram may indicate that the best balancing of a certain portion of a runway will require a haul distance of 2,200 feet along the site. However, it may be better to balance yardage with an adjacent taxiway in which the haul distance will be only 1,200 feet. The mass diagram can deal only with the runway or the taxiway, not with both simultaneously.

The mass diagram assumes that all material excavated in the cut sections is acceptable for use in the embankment sections. This is not necessarily true. However, all unacceptable quantities can be eliminated from the earthwork table.

The mass diagram is applicable to projects needing balanced earthwork. Balancing eliminates the double handling of quantities. If there is a short distance between an acceptable borrow pit and an embankment section, it may be more economical to use the borrow pit instead of a long balancing operation, This can be determined by a *work* or economy. study.

Location (Stations)	Type of work	Amount (cy)	Average distance/ % grade	Type of equipment	Production rate	Number of units	Net time
00 + 00 - 9 + 00	Cut	2,250	740 -2%	Scraper			
12 + 00 - 14 + 60	Fill						
9 + 00 - 11 + 00	Cut	300	225 -2%	Dozer			
11 + 00 - 12 + 00	Fill						
14 + 60 - 22 + 50	Fill	1,630	1,000 -1%	Scraper			
25 + 50 - 33 + 00	Cut						
22 + 50 - 24 + 30	Fill	400	220 -1	Dozer			
24 + 30 - 25 + 50	Cut						
33 + 00 - 35 + 00	Waste	380					

Figure 3-29. Mass-diagram analysis sheet

CLEARING, GRUBBING, AND STRIPPING

CHAPTER 4

Land clearing is the removal and disposal of all vegetation, rubbish, and surface boulders embedded in the ground. In the TO, land clearing also includes the removal and disposal of mines, booby traps, and unexploded bombs. Grubbing is the uprooting and removal of roots and stumps. Stripping is the removal and disposal of unwanted topsoil and sod.

Clearing, grubbing, and stripping are accomplished by using heavy engineering equipment. Hand- or power-felling equipment, explosives, and fire are also used. Factors that determine which method to use are: the acreage to be cleared the type and density of vegetation, the terrain's effect on the operation of equipment, the availability of equipment and personnel, and the time available for completion. For best results, a combination of methods is used in a sequence of operations.

Clearing, grubbing, and stripping are the same in road and airfield construction. In airfield construction, the areas to be cleared are usually larger than for road construction; the number of personnel and amount of equipment used are correspondingly greater; and the disposal of unsuitable materials requires more detailed planning and longer hauls.

FOREST TYPES AND ENVIRONMENTAL CONDITIONS

Clearing, grubbing, and stripping operations differ in every climatic zone because each zone has different forest and vegetative types. Forests are not uniform in type, growth, and density within climatic zones. Soils, altitudes, water tables, and other factors vary widely within each zone.

The general nature of a forest is determined from records of the principal climatic factors, precipitation, humidity, temperature, sunlight, and the direction of the prevailing winds. The nature and action of climatic factors during the growing season determine the amount and types of forests. From these records, a general interpretation of the forests in an area can guide detailed reconnaissance.

The climate classifications of forests are temperate, rain, monsoon, and dry, The following paragraphs describe these classifications.

TEMPERATE FORESTS

Temperate forests contain both softwood and hardwood trees. Hardwoods are dominant where the soils are old, deep, and fertile. Softwoods are dominant where the soils are young, shallow, and less fertile. The density of growth in these forests varies with topography and local climate conditions. Bogs are common in cold region, softwood forests. Bogs present a hazard to construction equipment during the clearing operations. Root systems vary according to geologic conditions and species. The types of root systems typical of various species are listed in Table 4-1.

RAIN FORESTS

Rain forests occur in tropical climates where rainfall is heavy throughout the year. They consist of tall, broad-leaved trees that grow as high as 175 feet. The trees have an umbrella-like foliage that permits little sunlight to penetrate. The undergrowth consists of thick vines that cling to the trees for support and grow to great heights. Where sunlight reaches the forest floor, the undergrowth is dense and varied. Because of continual precipitation, the root systems are on or near the ground surface and spread in a lateral pattern around the base of the trees,

MONSOON FORESTS

Monsoon forests occur in climates of heavy seasonal rains with strong, warm winds. The forests are dense, with varied species of hardwoods which are moderate-sized, broad - leaved, and have shallow root systems. The undergrowth is very dense with shrubs, vines, and plants.

DRY FORESTS

Dry forests occur in arid and tropical regions where there is little precipitation. The forests are either scrub or savanna. Scrub forests usually consist of broad-leaved hardwoods with dense thickets along watercourses. In open areas there are scattered growths of low, thorny, stunted shrubs and stunted trees that usually have long, tough taproots that are difficult to remove. Savanna forests are in more humid, dry-forest regions. These forests are park-like, with large trees widely and uniformly spaced. Savannas usually have continuous coverings of small grasses.

GEOLOGIC AND PERMAFROST CONDITIONS

An investigation of the geologic conditions of a forest can help when estimating the density and depth of the root systems of the trees. The investigation should be concerned with hardpan, marshy, and permafrost conditions.

Hardpan or Rock

Where a forest is closely underlaid by hardpan or rock, the tree roots branch and remain near the surface. This growth is easy to uproot. Where the soil is firm and the hardpan or rock is deep, the trees tend to form large, deep taproots that make uprooting difficult.

Inundated, Marshy, and Boggy Areas

In these areas, trees have thick, wide-spreading. and shallow root systems near the surface of the ground.

Permafrost

In northern regions where permafrost occurs, the root systems of trees are similar to those in hardpan or rock. Where the permafrost is near the surface, roots branch out and lie close to the surface. Where permafrost is far below the surface, trees develop taproots.

PREPARATION

RECONNAISSANCE AND PLANNING

The types of trees, vegetation, soil, and terrain encountered while clearing the land must be determined as accurately as possible from climatic and geological maps, intelligence reports, and aerial and ground reconnaissance. (Refer to Chapter 2 of this manual for more detailed in formation.)
After such information has been verified, estimate the quantity of work, select the available equipment, determine the number of

Table 4-1. Species of trees and their normal root systems

Species	Normal Root System
Alder	Shallow, wide-spreading laterals
Ash	Deep in porous soils, shallow and spreading in rocky soils
Aspen	Shallow laterals
Basswood	Deep, wide-spreading laterals
Birches:	
Black, cherry, sweet	Deep, wide-spreading laterals
Paper, white	Shallow laterals
Yellow	Shallow, wide-spreading laterals
Cedars	Shallow, wide-spreading laterals
Cherry	Moderately deep, wide-spreading laterals
Chestnut	Taproot
Cypress	Several descending roots and many shallow, wide-spreading laterals
Elm	Shallow, wide-spreading laterals; occasionally a taproot
Firs:	
Balsam	Shallow, wide-spreading laterals
Douglas	Wide-spreading laterals
Lowland white	Deep, wide-spreading laterals
Noble	Moderately deep, wide-spreading laterals
White	Shallow laterals
Gums	Deep, wide-spreading laterals
Red, sweet	Shallow, wide-spreading laterals
Hackberry	Shallow, wide-spreading laterals
Hemlock	Shallow, wide-spreading laterals
Hickory	Deep taproot
Juniper	Deep laterals
Larch	Deep, wide-spreading laterals
Laurel	Deep, wide-spreading laterals
Locust	Deep, wide-spreading laterals
Magnolia	Deep, wide-spreading laterals
Mahogany	Shallow, wide-spreading laterals
Maple	Shallow, wide-spreading laterals
Oak	Deep taproot
Pine:	
Eastern white	Moderately deep; no taproot
Jack	Moderately deep, wide-spreading laterals
Loblolly	Short taproot (young), laterals
Lodgepole	Deep, wide-spreading laterals; always with taproot
Longleaf	Deep taproot; well-developed laterals
Nut	Shallow to moderately deep laterals
Pitch	Taproot (young); later laterals
Ponderosa	Moderately deep, wide-spreading laterals
Red, Norway	Strong taproot and laterals
Shortleaf	Very deep taproot
Slash	Deep, strong taproot with laterals
Stone (Foxtail)	Taproot supplemented by laterals
Sugar	Taproot (seedlings), deep laterals
Western white	Taproot (seedlings); deep, wide-spreading laterals
Poplar	Shallow laterals
Yellow	Deep, wide-spreading laterals
Quassia	Shallow, wide-spreading laterals
Redwoods	Several descending and many shallow, wide-spreading laterals
Spruce	Shallow, wide-spreading laterals
Sycamore	Shallow laterals
Tamarack	Shallow, wide-spreading laterals
Willows	Wide-spreading laterals

personnel needed, and plan a sequence of operations to complete the clearing rapidly and efficiently. In all clearing operations, the decisive factors controlling the method of clearing are the type and amount of equipment and the time available for completion.

TIMBER CRUISING

Timber cruising is performed to estimate the size, the height, and the number of each tree species in a given area. It is used either to determine the quantity of usable timber or to estimate the amount of work required in clearing. A sample, usually 10 percent of the area, is studied and the result is applied to the entire area. The sample may be increased or decreased. In small areas, a 100-percent cruise is usually made.

In timber cruising for land clearing only, record the diameters of the trees at breast height (DBH) taken at 4 1/2 feet above the ground, and record the species and number of trees. This information is used to plan the clearing operation and select the type of equipment most efficient for the diameters and species.

CLEARING CONSIDERATIONS

PERMAFROST

Clearing of ground cover over permafrost which is near the freezing point may result in thawing of material, causing considerable ground-surface subsidence.

SAFETY

Careful consideration must be given to the safety of personnel and equipment during clearing operations. Protective, tractor-mounted cabs should be used when extensive clearing operations are anticipated. Protective cabs permit greater flexibility in clearing operations and increase operator efficiency. With this protection, damage to the dozer is reduced and continuous production results.

Proper supervision and planning can help prevent accidents caused by falling trees, uprooted stumps, stump holes, and rough or broken terrain during the clearing operation. All equipment used in clearing should, if practicable, be equipped with heavy steel plating for protection of the undercarriages. This will prevent stumps, logs, and boulders from damaging vulnerable equipment parts.

CAMOUFLAGE

To provide cover and concealment (camouflage) for the construction site, do not remove standing trees and brush outside the designated cleared area unless necessary. When uprooting trees with bulldozers, take care to control their fall and avoid breaking surrounding trees.

TIMBER SALVAGE

Trim all timber useful for logs, piles, and lumber, and stockpile it for future use in bridge, culvert, and other construction applications. Push or skid this timber into a salvage area where it can be moved to a sawmill with little difficulty.

TEMPORARY DRAINAGE

Phased development of the drainage system in the early stages of clearing, grubbing, and stripping is essential to ensure uninterrupted construction. Delays caused by flooding, subgrade failures, heavy mud conditions, and the subsequent immobilization of construction equipment can be eliminated by careful development of the drainage system before, or concurrent with, other construction. Use the original drainage features as much as possible without disturbing natural grades. Grade drainage ditches downhill.

Fill holes left by uprooted trees and stumps with acceptable soil, and compact the ground to prevent the accumulation of surface water. Use dozers and graders for this work. Slope the ground toward drainage ditches to prevent pending on the surface. Backfill existing ditches at the latest possible time to permit the best use of the original drainage.

DISPOSAL

Use waste areas or burning to dispose of cleared materials. The choice of method depends on the type of construction, environmental concerns, the location, the threat, and the time available. Generally, the material is pushed and skidded off the construction site and into the surrounding timber to speed disposal and keep the area cleared for equipment operation. To dispose of material as rapidly as possible, assign specific units of equipment to accomplish this concurrently with the clearing and grubbing. The disposal method should be consistent with the methods of camouflage, salvage, and drainage used for clearing.

WASTE AREAS

In airfield construction, consideration must be given to the areas used for disposal of construction waste.

Dumps Adjacent to Work Areas

In forward combat areas where saving time is essential, the quickest and most convenient method of material disposal is to pile the materials adjacent to the work area. Study the construction plans to determine where the debris can be piled without interfering with drainage or potential work areas.

Off-Site Areas

In constructing the main project, it may be necessary to clear some adjacent land to dispose of the cleared material. Locate this clearing as close to the main project as possible to shorten the hauling distance, Use the same methods to clear disposal areas that arc used in clearing work areas.

Revetments

Cleared material can be disposed of by using it as fill material in revetments around hardstands when protective measures are needed.

Burning

Do not use fire for clearing land unless suitable equipment and sufficient personnel are not available for other methods of clearing. When burning is required, closely follow recommended procedures.

Under favorable tactical conditions, brush and timber debris may be burned. To limit the likelihood of detection because of smoke, keep fires burning as hot as possible and do not push new material into the fire rapidly. Do not permit fires to burn at night unless tactical conditions are extremely favorable and approval has been obtained from headquarters.

Fire Control. Strip the area around any debris to be burned before fires are started to provide a firebreak. If large areas are to be burned, establish firebreaks on all sides as a precaution against shifting winds. Maintain a fire guard over the fires as an additional safety measure. In dry weather, hand shovels, water buckets, and other expedient fire-fighting equipment should be available to extinguish fires caused by flying sparks.

Burning Pits. The most satisfactory method for burning large quantities of brush and timber is to burn them in a pit or trench dug by a bulldozer or scraper. The sides of the pit will reflect the heat back into the fire, producing a very hot fire, Burning will be rapid and complete. Push the material into the pit with a bulldozer. Start the fire with limbs and small brush to get a good bed of coals. Gradually increase the size of the material as the intensity of the fire increases. Get as little dirt as possible in the pit because it tends to smother the fire and fill the pit. A soldier should be detailed to tend the fire and ensure that the pile is kept compact, This method cannot be used in swampy areas where groundwater will seep into the trench.

Log Piles. If it is not desirable to construct burning pits, burn piles of logs by loosely piling them so that the heat and flames can pass through, It is always best to start the fire with brush. After a large bed of coals is formed, add a few logs at a time to obtain a good blaze.

To burn piles of green, wet logs, it may be necessary to use fuel oil to furnish enough heat to dry out the logs and start the burning process. Pile the logs parallel, one on top of the other. The fuel oil is carried to the center of the pile by a pipe in which holes have been drilled or cut. Once the pile is burning well, the fuel can be cut off and the pipe removed. Care must be taken to avoid ground contamination.

Fuel oil is also a quick and convenient means of starting brush fires, particularly if the brush is green and wet. If material is to be pushed onto the pile while the pipe is being used, it is best to bury the portion of the pipe outside the pile to protect it from damage from tractor grousers and bulldozer blades.

Clearing and Piling Stumps. In preparing stumps for burning, remove as much dirt as possible from the roots. Dirt on the roots will retard combustion and smother the fire. When the stumps are pushed out, leave them with the roots exposed to sun and wind so the dirt till dry quickly. Scrubbing with the side of the bulldozer blade will knock off much of the dry dirt. Pile the stumps as close together as possible with the trunks pointing toward the center of the pile. Keep the stumps together after they start to burn. This procedure will speed up the burning.

AIRFIELD APPROACH ZONES

Airfield glide angles and approach zones are further discussed in Chapter 11 of FM 5-430-00-2/AFPAM 32-8013, Vol 2. Obstructions extending above the glide angle must be removed. Although glide-angle requirements may be met by only topping trees, it is best to fell or uproot trees that extend above the glide angle. Disposal is no problem in the approach zone, because all demolished material is left in place.

PERFORMANCE TECHNIQUES

CLEARING WITH EQUIPMENT

The use of engineer equipment is the most rapid and efficient method of clearing. The use of such equipment is limited only by un usually large trees, stumps, and terrain that decrease the maneuverability of the equipment and increase maintenance requirements. This type of equipment includes bulldozers; tree-dozer, tractor -mounted units; tractor -mounted clearing units; winches; power saws; rippers; and motor graders. In addition, pioneer tools are used for some clearing operations. Table 4-2 summarizes the limitations and proper applications of engineer equipment in clearing operations. Use production rates of equipment under normal operating conditions for determining the total time required for the job. Clearing rates are discussed in FM 5-434. Limitations and applications for each type of equipment follow.

Bulldozer

When clearing an area in dry or temperate forests, the bulldozer is the most efficient mechanical equipment for removing small brush, trees, and stumps up to 6 inches in diameter. Although more time and effort are required, bulldozers can also remove trees up to 30 inches in diameter when tractor-mounted clearing units and power saws are not available. Because of its ability to push, move, and skid felled trees and brush, the bulldozer is used extensively as the primary unit of equipment in all clearing operations.

Table 4-2. Applications and limitations of engineer equipment in land clearing

Equipment	Applications	Limitations
Bulldozer	-Primary equipment for all land clearing. -Excellent for removing brush, trees, and stumps up to 6 inches in diameter. -Push, pull, or skid cleared material for disposal.	-Trees over 6 inches in diameter require special and slower methods of removal by dozer. -Maneuverability limited in muddy or swampy terrain and in dense, heavy growth.
Tree-dozer, tractor-mounted unit (Rome Plow)	-Medium clearing of brush and trees at ground level rather than uprooting.	-Skilled personnel required for cutting of trees; other units required for completion of clearing when burning is not permitted.
Tractor-mounted clearing unit	-For extensive clearing operations requiring heavy pulling. -Uproot trees and stumps of almost unlimited diameters. -Skid cleared material for disposal. -Extricate mired equipment. -Excellent for operation in jungles, swamps, and bottom lands with heavy growth.	-Skilled personnel required for rigging. -Slow in clearing an area; other units required for speedy completion. -Not TOE.
Winches (towing):		
Tractor-mounted	-For general light and medium pulling. -Uproot trees and stumps up to 24 inches in diameter. -Skid cleared material for disposal. -Extricate mired equipment.	-Pulling capacity limited by size of tractor. -Terrain affects maneuverability of tractor.
Truck-mounted	-Expedient for light pulling of trees up to 6 inches in diameter. -Skid small trees and brush. -Extricate mired equipment.	-Rigging personnel required. -Terrain must be suitable for truck use. -Pulling capacity too limited for most operations.
Felling equipment:		
Chain saw	-Controlled felling of trees of almost unlimited diameters. -Saw timber for salvage. -Rapid felling.	-Other units required for uprooting stumps and disposing of felled timber. -Pneumatic saws are very dangerous to use on steep, rugged ground. Air hoses frequently are fouled and broken by rolling logs and chunks. Gasoline chain saws are far easier to handle than the pneumatic ones because there are no hoses to contend with. They can be used in any type of terrain with a reasonable degree of safety if operated by skilled operators.

Table 4-2. Applications and limitations of engineer equipment in land clearing (continued)

Equipment	Applications	Limitations
Circular or chain saw mounted on tractor	-Saw timber for salvage. -Rapid felling. -Excellent for clearing heavy, dense growth in rough and broken terrain.	-Other units required for uprooting stumps and disposing of felled lumber. -Maneuverability limited in muddy or swampy terrain and in terrain too steep for tractor to negotiate. -May bind in unbalanced tree, requiring extensive looping of tractor pull line.
Ripper	-Cut free roots. -Loosen surface boulders. -Loosen soil for stripping.	-Depth of shank penetration limits use to shallow roots. -Maneuverability limited in muddy or swampy terrain and in dense, heavy growth.
Grader	-Light clearing of grass, weeds, and small brush/vegetation. -Windrow cleared material. -Grade cleared area for drainage.	-Maneuverability limited to level terrain free of trees, stumps, and boulders. -Careful operation required to prevent damaging blade.

When clearing with bulldozers, the sequence of operations depends on the type of trees, the terrain, and planned construction. After establishing the boundaries of the clearing, select spoil areas for disposal of all cleared material based on the shortest haul, a downgrade slope, effective camouflage, and general accessibility.

Start clearing at the disposal area and move in each direction away from it. Use one or two dozers to clear the small trees and brush only. Another pair of dozers will remove the larger trees and stumps bypassed by the previous units. If necessary, add more dozers for a third cycle of operation to take care of the heaviest removals.

Move the cleared material to the spoil area by skidding, pushing, or pulling. Disposal should be done with uprooting and removing. It is best to have a separate crew assigned for disposal.

Multiple operations are possible when other types of equipment are available, using each type where it is most effective. Use power saws, for example, to fell large trees. Use clearing units to uproot large stumps and work in areas inaccessible to dozers. Use bulldozers to clear, stockpile, and dispose of light material. The operational methods used by bulldozers in clearing depend on the size of the trees. The methods briefly discussed below are discussed fully in FM 5-434.

Small Trees, 6 Inches or Less in Diameter, and Brush. In clearing small trees and brush, operate the bulldozer with the blade straight and digging slightly. It may be necessary to back up occasionally to clear the blade. The cleared material can either be pushed into windrows for later removal or pushed off to one side of the area to be cleared.

Medium Trees, 6 to 12 Inches in Diameter. To push over trees that range from 6 to 12 inches in diameter, set the blade of the bulldozer as high as possible to gain added leverage (Figure 4-1). As the tree falls, the bulldozer is backed up quickly to clear the roots. With the blade lowered, the dozer travels forward again and digs the roots free by lifting the blade. The felled tree is then ready for removal to the spoil areas.

Large Trees. Removing large trees (over 12 inches in diameter) is much sIower and more difficult than clearing brush and small trees. First, gently and cautiously probe the tree for dead limbs that could fall and injure you. Then, position the blade high and center it for maximum leverage. Determine the direction of fall before pushing the tree over; the direction of lean, if any, is usually the direction of fall, If possible, push the tree over the same as you would a medium tree.

However, if the tree has a large, deeply embedded root system, use the following method (Figure 4-2, page 4-10):

Step 1. Opposite the direction of fall, make a cut deep enough to cut some of the large roots. Use a V-ditch cut around the tree, tilted downward laterally toward the tree roots.

Step 2. Cut side two.

Step 3. Cut side three.

Step 4. To obtain greater pushing leverage, buld an earth ramp on the same side as the original cut. Then push the tree over. As the tree starts to fall, reverse the tractor quickly to get away from the rising root

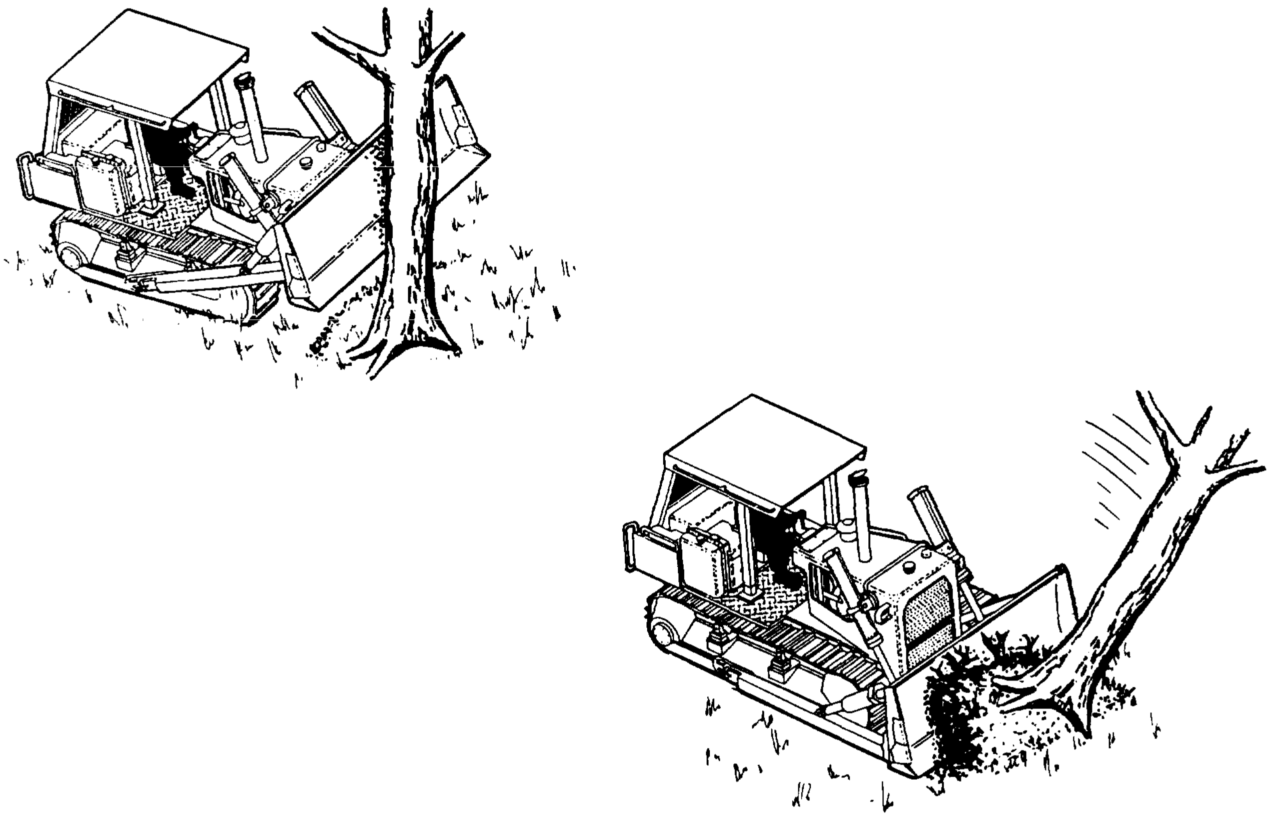


Figure 4-1. Bulldozer removing medium-sized trees, 6 to 12 inches in diameter

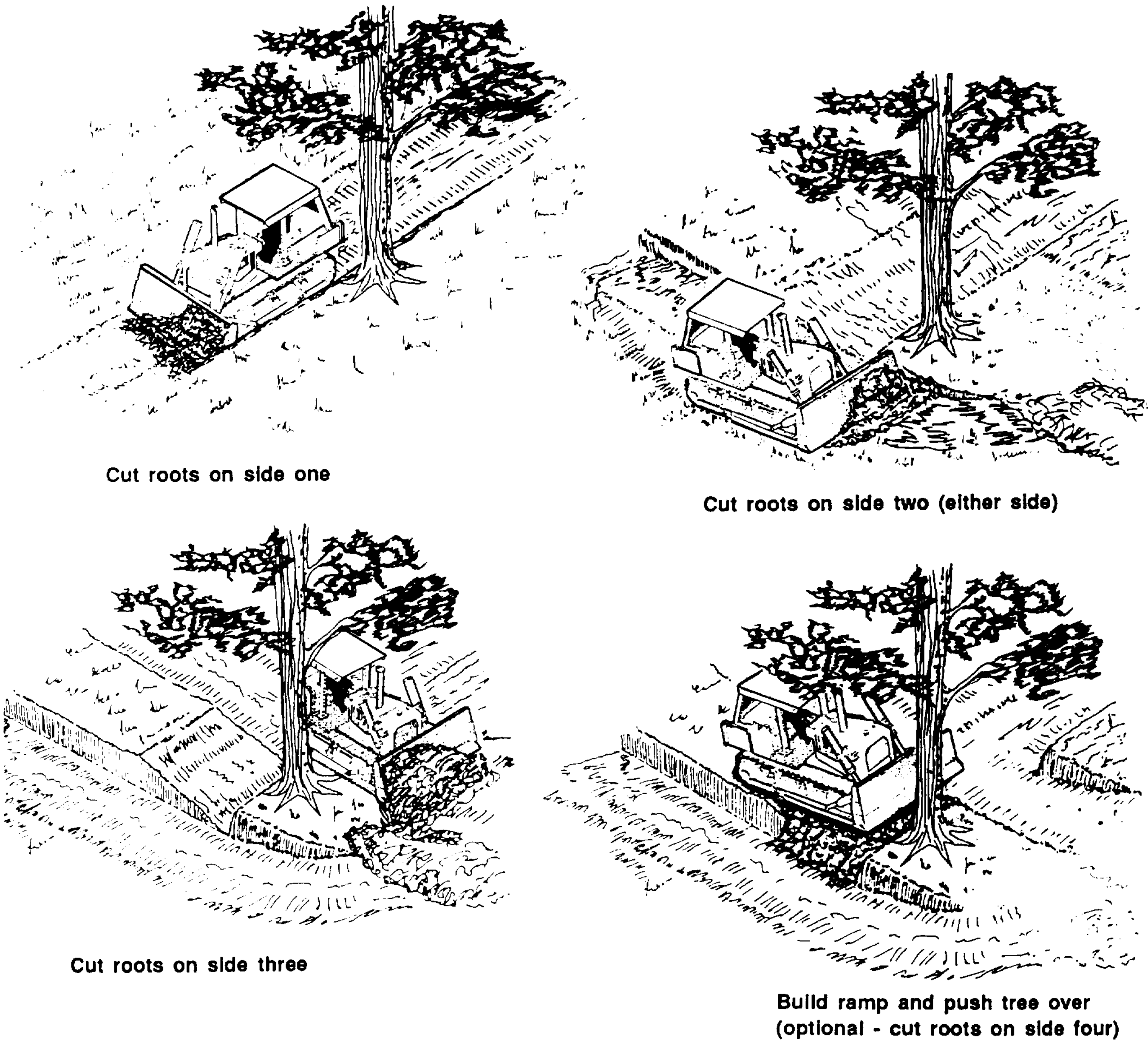


Figure 4-2. Four steps for removing large trees with a bulldozer

mass. After felling the tree, fill the stump hole so that water will not collect in it.

NOTE: The roots on the fourth side may need to be cut also.

Tree-Dozer, Tractor-Mounted Unit

The tree-dozer, or Rome plow, is a tractor with a blade that *stings* and *slices* large trees. A sharp projection on the left side of the blade splits the trees, while the cutting edge shears them off at ground level. The operator is protected by a steel canopy and a guide bar that controls the direction of falling trees.

The tree-dozer is a simple and efficient piece of equipment used for military land-clearing operations. It does not appreciably disturb the soil. It provides—

- Clear fields of fire and security around cantonments, airfields, and other facilities.

- Right-of-way clearance to desired depths along roads and railroads, thereby reducing the enemy's capability of ambush.

Before committing a tractor equipped with the tree-dozer mounting, investigate the soil condition in the area of operation to determine if it will support the equipment. Use the tree-dozer mounting to make cuts through any kind of forest except heavy swampland. Shear trees at ground level, sweep them into piles or windrows, and dispose of them. One tractor equipped with a tree-dozer mounting can clear approximately 1 to 2 acres per hour, depending on the tree density and size. Use one of the following clearing methods:

Ž When the tractor can move forward almost continuously, it shears to ground level anything in its path, Fast production can be obtained by laying out long areas (200 to 400 feet wide) that can be cut from the outside toward the center in a counterclockwise direction. The cut material then slides off the trailing (right) end of the tree-dozer mounting and leaves the uncut area free of fallen debris. The windrows are placed lengthwise on the borders of the areas. Piling is done by sweeping with the tree-dozer mounting. Sweep a blade width at a time. Work from the center of each area, at a right angle to the border (Figure 4-3).

Ž Another method is shown in Figure 4-4, page 4-12. Again, long areas are laid out in 200- to 400-foot widths, but the cutting is done from the center toward the sides in a clockwise direction. This allows the cut material to fall toward the center, which becomes the windrow site. The piling is done with the tree-dozer mounting, following the pattern outlined on the right side of Figure 4-4. When windrowing, the operator keeps the cutting edge on the ground while pushing into the windrow and raises it

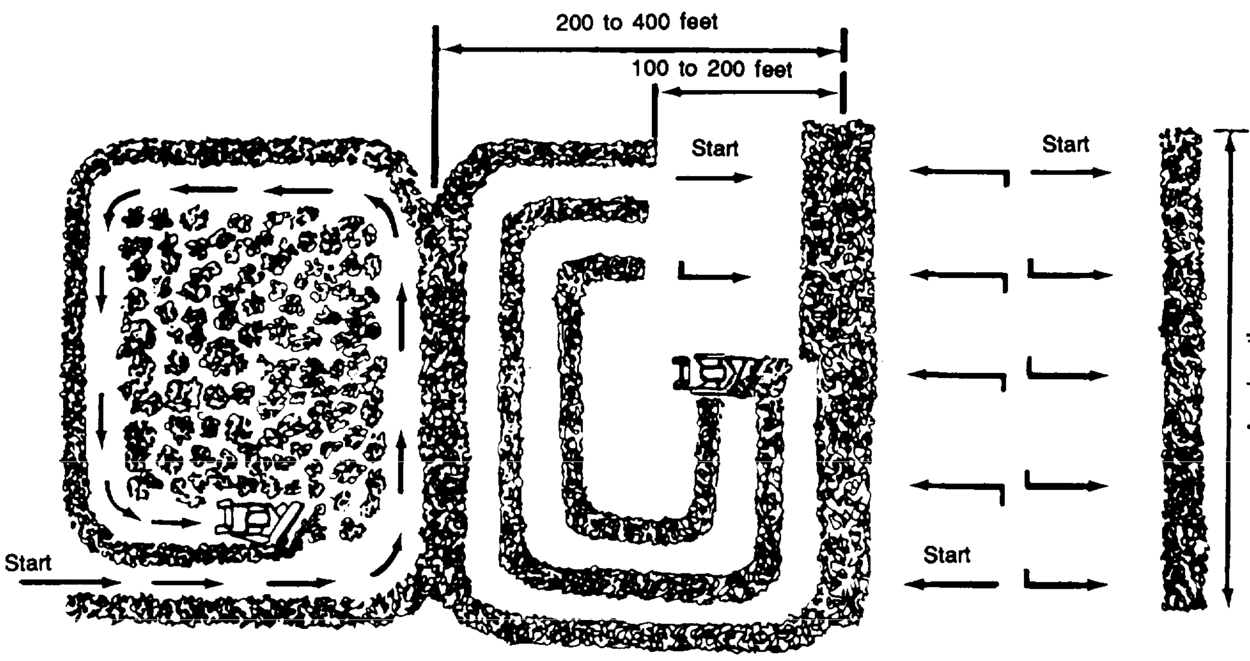


Figure 4-3. Cutting vegetation to ground level and piling cut material using the counterclockwise method

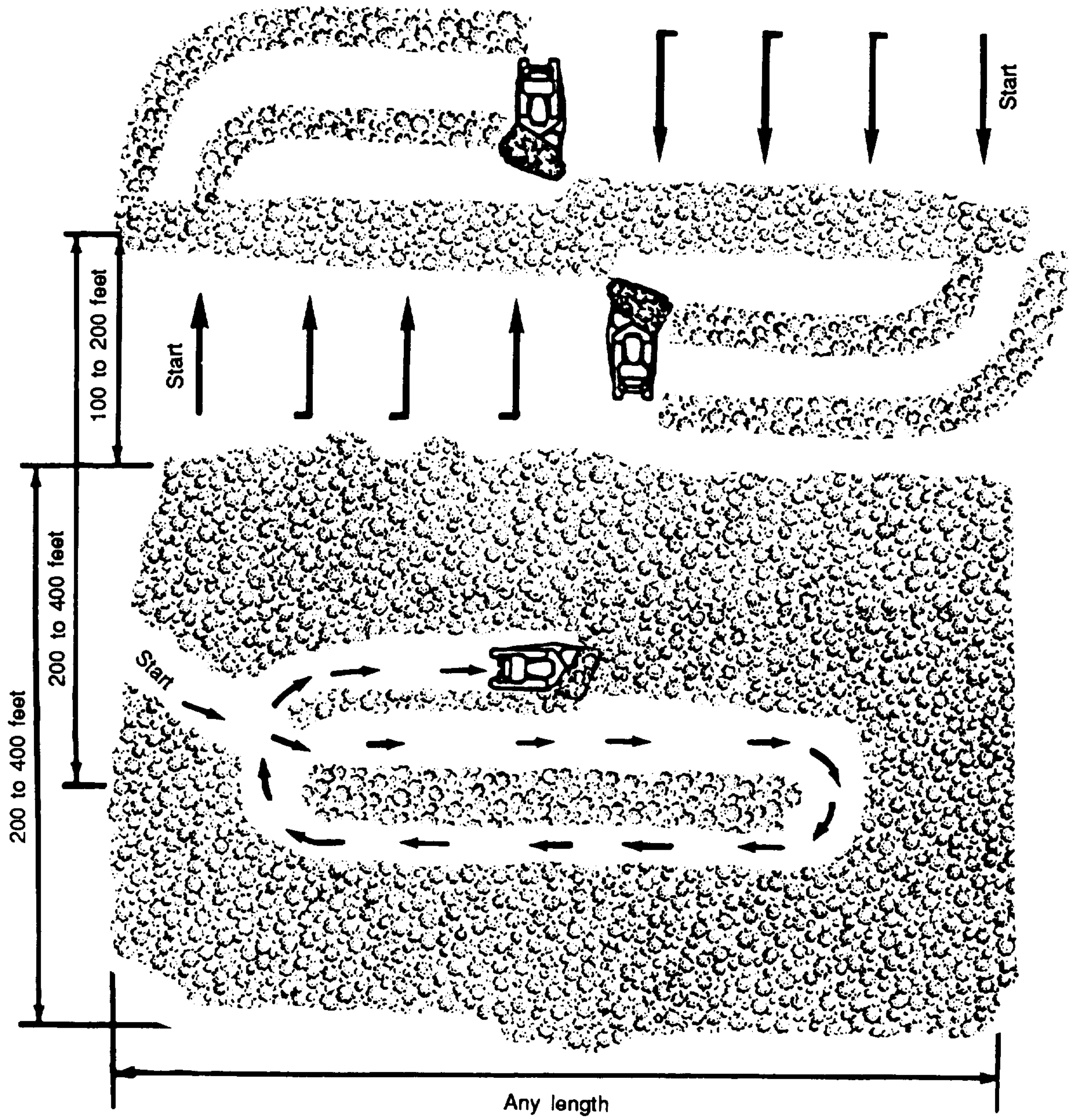


Figure 4-4. Cutting vegetation to ground level and piling cut material using the clockwise method

when backing away. This allows accumulated soil to sift away and lessens soil deposits in the windrow.

- On extreme slopes, rapid production is obtained by working in a semicircular pattern, from left to right, at approximately right angles to the windrow (Figure 4-5). If the terrain is steep, the windrows should be on the contour, and the tractor should work from the uphill side and push downhill to the windrow.

- Where the vegetation is dense and small, the highest production can be obtained by cutting and windrowing simultaneously. Work from left to right at a

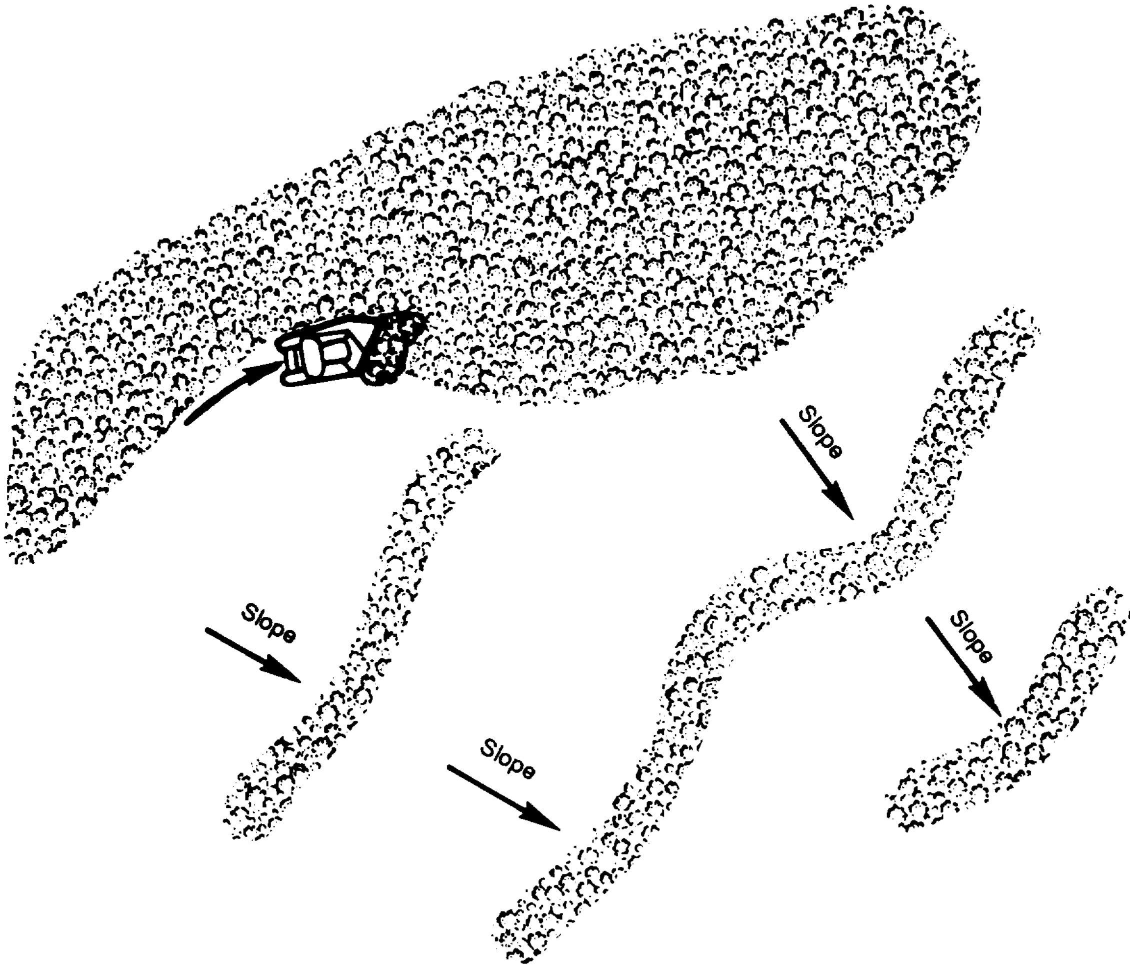


Figure 4-5. Clearing on steep slopes covered with large trees

90-degree angle to the windrow, with the trailing edge of the tree-dozer working against the uncut material. This prevents cut material from sliding off the moldboard and allows the cut material to accumulate on the mold board.

When the moldboard is filled, the operator should stop the tractor and deposit the cut material. The operator should then reverse to the starting point and repeat the operation to the right (Figure 4-6, page 4-14), reducing the time lost in backing up. When the tractor reaches the previously cut material, the operator should deposit cut material and form another windrow.

The area of vegetation should be laid out as shown in Figure 4-6, with the operator working in patches, from inside to outside in a counterclockwise direction and at right angles to the windrows. Sweeping and piling the resulting debris can be accomplished much faster when tractors are used in teams traveling abreast.

Winches

Towing winches mounted on tractor-dozer units or trucks are limited in use for clearing operations because of their small capacities in comparison with the tree- and stump-pulling units.

Tractor-Mounted Winch. Use tractor-mounted winches for uprooting trees and stumps up to 24 inches in diameter.

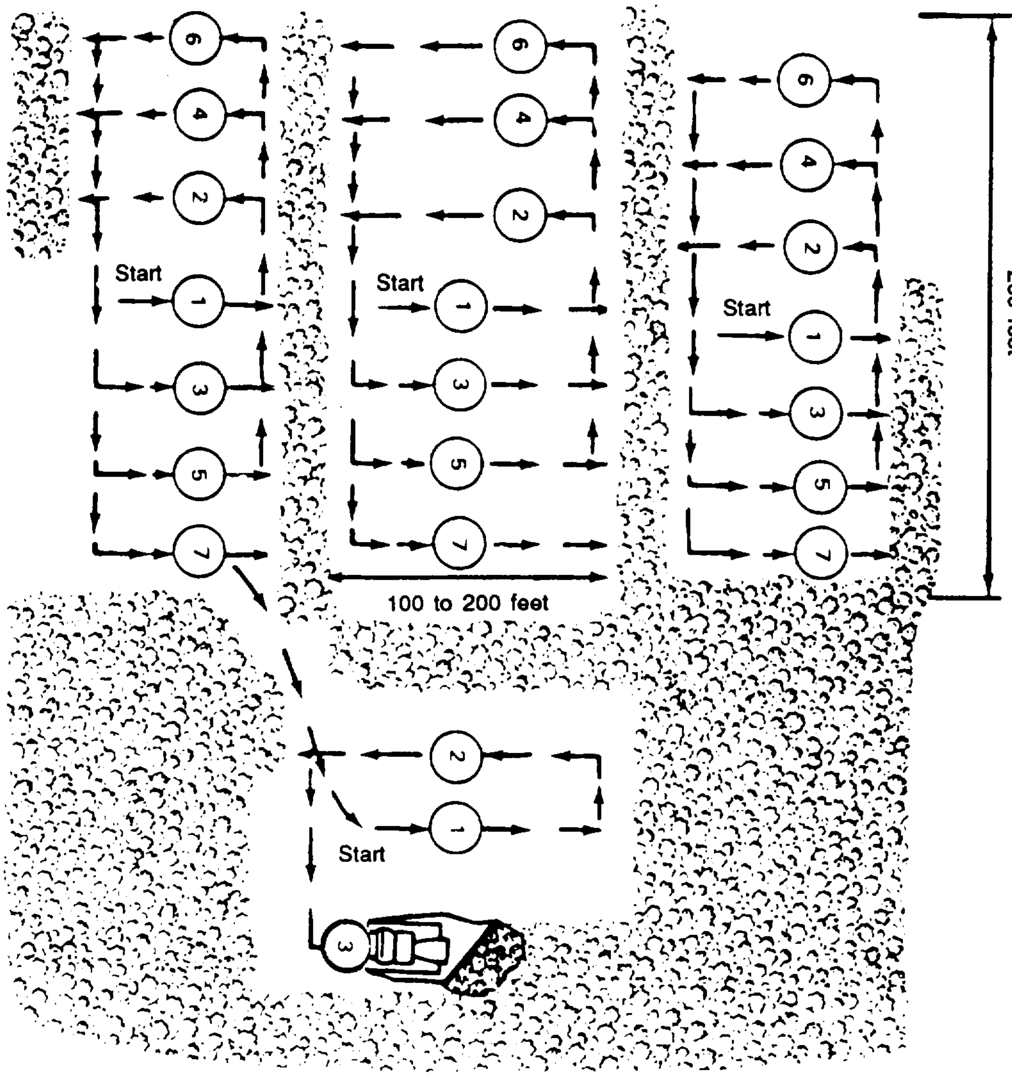


Figure 4-6. Cutting and piling dense growths of small-diameter vegetation on level terrain

hoisting and skidding felled trees, and extricating mired equipment. On the tractor-dozer unit, the winch is mounted in the rear and is directly geared to the rear power take-off on the tractor. The line pull developed varies with the size of the tractor, the speed at which the winch is operated, and the number of layers of rope on the drum. The line pull of a tractor-mounted winch is only about one-half the pull of a standard tree- and stump-pulling unit.

Truck-Mounted Winch. As an expedient, truck-mounted winches can be used on trees up to 6 inches in diameter. Their capacities are too limited for heavy work. Their best use is for skidding felled trees and logs to a disposal area, if the haul road is sufficiently cleared for trucks to operate.

Felling Equipment

Felling can be done with hand tools or power equipment. Axes, two-man saws, shovels, pick-mattocks, and machetes are used to chop or saw down standing timber; dig and uproot stumps: and slash grass, vines, and undergrowth. Clearing by hand is usually too slow and difficult for military requirements unless explosives or mechanical methods are used. When labor is plentiful, forests are dense, and terrain is rough, this method of clearing can be used with good results. Power equipment and chain and circular saws are the principal ways of felling limber.

Ripper

In land clearing, the ripper is used to help in the removal operations of bulldozers and tree- and stump-pulling units. The ripper cuts and breaks tree roots and loosens boulders from the ground. The short depth of shank penetration limits its use to shallow root systems. Prior to stripping operations, the ripper is used to loosen and break up frozen soil or organic material for easier removal by graders or scrapers.

Grader

The grader is used 10 cut grass and weeds, remove small brush, and clear the area of dead vegetation. The terrain must be level and free from boulders and trees. Used with rippers and bulldozers, graders can windrow the cleared material for later removal by other equipment. The grader is extremely limited in most clearing operations.

CLEARING WITH EXPLOSIVES

Explosives may bc used to fell standing trees, uproot entire trees and stumps, and remove and dispose of large boulders. Explosives, however, have several disadvan - tages. The sound of the explosive can travel farther than the sound of the construction equipment. In loose soil, the initial charge may be entirely a expended in compacting the soil under a tree or stump, and a second charge may be required to remove it. Deep taproots often are only broken by explosives and have to be removed by hand. Also, explosives generally take time to place and they create large craters, which require borrow excavation and compaction to backfill. In spite of these disadvantages, it is still sometimes necessary to use explosives to clear an area where the terrain precludes or seriously impedes the operation of heavy equipment. Refer to FM 5-250 for the correct application of explosives and demolitions.

Trees and Stumps

Methods of tree and stump blasting vary with the size and condition of the tree, root structure, and ground conditions, Figure 4-7, page 4-16, shows the methods of placing charges to blast stumps with different root structures. Table 4-1, page 4-3, shows the type of root system for several tree species in temperate forests.

The size of the charge required depends on the strength of the explosive available; the size, variety, and age of the tree or stump; and soil conditions.

Various types of tools (such as pinch bars, earth augers, and spoons) can be used when drilling holes for the charge. Wood augers can be used for taproots. For loading and tamping, any smooth, wooden pole about 5 feet long and 1 1/2 inches in diameter can be used. The handle from a long-handled shovel is excellent because the crook of the blade end provides a good grip. All holes must be tamped firmly with earth to retain the full force of the explosion.

Boulders

When boulders cannot be used in an embankment or fill, they must be removed from the construction area. Blasting is a quick and easy method of dislodging boulders. Mudcapping, blockholing, or snakeholing (described in Chapter 3 of FM 5-250 or Chapter 6 of FM 5-34) may be used. Refer to Chapter 3 of TM 5-332 for quantities and types of explosives to be used and details regarding blasting rock.

REMOVAL OF SURFACE ROCKS

All surface rocks must be removed in certain types of construction. There are three methods used in this operation: hand, bulldozers, and cranes or scoop loaders with trucks. The choice of method depends upon the situation.

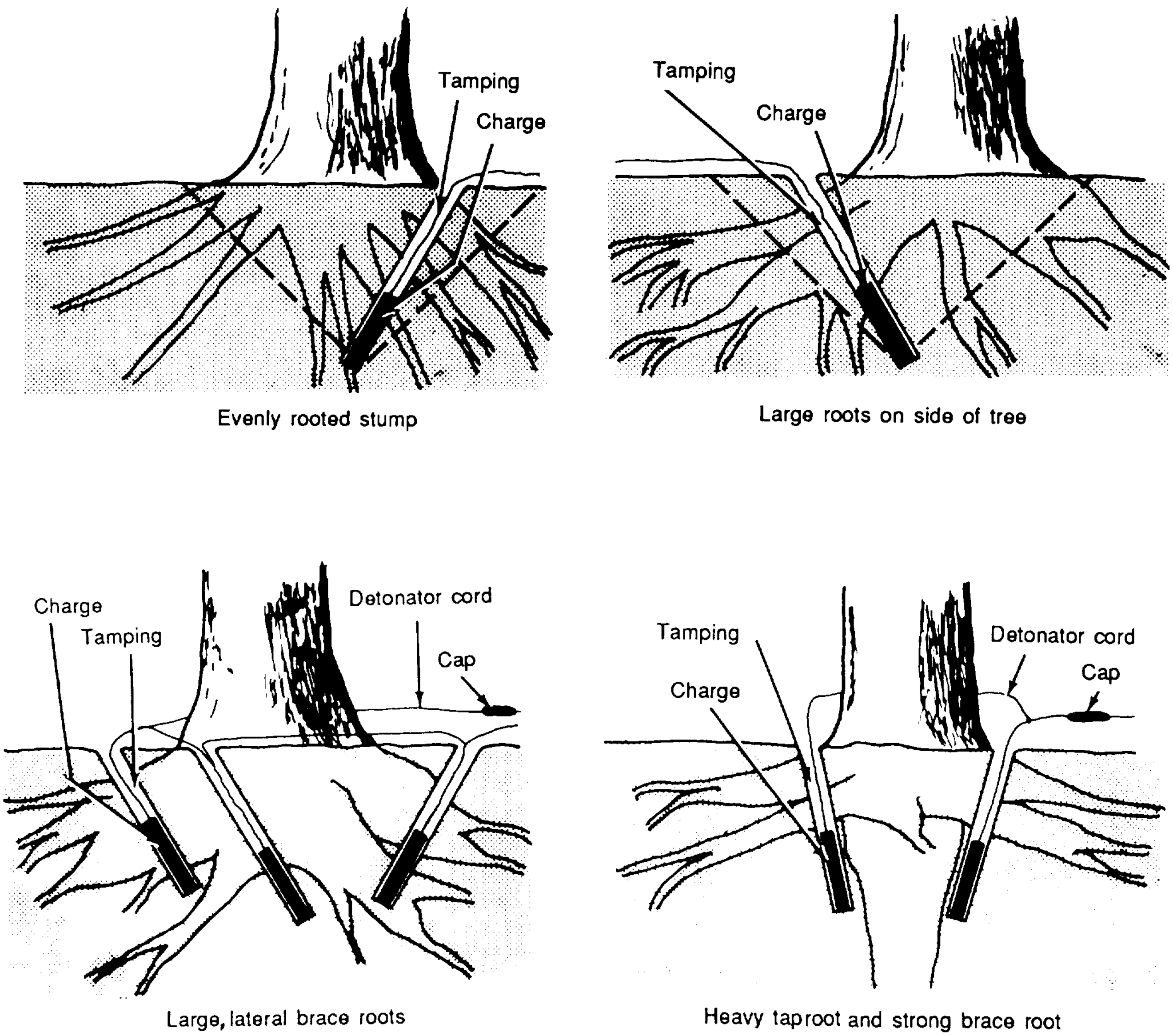


Figure 4-7. Stump-blasting methods for different root structures

Hand

When there is sufficient time and personnel, rocks are picked up and loaded into hauling units by hand. This slow method is used in military construction only as an expedient.

Bulldozer

The bulldozer is the most commonly used engineer equipment for moving rocks to a fill or disposal area. The rocks may be windrowed by dozers for later removal by scrapers or shovels and trucks. If the distance is short, they may be pushed by dozers to the designated disposal area.

Cranes or Scoop Loaders with Trucks

Clearing surface rock by this method alone is possible, but it is slow. If the rocks are first windrowed or moved into piles by bulldozers or graders, the shovel will load the trucks quickly and efficiently. The rocks can then be hauled long distances for disposal.

STRIPPING

Stripping consists of removing and disposing of the topsoil and sod that cannot be used as a subgrade, foundation under a fill, or borrow material. Examples of this material are organic soils, humus, peat, and muck, Unsuitable soil must be removed to a depth at which compaction and thickness requirements are satisfied. Stripping is done concurrently with clearing and grubbing by using bulldozers, graders, scrapers, and sometimes shovels, Good topsoil and sod should be stockpiled for later use on bare areas for dust or erosion control or for camouflage.

REMOVAL OF STRUCTURES

An airfield construction site may be acceptable except for obstructions such as houses, railroads, power lines, and other structures on the proposed site or near the operation of aircraft. The primary selection of a site always involves compromises, The survey party often selects a site where limited clearing of structures will be necessary before full-scale operations can take place.

Power Lines

Power lines obstructing forward area construction or glide angles should be removed. In rare instances, the lines may be needed intact and a different approach to the airfield must be used. If the lines cross the runway, relocate them around the nearest end of the runway. As a last resort, underground installation may be used if armored underground cable or conduit is available that will adequately insulate the lines, In rear areas, power lines that are not in an approach zone or not high enough to extend into the glide angle when located in an approach zone may be marked with suitable warning lights.

Roads and Railroads

In general, roads and railroads present no obstructions when located near airfields, if the traffic does not interfere with the approach or takeoff of aircraft. Do not destroy main paved highways and railroad lines because they may be required for ground operations. It is desirable to have the airfield located near a good road or railroad so supplies may be readily transported to the site.

Buildings

Buildings may be completely razed with explosives or heavy equipment, leaving no salvage. They may be razed in a manner to conserve usable material, or they may be relocated.

REMOVAL OF BURIED EXPLOSIVES

Mines, booby traps, unexploded ordnance (UXO), and other buried explosives must be located and removed or neutralized before any construction operations begin. Check all reports and data on an area to locate and identify explosives.

Establish the boundaries of the construction area first, then make a visual search of the most likely places for explosives. They are usually near existing structures, houses, and roads; in disturbed ground hollows where the earth has visibly settled; and under stockpiles, pickets, or stakes placed in unnatural locations. If time allows, thoroughly investigate the area with mine detectors or by probing methods.

The safety of personnel and equipment is primary at all times during the removal of mines. The speed of clearance is secondary, Whenever possible, use trained explosive ordnance disposal (EOD) detachments, particularly when the size of bombs precludes detonation in place.

The method of mine and UXO removal is a command decision. For minefield with booby traps or other antihandling devices, it is best to destroy the mines in place by explosives or mechanical means. UXOs may be disarmed by ordnance personnel and then manually removed from the area and disposed. If ordnance personnel are not available, destroy the device in place.

Surface mines or bomblets may be able to be pushed from an area by using a drag chain or dozer. If devices are destroyed in place, all resulting holes and craters must be filled and compacted with acceptable material, using dozers or graders. For additional information, see FM 5-434.

SUBGRADES AND BASE COURSES

CHAPTER 5

This chapter discusses the functions of the subgrade, subbase, and base courses and covers the selections of materials and construction procedures. Chapters 8 and 9 of this manual discuss the determination of the base- and surface-course thickness for roads. Chapters 11 and 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2, include similar information for all classes of airfields and heliports as well as all commonly used types OJ surface materials. Additional information on soils, compaction, and California Bearing Ratio (CBR) requirements is contained in Chapters 2, 6, and 10 of FM 5-410 and in Chapter 2 of FM 5-530.

DESIGN CONSIDERATIONS

Pavement structures may be rigid or flexible. In rigid pavements, the wearing surface is made of portland cement concrete. A rigid pavement made of concrete will have great flexural strength, permitting it to act as a beam and allowing it to bridge over minor irregularities that may occur in the base or the subgrade upon which it rests.

All other types of pavements are classified as flexible. In a flexible pavement, any distortion or displacement occurring in the subgrade is reflected in the base course and continues upward into the surface course. Flexible denotes the tendency of all courses in ibis type of structure to conform to the same shape under traffic.

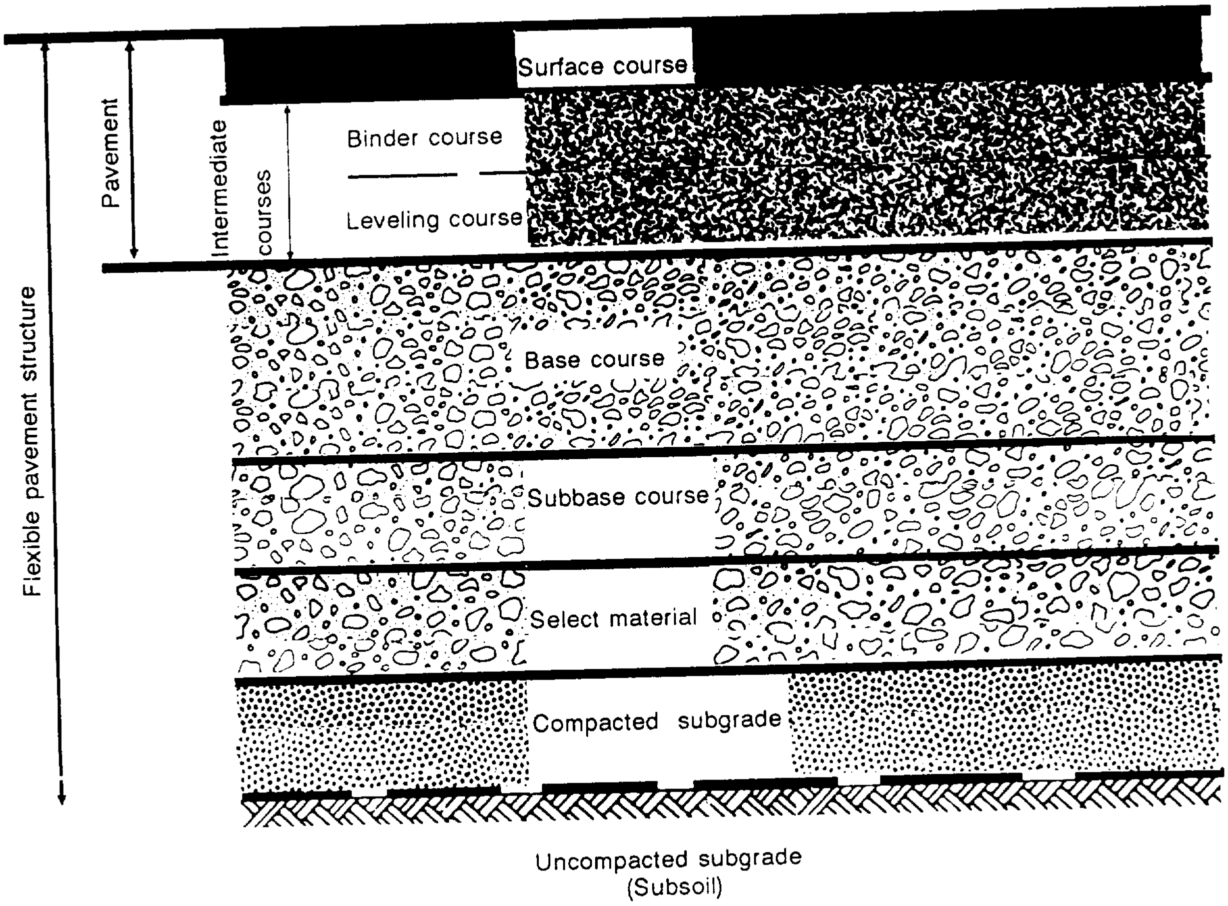
Flexible pavements are used almost exclusively in the TO for road and rear area airfield construction. They are adaptable to almost any situation, and they fall within the construction capabilities of the combat heavy engineer battalion and its support units. Rigid pavements are generally not suited to TO construction requirements (unless the materials are more readily available) and are not discussed in detail in this chapter.

FLEXIBLE-PAVEMENT STRUCTURE

A typical flexible-pavement structure is shown in Figure 5-1, page 5-2. It illustrates the terms used in this manual that refer to the various layers. All the layers shown in Figure 5-1 are not present in every flexible pavement. For example, a flexible-pavement structure may consist only of an asphaltic-concrete surface, a base course, and the subgrade. The design of flexible pavements must include a thorough investigation of the subgrade conditions; borrow areas; and sources of select, subbase, and base materials.

TESTS

Engineers should classify soils according to Chapter 2 of FM 5-410, and then select representative samples for detailed tests. Detailed tests determine compaction characteristics, CBR values, and other properties



NOTE: All layers and coats are not present in every flexible-pavement structure. Intermediate courses may be placed in one or more lifts. Tack coats may be required between the intermediate courses and under the surface course. A prime coat may be required between the highest aggregate surface and the first layer of asphalt.

Figure 5-1. Typical flexible-pavement section

needed for designing the flexible-pavement structure. Subbase-and base-course materials are tested for compliance with specification requirements for gradation, liquid limit (LL), plasticity index (PI). and CBR values. When tests arc completed, limiting conditions in the subgrade and subsoil must be determined. Materials are selected for each layer based on their characteristics (gradation, LL, PI. and CBR values).

DISTRIBUTION OF LOADS

Flexible-pavement design is based on the principle that the magnitude of stress induced by a wheel load decreases with depth below the surface. Consequently, the stresses induced on a given subgrade material can be decreased by increasing the thickness of the overlying layers (subbase, base, and surface courses). Figure 5-2 shows the distribution of a single-wheel load on two

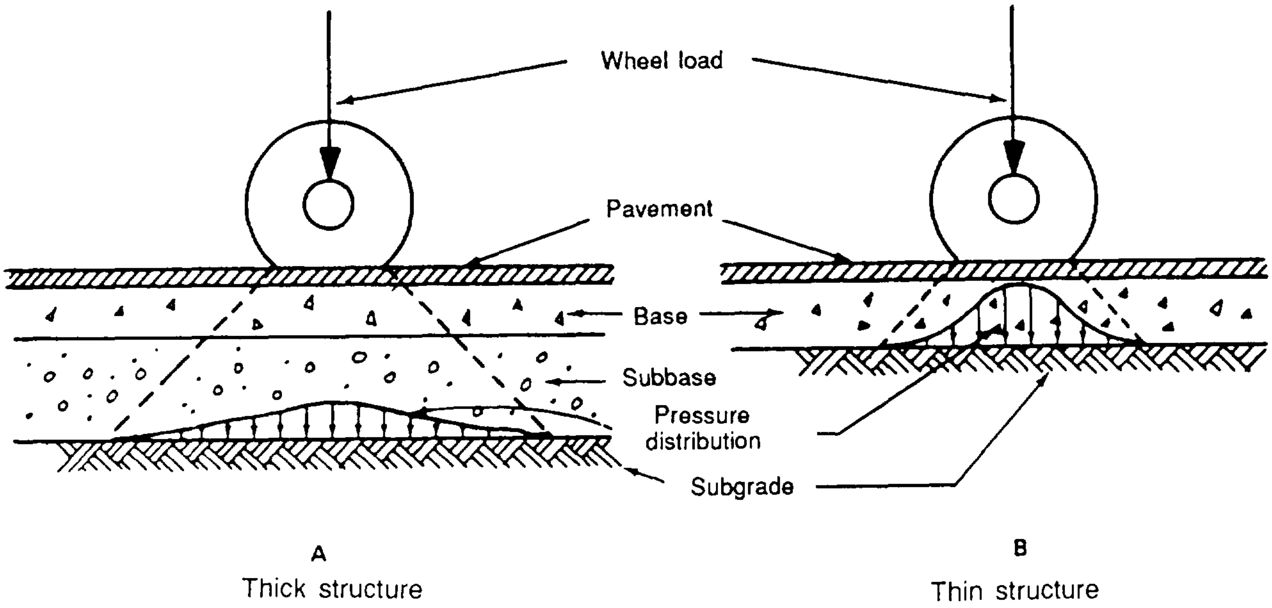


Figure 5-2. Distribution of pressures under single-wheel loads

sections of flexible pavement. one with a thick and one with a thin flexible-pavement structure. In both cases, the subgrade is the found ation that eventually carries any load applied at the surface. Figure 5-2 demonstrates that the magnitude of the stresses on the subgrade decreases as the flexible-pavement structure is thickened. In the left diagram in Figure 5-2, the flexible-pavement structure is thick, the load at the subgrade level is spread over a wide area, and the stresses on the subgrade arc low. In the right diagram the structure is thin, the load at the subgrade level is confined to a much smaller area, and the stresses on the subgrade level are significantly higher. The pattern of decreasing stresses with increasing depth is the basis of the conventional flexible-pavement design in which subgrade materials of low-bearing capacity arc covered with thick flexible-pavement structures.

The distribution of pressures under a multiple-wheel assembly is shown in Figure 5-3, page 5-4. Multiple-wheel assemblies are beneficial because the stress distributions produced by the tires do not overlap to a large degree at shallow depths. This is illustrated at line A-A in Figure 5-3. Therefore, multiple-wheel assemblies are beneficial on thin, flexible- pavement structures constructed on subgrades with high-bearing capacity.

The intensity of stress at a given point in a flexible pavement is affected by the tire-contact area and tire pressure. The major difference in stress intensities caused by variation in tire pressure occurs near the surface. Consequently, the surface course (pavement or a well-graded crushed aggregate) and base course arc the most seriously affected by high tire pressures.

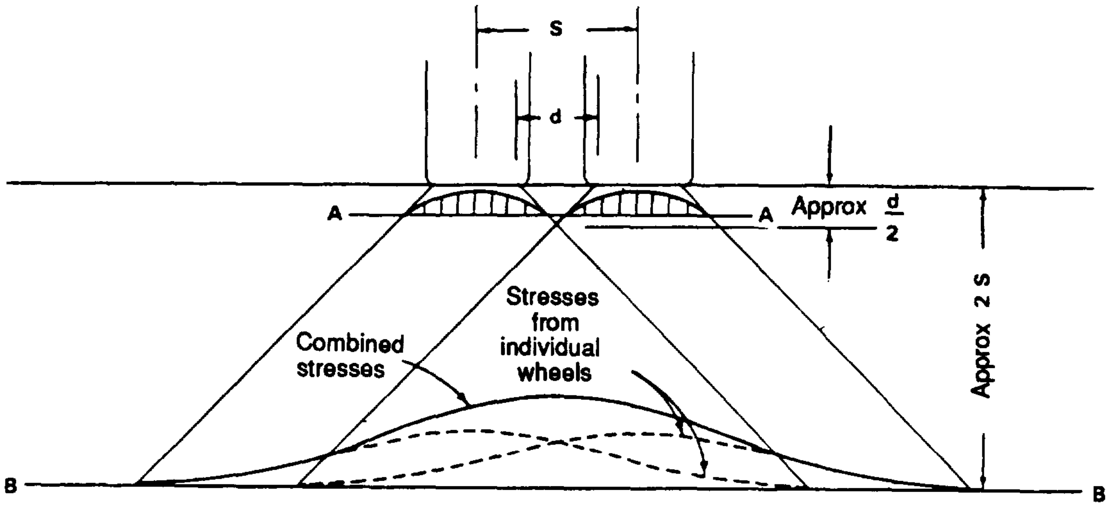


Figure 5-3. Distribution of pressures produced by muitiple-wheel assemblies

SUBGRADES

Using information from a deliberate soil survey as outlined in Chapter 2 of FM 5-530, consider the following factors when determining the suitability of a subgrade:

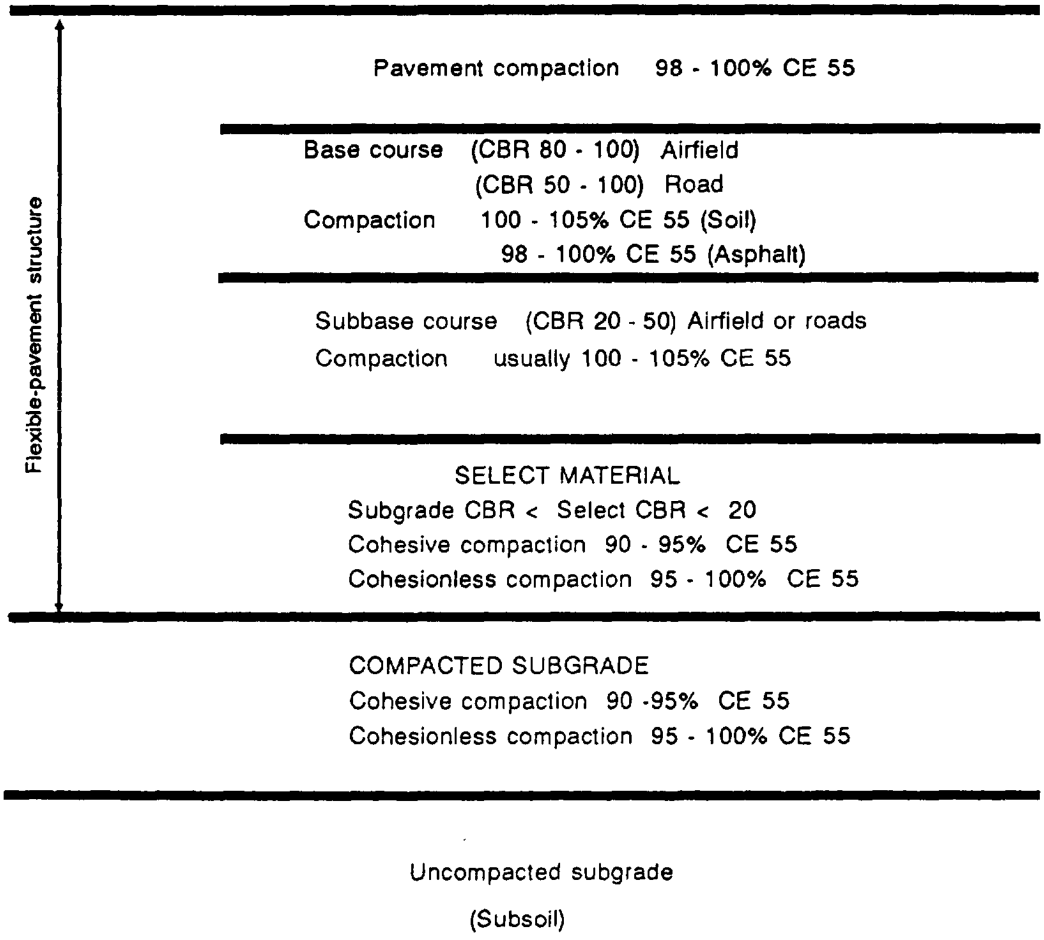
- General characteristics of the subgrade soils.
- Depth to bedrock.
- Depth to the water table.
- Compaction that can be attained in the subgrade.
- CBR values of uncompacted and compacted subgrades.
- Presence of weak or soft layers or organics in the subsoil.
- Susceptibility to detrimental frost action or excessive swell.

GRADE LINE

Classify the subgrade soil in accordance with the Unified Soil Classification System (USCS) (as described in FM 5-410) and consider the previously listed factors to determine the suitability of a subgrade material. When locating the grade line of a road or airfield, consider the suitability of the subgrade, the depth to the water table, and the depth to bedrock. Generally, the grade line should be established to obtain the best possible subgrade material consistent with the design parameters. However, simplicity of construction must also be considered.

COMPACTION

Compaction normally increases the strength of subgrade soils, The normal procedure is to specify compaction according to the requirements in Figure 5-4. A specification block should be used to determine limits for density and moisture content.



NOTES:
1. A cohesive soil is one with PI > 5.
2. A cohesionless soil is one with a PI ≤ 5.
3. Percent compaction is a percent of the maximum density at CE 55.
4. Each layer shown will not necessarily be used in the final design.
5. The minimum compacted layer thicknesses are 4" for a road and 6" for an airfield.

Figure 5-4. Recommended compaction requirements for rear areas

Compaction is relatively simple in fill sections because all the layers are subjected to construction processes and can be compacted during construction. Compaction is more difficult in cut sections. Compaction must be obtained during construction to a depth at which the natural density of the material will resist further consolidation under traffic.

Specific requirements for minimum depth of subgrade compaction for both cohesive and cohesionless soils for road designs are described in Chapter 9 of this FM. This same information for airfield designs is described in Chapter 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2.

SUBGRADE COMPACTION—NORMAL CASES

Cohesionless soils (except silts) can be compacted from the surface with heavy rollers or very heavy vibratory compactors. Cohesive soils [including silts) cannot be compacted in thick layers. In cut areas consisting of cohesive soils, it may be necessary to remove the subgrade material and replace it with sequential lifts capable of being compacted to the required density. As a rule of thumb, initially replace material in 6-inch lifts, then adjust the lift thickness up or down, as necessary, to determine the minimum compactive effort. Compaction of cohesive soil is best achieved with penetrating rollers such as tamping or sheepsfoot.

Compare the subsoil with compaction requirements for the subgrade (as described in Chapter 9 of this FM and Chapter 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2) to determine if consolidation of the subsoil is likely to occur under the design traffic load. If such consolidation is likely to occur, provide a means for compacting the subsoil or design a thicker flexible-pavement structure to prevent subsoil consolidation.

Cohesive materials, including those of relatively low plasticity showing little swell, should be compacted at the optimum moisture content determined from the density-moisture curves developed for that soil using the 55-blows-per-layer compactive effort (CE 55) test. CE 55 may also be designated for ASTM 1557, the American Society of Testing and Materials code for density-moisture curves. Cohesionless, free-draining materials should be compacted at moisture contents approaching saturation.

SUBGRADE COMPACTION—SPECIAL CASES

Although compaction normally increases the strength of soils, some soils lose stability when scarified and rolled. Some soils shrink excessively during dry periods and expand excessively when they absorb moisture. Special treatment is required when these soils are encountered. The following paragraphs describe the soils in which these conditions may occur and suggested methods of treatment.

Soils That Lose Strength

The types of soils that decrease in strength when remolded are generally in the USCS CH, MH, and OH groups. They are soils that have been consolidated to a very high degree, either under an overburdened load by alternate cycles of wetting and drying or by other means, and they have developed a definite structure. They have attained a high strength in the undisturbed state. Scarifying, reworking, and rolling these soils in cut areas may reduce the soil's load-bearing capacity.

When these soils are encountered, obtain CBR values for the soil in both the undisturbed and disturbed conditions. Compact the soil that is removed to the design density at the design moisture content. Other samples should be compacted to the design density across the range of specified moisture contents. If the undisturbed value is higher than the laboratory test results, no compaction should be attempted and construction operations should be conducted to produce the least possible soil disturbance. Since compaction should be avoided in these cases, the total thickness design above the subgrade may be governed by the required depth of compaction rather than the CBR method. (See Chapter 3 of TM 5-825-2.)

Silts

The bearing capacity of silts, very fine sands, and rock flour (predominantly USCS ML and SC groups) is reasonably good if properly compacted within the specified moisture range. Compaction of deposits of silt, very fine sand, and rock flour located in areas with a high water table can pump water to the surface. The material becomes quick or spongy and practically loses all load-bearing capacity. This condition can also develop when silts and poorly draining, very fine sands are compacted at a high moisture content. Compaction reduces the air voids so that the available water fills the void space. Therefore, it is difficult to obtain the desired compaction in these silts and very fine sands at moisture contents greater than optimum.

Water from a wet, spongy silt subgrade often enters the subbase and base during compaction through capillary action. This additional moisture may have detrimental effects on bearing capacity and frost susceptibility. If the source of water can be removed, it is usually not difficult to dry these deposits. These soils usually crumble easily and scarify readily. If the soils can be dried, normal compaction effort should be applied. However, removing the source of the water is often very difficult and, in some cases, impossible in the allotted construction period.

In areas with a high water table, drying is not possible until the water table is sufficiently lowered. Compaction operations will continue to cause water to be pumped to the surface. Areas of this nature are best treated by replacing the soil with subbase and base materials or with a dry soil that is not adversely affected by water.

Do not disturb the subgrade where drainage is not feasible or a high water table cannot be lowered. Also, do not disturb the subgrade in cases where soils become saturated from sources other than high water tables and cannot be dried (as in construction during wet seasons). Compaction of lifts during wet periods can cause fines from the subgrade to contaminate upper layers of the flexible-pavement structure.

The pumping and detrimental actions previously described should be anticipated whenever silts or very-fine-sand subgrades are located in areas with a high water table. Pumping action limits the ability to obtain the desired compaction in the immediate overlying material.

Swelling soils

Soils are characterized as swelling if they display a significant increase in volume with the addition of moisture. These soils can cause trouble in any region where construction is accomplished during a dry season and the soils absorb moisture during a subsequent wet season. If the moisture content of the compacted soil increases after compaction, the soil will swell and produce large, uplift pressures. This action may result in unacceptable differential heaving of flexible pavements. For military construction, swelling soils are placed at moisture contents that will not result in more than a 3-percent change in volume if soil moisture is later increased.

Preswelling is a common method for treating subgrade soils with expansive characteristics. The soil should be compacted at a moisture content at which a 3-percent or greater swelling has already occurred. This reduces the impact of future expansion. Proper control of moisture content is the most important item to remember for swelling soils.

SELECTION OF SUBGRADE AND SUBSOIL DESIGN CBR VALUES

The CBR test described in FM 5-530 includes procedures for conducting tests on samples compacted in test molds (design density and soaked for four days) and for taking in-place CBR tests on undisturbed samples. These tests are used to estimate the CBR that will develop in the prototype structure. Where the design CBR is above 20, the subgrade must also meet the gradation and Atterberg limit requirements for a subbase given in Table 5-1.

SUBGRADE STABILIZATION

Subgrades can be stabilized mechanically (by adding granular materials), chemically (by adding chemical admixtures), or with a stabilization expedient (sand-grid, matting, or geosythetics). Stabilization with chemical admixtures (lime, port-land cement, fly ash, and such) is generally costly but may prove to be economically feasible, depending on the availability of the chemical stabilization agent in comparison with the availability of granular material. Chemically stabilized layers should be designed according to the criteria presented in Chapter 9 of FM 5-410.

Table 5-1. Recommended maximum permissible value of gradation and Atterberg limit requirements in subbases and select materials for roads and airfields

Maximum Permissible Value						
	Maximum design CBR	Size in inches	Gradation requirements % passing No. 10	No. 200	Atterberg Limits LL*	PI*
Subbase	50	2	50	15	25	5
Subbase	40	2	80	15	25	5
Subbase	30	2	100	15	25	5
Select Material	20	3	--	--	35	12

*Determination of these values will be made according to ASTM D4318.

If mechanical stabilization is used and the stabilized material meets the gradation and Atterberg limit requirements in Table 5-1, it can be assigned a subbase CBR rating. If it does not meet the requirements for a subbase, the material must be considered a select material.

A stabilization expedient may provide significant time and cost savings as a substitute to other means of stabilization or low strength fill. The most popular of the man-made stabilizers are sand grid, roll-matting, and various types of geosynthetics, especially geotextiles. Matting and sand grid are expedient methods of stabilizing cohesionless soils such as sand for unsurfaced road construction. Geotextiles and other geosynthetics are primarily used to reinforce weak subgrades, maintain the separation of soil layers, and control drainage through the road or airfield design. The availability of these materials must be weighed with the considerable time savings for use of expedients in combat construction.

FROST SUSCEPTIBILITY OF SUBGRADE

In areas subjected to seasonal freezing and thawing, subgrade materials may exhibit frost heave and thaw weakening. Table 5-2 lists the frost-susceptibility ratings of soils. Those materials with the F3 and F4 classifications are extremely frost-susceptible, especially if the water table is less than 5 feet below the top of the subgrade. Silty soils are particularly susceptible and their CBR value may be less than 1 during thawing periods. The thaw period and resulting degraded soil strength may last from one to four weeks. Emphasis must be placed on reducing traffic loads during this period to help reduce the possibility of damage.

SELECT MATERIALS AND SUBBASE COURSES

When designing flexible pavements, locally available or other inexpensive materials may be used between the subgrade and base course. These layers are designated in this manual as select materials or subbases. Those with design CBR values less than or equal to 20 are called select materials, and those with CBR values greater than 20 are called subbases.

Where the CBR value of the subgrade, without processing, is in the range of 20 to 50,

Table 5-2. Frost-design soil classification

Frost Group	Type of Soil	% By Weight < 0.02 mm	Typical Soil Types Under the USCS
NFS	(a) Gravels ($e \geq 0.25$)	0 - 3	GW, GP
	Crushed stone	0 - 3	GW, GP
	Crushed rock	0 - 3	GW, GP
	(b) Sands ($e \leq 0.30$)	0 - 3	SW, SP
	(c) Sands ($e \geq 0.30$)	3 - 10	SP
S1	(a) Gravels ($e < 0.25$)	0 - 3	GW, GP
	Crushed stone	0 - 3	GW, GP
	Crushed rock	0 - 3	GW, GP
	(b) Gravelly soils	3 - 6	GW, GP, GW-GM, GP-GM, GW-GC, GP-GC
S2	Sandy soils ($e \leq 0.30$)	3 - 6	SW, SP, SW-SM, SP-SM, SW-SC, SP-SC
F1	Gravelly soils	6 - 10	GW-GM, GP-GM, GW-GC, GP-GC
F2	(a) Gravelly soils	10 - 20	GM, GC, GM-GC
	(b) Sands	6 - 15	SM, SC, SW-SM, SP-SM, SW-SC, SP-SC, SM-SC
F3	(a) Gravelly soils	> 20	GM, GC, GM-GC
	(b) Sands, except very fine silty sands	> 15	SM, SC, SM-SC
	(c) Clays (PI > 12)	-	CL, CH, ML-CL
F4	(a) Silts	-	ML, MH, ML-CL
	(b) Very fine sands	> 15	SM, SC, SM-SC
	(c) Clays (PI < 12)	-	CL, ML-CL
	(d) Varved clays and other fine-grained, banded sediments	-	CL or CH layered with ML, MH, SM, SC, SM-SC, or ML-CL

NOTE: e = void ratio.

select materials and subbases may not be needed. However, the subgrade cannot be assigned a design CBR value greater than 20 unless it meets the gradation and plasticity requirements for subbases.

Where subgrade materials meet plasticity requirements but are deficient in grading requirements, it may be possible to treat an existing subgrade by blending in stone, lime rock, sand, or similar materials to produce an acceptable subbase and raise the design CBR value.

MATERIALS

Select Materials

Select materials will normally be locally available, coarse-grained soils (classified G or S), although fine-grained soils in the ML and CL groups may be used in certain cases. Consider lime rock, coral, shell, ashes, cinders, caliche, and disintegrated granite when evaluating sources of select material. To qualify as a select material, a material must meet the gradation and Atterberg limit requirements established in Table 5-1. A **maximum** aggregate size of 3 inches will aid in meeting aggregate gradations.

Subbase Materials

Subbase materials may consist of naturally occurring, coarse-grained soils or blended and processed soils. Lime rock, coral, shell, ashes, cinders, caliche, and disintegrated granite may be used as subbases when they meet the requirements in Table 5-1, page 5-8. The existing subgrade may meet the requirements for a subbase course, or it may be possible to treat the existing subgrade to produce a subbase. Do not admix native or processed materials unless the unmixed subgrade meets the LL and PI requirements for subbases.

A suitable subbase may be formed using material stabilized with commercial admixtures. Portland cement. hydrated lime, fly ash, and bituminous materials are commonly used for this purpose. The plasticity of some materials can be decreased by adding lime or portland cement, enabling them to be used as subbases.

COMPACTION

Select and subbase materials can be processed and compacted using normal compaction procedures. Specify compaction according to the criteria described in Figure 5-4, page 5-5.

SELECTION OF DESIGN CBR

CBR tests are usually conducted on remolded samples. However, where existing similar construction is available, conduct CBR tests on material in place when it has attained its maximum expected water content or on undisturbed, soaked samples. The procedures for selecting test values described in the section on subgrades also applies to select and subbase materials. In order to be used as a select or subbase, the material must comply with the requirements indicated in Table 5-1, including CBR value, gradation, and Atterberg limits. If a material meets the requirements for gradation and Atterberg limits for the next higher design CBR category, but the material's CBR value does not meet the maximum design CBR for that category, assign the material design a CBR value equal to the measured CBR results. For example, a material with a measured CBR value of 37, which meets the gradation and Atterberg limit requirements for a CBR 40 subbase, should be used as a CBR 37 subbase. Conversely, if the material failed to meet the CBR 40 subbase requirements (gradation and Atterberg limits) but met the CBR 30 subbase requirements, it would be used as a CBR 30 subbase rather than a CBR 37.

Some natural materials develop satisfactory CBR values but do not meet the gradation requirements in Table 5-1. These materials may be used as select or subbase materials, as appropriate, if supported by adequate in-place CBR tests on construction projects using the materials that have been in service for several years.

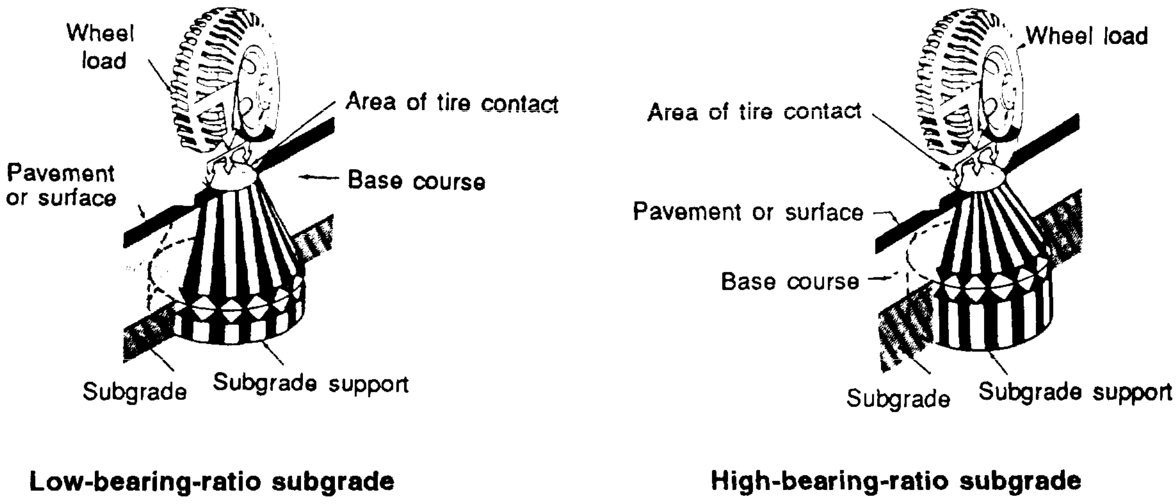
The CBR test is not applicable for use in evaluating materials stabilized with chemical admixtures. These chemically stabilized soils must be assigned an equipment CBR value based on the type of admixture and method of application. (See Chapter 9 of FM 5-410.) Ratings as high as 100 can be assigned to these materials when proper construction procedures are followed.

POTENTIAL FOR FROST ACTION

Select and subbase materials which are subjected to freezing and thawing may exhibit detrimental frost effects. Although these materials generally are not affected by excessive frost heave, they may lose up to 50 percent of their strength during thawing conditions. This is especially true of materials which have more than 20 percent fines (particles passing the Number (No.) 200 sieve). If possible, materials listed in Table 5-2, page 5-9, as NFS, S1, S2, F1, or F2 should be used as subbase and select materials in seasonal frost and permafrost areas.

BASE COURSE

The purpose of a base course is to distribute the induced stresses from the wheel load so that it will not exceed the strength of the underlying soil layers. Figure 5-5 shows the distribution of stress through two base courses. When the subgrade strength is low, the stress must be reduced to a low value and a thick base is needed.



Fjgure 5-5. Distribution of stress in base courses and affects of subgrade strength on basecourse thickness

When the subgrade strength is higher, a thinner base course will provide adequate stress distribution. Because the stresses in the base course are always higher than in the subgrade (Figure 5-5), the base course must have higher strength.

The base course is normally the highest-quality structural material used in a flexible-pavement structure, having CBR values near the CBR standard material (crushed limestone). Base courses are always cohesionless materials and are normally processed to obtain the proper gradation.

REQUIREMENTS

Give careful attention to the selection of materials for base courses. The materials should be dense and uniformly graded so that no differential settlement occurs. For continuous stability, base courses must meet gradation and plasticity requirements.

Gradation

Normally, a material used as a base course must meet the gradation requirements outlined in Tables 5-3, 5-4 and 5-5, page 5-12 (depending on the type of material). Determine gradation of the proposed material using mechanical analysis. If strict adherence to gradation requirements is not feasible, a safe rule of thumb is to avoid using materials which have more than 15 percent fines (particles passing the No. 200 sieve).

Plasticity Index and Liquid Limit

In addition to the gradation requirement, a base-course material must meet the same PI and LL requirements for a subbase material as indicated in Table 5-1, page 5-8. Material with a LL greater than 25 or a PI greater than 5 should not be used as a base course.

Compaction

Base-course material must be capable of being compacted to meet the requirements given in Figure 5-4, page 5-5. When constructing a base course, lift thickness must be based on the ability to attain the required density. Lift thickness is dependent on the type of material, the compaction equipment used, and the method of construction.

The CBR of the finished base course must conform to that used in the design. The total compacted thickness must equal that obtained from the flexible-pavement design curves, Table 5-6, page 5-13 lists nine types of materials commonly used as base

Table 5-3. Generally suitable base course materials

	Rock Type	Use as an Aggregate: Concrete	Use as an Aggregate: Asphalt	Use as a Base Course or Subbase
IGNEOUS	Granite	Fair to Good	Fair to Good**	Good
	Gabbro-Diorite	Excellent	Excellent	Excellent
	Basalt	Excellent	Excellent	Excellent
	Felsite	Poor*	Fair	Fair to Good
SEDIMENTARY	Conglomerate Breccia	Poor	Poor	Poor
	Sandstone	Poor to Fair	Poor to Fair	Fair to Good
	Shale	Poor	Poor	Poor
	Limestone	Fair to Good	Good	Good
	Dolomite	Good	Good	Good
	Chert	Poor*	Poor**	Poor-Fair
METAMORPHIC	Gneiss (nice)	Good	Good	Good
	Schist	Poor to Fair	Poor to Fair	Poor to Fair
	Slate	Poor	Poor	Poor
	Quartzite	Good	Fair to Good**	Fair to Good
	Marble	Fair	Fair	Fair

*Reacts (alkali aggregate)
**Antistripping agents should be used

Table 5-4. Desirable gradation for crushed rock or slag, and uncrushed sand and gravel aggregates for nonmacadam base courses

Sieve Designations	Percent Passing Each Sieve (Square Openings) by Weight — Maximum Aggregate Size: 2 inch	1 1/2 inch	1 inch	1-inch sand clay
2 inch	100			
1 1/2 inch	70-100	100		
1 inch	55-85	75-100	100	100
3/4 inch	50-80	60-90	70-100	
3/8 inch	30-60	45-75	50-80	
No. 4	20-50	30-60	35-65	
No. 10	15-40	20-50	20-50	65-90
No. 40	5-25	10-30	15-30	33-70
No. 200	0-10	5-15	5-15	8-25

Table 5-5. Desirable gradation for crushed rock, gravel, or slag aggregates for macadam base courses

Sieve Designations	Percent Passing Each Sieve (Square Openings) by Weight — Maximum Aggregate Size: 2 inch	1 1/2 inch	1 inch
2 inch	100		
1 1/2 inch	70-100	100	
1 inch	45-80		100
1/2 inch	30-60		
No. 4	30-50	20-40	25-45
No. 10	15-40		
No. 40	5-25		
No. 200	<10	<10	<10

Table 5-6. Assigned CBR ratings for base-course materials

Type	Design CBR
Graded, crushed aggregate	100
Water-bound macadam	100
Dry-bound macadam	100
Bituminous base course, central plant, hot mix	100
Lime rock	80
Bituminous macadam	80
*Stabilized aggregate (mechanically)	80
Soil cement	80
Sand shell or shell	80

*It is recommended that stabilized-aggregate base-course material not be used for tire pressures in excess of 100 psi.

courses for roads and airfields. A typical design CBR is given for each type of material. Laboratory CBR tests to determine design CBR are not necessary.

MATERIALS

Natural, processed, and other materials are used for base courses. Descriptions of these materials follow.

Natural Materials

A wide variety of gravels; sands: gravelly and sandy soils; and other natural materials such as lime rock, coral, shells, and some caliches can be used alone or blended as a suitable base course. Sometimes natural materials require crushing or removal of oversize material to maintain gradation limits. Some natural materials may be suitable for use as a base course by mixing or blending them with other materials.

Sand and Gravel. Many natural deposits of sandy and gravelly materials make satisfactory base-course materials, Gravel deposits differ widely in the relative proportions of coarse- and fine-grained material and in the character of the rock fragments. Satisfactory base materials can often be produced by blending materials from two or more deposits. Uncrushed, clean, washed gravel is normally not suitable for a base course because not enough fines are present. Fines act as a binder and fill the voids between coarser particles.

Sand and Clay. Natural mixtures of sand and clay are often located in alluvial deposits of varying thicknesses. Often there are great variations in the proportions of sand and clay from the top to the bottom of the deposit. Depending on the proportions of sand and clay, these deposits may also provide suitable base-course materials. With proper proportioning and construction methods, satisfactory results can be obtained with sand-clay soils.

Deposits of partly disintegrated rock that consist of fragments of rock, clay, and mica flakes should not be confused with sand-clay soils. The mica flakes make the deposit unsuitable for use as a base course. Mistaking these deposits for a sand-clay soil may result in base-course failure.

Processed Materials

Processed materials are made by crushing and screening rock, gravel, or slag. A properly graded, crushed rock base-course material produced from sound, durable rock makes the highest-quality base material. Existing quarries; ledge rock; cobbles and gravel: talus deposits; coarse mine tailings; and similar hard, durable rock fragments are the sources of processed materials. Table 5-3 shows the common rock types that are generally suitable for base-course material. Generally, rock which is hard enough to require blasting during excavation makes suitable base-course material.

Base courses made from processed materials can be divided into three general types: stabilized, coarse-graded, and macadam.

Stabilized. In a stabilized base course, material ranging from coarse to fine is mixed to meet the gradation requirement given in Table 5-4. The mixing process can be accomplished in advance (at a processing plant) or during the placing operation. Because the aggregates produced in crushing operations or obtained from deposits are often deficient in fines, it may be necessary to blend in selected fines to get a suitable gradation. Screenings, crusher-run fines, or natural clav-free soil may be added for this purpose.

Coarse-Graded. A coarse-graded type of base course is composed of crushed rock, gravel, or slag, When gravel is used. 50 percent of the material by weight must have two or more freshly fractured faces, with the area of each face equal to at least 75 percent of the smallest midsectional area of the piece.

Macadam. The term *macadam* is usually applied to construction in which a coarse, crushed aggregate is placed in a relatively thin layer and rolled into place. Fine aggregate or screenings are placed on the surface of the coarse-aggregate layer and rolled and broomed into the coarse rock until it is thoroughly keyed in place. Water may be Used in the compacting and keying process. When water is used, the base is termed a *water-bound macadam.* The crushed rock used for macadam base courses should consist of clean, angular, durable particles free of clay, organic matter. and other unwanted material or coating, Any hard, durable, crushed aggregate can be used, provided the coarse aggregate is one size and the fine aggregate will key into the coarse aggregate. Aggregates for macadam-type construction should meet the gradation requirements given in Table 5-5, page 5-12.

Other Materials

In some TO areas, deposits of natural sand and gravel and sources of crushed rock are not available. This has led to the development of base courses from materials that normally would not be considered. These include caliche, lime rock, shells, cinders, coral, iron ore, rubble, and other similar materials. Some of these materials arc weak rock that crush or degrade under construction traffice to produce composite base materials similar to those described in the preceding paragraphs. Others develop a cementing action that results in a satisfactory base.

These materials cannot be judged on the basis of the gradation limits used for other materials. Rather, they arc judged on the basis of service behavior. Strength tests on laboratory samples arc not satisfactory because the method of preparing the sample seldom replicates the characteristics of the material in place. The PI is a reasonably good criterion for determining the suitability of these materials as base courses. As a general rule, a low PI (≤ 5) is a necessity. However, observation of these types of base materials in existing roads and pavements is the most reliable indicator of whether or not they will be satisfactory.

Coral. Coral is commonly found along the coastlines of the Pacific Ocean and the Caribbean Sea. Coral is normally very angular and, as such, its greatest assets as a construction material are its bonding properties. These properties vary, based on the amount of volcanic impurities, the proportion of fine and coarse material, and the age and length of exposure to the elements. Proper moisture control, drainage, and compaction arc essential to obtain satisfactory results. Avoid variations of more than 1 percent from optimum moisture content. Uncompacted and poorly drained coral is susceptible to high capillary rise, resulting in too much moisture and loss of stability. Sprinkling with sea water or sodium chloride in solution promotes bonding when rollers are used. As a rule of thumb, coral should cure for a minimum of 72 hours after compaction is completed.

Caliche. Caliche is a by-product of chemical weathering processes. It is composed of limestones, silts, and clays cemented together by lime, iron oxide, or salt. Caliche has been used extensively in arid regions as a base material because of its ability to recement when saturated with water, compacted, and given a setting period. Caliche varies in content (limestone, silt, and clay) and in degree of cementation. It is important that caliche of good, uniform quality be obtained from deposits and that it be compacted within a specified moisture range.

After caliches have been air-dried for 72 hours, the LL of the material passing the No. 40 sieve should not exceed 35, and the PI should not exceed 10. For base-course material, caliches should be crushed to meet the following gradations:

Percent passing 2-inch sieve	100
Percent passing No. 40 sieve	15-35
Percent passing No. 200 sieve	0-20

Stripping should be used to remove undesirable material from surface deposits of caliche.

Tuff. Tuff and other cement-like materials of volcanic origin may be used for base courses. Tuff bases are constructed in the same manner as other base courses except that the oversize pieces are broken and the base is compacted with sheepsfoot rollers after the tuff is dumped and spread. The surface is then graded and final compaction and finishing are accomplished.

Rubble. The debris or rubble of destroyed buildings may be used in constructing base courses. Jagged pieces of metal and similar objects must be removed: large pieces of rubble should be broken into 3-inch pieces or smaller. Caution should be exercised when using rubble in a tactical environment to avoid mines or booby traps.

Bituminous Base. In general, a bituminous base course may be considered equal, on an inch-for-inch basis, to other types of high-quality base courses. Bituminous mixtures are frequently used as base courses beneath high-use bituminous pavements, particularly for rear-area airfields carrying heavy traffic, Bituminous bases may be advantageous when locally available aggregates are relatively weak and of poor quality, when mixing-plant and bituminous materials are readily available, or when a relatively thick structure is required for the traffic.

When a bituminous base course is used, it is placed in lifts no more than 3 1/2 inches thick. If a bituminous base is used, the binder and leveling courses may be omitted and the surface course may be laid directly on the base course.

SELECTION OF BASE COURSE

Selection of the type of base-course construction depends on the materials and equipment available and the anticipated weather conditions during construction. A complete investigation should be made to determine the location and characteristics of all natural materials suitable for base-course construction. Base courses of untreated natural materials are less affected by adverse weather and normally require less technical control. Untreated bases are relatively easy and fast to build and are preferable to bituminous or cement-stabilized types. This is true even where suitable admixture materials for such construction are readily available, which is not true in many areas of the world.

SPECIAL CONSIDERATIONS FOR SEASONAL FROST AND PERMAFROST CONDITIONS

Since base-course materials are near the surface of the road or airfield, the amount of strength loss during thawing periods will have a strong influence on the life of the facility. If possible, materials listed in Table 5-2, page 5-9, as nonfrost susceptible (NFS), or possibly frost susceptible (S1 or S2) should be used as base courses in seasonal frost and permafrost areas.

CONSTRUCTION OPERATIONS

Construction operations for roads and airfields include the following tasks which are organized according to the construction schedule and quality control plan for the project.

Fine Grading

The subgrade is fine graded to achieve the desired cross section established by final grade stakes. Before placing select material, subbase, and base course, the subgrade should be compacted to attain the required density, and ruts and other soft spots should be corrected.

Hauling, Placing, and Spreading

Placing and spreading material on the prepared subgrade may begin at the point nearest the borrow source or at the point farthest from the source. The advantage of working from the point nearest the source is that the haul vehicles can be routed over the spread material, which compacts the base and avoids damage to the subgrade. An advantage of working from the point farthest from the source is that hauling equipment will further compact the subgrade. Also, this practice will not overwork the base course, which can cause unwanted segregation. This method also reveals any weak spots in the subgrade so that they can be corrected prior to placement of the base courses, and interferes less with spreading and compaction equipment.

The self-propelled aggregate spreader is the preferred piece of equipment for placing a base course. If a self-propelled spreader is not available, base-course material can be spread using towed spreaders, scrapers, or dump trucks. If equipment capable of spreading the aggregate in even lifts is not available, the material can be initially dumped in long windrows and subsequently spread with graders, dozers, or front-end loaders.

Lift thickness should be based on the ability to compact the material to the required density. A good rule of thumb is to initially place the base course in 6-inch lifts. After testing the compacted density, increase or decrease the lift thickness as necessary to meet the project requirements.

Blending and Mixing

Materials to be blended and mixed should be spread on the road, runway, or taxiway in correct proportions, with the finer material on top. Fold the fine material into the coarser aggregate with the grader blade. If available, dry-mix the material using blades, disks, harrows, or rototillers, leaving the material in windrows. When a grader is used, thoroughly mix the materials by blading the windrows of materials from one side of the area to the other, with the blade of the grader set to give a rolling action to the material. The coarse and fine aggregates can also be mixed in mechanical plants (mobile or stationary) or on a paved area with graders and bucket loaders. Proportionally distribute the coarse and fine aggregates by weight or volume in quantities so that the specified gradation, LL, and PI requirements are attained after the base has been placed and compacted. Mixing operations should produce uniform blending.

When mechanical mixing is used, place the coarse and fine aggregates in separate stockpiles or adjacent windrows to permit easy proportioning. When bucket loaders are used, place the fine- and coarse-aggregate portions in adjacent windrows on a paved area. Blade the windrows together to meet the requirements specified for the project.

Watering Base Materials

As in subgrade-compaction operations, obtaining the specified compacted density requires that the material be placed and compacted at a moisture content inside the specification block. The moisture content of the base material at the site can be obtained by a nucIear densometer, a speedy moisture tester, or by expedient methods. Given the on-site moisture content, the engineer in charge can calculate exactly how much water is to be added or if the base needs to be aerated to achieve the specified moisture content range.

Controlled watering can be done with a truck-mounted waler distributor. Asphalt distributors should not be used because the pump lubrication system is not designed for water. Any container capable of movement and gravity discharge of water may be used as an expedient water distributor.

Compacting

Base-course compacting must produce a uniformly dense layer that conforms to the specification block. Compact base-course material with vibratory or heavy.

rubber-tired rollers. Maintain moisture content during the compaction procedure within the specified moisture-content range. Compact each Iayer through the full depth to the required density. Measure field densities on the total sample. Use a *test strip* to determine which rollers are most effective and how many roller passes are necessary to achieve the desired compaction. The care and judgment used when constructing the base course will directly reflect on the quality of the finished flexible pavement. Base-course layers that contain gravel and soil-binder material may be compacted initially with a sheepsfoot roller and rubber -tired rollers. Rubber-tired rollers are particularly effective in compacting base materials if a kneading motion is required to adjust and pack the particles. Base courses of crushed rock, lime rock, and shell are compacted with vibratory, steel-wheeled, or rubber -tired rollers. Select the equipment and methods on each job to suit the characteristics of the base material. When using rollers, begin compaction on the outside edges and work inward, overlapping passes by one-half of a roller width.

Finishing

Finishing operations must closely follow compaction to furnish a crowned, light, water-shedding surface free of ruts and depressions that would inhibit runoff. Use the grader for finishing compacted aggregate bases. Blade the material from one side of the runway, taxiway, or road to the middle and back to the edge until the required lines and grades are obtained. Before final rolling, the bladed material must be within the specified moisture-content range so it will consolidate with the underlying material to form a dense, unyielding mass. If this is not done, thin layers of the material will not be bound to the base, and peeling and scabbing may result. Final rolling is done with rubber-tired and steel-wheeled rollers.

SPECIAL PROCEDURE FOR MACADAM BASES

Construction of macadam base courses requires the procedures that follow.

Preparing Subgrade

If a macadam base course is constructed on a material with high plasticity, there may be base infiltration. This can be prevented with a blanket course of fine material such as crusher screenings or 3 to 4 inches of sand. The blanket course should be lightly moistened and rolled to a smooth surface before spreading the coarse macadam aggregate. A membrane or a geotextile fabric may be used in lieu of the blanket course.

Spreading

Macadam aggregate must be placed and spread carefully to ensure that hauling vehicles do not add objectionable material to the aggregate. Care is particularly necessary when placing the aggregate at the point nearest to the source and routing hauling vehicles over the spread material. If the compacted thickness of the lift is 4 inches or less, spread the loose macadam aggregate in a uniform layer of sufficient depth to meet requirements. For greater compacted thickness, apply the aggregate successively in two or more layers. Spreading should be from dump boards, towed aggregate spreaders, or moving vehicles that distribute the material in a uniform layer. When more than one layer is required, construction procedures are identical for all layers.

Compacting

Immediately following spreading, compact the coarse aggregate the full width of the strip by rolling it with a steel-wheeled roller. Rolling should progress gradually from the sides to the middle of each strip in a crown section, and from the low side to the high side where there is a transverse slope across the road, runway, or taxiway. Continue rolling until the absence of creep or wave movement of the aggregate ahead of the roller indicates that the aggregate is stable. Do not attempt rolling when the subgrade is softened by rain.

Applying Screenings

After the coarse aggregate has been thoroughly stabilized and set by rolling, distribute sufficient screenings (fine aggregates) to fill the voids in the surface. Roll continuously while screenings are being spread, so the jarring effect of the roller will cause them to settle into surface voids. Spread screenings in thin layers by using hand shovels, mechanical spreaders, or moving trucks. Do not dump them in piles on the coarse aggregate. If necessary use hand or drag brooms to distribute screenings during rolling.

Do not apply screenings 100 thick because they will bridge over the voids and prevent the direct bearing of the roller on the coarse aggregate. Continue spreading, sweeping, and rolling until no more screenings can be forced into the voids. Start sprinkling the surface with water after the screenings have been spread. The sprinkling causes the screenings to be flushed down into the voids of the aggregate. The surface is then rolled. Do not saturate and soften the subgrade.

Continue sprinkling and rolling until a mixture of screenings and water forms, fills all voids, and gathers in a small wave before each roller. When a section of a strip has been grouted thoroughly, allow it to dry completely before performing additional work.

FINISHED SURFACES

The base-course surface determines the smoothness of the finished pavement. If the finished base dots not conform to the specified grade when tested with a 12-foot straightedge, the finished pavement also will not conform. The base surface should be smooth and conform to specified design requirements.

When tested with a 12-foot straightedge applied parallel and perpendicular to the centerline of the paved area, the surface of the base course should not show any deviation in excess of 3/4 inch for roads and airfields [for propeller-type aircraft) or 1/8 inch for jet aircraft. Correct any deviation in excess of these figures, and remove material to the total depth of the lift, replacing with new material and compacting as specified above.

SLUSH ROLLING

The purpose of slush rolling (rolling with enough water to produce a slushy surface) is to achieve compaction when conventional methods fail. Slush rolling should be permitted only on a free-draining, cured base course. Slushing requires a considerable amount of water on the surface. The quantity varies greatly with the type of material, the temperature, and the humidity. If the surface is generally satisfactory but has some large areas requiring slushing, slush only the rough areas. Slushing brings fines to the top and creates voids. In general, slush rolling should not be used on a high-quality base-course material. It should be used only when required by the specifications or when conventional compaction methods have failed.

Applying Water

Engineers must calculate a water application rate in terms of gallons per square yard in order to allow the water distributor operator to accurately apply water. A reasonable estimate for applying water to a 6-inch lift is 0.5 to 1.0 gallon per square yard. The rollers must follow immediately behind the water truck to achieve the desired results because the roller should carry a wave of water ahead of it as it passes over the base course.

Rolling Equipment

Use pneumatic-tired, vibratory, or steel-wheeled rollers to obtain a smooth finish on the base course. Continue rolling until compaction has been obtained.

Finishing

There are usually small rivulets or ridges of fines left on the surface after slushing is completed. Where these are excessive or when the thickness of the blanket of fines is excessive, sprinkle the surface with water and hone (dress lightly) with a grader blade. This delicate operation requires a good operator and a sharp, true blade. Follow the grader immediately with a pneumatic roller to reset the surface.

WET ROLLING

All base courses require a final surface finish. The final finish should be obtained immediately after final compaction or proof rolling. For less critical base courses or where deemed necessary by the project engineer, wet rolling and slush rolling may be used to obtain the final finish. Both methods have strong points and, in some cases, a job may require a combination of the two.

Applying Water

Wet rolling does not require the large amount of water demanded for slush rolling, and the base course does not need to go through the curing period required by the slush-rolling method. Apply enough water to the base course to raise the moisture content of the upper 1 to 2 inches of the base course to approximately 2 percentage points above the minimum moisture content. The percent of moisture will vary with the type of material and is a matter of judgement by the project's quality control manager.

Finishing

Finish the surface by having the grader blade lightly cut the final surface. The light blading will loosen the fines; the coarser particles of the base course will be carried along by the blade to form a windrow at the edge of the section being finished. This coarse aggregate can be evenly distributed over the area and incorporated into the surface of the base by a steel-wheeled roller closely following the grader. Additional water may be required, and rolling by the steel-wheeled and pneumatic-tired rollers must be continued until a smooth, dense surface is obtained. This method can also be used for correcting minor surface irregularities in the base course.

QUALITY CONTROL

Quality control is essential to any project's success. Although visual inspection is important, it is not, by itself, sufficient to control the construction of all courses described in this chapter, particularly those which contain considerable fine material. Depending on the type of base, control tests will include determinations of gradation, mixing proportions, plasticity characteristics, moisture content, field density, lift thickness, and CBR values. These tests are described in detail in FM 5-530. Prior to starting construction, a detailed quality control plan should be developed which addresses testing procedures, frequency, location and, most importantly, remedial actions.

DRAINAGE

CHAPTER 6

Inadequate drainage is the most common cause of road and airfield failure. Therefore, drainage is a vital consideration in planning, designing, and building military roads and airfields. It is important during both construction and use.

SECTION I. CONSTRUCTION DRAINAGE

Commanders and construction supervisors must ensure continuous maintenance of the drainage system during construction of a military road or airfield. The construction drainage system is temporarily established to prevent construction delays and structural failure before completion. Generally, long delays will result if drainage is not continuously emphasized by the command. Construction drainage must be completed before needed; when a storm begins it is too late to start drainage work. Construction-drainage measures used during different phases of construction are discussed below.

PRELIMINARY MEASURES

RECONNAISSANCE

Prior to the start of construction, preliminary reconnaissance of an area should disclose features that require advance drainage planning and operations. These features include—

- Springs and seepage on hillsides which may indicate perched or high water tables detrimental to cuts.
- Trees adjacent to dry or low-flow streams that could receive their root water from a groundwater table flowing near the surface. Compacted fills across such areas could change the movement of the flow.
- Vegetation or cover that, if removed during the clearing and grubbing phase of construction, could increase surface runoff.
- The presence of level areas which have good vegetation and adjacent slopes. These areas may indicate a shallow groundwater table with capillary water movement and may require intercepting subsurface drainage.
- Streams that should be checked for normal high-water and flood indicators.

PROTECTIVE MEASURES

Controlling runoff during construction can be costly, The following measures can help maintain satisfactory drainage during construction:

- Make maximum use of existing ditches and drainage features. Where possible, grade down hill to allow economical grading and to take advantage of natural drainage

- Use temporary ditches to help construction drainage. Ensure efforts are made to drain pavement subgrade excavations and base courses to prevent detrimental saturation. Carefully consider the drainage of all construction roads, equipment areas, borrow pits, and waste areas.

- Be aware of areas where open excavation can lead to excessive erosion. The discharge of turbid water to local streams will require temporary retention structures.

- Hold random excavation to a minimum, and sod or seed finished surfaces immediately.

- Plan timely installation of final storm-drain facilities and backfilling operations to allow maximum use during construction.

AREA CLEARING

The following constraints should be considered:

Existing Drainage

Clear excess vegetation from streams. This increases the velocity and quantity of flow. Widening the stream can also increase the flow. Bends and meanders can be cut to straighten the stream. Use care in making major alignment changes because they can change the hydraulic characteristics of the stream. This change could adversely affect other parts of the stream.

Vegetation Removal

Military projects may require the removal of all vegetation from large areas. Consider the following factors with regard to construction stripping:

- Select a disposal area that will not interfere with or divert the drainage pattern of surface runoff. If the drainage pattern is disturbed, the stripped material may form a barrier resulting in pending and may otherwise affect adjacent areas.

- Be aware that removing the vegetation from an area can lead to excessive surface runoff and erosion. This could lead, in turn, to silting of channels and flooding of low areas.

- Consider that serious bank erosion due to surface runoff may occur if vegetation adjacent to the banks of streams or ditches is removed. To avoid this, it may be necessary to leave the vegetation or to provide a berm with a chute.

- In large, cleared areas, control runoff sediments to prevent failure of other structures and possible adverse environmental effects. This is more of a contern in permanent construction than in wartime TO construction.

Clearing, Grubbing, and Stripping

Any clearing, grubbing, and stripping must include filling holes and back dragging or grading to a slight slope. This will ensure proper runoff and prevent water from collecting and saturating the subgrade. If filling and grading is not done, the advent of rain will make it necessary to strip off any wet soil until dry soil is reached to start the fill. Some ditching may be required to direct the surface flow to an outlet point.

Fill

When placing fill, exercise firm control over the project to prevent adverse effects from improper drainage procedures. Some of the factors requiring attention are—

- The fill section must be rolled smooth at the end of each working day to seal the surface. No areas should be left that can hold standing water.

- The fill surface must be kept free of ruts caused by trucks and other equipment. These depressions collect rain and saturate the subgrade. Also, the surface must be crowned to discharge runoff quickly.

- When the fill area is large, it may be necessary to create swales (depressed areas) to conduct surface runoff to discharge outlets.

- To allow fill to proceed, it may be necessary to install temporary culverts in the fill area in places other than final design locations. After the fill area has reached design depth, the design culverts can be properly trenched in place.

Ditching

Use interception ditches during construction to collect and divert surface runoff before building the designed system. Prior to construction, conduct a site investigation of the general layout, consistent with the work plan. When interception ditches cannot be made part of the design drainage. consider removing the ditches by backfilling and compacting.

Locate interceptor ditches on hillsides and at the foot of slopes to intercept and divert runoff from the construction site. Make these ditches part of the final drainage system wherever possible. Roadside ditches, required during all construction stages, should be placed at design locations.

During construction, use deep ditches for subsurface drainage. They intercept groundwater flow, as shown in Figure 6-1. If groundwater flow must be intercepted but ditching is not possible, modify the ditch into a subsurface drainpipe system.

Ditching may be required in swamp areas to either continue drainage ditches to an outlet point or drain the area.

Engineers may use explosives in such cases, since the soil may not be capable of supporting construction equipment. Draglines should also be considered.

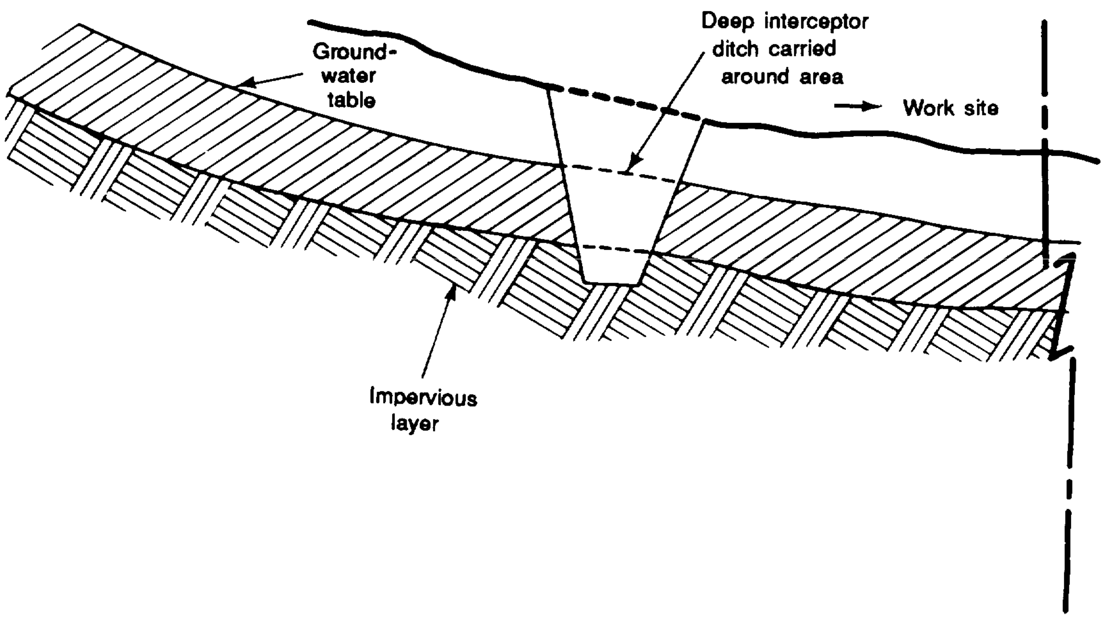


Figure 6-1. Deep interceptor ditch

Timber or steel mats can be used to provide a firm foundation and support equipment during the operation.

Culverts

Culverts are required during construction to allow surface runoff. If streams must be diverted to allow the construction of permanent culverts, use temporary culverts in the construction area. Never close natural drainage channels, even if they are currently dry. If these channels are closed, surface runoff from sudden storms could cause a serious problem. These conditions must be anticipated. Construction drainage must keep pace with the construction project.

DRAINAGE HYDROLOGY

The hydrologic cycle is the continuous process which carries water from the ocean to the atmosphere, to the land, and back to the sea. A number of different subcycles can take place concurrently in the overall cycle and are discussed below.

PRECIPITATION

Rainfall is the moisture-delivery mechanism of primary concern to most military drainage designers. Snowmelt may be of greater concern in colder climates or in the design of reservoirs in milder regions. These concerns are beyond the scope of this manual, but they are included in TM 5-852-7. The amount of rainfall that evaporates depends on the surface temperature of ground features, the air temperature, the wind speed, and the relative humidity. Evaporation occurs while rain is falling to the ground and after it lands on vegetation and other ground cover.

INTERCEPTION

Rainfall coming to rest on vegetation is said to have been intercepted. Large quantities of water can be trapped in the leaf canopy of trees and plants. Rain does not reach the soil until the holding capacity of the vegetation canopy is exceeded.

INFILTRATION

A significant portion of the water that actually strikes the soil soaks into the ground. This process is call infiltration. The rate of absorption and the quantity of water absorbed depends on the soil type, the vegetation, the terrain slope, and the soil moistness prior to the rain. Storm-water runoff begins to accumulate only when the rate of rainfall exceeds the rate of infiltration.

DETENTION

Before overland water flow can begin its downhill motion, it must be deep enough to overcome any obstacles to its movement. Detention is the amount of water required to fill depressions of any size in the earth's surface. Except by infiltration or evaporation, no water can leave a depression until the holding capacity of the depression has been exceeded.

TRANSPIRATION

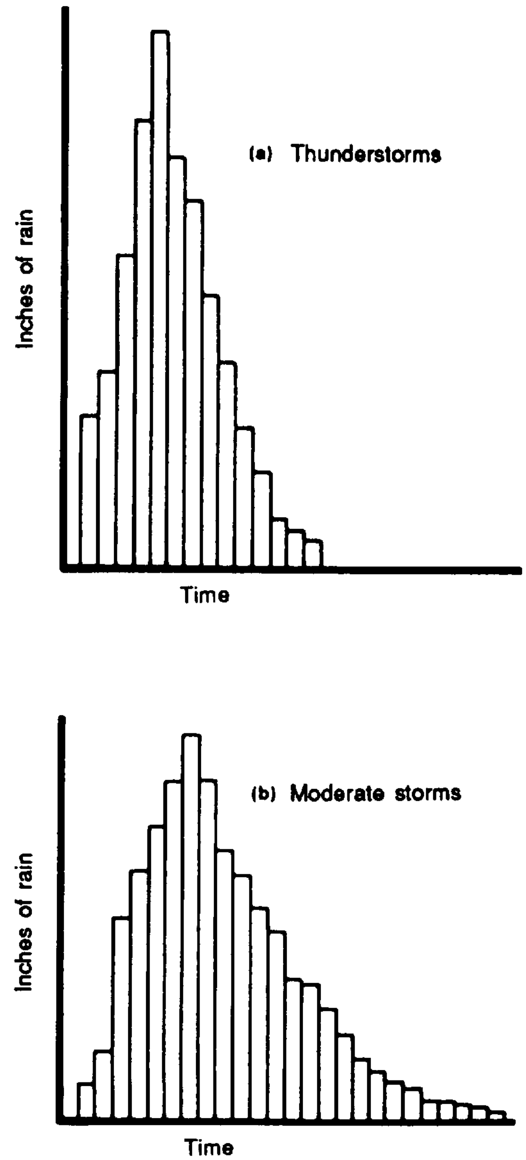
On a long-term basis, vegetation returns water to the atmosphere through a process called transpiration. Because of the time involved, transpiration has no immediate effect on water runoff in an area.

RUNOFF

Evaporation, interception, infiltration, detention, and transpiration are all moisture losses, Runoff is precipitation minus these moisture losses.

STORMS

Storms can deliver a large quantity of water to the earth in a short period of time. For



that reason, the study of storms is an important part of the study of drainage hydrology. This section discusses storms in terms of duration, frequency, and intensity. It describes procedures for determining maximum storms and introduces the subject of runoff.

Duration

Duration is the length of time a storm lasts, After many years of observation, hydrologists have determined that a storm of long duration usually has low intensity. In contrast. a high-intensity storm usually has a short duration. Figure 6-2 shows typical storm hydrography developed by the National Weather Service. Time, usually measured in hours, is depicted horizontally. The amount of rain for each unit of time is measured vertically in inches. The total amount of precipitation is the area of the graph. The five main types of storms are described below.

Thunderstorms

Thunderstorms, represented by Figure 6-2(a), are local atmospheric disturbances of short duration and high average rate of rainfall (intensity). They are characterized by thunder, lightning, torrential rain, and sometimes hail, Thunderstorms tend to govern the design of drainage for small areas.

Moderate Storms

Moderate storms, represented by Figure 6-2(b), cover larger areas for several hours with moderate intensity. These storms

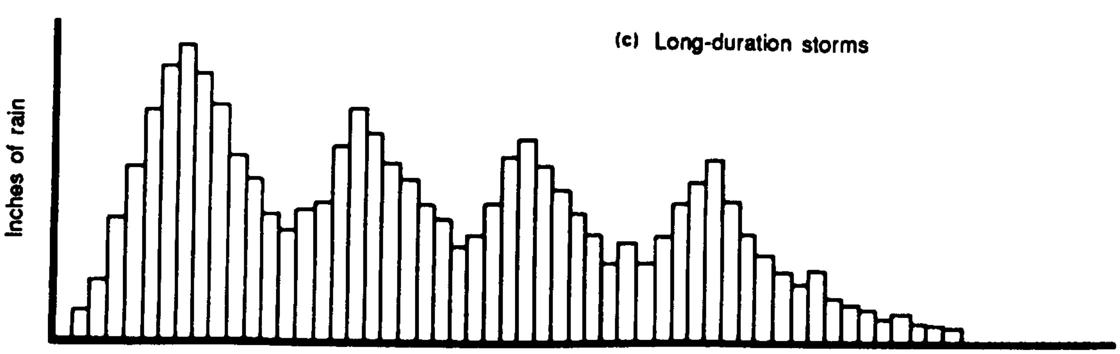


Figure 6-2. Typical storm hyetographs

develop greater total precipitation than thunderstorms. The moderate storm normally controls the design of drainage structures for medium-sized basins.

Long-Duration Storms

Long-duration storms, represented by Figure 6-2(c), page 6-5, often have several peaks of high rainfall. Durations may be up to several days, developing very large amounts of precipitation at relatively low average rates of rain fall. With a low average rate of rainfall, such storms have little or no impact on small- or medium-sized drainage basins, but they normally control the design of drainage structures for large basins.

Monsoons

Monsoons arc seasonal winds of the Indian Ocean and southern Asia. These winds blow from the south during April to October and from the north during the rest of the year. Heavy rains usually characterize the April-to-October season. This rain is not normally continuous; it rises to a peak and then subsides in a cyclic fashion.

Tropical Cyclones

Hurricanes and typhoons are storms caused by severe cyclonic disturbances over a wide area. Precipitation is normally heavy and long.

Design Life Versus Actual Life of a Structure

The design storm is an idealized storm that is expected 10 be equalled or exceeded at least once during the design life of a drainage system. For example, if a drainage system has been designed for an estimated life of five years, then the design storm will have a five-year frequency. The frequency of a design storm is the average return period of a storm. For example, if a two-year frequency storm has an intensity of 1.5 inches of rainfall per hour, it can be expected that a storm of that intensity or greater will recur an average of once every two years. Two years is also called the return period. The reciprocal of the return period is the probability of having a storm of that value or greater in any one year.

For the two-year -frequency storm, the probability of having a storm equalling or exceeding the value in any one year is 0.5 (two out of four times).

The design-storm frequency for TO construction is normally two years. If construction with a longer estimated life is desired, **the appropriate design storm should be specified in the authorizing directive.**

As with any statistical method of describing essentially random, natural events such as weather, there is a degree of uncertainty. The two-year design storm occurs on the average every two years; it is not guaranteed to occur every two years. Statistically, the probability of a storm equal to or greater than the two-year design storm occurring in any two years is 0.75 (three out of four times). Details of the statistics involved can be found in hydrology textbooks.

WEATHER DATA

If there are extensive rainfall and rain-rate records for the location of interest, and if hydrologists have examined those records statistically to formulate intensity-duration tables, then those tables can be obtained through the Air Force staff weather officer. The staff weather officer is normally located at division level.

Within the United States, the data will generally come from the National Weather Service, either directly or through the Air Force. Overseas, the staff weather officer may be able to obtain data from local government sources, but it may take considerable time to obtain. However, it is unlikely that such pinpoint data is available in many overseas TO locations. When weather data is not available, use rainfall isohyetal maps. Isohyetal maps have contours of equal rainfall intensity just as topographic maps have contours of equal elevation. Figure 6-3 is an isohyetal map of the world, in this case showing the iso-intensity lines for a 60-minute, 2-year storm.

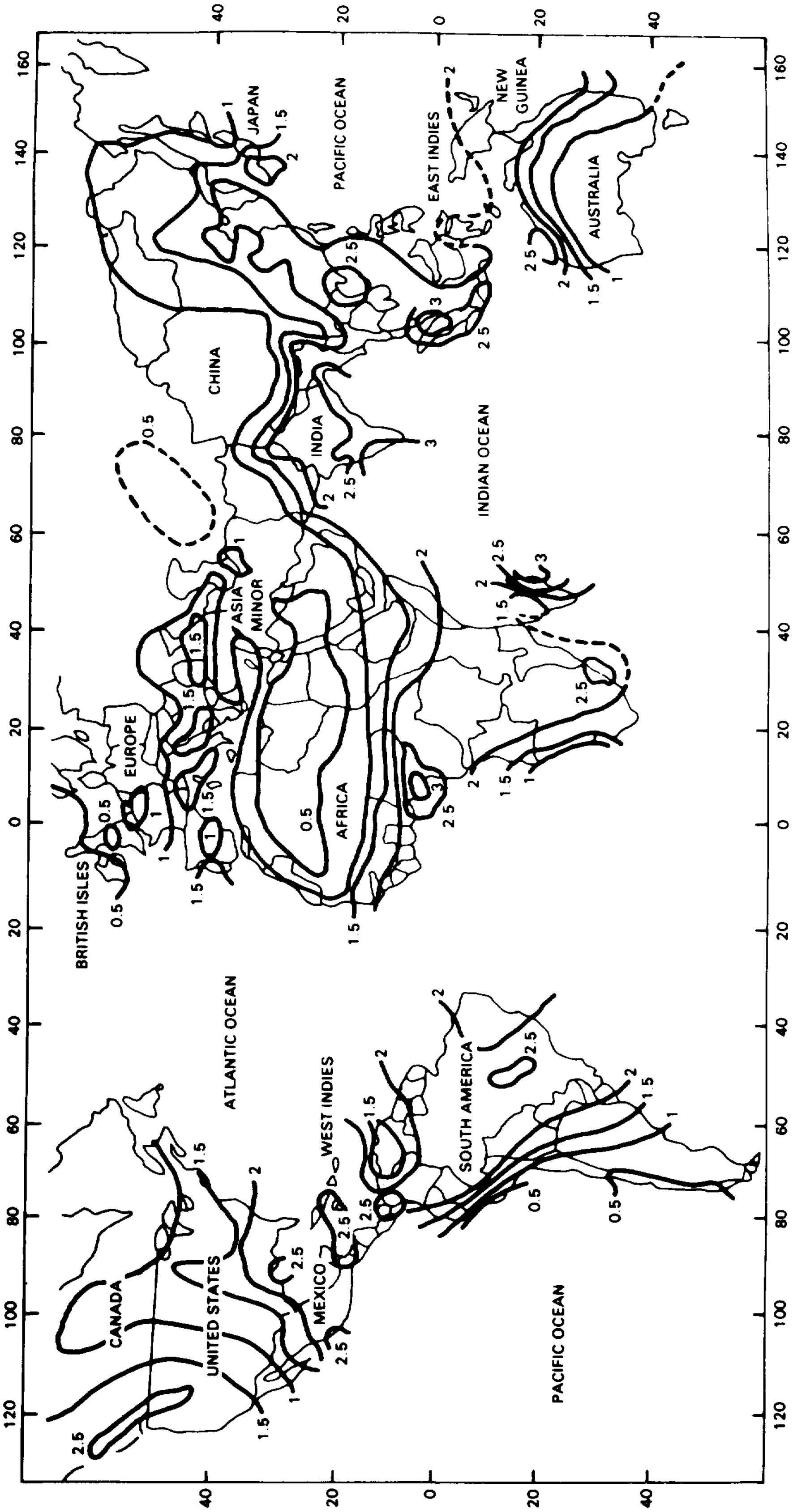


Figure 6-3. World isohyetal map

To properly read a value on the isohyetal map, find the project location and read the value of the appropriate isohyet(s).

Do not interpolate. If a project location falls—

- On an isohyetal line, read the value of that isohyet.
- Between two isohyets, read the larger value.
- Within an encircling isohyetal line, read the value of the encircling isohyet.

Examples (in inches per hour (in/hr)):

Southern Australia	1.0
North Dakota	1.5
Florida	2.5
Washington, DC	1.5
Vietnam	2.5
Cuba	2.5
New Orleans, Louisiana	2.5

Note that the intensities just found are for a 60-minute storm. This must now be adjusted to the critical duration of the project under construction. Once the critical duration has been determined, make the adjustment using the standard rainfall intensity-duration curves in Figure 6-4.

The standard curves are numbered 1.0, 2.0, 3.0, and 4.0, with intermediate values readily interpolated. Note that curve number 1 passes through 1 inch per hour at 60 minutes, curve number 2 passes through 2 inches per hour at 60 minutes, and so on.

Where intensity is known for any nonarctic location (taken from the isohyetal map, Figure 6-3, page 6-7) and critical duration is calculated, the intensity (1) can easily be determined. (The standard intensity-duration curves are applicable to any frequency, not just a 2-year frequency.)

To use Figure 6-4, enter the graph using the Duration in Minutes (Tc). Follow the line vertically until it intersects the curve whose number corresponds to the 60-minute intensity determined from the isohyetal map (or from pinpoint data, if YOU choose not to draw your own intensity-duration curve). Read horizontally to the left to determine the rainfall intensity (I) in inches per hour. The following is an example:

I_{60}, 2-yr Intensity (in/hr)	Critical Duration (min)	I_{adj} (in/hr)
1.0	50	1.2
1.5	30	2.4
2.0	10	5.2

RUNOFF

Precipitation supplies water to the surface, but evaporation, interception, and infiltration begin to draw water at the start of the storm. Eventually, if the storm is strong enough, vegetation and other surface characteristics, such as depressions and soil, will become saturated, allowing water to flow freely over the surface. This condition is called runoff and is usually measured in cubic feet per second (cfs). Runoff begins sometime after the beginning of precipitation and may continue long after precipitation ends.

The total quantity of runoff from a given area, after it is collected in channels and streams, is the flow estimate used to design an area's drainage structures.

Transpiration and evaporation also draw from the water supplied by precipitation. However, these are relatively small losses and will not usually affect military drainage design. Estimating runoff will be discussed later in this chapter. Once the runoff has been determined, necessary ditches and culverts can be designed.

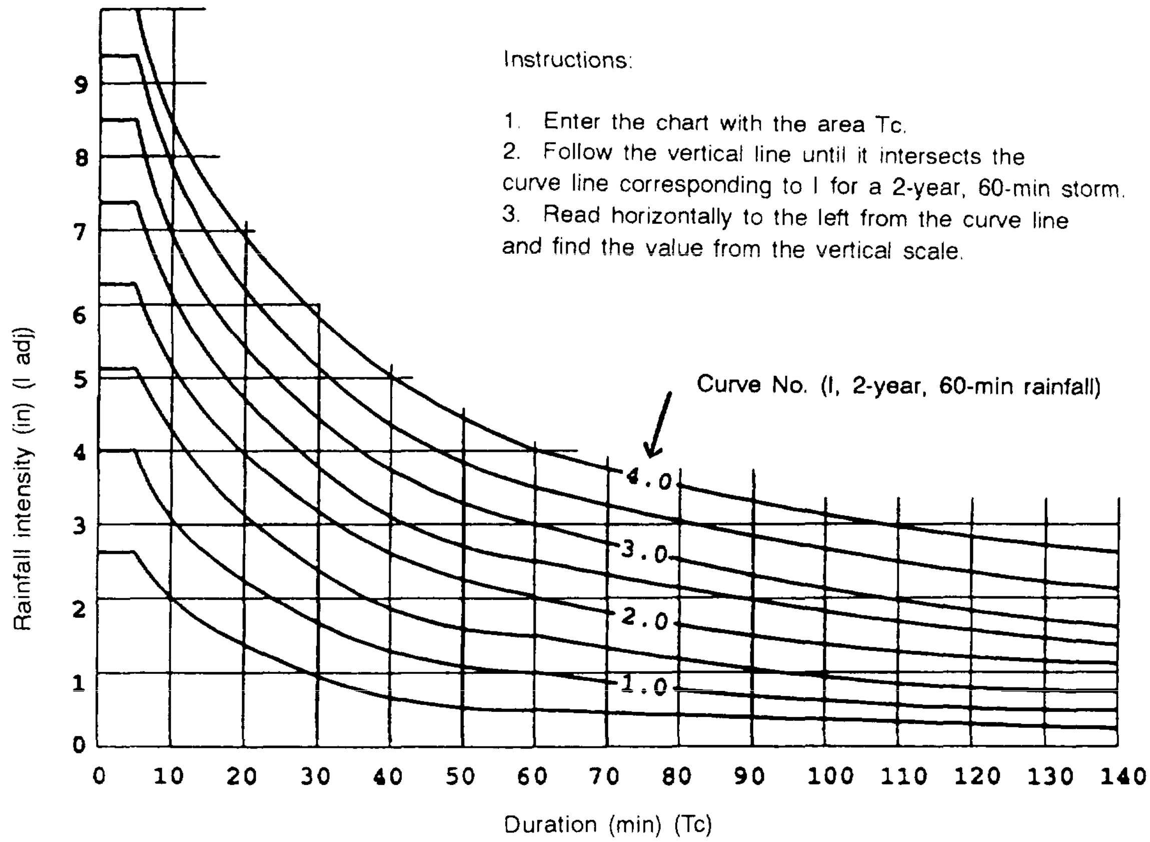


Figure 6-4. Standard rainfall intensity-duration curves

THE HYDROGRAPH

Stream-water flow may originate from surface runoff, groundwater, or both. Runoff reaches the stream as overland flow. Groundwater flow results from side-bank seepage and springs. The hydrography depicts the fluctuations of flow with regard to time.

The elements of a hydrograph are base flow, lag time, peak flow, lime of concentration (TOC), and flow volume. Each stream will have its own characteristic hydrography with widely varying values for the elements. A typical stream-flow hydrography is shown in Figure 6-5, page 6-10.

BASE FLOW

The base flow of a stream depends upon the amount of groundwater that seeps into the stream and its tributaries from their banks and the flow from permanent springs and swamps. Depending on the area, climate, and groundwater level, it may flow at a fairly constant rate. Conversely, the flow may fluctuate widely or even cease completely for some periods of time.

LAG TIMES

When precipitation begins over an area, there is an initial period during which the loss factors induced by interception, infiltration, and detention take effect before any surface runoff takes place. Stream flow will increase only when these initial losses have been satisfied and surface runoff begins. This is known as initial lag time. The length of this lag time is influenced by

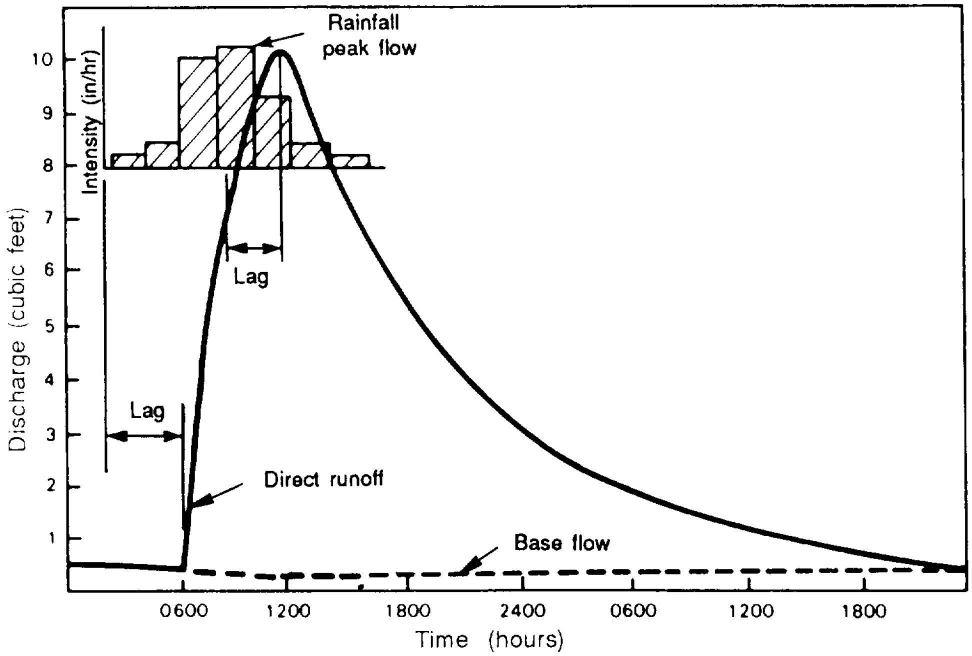


Figure 6-5. Typical hydrograph

vegetation and other terrain characteristics. For example, a grass-covered parking area will have a longer initial lag period than an asphalt parking lot of the same size.

A second lag time occurs between the time the stern] reaches its peak pecipitation rate and the time the stream reaches its maximum flow. The length of this secondary lag time is influenced by the size of the area drained. In small- and moderately - sized drainage areas, there will be only slight differences between storm peak and stream peak.

PEAK FLOW

The peak of the hydrography is the maximum stream flow that will occur during a particular storm. In general, peak flow is generated when the entire drained area is discharging its runoff. Peak flow is read directly from the maximum ordinate of the hydrograph This flow determines the size of the drainage structures required at the basin outlet.

TIME OF CONCENTRATION

The TOC is the time it lakes for an entire drainage basin to begin contributing runoff to the stream. Assuming uniform rainfall, the hydrography peaks at that point. TOC is critical to the drainage engineer, since it determines the duration of the storm that will demand the most from the drainage system; that is, the storm's critical duration.

VOLUME OF FLOW

The area under the curve of the hydrography indicates the total flow, in cubic feet, resulting from any particular storm. It is used in drainage design for determining pending times when it is practical to use culverts with submerged inlets.

CONSTRUCTING A HYDROGRAPHY

A hydrograph is constructed by measuring a stream's rise and fall and the times related to these changes in flow. When constructing a hydrograph—

- The base flow must be measured at a time when there have been no recent storms. A field reconnaissance must be made for this measurement.
- The peak flow can be estimated using the hasty runoff estimation presented in this chapter.
- The general shape of the curve of the hydrograph will be similar to that shown in Figure 6-5.

DRAINAGE-SYSTEM DESIGN

DESIGN DATA REQUIREMENTS

Before designing a drainage system, survey the various types and sources of drainage-related information, The survey should include, as a minimum, information concerning the area's topography, meteorological records, soil characteristics, and available construction resources.

TOPOGRAPHICAL INFORMATION

Give special attention to the vicinity of the proposed facility as well as the presence of any topographical features that may contribute runoff to the project area. The completed facility often will interfere with the site's natural drainage. Therefore, when analyzing the effects of the surrounding terrain—

- Identify all areas that contribute runoff to the site.
- Determine the general size and shape of these contributing areas.
- Determine the natural direction of surface-water flow, the slope of the land, and the type and extent of natural ground cover.
- Locate natural channels that can be used to move runoff within the project area or to divert it away from the work site.

METEOROLOGICAL DATA

Gather information on general climatic conditions, seasonal variations in rainstorms, and intensity and duration of representative storms. This data can then be applied to the location of the proposed facility.

SOIL CHARACTERISTICS

Obtain soil data from soil and geological maps, aerial photographs, or site tests performed by soil analysts. Soil data deals with the horizontal and vertical extent of soil types, the elevation of the groundwater table, and the drainage characteristics of the soil. The most important drainage characteristic of a soil is its permeability. Permeability limits the rate at which the rainfall infiltrates the ground, which greatly influences the presence and movement of subsurface water.

AVAILABLE RESOURCES

Make an initial investigation of the time, materials, equipment, and labor available to build a drainage system. Without a sufficient quantity of these essentials, the construction of an adequate system is impossible.

DESIGN PROCEDURES

Designing a drainage system involves numerous assumptions and estimates. The degree of protection to be provided is directly related to the importance of the established time-use period. The general location of the facility will be determined by its functional requirements.

The drainage system must be planned and designed for the predetermined location of the facility. There are three basic procedures in the design of any drainage structure:

- Determining the area (usually in acres) contributing runoff to the facility
- Estimating the quantity of runoff.
- Designing the drainage structure to carry the maximum expected runoff.

DETERMINING THE AREA CONTRIBUTING RUNOFF

When developing a tentative layout for the drainage system, identify all locations within the site requiring drainage structures because of topographical or manufactured features. This is best done from a topographic map of the area or a sketch of the project site.

Next, determine the acreage of the areas that contribute runoff to these required drainage structures. An analysis of existing channels is helpful in establishing locations for the required structures. Upon completion, this tentative plan should be field checked at the project site.

Establishing Drainage-Structure Locations

The initial step in developing a drainage-structure layout is to establish the location of the required drainage structures. Placement, in general. will be controlled by the topography. For example, a fill section which crosses a valley will require one or more culverts to permit the flow of storm runoff down the valley. A depression or enclosed area will require ditches or culverts at various points to remove accumulated rainfall.

Figure 6-6 shows an airfield with required culverts (X) and open channels or ditches (V). Note that at Point A the elevation is 65 feet, while at Point B the elevation is 55 feet. Culverts and ditches must be laid to carry water from high to low elevations. The alignment of these culverts and ditches

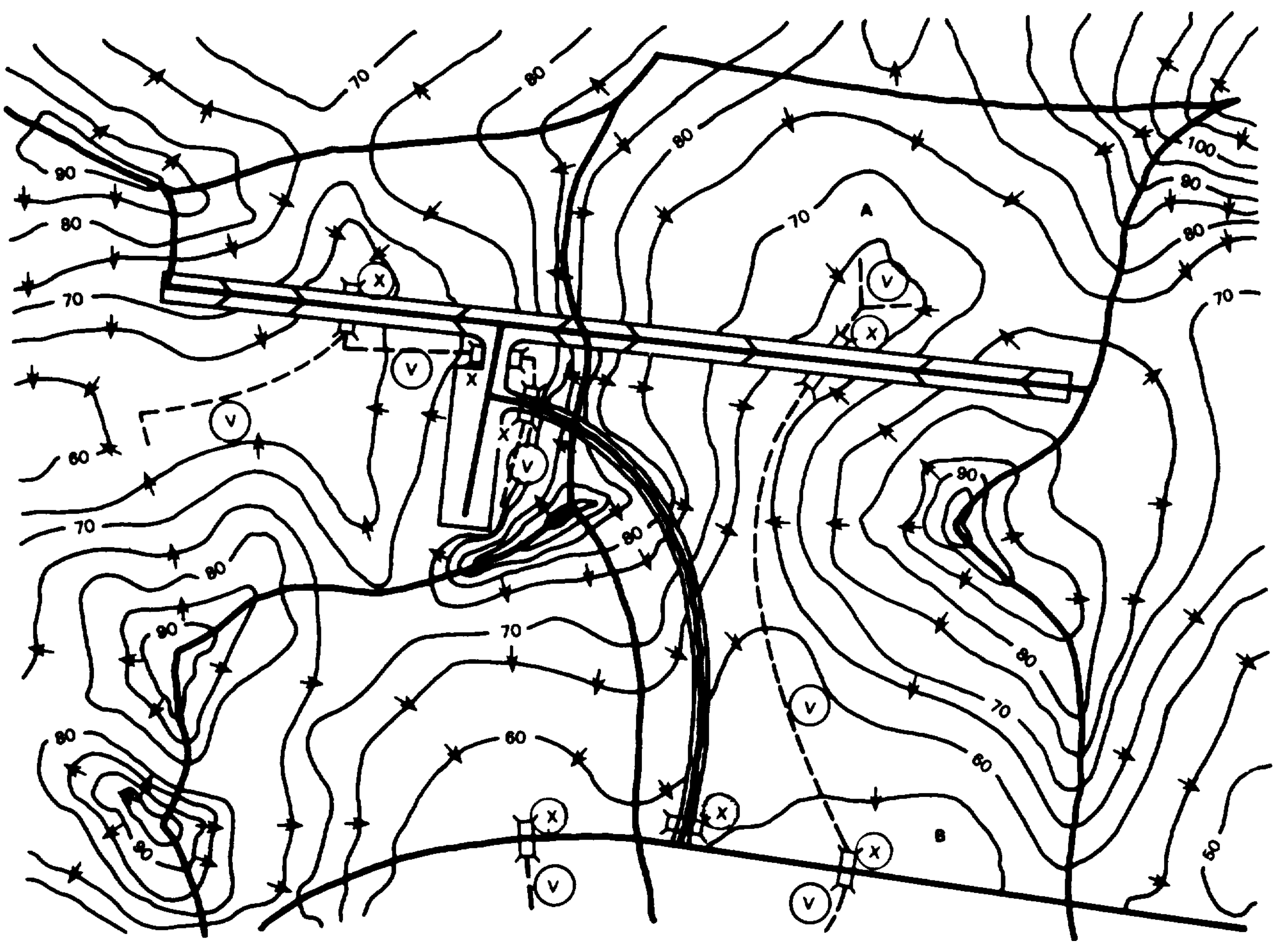


Figure 6-6. Typical airfield drainage features

should be as straight and smooth as possible, Sharp bends in ditches or near culverts will cause erosion. Not shown on this sketch arc the standard ditches constructed along the sides of all military roads and airfields.

Delineating Watersheds

After initially locating drainage structures, define the boundaries of the areas (or watersheds) contributing runoff to each of them. This process is known as delineation. Delineation is performed in six simple steps (refer to Figure 6-6):

Step 1. Locate all existing or proposed drainage structures on the topographic map or sketch (X and V).

Step 2. Identify and mark all terrain high points.

Step 3. Draw arrows representing water flow away from these high points. (These arrows must always be **perpendicular** to the contour lines because water flows downhill.)

Step 4. Continue drawing the arrows until they converge upon the culvert or the end of the ditch. Remember that runoff will flow parallel to a road or airfield when it is intercepted by side (or interceptor) ditches.

Step 5. Draw delineation lines. (These lines will run from high point to high point, indicating where the flow of surface runoff separates.) Delineation lines arc located so they cannot be crossed by any flow arrows. Flow arrows only cross delincation lines at culverts or ditches.

NOTE: Delineation lines are drawn between opposing arrows, (See Figure 6-7.)

Figures 6-7 and 6-8 depict the use of flow arrows and delineation lines for special, manufactured structures such as roads, airfields, and superelevated roads. When airfields or straight roadways are properly constructed, they arc shaped so that the highest portion of the cross section (the crown) is at the centerline, as illustrated in Figure 6-8. In the plan view, the delineation will be at the centerline precisely

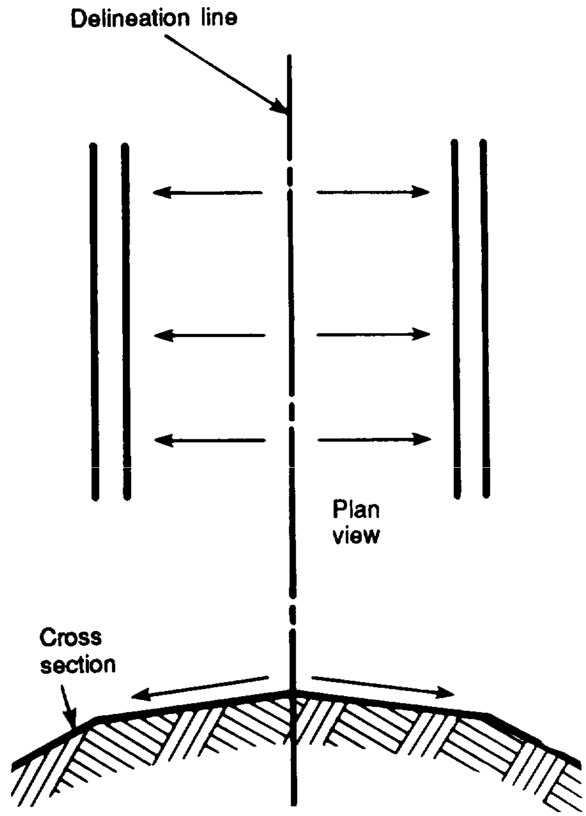


Figure 6-7. Delineation of roads and air fields

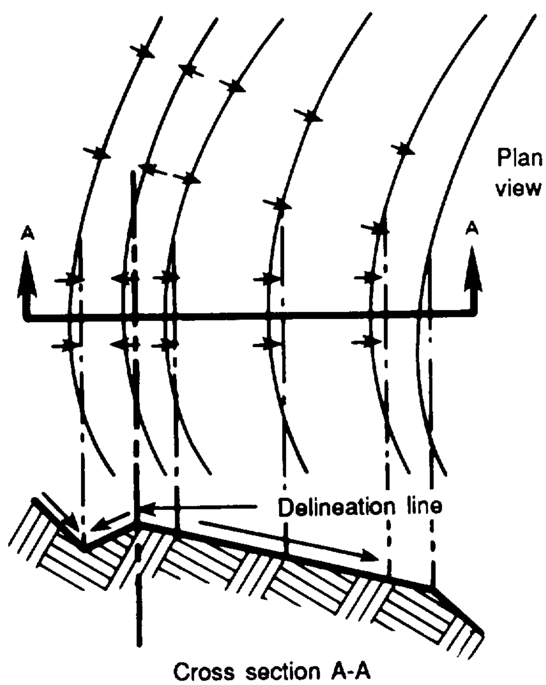


Figure 6-8. Delineation of superelevated roads

where the accumulated storm water would separate and flow in opposing directions. Figure 6-8, page 6-13, shows how super-elevated roadways (roadways that arc banked to ease the flow of traffic through a curve) arc delineated. In a properly constructed, superelevated road, storm water will always separate at the outside edge of the curve.

Several examples are provided to aid in visualizing special terrain features, including hills, ridges, valleys, and saddles, as shown in Figures 6-9 and 6-10 and Figure 6-11, page 6-16.

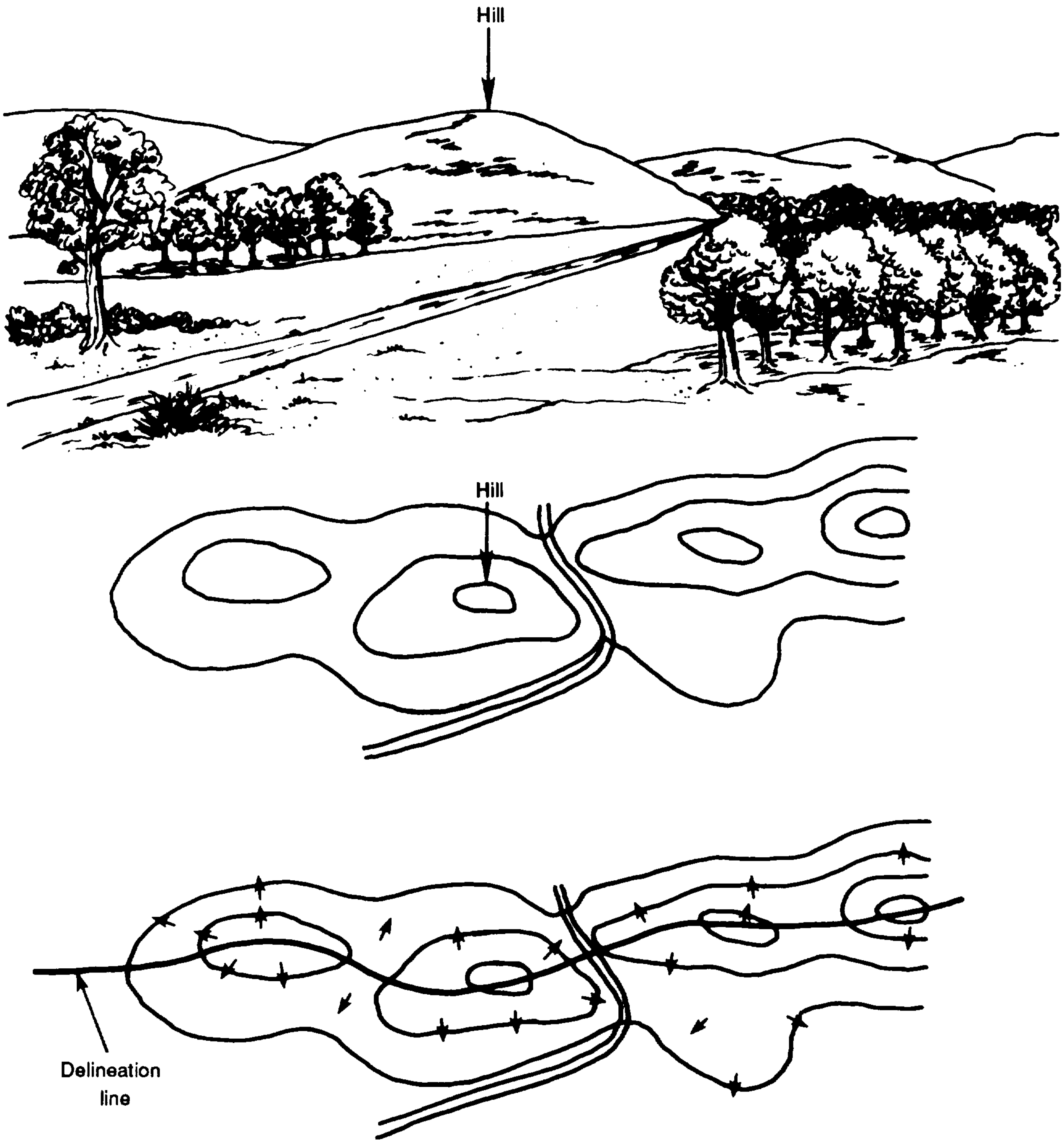


Figure 6-9. Delineation of a hill

Step 6. Since each cover or soil type will have an effect on the basin, if there arc multiple types of cover in the basin, each cover or soil type must be delineated and measured according to its respective cover type. (See Figure 6-12, page 6- 17.)

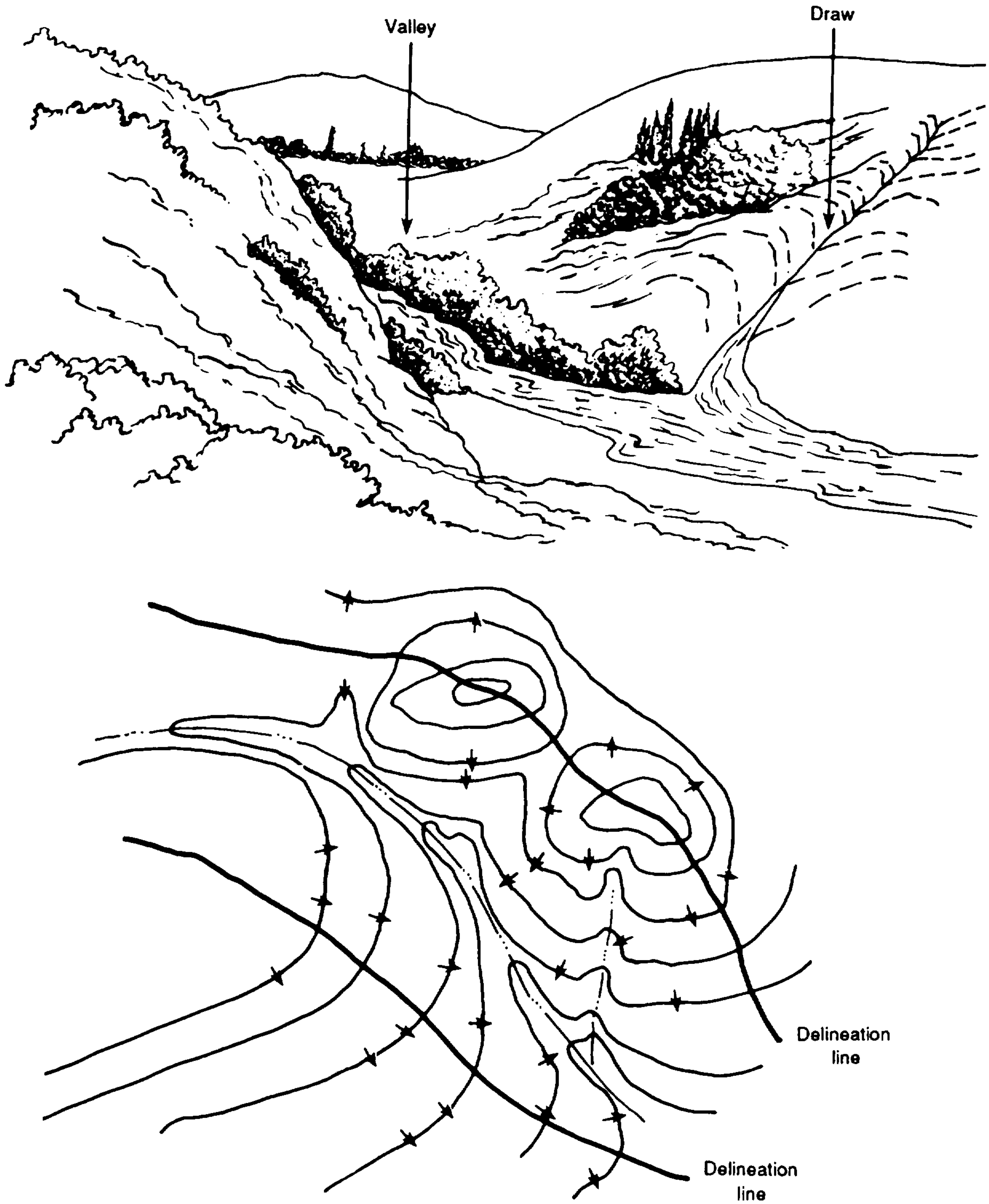
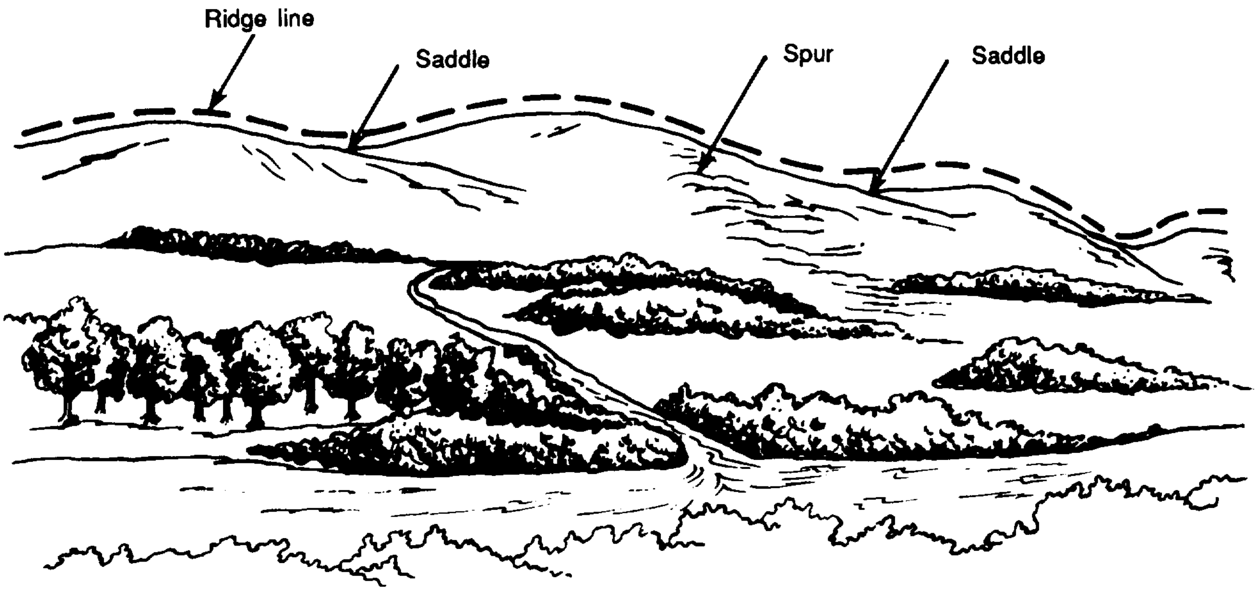


Figure 6-10. Delineation of valleys and draws



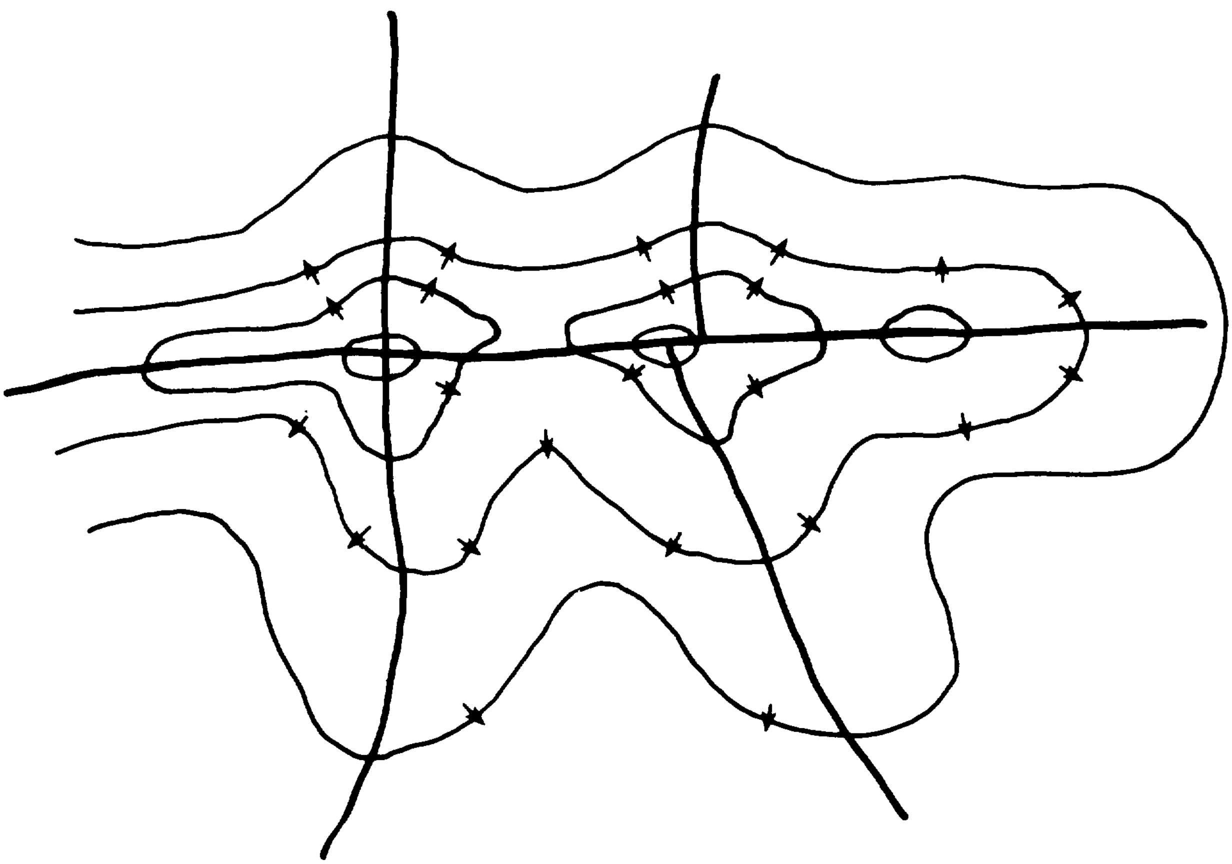


Figure 6-11. Delineation of ridges, spurs, and saddles

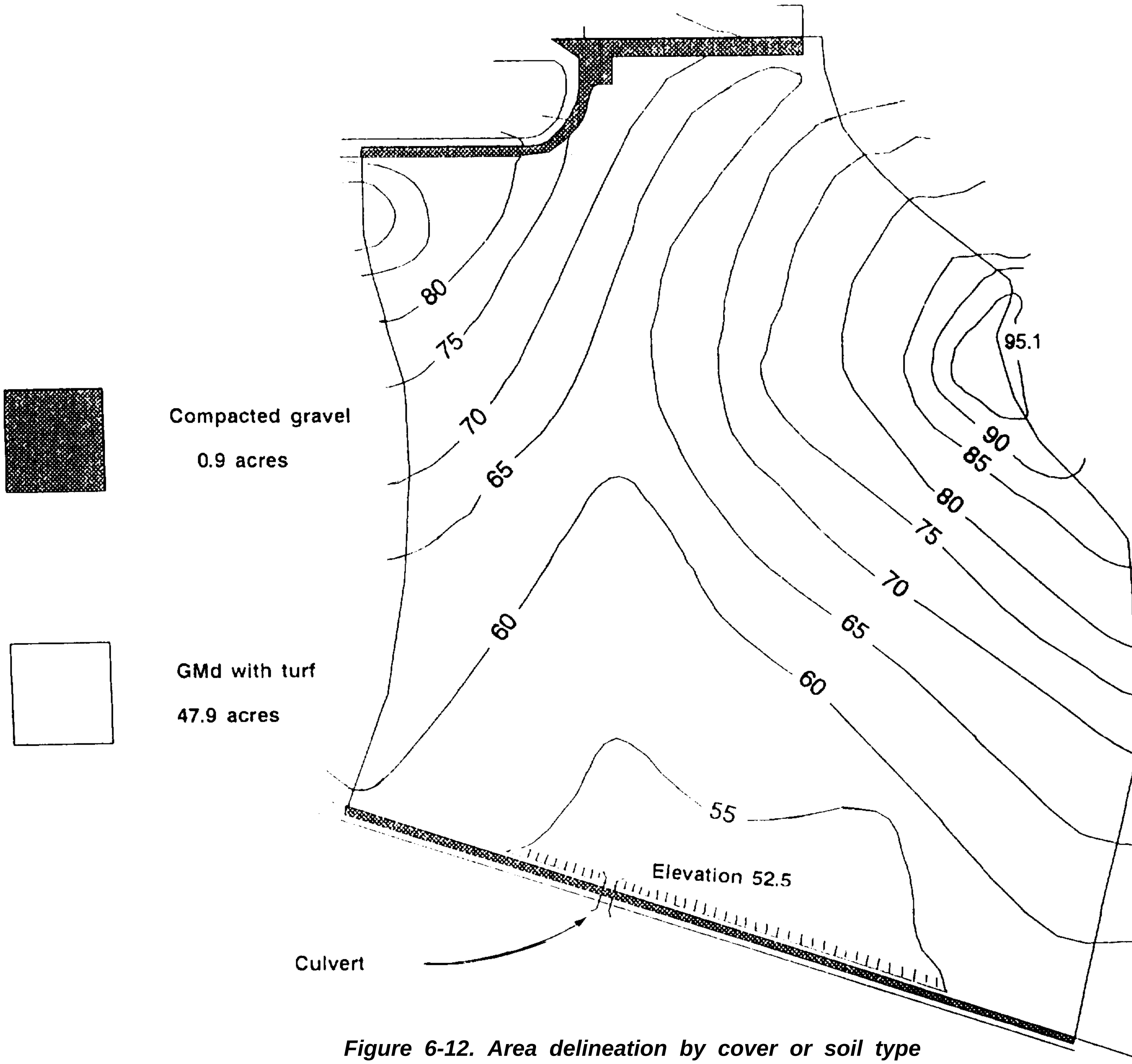
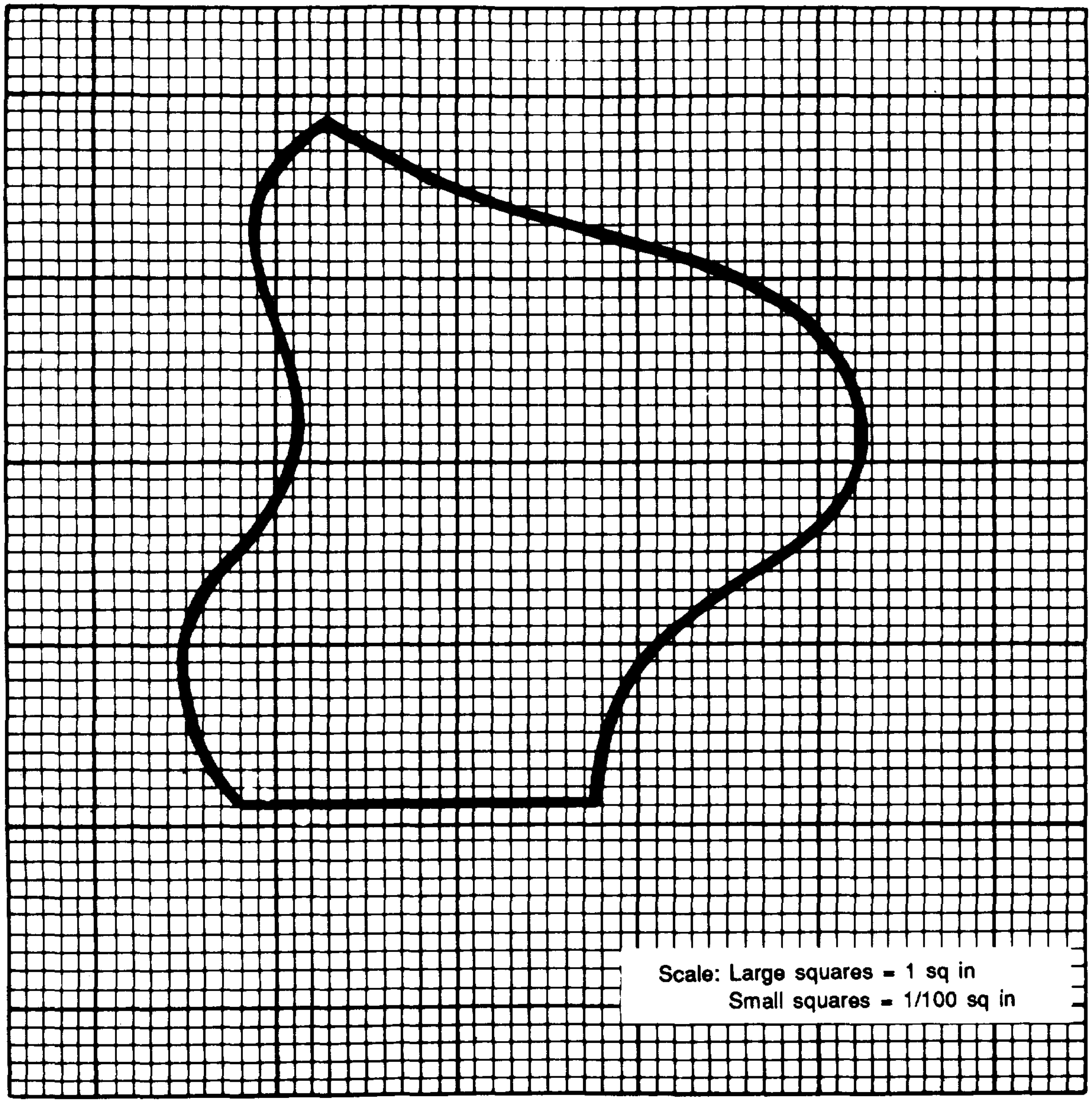


Figure 6-12. Area delineation by cover or soil type

Determining Drainage-System Size

After delineating the watershed determine its size in acres. Make this measurement carefully, since the size directly influences the calculation of runoff from the watershed at peak flow. Use any accurate method of measurement desired. A planimeter which measures the area of a plane figure as a mechanically coupled pointer traverses the figure's perimeter, is quite accurate and should be used, if available. However, several other methods arc suitable for field estimation.

Counting-squares method. To make a hasty approximation of an area, transpose the outline of the watershed to graph paper (or other suitable grid). Count the number of whole squares and estimate the values of the partial squares Multiply the total number of counted squares by the number of counted square feet represented by a single square. Then convert the measurement in square feet to acres (1 acre = 43,560 square feet), Figure 6-13, page 6-18, shows this technique.



Calculations:

Approximate number of large squares = 5 = 5 sq in
Approximate number of small squares = 564 = 5.64 sq in
Approximate area = 10.64 sq in

Example:

If one square represents 10,000 actual sq ft on the ground, then the delineated area represents:

$$\frac{10.64\ (10{,}000)}{43{,}560} = 2.44 \text{ acres}$$

Figure 6-13. Area measurement - counting-squares method

Geometric-shapes method. This method involves estimating the watershed shape in terms of rectangles, triangles, or trapezoids. Using the formulas below for determining the areas of these geometric shapes, determine the area of each shape and then total all areas to estimate the area of the watershed. This technique is shown in Figure 6-14, page 6-20.

Rectangle:
Area = base x height or $A = bh$

Triangle:
Area = base x height or $A = bh$

Trapezoid
Area = sum of bases x height
or $A = (b_1 + b_2)(h)$

Stripper method. The stripper method is a variation of the geometric-shapes method. This method is shown in Figure 6-15, page 6-21. Approximate the area by drawing a series of lines that are equidistant (stripper width) across the delineated area. Then measure the lines and total all of them. L = total of the lengths, This method is more applicable for field estimations. Use a stripper width of 1 inch.

The total of the lengths (L) is then multiplied by the stripper width. This would represent the total area on the map in square inches. Since the value of 1 square inch on the map would represent the map scale squared on land, the acreage can be found by multiplying L (in inches) x stripper width (in inches) x (map scale in feel per inch $(ft/in))^2$ and dividing the product by 43,560 ft^2/acre.

Example:

L = 12.5, map scale = 175 ft/in, stripper width = 1 inch (in)

Solution:

Step 1. $12.5 \times 1 \text{ in} = 12.5 \text{ in}^2$

Step 2. $12.5 \text{ in}^2 \times (175 \text{ ft/in})^2 = 382{,}812.5$ feet $(ft)^2$

Step 3. $\dfrac{382{,}812.5 \text{ ft}^2}{43{,}560 \text{ ft}^2/\text{acre}} = 8.78$ acres

ESTIMATING THE QUANTITY OF RUNOFF

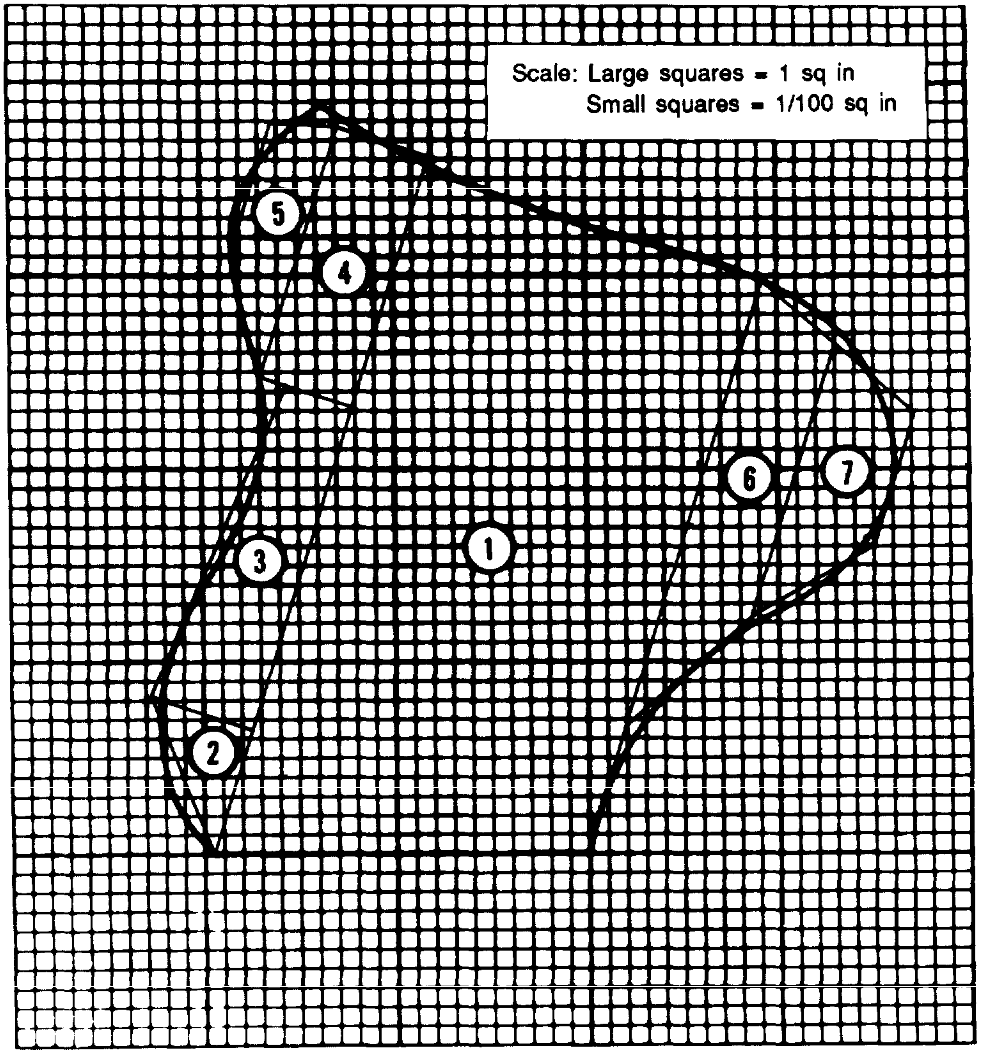
Drainage systems must be designed to accommodate the peak flow generated by runoff from contributing watersheds during the design storm, Many techniques arc available for determining the peak flow, but most arc too complex for general field use.

This manual will demonstrate the most comon method for estimating runoff—the rational method.

DESIGNING DRAINAGE STRUCTURES FOR MAXIMUM RUNOFF

To accommodate the peak flow of the design storm, design structures must provide a sufficient cross-sectional area and longitudinal slope for passing storm runoff. If pending or flooding of adjacent areas must be prevented, the design must be for peak flow. At the same time, water velocities generated at peak flow must not be so great as to cause damage to the drainage structure or excessive erosion and scouring 10 the protected facility.

Determine the capacity of drainage structures by calculating the runoff from all contributing drainage areas. Specific procedures for designing open channels and culverts are discussed later in the chapter.



Calculations:

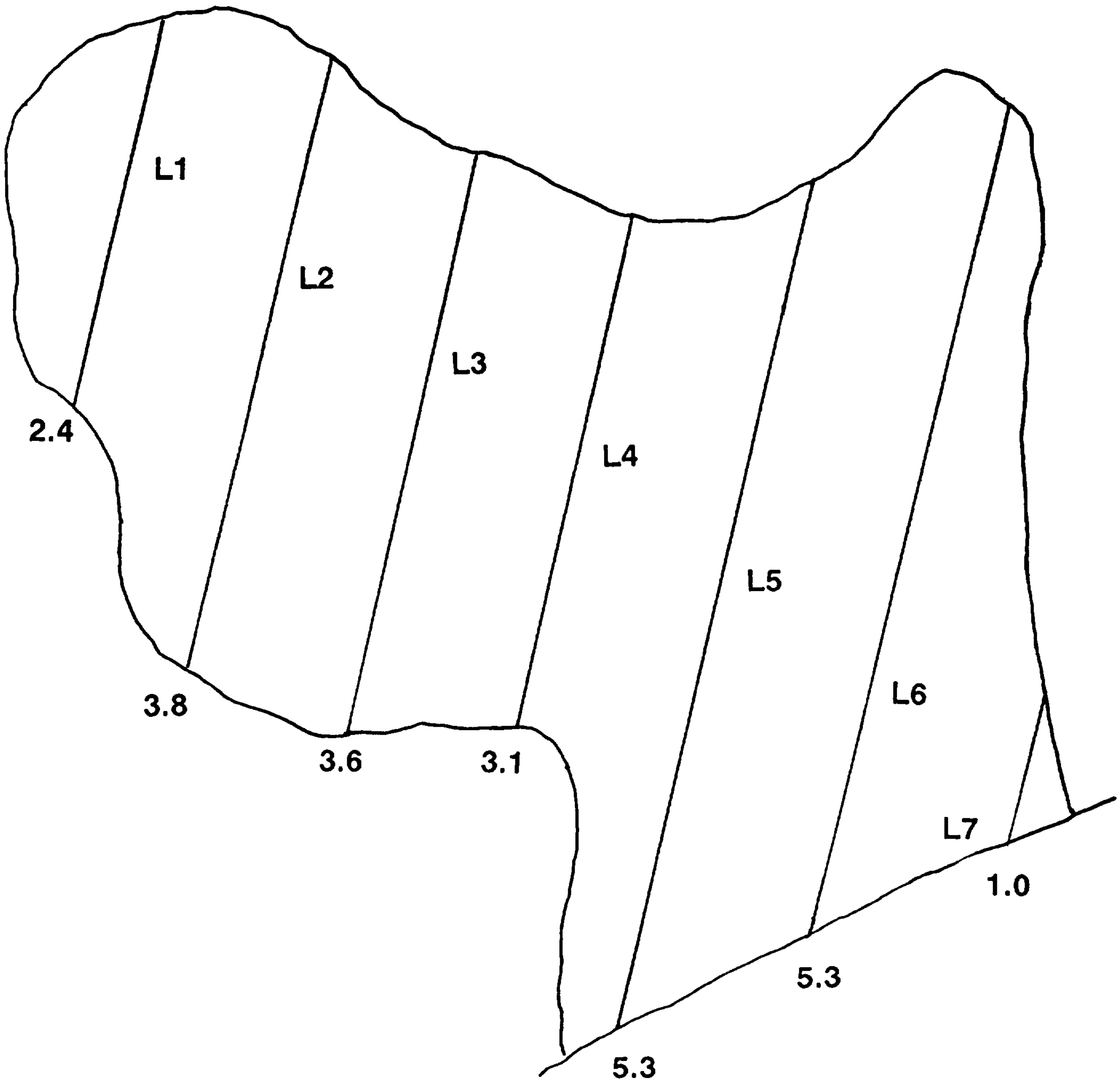
Area #1 = trapezoid = ½ (3.90 + 3.10) 1.94 = 6.79 sq in
Area #2 = triangle = ½ (0.5)(0.93) = 0.23 sq in
Area #3 = trapezoid = ½ (0.5 + 0.34) 1.35 = 0.57 sq in
Area #4 = rectangle = 0.30 x 1.56 = 0.47 sq in
Area #5 = trapezoid = ½ (1.43 + 0.75) 0.85 = 0.93 sq in
Area #6 = trapezoid = ½ (2.37 + 1.65) 0.41 = 0.82 sq in
Area #7 = trapezoid = ½ (1.65 + 0.94) 0.50 = 0.69 sq in
Total area = 10.5 sq in

Example:

If one square represents 10,000 actual sq ft on the ground, then the designated area represents:

$$\frac{10.5 \times 10{,}000}{43{,}560} = 2.41 \text{ acres}$$

Figure 6-14. Area measurement - geometric-shapes method



1.0 + 5.3 + 5.3 + 3.1 + 3.6 + 3.8 + 2.4 = 24.5 in

$\sum$ lengths = 24.5 in

Map scale: 1 in = 285 ft

Step 1. 24.5 in x 1 in = 24.5 in^2

Step 2. 24.5 in^2 x $(285\ ft/in)^2$ = 1,990,012.5 ft^2

Step 3. $\dfrac{1{,}990{,}012.5\ ft^2}{43{,}560\ ft^2/acre}$ = 45.7 acres

Figure 6-15. Area measurement - stripper method

ESTIMATING RUNOFF USING THE RATIONAL METHOD

ESTIMATING PRINCIPLES

The rational method is used to estimate the expected peak storm runoff at a given drainage basin outlet. Much of the input to the formula is based on judgment. Therefore, it is imperative that sound engineering judgment be used to determine the input data.

ASSUMPTIONS

The rational method is based on the following underlying assumptions and limitations:

- The area is not greater than 1,000 acres and is regular in shape, with a homogeneous cover and soil type.
- The entire drainage area is contributing runoff to the outlet point when peak runoff is obtained.
- The design rainfall intensity is uniform over the entire drainage area (that is, the rainfall is uniform over time and space).
- There are no active streams draining the area. (If an active stream drains the basin, use the hasty method found in FM 5-34.)

FORMULA

The rational method uses the following formula:

$$Q = CIA$$

where—

Q = peak runoff in cfs
C = runoff coefficient
I = intensity of rainfall in in/hr
A = drainage area in acres

The following conversion factor is applied to this formula:

$$\frac{1 \text{ acre} \times 1 \text{ inch}}{1 \text{ hour}} = 1.0083 \text{ cfs}$$

This is so close to unity that no correction factor is added; hence, the name *rational* (because a rational conversion of units] is used.

FORMULA VARIABLES

The rational formula has three variables. The C and I variables are explained here. The A variable is explained later in this chapter.

The C Variable

The runoff coefficient, or C variable, accounts for losses from precipitation. The C variable is the decimal fraction of the amount of water expected to run off relative to the amount of precipitation. It can be expressed as the ratio—

$$C = \frac{runoff}{rainfall}$$

Table 6-1 gives conservative values of C. Knowledge of an area's USCS classification (for example, GMd) or an estimate of the soil's perviousness allows selection of a C value. C values appear in the table for manufactured surfaces and for wooded areas as well. An area of SP soil (a pervious, sandy soil with a slope less than or equal to 2 percent) with turf has a C factor of 0.10; that is, only 10 percent of the rain falling on this soil will actually run off.

Table 6-1. Runoff coefficients

Soil or Cover Classification	C VALUES					
	Slope ≤ 2%		Slope >2 & <7%		Slope ≥7%	
	w/turf	w/o turf	w/turf	w/o turf	w/turf	w/o turf
GW,GP,SW,SP	.10	.20	.15	.25	.20	.30
GMd,SMd,ML, MH,Pt	.30	.40	.35	.45	.40	.50
GMu,GC,SMu, SC,CL,OL,CH,OH	.55	.65	.60	.70	.65	.75
Wooded area	.20	.20	.20	.20	.20	.20
Asphalt pavement		.95		.95		.95
Concrete pavement		.90		.90		.90
Gravel/macadam		.70		.70		.70

The remaining 90 percent becomes lost to runoff through infiltration and other factors. At the other extreme, an asphalt pavement has a C value of 0.95. Only 5 percent of the rain falling on asphalt will be lost. The remaining 95 percent is expected to become runoff.

NOTE: C values given in Table 6-1 are actually maximums of ranges of allowable values for the cover or soil categories. Using the maximum value, a conservative "worst-case" design runoff is calculated. To use values less than the maximums given in the table, refer to a reliable civil engineering text dealing with hydrology. The table is arranged with three columns for varying slope conditions.

C Versus Slope

As terrain becomes steeper, water flows sooner and more rapidly. This allows less time for infiltration to occur and results in the C value becoming larger for the natural cover or soil categories. For this reason, whenever the average slope of an area exceeds 2 percent, an adjustment must be made.

Table 6-1 is arranged with three columns for different slope conditions and their corresponding runoff coefficients, Use the column that corresponds with the average percentage of slope.

The C for a turfed soil is different from the C for bare soil. The turf (grass or other ground cover) exerts a drag on water, causing slower flow and providing more time for infiltration to occur; hence, a lower C results. Denuded soil (soil from which the turf or cover has been removed) requires an increased C because a swifter flow will result and less time will be available for losses to occur. If one cover type has more than one flow path, average the slopes and use the appropriate column in Table 6-1.

Example:

Flow path 1A = 2.3 percent and flow path 1B = 1.9 percent.

Solution:

$$\frac{2.3\% + 1.9\%}{2} = 2.1\ \%$$

C for Nonhomogeneous Areas

One of the assumptions made by the rational method is that there is a homogeneous cover and soil type throughout the area. Quite often this is not the case, especially in areas where humans have exerted their influence on the topography.

If one type of cover and soil predominates in 80 percent or more of the area, the area is called simple and the C value for that predominant soil and cover type controls. If no one type of cover and soil type predominates in 80 percent or more of the total area, the area is complex and the C value must be weighted; that is, the C value has to be adjusted to account for the proportion of C contributed by each subarea.

To help understand this, imagine a complex area with one subarea of average turf and the other of bare soil. The slope of the bare soil does not affect how fast (or slow) the water runs off the turfed area and, as a result, how much of the water soaks into the turfed area. The converse is also true. The slope of the turfed area does not affect the speed or amount of water that runs off the bare soil area. Table 6-1 shows C values with and without turf.

Weight the C value by multiplying the corrected C values by the area (in acres) that the C values affect. Then total the products and divide by the total acreage. Expressed mathematically, the formula is—

$$C = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \ldots C_NA_N}{A_1 + A_2 + A_3 + \ldots A_N}$$

where—

C_1A_1 = C value and area for first subarea
C_2A_2 = C value and area for second subarea

The I Variable

As explained previously in this chapter, rainfall intensities can be determined from pinpoint source data or isohyetal maps. The former method provides more accurate results if reliable data is available. The task of calculating the critical duration for any given drainage area is detailed here.

Time of Concentration

Under the assumptions listed at the beginning of this section and with the intensity-duration relationships presented earlier, only one particular storm will give a maximum discharge (Q) for a given area. This particular storm is the one that rains over the entire area being drained for a period of time just long enough to fill the outlet with runoff from all segments of the area at the same time. This time is called the area TOC. A storm of shorter duration than this TOC would not last long enough for the runoff from the more distant segments of the area to reach the outlet. The outlet woulrd be filled only with the runoff from nearby segments. Therefore, runoff would not be maximum

In Figure 6-16, all of the area below the 10-minute line will drain in 10 minutes or less. Runoff from the area between the 10- and 20-minute lines will reach the outlet in not less than 10 minutes but will have drained in not more than 20 minutes. Similarly, the runoff from the area between the 20- and 30-minute lines will reach the outlet in not less than 20 minutes nor more [ban 30 minutes. At the end of 30 minutes the entire area is draining. Therefore, the TOC at the outlet for this area is 30 minutes.

If a storm of 20-minute duration sweeps over the area in a uniform fashion, only a fraction of the total area inside the 20-minute boundary simultaneously contributes runoff to the outlet at the end of the storm. All runoff from the upper third of the area reaches the outlet after the rainfall has ceased and after much of the lower acreage has finished contributing runoff.

If a 30-minute-duration storm sweeps over the same area in a uniform fashion, the entire area contributes runoff to the outlet in the 30-minute time frame (the TOC mentioned above).

If a storm with a duration longer than the TOC occurs, the drainage designer can easily picture (using the standard intensity-duration curve) that the intensity, I, will be less than the I of the 30-minute storm. Examination of the rational-method equation, $Q = CIA$, reveals that since C and A would not change as I decreases, Q must decrease as well. The critical-storm duration which yields the design Q must then be equal to the contributing area's TOC.

Determining TOC

Determine the area TOC by determining representative flow paths. A flow path is the path that a typical drop of water will follow from the time it hits the ground until it reaches the area outlet, The flow path is called representative, because not all drainage areas are as regular in shape as the area in Figure 6-16. The path selected must be representative of the time at which most of the area will be contributing water to the outlet point. Establishing representative flow paths is based largely on experience and judgment (trial and error).

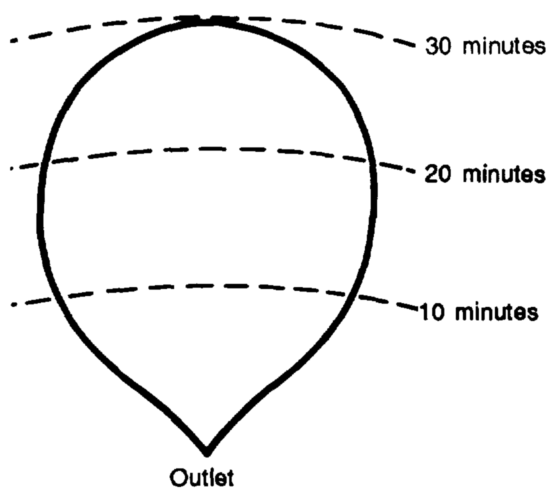


Figure 6-16. TOC, regular area

Unlike the area depicted in Figure 6-16, the area in Figure 6-17 is irregular as most natural areas will be, In irregular areas, it is especially critical that the flow paths chosen truly represent the time required for most of the area to drain. All the area below the 10-minute line will drain in 10 minutes or less. The area between the 10- and 20-minute lines will drain in not less than 10 minutes nor more than 20 minutes and so forth up to the 40-minute line. Flow lines a, b, and c have been determined; 90 percent of the total area (90 acres) lies below the 30-minute line and will drain in 30 minutes or less. Water from the remaining 10 acres will reach the outlet in not less than 30 minutes nor more than 40 minutes. Flow lines a and c should be chosen as the representative flow paths and used to determine the TOC because they are indicative of the time it will take for most of the water from the area to reach the outlet, Line b is not representative.

For simplicity, in this example let C arbitrarily equal 1.0 and assume that the 1-hour, 2-year intensity is 2,0 inches per hour. If a 40-minute-duration storm occurs, in 40 minutes the entire area will be wet and contributing water to the outlet point, The standard intensity-duration curve in Figure 6-4,

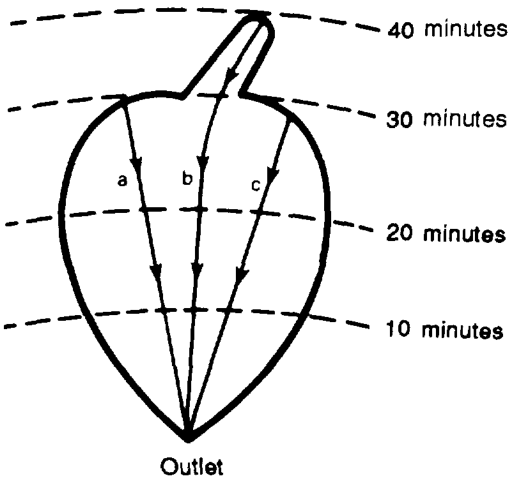


Figure 6-17. TOC, irregular area

page 6-9, shows that the I for a 40-minute storm is 2.7 inches per hour; therefore, the estimated runoff is—

$Q = CIA$
$Q = (1.0)(2.7 \text{ in/hr})(100 \text{ acres}) = 270 \text{ cfs}$

A storm of 30-minute duration will have an intensity of 3.2 inches per hour. At the end of 30 minutes, 90 percent of the area (90 acres) will be contributing water to the outlet and the volume will be—

$Q = CIA$
$Q = (1.0)(3.2 \text{ in/hr})(90 \text{ acres}) = 288 \text{ cfs}$

which is larger than the 270 cfs estimated for the entire area.

Flow paths must be chosen that represent the time required for most of the area to drain. As shown above, a shorter storm of higher intensity may cause a larger flow. After all the chosen paths have been timed, the times should correspond to each other within a few minutes. If times are not relatively close, make a careful check to determine why, and assess the area to determine which of the times will produce the critical flow. Apply rainfall adjusted to this critical duration over the entire watershed. The design runoff from the watershed in Figure 6-17 would be—

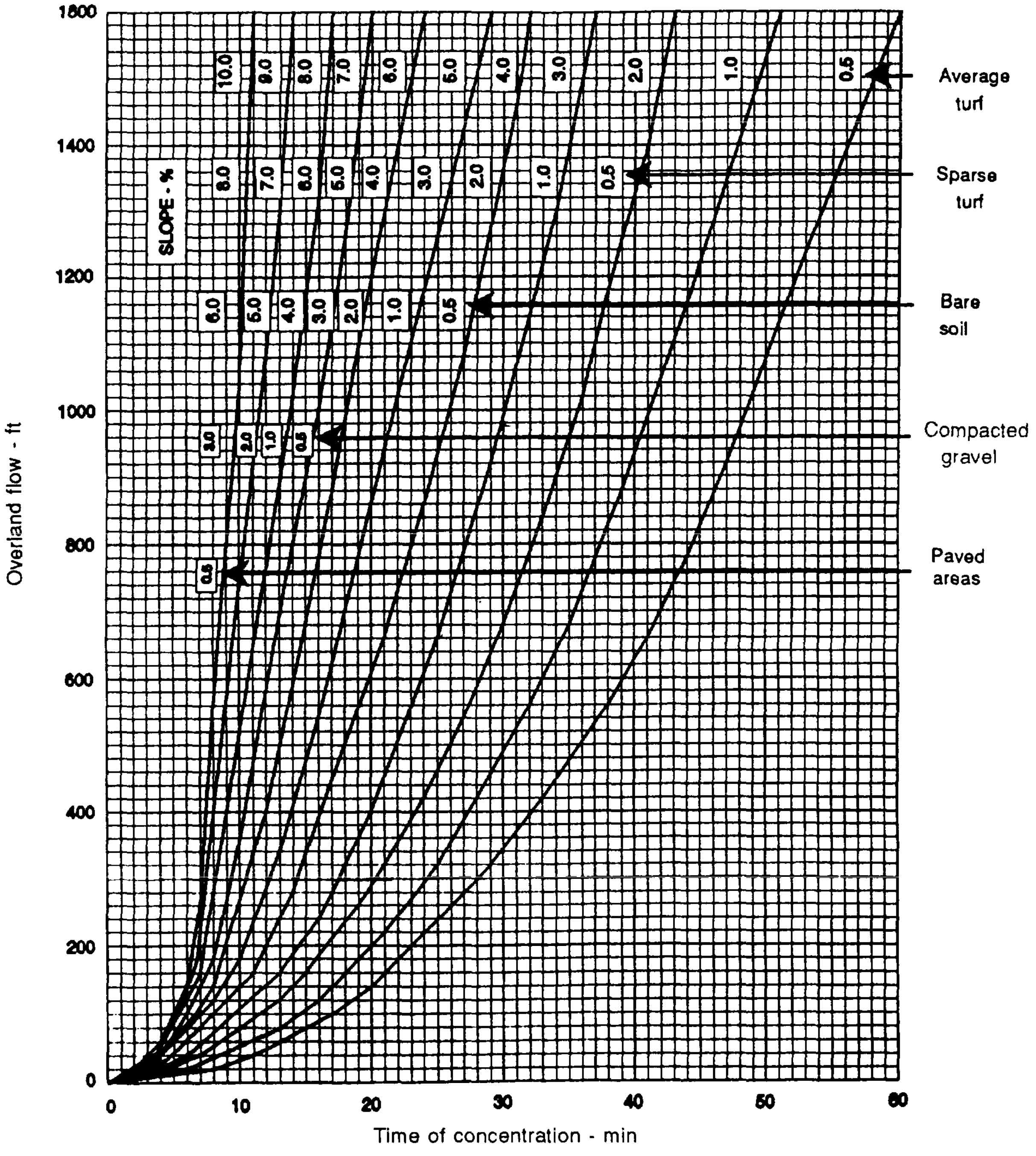
$Q = CIA$
$Q = (1.0)(3.2 \text{ in/hr})(100 \text{ acres}) = 320 \text{ cfs}$

After representative flow paths have been established, estimate the time it will take for water to reach the outlet if it travels along the established path. To do this, determine (through observation) the nature of the surface cover and the slope of the flow path. Slope affects the velocity of the water in that the steeper the slope, the faster the water runs. Water will also travel faster across a paved area than across a grassy area of the same slope because grass slows the flow. Flow is slower over bare soil than over pavement but faster than grass. Flow in a ditch is more rapid than overland flow over turfed, bare, or compacted gravel surfaces.

Estimating Flow Time for Single Covers

After establishing the location, the cover, and the slope of a flow path, Figure 6-18 can be used to estimate the travel time along the flow path. It is important to understand what the illustration is depicting as well as how to use it properly.

Notice that there is a series of curves, each with linear and curvilinear portions. The slope of the curve indicates the velocity at a given point along the flow path. In the curvilinear portion, the slope is initially zero and gradually steepens until it becomes linear. This represents the fact that water



NOTE: It is valid to interpolate between percent slope curves.

Figure 6-18. Flow travel time

initially moves very slowly and begins to pick up speed only as its accumulated depth increases. It is initially slow flowing in a laminar or sheet-flow manner and gradually becomes turbulent (and faster) as it progresses downhill. At some point, the turbulent-flowing water reaches some steacly-state velocity. It is apparent from Figure 6-18 that the slope and “slickness” of the flow path dictate how quickly the transition occurs from slow-moving laminar flow to rapid, fully developed turbulent flow

To estimate the travel time in sheet-flow conditions, use Figure 6-18. Enter the right-hand vertical edge at the appropriate cover type of the flow path. Proceed horizontally to the left until reaching the curve labeled with the slope of the path. Follow the curve up or down until you reach the intersection of the horizontal line equaling the flow-path length, which is determined on the left-hand vertical edge. Read the travel time, which is found by drawing a line vertically from the intersection to the lowermost axis. Some examples of the use of this graph are as follows:

Path	Cover	Length (ft)	Slope (%)	Travel Time (rein)
1	Sparse turf	800	3,0	19
2	Paved area	500	1.5	8
3	Average turf	600	2.5	26

Note that it was necessary to interpolate to find the travel time for path 3. It is valid to interpolate between labeled lines, but never extrapolate above or below the limiting-curve values. Use the limiting curve in that situation To estimate the travel time in ditch-flow conditions, usc Table 6-2. Using the slope of the flow path, enter the chart, then read the right-hand column under velocity. To calculate the velocity, divide the length of the path by the velocity obtained from the chart.

Example:

Path 1B

Cover - Ditch
Length - 1,015 ft
Slope - 0.9%
Chart velocity - 135 feet per minute (fpm)

Table 6-2. Estimating flow velocity

Slope	Velocity (fpm)	Slope	Velocity (fpm)
0.5	115	1.8	188
0.6	120	1.9	194
0.7	125	2.0	200
0.8	130	2.1	205
0.9	135	2.2	210
1.0	140	2.3	215
1.1	146	2.4	220
1.2	152	2.5	225
1.3	158	2.6	230
1.4	164	2.7	235
1.5	170	2.8	240
1.6	176	2.9	245
1.7	182	3.0	250

NOTE: Steady state will be at 250 fps.

Solution:

$$\frac{1{,}015}{135\ \text{fpm}} = 7.5 \text{ minutes (min)}$$

Estimating Flow Time for Multiple Covers

In many cases, a flow path traverses more than one cover type. Estimating travel time accurately then becomes more complicated because it is not appropriate simply to add times obtained from Figure 6-18, page 6-26. Laminar flow occurs only once along a flow path, no matter how many cover types are traversed. For subsequent covers, it becomes necessary to estimate flow velocity using Table 6-2, page 6-27.

To estimate travel time in a ditch, use Table 6-2. Enter the table using the slope of the flow path. Then read right to the velocity column and find the velocity in feet per minute. By knowing the flow-path length and the table velocity, the travel can be calculated.

Example:

Assume that paths 1 and 2 from the preceding example were actually the upper and lower lengths of one combined flow path. To estimate their combined travel time, first estimate each separately in the order the water would flow through them.

Solution:

Estimate the travel time of path 1. Since path 1 is uphill from path 2, nothing has changed from before. The travel time remains 19 minutes. Estimate the travel time of path 2. Remember that the flow entering at the upstream end of path 2 is al ready moving. To estimate travel time, divide the length of the path by the estimated velocity listed in Table 6-2. In this case, for a 500-foot, paved path at 1.5 percent—

$$\text{Travel time} = \frac{500\ \text{ft}}{(165\ \text{fpm})} = 3.0 \text{ min}$$

(Note that path 2 had an 8-minute travel time when considered alone.)

Arid partial travel times to get the total travel time:

Travel time = 19 min + 3.0 min = 22 min

It may be helpful, at times, to estimate the travel time through a culvert. A reasonable assumption of culvert velocity is 5 feet per second (fps) (300 fpm), although more precise determinations can be made with information presented later in this chapter.

Selecting Design TOC

The usual procedure is to establish several trial flow paths that are thought to be representative of the area and determine a travel time for each path. Compare the time for water to travel along each of the flow paths chosen. If the times arc within a few minutes of each other, select the longest time as the area TOC. If the times arc not within a few minutes of each other, make a complete analysis of the area. New flow paths may be needed to determine which of the times is representative of the bulk of the area draining. The largest representative time is chosen as the design TOC.

The A Variable

The drainage area, A, (the area contributing storm-water runoff to the culvert or ditch being designed) must be calculated in acres, This procedure was presented earlier in this chapter.

APPLYING THE RATIONAL METHOD

Application of the rational method of estimating drainage varies according to the type of drainage area. One type is a single, independent area which does not receive any drainage from an upstream area. Another type is a dependent or successive area that receives runoff from another area.

Single Areas

The rational method of estimating single areas is reasonably simple and straightforward, if it is done methodically. The steps are summarized in the proper order. lf this summary is followed step-by-step, the procedure will be correct and the estimate obtained will be as valid as the judgments that are made.

Step 1. Delineate the area to determine the area contributing runoff to your project location. Refer to page 6-13, Delineating Watersheds.

Step 2. Delineate subareas by soil or cover type. Refer to page 6-15.

Step 3. Determine acreage for each basin and subarea. Refer to page 6-17.

Step 4. Classify the drainage basin as simple or complex. A simple watershed has one cover or soil type over 80 percent of its total area. A complex watershed has no single cover or soil type covering at least 80 percent of its total area.

Step 5. Determine representative flow paths. Refer to the information on pages 6-24 and 6-25.

Step 6. Divide flow paths into two sections: laminar and ditch flow. As a rule of thumb, use 500 feet plus or minus 200 feet as a point where laminar flow will change to ditch or steady-state flow. Generally, overland flow concentrates into natural rivulets or channels after roughly 500 feet of travel. This distance may vary up to several hundred feet either way, depending upon such factors as soil type, vegetation, and slope, It is always best to visually investigate on-site to look for evidence of channeling, and take measurements accordingly. However, since in this problem it is not possible to visually investigate the drainage area, clues to determine when overland flow (sheet flow) ends and ditch flow begins must come from topographic information alone. Some of these clues may be the beginning of uphill swales or flow paths that converge in a swale. This convergence may take place in a valley where multiple paths meet.

Step 7. Determine the slope of each section of flow path. See page 6-25.

Step 8. Determine the average slope of the basin or each subarea based on flow paths. Define the slope as being either $\leq$ 2%, > 2% and < 7%, or $\geq$ 7%.

Step 9. Find the C value of the basin or each subarea based on soil or cover type and slope from Table 6-1, page 6-22. Select C from the appropriate column in Table 6-2, page 6-27, making sure that you pick the right C-value column. Notice that the columns are arranged with respect to slope and cover type.

Calculate C_{wtd}, if the area is complex.

$$C_{wtd} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \dots C_NA_N}{A_1 + A_2 + A_3 + \dots A_N}$$

Step 10. Determine the travel time for each flow path and select the longest flow path as the basin TOC. Refer to sheet flow on page 6-27 and ditch flow in Table 6-2.

Step 11. Find I for a 2-year, 60-minute storm on the world isohyetal map (Figure 6-3, page 6-7).

Step 12. Adjust I based on the TOC (step 10), using Figure 6-4, page 6-9.

Step 13. Calculate Q using C from step 12, and A from step 3 as follows: Q=CIA.

Example:

Estimate the amount of runoff expected to arrive at the culvert in Figure 6-19, page 6-30.

The location is Giessen, Germany, and the design life is two years.

NOTE: A number of steps, quantities, and calculations will be "given" to illustrate the process.

Solution:

Step 1. Delineate the area. (Given for this example.)

Step 2. Delineate the subareas by soil or cover type. (Given for this example.)

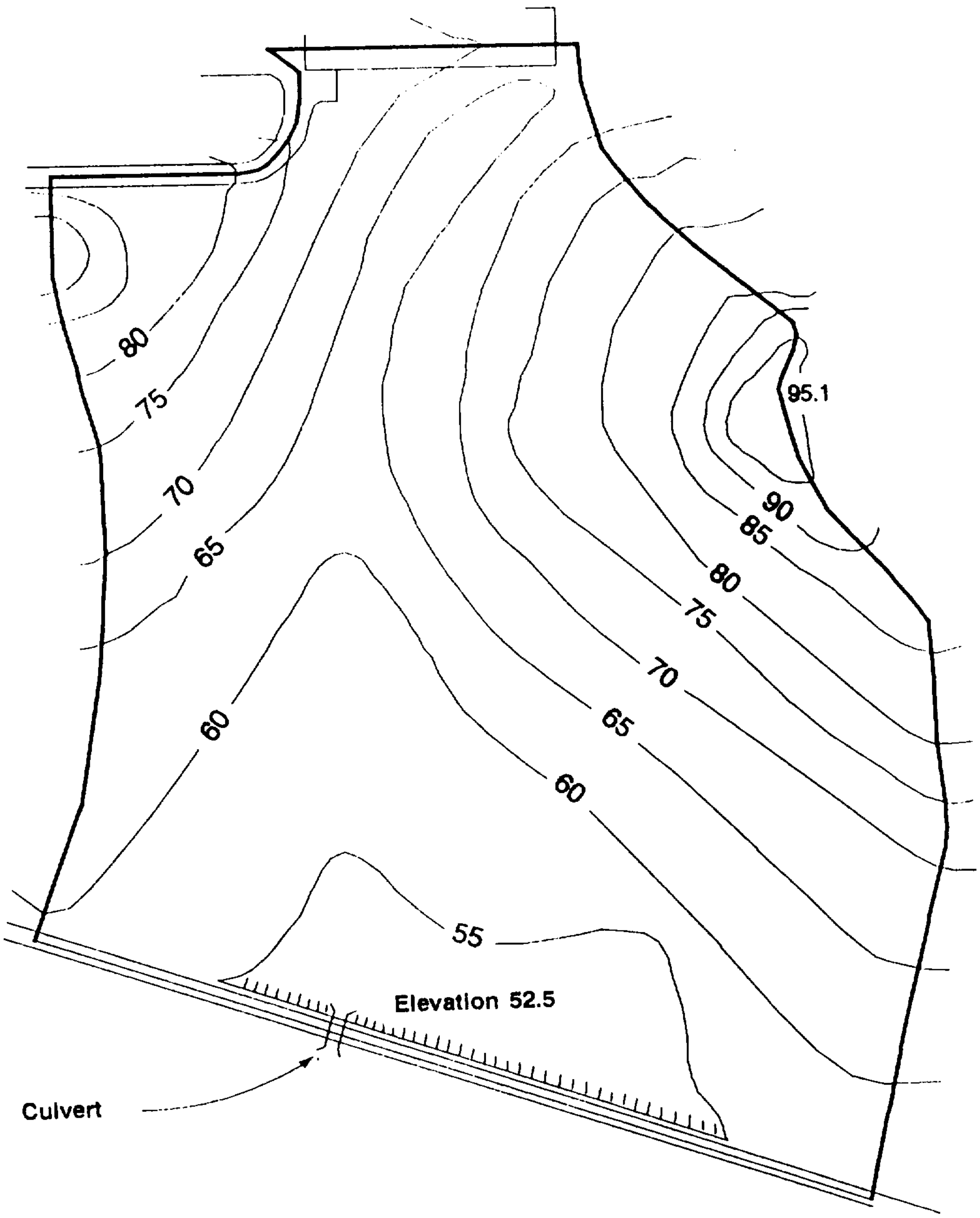


Figure 6-19. Delineating runoff area

Step 3. Determine the acreage for each basin or subarea. The following acreage is given for this example:

Average turf GMd	47.9 acres
Compacted gravel	0.9 acre
Total acreage (A)	48.8 acres

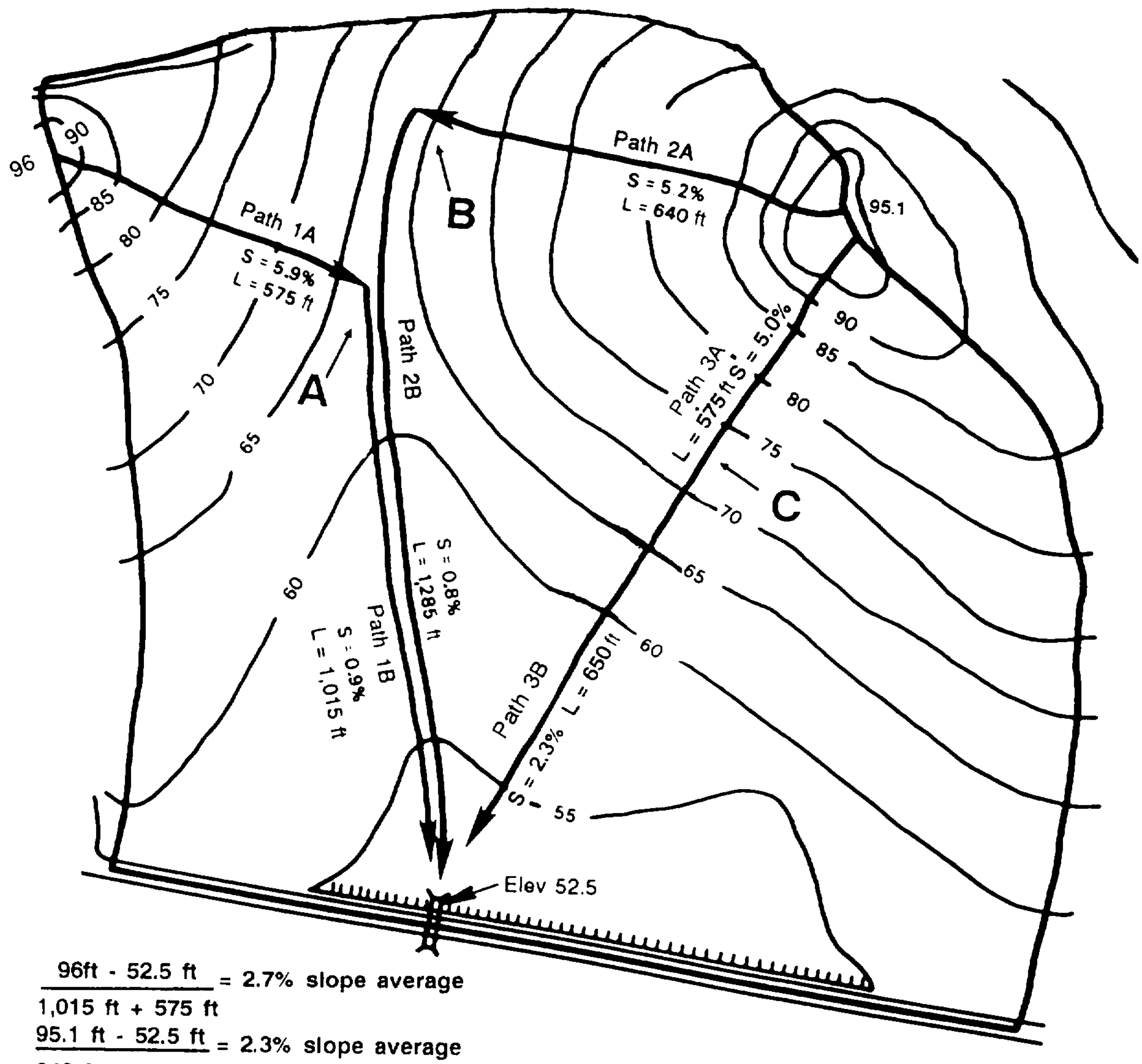
Step 4. Classify the drainage basin as simple or complex. Divide the largest soil or cover group (the GMd soil] with turf subarea equaling 47.9 acres by the total area, 48.8 acres.

$$\frac{47.9}{48.8} = 0.98 \text{ or } 98\%$$

Since the percentage is greater than 80, this area will be treated as a simple area consisting of 48.8 acres of turfed GMd soil.

Step 5. Determine the representative flow paths. It is necessary to determine the average slope of the entire simple area shown in Figure 6-19. In order to do this, use representative flow paths, If the north - south running swale is imagined as the dividing line, approximately one-third of the watershed area lies to the west (left) and two-thirds to the east (right). To determine an average slope, the number of slope measurements taken on the western slope should be balanced by twice that number taken from the east. Figure 6-20 reflects this guideline, showing three slope measurements, two on the east and one on the west.

NOTE: The selection of representative flow paths is a judgment call based on the best evaluation of the topographic features.



Path 1 $\dfrac{96\text{ft} - 52.5\text{ ft}}{1{,}015\text{ ft} + 575\text{ ft}} = 2.7\%$ slope average

Path 2 $\dfrac{95.1\text{ ft} - 52.5\text{ ft}}{640\text{ ft} + 1{,}285\text{ ft}} = 2.3\%$ slope average

Path 3 $\dfrac{95.1\text{ ft} - 52.5\text{ ft}}{575\text{ ft} + 650\text{ ft}} = 3.6\%$ slope average

Figure 6-20. Determining average slope

Step 6. Divide flow paths into two sections: sheet and ditch flow. Generally, overland flow concentrates into natural ditches after 500 feet. Path 1 changes to ditch flow in the valley where it slopes down. See section A of Figure 6-20, page 6-31. The same is true for path 2. By looking at the contour lines near path 3, we can see they are relatively flat. Flow occurs across a wide area, and no clear point can be seen where the flow changes. A good estimate would be at 575 feet.

Step 7. Determine the slope of each flow path. The representative flow paths selected in an earlier step, if properly selected, can provide very good slope information with a minimum of effort. Other slope lines may be selected for practice and to gain confidence in using this procedure. Remember to maintain the 2-to-1 balance in finding slopes in this basin and to delete redundant information. The slope must be measured over a path that water would actually follow as it flows downhill. Normally, all work would be performed on one consolidated map. For instance, to determine the slope for path 1A—

$$S = \frac{96 \text{ ft} - 62 \text{ ft}}{575 \text{ ft}} = 0.059 \text{ or } 5.9\%$$

This procedure is repeated for every flow path illustrated in Figure 6-20.

A tabular solution is recommended to determine TOC.

Path	Cover	Length (ft)	Slope (%)	Time (min)
1A	Average turf	575	5.9	14.5
1B	Ditch section	1,015	09 (135 fpm)	7.5
				22.0
2A	Average turf	640	5.2	16.5
2B	Ditch section	1,285	0.8 (130 fpm)	9.9
				26.4
3A	Average turf	575	5.0	16,0
3B	Ditch section	650	2.3 (215 fpm)	3.02
				19.02

The slope of the original path 3 is unchanged, remaining at 3.6 percent. Paths 2A and 2B are now one single path, 2, with an average slope of 2.3 percent. (The average of 5.2 percent and 0.8 percent is not 2.3 percent.) Redetermine the overall slope (as done earlier). The earlier path 1B has been deleted, leaving only the original path 1A (with S = 5.9 percent) as the new path 1. The reason for deleting 1B is that it provides the same information already provided by the new path 2. Path 2B could have been deleted instead of path 1B with no change to the final result.

Step 8. Determine the average slope of the basin or subarea based on flow paths. With the three flow paths now determined, the average slope of the simple area is—

$$\frac{5.9\% + 2.3\% + 3.6\%}{3} = 3.93\%$$

Step 9. Find the C value of the basin or subarea based on soil or cover type and slope from Table 6-1, page 6-22. Since we know that the average slope is 3.93 percent, we can use Table 6-1. Using the column marked slope >2% and <7% with turf, we have a C value of 0.35.

Step 10. Determine the travel time of each flow path and select the longest flow-path travel time as the basin TOC. Times for paths 1B and 2B were obtained by dividing their flow lengths by approximate velocities obtained from Table 6-2, page 6-27. The travel times for each of the complete flow paths (22.0, 26.4, and 19,02 minutes, respectively) are obtained from Figure 6-18, page 6-26. The variation between the smallest and largest time, although not small, is not excessively large, either. Perhaps path 1 is not representative and some ditch flow occurs along path 3 that could not be determined from the topographic information available. Both of these possibilities are likely to be true. However, without an actual field investigation to justify revising either path 1 or 3, accept the travel times already determined and select the largest as the basin TOC. Thus, TOC = 26.4 minutes.

Step 11. Determine the I value for a 2-year, 60-minute storm. To determine factor I, a source of rainfall data is necessary. The choice is between using pinpoint data (the most accurate means of determining 1) or referring to an isohyetal map. Since pinpoint data is not available, use an isohyetal map. Refer to the isohyetal map in Figure 6-3, page 6-7. Knowing that the airfield is located near the demilitarized zone (DMZ) in Korea, as shown in Figure 6-3, rainfall intensity of the 1-hour, 2-year storm is determined to be 2.5 inch per hour or—

$$I_{60,\ 2\ yr} = 2.5\ in/hr$$

Step 12. Adjust the I value. To determine I, $I_{60 min,\ 2\ yr}$ must be adjusted so that its duration is equal to the basin TOC, 26.4 minutes. Use the set of standard intensity-duration curves in Figure 6-4, page 6-9, to make the adjustment. Using curve 5 (for 2.5 inches per hour) and sliding along until the 26.4-minute imaginary vertical line is in tersected, the intensity (adjusted to 26.4 minutes) is found to be 4.2 inches per hour; thus--

$$I_{adj} = I_{26.4\ min,\ 2\ yr} = 4.2\ in/hr$$

Step 13. Calculate Q using C from step 9, I_{adj} from step 12, and A from step 3.

All the variables have been determined to solve the equation $Q = CIA$, as follows:

C = 0.35
I = 4.2 in/hr
A = 48.8 acres
Q = 0.35 x 4.2 in/hr x 48.8 acres
Q = 71.74 cfs or 71.7 cfs

The determination of Q is the final solution to the example.

If area 5 had been a complex area, steps 1, 2, 3, and 5 would be unchanged. The only difference would occur in step 4, which would be changed as follows:

Step 4 (for a complex area). An S_{avg} for **each** soil or cover area must be determined (except for manufactured covers). A C value for **each** soil or cover must be determined based on the average slope for each cover area. Once all C corrections are made, then an area-weighted C or C_{wtd} can be determined. C_{wtd} would be used in solving $Q = CIA$.

Successive Areas

Up to this point, the drainage areas discussed have been single, independent areas, whether simple or complex, These independent areas do not receive runoff from an upstream area. Some drainage systems, however, consist of a series of drainage areas with upstream areas discharging runoff into lower areas. The areas receiving this runoff are called dependent areas. The runoff accumulates and increases in its passage through the system.

Sometimes, two or more areas discharge runoff into the same dependent downstream area. Such contributing areas arc called parallel areas.

Unfortunately, the increase in runoff is not the simple summation of the peak runoff of each individual area. The individual peak flows arc acted upon by various factors, including storage and peak-tlow reduction while in the drainage network, Also, the peak flow from upstream areas and the peak flow from downstream, dependent areas will probably not arrive at the lower outlet simultaneously. Hence, the total peak flow must be less than the total of the individual peak flows.

Calculating TOC

Because of the accumulation of peak flow in successive areas, calculation of TOC for those areas must be different from the method used for single, independent areas. To estimate the amount of the accumulated runoff with some precision, a procedure has been developed to recalculate TOC for each of the successive drainage areas as water travels downstream. Naturally, as TOC increases, rainfall intensity, I, decreases.

The term *TOC* must be modified to reflect calculation differences. Consider two areas, an upstream area W and a downstream area X, as shown in Figure 6-21. The maximum travel time from the most hydraulically remote represen [alive points in this series of two areas to the outlet of area X is defined as TOC. The maximum representative flow times for runoff **originat-ing** in both areas (X and W) to arrive al [heir respective outlets (X and W) arc defined as inlet time X and inlet time W, respectively.

The TOC at teh outlet upstream in area W is given as TOC_W, which equals inlet time W for this independent area. The' ditch *time (DT)* or transit time through area X, from outlet W to outlet X, is DT_{W-X}. The total of these two elements (TOC_W and DT_{W-X}) is inlet $time_{W-X}$.

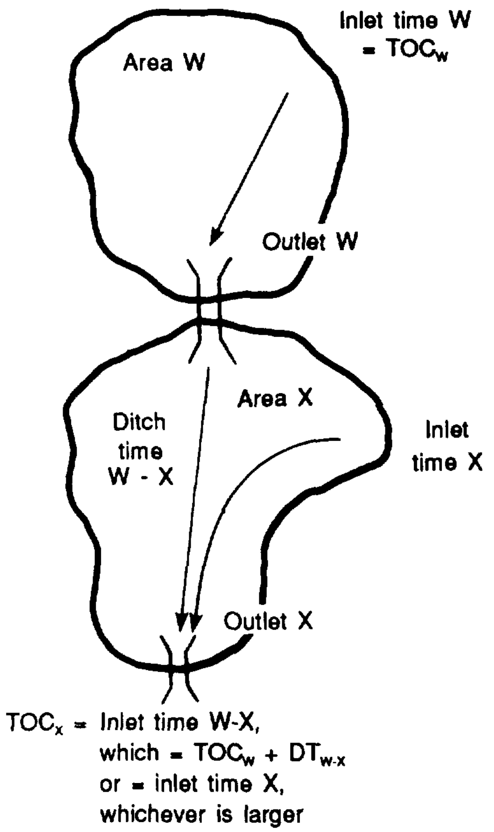


Figure 6-21. Schematic example for successive areas

To a designer engaged in sizing the culvert which serves as outlet W, the TOC would simply be TOC_W = *Inlet Time*$_W$. However, when sizing culverts that occur further down in successive areas (for instance, the culvert at outlet X), the designer requires the time it takes water to arrive from the most hydraulically remote location, which might be in either area X or area W. To determine this maximum representative time (TOC_X) the designer must compare the travel times for runoff origination in both areas.

The travel time for waler originating in area W and arriving at outlet X is equal 10 the inlet lime at area W (which is the same as TOC_X, since area W is dependent) plus the transit time as the water flows in a ditch through area X. This composite time, called inlet $time_{W-X}$, must be compared to the time for water that originates in area X, or inlet $time_X$. TOC_X, the maximum repre-sentative time for water to arrive at outlet X, is the larger time value identified in the comparison.

Estimating Successive Area Runoff

The modified definition of TOC is applied in estimating runoff for successive areas. This calculation requires collection of certain es-sential data and application of the prin-ciples used for single areas, appropriately modified, to determine the desired Q.

Preparatory Work.

Step 1. Delineate every subarea in the series of areas.

Step 2. Determine the intensity of the 2-year, 60-minute rainfall.

Step 3. Determine A, C, and inlet time. For each subarea, deter,ome the acreage (A); variable C, corrected for slope and weighted (as necessary); and the inlet time.

Step 4. Determine the DT (or transit time).

For each dependent subarea, the DT is determined from the upper outlet 10 the lower outlet using culvert flow (assume 300 fpm, unless you have more precise data).

Determining Specific Q, Working systematically, start at the uppermost subarea and proceed downstream. Refer to Figure 6-21 and note that subarea W is upstream from subarea X.

Step 1. Determine the subarea TOC using these simple rules:

The rule for an independent subarea is—

$$TOC_w = Inlet\ time_w$$

The rule for a dependent area is—

$$TOC_w + DT_{w\text{-}x}$$

Compare inlet $time_x$ with inlet $time_{w\text{-}x}$. Select the larger value of TOC_x, based on the comparison.

Step 2. Adjust $I_{60min,\ 2yr}$ to $I_{TOC,\ 2yr}$.

Step 3. Calculate Q for subarea W.

$$Q_w = C_w I_{TOCx} A_w$$

Step 4. Proceed downstream to subarea X and repeat steps 1, 2, and 3.

$$Q_x = C_x I_{TOCx} A_w$$

Step 5. Total the accumulated runoff, To get the total runoff at the Outlet of subarea X, use the following equation:

$$Q_{outlet\ x} = Q_x + Q_w$$

Step 6. Continue working downstream. Proceed until the runoff at the lowest outlet in the series is calculated. Use the drainage basin's corresponding rainfall in - tensity for the I value; use the total area for all basins and the subareas for the area. A value, in the rational-method formula.

$$Q = CIA$$

NOTE: Remember that when using the rational method, the area limit is 1,000 acres. Always check the accumulated acreage to ensure that it does not exceed 1,000 acres.

Example:

Using the rational method and Figure 6-22, page 6-36, determine the runoff, in cfs, expected at culverts 3 and 4 at the Span II Army Airfield at Giessen, Germany. The soil type is GMd.

Solution:

Step 1. Delineate the area; it is found to have the following:

Subarea A.

Compacted gravel	6.2 acres	Simple
Average turf	0.2 acre	Area
	6.4 acres	

Subarea B.

Compacted gravel	0.9 acre	Simple
Average turf	12.5 acres	Area
	13.4 acres	

Step 2. Delineate subareas by soil or cover type. (See step l.)

Step 3. Determine acreage for each basin or subarea. (This information is given in step 1,)

Step 4. Classify the basin as simple or complex.

Subarea A.

Compacted gravel	6.2 acres	
Average turf	0.2 acre	Simple
Total	6.4 acres	

6.2 acres/6.4 acres = 0.97 or 97%

Subarea B.

Compacted gravel	0.9 acre	
Average turf	12.5 acres	Simple
Total	13.4 acres	

12.5 acres/13.4 acres = 0.93 or 93%

Step 5. Determine representative flow paths. See Table 6-3, page 6-37 (given information).

Step 6. Divide flow paths into two sections: sheet flow and channel flow. See Table 6-3 (given information).

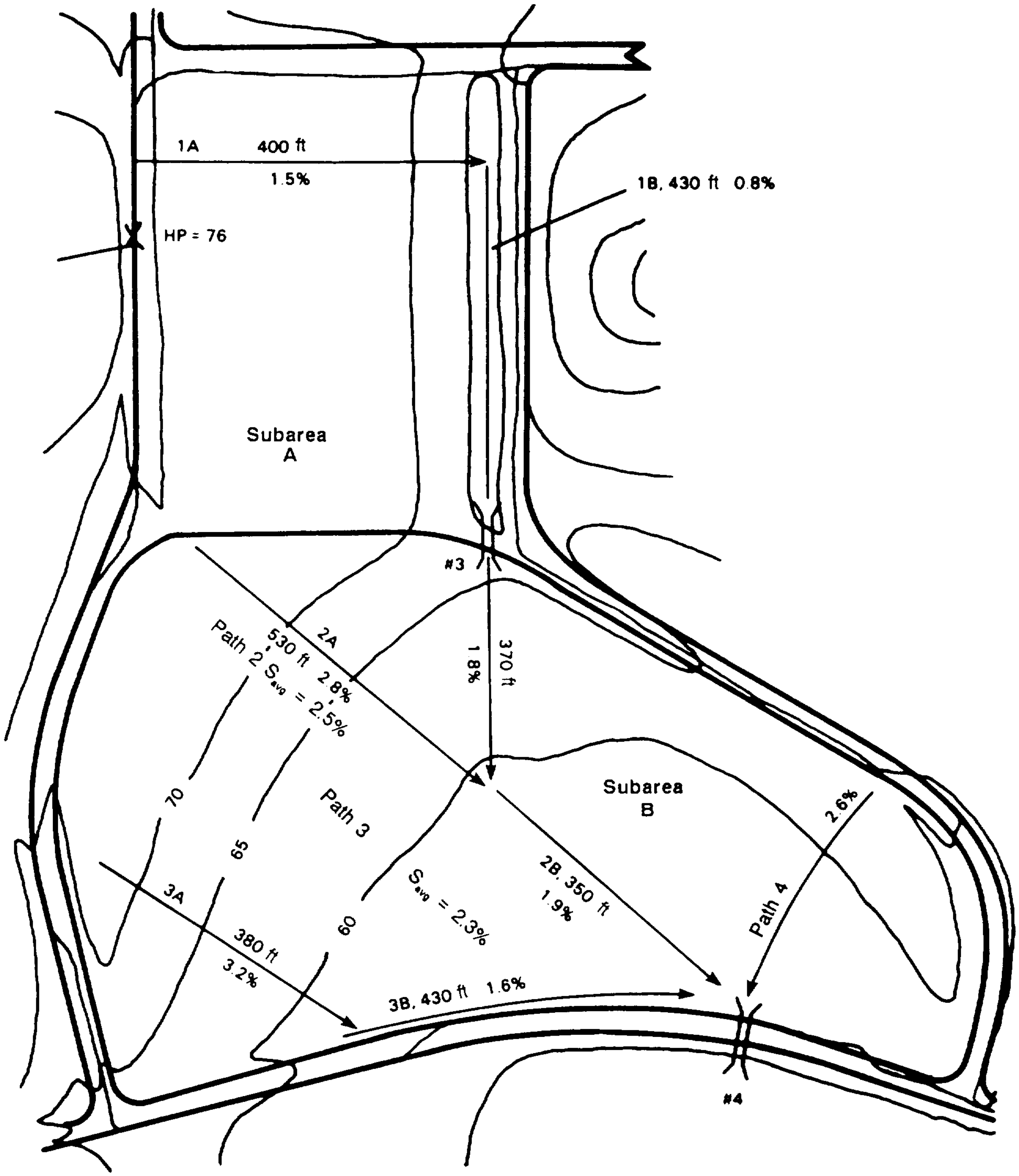


Figure 6-22. Successive areas - example

Step 7. Determine the slope of each section of the flow path.

Step 8. Determine the average slope of the basin or each subarea, There are two paths in subarea 1, paths 1A and 1B. Both of these paths will have an average slope of less than 2 percent. Subarea 2 has three major paths. Path 2 has an average slope of 2.3 percent, path 3 has an average slope of 2.4 percent, and path 4 has an average slope of 2.6 percent. (See

Table 6-3. Determining travel time

Path	Cover/type	Length (ft)	Slope (%)	Travel time (min)	
1A	Compacted gravel	400	1.5	13.0	
1B	Ditch	430	0.8	3.3	at 130 fpm
				16.3	
2A	Average turf	530	2.8	23.5	
2B	Ditch	350	1.9	1.8	at 130 fpm
				25.3	
3A	Average turf	380	3.2	18.5	
3B	Ditch	430	1.6	2.4	at 130 fpm
				20.9	

Figure 6-22.) Using this information, we can now get an average slope for paths 2, 3, and 4.

$$\text{Average Slope} = \frac{2.4\% + 2.3\% + 2.6\%}{3}$$

2.4%

Average slope for subarea 1 = < 2%
Average slope for subarea 2 = 2.4%

Step 9. Find the C value for each subarea, Subarea A has compacted gravel with an average slope less than 2 percent. Using Table 6-1, page 6-22, we find that the C value is 0.70. Likewise, the C value for subarea B, GMd with average turf with a slope of 2.4 percent, is 0.35.

Step 10. Deter mine the travel time of each flow path and select the longest flow-path travel time as the TOC. Obtain sheet-flow times from Figure 6-18, page 6-26, and ditch-flow travel time from Table 6-2, page 6-27. Determine the travel times and ditch time from Table 6-3.

$$\text{Ditch time}_{3\text{-}4} = \frac{370 \text{ ft}}{188 \text{ fpm}} + \frac{350 \text{ ft}}{194 \text{ fpm}}$$
$$= 3.8 \text{ min}$$

Ditch velocity = 188 fpm at 1.8% and 194 fpm at 1.9%

Inlet time A = 16.3 min
Inlet time B = 25.3 min

Step 11. Determine I and I_{adj}. (I value for Giessen, Germany is 1.7 in /hr from local rainfall records.)

Subarea A.

TOC_A = 16.3 min
$I_{16.3}$ = 2.6 in/hr

Subarea B.

Compare inlet $time_B$ = 25.3 min with TOC_A + DT = 16.3 + 3.8 = 20.1 min

Select the larger value of TOC_B = 25.3 min and $I_{25.3}$ = 1.8 in/hr

Step 12. Adjust I based on TOC using Figure 6-4, page 6-9. (This step was included in step 11.)

Step 13. Determine runoff.

To calculate runoff in subarea A, use—

$$Q_A = C_A I_A A_A$$
$$= 0.70(2.6 \text{ in/hr})(6.4 \text{ acres})$$
$$= 11.6 \text{ cfs}$$

To calculate runoff in subarea B, use—

$$Q_B = C_B I_B A_A$$
$$= 0.35(1.8 \text{ in/hr})(13.4 \text{ acres})$$
$$= 8.4 \text{ cfs}$$

$Q_{inlet\ 3}$ = 11.6 cfs
$Q_{inlet\ 4}$ = 11.6 + 8.4 = 20 cfs

NOTE: Although path 4 was used to obtain an accurate slope average, it is not used for travel time. The situation, as drawn on the map, clearly shows that path 4 could not be chosen for the TOC.

SECTION II. OPEN-CHANNEL DESIGN

An open channel is a conduit with a free-water surface used to convey water. The most common is a ditch, which is an open channel cut into the soil. If so desired, the ditch can be lined along the bottom from bank to bank.

The size, the shape, the method of construction, and the location of a ditch are determined largely by its purpose. These factors, once determined, will influence the design capacity and maintenance requirements.

DESIGN FACTORS

LOCATION

There are three main types of ditches used in road and airfield construction.

An *interceptor ditch* is generally located on a hillside above a roadway or other feature requiring protection. Its function is to intercept runoff and direct the flow to a more desirable location. It is usually located above sidehill cuts to prevent erosion of the cut.

A side ditch is located along the side of a road. It collects runoff from the road and adjacent areas and transports it to a culvert or diversion ditch.

When the Topography allows, a *diversion ditch* is built to transport water away from roadways or airfields. It can be used in conjunction with interceptor and side ditches to transport water between culverts or to divert an existing stream channel around a project.

CROSS SECTION

The location and peak quantity of runoff expected will determine the ditch cross-sectional area required. The most common shapes of cross sections—triangular (symmetrical and nonsymmetrical), trapezoidal, and segmental-are shown in Figure 6-23.

In the TO, the shape of a ditch is also dictated, to a great extent, by the choice of engineer equipment available for its construction. Two items of equipment are uniquely suited for speedy ditch excavation: the motor grader and the wheeled tractor-scraper. Other items of equipment that can be used to excavate a ditch section include the backhoe; bulldozer; front-end loader; trenching machine; and crane equipped with a dragline, clamshell, or shovel front.

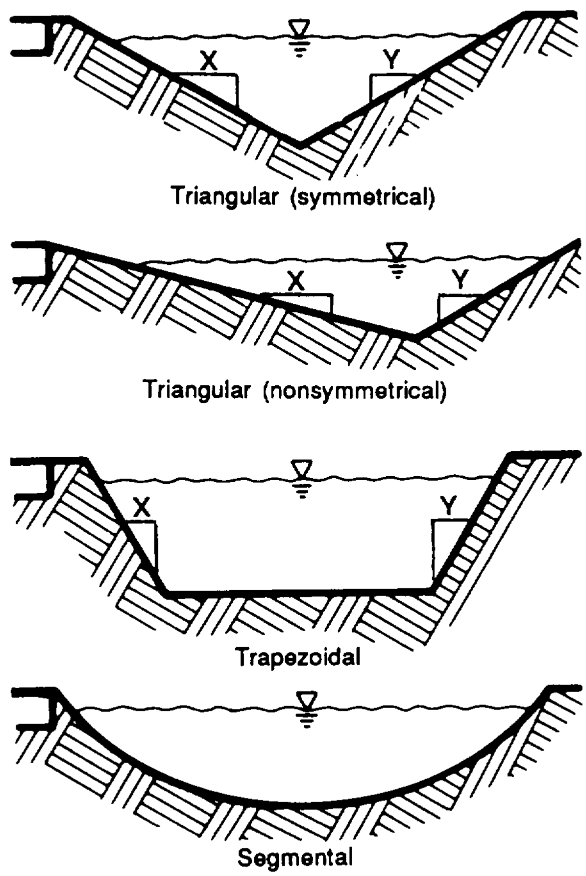


Figure 6-23. Ditch cross sections

However, production rates for these items are relatively low compared to the grader or scraper; hence, the grader or scraper is more likely to be used.

Triangular or V ditches are commonly installed to handle flows up to 60 cubic feet per second. The road grader is well designed to quickly excavate the necessary cross section to handle this flow, provided that the ditch is built in soil rather than rock. Grader efficiency drops significantly when cross sections of larger dimensions are required.

For flows larger than 60 cubic feet per second, the *trapezoidal ditch* is commonly specified. The flat bottom and midsection of this ditch can be excavated rapidly by a wheeled scraper, and the side slopes can be dressed back by subsequent passes of a road grader.

Smaller bottom widths can be provided using any of the previously mentioned items of construction equipment, including the road grader with its blade turned to a high angle. Production rates, however, will be much lower than those of the wheeled scraper.

Note that the 60 cubic feet per second guideline is flexible. If scrapers are not available to excavate a ditch carrying 100 cubic feet per second but a grader is, common sense dictates that the grader be used to construct an oversized V ditch rather than using low-production-rate equipment to construct a trapezoidal ditch,

The *segmental-ditch* shape results when explosives arc used to create the ditch. This technique is often used when the terrain is too soft to support excavating math inery. Ditches cut by hand will often bear this shape as well.

SIDE-SLOPE RATIOS

Ditches have two sides and two associated side-slope ratios. Side slope is the slope of the banks of the channels, normally expressed as a ratio of feet horizontal to feet vertical. For example, 3:1 is a side slope of 3 feel horizontal to 1 foot vertical. When the sidewalls on opposite sides are inclined equally, the ditch is called symmetrical. *Nonsymmetrical* ditches have side slopes that differ.

The designer selects appropriate side-slope ratios. The selection is critical to ensure that the ditch serves its purpose. Ditch sidewalls that are too steep invite excessive erosion and are likely to cause the ditch to clog with sediment. Even more serious is the risk of a severe accident, if a vehicle should run into the ditch and become entrapped or overturn because the side slope is too severe. Only one side slope is required for symmetrical ditches. For clarity, the terms *front slope* or *ditch slope* and *back slope* are used to differentiate between the dissimilar slopes. Figure 6-24, page 6-40, illustrates this terminology.

The sidewall of a roadside ditch located adjacent to the shoulder is called the front slope of the ditch. The far slope, called the back slope, is simply an extension of the cut face in an excavation. The following rules of thumb are applicable **only in shallow ditches in relatively flat terrain:**

- Roadside ditches may be cut nonsymmetrically at 3:1/1:1 (front slope/back slope).

NOTE: For calculation purposes, the horizontal component of the roadside ditch will be referred to as X. Likewise, the horizontal component on the back slope will be referred to as Y. (See Figure 6-23.)

- Diversion ditches may be cut symmetrically at 1:1.
- Ditches intended to be subject to cross traffic may be cut symmetrically at 3:1 or more gently.

In most cases, interceptor and diversion ditches are installed far enough from the traveled way not to present a hazard to passing vehicles on the roadway or aircraft on the runway. Since there is very little danger from either side, symmetrical side-slope ratios are specified for these types of ditches.

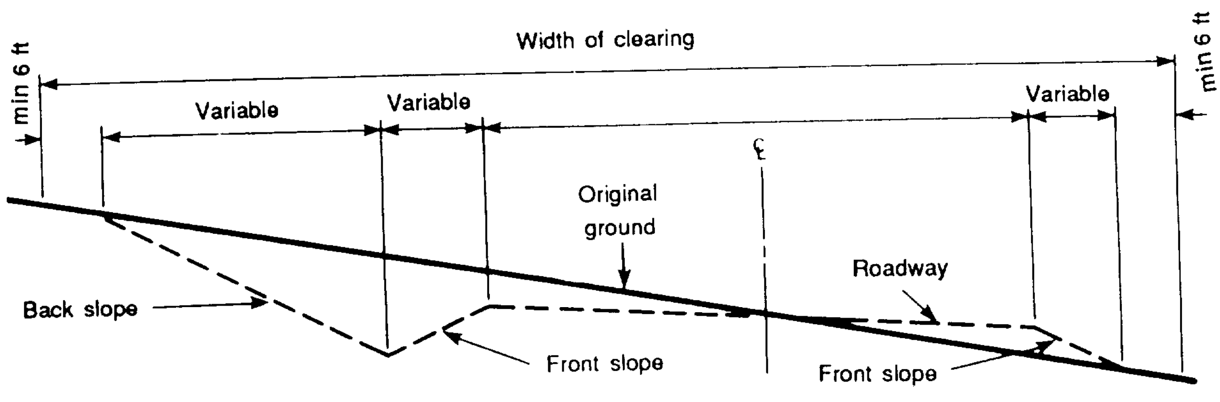


Figure 6-24. Definition sketch

Tables 6-4 and 6-5 are also useful in selecting front slopes for fill sections.

TYPES OF FLOW

Several types of flow are associated with open channels. Some of these types occur simultaneously in the same channel. An understanding of these types of flow and their interrelationship is essential to the effective design of drainage systems.

An *open-channel flow* has a free surface and no hydraulic pressure. Some examples of open-channel flow include ditches, canals, streams, and culverts not flowing full. Empirical formulas with experimentally derived coefficients are used in designing an open channel. These hydraulic formulas reflect certain hydraulic theories and assumptions governing design analysis of free-flow channels.

A *steady flow* is assumed in an open channel with a uniform depth during the design period. Changes in flow are generally slow, and any errors that may be introduced by this assumption are not significant.

A continuous flow is assumed according to the principle of the conservation of mass.

A *uniform* flow is assumed when the depth of water throughout a channel is constant in dimension and slope. This means that the slope of the water surface is the same as the slope of the channel bottom. This assumption is essentially correct for channels of moderate slope and length. Flows in ditches, canals, and rivers are uniform, but flows over spillways or waterfalls are not uniform.

The strength of viscosity forces and hence the thickness of moving fluid, determine whether channel flow is *turbulent or laminar.*

Turbulent flow is assumed for purposes of open-channel design. This type of flow occurs when viscosity forces are relatively weak and the individual water particles move in random patterns within the aggregate forward-flow pattern.

Laminar flow occurs when viscosity forces predominate and the particles of the fluid move in smooth, parallel paths. An example of this type of flow is honey poured from a container; honey has high-viscosity strength compared to water. The only type of laminar flow considered in this manual is sheet flow, which occurs where depth is extremely shallow. It is assumed that the flow

Table 6-4. Recommended requirements for slope ratios in cuts and fills - homogeneous soils

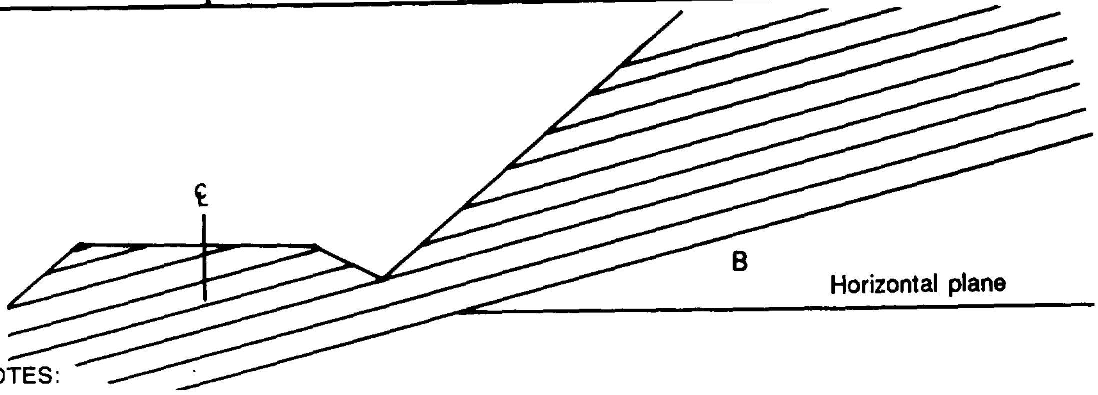
USCS Classification	Slopes Not Subject to Saturation		Slopes Subject to Saturation	
	Maximum Height of Earth Face	Maximum Slope Ratio	Maximum Height of Earth Face	Maximum Slope Ratio
GW, GP, GMd SW, SP, SMd	Not critical	1½:1	Not critical	2:1
GMu, GC SMu, SC ML, MH CL, CH	Less than 50 feet	2:1	Less than 50 feet	3:1
OL, OH, Pt	Generally not suitable for construction			

NOTES: 1. Recommended slopes are valid only in homogeneous soils that have either an in-place or compacted density equaling or exceeding 95% CE 55 maximum dry density. For nonhomogeneous soils, or soils at lower densities, a deliberate slope stability analysis is required.

2. Back slopes cut into loose soil will seek to maintain a near-vertical cleavage. Do not apply loading above this cut face. Expect sloughing to occur.

Table 6-5. Recommended requirements for slope ratios in cuts - rock with bedding or other planes of weakness

Effective dip angle (degree) B	0 - 18	18 - 35	35 - 75	75 - 90 or any negative angle
Maximum side slope ratio (H:V)	1:2	1:1	Cut to dip angle	1:4 to 1:2



NOTES:

1. Solid rock (with no plane of weakness) may be cut 1:4 to 1:2.
2. Effective dip angle = true dip - apparent dip where--
   a. True dip is the angle between the horizontal and bedding planes, measured perpendicular to strike.
   b. Apparent dip equals:

$$\frac{\text{angle between centerline and strike}}{90^\circ} \times \text{true dip}$$

in natural and designed channels will be steady, continuous, uniform, and turbulent.

DESIGN EQUATIONS FOR OPEN CHANNELS

This section deals with open-channel design equations. Because of the variables and assumptions to be made, trial techniques are required to determine the shape and depth of a particular channel before a final solution is reached.

Continuity Equation

The equation of continuity is expressed as follows:

$$Q = AV$$

where—

Q = rate of flow in cfs
A = cross-sectional area in sq ft
V = velocity in fps

Manning's Velocity of Flow Equation

Many empirical equations have been proposed for determining turbulent flow. The most widely used is the equation presented by Manning in 1889. It states—

$$V = \frac{1.486\, R^{2/3}\, S^{1/2}}{n}$$

or, after transposing,

$$R = \left[\frac{Vn}{1.486\sqrt{S}}\right]^{3/2}$$

where—

V = velocity of flow in fps
R = hydraulic radius
or

$$R = \frac{\text{cross-section area of water}}{\text{wetted perimeter}}$$

S = slope or grade of the channel in feet per foot (ft/ft)
n = roughness coefficient or friction factor, which depends upon the material comprising the channel lining

NOTE: When using metric units—

$$V = \frac{R^{2/3}\, S^{1/2}}{n}$$

Manning's equation can easily be solved mathematically using the equation. However, to assist in the design of open channels, the equation has been prepared as a nomograph. (See Figure 6-25.)

Roughness Coefficient (n)

The roughness or resistance coefficient is a measure of the resistance to flow caused by surface-contact irregularities. It varies with soil type, channel condition, and type of ditch lining used. Use Table 6-6 to estimate the roughness coefficient, n, used in the solution of the equation. The coefficient can be changed only if the ditch lining is changed or modified. The effect of the roughness coefficient on velocity can be altered by changing the side slopes of the ditch, thereby changing the water con tact area. Changing the roughness coefficient in this way changes the ditch capacity.

Longitudinal Slope or Grade (S)

Under normal conditions, the ditch slope (the longitudinal fall of the channel in feet per foot or in percentage) will be determined by the slope of the terrain. For short ditch lengths only, a variation from the natural slope of the terrain can be achieved by modifying the cutting depth of the ditch. By varying the cutting depth within the ditch length, the slope can be increased or decreased independently of the terrain slope.

In channel design, slope percentage and the resulting change in velocity are important considerations. Slopes over 2 percent may have too high a velocity, resulting in erosion. Slopes under 0.5 percent will generally have too low a velocity, resulting in sedimentation deposits. Deposition (the depositing of sediment on the bottom of the ditch) normally occurs at velocities below 3 feet per second.

Velocity of Flow (V)

Many ditches with differing side slopes and cross-sectional areas of flow will carry the same rate of runoff on a similar longitudinal slope. In each case, however, the velocity of flow will be different. Since excessive velocity in a ditch will cause erosion and

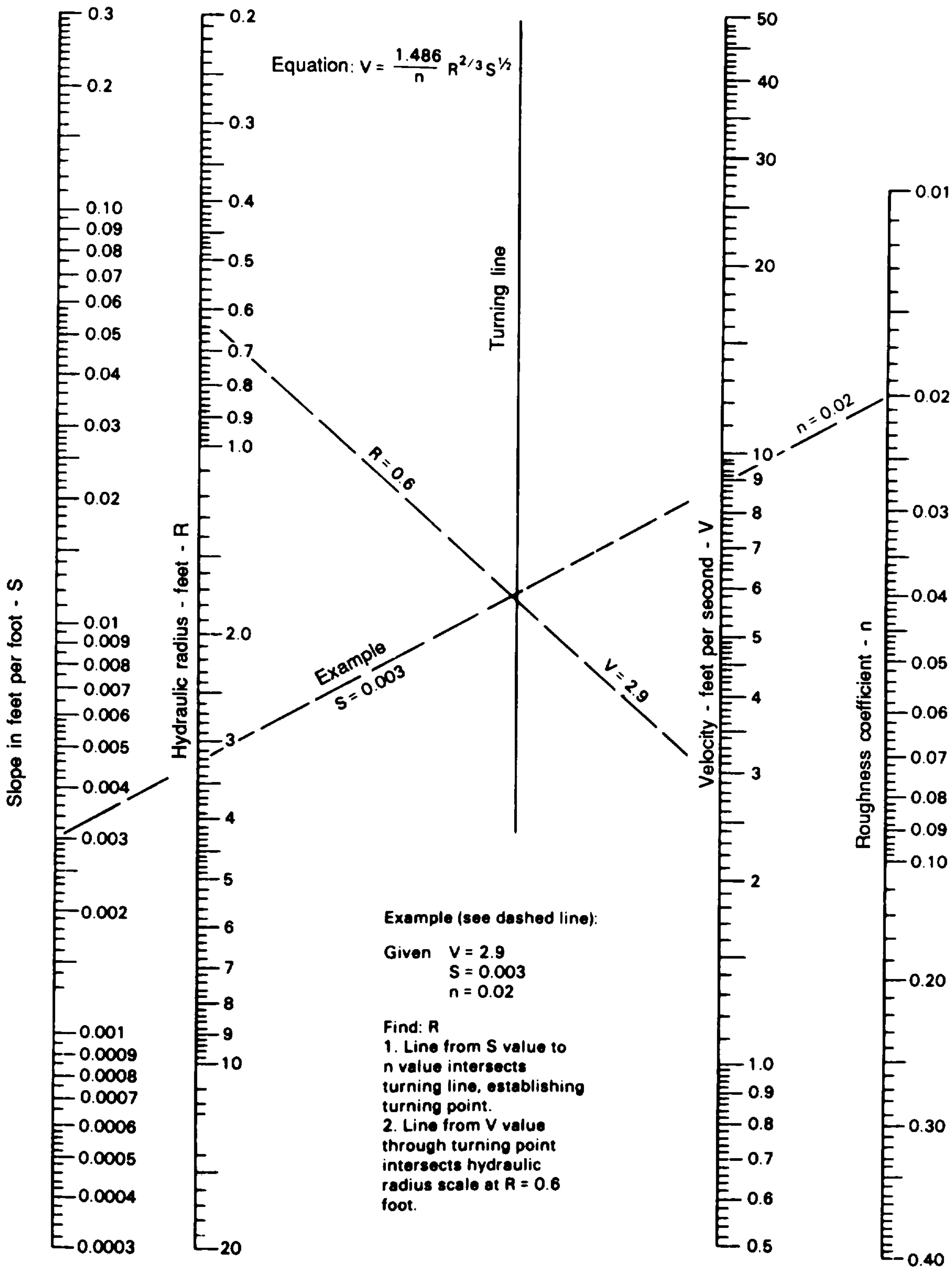


Figure 6-25. Nomograph for Manning's equation

Table 6-6. Values for Manning's n and maximum permissible velocities of flow in open channels

Ditch Lining	Manning's n	V_{max} (fps)
1. Natural earth:		
a. Rock: smooth and uniform	0.038	20
jagged and irregular	0.043	18
b. Soils: GW	0.023	7
GP	0.025	8
GMd	0.024	5
GMu, MH	0.023	4
GC	0.025	7
SW, SP	0.023	2
SMd, SMu	0.022	3
SC, ML	0.024	4
CL, CH, OL, OH, PT	0.023	3
2. Paved surfaces:		
a. Concrete (all surfaces)	0.014	20
b. Concrete bottom w/sides of--		
dressed stone in mortar	0.016	20
random stone in mortar	0.018	19
dressed stone or smooth concrete rubble (riprap)	0.023	15
rubble or random stone	0.027	15
c. Gravel bottom, w/sides of--		
formed concrete	0.018	10
random stone in mortar	0.022	10
random stone or rubble	0.028	10
d. Brick	0.016	10
e. Asphalt	0.015	20

NOTE: To calculate $V_{assumed}$, take V_{max} and subtract 1. For example, $V_{assumed}$ for a GP soil is 8 - 1 or 7 fps.

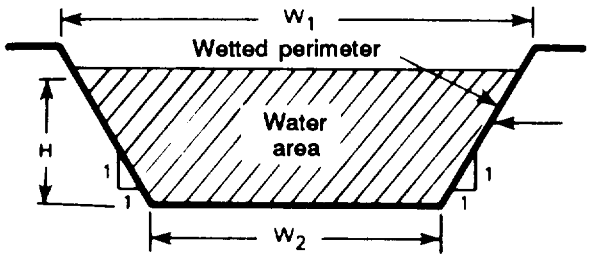
possibly damage adjacent structures, it must be contained within limits. Table 6-6 lists the maximum permissible velocities, depending on soil and other factors.

Velocity Relationships

- As slope increases, velocity (V) increases.
- As quantity of runoff (Q) increases while area (A) remains constant, velocity (V) increases.
- If Manning's n increases, velocity (V) decreases.
- If velocity (V) increases, erosion increases.

Hydraulic Radius (R]

The hydraulic radius (R) is the area of the water cross section of the ditch divided by its wetted perimeter, calculated as shown in Figure 6-26. It relates the surface area of friction resistances with the volume of water being carried by the ditch. The hydraulic radius can be calculated using an electronic calculator as shown in Figure 6-27, page 6-46.



Example.

Assume. $H = 2$ ft, $W_1 = 14$ ft, $W_2 = 10$ ft
Find. hydraulic radius (R)
$A = \frac{1}{2}(H)(W_1 + W_2) = \frac{1}{2}(2)(14 + 10)$
$= 24$ sq ft
In a 1:1 triangle, hypotenuse $= \sqrt{2}$
Wetted perimeter $= 2\sqrt{2} + 10 + 2\sqrt{2} = 15.7$ ft

$$\frac{\text{Hydraulic radius area}}{\text{Wetted perimeter}} = \frac{24}{15.7} = 1.53 \text{ ft}$$

Figure 6-26. Hydraulic radius

DESIGN CONSIDERATIONS

There are certain factors known for each ditch being designed. Each of these factors will affect design details. Items such as the location, the peak flow or runoff carried, the effect of terrain on slope, and the soil type or material to be used to line the ditch all have a bearing on channel design.

LOCATION

The location of the ditch will determine its general shape and the side slopes to be used in its design.

QUANTITY OF RUNOFF (Q)

Designers must know the quantity of runoff the channel will have to carry. Usually, this is estimated using the rational method of runoff determination. However, it may also be estimated based on knowledge of the slope, the diameter, and the type of culverts used to discharge into the channel. The value of Q will also determine which type of ditch section, triangular or trapezoidal, will be used. This depends on whether Q is greater than or less than 60 cubic feet per second.

SLOPE (S)

The slope will be determined by the terrain. In general, the slope used will be the natural ground slope. Small modifications of the slope can be made for short ditch sections.

PROPOSED DITCH LINING

The ditch lining determines the velocity and roughness coefficient or resistance factor, n. (See Table 6-6.)

Table 6-6 gives maximum erosion velocities for each type of soil and lining. The lower velocity on the chart indicates the velocity at which erosion will start occurring in some portion of the ditch. At the high velocity value, the entire length of the ditch will probably be eroding.

Table 6-6 also provides Manning's roughness coefficient (n) which represents the friction resistance of the ditch, channel, or stream for various soil types and linings. Use the average value of n for design purposes.

1. If your calculator has the function $\sqrt{x}$ and y^x you can determine hydraulic radius (R) more quickly and accurately than with the nomograph provided in the workbook. Transposing Manning's equation,

$$V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n} \quad \rightarrow \quad R = \left(\frac{V \times n}{1.486\ \sqrt{S}}\right)^{1.5}$$

Since n and S are constant for various trial iterations, you need only enter different values for V to arrive at an acceptable value for R. Therefore, if S and n are constant, the above equation for R becomes:

$$R = [V \times K]^{1.5} \text{ where } K = \frac{n}{1.486\ \sqrt{S}}$$

2. To calculate R, use the above equation for numerous iterations until you have "bracketed" flow rate, Q, within acceptable limits. Below are keystrokes which will work on most calculators.

a. First, input and store values for K.

.xxx	Enter value for S
$\sqrt{x}$	Square root of S
x 1.486	Times constant of 1.486
= 1/x	Calculate and invert denominator
x .xxx	Times value for n
= STO or M+	Store constant K

b. To calculate R for various values of V,

RCL or MRC	Recall K
x V	Times value for velocity
=	Equals
y^x 1.5	Raise product to 3/2 power
=	Displays value for R

c. Repeat 2b for different V values until you reach an acceptable R and Q.

Figure 6-27. Calculating hydraulic radius

DESIGN TECHNIQUES

Once design considerations have been examined, the interactive design procedure can begin.

DESIGN STEPS

The steps used in design follow:

Step 1. Determine the peak volume of storm-water runoff, Q. Calculate the total area(s) contributing flow to the ditch. (Use the rational method.) Using the appropriate formula, find Q.

Step 2. Determine the slope, S, in feet per foot. If the slope is already known as a percentage, it may be converted to units of feet per foot by simply dividing by 100.

Step 3. Select trial values for resistance, n, and velocity, V. From Table 6-6, page 6-44, select a value of the resistance or roughness coefficient, n, and a velocity, V, for the soil type in which the ditch is to be constructed.

The initial trial velocity should be held to 1. feet per second below the high value. Usually, the channel will be carrying less than design flow, reducing the velocity and making sedimentation likely. If a high value is chosen for the design velocity, this deposited material will be removed by the water during a peak flow without causing extensive damage to the channel.

Step 4. Determine the hydraulic radius. From the slope, S; Manning's n; and the velocity, V; find the hydraulic radius of the ditch, using the nomograph or equation.

Call this R_m to distinguish it from the R values in Appendix C of this manual, which will be R_t.

Step 5. Determine the type of ditch cross section. Where Q is greater than 60 cubic feet per second, use a trapezoidal ditch. Where Q is equal to or less than 60 cubic feet per second, use a triangular ditch. Whether the ditch is symmetrical or nonsymmetrical will depend on the specific location.

Step 6. Select the appropriate hydraulic radius and area table, From Appendix C, select the appropriate hydraulic radius and area table for the desired ditch cross section. Identify the column headed with the tentative side-slope ratios. Enter the R_m table, locating the value of R_t that corresponds with R_m. Then find the cross-sectional area and ditch depth corresponding to R_m and R_t. In using the tables, if the exact R_m value is not available, use the next smaller R_t value listed in that column.

Step 7. Calculate Q. Use the equation $Q = AWV$, where area, A_w, and velocity, V, are determined in steps 6 and 3, respectively. If the calculated Q from step 7 is not more than 5 percent greater than the design Q, the ditch selected can be used. If the calculated Q is more than 5 percent greater than the design Q, reduce the velocity and repeat steps 4, 5, 6, and 7.

If the calculated Q is smaller than the design Q by more than 5 percent, increase the velocity, However, do not make it any larger than the maximum for the soil or lining based on Table 6-6. If the calculated Q is still less than the 95-percent limit, the cross section must be increased by flattening the side slopes or by increasing the bottom width (if a trapezoidal section is used).

Step 8. Provide freeboard. Add 0.5 foot to the water depth to provide freeboard. Freeboard is the additional ditch depth over that required to carry the design flow. This added depth allows the ditch to carry the design capacity, even with sediment in the ditch bottom. The total depth, including freeboard, will be the cutting depth and the depth at which the ditch grade will be set.

Example:

Design a ditch to carry a peak volume of storm-water runoff equaling 47,3 cfs from a culvert to a stream that is 289 feet from the outlet of the culvert. The ditch invert (or bottom) elevation at the outlet of the culvert will be 7.0 feet **above** sea level. The ditch invert will be constructed above the stream high watermark at an elevation of 5.5 feet above sea level. The ditch lining will be the bare (unturfed) GMd soil, as excavated.

$Q = 47.3$ cfs

$$\text{Slope} = \frac{\text{invert at culvert or 7.0 ft - 5.5 ft}}{\text{distance from culvert to stream}}$$

$$= \frac{1.5 \text{ ft}}{289 \text{ ft}} = 0.0052 \text{ ft/ft}$$

Soil (GMd): V = 3 to 5 fps (Table 6-6)
= 0.024 (Table 6-3, page 6-37)

Solution (Nomograph and Table Method):

The solution of a ditch problem is always a trial technique in which several values of velocity arc used. Tables are recommended to tabulate the results. The tables should be similar to the ditch design work sheet shown in Table 6-7.

Step 1. Select a ditch. Since design flow, Q, is less than 60 cubic feet per second, the ditch should be triangular. The channel is not a roadside ditch, so it should be symmetrical. Select side slopes for a 3:1 triangular ditch for the first trial. (This assumes that periodic vehicular crossings are expected.) These factors can be changed if the ditch design is not suitable. Enter the information in the columns under ditch selection on the ditch design work sheet.

Step 2. Select the velocity. The erosion velocity for the soil is 3 to 5 feet per second. This means that at 3 feet per second the soil in the ditch may begin to erode, and at 5 feet per second the whole ditch will be eroding. Since it is preferable not to exceed the V_{max} of 5 feet per second, the best initial choice is usually 1 feet per second lower than V_{max}, or 4 feet per second. Enter this figure on the design work sheet.

Step 3. Determine the hydraulic radius, R_m, from the nomograph. Using the nomograph (Figure 6-26, page 6-45), first locate a turning line. The line is in the center on the nomograph. Find the turning point by locating the slope, S (in feet per foot), in the left column of the chart and the roughness coefficient, n, in the right column. Draw a straight line to connect the two points. The point at which the new line crosses the turning line is the turning point. This turning point will remain (he same as long as neither S nor n changes. The hydraulic radius, Rm, is found by connecting the velocity (in the second column from the right) and the turning point by drawing a straight line through to the R scale. Then read the hydraulic radius (R) in the second column from the left. This gives the required R for any given V in Manning's equation. In this example, R = 0.86. Enter this value on the work sheet.

The R value can be computed by using the calculator method shown in Figure 6-27, page 6-46.

Step 4. Find the hydraulic radius, R_t, in the ditch table. Locate the appropriate table among Tables C-2 through C-10 in Appendix C of this manual. There are four types of tables: V-triangular or symmetrical, V-triangular or nonsymmetrical, trapezoidal-symmetrical, and trapezoidal-nonsymmetrical. For this example, use Table C-2 for a symmetrical V ditch.

Locate the pair of columns representing the side slopes of the ditch being designed (for example, 3:1). Locate the R_t values that fall above and below, or the one that is exactly equal to, the R_m value found from the nomograph of Manning's equation. In this example, the R_m = 0.86 cannot be found in these columns, but the values 0.85 and 0.90 arc given, Use the lower value (0.85) on the work sheet.

Step 5. Record the area and depth. With R_t = 0.85, the corresponding area found in Table C-2 is 9.72 square feet, and the depth (d) found in the column at the far left is 1.8 feet, Record these values under the appropriate headings on the ditch design work sheet.

Table 6-7. Ditch design work sheet

Ditch selection		Assumed	Nomograph	Table, Appendix C			Calculate
Type	Side slope	V	R_m	R_t	A(sq ft)	d(ft)	Q(cfs)
V	3:1 SYM	4.5	1.0	0.95	12.0	2.0	54.0
V	3:1 SYM	4.2	0.92	0.90	10.83	1.9	45.5

Step 6. Check for Q. Check to see if this particular ditch will mcct the requirements of the design by performing the calcula - tions-

$Q = A_w V$

$Q = (9.72 \text{ sq ft})(4.0 \text{ fps})$
$Q = 38.9 \text{ cfs}$

This calculated quantity of flow (Q) must fall within 5 percent of the design flow of 47,3 cubic fect per second, or between 44.9 and 49.7 cubic feet per second. If this requirement is not met, as in this case, try a new velocity. If the calculated Q is less than 95 percent of the design flow, use a higher velocity. For this example, since 38.9 is less than 44.9 cubic feet per second (the lower limit of the acceptable range), a velocity of 4,5 feet per second would be an acceptable assumption for the next trial.

$V = 4.5$
$R_m = 1.0$
$R_t = 0.95$
$A_w = 12.0$ sq ft. d = 2.0 ft
$Q = (4.5 \text{ fps})(12.0 \text{ sq ft})$
$Q = 54.0$ cfs, which exceeds 49.7 cfs, so the ncx[trial velocity must be less than 4.5 fps: try 4.2 fps.

$V = 4.2$
$R_n = 0.92$
$R_t = 0.90$
$A_w = 10.83$ sq ft, d = 1.9 ft
$Q = (4.2 \text{ fps})(10.83 \text{ sq ft})$
$Q = 45.5 \text{ cfs}$

Q is greater than the lower limit of 44.9 and less Lhan the high limit of 49.7. This ditch is within the range and thus meets the ditch water depth and velocity requirements. Enter this value of 9 on the ditch design work sheet.

Step 7. Determine if the ditch is appropriate. This process describes a symmetrical, triangular ditch in GMd soil, with a slope of 0.0052 feet per foot and 3:1 side slopes. It carries 45.5 cubic feet per second runoff with a water depth of 1.9 feet and a velocity of 4.2 feet per second.

Since the velocity in the ditch is greater than 3 fps, the ditch can be considered self-cleaning and requires little maintenance. Peak runoff will remove any silt buildup from the channel bottom. With V = 4.2 feet per second, there may be some erosion of the ditch, but it should not be a significant maintenance problem. The shape, lining, and slopes are acceptable.

Step 8. Determine the cutting depth. The water level in the ditch should be at least 0.5 foot below the edge of the ditch as a safety factor. Accordingly, the cutting depth is the water depth plus 0.5 foot of freeboard.

Cutting depth = d + freeboard

Using the rutting depth just calculated,

Cutting depth = 1.9 + 0.5
= 2.4 ft

Since there are 2.5 feet (8.0 - 5.5 = 2.5) of available cutting depth at the end of the ditch, the design is acceptable.

Conclusion: Use this ditch.

Alterna Live Solution (Calculator Method):

If it is more convenient to use the calculator method than the nomographmo the following procedure is used:

Step 1. Select a ditch. As before, the ditch is V- type and symmetrical. Assume side slopes of 3:1 have been selected for the trial cross section.

Step 2. Select the velocity. The assumed trial velocity should always be 1 feet per second less than the maximum erosion velocity of the soil. In this case, it will be 4 feet per second.

Step 3. Determine the hydraulic radius, R_m.

$$R_m = \left[\frac{V\,n}{1.486\sqrt{S}}\right]^{3/2}$$

where—

V = 4 fps
n = 0.024
S = 0.0052 ft/ft

$$R_m = \left[\frac{4(0.024)}{1.486\sqrt{0.0052}}\right]^{3/2}$$

$$= \left[\frac{0.096}{1.486\,(0.072)}\right]^{3/2}$$

$$= \left[\frac{0.096}{0.107}\right]^{3/2} = 0.848$$

Stcp 4. Determine the area of water, A_w, and depth of water, d. As explained in Figure 6-28—

$A_w = 1/2(xd + yd)d$ for any triangular ditch.

For this problem, x = 3, y = 3

$$A_w = 1/2(3d + 3d)d$$
$$= 1/2(6d)d$$
$$= 3d^2$$

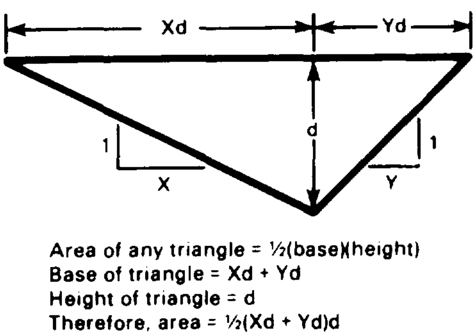


Figure 6-28. Determination of the area of a triangular ditch

The wetted perimeter (wp), as explained in Figure 6-29, is—

$$wp.\ C_1 + C_2 = (xd)^2 + d + (yd)^2 + d$$

Substituting values of x and y–

$$wp = \sqrt{(3d)^2 + d^2} + \sqrt{(3d)^2 + d^2}$$

$$= 2\sqrt{(9d^2 + d^2)}$$

$$= 2\sqrt{10d^2}$$

$$= 2d\sqrt{10}$$

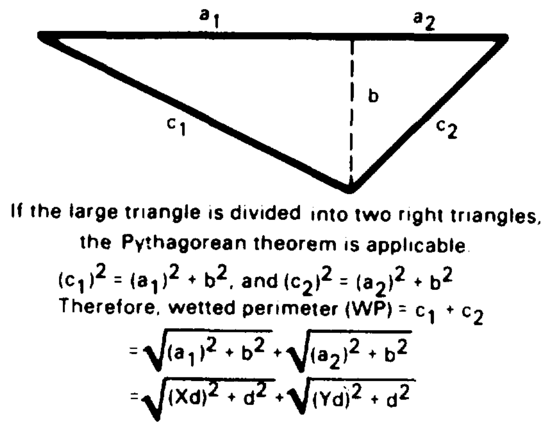


Figure 6-29. Determination of the wetted perimeter of a triangular ditch

Therefore, the hydraulic radius (Rm) is—

$$Rm = \frac{A_w}{wp}$$

$$= \frac{3d^2}{2d\sqrt{10}} = \frac{3d^2}{2(3.16)d} = \frac{3d^2}{6.32d} = 0.474d$$

For depth (d), the depth of water for the conditions **assumed** in step 3, is calculated. Since Rm = 0.848 from step 3, and Rm is also equal to 0.474d, the depth of water (d) can bc computed.

$$d = \frac{0.848}{0.474} = 1.79 \text{ ft}$$

This allows A_w to be computed.

$$A_w = 3d^2 = 3(1.79 \text{ ft})^2 = 9.61 \text{ sq ft}$$

Step 5. Check for Q.

$$Q = A_w V = 9.6(4) = 38.4 \text{ cfs}$$

The Q calculated for the ditch must be within ± 5 percent of the design Q of 47.3 cfs, or between 44.9 and 49.7. The calculated ditch Q is 38.4, which is less than the lower limit and is unacceptable. Since the calculated Q is below the lower limit, raise the velocity for the next trial calculation. For the second trial, the assumed velocity will be 4.5 feet per second.

Step 6. Make a second trial calculation.

$$R_m = \left[\frac{Vn}{1.486\sqrt{S}}\right]^{3/2} = \left[\frac{4.5\,(0.24)}{1.486\,(0.072)}\right]^{3/2}$$

$$= \left[\frac{0.108}{0.107}\right]^{3/2} = (1.01)^{3/2}$$

$$R_m = 1.015$$

$$d = \frac{R_m}{0.474} = \frac{1.01}{0.474} = 2.13 \text{ ft}$$

$$A_w = 3d^2 = 3(2.13)^2 = 13.6 \text{ sq ft}$$

$$Q = A_w V = (13.6)4.5 = 61.2 \text{ cfs}$$

Q is too high; try V = 4.2 fps.

Step 7. Make a third trial calculation.

$$d = \frac{0.912}{0.474} = 1.92 \text{ ft}$$

$$A_w = 3d^2 = 3(1.92)^2 = 11.06 \text{ sq ft}$$

$$Q = A_w V = (11.06)4.2 = 46.5 \text{ fps}$$

Q is acceptable, because it is between the limits.

Step 8. Determine the ditch. With the Q being acceptable, the ditch will have a water depth of 1.92 feet and side slopes of 3:1. Additional depth must be added to the water depth for freeboard. This additional depth can be selected depending upon the conditions external to the ditch but cannot be less than 0.5 foot. Therefore—

Cutting depth = water depth (d) + 0.5
= 1.92 + 0.5
= 2.42 ft or 2.4 ft

The velocity of 4.2 feet per second is acceptable. Some erosion may be anticipated, but it will not be serious. In addition, because of the high velocity, the flow will clean out the sediment from previous low flow,

SPECIAL CHANNELS

Some facilities will have special types of channels where surface runoff will be intercepted and removed. These channels will be similar to open channels, except they will tend to be very wide and shallow. To determine runoff capacity (in cfs) in this type of channel, open channel hydraulics, as previously discussed, must be modified.

Gutters

Gutters are shallow, paved drainage channels used in more permanent construction adjacent to paved or hard-surfaced areas. They provide positive removal of runoff, protection for easily eroded soils adjacent to the pavement, and prevention of softening of turf shoulder areas commonly caused by a large volume of runoff from adjoining pavements.

A cross section of a typical runway gutter is shown in Figure 6-30, page 6-52. This gutter conforms to US Air Force safety requirements and design charts of that particular gutter. Safety and operational requirements for fast landing speeds make it desirable to provide a continuous, longitudinal grade in the gutter. Close conformity to the runway gradient requires the use of sump inlets (drop inlets covered by gratings). A sufficient number of inlets should be provided in the gutter to prevent the depth of flow from exceeding 3 inches.

Median Channels

Channels will be placed down the center of medians, dual roads, runways, taxiways, and other similar structures. These than - nels will be symmetrical but very wide and shallow. The input flow into such channels is from surfaced and unsurfaced areas adjacent to the channel. The flow can be determined by the rational method. Since the factors of slope (S), Manning's n, flow (Q), and the general width are known, the depth can be determined by trial using the flow equation or nomograph for flow in open channels.

BERMS

The uncontrolled inflow from drainage areas adjacent to open channels has been a source of numerous erosion failures. Combating this problem requires special consideration during the design of the surface drainage system. Local runoff inflow can

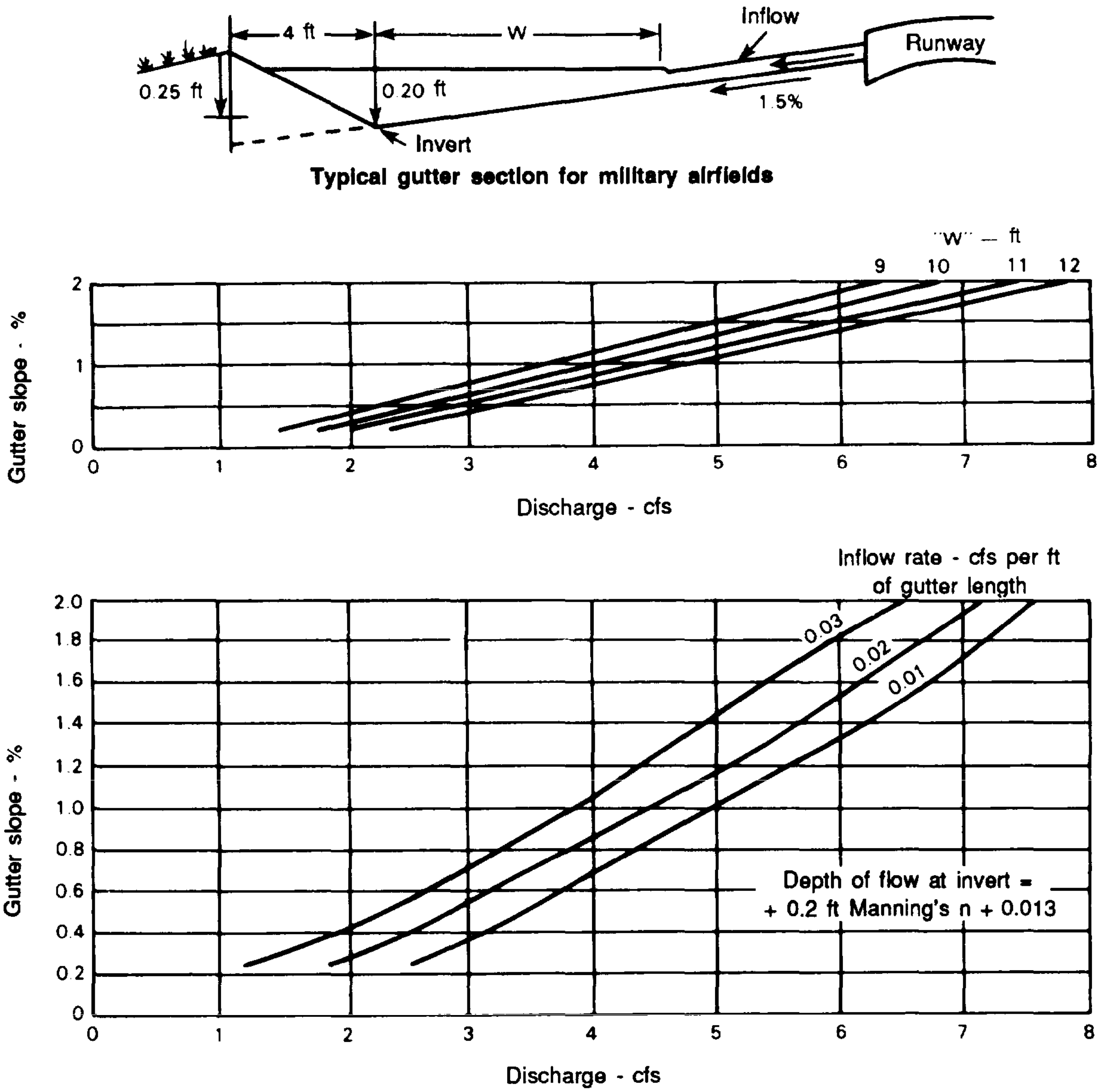


Figure 6-30. Typical run way gutter

be particularly detrimental. Because of normal irregularities in grading operations, runoff becomes concentrated and causes excessive erosion as it flows over the sides of the channel. Experience shows that constructing a berm (raised lip), Figure 6-31, prevents this problem. Place the berm, usually made of earth, at the top edge of the channel. This berm prevents inflow into the channel except at designated points where an inlet, properly protected against erosion, is provided.

Where excavated material is wasted, as in a levee or dike parallel to the channel, there must be frequent openings through the levee to permit inflow to the channel. A suitable berm allows a minimal amount of excavated material to flow back into the channel. This prevents sloughing from the spoil bank into the channel.

Runoff, Q, from the area is determined by the rational method. This runoff is collected and conveyed by the channel formed

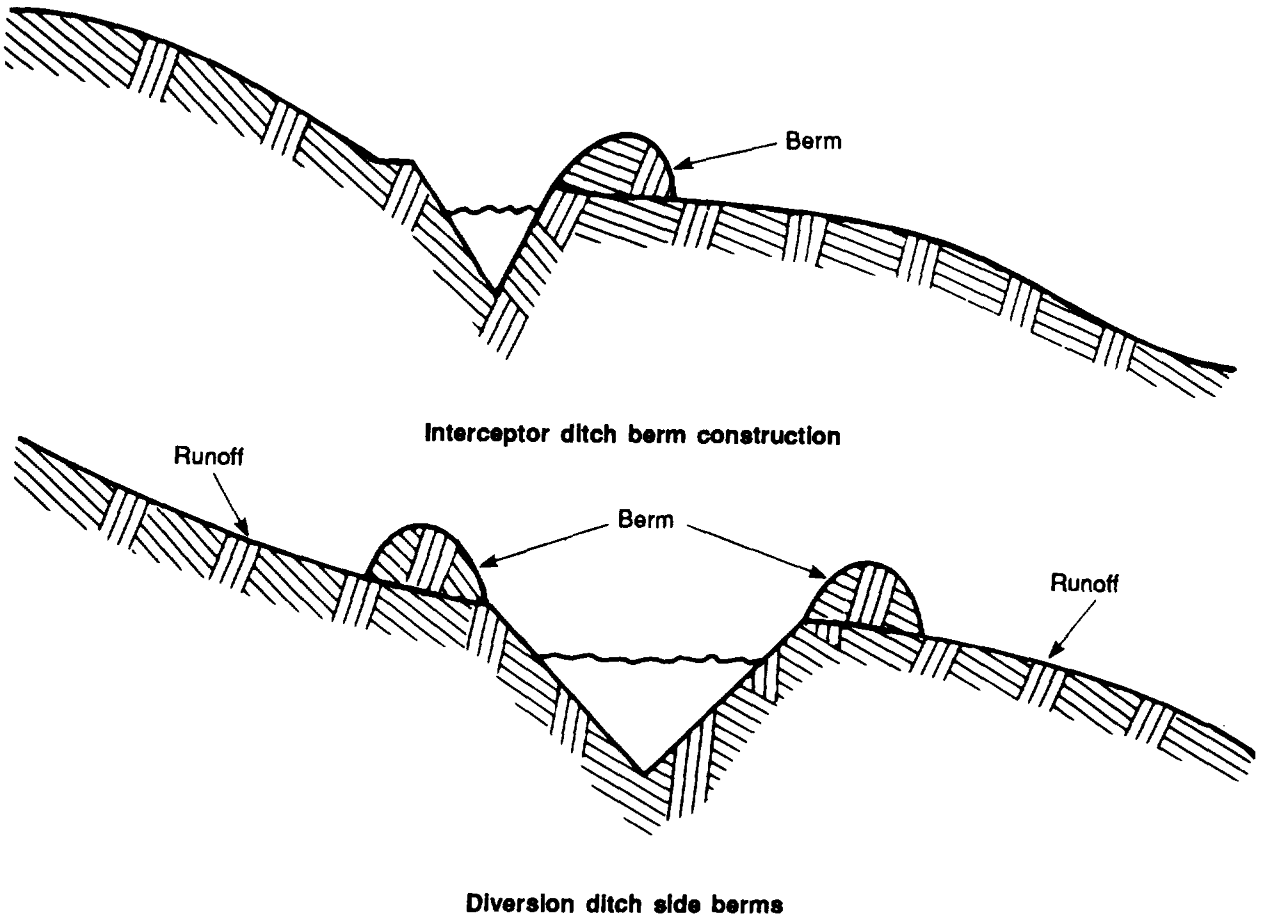


Figure 6-31. Typical berm emplacement

by the berm. The factors of Q, n, S, and width are used to solve for depth by the trial method.

CONSTRUCTION AND MAINTENANCE OF CHANNELS

Ditch construction normally requires either a grader or a scraper. Sometimes, these two items arc used in combination. Depend ing on the location, other factors must be considered in designing and building ditches.

Interceptor ditches should be set on grades, thus allowing for quick removal of water without erosion. When the grade is not sufficient to permit quick water removal, the ditch may require an asphalt or concrete lining. Berms may be constructed on the downhill side of interceptor ditches, as shown in Figure 6-31, to prevent overflow and excessive erosion of the downhill slopes.

Abrupt changes in a ditch's normal flow pattern will induce turbulence and cause excessive erosion. These conditions develop most frequently at channel transitions, junctions, and storm drain outlets. Accordingly, special attention must be given to these locations during design.

Channel maintenance problems exist in all drainage systems. The basic design of the ditch must consider these maintenance problems before they arise. Therefore, designers must have a thorough understanding of the two basic ditch maintenance problems: sedimentation and erosion. Channel protection and ditch shape, when used together, control maintenance problems most effectively.

SEDIMENT CONTROL

Water flowing overland tends to carry sediment into any open channels. When the velocity in the channel is 3 fps or less, this sediment can be deposited in the channel. Since most storms are less intense than the design storm, the channel bottom may accumulate a large volume of loose sediment. When peak flow does occur, the velocity should be great enough to scour the channel bottom clean of loose sediment. If the sediment is not removed, it will compact and gradually reduce the depth of the ditch. At peak flow, the ditch may overflow and cause damage to adjacent structures before the channel is cleaned out; therefore, the channel should be kept clean through maintenance.

When designing channels, keep peak velocity flow above 3 fps. This keeps the channel self-cleaning. A higher velocity is preferred, but such velocity must not exceed the maximum velocity for the soil. (See Table 6-8.) When the soil type requires a maximum allowable velocity under 3 fps, sedimentation will be a maintenance problem. One solution is to line the ditch with asphalt or concrete. This reduces the coefficient of friction, thus increasing the velocity. When sedimentation is expected and linings cannot be used, increase the allowable freeboard to eliminate overflow.

EROSION CONTROL

Water flowing through open channels is turbulent. This turbulence increases as the velocity increases. Together, velocity and turbulence erode and carry away the soil of the channel and endanger nearby structures such as roads, bridges, and culverts. Velocity and turbulence may also endanger the channel itself. In addition, the eroded material may sometimes be deposited within the channel in areas where it will cause damage. In these cases, channel maintenance and repair will be a constant concern.

Erosion most commonly occurs when the velocity of flow exceeds the velocity at which the soil of the channel will erode. Table 6-8 shows the maximum velocities. Erosion can be prevented by lowering the velocity below the soil-erosion velocity. This can be accomplished by lining the natural channel material with a more erosion-resistant material or by reducing the side slopes. Table 6-9 shows the recommended side slopes. Erosion should be considered and accounted for in the design of channels.

Table 6-8. Suggested maximum velocities

Soil type or lining (earth; no vegetation)	Maximum permissible velocities (fps) *		
	Clear water	Water carrying fine silts	Water carrying sand and gravel
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.7	2.5	2.0
Silt loam (noncolloidal)	2.0	3.0	2.0
Ordinary firm loam	2.5	3.5	2.2
Volcanic ash	2.5	3.5	2.0
Fine gravel	2.5	5.0	3.7
Stiff clay (very colloidal)	3.7	5.0	3.0
Graded, loam to cobbles (noncolloidal)	3.7	5.0	5.0
Graded, silt to cobbles (colloidal)	4.0	5.5	5.0
Alluvial silts (noncolloidal)	2.0	3.5	2.0
Alluvial silts (noncolloidal)	3.7	5.0	3.0
Coarse gravel (noncolloidal)	4.0	6.0	6.5
Cobbles and shingles	5.0	5.5	6.5
Shales and hard pans	6.0	6.0	5.0

* As recommended by Special Committee on Irrigation Research, ASCE, 1926

Table 6-9. Recommended side slopes

Type of channel	(Horizontal:vertical)
Firm rock	Vertical to ¼:1
Concrete-lined stiff clay	½:1
Fissured rock	½:1
Firm earth with stone lining	1:1
Firm earth, large channels	1:1
Firm earth, small channels	1½:1
Loose, sandy earth	2:1
Sandy, porous loam	3:1

Decreasing the Hydraulic Radius

Reducing the hydraulic radius will decrease the velocity. This decrease in hydraulic radius can be accomplished by increasing the wetted perimeter, wp, in relation to the area, A (while the Q remains constant). This can be done by widening the ditch, flattening the side slopes, or widening the bottom. This increases the wetted perimeter without materially increasing the area. The required changes in ditch design are determined by the trial approach, since the amount of runoff, Q, must be retained while reducing the velocity.

Lining the Channel

Erosion can be controlled by lining the bottom and sides of the channel.

Grass or Turf

Since natural linings take considerable time to grow or effort to place, they are seldom used in the TO.

Riprap

Riprap lining involves placing rocks or rubble in the bottom and on the sides of the ditch to prevent soil erosion. Rocks should be hand-placed in at least two layers and compacted individually. Riprap not only prevents erosion but decreases velocity in the channel because of its high n value. Riprap also helps prevent erosion when making transitions from paved to soil ditches or from other high-velocity ditches to those in which lower velocity is required. Gabions are another method of lining ditches. Details on riprap and gabions are given on pages 6-116 through 6-123.

Pavement

When the design and construction are done properly, paving the ditch with asphalt or concrete will prevent erosion. Because of the low n values of different types of pavement, velocity may increase too much, causing erosion where the pavement ends. Special protection, such as a stilling basin or rock lining, may be required at this point to slow the velocity before allowing the flow to continue into the natural soil channel. Because of pavemen 1's low n values, it can be used effectively to increase flow velocities if they are too low, thus preventing deposition.

Installing Check Dams

The water velocity in a channel can also be reduced by decreasing the slope. However, except for local variations, building ditches at slopes other than that of the surrounding ground is impractical, One method for decreasing the slope is to install check dams or weirs, as shown in Figure 6-32, page 6-56. Check dams should be considered when the slope ranges between 2 and 8 percent. Channels with slopes of 2 percent or less generally do not require extensive erosion controls. With slopes in excess of 8 percent, it is usually more economical to pave the ditch with asphalt or concrete than to build check dams.

Design

Correct spacing between check dams can be determined by using the following formula:

$$S = \frac{100(H)}{A - B}$$

where—

S = spacing, in feet, between check dams **(This value should not be less than 50 feet.)**

H = height from the channel bottom to the lower edge of the weir notch **(This value should not be greater than 3 feet unless the dam is to be structurally designed. To prevent unnecessary work, the practical lower limit for H is 1 foot.)**

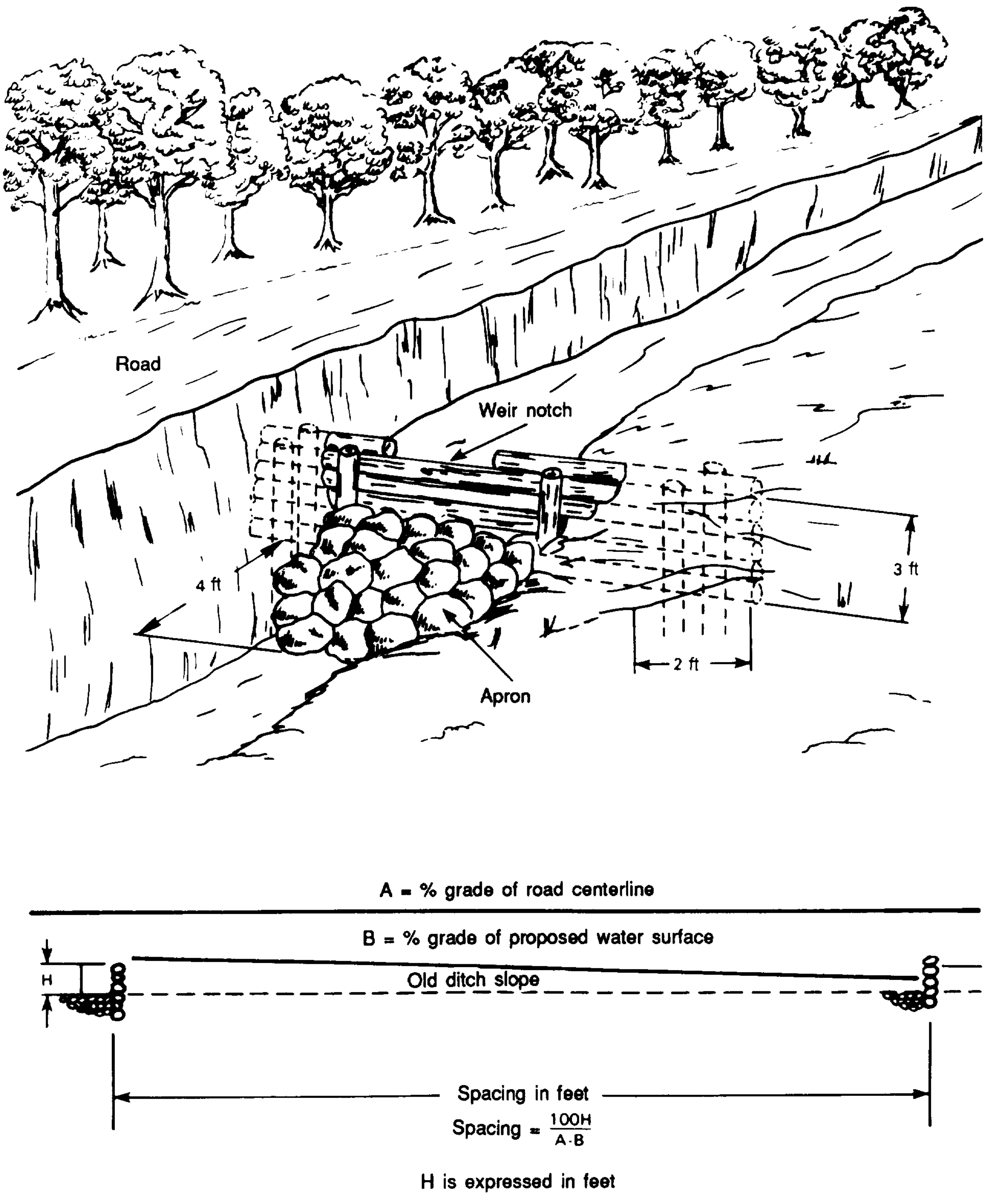


Figure 6-32. Check-dam design

A = slope of the original ditch in percent

B = desired slope in percent **(This value should be set at 2 percent. This is the maximum slope that will not require additional erosion control.)**

Example:

Original slope = 5%

Desired slope = 2%

H = 3 ft

Find S.

$$S = \frac{100(H)}{A - B} = \frac{100(3)}{5-2} = \frac{300}{3} = 100$$

The spacing between check dams is 100 feet. To prevent erosion of the sides of the channel at the check dam, a weir notch is built in the center of the dam. The weir notch must be designed to carry the flow in the ditch. Weir-notch dimensions for various flow rates are given in Table 6-10.

Maintenance

Erosion is a common problem with check dams. It usually occurs when the weir

Table 6-10. Discharge for weir notches in check dams

Compute by formula $Q = 3.39L(H^{3/2})$

Where Q = discharge in cfs
L = length of weir notch in ft
H = depth of weir notch in ft

H	Q							
L	1	2	3	4	5	6	7	8
0.5	1.2	2.4	3.6	4.8	6.0	7.2	8.4	9.6
1.0	3.4	6.8	10.2	13.6	17.0	20.3	23.7	27.1
1.5	6.2	12.5	18.7	24.9	31.1	37.4	43.6	49.8
2.0	9.6	19.2	28.8	38.3	47.9	57.5	67.1	76.7
2.5	13.4	26.8	40.2	53.6	67.0	80.4	93.8	107.2
3.0	17.6	35.2	52.8	70.5	88.1	105.7	123.3	140.9
3.5	22.2	44.4	66.6	88.8	111.0	133.2	155.4	177.6
4.0	27.1	54.2	81.4	108.5	135.6	162.7	189.8	217.0
4.5	32.4	64.7	97.1	129.4	161.8	194.2	226.5	258.9
5.0	37.9	75.8	113.7	151.6	189.5	227.4	265.3	303.2
L	9	10	11	12	13	14	15	16
0.5	10.8	12.0	13.2	14.4	15.6	16.8	18.0	19.2
1.0	30.5	33.9	37.3	40.7	44.1	47.5	50.9	54.3
1.5	56.0	62.3	68.5	74.7	81.0	87.2	93.4	99.6
2.0	86.3	95.9	105.5	115.0	124.6	134.2	143.8	153.4
2.5	120.6	134.0	147.4	160.8	174.2	187.6	201.0	214.4
3.0	158.5	176.1	193.8	211.4	229.0	246.6	264.2	281.8
3.5	199.8	222.0	244.2	266.4	288.6	310.8	333.0	355.2
4.0	244.1	271.2	298.3	325.4	352.6	379.7	406.8	433.9
4.5	291.2	323.6	356.0	388.3	420.7	453.1	485.4	517.8
5.0	341.1	379.0	416.9	454.8	492.7	530.6	568.5	606.4

notch is too small or is clogged with debris so that water flows over the top of the dam. Scour begins on the area exposed to the hydraulic jump. If allowed to continue long enough, the erosion will extend to the area around the dam, as seen in Figure 6-33. If this occurs, there will be more damage caused to adjacent structures than if the dam had never been installed.

Some protection usually takes the form of an extended horizontal or sloping apron. Scour can also be prevented by anchoring the sides and bottom into at least 2 feet of compacted material. Then place riprap along at least 4 feet of the downstream channel. These construction techniques will prevent scour and undercutting.

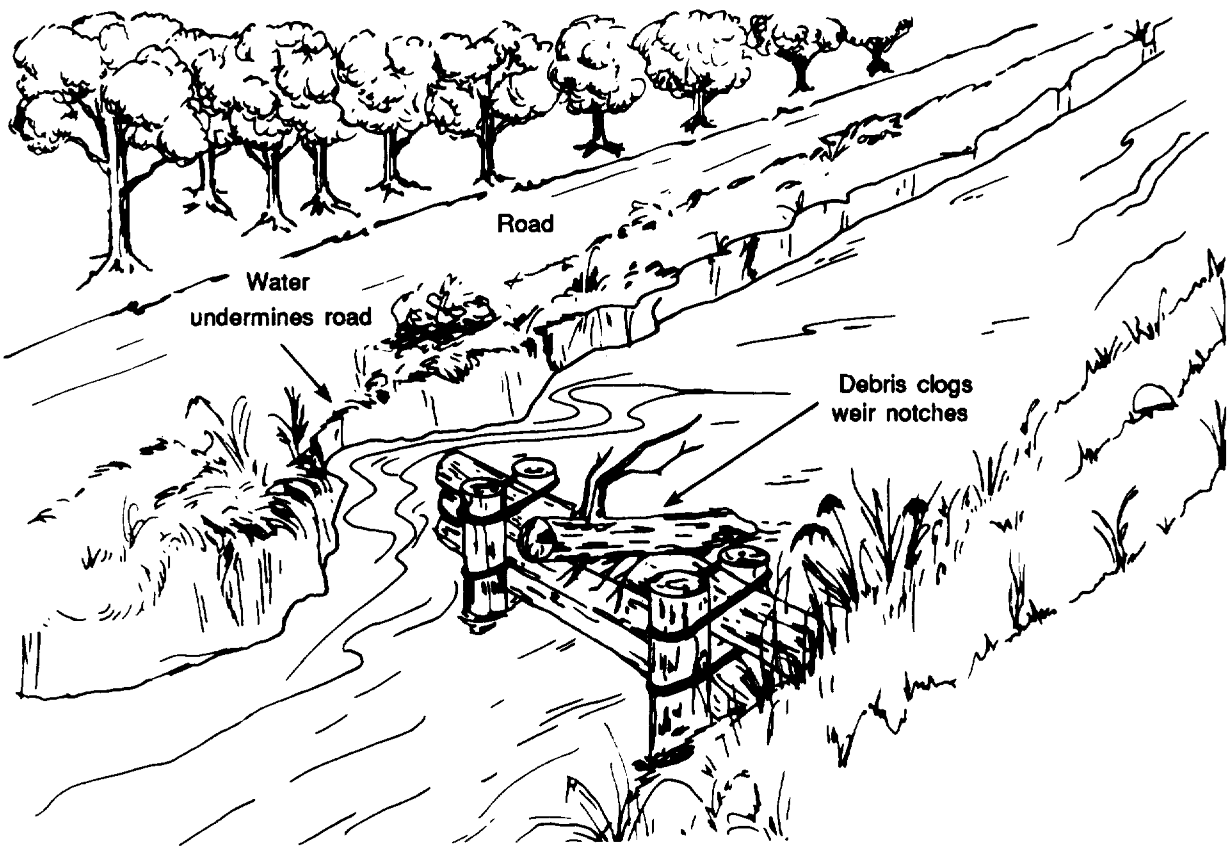


Figure 6-33. Check-dam maintenance

SECTION III. CULVERTS

A culvert is an enclosed waterway used to pass waler through an embankment or fill. The flow in a culvert depends upon several factors, including the lining, slope, headwalls, wing walls, and downstream conditions. Accordingly, culvert types and construction procedures are discussed prior to considering culvert hydraulics and design features.

CULVERT TYPES AND DESIGNS

PERMANENT CULVERTS

Permanent structures can be constructed from corrugated metal pipe (CMP), concrete multiplate pipe arch, vitrified clay (VC), polyvinyl chloride (PVC], or other material.

Corrugated-Metal-Pipe Culverts

Because it is commercially available in numerous shapes, lengths, and diameters, CMP is commonly used in military construction. For TO construction, CMP is made of an aluminum-and-steel alloy. It comes in nestable half sections that, when assembled, give 2-foot effective lengths. It is available in the diameters and gages listed in Table 6-11. The minimum diameter recommended is 18 inches for lenghts up to 20 feet and 24 inches for all other lengths. Small diameters may become clogged with debris and are difficult to maintain.

Specific construction techniques are employed in placing CMP. A retaining wall called a headwall is placed at the upstream end of the culvert. Headwalls arc always used upstream; they arc desirable but not mandatory for the downstream end. The headwall supports the soil mass at the end of the culvert and helps to protect against erosion.

Table 6-11. Sizes of corrugated metal pipe

Diameter (inches)	Gage	Thickness (inches)
12, 18, 24, 30, 36, 42, 48, 60, 72	8	0.179
	10	0.135
	12	0.105
	14	0.075
	16	0.0598
2-foot effective length (nestable half sections)		
Material: aluminum-steel alloy		

CMP joints must be lapped so that water flowing through the culvert passes over the joint rather than into it. Failure to properly overlap the pipes will tend to force the flowing water through the joints and into the fill. All joints must be sealed, preferably with caulk or bituminous material. If joints are not sealed, voids may be generated around culverts, causing collapse of the fill. To increase the velocity for a greater quantity of flow, line the culvert with asphalt. Commercial CMP is available with asphalt linings.

Assembling Nestable CMP

CMP has flange-type fittings which arc easily fastened together by nuts and bolts that come with the sections. Vise grips and a ratchet set make assembly faster and easier.

Concrete-Pipe Culverts

When available, precast concrete pipe should be used instead of CMP for culverts. It has two advantages over CMP. First, it is stronger and requires less cover than CMP to support the same load, Second, the interior surface of concrete pipe is smoother (Manning's n = 0.013) than CMP (Manning's n = 0.024). Because of these advantages, concrete pipe of the same diameter and slope as CMP will carry a higher flow.

Concrete pipe is fabricated in circular and noncircular cross sections, It is available in many lengths and widths. Seal joints between the sections to prevent excessive leakage and subsequent weakening of the fill section Substantial headwalls are required at both ends to prevent separation at the joints. Start assembly downstream and work upstream, ensuring male ends point downstream.

Concrete-Box Culverts

Consider concrete-box culverts where the full area of the waterway must be used. One advantage is that the box can be designed to withstand external loads with little or no cover. Box culverts are especially adaptable to rock sites, since the bottom of the culvert can be placed directly on the rock.

Concrete-box design requires knowledge of construction techniques for reinforced concrete structures. Box culverts can be precast, but construction in the TO will probably require that they be cast in place.

EXPEDIENT CULVERTS

Expedient field-type culverts are built of material available on-site such as logs, oil drums, and sandbags filled with a sand-cement mixture. Some examples of culverts built of these materials appear in Figures 6-34, 6-35 and 6-36. Expedient culverts built and sized properly should serve until permanent structures can be built. For evaluating hydraulics, end areas equivalent 10 CMP can be used for similar slopes.

CULVERT CONSTRUCTION

Proper placement is one of the most important factors during culvert construction. It is a major contributor to survival of the culvert under adverse conditions. Some things to consider in placing culverts arc culvert alignment: slope; fill placement; compaction under, around, and over the culvert; culvert length; and protection against erosion.

Alignment

The relationship of the culvert to the streambed is of major importance. Improper location can cause the stream to seek an alternative path other than the culvert. This could quickly close a road or airfield to traffic. To lessen this effect, use the alignment techniques shown in Figure 6-37, page 6-62.

To maintain an existing drainage pattern, place the culvert directly in the streambed, as in view (A) of Figure 6-37. Even though this may be diagonal to the fill, if the hydraulics of the channel are not changed, the stream will not change its direction.

Prevent the stream from shifting its course at the culvert inlet or outlet. Sometimes the structure will cut across a stream meander as in view (B) of Figure 6-37 This leads to doubt as to where to lay the culvert in the streambed. In this case, it is best to cut a new channel 10 lead the stream away from the structure. The old streambed must be filled and dammed with erosion-resistant material at the junction of the old and new channels. The dam can be built of sandbags, logs, riprap, or other similar material.

Provide a smooth transition into and out of the culvert. The structure may cut across a bend of the stream, as in view (C) of Figure 6-37, with a straight run of the stream through the structure. If the bend is close to the structure, it is preferable to recut the stream as shown, and lay the culvert in the new streambed. Care should be taken to fill in and dam the entrance to the old streambed and the junction of the two streambeds, as descibed above.

Move the water past the project as quickly as possible. When the channel flows paral- -lel. to the structure, erosion will even tually occur. To prevent erosion, dig a new channel routing the flow through the culvert and away from the structure, as shown in view (D) of Figure 6-37. Again, be sure to fill and dam the old streambed at the junction point.

The alignment of ditch relief culverts is shown in Figure 6-38, page 6-63. The amount of flow and the slope of the ditch determine the spacing between culvert inlets. On a road with a 5-percent grade, relief culverts should be spaced 500 feet

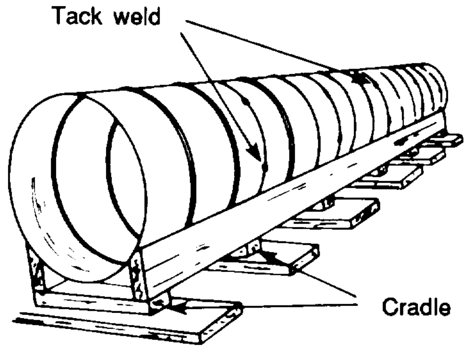


Figure 6-34. Oil-drum culvert

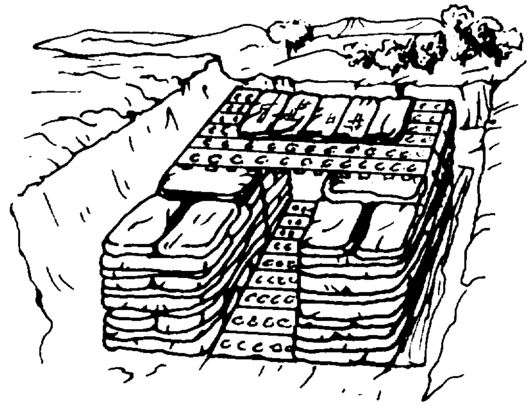


Figure 6-35. Landing-mat and sandbag culvert

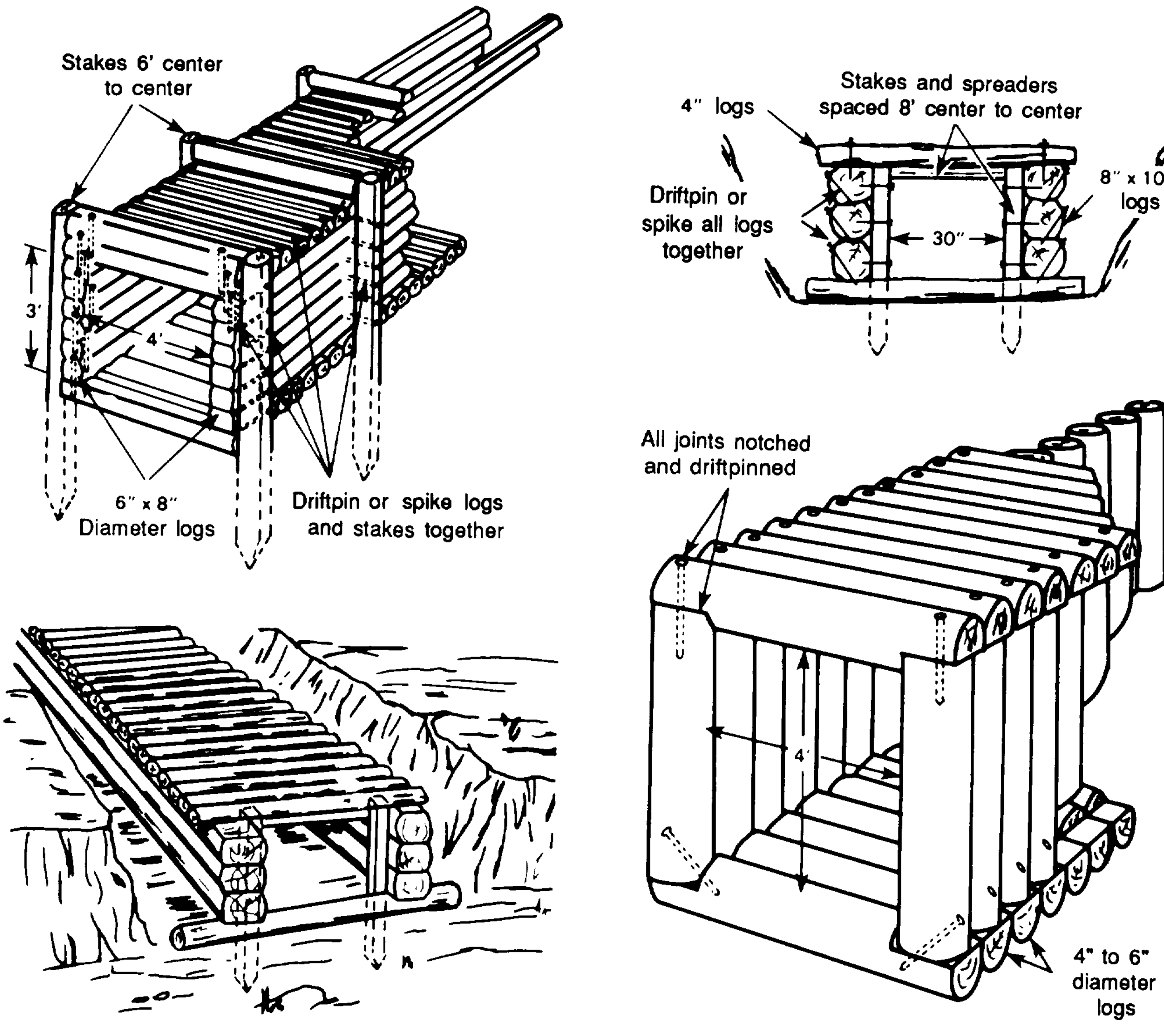


Figure 6-36. Log culverts

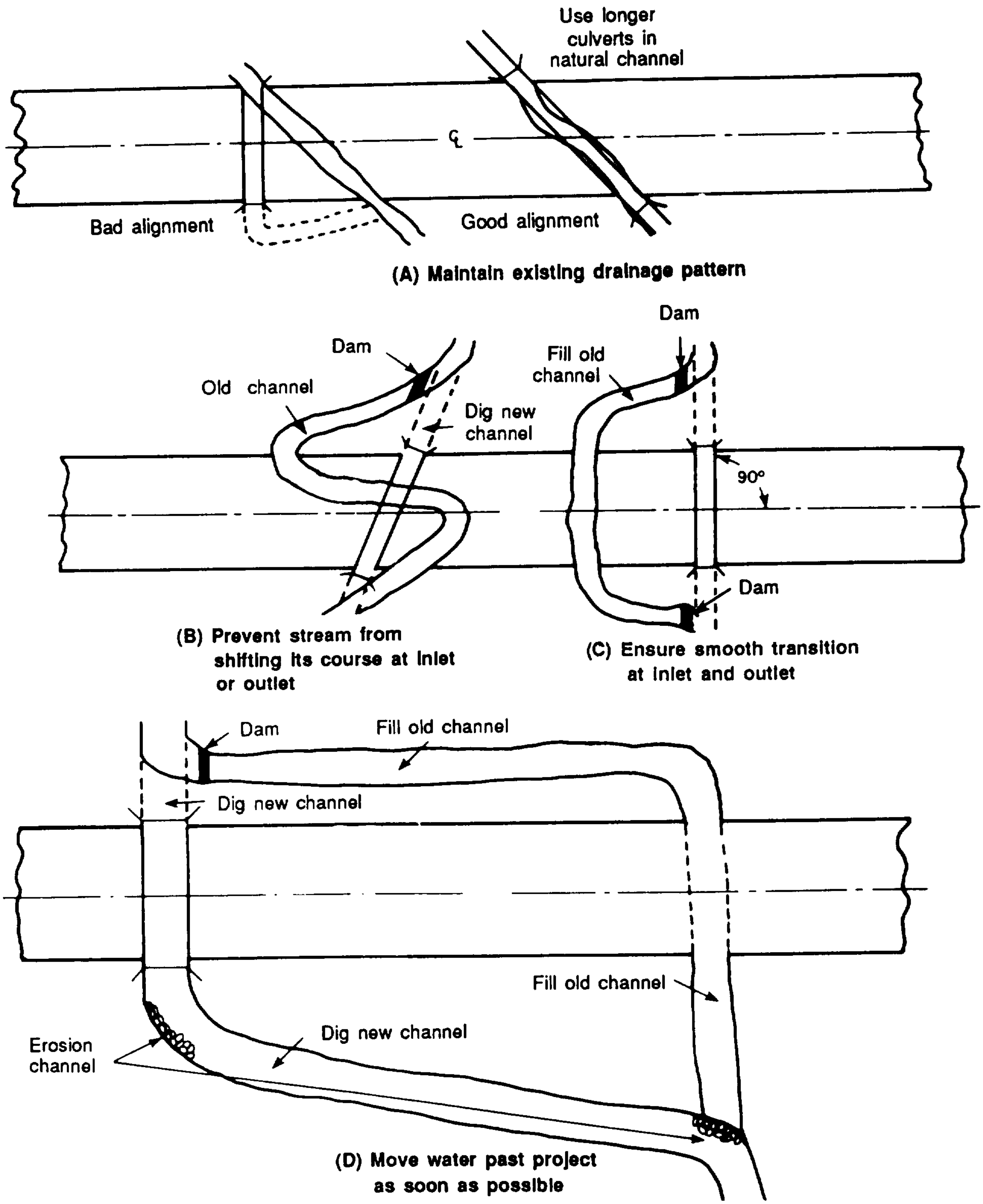


Figure 6-37. Culvert alignment

apart. On an 8-percent grade, spacing should be reduced to 300 feet.

Slope

Culverts normally should be installed with the invert of both the inlet and the outlet of the culvert at streambed or channel elevation. The invert is the lowest point in the

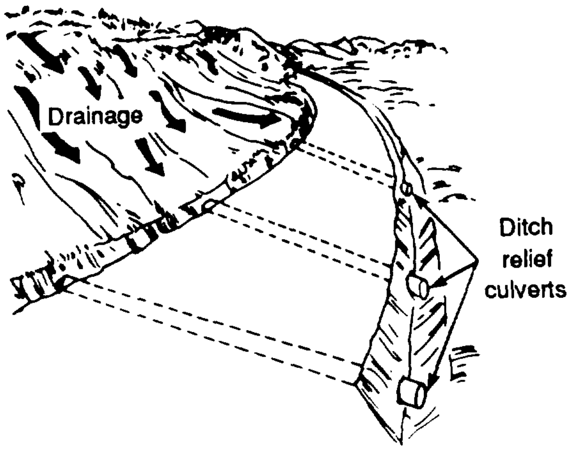


Figure 6-38. Ditch relief culverts

internal cross section of an artificial channel or the bottom of the culvert. The normal grade of the culvert can be modified by two techniques, as shown in Figure 6-39.

- Drop inlets can be used to lower the inlet of the culvert, This will tend to reduce the slope of the culvert. An example of a drop inlet is shown in view (A) of Figure 6-39 and a discussion begins on page 6-89.

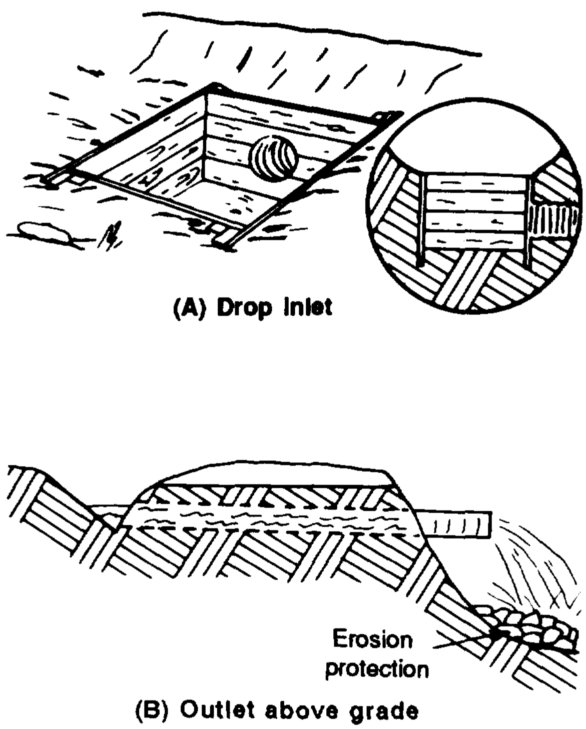


Figure 6-39. Changing normal grade of culvert

- The outlet of the culvert can be raised to reduce the slope, as shown in view (B) of Figure 6-39.

For culverts to be self-cleaning, flow velocity should be at least 3 fps. To achieve this velocity, the culvert should not be set on less than a 0.5 percent grade, if practicable. To prevent excessive outlet velocity, the culvert grade should not exceed 2 percent.

In general, free-falling outlets and culverts with slopes of more than 2 percent will require outlet erosion protection. In mountainous country where there are excessive stream slopes or in fills across dry valleys, culverts may have to be set at grades other than the terrain or streambed slope. In the TO, it may be advisable to accept the erosion problem and install the culvert with a slope for an excess of 2 percent. This will help to prevent a blockage caused by debris passing through the pipe.

Depth of Fill

The distance measured from the culvert invert to the edge of the shoulder or top of the fill, as shown in Figure 6-40, page 6-64, is the depth of fill. The depth of fill must be equal to or greater than the cover plus the pipe diameter. Otherwise, a smaller diameter culvert or a drop inlet must be used. For a road culvert made of CMP, the largest diameter allowable must equal two-thirds the minimum fill depth.

Cover

The depth of compacted soil from the top of the culvert (crown) to the finished construction grade is called cover. The culvert and the surrounding compacted soil must have sufficient strength to carry the compacted soil backfill (dead loads) and the wheel and impact loads (live loads) of the traffic. Live loads are more damaging than dead loads on culverts under shallow cover, and dead loads are more damaging than live loads on culverts under deep cover. Accordingly, both minimum protective cover and maximum permissible depths of backfill must be included in design considerations for culverts. During construction, provide

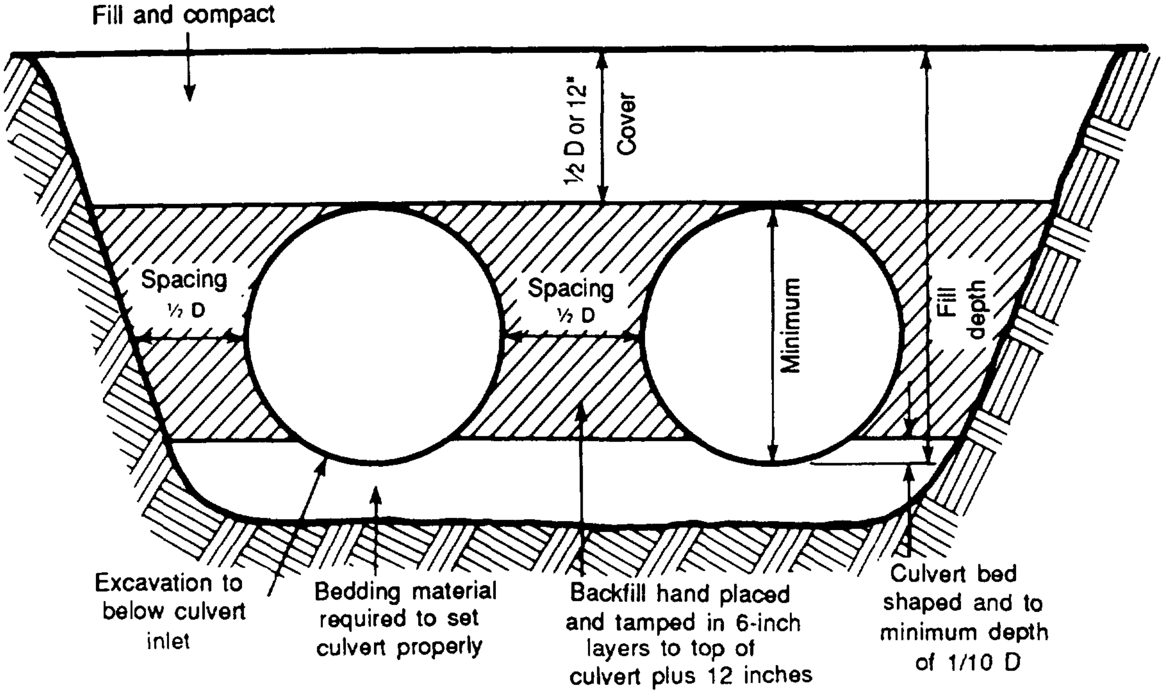


Figure 6-40. Culvert specifications for CMP

adequate additional cover to protect the culvert from damage in places where heavy construction equipment will be crossing frequently.

The minimum cover required to protect the culvert pipe against live loads will depend upon the type of load. For road culverts, the minimum cover is one-half the diameter of the culvert, or 12 inches, whichever is greater. The cover over culverts used in airfields must be specifically designed for the heaviest aircraft using the facility. Table 6-12 lists characteristics of US aircraft. Use this table to determine the landing-gear configuration, wheel loads, and culvert weight type. This will, in turn, allow you to select the proper fill requirements category in Table 6-13, page 6-66, or Table 6-14, page 6-67.

Tables 6-13 and 6-14 give the cover requirements for culverts under airfields. These tables are composed of a series of charts based on aircraft load and gear configurations. Each of these charts is headed by a culvert weight type. These charts are good for 5,000 pounds more than listed. (For example, the chart headed "60,000-lb Single-wheel" is good for single-wheel loads up to and including 65,000 pounds.)

Each chart is subdivided into columns headed by culvert pipe diameters, in inches, and rows based on the gage of the metal. Use linear interpolation for the incremental diameters not listed. The body chart gives the cover required, in feet, for the diameter and gage of pipe concerned.

To use these tables, find the chart that applies to the particular aircraft wheel or gear load and landing-gear type. Then use Table 6-12 to find the appropriate culvert type. When the proper chart is located, read down the left column of the table to determine the appropriate row for the gage concerned. When the proper row has been found, read horizontally to the right and find the minimum cover, in feet, under the column headed with the pipe diameter in

Table 6-12. Characteristics of US aircraft

Aircraft	Name	Max wt (kip)	Gear type	Load on main gear (kips)	Culvert WT type
A-7	Corsair II	42	S	18	2
A-10A	(CAS)	51	S	23	2
A-37B	Dragonfly	15	S	7	1
F-111A/D/E	(VGF)	100	S	44	4
F-111F	(VGF)	100	S	48	4
F-4C/D/E/G	Phantom II	58	S	25	3
F-14C	Tomcat	72	S	34	4
F-15A/B	Eagle	56	S	25	2
F-15C/D	---	68	S	30	2
F-15E	---	81	S	35	2
F-16A	(LWF)	33	S	14	2
F-18	Hornet	20	S	9	1
AV-8B	Harrier	28	DU-S(BIC)	10	2
AV-16A	Advanced Harrier	30		11	2
B-1	(Adv SB)	477	T-T-TA	222	15
B-52G/H	Stratobomber	488	T-T	265	16
FB-111A	(VGSB)	114	Single	54	5
C-5B	Galaxy	840	T-DE-TA	400	15
C-9A	Nightingale	108	Twin	52	7
C-17	---	580	T-TA-TR1	267	15
C-130E	Hercules	175	S-TA	84	13
HC-130H	Hercules	175	S-TA	84	13
KC-135A	Stratotanker (707)	301	T-TA	142	14
C-141A/B	Starlifter	323	T-TA	153	14
VC-137B	(707)	334	T-TA	167	14
E-3A	(707 AWACS)	323	T-TA	155	14
E-4B	(747 AABNCP)	800	DU-T-TA	372	16
SR-71C	(ASR)	170	Triple	77	9
OV-10A/B	Bronco	14	DU-S(TA)	55	1
UV-18A	Twin Otter	13	Single	6	1

Table 6-13. Minimum cover requirements, in feet, for airfields (CMP)

Weight type	1						2						3						4					
	5,000-lb Single-wheel						15,000-lb Single-wheel						25,000-lb Single-wheel						40,000-lb Single-wheel					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
GAGE																								
16	1.0	1.0					1.0	1.5					1.5	2.0					2.0	3.0				
14	1.0	1.0	1.5				1.0	1.0	2.0				1.0	1.5	2.5			1.5	2.5	3.5				
12		1.0	1.0	1.5				1.0	1.0	2.0				1.5	2.0	3.0				1.5	2.5	3.5		
10		1.0	1.0	1.0	1.0	1.0		1.0	1.0	1.0	1.5	2.0		1.0	1.5	2.0	2.5	3.0		1.5	2.0	2.5	3.0	3.5
8			1.0	1.0	1.0	1.0			1.0	1.0	1.0	1.5			1.0	1.5	2.0	2.5			2.0	2.0	2.5	3.0

Weight type	5						6						7						8					
	60,000-lb Single-wheel						20,000-lb Twin-wheel assy						40,000-lb Twin-wheel assy						60,000-lb Twin-wheel assy					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
GAGE																								
16	2.5	4.0					1.0	1.5					2.0	2.5					2.5	3.5				
14	2.0	3.0	4.0				1.0	1.0	1.5				1.0	2.0	3.5				2.0	3.0	4.0			
12		2.5	3.5	4.0				1.0	1.5	1.5				1.0	2.0	3.5				2.0	3.0	4.0		
10		2.0	3.0	3.5	4.0	4.5		1.0	1.0	1.0	1.5	2.0		1.0	2.0	2.0	2.5	3.0		2.0	3.0	3.5	3.5	4.0
8			2.5	3.0	3.5	4.0			1.0	1.0	1.0	1.5			2.0	2.0	2.0	2.5			2.0	3.0	3.5	3.5

Weight type	9						10						11						12					
	80,000-lb Twin-wheel assy						100,000-lb Twin-wheel assy						120,000-lb Twin-wheel assy						40,000-lb Single-tandem assy					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
GAGE																								
16	2.5	4.5					3.0	5.0					3.0	5.5					1.5	2.0				
14	2.5	3.5	4.5				3.0	4.0	5.5				3.0	4.5	6.5				1.0	1.5	3.0			
12		3.0	4.0	4.5				3.5	4.5	5.5				3.5	5.0	6.5				1.0	1.5	3.0		
10		2.5	3.5	4.5	4.5	5.0		3.0	4.0	5.0	5.5	6.0		3.5	5.0	5.5	6.5	7.0		1.0	1.5	1.5	2.0	2.5
8			3.0	3.5	4.5	4.5			3.5	4.0	5.0	5.5			4.0	5.0	5.5	6.5			1.5	1.5	1.5	2.0

Weight type	13						14						15						16					
	60,000-lb Single-tandem assy						150,000-lb Twin-tandem assy						200,000-lb Twin-tandem assy						265,000-lb Twin-tandem assy					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
GAGE																								
16	2.0	3.0					2.5	5.5					3.0	6.0					3.5	7.0				
14	1.5	2.5	3.5				2.0	4.5	6.5				2.0	4.5	7.5				2.5	6.0	9.0			
12		1.5	2.5	3.5				3.5	5.5	6.5				3.5	6.5	7.5				4.5	8.0	9.0		
10		1.5	2.5	3.0	3.0	3.5		3.0	5.0	6.0	6.5	7.0		3.5	6.0	7.0	7.5	8.0		4.0	7.0	8.5	9.0	9.5
8			2.0	2.5	3.0	3.0			4.5	5.5	6.0	6.5			5.5	7.0	7.0	7.5			6.5	8.0	8.5	9.0

*Corrugated metal pipe, Federal Specification QQ-C-806a and Am 1, AASHO M36-57, or AREA Specification 1-4-6 (1953)

NOTES:

1. Pipe produced by certain manufacturers exceeds the strength requirements established by the indicated standards. When additional strength is proven, the minimum cover may be reduced accordingly.
2. Table to be used for both trench- and embankment-type installations.
3. Cover for pipe within landing or taxiway strips or similar traffic areas will be in accordance with this table except as provided in note 4.
4. Pipe placed under airfield rigid pavements will have a minimum cover measured from the bottom of the slab of 0.5 ft for 25,000-lb single-wheel load, 40,000-lb twin-wheel assembly, and 40,000-lb single-tandem assembly. Minimum cover for greater loadings will be 1.0 ft.
5. In seasonal frost areas, minimum pipe cover will meet the requirements of Table 2 of Change 1 to EM 1110-345-283 for protection of storm drains against freezing conditions.

question. Two examples of this tabulation follow.

Example 1:

C130E Hercules—12-gage pipe; from Table 6-12, page 6-65, C130E requires culvert WT type 13.

60,000-lb single-tandem—12-gage pipe. (Use 60,000-lb single-tandem assembly (assy), chart 13.)

Weight type	13					
	60,000-lb Single-tandem assy					
Pipe dia (in)	12	24	36	48	60	72
GAGE						
16	2.0	3.0				
14	1.5	2.5	3.5			
12		1.5	2.5	3.5		
10		1.5	2.5	3.0	3.0	3.5
8			2.0	2.5	3.0	3.0

Table 6-14. Minimum cover requirements, in feet, for airfields (reinforced concrete pipe)

Weight type	1						2						3						4					
	5,000-lb Single-wheel						15,000-lb Single-wheel						25,000-lb Single-wheel						40,000-lb Single-wheel					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
Strength Class I					1.5	1.5					2.5	2.5					3.5	3.5					—	5.0
Class II	1.5	1.5	1.5	1.5	1.5	1.0	2.5	2.5	2.5	2.5	2.0	2.0	3.5	3.5	3.0	3.0	3.0	2.5		5.5	4.5	4.0	4.0	4.0
Class III	1.0	1.0	1.0	1.0	1.0	1.0	2.0	2.0	2.0	2.0	2.0	2.0	3.0	2.5	2.5	2.5	2.5	2.0	4.0	3.5	3.0	3.0	3.0	3.0
Class IV	1.0	1.0	1.0	1.0	1.0	1.0	1.5	1.5	1.5	1.5	1.5	1.5	2.0	2.0	2.0	2.0	2.0	2.0	3.0	2.5	2.5	2.5	2.0	2.0
Class V	1.0	1.0	1.0	1.0	1.0	1.0	1.5	1.0	1.0	1.0	1.0	1.0	2.0	1.5	1.5	1.5	1.5	1.5	2.5	2.0	2.0	2.0	2.0	2.0

Weight type	5						6						7						8					
	60,000-lb Single-wheel						20,000-lb Twin-wheel assy						40,000-lb Twin-wheel assy						60,000-lb Twin-wheel assy					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
Strength Class I					—	—					2.5	2.5					6.0	5.0					—	—
Class II	—	—	—	—	5.5	5.0	2.5	2.0	2.0	2.0	2.0	2.0	6.0	5.5	4.0	3.5	3.5	3.5	—	—	—	—	5.5	5.0
Class III	5.5	4.5	4.0	4.0	4.0	4.0	2.0	1.5	1.5	1.5	1.5	1.5	3.5	3.0	3.0	2.5	2.5	2.5	5.5	4.5	4.0	3.5	3.5	3.5
Class IV	3.5	3.0	3.0	3.0	3.0	3.0	1.5	1.0	1.0	1.0	1.0	1.0	2.5	2.0	2.0	2.0	2.0	2.0	3.5	3.0	2.5	2.5	2.5	2.5
Class V	3.0	2.5	2.5	2.5	2.5	2.5	1.0	1.0	1.0	1.0	1.0	1.0	2.0	1.5	1.5	1.5	1.5	1.5	2.5	2.5	2.0	2.0	2.0	2.0

Weight type	9						10						11						12					
	80,000-lb Twin-wheel assy						100,000-lb Twin-wheel assy						120,000-lb Twin-wheel assy						40,000-lb Single-tandem assy					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
Strength Class I					—	—					—	—					—	—					5.5	4.5
Class II	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	5.5	5.0	3.5	3.0	3.0	3.0
Class III	—	6.5	6.0	5.0	4.5	4.5	—	7.5	7.0	6.0	6.0	5.5	—	—	—	—	7.0	6.5	3.0	2.5	2.5	2.0	2.0	2.0
Class IV	4.0	4.0	3.5	3.0	3.0	3.0	5.0	4.5	4.0	4.0	4.0	3.5	5.5	5.0	5.0	4.5	4.0	4.0	2.0	1.5	1.5	1.5	1.5	1.5
Class V	3.5	3.0	3.0	2.5	2.5	2.5	4.0	3.5	3.0	3.0	3.0	3.0	4.5	4.0	4.0	3.5	3.5	3.5	1.5	1.0	1.0	1.0	1.0	1.0

Weight type	13						14						15						16					
	60,000-lb Single-tandem assy						150,000-lb Twin-tandem assy						200,000-lb Twin-tandem assy						265,000-lb Twin-tandem assy					
Pipe dia (in)	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72	12	24	36	48	60	72
Strength Class I					—	—					—	—					—	—					—	—
Class II	—	—	—	5.5	5.0	4.5	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—	—
Class III	5.0	4.0	3.5	3.0	3.0	3.0	—	—	—	—	7.0	6.0	—	—	—	—	—	—	—	—	—	—	—	—
Class IV	3.0	2.5	2.0	2.0	2.0	2.0	6.0	5.0	4.0	4.0	3.5	3.5	7.5	6.5	6.0	5.0	5.0	4.5	9.5	8.0	7.5	7.0	6.5	6.0
Class V	2.0	2.0	1.5	1.5	1.5	1.5	4.5	4.0	3.5	3.0	3.0	3.0	6.0	5.0	4.5	4.0	3.5	3.5	6.5	6.0	6.0	5.5	5.0	5.0

*Reinforced concrete culvert, storm drain, and sewer pipe, ASTM C76-59T or AASHO M170-57

NOTES:

1. Pipe produced by certain manufacturers exceeds the strength requirements established by the indicated standards. When additional strength is proven, the minimum cover may be reduced accordingly.
2. Table to be used for both trench- and embankment-type installations.
3. Cover for pipe within landing or taxiway strips or similar traffic areas will be in accordance with this table except as provided in note 4.
4. Pipe placed under airfield rigid pavements will have a minimum cover measured from the bottom of the slab of 0.5 ft for 25,000-lb single-wheel load, 40,000-lb twin-wheel assembly, and 40,000-lb single-tandem assembly. Minimum cover for greater loadings will be 1.0 ft.
5. Dash (—) indicates allowable load is less than load on pipe. Blanks indicate that pipe is not specified by applicable standards.
6. In seasonal frost areas, minimum pipe cover will meet the requirements of Table 2 of Change 1 to EM 1110-345-283 for protection of storm drains against freezing conditions.

From chart:

Pipe dia (in)	24	*30**	36	*42**	48
Cover required (ft)	1.5	*2.0**	2.5	*2.75**	3.0

**Linear interpolation between sizes listed.*

Example 2: 60,000-lb single-wheel—8-gage pipe. (Use chart 5 in Table 6-13,)

Pipe dia (in)	36	*42**	48	*54**	60
Cover required (ft)	2.5	*2.75**	3.0	*3.25**	3.5

**Linear interpolation between sizes listed.*

The maximum permissible depth of fill (cover) for CMP is given in Table 6-15 for steel and Table 6-16 for aluminum alloy. Reinforced concrete pipe made under standard specifications can be used in fill up to 50 feet.

Bedding (Foundations)

The minimum bedding depth for pipe culverts is one-tenth the diameter of the pipe as shown in Figure 6-40, page 6-64. The bedding is formed and shaped to fit the bottom of the culvert. In addition, the foundation is cambered (curved slightly upward), as shown in Figure 6-41 along the centerline of the culvert to allow for settlement and to ensure tightness in the joints. At no time should the invert elevation increase as the flow proceeds downstream.

Table 6-15. Maximum permissible cover for CMP

	Maximum permissible cover (ft)									
Diameter (in)	Circular section gage					Vertically elongated section gage				
	16	14	12	10	8	16	14	12	10	8
8	80									
10	60	80								
12	60	70	80							
15	50	70	80							
18	40	60	80							
21	35	50	80							
24	15	45	70	80						
30	—	30	45	70	80					
36	—	15	30	45	70					
42	—	—	25	35	60					
48	—	—	20	25	35	—	—	25	70	80
54	—	—	15	20	30	—	—	20	50	80
60	—	—	—	15	25	—	—	—	45	80
66	—	—	—	15	20	—	—	—	35	70
72	—	—	—	25	15	—	—	—	25	60
78	—	—	—	—	—	—	—	—	—	25
84	—	—	—	—	—	—	—	—	—	20

NOTES:

1. Except for gage tables, CMP will conform to the requirements of Federal Specification QQ-C-806a and Am1, AASHTO Standard M36-57 or to AREA Specification 1-4-6 (1953).
2. Table will be used for normal installation conditions and for loads ranging from dead (earth) load only to dead load plus H-20-44 live load. H-20-44 live load is equivalent to a 32,000-lb axle load.
3. Vertical elongation will be accomplished by either shop fabrication or field strutting and will generally be 5% of the pipe diameter.

Table 6-16. Maximum permissible cover for corrugated aluminum-alloy pipe

Diameter (in)	Maximum permissible cover (ft)									
	Circular section gage					Vertically elongated section gage				
	16	14	12	10	8	16	14	12	10	8
8	50									
10	35	40								
12	30	35	40							
15	25	30	35							
18	20	25	30							
21	20	25	30							
24	10	20	30	35						
30	—	20	25	30	35					
36	—	10	20	25	30					
42	—	—	15	20	30					
48	—	—	15	20	25	—	—	20	25	30
54	—	—	10	15	20	—	—	15	20	25
60	—	—	—	10	20	—	—	—	15	25
66	—	—	—	10	15	—	—	—	15	20
72	—	—	—	—	10	—	—	—	10	15
78	—	—	—	—	—	—	—	—	—	15
84	—	—	—	—	—	—	—	—	—	10

NOTES:

1. Corrugated-aluminum-alloy pipe will conform to the requirements of AASHTO Standard M196-621.
2. Table will be used for normal installation conditions for loads ranging from dead (earth) load only to dead load plus H20-44 live load or dead load plus Cooper E-60 railway loading.
3. Vertical elongation will be accomplished by either shop fabrication or field strutting and will generally be 5% of the pipe diameter.

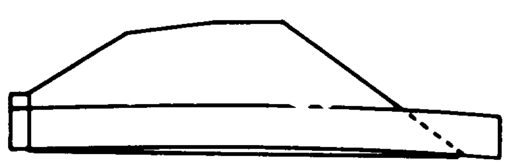


Figure 6-41. Elevation of center section of culvert

Culverts are constructed on firm, well-compacted foundations. The composition of the soil on the bottom of the stream should be determined. Good granular material will be required to form a proper compacted and shaped bed. If a stream bottom is composed of poor material, such as organic matter, muck, silt, or large material that could puncture the CMP, remove and replace the material. The depth of material to be removed will depend upon information from borings. When the bearing strength of the soil is completely inadequate and uneven settlement is expected provide cradle footings. These footings may require piling as shown in Figure 6-42, Footings used for timber culverts as shown in Figure 6-43, can also be adapted for various types of culverts and soils.

Culverts can be placed on properly prepared rock foundations, Trench the rock and backfill with firmly compacted soil for bedding the culvert, as shown in Figure 6-44.

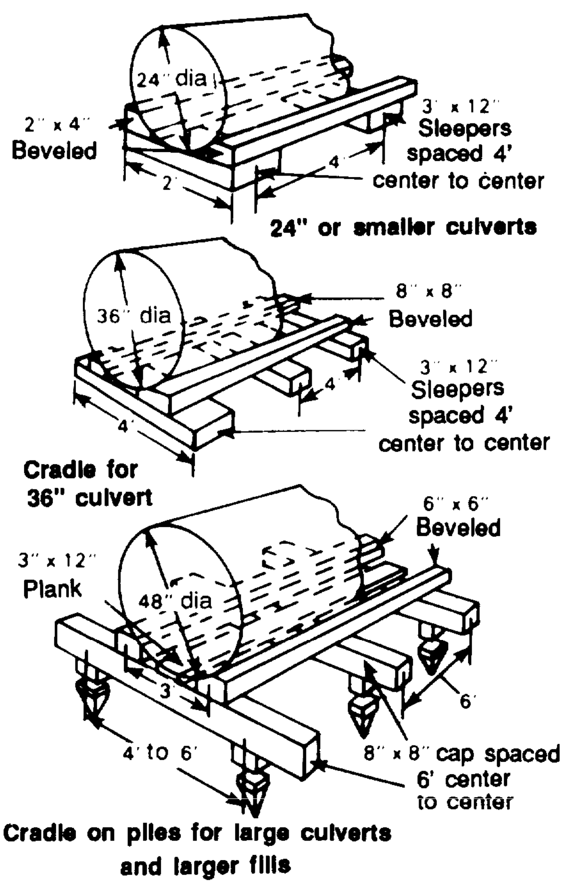


Figure 6-42. Cradle footings

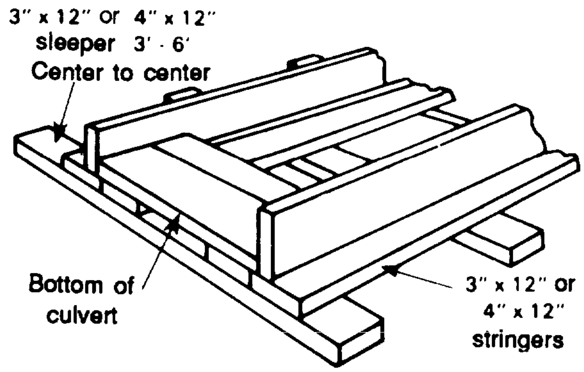


Figure 6-43. Footings for timber culverts

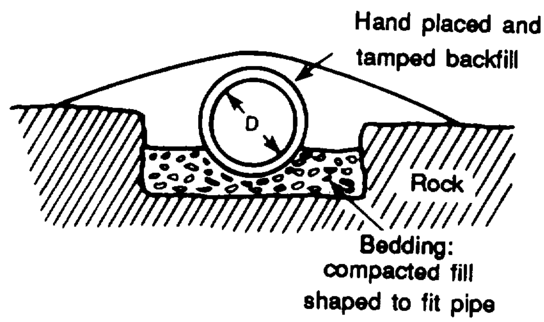


Figure 6-44. Concrete-culvert installation in rock

Backfill

Take care in backfilling around the culvert, since the backfill supports the culvert against soil pressure generated by surface loads as shown in Figure 6-39, page 6-63. The soil selected should be placed carefully in well-compacted layers kept at the same elevation on both sides of the culvert. Compaction is done in 6-inch layers if hand- or air-operated or if other mechanical tampers are used. If logs, hand tamping, or other expedient methods are used, place the soil in 4-inch layers. Carry the compaction from the culvert bed material to 12 inches or one-half the diameter above the top of the culvert (whichever is greater).

Length

Culvert length is determined by the width of the embankment or soil mass through which the culvert carries water. Culverts that do not include a downstream headwall must be long enough 10 extend a minimum of 2 feet beyond the toe of the embankment to prevent erosion.

Headwalls, Wing Walls, and Aprons

Hcadwalls and aprons arc constructed to guide water into the culvert, prevent or control erosion, reduce seepage, hold the soil in place, and support the ends of the culvert. Headwalls close to roads should not protrude above the shoulder grade. They should be at least 2 feet outside the shoulder so they do not present a traffic hazard.

Use headwalls on the upstream end of all culverts. If possible, use a headwall on the downstream end as well. When concrete pipes arc used, headwalls are mandatory at both ends. Headwalls, both upstream and downstream, should have wing walls or retaining walls set at an angle 10 the headwall, This will help support the fill and d irect the water flow to prevent erosion. The upstream wing wall will guide the water into the culvert and assist in improving the culvert hydraulics. The downstream wing wall, combined with an apron, will help reduce the velocity of the stream, and thereby lessen erosion al the outlet Ideally, headwalls and wing walls should be reinforced concrete or mortared stone. Standard designs are in the TM 5-302 series manuals. They can, however, be made of expedient material such as lumber, logs, or sandbags. These structures are shown in Figure 6-45. For speedy construction in the TO, sandbags filled with a soil-and-cement mixture may provide the best headwall possible.

When headwalls arc not used on the downstream end, the culvert should project beyond the toe of the fill at least 2 feet. Use riprap to protect the projecting area culvert riprap, as shown Figure 6-46, page 6-72.

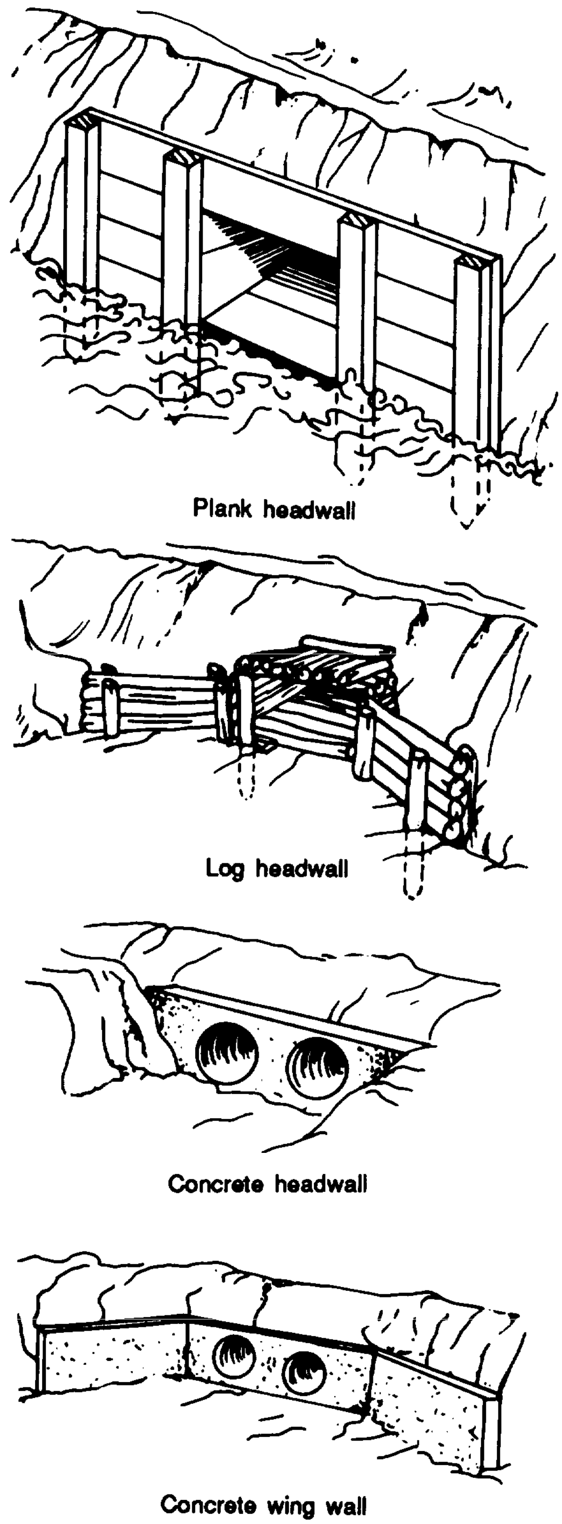


Figure 6-45. Headwalls and wing walls for culverts

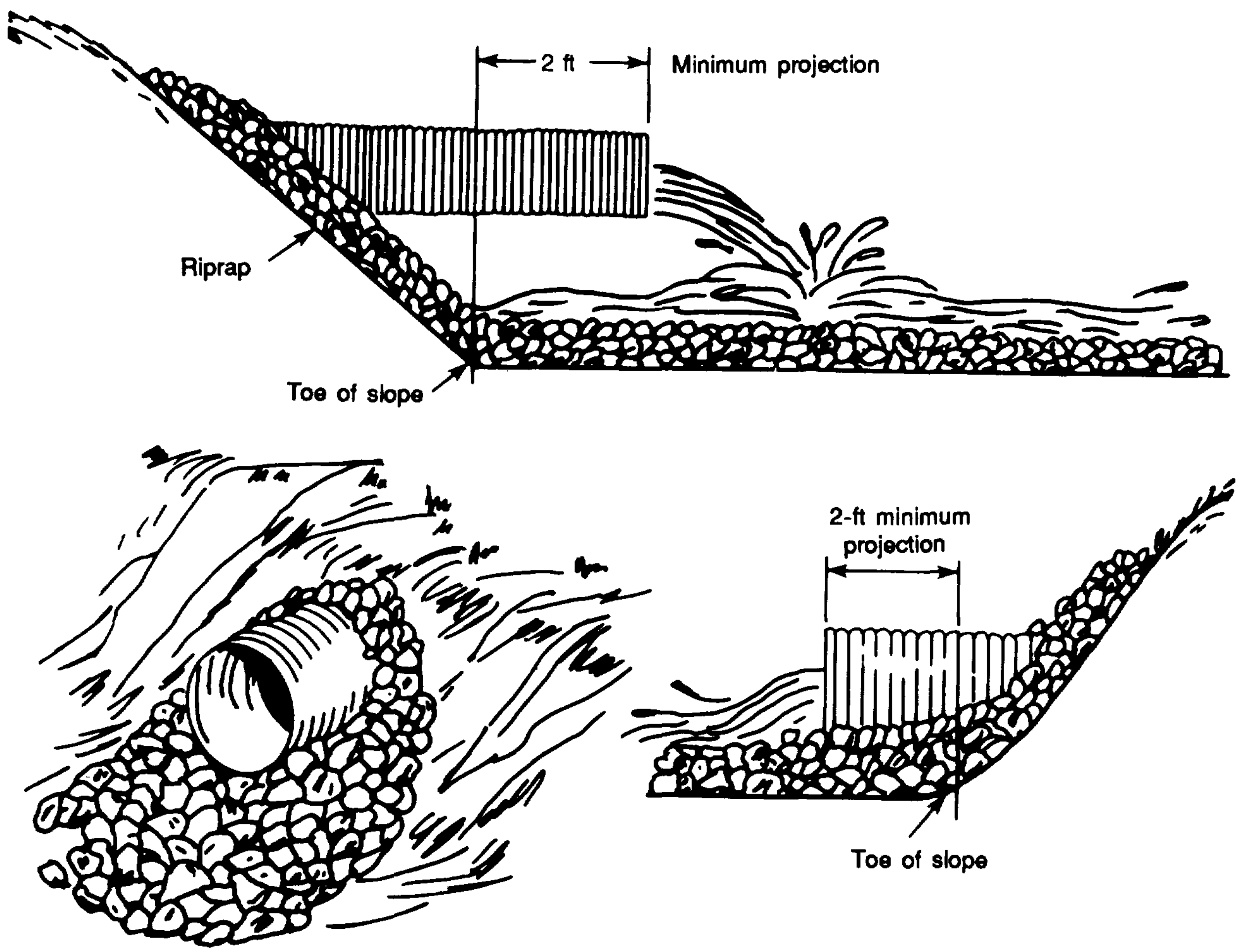


Figure 6-46. Culvert outlets without headwalls

Erosion Control

Culverts discharging into open channels should have antiscour protection to prevent erosion. Two types of channel instability can develop downstream from culvert and storm-drain outlets. These conditions, known as gully scour and scour hole, are shown in Figure 6-46. Predict the type of scour for a given field situation by comparing the original or existing channel slope or drainage basin downstream of the outlet to what is required for stability.

Gully scour is expected when channel flow exceeds that required for stability. It begins at a point downstream where the channel is stable and progresses upstream. If sufficient differential in elevation exists between the outlet and the stable channel, the outlet structure will be completely undermined. Erosion of this type may be considerable, depending on the location of the stable channel section relative to the outlet in both the vertical and downstream directions. View (A) of Figure 6-47 illustrates this condition.

A scour hole or localized erosion, as shown in view (B] of Figure 6-47, is to be expected if the downstream channel is stable. The severity of damage to be anticipated depends on existing conditions or those created at the outlet. In some instances, the extent of the scour hole may be insufficient to produce either instability of the embankment or structural damage to the out-

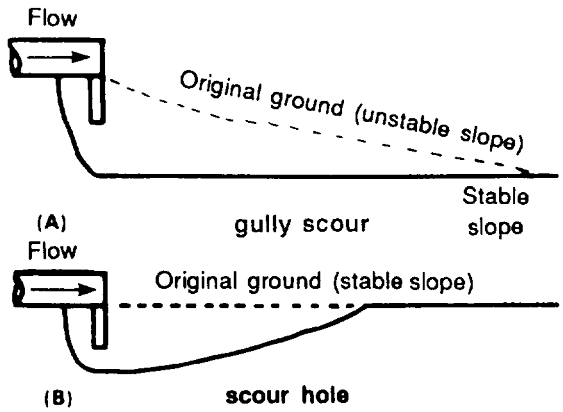


Figure 6-47. Types of scour at culvert outlets

let. However, in many situations, flow conditions produce scour that erodes the embankment and causes structural damage to the apron, end wall, and culvert. This type of outlet erosion of the bottom of the ditch and of the side banks is shown in Figure 6-48.

Erosion is best controlled by two methods: velocity reduction and channel protection. Reducing the slope of the culvert reduces the velocity. This solution, however, may require a larger pipe since it changes the capacity of the culvert.

Headwalls, wing walls, or channel linings can absorb the energy of flowing water and reduce its velocity to acceptable levels. When wing walls are not provided, line the stream bank and bottom for some distance downstream with riprap or other material. Ditch linings were described in greater detail earlier in this chapter.

CULVERT DESIGN

The hydraulic load on a culvert is the amount of water that will flow to the ulvert inlet, either as direct surface or channel-stream flow. Surface flow is determined by the rational and channel-flow-equations methods discussed earlier.

Hydraulics of Culverts

Culvert quantity of flow (Q) is the amount of water the culvert will carry in a unit of time. This capacity is expressed in cfs. For a particular culvert of known size (A), shape, and interior roughness (n), the discharge capacity is controlled by one or more of the following factors:

- Height of the water above the culvert inlet.
- Hydraulic gradient (S) of the culvert.
- Length (L) of the culvert.
- Elevation of the tailwater at the culvert outlet.

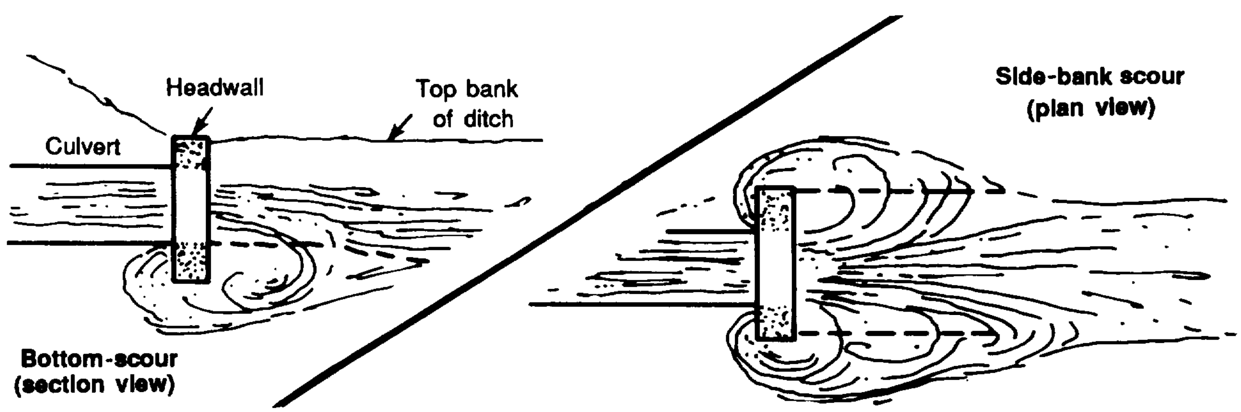


Figure 6-48. Outlet erosion

The type of inlet is not generally considered in military culvert design. However, it should be remembered that the discharge capacity of a culvert will be increased by a smooth, transition type of inlet with head-walls and wing walls. For construction in the TO, the culvert should have a design capacity sufficient to pass the peak runoff from the design storm.

Hydraulic Gradient

The hydraulic gradient (S) of a culvert is one of the culvert discharge capacity controls. It can be satisfactorily estimated as the slope in ft/ft. The gradient is calculated by dividing the head (H) on a culvert by the culvert length (L): S = H/L. The head is the difference in elevation between the following:

- The two ends of a culvert, if both the inlet and the outlet are not submerged.
- The water surface directly above the inlet and the top of the outlet, if the inlet is submerged and the outlet is not submerged.
- The water surface directly above the inlet and the outlet, if both the inlet and outlet arc submerged.

The hydraulic gradient and head are illustrated in Figure 6-49.

The normal flow pattern for culverts, for which Table 6-17 is used, is shown by view (A) of Figure 6-49, In this case, the water is at crown elevation al the inlet, and the outlet is free-flowing.

An accumulation of water at the inlet of the culvert is called *ponding*. When pending occurs, the outlet will normally be free-flowing, but the water will be at some depth above the inlet. When this depth at the inlet is 1.2D or less (where D is the diameter of the culvert), use Table 6-17 to directly determine Q and V. When the water depth at the inlet invert is greater than 1.2D and the outlet is free-flowing, use a nomograph (Figure 6-50, page 6-76) to determine Q, and use the continuity equation Q = VA to determine the velocity.

When both the inlet and the outlet are completely submerged, use Figure 6-50 exclusively.

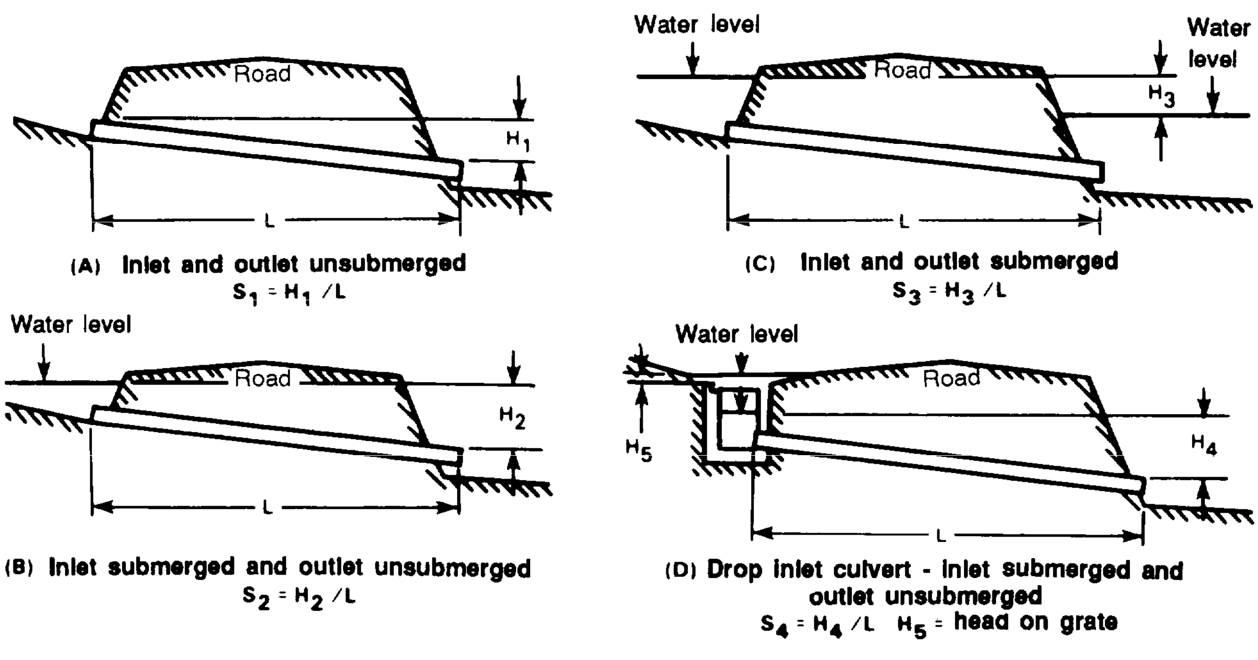


Figure 6-49. Hydraulic gradient, S, and heads, H, for culverts

*Table 6-17. Capacity and critical slope of culverts**
(Manning's n of 0.012, 0.018, and 0.024)

SLOPE (PERCENT)	DIAMETER OF PIPE (INCHES) 8	10	12	15	18	21	24	30	36	42	48	54	60	66	72	78	84	90	96
	n = 0.012																		
0.4	0.8	1.5	2.4	4.3	6.8	10	15	26	40	59	83	110	140	180	230	280	330	400	470
0.6	0.9	1.6	2.6	4.5	6.8	10	15	26	40	59	83	110							
0.8	0.9	1.6	2.6																
	n = 0.018																		
0.4	0.6	1.1	1.7	3.2	5.1	7.6	11	20	31	48	70	92	120	160	200	250	300	350	420
0.6	0.7	1.3	2.1	3.8	6.0	9.0	13	23	36	55	79	100	140	180	220	270	330	400	470
0.8	0.8	1.4	2.3	4.2	6.7	10	14	25	39	59	83	110	140	180	230	280	330	400	470
1.0	0.9	1.5	2.5	4.4	7.0	10	15	26	40	59	83	110	140	180	230				
1.2	0.9	1.6	2.5	4.5	7.1	11	15	26	40										
1.4	0.9	1.6	2.6	4.5	7.1	11													
1.6	0.9	1.6	2.6	4.5															
1.8	0.9																		
	n = 0.024																		
0.4	0.4	0.8	1.3	2.4	3.9	5.8	8.4	15	25	37	53	72	96	120	160	190	230	280	330
0.6	0.5	1.0	1.6	2.9	4.8	7.1	10	18	30	45	64	87	120	150	190	230	280	340	390
0.8	0.6	1.1	1.8	3.3	5.4	8.1	12	21	34	50	72	97	130	160	210	250	310	370	430
1.0	0.7	1.2	2.0	3.7	5.9	8.9	13	22	36	54	77	100	140	170	220	270	320	390	450
1.2	0.8	1.3	2.2	4.0	6.4	9.4	13	24	38	57	80	110	140	180	230	280	330	400	470
1.4	0.8	1.4	2.3	4.2	6.6	9.9	14	25	39	59	82	110	140	190	230	280	330	400	470
1.6	0.8	1.5	2.4	4.3	6.8	10	14	25	40	59	83	110	140	190	230	280			
1.8	0.9	1.5	2.5	4.4	7.0	10	14	26	40	59	83	110							
2.0	0.9	1.6	2.5	4.5	7.0	10	15	26	40										
2.2	0.9	1.6	2.6	4.5	7.1	10	15												
2.4	0.9	1.6	2.6	4.6	7.1	10													
2.6	0.9	1.6	2.6	4.6															
2.8	0.9	1.6	2.6																
3.0	0.9	1.6																	
3.2	0.9																		

*Culverts with free outlet with water surface at inlet same elevation as top of pipe and outlet unsubmerged.

NOTES:

1. Values are in cfs.
2. Heavy horizontal lines indicate "critical slope."
3. Steeper slopes than "critical" do not result in increased discharge.
4. Numbered "stepped" lines indicate appoximate velocities in fps.
5. Manning's n = 0.012 applies to clay or concrete pipe; excellent condition of surfacing alignment.
6. Manning's n = 0.018 applies to CMP; invert paved to 50% of diameter.
7. Manning's n = 0.024 applies to CMP; standard unpaved.

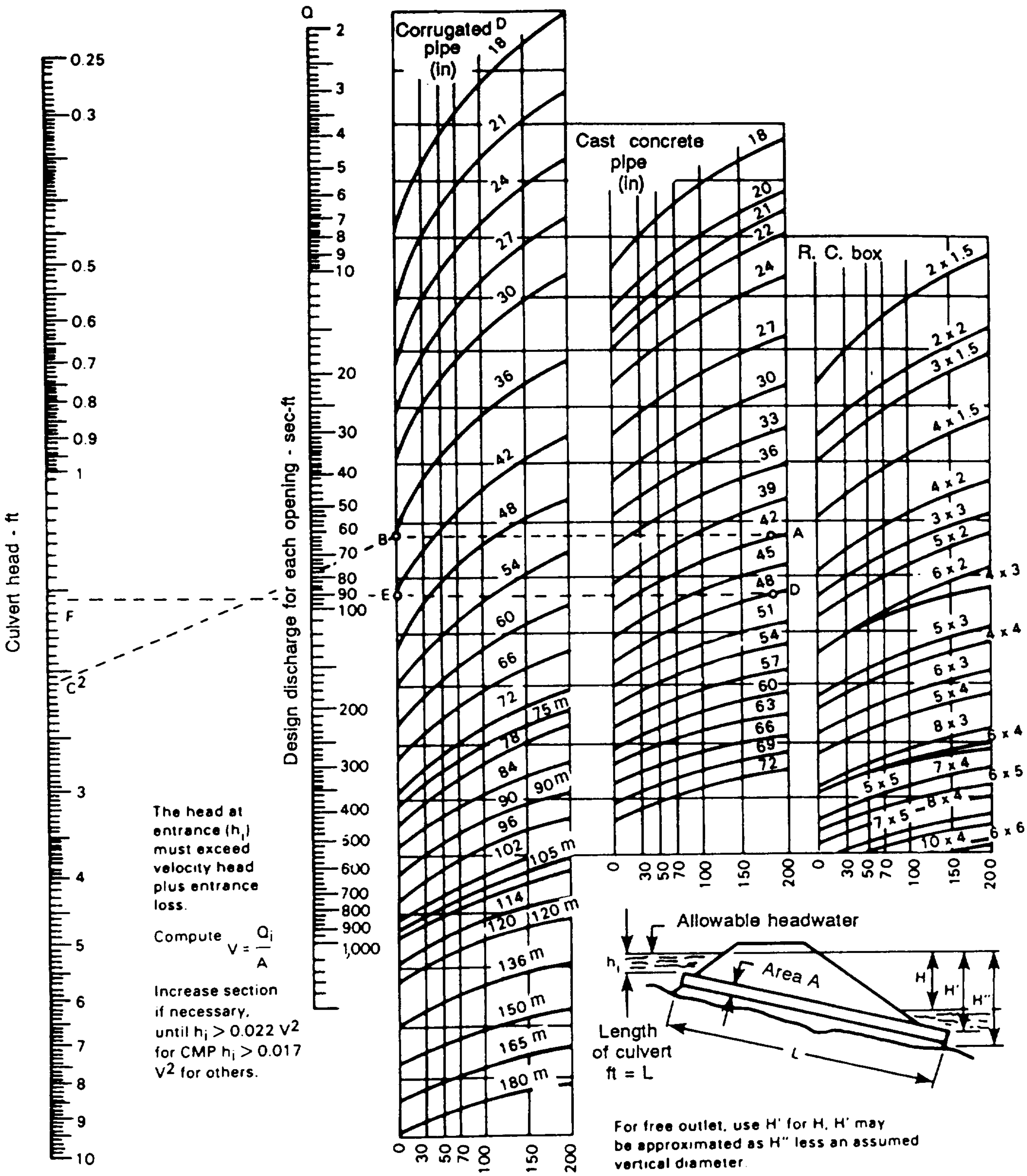


Figure 6-50. Nomograph for culverts flowing full with ponding inlets

Critical Slope

For a given size, A, of the culvert and a given head, H, on the culvert, the discharge capacity of the culvert will increase as the hydraulic gradient increases, This continues until the hydraulic gradient becomes equal to or greater than the critical slope, Sc. As the hydraulic gradient is increased beyond the critical slope, culvert discharge remains constant. The area decreases because the pipe does not flow full, and the velocity of flow in the culvert increases because V = Q/A. The critical slope is the maximum discharge.

Roughness

For the selection of the pipe roughness values (n values) for flow determination refer to the footnotes to Table 6-17, page 6-75.

Box-Culvert Flow

Flow characteristics of box culverts may be different from those of pipe culverts, even those with the same slope, lining, and inlet and outlet conditions. Flow, Q, can be determined as previously noted, for the conditions of inlet and outlet water elevations.

Assuming that the water elevation of the inlet is at the top of the box and the outlet is free-flowing, the difference in inflow characteristics between pipe and box culverts of the same material and slope is negligible. Box-culvert sizes can therefore be determined by computing the cross-sectional area required for a pipe and then desgning a box of the same material, slope and cross-sectional area.

When the water elevation is above the top of the box inlet, usc the nomograph in Figure 6-50. In this case, make trial solutions until there is correlation between the box size and pending depth.

Design of Culverts with Submerged Inlets

Submerging of the culvert inlet results in pending at-the site. The elevation of the pond surface, which will determine the depth of submergence, is a function of the extent of the pond, the requirement of safety to the structure, and the time it will take to empty the pond. These factors are determined by the runoff rate, the pond volume, and the culvert-flow rate. They are derived using the following steps:

Step 1. Determine the rate of runoff (Q) the culvert must drain or, in the case of pending, the inlet drain capacity, Q_d.

Step 2. Determine the length of the culvert.

Step 3. Determine the head on the culvert.

Step 4. Using Figure 6-49, page 6-74, determine the size and type of pipe or box culvert required to handle the quantity of flow, Q.

To use this table, enter the nomograph (Figure 6-50) at the intersection of the length for the type and size of culvert (for example, point A in the nomography). Extend the line horizontally to the turning-point line (point B in the nomography), and extend the line to the culvert head, H, in feet (point C in the nomography). Find the discharge, Q, in cubic feet per second, for the pipe selected at the intersection with the discharge line.

Step 5. Compute the discharge velocity, V, in fps. Use the equation, $V = Q/A$. If the discharge velocity is greater than the maximum permissible velocity for the outfall or the height of the water, in feet, above the top of the culvert inlet is less than $0.022(V^2)$ for CMP or $0.0\ 17(V^2)$ for concrete pipe or boxes, either select pipes of larger diameter or decrease the slope of the culvert.

Example (Submerged Inlet, Unsubmerged Outlet):

Determine the most economical pipe size and number of pipes required for a culvert across an airfield. The following are known conditions:

> The outfall from the culvert is a natural drainage channel with dense turf in a GP soil.

- The design flow, Q, is 210 cfs.
- 42-inch and 48-inch concrete pipes are available.

Solution:

Step 1. Q = 210 cfs (given).

Step 2. L = 180 feet (given).

Step 3. Determine head, H.

H 42-in = 598.6 ft - (593.0 ft + 3.5 ft]
= 2. 1 ft

H 48-in = 598.6 ft - (593.0 ft + 4.0 ft)
= 1.6 ft

Step 4. Determine the size and number of pipes required to handle the flow, Q.

(a) On the cast-concrete-pipe portion of the nomograph, draw a horizontal line from the intersection of the 180-ft L line and the 42-in pipe line (point A) to the turning-point line (point B). From B, draw a line to an H of 2.1 ft (point C). The intersection of this line with the “Q” portion of the nomograph shows the maximum discharge of one 42-in pipe to be about 78 cfs. For 210 cfs, three would be required.

(b) Similarly, for the 48-in pipe, 180 ft length, and 1.6 ft H, proceed from D to E to F and find that one 48-in pipe would have a capacity discharge of about 92 cfs. Again, three would be required for 210 cfs.

Step 5. Compute the discharge velocity, V, and check for excessive outlet velocity. Check smaller pipe.

(a) Since three pipes are used, assume each will carry one-third the total Q or 70 cfs, whichever is greater.

(b) If the pipe flows full, the exit velocity is—

$$V = \frac{Q}{A} = \frac{78}{3.14(1.75)^2} = 8.1 \text{ fps}$$

(c) If the exit velocity is based on a design flow of 70 cfs, the pipe would be flowing only partially full and the exit velocity would be 9.2 fps. This velocity is greater than when the pipe flows full because the resistance to flow decreases until the pipe is flowing approximately 0.8 full.

(d) Check to see that $H_1 \geq 0.017(V^2)$.

H_1 = 598.6 ft - (594.0 ft + 3.5 ft) = 1.1 ft
and $0.017 \times 9.2^2 = 1.44$

(e) Since allowable outfall velocity is exceeded and $H_1 < 0.017\ (V^2)$, use three 48-inch pipes as the most economical available size.

Design of Pipe Culverts with Unsubmerged Inlets

The factors to be applied to the design of these culverts arc determined as follows:

Step 1. Determine the rate of runoff. Use the area the culvert must drain. This will be the required capacity, Q_p, of the culvert.

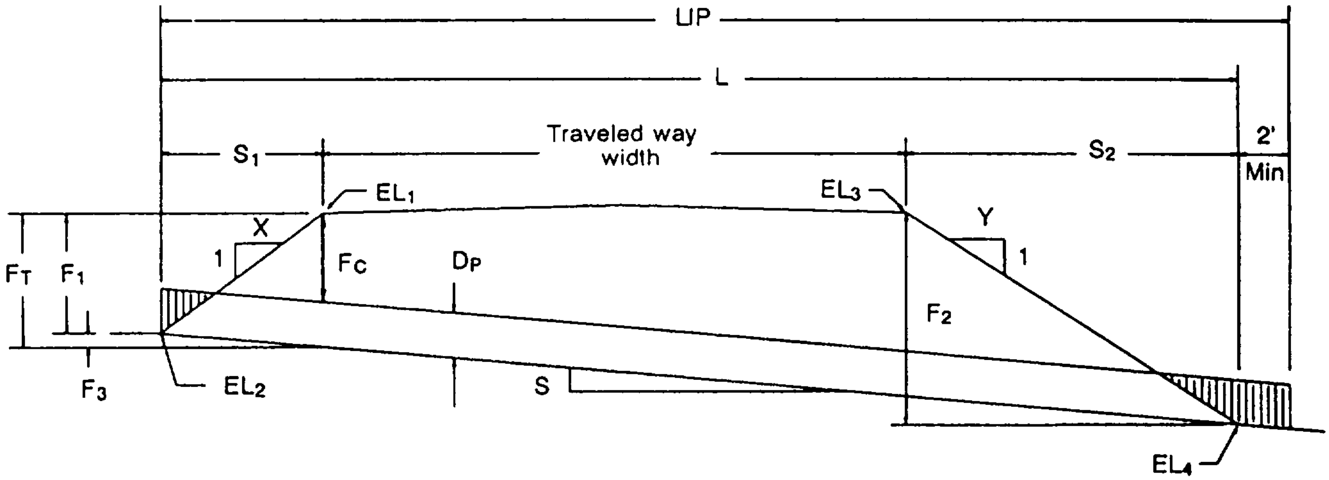
Step 2. Determine culvert use. Will it be used for a road or for an airfield?

Step 3. Calculate the critical dimensions from the cross sections. See Figure 6-51. Determine the length in place (LIP) and the fill critical (F_C).

Step 4. Determine the largest pipe for the fill. Begin from the cross section at the outside edge of the shoulder. Consider only pipes that are available and for which cover is adequate.

For roadway loadings, the maximum culvert diameter is equal to two-thirds of the minimum fill (F_T from Figure 6-51, page 6-79). The cover required for culvert protection is equal to one-half the diameter of the culvert or 12 inches (whichever is greater).

For runways and taxiways sustaining aircraft wheel loads, refer to Table 6-12, page 6-65, for the wheel loads and Table



$F_1 = EL_1 - EL_2 =$ ______________

$S_1 = F_1 \times X =$ ______________

$F_2 = EL_3 - EL_4 =$ ______________

$S_2 = F_2 \times Y =$ ______________

$$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2} = \text{-----------} = \text{\_\_\_\_\_\_\_\_\_\_}$$

$F_3 = S \times S_1 =$ ______________

$F_T = F_1 + F_3 =$ ______________

$LIP = S_1 + TW + S_2 + 2' =$ ______________

$F_C = F_T - D_P$

- F_1 (Fill 1) = roadway elevation at the outer edge (EL_1 - the elevation at the toe of the slope (EL_2),
- S_1 (Slope horizontal distance on the Inlet side of the road) = Fill 1 x the "X" value (horizontal component of the Inlet side slope ratio).
- F_2 (Fill 2) = roadway elevation at the outer edge of the road on the outlet side (EL_3) – the elevatlon at the toe of the slope on the outlet end,
- S_2 (Horizontal distance of the roadside slope on the outlet end) = Fill 2 x the "Y" value (horizontal component of the outlet side slope ratio).
- S (Slope) = the difference between EL_2 and EL_4 divided by the sum of S_1 + TW (top width across the road) + S_2.
- F_3 (Fill 3) = fill amount from the existing ground on the inlet side to the culvert inlet.
- F_T (Fill Total) = Fill 1 + Fill 3.
- LIP (Length In Place) = S_1 + TW + S_2 + 2^1.
- F_C ((Fill Critical) = Fill total minus DP (diameter of the pipe).

Figure 6-51. Calculating cover

6-13, page 6-66, or Table 6-14, page 6-67, for the minimum cover required.

Step 5. Determine the culvert capacity, Q_p, and outlet velocity. Use values based on culvert material and slope and Table 6-17, page 6-75.

Step 6. Determine the number of culvert pipes required. Divide the area runoff, Q, by the pipe capacity, Q_p. Round up to the next whole number.

Step 7. Determine the order length (OL). The OL is calculated by multiplying the number of pipes (NP) times the LIP times a waste factor of 1.15, (See step 3 for the LIP and step 6 for the NP.)

Step 8. Determine the maximum permissible discharge velocity, V_{max}. Use Table 6-6, page 6-44, to calculate V_{max} for the channel lining into which the culvert outlet will discharge. Determine the correct pipe to be used. Apply the following criteria in your calculations:

- Be sure the outlet velocity is equal to or less than the maximum velocity of the channel lining into which the culvert outlet will discharge, If outlet velocity exceeds the soils V_{max}, the outlet must be protected against erosion.
- Use the least number of culvert pipes possible to carry the total flow and still be consistent with the above criteria.

Example (Unsubmerged Inlet):

Determine the most economical pipe size and the quantity of pipe required for a culvert located under a runway, with the general data cross section given below. The maximum using aircraft weight cIass for this example will be an SR-71C.

No headwall will be constructed down-stream. The following are known conditions:

- Aircraft SR-71C. (Refer to Tables 6-12 and 6-13, pages 6-65 and 6-66.)
- Culvert weight type = 9.
- Q to be handled by culvert = 32 cfs.
- Soil type is bare SC; therefore, the maximum allowable outlet velocity = 3-4 fps.
-
- Pipe sizes available: 24-, 30-, 36-, and 42-inch CMP, 10 gage.

Referring to Figure 6-52, note that the following information is required before design can be accomplished:

- Horizontal length of one culvert, L.
- Slope of culvert pipe needed to determine the flow characteristics of various pipe diameters (see Table 6-17, page 6-75) and F_c, which is a portion of the critical fill depth, F_t.
- Critical fill depth, F_t, in order to determine if the cover over the pipe meets the requirements of Table 6-13.

Solution:

Step 1. Determine the runoff rate. Given for this example, Q = 32 cfs.

Step 2. Determine airfield culvert use.

Step 3. Calculate the critical dimensions. Computations should be made in the following sequence (see Figures 6-52 and 6-53):

- Determine the difference in elevation between the edge of the runway and the culvert upstream invert.

$F_1 = EL_1 - EL_2$

$F_1 = 607.00 - 600.80 = 6.20$ ft

- Determine the horizontal distance between the culvert invert and the shoulder edge.

$S_1 = F_1 \times X$

$S_1 = 6.20 \times 10 = 62$ ft

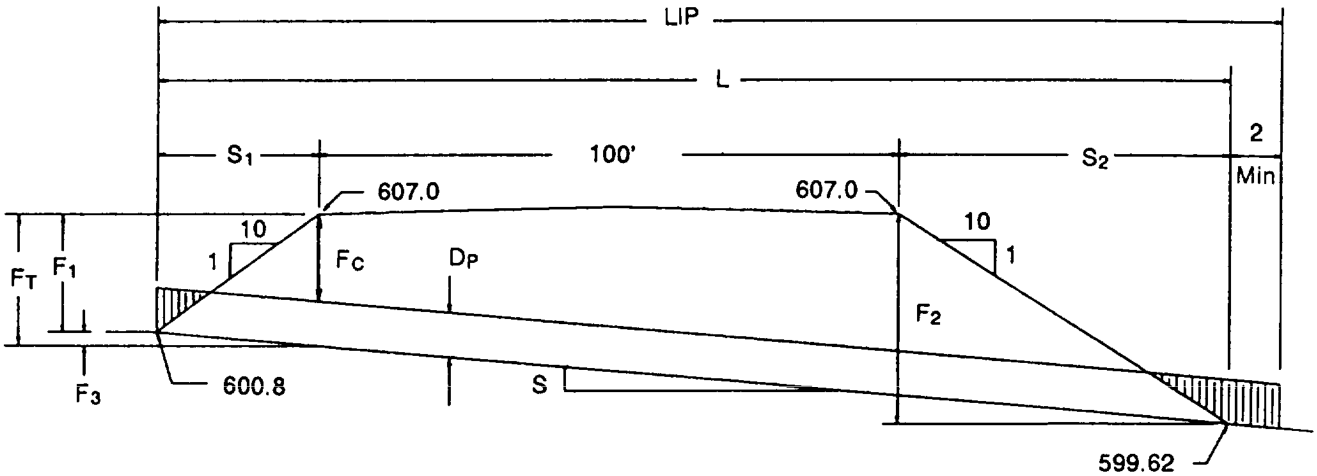


Figure 6-52. Unsubmerged inlet culvert problem

$F_1 = EL_1 - EL_2 = 607.0' - 600.8' = 6.2'$

$S_1 = F_1 \times X = 6.2' \times 10 = 62'$

$F_2 = EL_3 - EL_4 = 607.0' - 599.62' = 7.38'$

$S_2 = F_2 \times Y = 7.38' \times 10 = 73.8'$

$F_3 = S \times S_1 = 0.005 \times 62' = .31'$

$F_T = F_1 + F_3 = 6.2' + .31' = 6.51'$

$LIP = S_1 + TW + S_2 + 2' =$

$62' + 100' + 73.8' + 2' = 237.8' \approx 238'$

$$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2} = \frac{600.8 - 599.62}{62 + 100 + 73.8} = \frac{1.18}{235.8} = 0.005 \text{ OR } 0.5\%$$

$F_C = F_T - D_P$

- F_1 (Fill 1) = roadway elevation at the outer edge (EL_1 – the elevation at the toe of the slope
- S_1 (Slope horizontal distance on the Inlet side of the road) = Fill 1 x the “X” value (horizontal component of the inlet side slope ratio).
- F_2 (Fill 2) = roadway elevation at the outer edge of the road on the outlet side (EL_3) – the elevation at the toe of the slope on the outlet end.
- S_2 (Horizontal distance of the roadside slope on the outlet end) = Fill 2 x the “Y” value (horizontal component of the outlet side slope ratio).
- S (Slope) = the difference between EL_2 and EL_4 divided by the sum of S_1 + TW (top width across the road) + S_2.
- F_3 (Fill 3) = fill amount from the existing ground on the inlet side to the culvert inlet
- F_T (Fill Total) = Fill 1 + Fill 3.
- LIP (Length In Place) = S_1 + TW + S_2 + 2'.
- F_C (Fill Critical) = Fill total minus DP (diameter of the pipe).

Figure 6-53. Calculating cover (sample problem)

- Determine the difference in elevation between the edge of the runway and the culvert outlet and fill slope.

$F_2 = EL_3 - EL_4$
$F_2 = 607.00 - 599.62 = 7.38$ ft

- Determine the horizontal distance between the edge of the runway and the intersection of the culvert and fill.

$S_2 = F_2 \times Y$
$S_2 = 7.38 \times 10 = 73.8$ ft

- Determine the actual slope of the culvert.

$$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2}$$

$$S = \frac{600.8 - 599.62}{62 + 100 + 73.8} = \frac{1.18}{235.8}$$

$= 0.005$ or 0.5%

- Determine the incremental elevation difference between the elevation of the upstream invert and the invert of the culvert at the critical fill section.

0.005 ft/ft = 0.5 percent
$F_3 = S \times S_1$
$F_3 = 62 \times 0.005 = 0.31$ ft

- Determine the depth of critical fill section.

$F_t = F_1 + F_3$
$F_t = 6.2 + 0.31 = 6.51$ ft (Lenglh of horizontal projection of the culvert.) The upstream invert elevation to downstream invert elevation is—

LIP = $S_1 + TW + S_2 + 2$ ft
LIP = 62 + 100 + 73.8 + 2 = 237.8 ft: (Round Up to 238 ft)

NOTE: These computations depict the cross section and Figure 6-53, page 6-81, summarizes the calculations.

From these calculations, it has been determined that—

- The actual length of the culvert is 235.8 feet, rounded up to an even value of 236 feet. A 2-foot projection is added because there is to be no downstream headwall. This gives a culvert length of 238 feet for a single pipe.
- The slope of the culvert is 0.005 foot per foot or 0.5 percent.
- The depth of fill at the critical section is 6.5 feet (rounded to the nearest tenth of a foot).

Step 4. Determine the largest pipe for the fill. Prepare a table as shown in Table 6-18 and fill in all known values. Start by entering the largest pipe available, D_P, into the table. Subtract Dp from total fill, F-T, to get fill critical, F_C. F_C represents the actual cover over the pipe at the critical section. Compare F_C to cover required, C_R.

To find C_R for aircraft, refer to Table 6-12, page 6-65, which indicate that an SR-71C aircraft has a culvert weight type (WT) of 9. The CMP available is 10 gage. With this information, refer to Table 6-13, page 6-66, and select chart 9, corresponding to culvert WT 9. The diameters and cover are given in chart 9 under the 10-gage line. Starting with the largest pipe available, 42-inch or 3.5 feet, C_R is 4.0 feet (interpolated between pipe diameters.) Enter this value in Table 6-18. The 42-inch pipe (3.5 feet) cannot be used because its actual cover is 3.01 feet against a required cover of 4.0 feet. Repeat the process using the next smaller pipe available, 36-inch pipe. It has a F_C of 3.51 feet and a C_R of 3.5 feet. This pipe will work.

Step 5. Determine the culvert capacity. Use Table 6-17, page 6-75, to find the capacity and velocity for the 36-inch pipe.

Table 6-18. Unsubmerged inlet sample data

$F_C = F_T - D_P$

Total Fill	F_T	6.51	6.51		
Pipe Dia	D_P	3.5	3.0		
Fill Critical	F_C	3.01	3.51		
Cover Reqd	$*C_R$	4.0	3.5		

*For aircraft, see Tables 6-12 and 6-13, pages 6-65 and 6-66; for roads, 2/3 F_T.

The slope of the culvert is 0.5 percent. Entering the graph from the top (pipe dameter), move down the column until you intersect with the slope in the left-hand column. This will slate the quantity of flow in the pipe. The velocity of flow is shown by following the respective shaded or unshaded areas down and to the left until it ends in the velocity column, 4 feet per second.

The 36-inch pipe has a capacity of 27.5 cubic feel per second for the 0.5 percent slope. The velocity indicated is 4 feet per second.

Step 6. Determine the number of pipe required to carry the flow by dividing the total (Q) of 32 cubic feet per second (step 1) by the pipe capacity, 27.5 cubic feet per second (step 5) and rounding up to the next whole pipe. For this example, the number of pipe required is 32/27.5 = 1.2 or two 36-inch diameter pipes.

Step 7. The next step is determining the length of pipe to be ordered. The order length is calculated by multiplying NP (Step 6) times LIP (step 3) times a waste factor (WF). Since pieces of material will be damaged in manufacturing, handling, transporting, and assembling, an additional amount over the actual in-place length will be required. This value has been determined to be 15 percent of the total length of pipe required. For this project, the pipe selected will be 36 inches in diameter with a length in place of 476 feet. The length to order will be—

NP x LIP x WF
2 x 476 x 1.15 = 1,094.8 ft

The pipe comes in 2-foot increments; therefore, the value of 1,094.8 is rounded up to the next even value, or 1,096 feet of pipe.

PONDING AREAS

Ponding is the accumulation of runoff at the inlet of a drainage structure resulting from the inability of the system to discharge more of the runoff than the rated capacity of the structure.

Military drainage structures in the TO are designed to discharge the runoff based on the 2-year design storm which, by definition, is expected to be equaled or exceeded at least once in the design period. Thus, a storm more severe than the design storm may occur and generate excessive runoff, overloading the drainage structures. In anticipation of this event, design the areas around the inlets of drainage structures to accommodate a certain amount of pending.

In some cases, due to limitations in culvert cover and space, terrain conditions, time, materials, or other conditions, a system may not be able to take care of the runoff from the design storm. To provide for this, include sufficient pending areas in the original plan to prevent flooding of vital areas. The pending areas store excess runoff until the intensity of the storm decreases and the structure can handle the flow.

When possible, military drainage systems arc designed to pass the runoff from the design storm without pending. However, some provision for pending may be made in those areas where flooding for a period of time will not affect the facility's operational status.

The following specifications are generally adhered to in the design of pending areas for military installations:

- The edge of the pond must be at least 75 feet from the edge of the pavement when used for runway and taxiway design.
- When a pond is used with road fills or embankments, the depth of the pond will be determined by the porosity of the fill, the amount of freeboard required to prevent overtopping of the road, the height of the headwall and wing wall, and the time allowed for the pond to empty. When pending is anticipated, the adjacent fill side slope should be made less steep to prevent the sloughing of saturated fill.
- The pond must be drained before damaging infiltration of the subgrade can occur. The actual time during which pending is allowable will depend upon the type and condition of the soil in the pending area and the embankment. In general, this period will be no more than four hours from the start of the storm.

In designing pending areas, the following assumptions are made to simplify the calculations and to retain satisfactory accuracy:

- A culvert discharges its design capacity before runoff starts to accumulate at the inlet and form a pond.
- An increase in the head because of pending does not increase the discharge capacity of the culvert.

The determination of permissible volume, the preparation of runoff curves, and the soil analysis must be done before designing pending areas to meet the above specifications and assumptions.

PERMISSIBLE VOLUME

The volume of permissible pending is determined by the elevations of the area available for such pending and the surrounding areas. A contour map showing the final grading plan is required to compute the volume of permissible pending. By inspection, a contour line may be selected to provide a pending area located a safe distance or elevation from the pavement. Pending volumes may be computed from the con tour map by the average-end-area method. This method is the average of the areas, in square feet, enclosed by two adjacent contour lines and multiplied by the contour interval in feet.

$$V = \frac{A + B}{2} x\ b$$

where—

V = volume in cubic feet

A = area of the first contour in square feet

B = area of the next contour in square feet

b = vertical distance, in feet, between contours (contour interval)

As an example of computing the volume for pending, consider the contours shown in Figure 6-54. Water can be safely ponded to the 66-foot contour line. The bottom of the inlet end of the culvert is at an elevation of 62 feet. Use a planimeter, or any other method, to determine the total area enclosed by each contour. In this case, the 66-foot contour line encloses 25,000 square feet. The 64-foot contour line encloses 10,000 square feet. It should be noted that the contours are concentric: the 66-foot contour -line area includes the area bounded by the 64-foot contour line.

$$\text{Volume 62-64} = \frac{10{,}000 + 0}{2} x\ 2$$

$$= 10{,}000 \text{ cubic feet}$$

$$\text{Volume 64-66} = \frac{25{,}000 + 10{,}000}{2} x\ 2$$

$$= 35{,}000 \text{ cubic feet}$$

The total volume of the pending area is—

10,000 + 35,000 = 45,000 cubic feet

A further example of the computation of pending volumes by the average-end-area method is shown in Figure 6-55, page 6-86. Assume that the pending area extends to the 68-foot contour line. Determine the volume of the pond. The 68-foot line encloses a total area of 30,000 square feet.

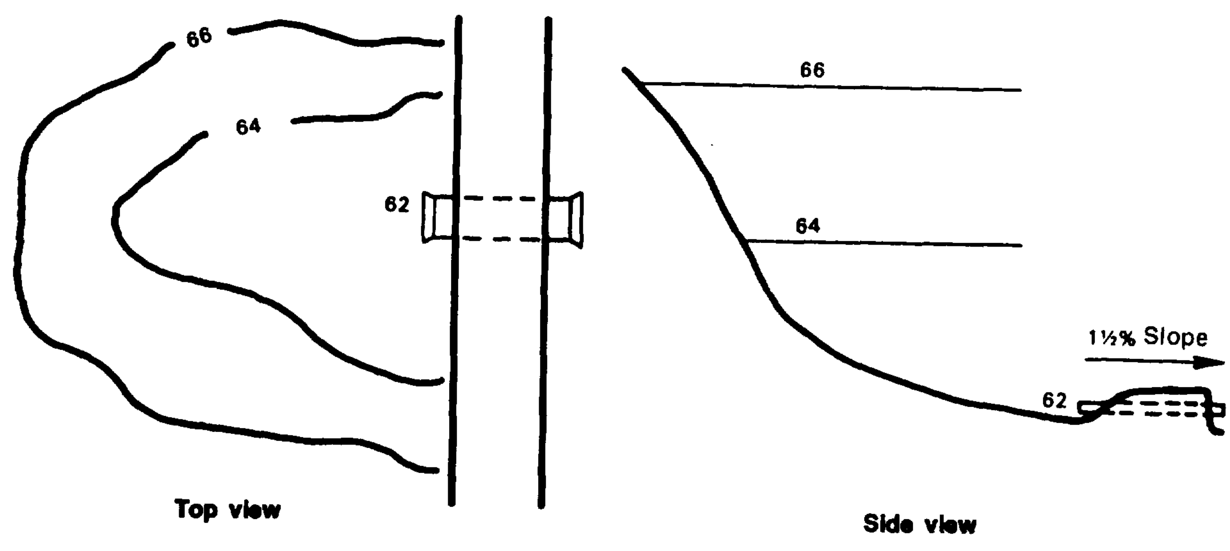


Figure 6-54. Pending area

Volume 62-64 = 10,000 cubic feet

Volume 64-66 = 35,000 cubic feet

Volume 66-68 $= 2 \times \frac{30,000 + 25,000}{2}$

= 55,000 cubic feet

The total volume available for pending will be 10,000 + 35,000 + 55,000 = 100,000 cubic feet, if the pending area extends to the 68-foot contour line.

RUNOFF CURVES

To determine the amount of water an area will contribute to a pond, a cumulative runoff curve must be plotted. The following example shows how such a curve is prepared:

Example:

An area has 33.3 acres, consisting of 23.3 acres of impervious soil and 10 acres of paved surface. The weighted C value is 0.75, the TOC is 13 minutes, and the location intensity is 2.2 inches per hour for a 1-hour storm with a 2-year frequency.

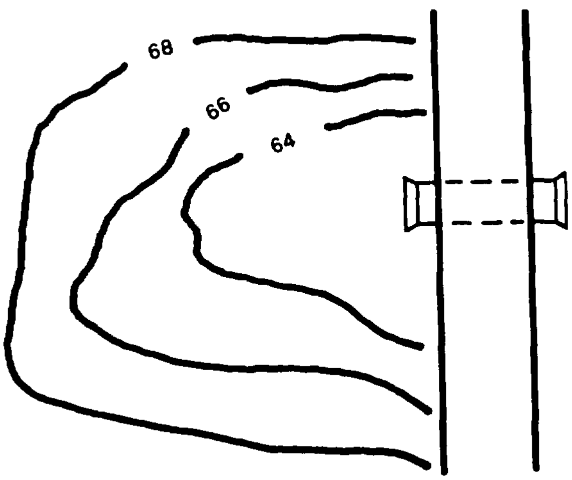


Figure 6-55. Pending area enlarged

Prepare a cumulative runoff curve based on the-data given. Pending may be required to reduce the culvert size.

Tabulation of Data—Rational Method

All data required to plot a cumulative runoff curve, using the rational method, is shown in Table 6-19. Each column is prepared as follows:

Column 1 is a tabulation of time in minutes. Any similar combination of time increments can be used as long as enough properly spaced points are obtained to plot a smooth curve. The cumulative runoff curve is constructed by plotting time, in minutes (column 1), against volume in cubic feet (column 6).

Column 2 is the product of the runoff coefficient, C (as determined for the area using the rational method), and the area in acres.

Column 3 is the intensity, in inches per hour, for each one of the minute values of column 1. These values are obtained from the standard intensity-duration curve number 2.2 in Figure 6-4, page 6-9. The value of 2.2 is the location intensity, in inches per hour, of the 1- hour, 2-year storm as specified in the given conditions.

Column 4 is the rate of runoff, Q, in cfs, for the entire interval of time shown in column 1. It is obtained by multiplying columns 2 and 3 (Q = CIA).

Column 5 is the time, in seconds, given in column 1.

Column 6 is the quantity of water supplied to the pond for the time given in column 1. For the first five minutes of rainfall, the quantity of water entering the pond is given by—

Column 6 = (column 4)(column 5)

Column 6 = (163)(300) = 48,900 cubic feet

Table 6-19. Cumulative runoff data tabulation, rational method

1	2	3	4	5	6
Time (min)	**C x area 0.75 x 33.3**	**Intensity (in/hr)**	**Q (cfs)**	**Time (sec)**	**Volume (cu ft)**
5		6.5	163	300	48,900
10		5.5	138	600	82,800
15		4.6	115	900	103,500
20		4.2	105	1,200	126,000
30		3.4	85	1,800	153,000
40		2.9	72.5	2,400	174,000
50		2.5	62.5	3,000	187,500
60		2.2	55	3,600	198,000
120		1.3	32.5	7,200	234,000
180		1.0	25	10,800	270,000

After 20 minutes from the start of rainfall, the volume of water would be equal to 105 x 1,200 or 126,000 cubic feet, if no water was released from the pond.

Preparation of Cumulative Runoff Curve

The cumulative runoff curve is obtained by plotting the volume (column 6, Table 6-19) obtained by the rational method, on the vertical axis and the time, in minutes (column 1, Table 6-19), on the horizontal axis. The cumulative runoff curve for the data listed in Table 6-19 is shown in Figure 6-56, page 6-88.

Analysis of Cumulative Runoff Curve

Based upon the data given in the problem, the flow for the culvert design shown in Figure 6-55 would be determined as follows:

$$Q = CIA$$

where—

C = 0.75

I = 4.9 for a TOC of 13 minutes and a location intensity of 2,2 inches per hour for the 1-hour, 2-year storm

A = 33.3 acres

Q = 0.75 x 4.9 x 33.3 = 122.4 cfs. The number of culverts at a slope of 1.2 percent (Figure 6-55) when Q = 122.4 would be as follows:

30-inch pipe at 24 cfs = 6 pipes
36-inch pipe at 38 cfs = 4 pipes
42-inch pipe at 57 cfs = 3 pipes
48-inch pipe at 80 cfs = 2 pipes
60-inch pipe at 140 cfs = 1 pipes

These pipes in the above sizes and quantities will pass the flow without pending.

ANALYSIS FOR PONDING

The safe volume for pending is 100,000 cubic feet. The requirement is to reduce the flow past the outlet point by using a single culvert and allowing pending at the inlet.

Table 6-17, page 6-75, reveals that a 30-inch CMP culvert (n = 0.024) on a 1.2-percent slope discharges 24 cfs. Assuming that the pipe always discharges at the rated capacity, the cumulative discharge is a straight-line function for any time interval with a line passing through the origin.

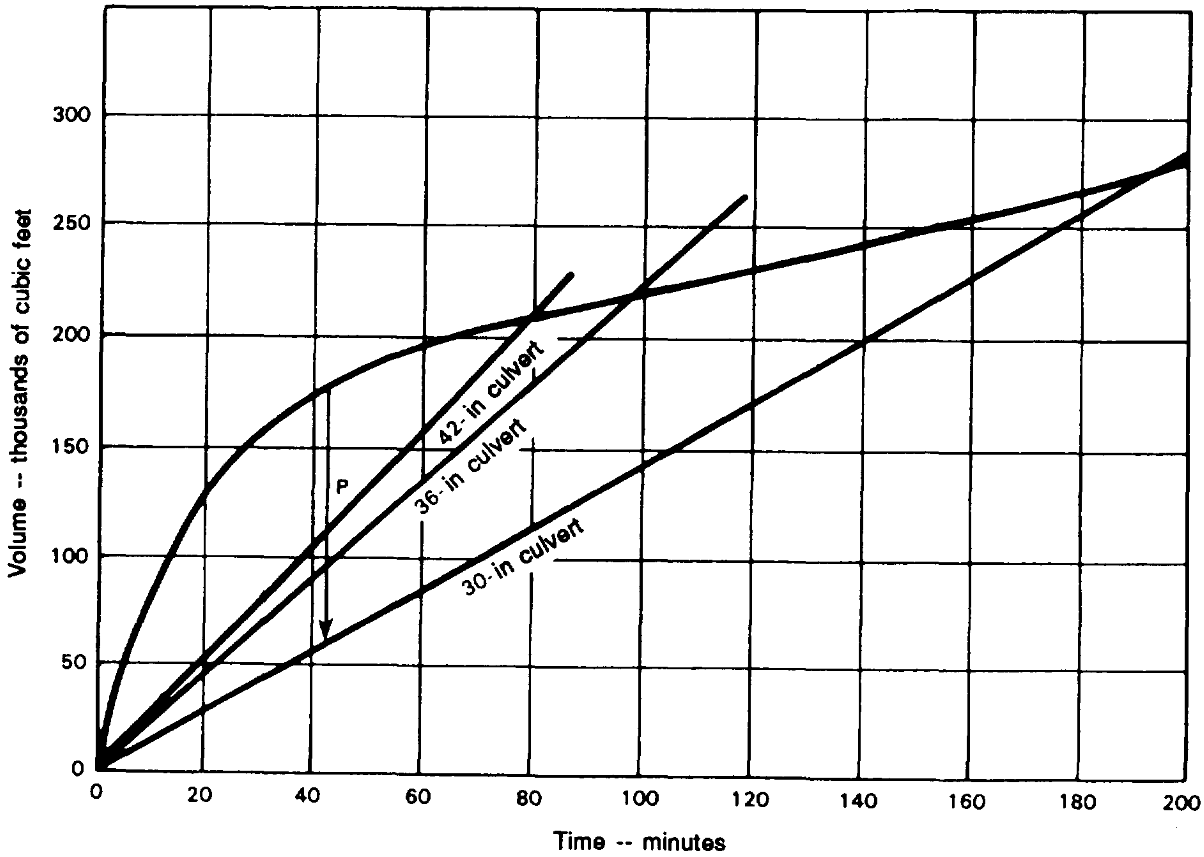


Figure 6-56. Cumulative runoff curve

Continue these straight lines so that they intersect thew cumulative runoff curve. At the point of intersection. the cumulative funff will have equaled the cumulative amount of water passed by the culvert. From then on, there will be no pending.

Knowing how long the pond will exist behind the inlet, next determine whether or not the pending area is large enough. Figure 6-56 shows that the greatest vertical distance betweem the cumulative runoff curve and the 30-inch cumulative discharge volume is line P. Line P represents the maximum volume of water ponded behind the 30-inch culvert.

Measuring line P by the vertical scale used in Figure 6-56 indicates that the maximum pending volu une will be 115,000 cubic feet. a 42 minutes the cumulative supply curve shows that 175,000 cubic feet of water have been supplied to the pond by the rainstorm. At the end of 42 minutes, the 30-inch culvert has theoretically been able to discharge 60,000 cubic feet. Therefore, the difference between the quantity supplied (175,000 cubic feet) and the quantity discharged (60,000 cubic feet) is 115,000 cubic feet, which must still be in the pond.

In view of the fact that the safe pending volume is only 100,000 cubic feet, 30-inch CMP is unsatisfactory because the safe pending volume would be exceeded.

Make the same calculations for 36-inch culvert. In this case, the pond volume will be 75,000 cubic feet with a pond time of 95 minutes. Since the safe pending volume of 100,000 cubic feet and the 4-hour limit on the pending time arc not exceeded the 36-inch culvert is satisfactory. In addition, the excess volume of 25,000 cubic feet will be available for storms that may exceed the design storm.

ADVANTAGES OF PONDING

Although pending is an added safeguard against the effects of storms more severe than the design storm, its primary use is as an economy measure. Pending allows for reductions in the size of culvert pipe necessary to handle runoff. However, if additional pipe or other construction is required for pending, check the additional cost against the savings in reduction of pipe size. Ponding appreciably reduces pipe sizes for areas that have a short TOC. For longer TOCs, pending has little or no effect on pipe sizes. The use of pending as an economy measure is often restricted by the area available for pending. This area should be sufficient to satisfy the requirements of the design storm, It should also have enough reserve capacity to take care of storms more intense than the design storm, Facilities for pending should be coupled with initial grading operations, if possible, to secure the most efficient use of personnel and equipment.

DROP INLETS AND GRATINGS

Drop inlets, vertical entrances to a culvert or a storm drain, may be used to lower the elevation of the culvert inlet below ditch elevation. This is done where fill does not provide sufficient cover or where discharge velocity is erosive and can be controlled only by changing the slope. For storm drains (underground culverts or conduits designed to carry surface runoff), drop inlets are used to collect surface runoff from paved and turfed areas, street gutters, and ditches. A typical drop inlet is shown in Figure 6-57, page 6-90.

A framework of bars, or a perforated plate called a grating] passes the storm runoff into a drop inlet. The grating serves as both a filter and a cover for the inlet. Gratings arc shown in Figure 6-58, page 6-91.

CONSTRUCTION OF DROP INLETS AND GRATINGS

Drop inlets should always be protected with some type of grating. An expedient grating can easily be fabricated using reinforcement bars welded together. These gratings should be spaced to readily admit debris which will pass unobstructed through the culvert. A drop inlet may be constructed of concrete, brick, timber, or CMP sections.

Inlet grating should be fabricated of steel bars, steel plate, cast iron, or reinforced concrete with adequate strength to withstand the anticipated load. The loads may range from the weight of debris collected over the grating to vehicle or aircraft wheel loads.

Inlet grating should be placed 0.2 foot below the grade. This will allow for the settlement of the area around the grating and will provide a sumped area to ensure complete drainage around the grating and positive interception of surface and gutter runoff.

To determine the proper size of grating-

1. Determine the peak rate of runoff, Q, from the area that drains into the drop inlet. This will be the required discharge capacity, Q, of the grating.

2. Determine the load (depth of water or head) on the grating at times of peak runoff. When a drop inlet is used in a ditch, gutter, or pending area, the head will be the depth of water running in the ditch or gutter, or the depth of the pond.

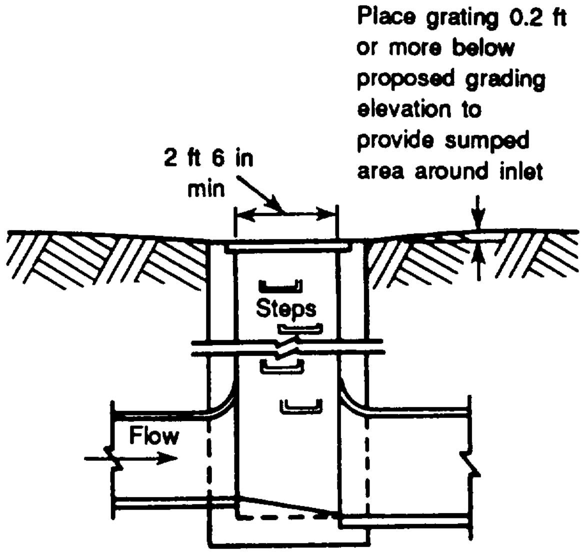
3. Knowing the head, H, and the design discharge capacity, Q, refer to Table 6-20, page 6-91, to select the minimum size grating required.

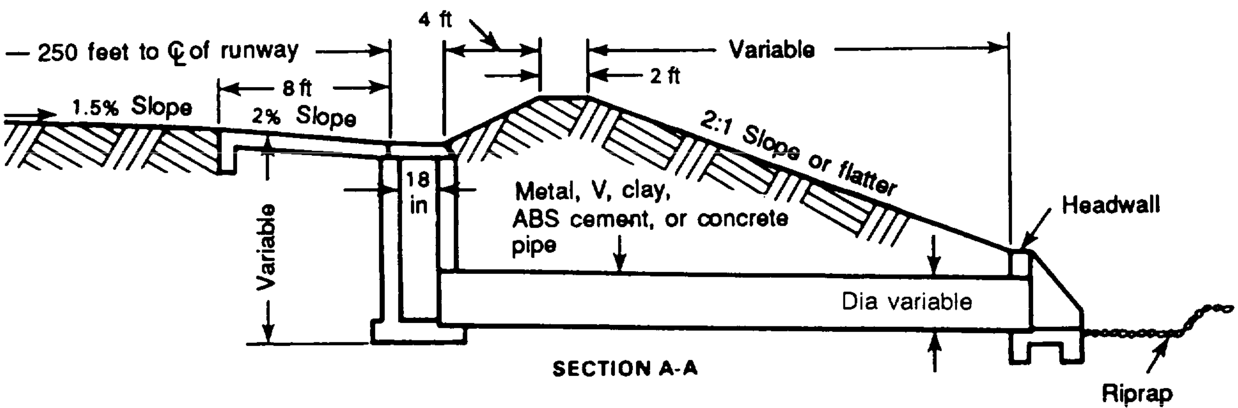
4. Multiply the required grating size by the appropriate safety factor to determine the actual grating size to use.

A safety factor of 50 percent (1.5 x total grating area) for paved areas and 100 percent (2 x total grating area) for turfed areas should be added to the grating-area measurement to compensate for debris caught in the openings.

Grating openings should be at least 18 inches long and should be placed parallel to the direction of flow.

The area of the grating openings is estimated at 50 percent of the total grating area. When using bars with large openings, determine the area of grating openings.





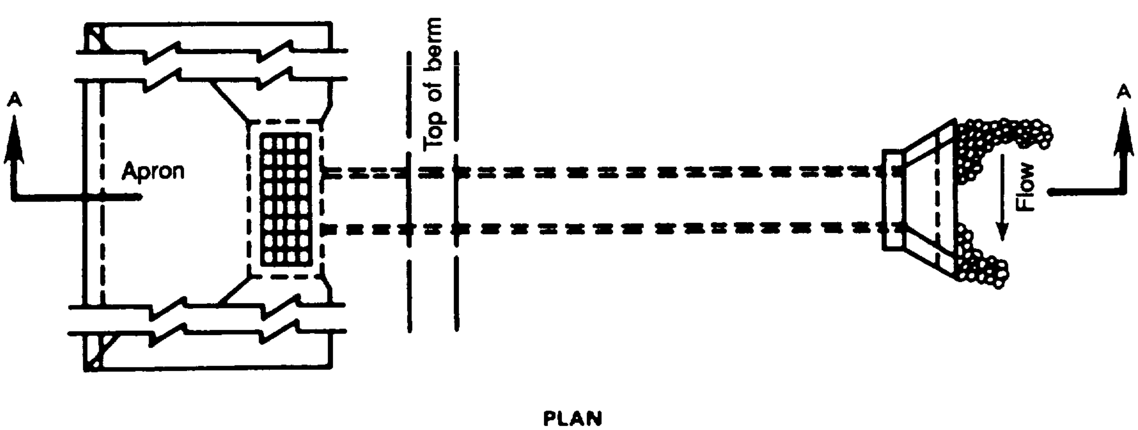


Figure 6-57. Drop inlets

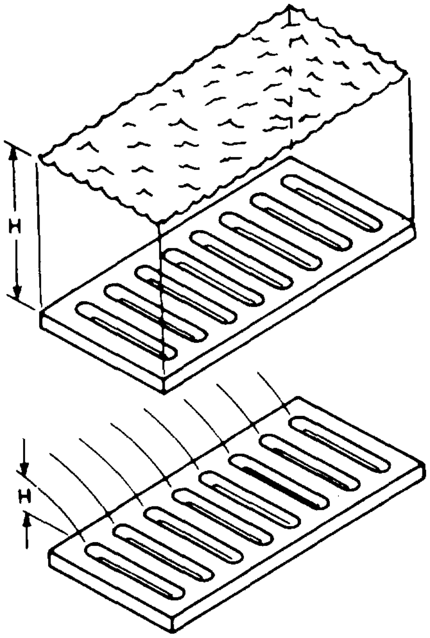


Figure 6-58. Grating heads

MAINTENANCE OF DROP INLETS AND GRATINGS

Drop inlets and gratings require continuous maintenance because debris collects at the openings. A maintenance schedule must be established to check and clean debris from the inlets and gratings. If these openings are not kept clean, a pond could form at the inlet, resulting in damage to nearby structures. This damage could consist of saturation of the subgrade by the pond or direct flooding of adjacent areas. Periodic maintenance of drop inlels should include removing the cover and inspecting and cleaning the chamber.

Expedient drop inlets of the type shown in Figure 6-39, view (A), page 6-63, must be covered with bars, If the box is left open, it will tend to fill with debris making cleaning difficult, especially if a pond has developed.

Table 6-20. Discharge capacity of square grate inlets

Grate size (in)	Grate opening (sq ft)	Head of water on grate (ft)																	
		0.2	0.4	0.6	0.8	1.0	1.2	1.4	1.6	1.8	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0
6x6	0.12	0.3	0.4	0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.2	1.3	1.5	1.6	1.8	1.9	2.0	2.1
9x9	0.28	0.7	1.0	1.2	1.4	1.5	1.7	1.8	1.9	2.0	2.1	2.6	3.0	3.4	3.7	4.0	4.3	4.5	4.8
12x12	0.50	1.1	1.7	2.1	2.4	2.7	2.9	3.2	3.4	3.6	3.8	4.7	5.4	6.0	6.6	7.1	7.6	8.1	8.5
15x15	0.78	1.3	2.7	3.2	3.8	4.2	4.6	5.0	5.3	5.6	5.9	7.3	8.4	9.4	10.3	11.1	11.9	12.6	13.3
18x18	1.12	1.6	3.8	4.7	5.4	6.0	6.6	7.1	7.6	8.1	8.5	10.5	12.4	13.5	14.8	16.0	17.1	18.1	19.1
21x21	1.53	1.9	5.2	6.4	7.4	8.2	9.0	9.7	10.4	11.0	11.6	14.2	16.4	18.4	20.1	21.8	23.3	24.7	26.0
24x24	2.00	2.2	6.1	8.3	9.6	10.7	11.8	12.7	13.6	14.4	15.2	18.6	21.5	24.0	26.3	28.4	30.4	32.2	34.0
30x30	3.12	2.7	7.6	13.0	15.0	16.8	18.4	19.9	21.2	22.5	23.7	29.1	33.6	37.5	41.1	44.4	47.5	50.3	53.1
36x36	4.50	3.2	9.1	16.7	21.6	24.7	26.5	28.6	30.6	32.4	34.2	41.9	48.3	54.0	59.2	63.9	68.3	72.5	76.4
42x42	6.12	3.8	10.6	19.5	29.4	32.9	36.0	38.9	41.6	44.1	46.5	57.0	65.9	73.4	80.9	87.0	93.0	98.7	104.0
48x48	8.00	4.3	12.1	22.3	34.3	43.0	47.1	50.8	54.3	56.6	60.8	74.4	85.9	96.0	05.2	113.7	121.5	128.9	135.9

NOTES:

1. Capacity is in cubic feet per second.
2. Values to left of HEAVY line were calculated from the weir formula: $Q = 3LH^{1/2}$, L = perimeter.
3. Values to right of HEAVY line were calculated from the orifice formula : $Q = 5.37AH^{1/2}$, A = grate opening.
4. Clear opening between grate bars was taken to be 50% of total grate area.
5. Grate size should be increased 50% in paved areas.
6. Grate size should be increased 100% in turfed areas.

SUBSURFACE DRAINAGE

SUBSURFACE DRAINAGE CRITERIA

When surface failures prove that natural subsurface drainage is inadequate, it becomes necessary to determine if a subsurface drainage system is needed, and if so, what type to install. Generally, subsurface drainage may be divided into three classes: base drainage subgrade drainage, and intercepting drainage.

Base drainage generally consists of subsurface drainpipes laid parallel and adjacent to pavement edges with pervious material joining the base and the drain. Figure 6-59 shows a typical section of base drainage.

Base drainage is required where frost action occurs in the subgrade beneath the pavement and where ground water rises to the bottom of the base course through natural conditions or from ponding of surface runoff. Where pavement becomes temperarily inundated and there is little possibility that the water will drain from the base into the subgrade base drainage will be required. Table 6-21, page 6-94, establishes the criteria to follow in these cases.

Base drainage is also required at the low point of longitudinal grades in excess of 2 percent where the subgrade coefficient of permeability is less than 1 x 10^{-3} fpm. The coefficient of permeability, a property of each soil type, is defined as the discharge velocity at a unit hydraulic gradient. Determine the coefficient of permeability experimentally, either by laboratory test or by an actual field test of the soil involved. The coefficient is expressed in units of velocity such as fpm or centimeters per second (cm/see). Base drainage is required if the subgrade coefficient of permeability is smaller than the coefficient of permeability indicated in Table 6-22, page 6-94.

Subgrade drainage is required for permanent construction when seasonal fluctuations of groundwaler may be expected to rise to less than 1 foot below the bottom of the course. Figure 6-60, page 6-95, shows a typical example of a subgrade drainage section. Figure 6-61, page 6-96, will serve as a guide for spacing drains. These drains, although similar to base drains, have a larger area of filter material in contact with the subgrade.

Intercepting drainage is required when water seeping into a pervious layer will raise the groundwater locally to a depth of less than 1 foot below the bottom of the base course. This condition is often encountered in thin, pervious soil layers; in exposed rock cuts; or in seepage from springs. A typical intercepting drainage section is shown in Figure 6-62, page 6-97.

SUBSURFACE DRAINAGE TECHNIQUES

Subsurface water can be controlled through a combination of techniques. The techniques and combinations depend on the conditions existing in the area to be drained. The techniques that follow should be considered when planning and designing subsurface drainage.

Depth of Base Course

The base course may be built up to a specified depth above the groundwater table. Generally, the finished grade must be at least 5 feet above the mean groundwater table level. This technique is feasible when—

- A gravity drainage system is impractical.
- The condition to be controlled is limited to a small area such as a narrow swamp crossing.
- Adequate base-course material is available.

Deep Ditch

Where ditches will not interfere with operations or become a hazard to traffic, deep V-ditches with free outfall may be feasible. Easily built and readily enlarged, these ditches provide positive interception and

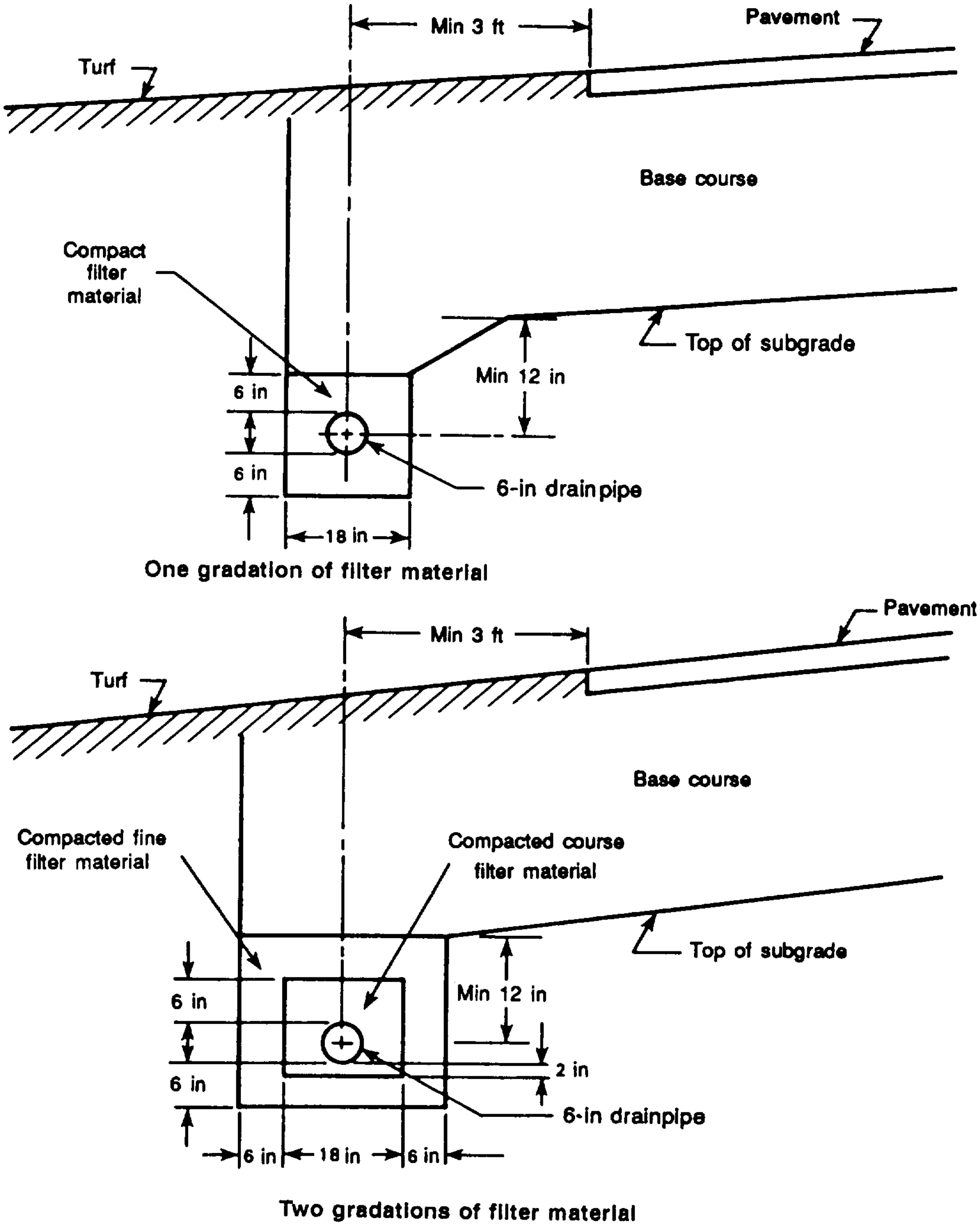


Figure 6-59. Typical base drainage installations

drainage of subsurface water before it reaches the area to be protected. Erosion, maintenance, traffic, and right-of-way are factors to be considered before using this solution,

Natural Drainage Channels

Where possible and practical, water in existing natural drainage channels should be lowered when corresponding effective lowering of the groundwater table will occur. This technique may be particularly effective in pervious soil.

Table 6-21. Base drainage criteria for K

Depth of ground water (ft)	Coefficient of permeability less than
Less than 8	1×10^{-5} fpm
From 8 to 25	1×10^{-6} fpm
Over 25	1×10^{-7} fpm

Note: Capillary action is related inversely to permeability. A soil with $K = 10^{-7}$ is more prone to capillary action than a soil with $K = 10^{-8}$.

Blind Drains

Blind or *French* drains are constructed by filling a ditch or trench with broken or crushed rock. The top surface of the rock may be left exposed so that the trench will act as a combination surface and subsurface drain, or the rock may be covered by a relatively impervious soil so that no surface water can penetrate. The latter is the general practice. In general, French drains are not recommended for permanent construction because they tend to silt up over time, In TO construction, these drains are often used as a substitute for perforated or open-joint pipe or on filter materials used with such piping.

Subsurface Pipe

In cases where a V-type or other open-ditch type drainage system is not practical, it may be necessary to resort to construction of subsurface drainage. Failure of the subsurface system stemming from improper control of the grade, the bedding, the pipe placement, the placement of filter material, or other installation work gives no warning prior to failure. Such failure is extremely difficult to repair when discovered.

The most common form of subsurface drainage is perforated pipe. Where the perforations do not extend completely around the circumference of the pipe, the pipe is generally laid with the holes down and the joints closed. Materials used in manufacturing this type of pipe arc corrugated metal, cast iron, vitrified clay, nonreinforced concrete, bituminized fiber, and asbestos cement.

Bell and spigot pipes can be laid with open join ts. If the filter material has been properly designed, collars are not needed over the joints. This type of pipe is generally made of vitrified clay, nonreinforced concrete, or cast iron.

Table 6-22. Drainage characteristics of soil

<table>
<tr><td></td><td colspan="6">K in cm/sec (log scale) (1 cm/sec = 2 ft/min)
10^{2} 10^{1} 1.0 10^{-1} 10^{-2} 10^{-3} 10^{-4} 10^{-5} 10^{-6} 10^{-7} 10^{-8} 10^{-9}</td></tr>
<tr><td rowspan="2">Drainage properties</td><td colspan="3">Good drainage</td><td colspan="2">Poor drainage</td><td>Practically impervious</td></tr>
<tr><td colspan="2">Drains very rapidly</td><td>Drains rapidly</td><td>Drains slowly</td><td>Drains very slowly</td><td>Drainage imperceptible</td></tr>
<tr><td>Soil classification</td><td colspan="2">GW-SW</td><td>GP-SP</td><td colspan="2">ML-OL GC-SC-CL GF-SF-MH</td><td>CL-CH-OH</td></tr>
<tr><td rowspan="2">Types of soil</td><td>Clean gravel</td><td colspan="2">Clean sand, clean sand and gravel mixtures</td><td colspan="2">Very fine sands; organic and inorganic silts; mixtures of sand, silt, and clay; glacial till; stratified clay deposits</td><td>"Impervious soils" Homogeneous clays below zone of weathering</td></tr>
<tr><td colspan="6">"Impervious soils" which are modified by the effects of vegetation and weathering</td></tr>
</table>

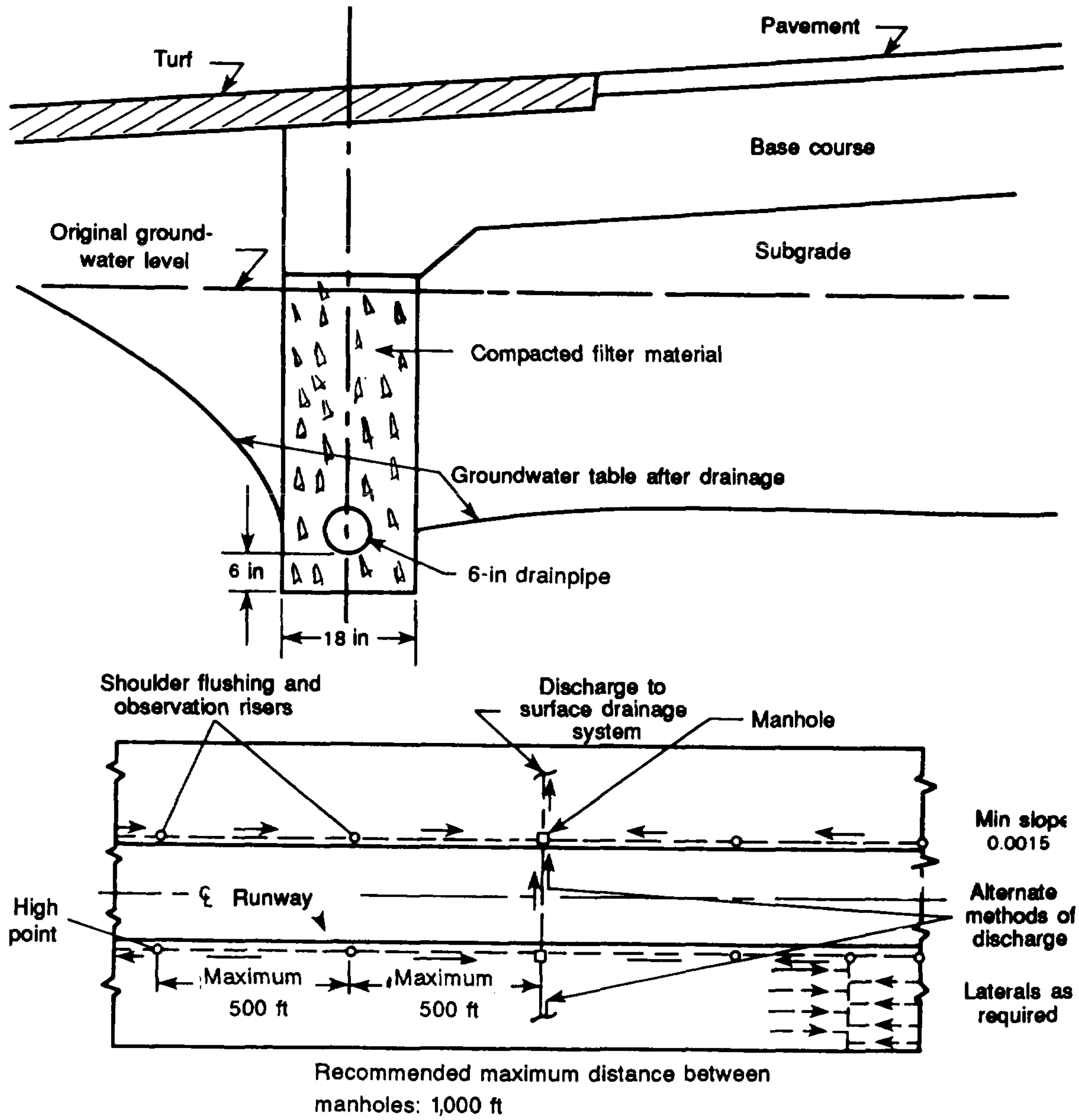


Figure 6-60. Typical subgrade drainage installation

Porous concrete pipe is laid with closed joints. It collects water by seepage through the wall of the pipe and should not be used where sulfated waters may cause disintegration of the concrete.

Farm tile is laid with butt joints slightly separated to permit collection of water through the joint. Because of its low resistance to high-impact loads, farm tile is not recommended for use on airfields.

Materials commonly used in manufacturing farm tile are clay or concrete.

Combination Drainage System

Combination drains, which attempt to handle both surface runoff and subsurface water in the same pipe system, are recommended. Surface runoff often carries sediment and soil from the drained area into the system. This clogs the system and causes flow stoppage. For this reason, subsurface drainage systems using some form of piping are

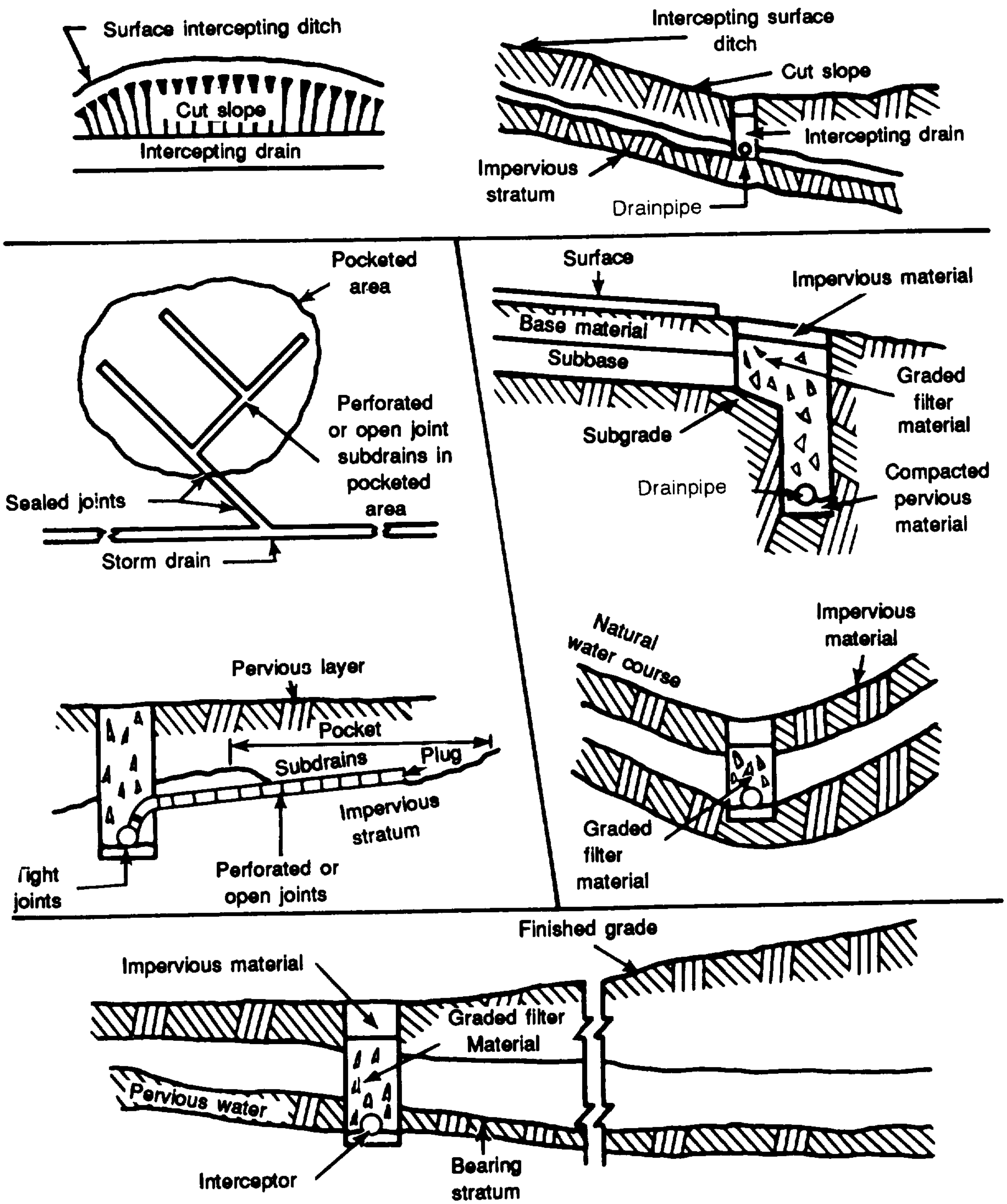


Figure 6-61. Types of subsurface drainage

generally sealed so that surface runoff cannot enter. The only drainage system which will satisfactorily handle both surface runoff and subsurface water is the open channel or ditch.

PIPE-LAYING CRITERIA

There are essentially four different types of pipe available for subsurface drainage, as previously mentioned. They should be laid according to the following specifications:

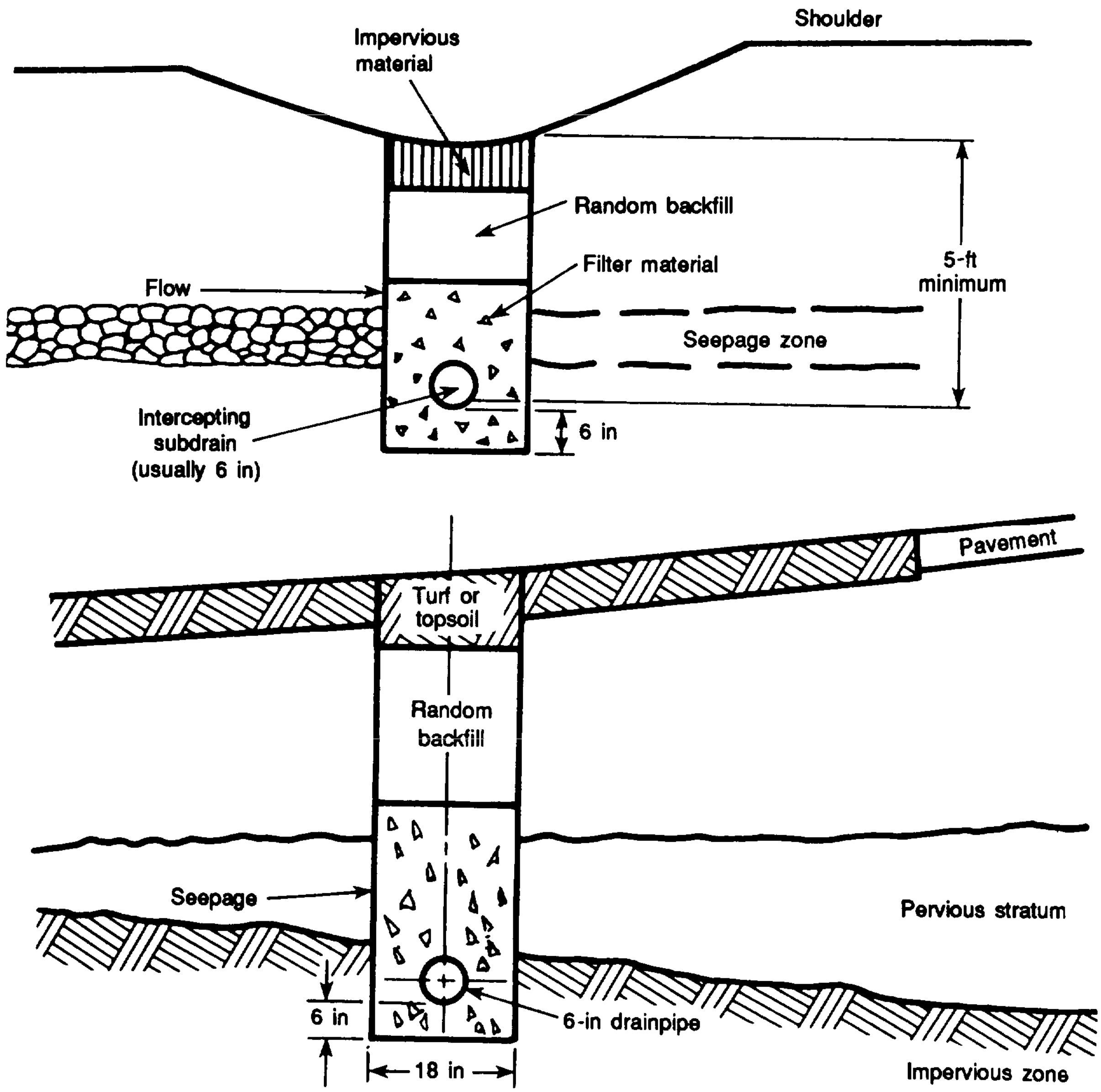


Figure 6-62. Typical intercepting drainage installations

- The minimum slope for subdrainage pipe is a 0.15-foot drop in elevation per 100 feet of length, The elevation of a pipe at any particular location is generally specified by the invert elevation, in which the invert is defined as the lowest point in the internal cross section of the pipe at the particular location.

- Manholes should be provided at intervals of not more than 1,000 feet, with flushing risers between manholes and at dead ends as shown in Figure 6-60, page 6-95.

- Pipe should be at least 6 inches in diameter with 6-inch pipe being used for all drains. With long intercepting lines or extremely severe groundwater conditions, 8-inch or larger pipe may be necessary.

Ž The center of subgrade drains should be located at least 1 foot below the bottom of the base course and not less than 1 foot below the groundwater table. Normally, subgrade drains are required only at the edges of pavement areas where the soil is pervious and drains well. However, local groundwater conditions and base and subgrade soil characteristics may require closer spacing of the drains, When the drain discharges into a culvert or any considerably larger pipe, it should discharge above the water level in the larger pipe. When the drain discharges into a pipe of equal or only slightly larger size, it is generally better 10 bring the drain in above the receiving drain and make a vertical connection between the two. This will pre - vent the water from backing up in the drainage pipe since these pipes rarely flow full

Ž When the impervious layer is at a reasonable depth, intercepting drains should bwe placed in the impervious layer below the intercepted seepage stratum. The quantity of water collected by an intercepting drain is difficult to determine, but in general, 6-inch pipe is sufficient for lengths up to 1,000 feet.

VERTICAL WELLS

Vertical wells arc sometimes constructed to allow trapped subsurface water to pass through an impervious soil or rock layer to a lower, freely draining soil layer. If drainage is obstructed, additional wells are built or the pocket is drained with an easily maintained lateral subdrain system. Vertical wells s are often used in northern latitudes where deep freezing is common. They permit fast runoff from melting snow to get through the frozen soil and reach a pervious stratum. Under such conditions, the bottoms of these wells are treated with calcium chloride or a layer of hay to prevent freezing.

FILTER MATERIAL

A layer of filter material approximately 6 inches deep should be placed around all subsurface piping systems. The selection of the proper filter material is very important since it determines, to a great extent, the success or failure of the drainage system. The improper selection of filter material can cause the drainage system to become inoperative in one of three ways:

- The pipe may become clogged through infiltration of small soil particles.
- Particles in the protected soil may move into or through the filters, causing instability of the surface.
- Free groundwater may not be able to reach the pipe.

Criteria have been developed, based upon the mechanical-analysis soil curve, to prevent the above failures.

A great deal can be learned about gradation characteristics of a particular soil by observing the soil curves on the mechanical analysis chart. Well-graded soils generally have a smooth, grain-size curve with gradual changes of slope. Poorly-graded, uniform soils generally have a very steep grain-size curve. Skip-graded soils have a grain-size curve with a characteristic hump in it. The filter material in skip-graded soil tends to segregate during placement. The grain-size curves in Figure 6-63 show various gradation characteristics.

A coefficient of uniformity, C_u, value of less than 20 is desirable to prevent segregation of coarse and fine-grain particles, especially during placement. For the same reason, skip-graded material should not be used. Placing the filter material while it is wet can reduce segregation.

Filter material can clog a pipe by moving through the perforations or openings.

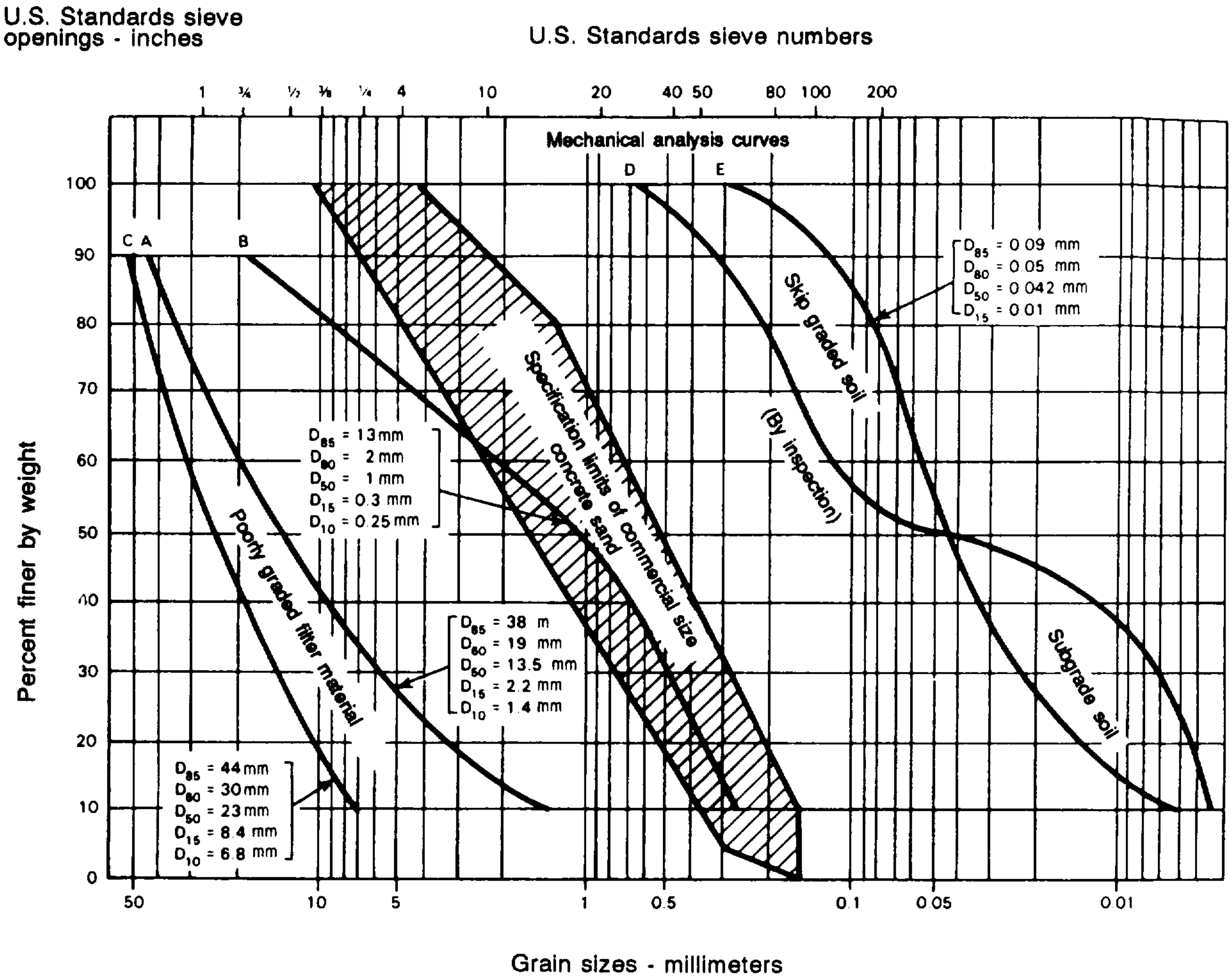


Figure 6-63. Mechanical analysis curves for filter material

Prevent this by using the following specifications:

For a slotted opening—

$$\frac{\text{85-percent size of filter material}}{\text{slot width}} > 1.2$$

For a circular hole—

$$\frac{\text{85-percent size of filter material}}{\text{hole diameter}} > 1.0$$

Use the following methods to prevent particles from the protected soil from moving into or through the filter or filters:

$$\frac{\text{15-percent size of filter material}}{\text{85-percent size of protected soil}} \leq 5$$

and

$$\frac{\text{50 percent size of filter material}}{\text{50 percent size of protected soil}} \leq 25$$

To permit free water to reach the pipe, the filter material must be many times more pervious than the protected soil. This condition is fulfilled when—

$$\frac{\text{15-percent size of filter material}}{\text{15-percent size of protected soil}} \geq 5$$

If it is not possible to secure a mechanical analysis of available filter materials and protected soil, concrete sand with mechanical-analysis limits as shown in Figure 6-63 may be used. Experience has indicated that a well-graded concrete sand is satisfactory as a filter material in most sandy, silty soils.

STEPS IN FILTER DESIGN

Obtain a mechanical analysis of each sandy soil from several readily available sources. Obtain a mechanical analysis of the subgrade soil (protected soil) at the bottom of the trench which will be in contact with the filter material.

Plot the mechanical analysis of each soil on semilogarithm paper and draw a curve for each soil.

From the mechanical-analysis curve for each soil, determine the diameter, in millimeters (mm), of the particle of each sample. Ten percent of these particles are finer (by weight) D_{10}. The subscript "10" indicates the percentage, by weight, of the soil sample which is finer (smaller in diameter) than the particle in size D_{10}. Similarly, determine the D_{15}, D_{50}, D_{60}, and D_{85} particle size for each soil.

Determine the slot width or hole diameter, in millimeters, of the type of pipe to be used.

Check the coefficient of uniformity, CU, to ensure—

$$C_u = \frac{D_{60}}{D_{10}} \leq 20$$

Check the design criteria for clogging of pipe openings to ensure—

(1) $$\frac{D_{35}}{\textit{slot width}} > 1.2 \quad \text{or}$$

(2) $$\frac{D_{35}}{\textit{hole diameter}} > 1.0$$

Check the design criteria for movement of particles from the protected soil (subgrade) through the filter material to ensure—

(1) $$\frac{D_{15} \textit{ of filter material}}{D_{85} \textit{ of protected soil}} \leq 5$$

(2) $$\frac{D_{50} \textit{ of filter material}}{D_{50} \textit{ of protected soil}} \leq 25$$

Check the design criteria for relative perviousness to ensure—

$$\frac{D_{15} \textit{ of filter material}}{D_{15} \textit{ of protected soil}} \geq 5$$

Select that filter material which best meets the above criteria.

Install pipe to grade, bedded and surrounded with the selected filter material as shown in Figure 6-58, 6-59, or 6-60, pages 6-91, 6-93, and 6-95, respectively.

SELECTING FILTER MATERIALS

Filter material should be selected with a view toward the simplest construction and the lowest cost. To further this end, try to use only one layer. If several layers of filter material are required, one layer should be confined to the region around the pipe openings and another layer placed between it and the protected soil, as shown in Figure 6-59, page 6-93. In this case, the designer selects a filter material for use around the pipe according to the filter design formulas. The second filter material is then designed to protect both the inner filter material and the surrounding soils. In other words, the design of a multilayer filter for a subdrain system should proceed outward from the inside filter material to the subgrade soil being protected.

Example (Selecting Filter Material):

A suitable filter material must be selected for a 6-inch pipe with 1/4-inch diameter perforations to protect a subgrade soil with an E curve (shown in Figure 6-63, page 6-99). The soils represented by curves A and B are readily available from local borrow pits.

Tabulate data from the mechanical-analysis curves.

Soil A	**Soil B**	**Subgrade (protected soil)**
D_{10} = 1.4 mm	D_{10} = 0.25 mm	
D_{15} = 2.2 mm	D_{15} = 0.30 mm	D_{15} = 0.01 mm
D_{50} = 13.5 mm	D_{50} = 1.0 mm	D_{50} = 0.042 mm
D_{60} = 19.0 mm	D_{60} = 2.0 mm	D_{60} = 0.054 mm
D_{85} = 38.0 mm	D_{85} = 13.0 mm	D_{85} = 0.09 mm

Check the coefficient of uniformity of both soils.

Soil A

$$C_u = \frac{D_{60}}{D_{10}} = \frac{19.0}{1.4} = 13.6$$

Soil B

$$C_u = \frac{D_{60}}{D_{10}} = \frac{2.0}{0.25} = 8.0$$

Thus, both soils A and B satisfy the require ment that the coefficient of uniformity be less than 20.

Apply design criteria to soil A,

Should be $\leq$ 5 to prevent movement of subgrade soils through the filter.

$\frac{2.2}{0.09} = 24.4$ which is not < 5.

Soil A is unsuitable because movement of the subgrade soil through the filter material is possible.

Apply design criteria to soil B.

$\frac{D_{15}\text{ (filter)}}{D_{85}\text{ (protected soil)}}$ Should be $\leq$ 5 to prevent movement of subgrade soils through the filter.

$\frac{0.30}{0.09} = 3.33$ which is < 5.

$\frac{D_{50}\text{ (filter)}}{D_{50}\text{ (protected soil)}}$ Should be $\leq$ 25 to prevent movement of subgrade soils through the filter.

$\frac{1.0}{0042} = 23.8$ which is < 25.

$\frac{D_{15}\text{ (filter)}}{D_{15}\text{ (protected soil)}}$ Should be $\geq$ 5 to permit water movement through the filter.

$\frac{0.30}{0.01} = 30$ which is > 5.

$\frac{D_{85}\text{ (filter)}}{\text{hole diameter}}$ Should be > 1.0 to prevent clogging of the pipe.

Note that the soil particle size is usually given in millimeters, while the hole size is usually given in inches. The two dimensions must be expressed in compatible units before the preceding formula is used. To make this conversion, multiply the size of the pipe perforations, in inches, by 25.4 which represents the number of millimeters per inch. For example, with soil B under discussion—

$$\frac{D_{85}\text{ (filter)}}{\text{hole diameter}} = \frac{13.0\text{ mm}}{1/4 \times 25.4} = \frac{13}{6.35} = 2.0$$

which is > 1.0.

Thus, soil B satisfies all the criteria for a good filter material while soil A dots not.

INSTALLATION OF A SUBDRAINAGE SYSTEM

The most efficient and most practical type of subdrainage system is one which adequately performs the operations for which it was intended and, in addition, was installed with the care and skill consistent with its purpose. Any attempt to lower the quality of construction or to usc a sketchy or inadequate subdrainage system can result in disastrous failures. Conversely, any attempt to install an elaborate system of underground piping where a simple V ditch would serve as well is inadvisable.

SECTION IV. SURFACE DRAINAGE DESIGN IN ARCTIC AND SUBARCTIC REGIONS

APPLICABILITY

This section discusses the problems involved in the design of drainage facilities in arctic and subarctic regions. While the design data presen ted has been developed primarily for Alaska, the methods used are generally applicable to other arctic and subarctic regions.

Arctic

Arctic is defined as the northern region in which the mean temperature for the warmest month is 50° Fahrenheit (F) or less, and the mean annual temperature is below 32° F. In general, the arctic coincides with the tundra region north of the limits of trees.

Subarctic

Subarctic is defined as the region adjacent to the arctic where the mean temperature is 32° F or below for the coldest month and 50° F or above for the warmest month, and where less than four months have a mean temperature above 50° F. In general, the subarctic coincides with the circumpolar belt of dominant coniferous forests.

HYDRAULIC CRITERIA FOR COLD CLIMATES

Rainfall

A study of rainfall intensity-frequency data recorded at arctic stations indicates a considerable variance between the average intensity of rainfall for a period of one hour and the average precipitation rates of comparable frequency for a duration of less than one hour. This is evidenced when compared with similar rainfall in the continental United States (CONUS). Even within the area of Alaska, there was a noticeable difference between the rains at Juneau and those at Fairbanks. The higher values for rainfall intensity were used to develop design intensity-duration (supply) curves, which are shown in Figure 6-64, For design purposes, a minimum rainfall rate of 0.2 in/hr is recommended, even where maps of intensity-frequency

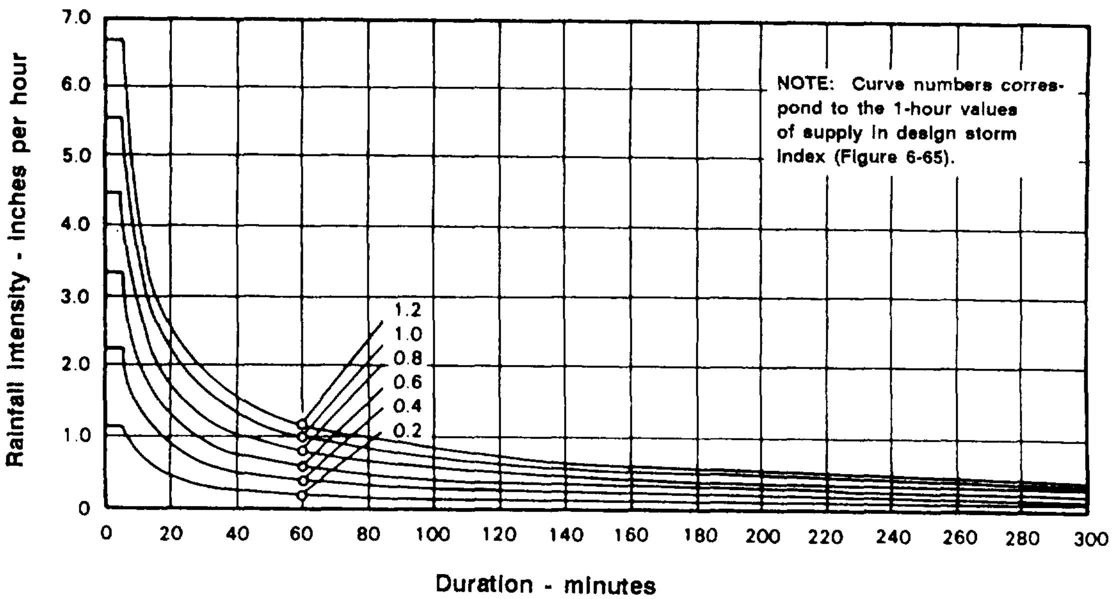


Figure 6-64. Intensity-duration curves, arctic and subarctic

Design-storm index. One-hour rainfall intensities having various average frequencies of occurrence in the arctic and subarctic regions of Alaska are shown on maps in Figure 6-65, page 6-104. This figure, on which rainfall depth curves are superimposed, is known as a design-storm index. The curves are labeled by the one-hour amounts of rainfall that are represented; these, in turn, are coordinated with the intensity-duration or supply curves of Figure 6-12, page 6-17, Figures 6-64 and 6-65, used in combination, provide a means whereby rainfall intensities sufficien tly accurate for runoff computations for any duration may readily be determined.

Elevaton and physiographic orientation. Present information is insufficient to establish quantitative conclusions of the effects of elevation and physiographic orientation for all locations. At the time a site is under investigation, it would be helpful to obtain even a short record of rainfall there. Such a record may be compared with the concurrent portion of a nearby long record and proper frequency values assigned to events in the short record, For example, a temporary gage might have a maximum hourly value of 0.60 inch some summer, and the same storm might produce the summer's maximum hourly value of 0.40 at a nearby long-record station. Similar comparison of other storms for the brief parallel record might confirm the ratio of 6 to 4 as an expression of the difference in the orientation, exposure, and elevation of the two stations. This ratio could then be applied to the known 2-year, 1-hour value of the long-record station to get the estimated 2-year, 1-hour value for the short-record station, If the long-record station has a 2-year, 1-hour value for the construction site, it would be 6/4 of 0.5 or 0.75. Arrangements usually can be made for borrowtng a rain gauge for temporary use from the meteorological agency of the area in which the project is to be located.

Infiltration

In permafrost areas, infiltration for design purposes should be considered zero. In other areas, a good guide may be obtained when test borings are made, Values normally would not exceed about 0.5 inch per hour for coarse sands and gravels and would be as low as 0.1 inch per hour for clayey soils with low permeability.

STORM-DRAIN DESIGN

The type and capacity of storm-drain facilities required are determined primarily by the promptness with which design-storm runoff must be removed in order to avoid serious interruption or hazard in the use of important operational areas as well as prevent serious damage to pavement subgrades. It is presumed that all phases of site reconnaissance have been carefully completed and that information is available which shows topography, natural drainage patterns, groundwater conditions, and seasonal frost and permafrost levels.

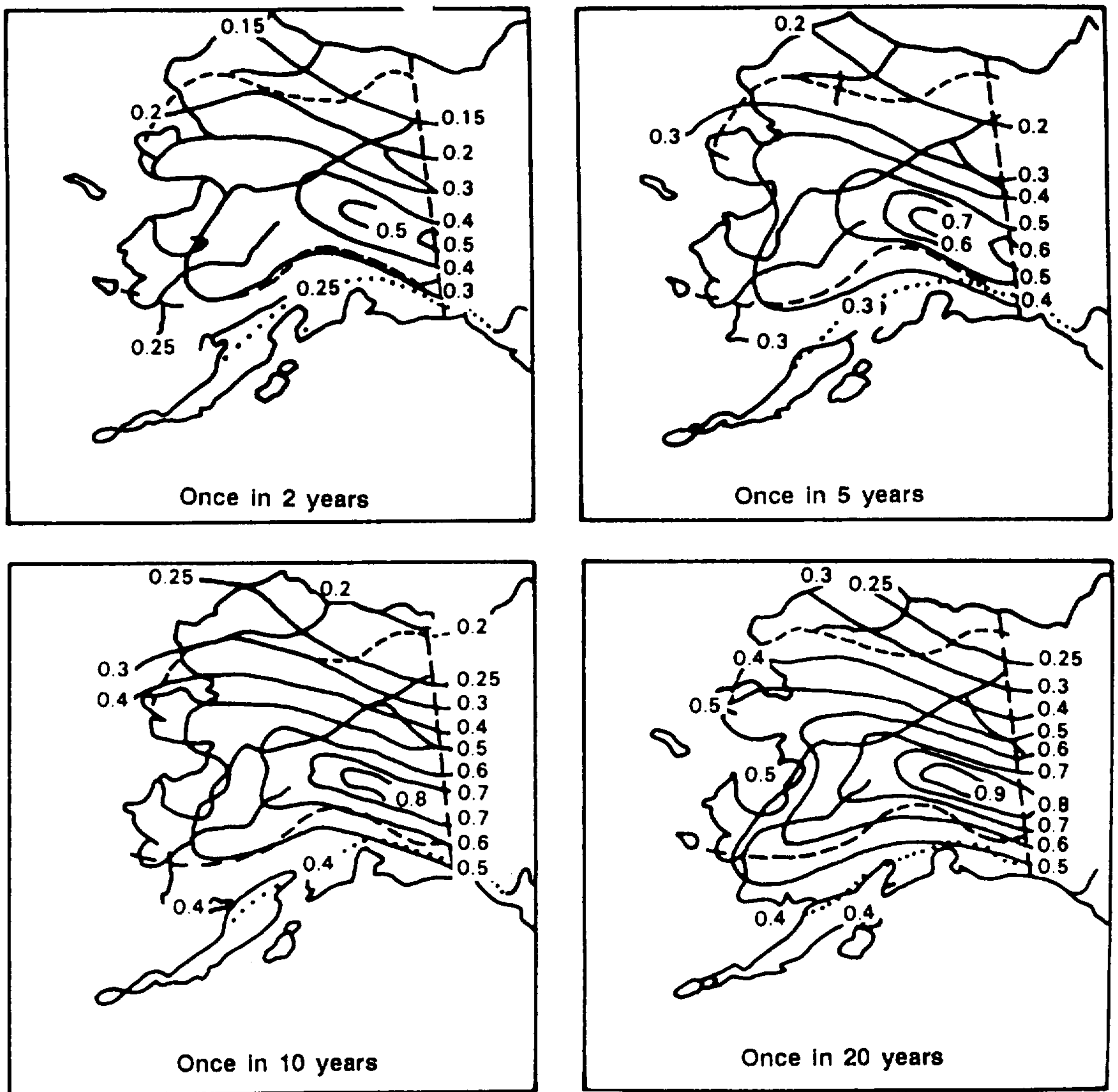
Basic Considerations

Even though rainfall magnitudes are small in arctic and subarctic regions, drainage is an important factor in selecting an airfield site and planning the construction. The planner should be aware of several features related to drainage in order to ensure a successful design. These features include the following:

- Sites should be selected in areas where cuts or the placement of base-course fills will not intercept or block obvious existing natural drainage ways,

- Areas with fine-textured, frost-susceptible soils should be avoided, if possible, In arctic and subarctic regions, most soils are of single-grain structure with a very small percentage of clay. As a consequence, cohesive forces between grain particles are very small and the material erodes easily. Frozen, fine-textured soil profiles may also contain large amounts of ice lenses and wedges.

- If the upper surface of the permafrost layer is deep and if provisions are made for lower temperatures, design features of a drainage system may be similar to



Legend

.... Approximate southern limit of the Arctic
— — Approximate southern limit of the permafrost
. . . . . Approximate southern limit of area covered

NOTES:

1. Observation of rainfall at National Weather Service stations provides the basic data for this figure. These stations are not located in the more elevated regions; consequently, wide deviations from the charted values may exist at altitudes above 2,500 feet.

2. The influence of local topographic characteristics should be taken into account and evaluated at the time of drainage design.

3. The lines of equal half-hour rainfall magnitudes correspond to the intensity-duration curves shown in Figure 6-64.

4. The once-in-50-years amount can be estimated by multiplying the once-in-2-years amount by the factor 2.2

Figure 6-65. Design-storm index for Alaska those used in frost regions of the United States.

- The flow of water in a drainage channel has an accelerating effect on the thawing of frozen ground. This may cause the surface of the permafrost to dip considerably beneath streams or channels which convey water. Bank sloughing and significant changes in channel grades become prominent. Sloughing is often manifested by wide cracks paralleling the ditches. For this reason, drainage ditches should be located as far as practicable from runway and road shoulders.

- In many subarctic regions, freeing drainage channels of drifted snow becomes a significant task before breakup each spring. In these areas, it is advantageous to have ditch shapes and slopes sufficiently wide and flat to accommodate heavy snow-moving equipment. In other locations where flow com tinues year round, narrow, deep ditches are preferable to avoid icings.

- Large, cut sections should be avoided in planning the drainage layout. Thawed zones or water-bearing strata may be en countered and later cause serious icings. Vegetative cover in permafrost areas should be preserved to the maximum degree practicable. Where disturbed, it should be restored as soon as construction permits,

- Inlets to closed conduits are commonly sealed before freeze-up and opened prior to breakup each spring.

- Fine-grained soils immediately above a receding frost zone arc very unstable. Consequently much sliding and caving is to be expected on unprotected ditch side slopes in such soils.

- Locating ditches over areas where permafrost lies on a steep slope should be avoided if possible. Slides may occur because of thawing and consequent wetting of the soil at the interface between frozen and unfrozen ground.

- Maintenance equipment for drainage facilities should include heavy snow-removing apparatus and a steam boiler with accessories for steam thawing of structures which have become clogged with ice. Pipes for this purpose are often fastened inside the upper portions of culverts prior to their placement.

Grading

Proper grading is a very important factor contributing to the success of a drainage system. The development of grading and drainage plans must be carefully coordinated. In arctic and subarctic regions, it is necessary to eliminate soft, soggy areas.

Temporary Storage

Trunk drains and laterals should have sufficient capacity to accommodate the project design runoff. Do not consider supplementary pending above the drain inlets in airfield drainage designs for the arctics and subarctic. Formulate plans in sufficient detail to avoid flooding even during the time of actual construction.

ICINGS

An *icing* is an irregular sheet or field of ice with no uniformity as to shape, thickness, or size. All icings are similar with regard to laminated structures, indicating that irrespective of shape, thickness, size, or cause, the actual process of formation is the same. Thin films of water traverse over layers of ice or other material and, when exposed to the cold air, freeze and form the first or an additional layer of ice. As water flow continues, the process is repeated, and an icing with horizontal laminations continues to grow until either the source of water supply is depleted or warmer weather begins.

Types of Icings

For the purpose of analysis, icings may be divided into three groups, depending largely on the nature of the source of water supply For icings formed along rivers or streams and adjacent areas having a source of water above or below the riverbed, the term river *icing* applies. If the source is Irom groundwater flow above permafrost, ground icing is the term most commonly used This term should not be confused with ground ice. which is often encountered in the arctic and subarctic as deposits in fine-graincd soils. The term *spring icing* should be confined to when the source of wa ter is from subpermafrost levels or sub-permafrost water under hydrostatic pressure. Spring icings are commonly large and thick. Human activity can disturb the ground regime sufficiently to cause or accelerate the formation of all types of icings.

River icings. Most arctic and subarctic streams carry large loads of sediment which are not fed into the channels in uniform quantities Consequently, the rivers are quite wide and relatively shallow. Many rivers have a braided pattern of several smaller streams within the confines of the main channels. These streams frequently shift in transverse position and often do so during one period of high flow. Winter flow is ordinarily very small and shallow. Freezing penetrates to the bottom of shallow streams quite readily, but the river discharge continues as groundwater flow beneath the riverbed. Because of thermal effects of flowing water, the soil below streambeds is unfrozen to greater depths than soil located elsewhere Consequently, It here is a large space for groundwater storage and flow above the permafrost and below all riverbeds. The head motivating ground water flow is ordinarily quite large and can result in large pressures above sec-it ions where the groundwater flow is retarded.

Groundwater-flow retardation is a natural process at many river sections because river beds arc not homogeneous in water-carrying capacity. Freezing of the water reduces channel area and capacity in some sections more than in others. The formation of anchor ice on the bottom of the streambed results in fur her constriction of the channel cross-section area. The water then finds avenues of escape to the top of the ice via weak points, cracks, and fissures. Here, exposed to the cold atmosphere, the water quickly freezes in thin sheets. This action is progressive, and icing continues to increase in thickness until the supply of water is exhausted or finds a new outlet, or until the beginning of warmer weather. A bridge may shade the streambed and also prevent the deposition of snow. Freezing then would be more rapid beneath the bridge than at either upstream or downstream locations. Subsequent penetrations of frost would diminish groundwater -flow capacity at the bridge section and induce the formation of an icing above or at the site. These icings can be of various shapes and sizes, depending upon the valley topography, the depth of the snow, the intensity of cold, the water supply, and other factors.

Ground icings. Ground icings may take the form of mounds having considerable thickness but small areas. They may also form as crustations if groundwater flow is induced to the surface at points which are not of great lateral spacing but are of about equal elevation, In addition to a supply of water, there is another requisite to the formation of an icing—an area where the water can be exposed to the cold atmosphere. A pavement kept clear of snow offers an excellent site over which flowing water can spread out into a thin film and then freeze. Icings from groundwatcr above the permafrost arc not likely to occur in the arctic, as the permafrost there is too close to the surface to permit any appreciable storage in the active layer. This occurrence is most severe in the southern zones of the subarctic and on slopes which face south. Groundwater flow may be induced to the surface in various ways. It is not essential that the seasonal frost reach the permafrost, although this very effectively blocks groundwater flow. Partial freezing of the active layer reduces the area of the section which ground water must pass. The path of least resistance may lead to the ground surface via a frost crack or fissure or through holes which have previously been made by burrowing animals Water coming to the surface in this way may flow considerable distances down slopes beneath a snow blanket without freezing.

Various met hods have been tried to prevent the occurrence of such icings. Some of these have met with partial success. The frost belt or dam has been advocated by Russian investigators, but this method is effective for only a few years. The thawing in summer is accelerated at the site of the frost dam, and eventually the permafrost degrades sufficiently to permit groundwater flow below the frost dam. Fences and barriers have been used quite effectively under special circumstances.

Spring icings. Icings that occur from artesian, subpermafrost water and springs are ordinarily quilt thick and cover considerable area. Reference is often made to the icing in the Momy River Valley of Siberia. This spring icing is about 15 miles long and 3 miles wide, with an average thickness of about 12 feet. It does not melt and form each year. Spring icings can be controlled quite readily. The temperature of the water emerging from the ground is ordinarily quite high and the water does not freeze quickly if confined to a conduit. In some cases, an insulated conduit may be required to convey the spring water to locations where the formation of icing will do no damage.

Measures Against Icings

River icings. In the case of river icings, depths to permafrost are ordinarily too large to be blocked effectively by accelerated freezing such as is induced by the frost dam or bell, In addition, the subbed river flow is often in excess of what can be stored as ice above the location of the bridge to be protected. The control of river icings then must be concerned with an insulation of streambeds at the critical section.

Ground icings. Ground icings can be controlled to some extent by inducing the ice to form upstream from the site in question. This can be accomplished by the installation of frost belts. In open terrain, some success can be achieved by merely keeping snow removed from a strip crossing the affected area in a direction transverse to groundwater flow. Groundwater flow will be blocked by freezing and forced to the surface upstream from the cleared area. The snow-free area also provides a cold space on which surface flow can spread out and freeze. If necessary, the depth of stored ice can be increased by erecting some barrier to the flow, such as an ordinary wooden-stave snow fence on top of the ice initially formed at the site of the frost belt. Because the process employs the removal of snow, it is essential to shift the position of the belt from year to year in order not to unduly influence the depth to permafrost. In timbered regions, it is obviously necessary to maintain the frost belt at one location to save the expense of tree removal.

Spring icings. If the source of water forming the icing is a spring, then it is necessary to resort to drainage or diversion to control the occurrence. This sometimes requires insulated channels. In the case of springs, flows are ordinarily too large to permit a storage of icc at or upstream from the site.

FORDS, DIPS, CAUSEWAYS, AND BRIDGES

FORDS

A ford is a shallow place in a waterway where the bottom, either naturally or by human improvement, permits the passage of personnel and vehicles. A ford is used instead of a bridge when time limitations, the lack of structural materials, the tactical situation, and the terrain configuration make its use necessary and practical.

An increase in water depth can close a ford for a considerable time. Streams in mountainous and desert country are subject to sudden changes in depth. The increase in depth can be so sudden as to endanger personnel or vehicles in the ford.

Stream bottoms can be of such material that much effort is required to make fords usable.

Reconnaissance

Route reconnaissance should include the selection of possible ford sites. Special emphasis is placed on the requirements to be discussed in this section, Ford reconnaissance and required reports are covered in FM 5-36.

Requirements

The characteristics of a good ford are a slow current (usually less than 2 miles per hour); low, sloping banks; good approaches; and a uniformly increasing bottom depth with a firm bottom material. Requirements of width depth, and bank slopes for fords are given in Table 6-23.

Location. A desirable location for a ford is in the reach of the stream between bends. At this location, the bottom depth is constant between the banks with only a slight channel in the center. The influence of river action on possible fording locations is shown in Figure 6-66. Because of the increased velocity of the water, bends result in a deep channel that is difficult to improve. In the reach, the center channel is not so deep or sharp and therefore is readily adaptable for improvement. In addition, the variation in the shape of the banks is not as pronounced in the reach area as it is at the bend.

Bottom Material. For a natural ford, the bottom material should be hard, durable, and interlocking. Such material will resist cutting by wheeled traffic and erosion. The general terrain will determine the bottom condition, as follows:

- In mountainous country, sudden freshet floods can transport large boulders and stones along the bottom. This material is deposited at the passing of high water or at locations where the widening of the stream reduces the velocity. It may be necessary to remove this material before traffic can cross the ford,

- In terrain of moderate or gentle slopes, the velocity will tend to prevent deposition of fine material. This tendency and the scouring action of the water will leave a good, firm bottom. The bottom material may be disturbed by traffic and will require protection.

- In slightly sloping or flat terrain, streams meander and have low velocity. In addition, there is sometimes a high water table. In such cases, the bottom material may be very soft. Such

Table 6-23. Requirements for military fords

	Maximum depth (ft)	Minimum width (one-way traffic) (ft)	Type of bottom	Maximum allowable slope on approaches*
Infantry	3½	3 (Single file)	Firm enough to prevent sinking	1:1
		7 (Column of 3s)	Boulders and obstacles removed	
Trucks	2	10	Firm and smooth	3:1
Light tanks	1-3	10	Firm and smooth	2:1
Medium tanks	2-4	10	Firm and smooth	2:1
Heavy tanks	4-6	12	Firm and smooth	2:1

*Based on hard dry surface. If wet and slippery, slope must be less.

material will require a completely improved ford with special emphasis on bottom requirements, Timber, lumber, matting, gravel, or gabions can be used to improve trafficabilty.

High-Water Determination. Table 6-23 indicates that a maximum water depth of 2 feet is allowable for truck traffic. To ensure maximum usc of the ford, it is necessary to determine the depth at which the stream will flow at frequently recurring times. Estimates of various depths of flow, as shown in Figure 6-67, page 6-110, can be made by direct stream observation, as follows:

Base Flow. Base flow is the normal flow that occurs in a stream when there has been no recent rain. The depth of this flow is dependent upon the quantity of ground water.

High-Water Flow. During the year, many rainfalls normally occur that cause flows above the base level, The velocity of such flows generates a slight erosion cut, as

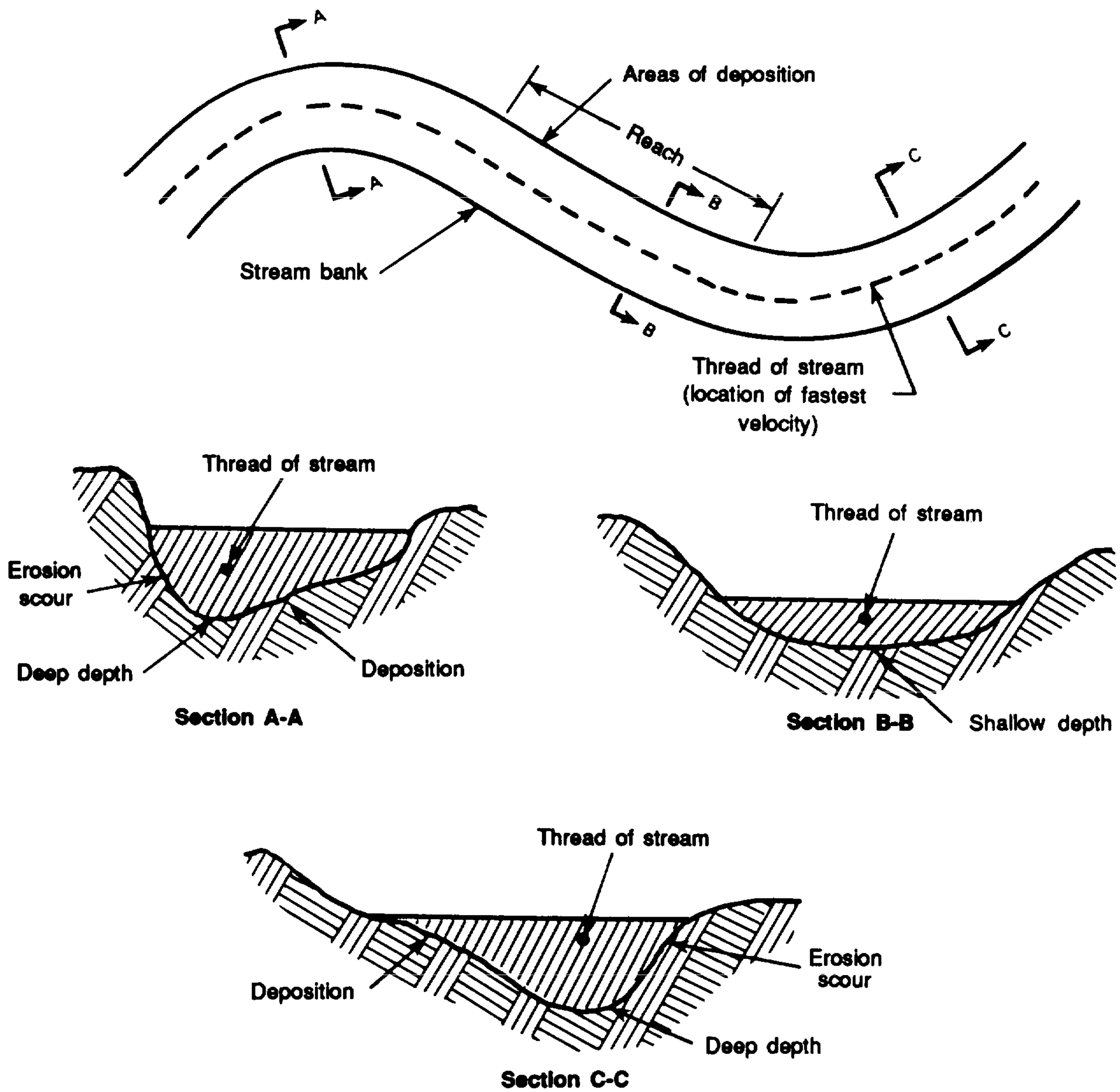


Figure 6-66. River action

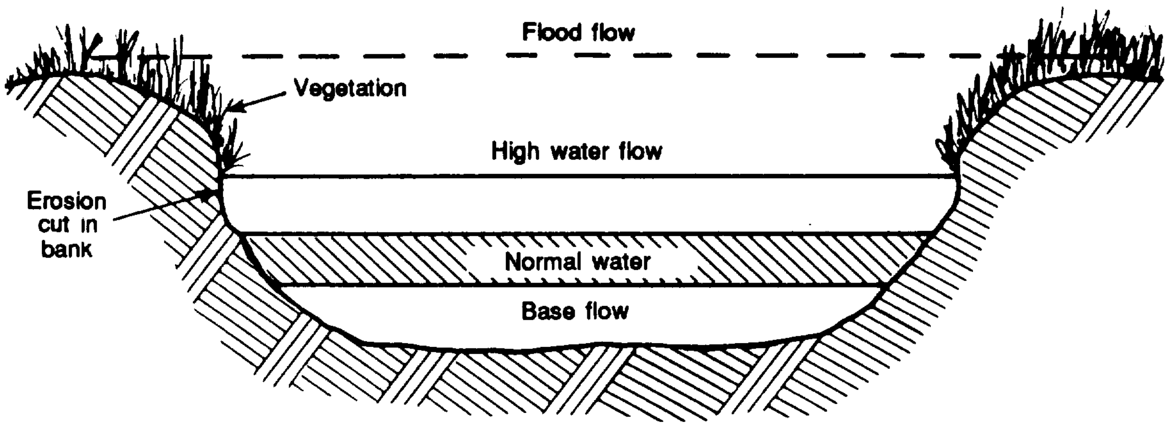


Figure 6-67. Stream cross section

shown in Figure 6-67. The occurrence of such flows tends to prevent the growth of vegetation. Since these flows can occur with relative frequency during the year, the depth of these flows could control the use of the ford. In the event the depth is greater **in places** than the fording depth of trucks (see Table 6-23, page 6-108), it may be necessary to fill these gaps with rock or gravel, However, such filling might cause the velocity of flow over the ford to be increased to the point that vehicles would have difficulty using the ford.

Flood Flow. In the absence of records, it is necessary to check stream banks for evidence of floodwater levels. The highest level of flood that occurs within a period of two years is of particular interest. The bend of a stream, especially with a high bank on the outside of the bend, would show an erosion cut that can be used to determine the flood level, as shown in Figure 6-67.

Approaches. Carefully note the height of the bank and the type of soil. This information helps to determine construction requirements.

Stream Velocity. The normal stream velocity at the fording site should not exceed 3 fps. This velocity would, in most cases, occur at low or moderate levels.

Cross Section. Make a cross section of a ford location similar to Figure 6-66, page 6-109 and Figure 6-67. Include full details of bank slopes, bottom slopes, bottom variations, and water depth. In addition, determine the average velocity of the stream from measurements taken at equal intervals across the stream.

Channel Condition. Make a record of the character of the streambed. Include vegetation density and type, whether or not the channel is scoured, and the type of soil. This information will determine the value of Manning's *n*.

CONSTRUCTION

Two phases of construction are required for fords—the development of the approaches and the preparation of the bottom.

The maximum slopes for ford approaches should be as recommended in Table 6-23. Place material cut from the banks off to the side and not in the stream, where it may form an obstruction. Because traffic will wet the slopes and cause eventual deterioration, provision should be made for protecting the surface.

Ford-bottom preparation will depend upon site conditions. Fill short, deep gaps with rock or gravel, preferably retained by wire mesh. Soft, mud bottoms can be improved by covering the bottom first with willow, brush mattresses, or timbers, and subsequently with metal planking, rock, or coarse gravel. Even a hard and tenacious bottom deteriorates under traffic conditions and requires protective maintenance.

Consider these factors when raising the bottom of the ford:

- The depth upstream from the ford increases in proportion to the amount of rise of the bottom of the ford.

- The velocity of flow over the ford increases at an increased fording depth so that vehicles may be difficult to operate and control.

MARKING

Place marking posts at each end of the ford and at as many intermediate points as may be necessary. Mark a post at each end with an index to indicate depth. Warning notices should be clearly and prominently placed to alert drivers that flooding can occur suddenly and without warning.

MAINTENANCE

Examine fords after each flooding. Repair scour damage upstream and downstream with riprap, Remove boulders and other debris to provide a clear passage for vehicles,

DIPS

Dips are paved fords used for the crossing of dry, wide, and shallow arroyos in semi-arid regions subject to flash floods.

Reconnaissance

The preferred location of a dip is in the straight run of an arroyo or wash, Determine the width between the banks and the top elevation of the banks. In addition, check the area above the dip site to determine if pending will occur and to what level. Determine the type of soil in the banks and bottom for construction requirements. In addition, note the size and type of bottom rock. This type of information gives an indication of the volume and velocity of flash floods which move or carry large material.

Investigate the area, especially the banks, to determine the general flash-flood high watermark. This information on the area of the waterway, along with an approximation of velocity as indicated by the rock or other debris on the bottom, gives some indication of the volume of flow. This information will be necessary if bridging is required.

Construction

The subgrade should be of erosion-resistant material or a rock-compacted base, and it should be set between the cutoff retaining walls. The pavement should be of concrete or compacted macadam. The general construction is shown in Figure 6-68.

Other factors of importance are as follows:

- The site should be in the reach of the stream and well away from any bends. There is more erosion in bends than in straight stretches. If a very severe flood occurred, a structure placed in a bend might be destroyed.

- The structure should be set at right angles to the flow to reduce scour.

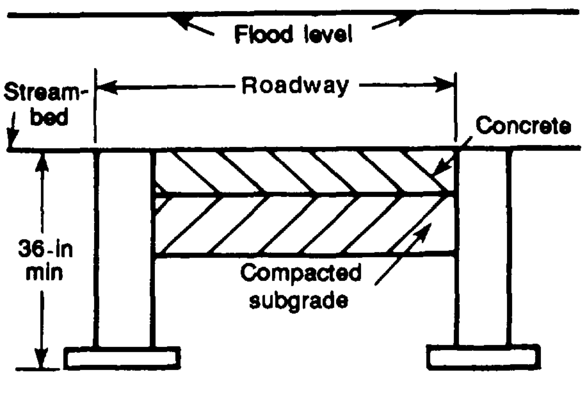


Figure 6-68. Cross section of dip section

- The structure must be set at true dry-streambed level to avoid scour erosion or silting. Under no circumstances should the structure be set above dry-streambed level.

Marking

The marking for dips is the same as for fords.

Maintenance

Dips, like fords, must be examined after each flooding, and scour damage upstream and downstream should be repaired with riprap, Remove boulders and other debris. When macadam is used, it can be anticipated that holes may be scoured in the roadway. Consider stockpiling rock adjacent to the area for immediate maintenance repair.

CAUSEWAYS

When it is essential to keep a roadway open during floods of medium intensity, a raised causeway can be used in place of a ford or dip, This type of structure must be well sited, carefully designed to pass the flow, and strongly built, It must incorporate a sufficient waterway at streambed level to permit the passage of the design volume of flow before the flood level reaches the top of the structure. A typical design is shown in Figure 6-69, The main design features of a causeway follow.

Cross-Sectional Area

A sufficient cross-sectional area must be provided to ensure that the flood level will not submerge the structure. Consider the following basic elements:

- The size and number of culverts or other elements must be sufficient to pass the flow. When the water elevation upstream and downstream is above the culvert crown, pipe equations should be used for flow design,

- The inverts or lowest parts of the culverts must be set at streambed level.

- The cover over the stream must be sufficient to protect the waterway against traffic loads,

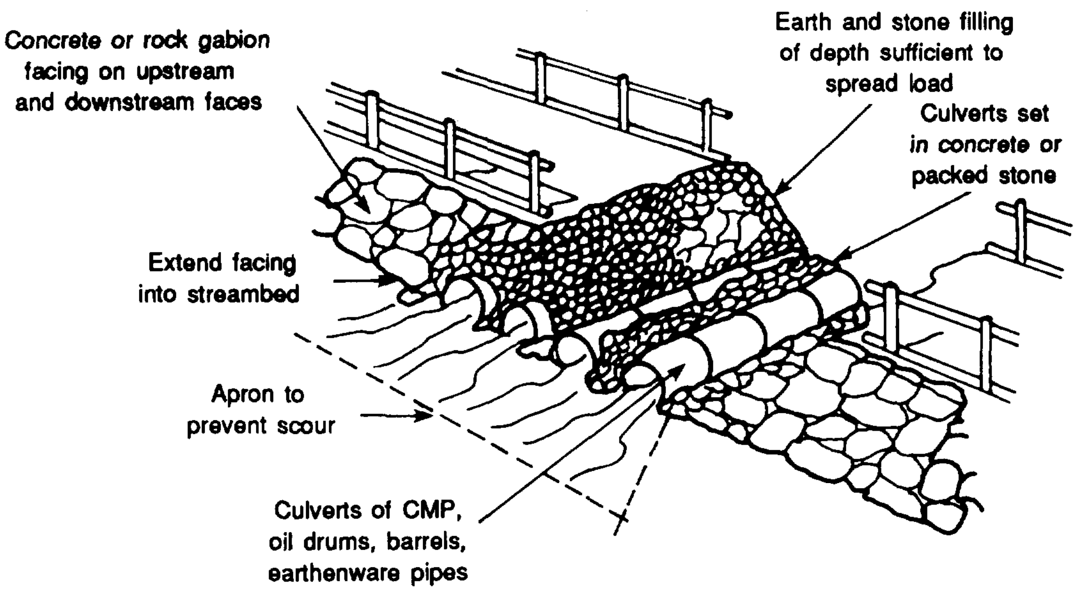


Figure 6-69. Overflow (causeway)

Embankment

Protect both the upstream and downstream faces of the embankment against scour and erosion. Heavy flooding or overtopping of the structure will require complete protection in the form of a concrete or rock-gabion facing. To further prevent excessive scour and erosion, carry the protective facings below the streambed and provide them with aprons. Anchor the ends of the structure securely into the banks in such a way that there is minimum obstruction to water flow.

Guardrails

Provide guardrails to guide and direct traffic. Because the structure can be overtopped, be sure to provide for ready replacement of the guardrails.

Maintenance

Inspect the structure for scour or erosion after each flow (hat causes partial submergence or overtopping. Repair any damage immediately otherwise, at the next heavy flow, the structure could be destroyed. Stockpiling of heavy rock and gabions at each end of a structure may be required.

BRIDGES

This section presents elements of bridge design other than the requirements for structural design Bridges must conform to the requirements of stream hydraulics in the same way as all cross-stream structures such as fords, dips, and causeways.

Location

The location of a bridge should be away from bends in the straight section or reach of the stream (Figure 6-66, page 6-109). In this location, there are moderate, even depths from bank to bank. The deep channel tends to be in the center. Abutments will be placed on the edge of the stream. Because of the even distribution of flow, scour and erosion at the stream pier and bank abutments arc not expected to be so excessive as to cause maintenance problems.

Height

The height of a bridge depends upon the flood-flow, high watermark. The height of that mark will determine the profile or superstructure level of the bridges as follows:

- When the flood-flow, high watermark is above the banks, a high-profile or high-level bridge must be constructed to keep the superstructure above the flood level. This type of bridge is well above bank level and may require a considerable length and height of approach.

- When the flood-flow, high watermark is below bank level. a low-profile or low-level bridge can be constructed. This type of bridge presents fewer problems than the high -level bridge, since approach ramps will not have to be constructed.

Abutments

The location of bridge abutments depends upon an analysis of the flood level and the cross section of the stream (Figure 6-70). Analysis proceeds as follows:

Abutments placed at 1A and 1B of Figure 6-70 present the most direct solution because they are located on high, dry ground,

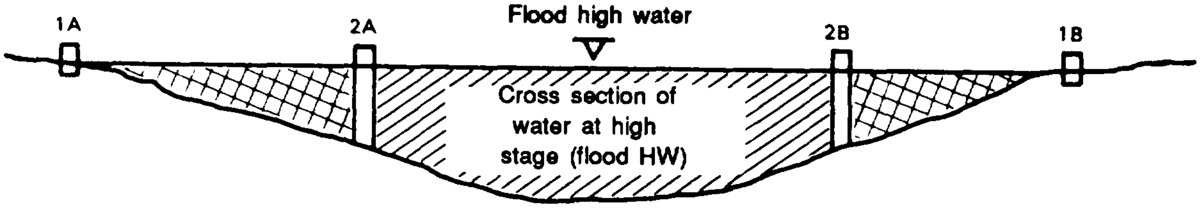


Figure 6-70. Waterway cross section at bridge site

Use of these locations, however, may necessitate more construction effort because of the increased length of the bridge. This increased length could result in the need for intermediate piers and spans. With this method, the full river waterway will be used for the passage of flood water.

Abutments placed at 2A and 2B would reduce construction time because fewer intermediate piers and spans would be required. In addition, the fill in areas 1A-2A and 1B-2B could be accomplished as the bridge is constructed. As can be seen from the figure, this placement reduces the cross section of the waterway, The following techniques could be used to accommodate this reduction in an available cross-sectional area:

- The elevation of the bridge superstructure could be raised to account for the rise in the flood level. In this case, use substantial abutments that are well protected against end scour.
- The superstructure elevation could be left substantially at the original level, and approach ramps over areas 1A-2A and 1B-2B could be constructed as causeways to allow for flow. Care must be taken to ensure that there is no excessive scour or erosion below the culvert outlet that would affect the roadway.
- The roadway approaches in areas 1A-2A and 1B-2B could be depressed below the superstructure level. In this case, the excess flood flow would pass over the roadway approaches, thus relieving the bridge flow. If this action is taken, the roadway may not be usable at all times. Since overflow is anticipated, the construction of these approaches is similar to construction of causeways without the culverts. If the bridge is designed properly, with fording depths as outlined in Table 6-23, page 6-108, it may be usable during floods under extreme conditions.
- When a depressed roadway or an elevated superstructure is used, the approach to the bridge must have a gentle slope to prevent vehicle impact on the abutment and to ensure traffic visibility.

Drainage

The soil behind bridge abutments can become saturated because of rain or other conditions. This saturation can take place whether the approach road is at the natural grade of the soil or it is a filled approach. When saturation occurs, static hydraulic pressure on the back face of the abutment generates additional overturning movement. With wood abutments, this condition is relieved naturally. However, if concrete abutments are used, the pressures can be relieved as follows:

Step 1. Use weep holes to pierce the abutments with bagged gravel backing on the soil side.

Step 2. Place gravel backing against the lower part of the abutment drained by a perforated pipe at the footing elevation. Set the pipe to drain out at the sides of the abutment.

EROSION CONTROL

Erosion must be controlled to maintain an effective and clear drainage system with a minimum of maintenance and to reduce hazardous dust conditions. Erosion may occur at any point where the force of moving water exceeds the cohesive strength of the material with which the water is in contact. Proper design of side slopes in cut and fill sections (based on the type of soil) will reduce the need for extensive erosion control measures. However, additional control is usually required. Most methods of control are based on dissipating the energy of water, providing an erosion-resistant surface, or some combination of these techniques. This chapter acquaints the military engineer with the means available to reduce or eliminate the erosive force of water.

NONUSE AREAS AND OPEN CHANNELS

Terracing is a control measure designed to dissipate the energy of overland flow in nonuse areas. Turfing, paving, Gunite lining, and placing riprap are control methods designed to cause turbulence and to increase retardation, thereby dissipating the energy of flow in open channels such as ditches and pjpe outfalls. In cases where even riprap will be eroded, the use of gabions is a speedy and relatively inexpensive means of dissipating energy.

TERRACING

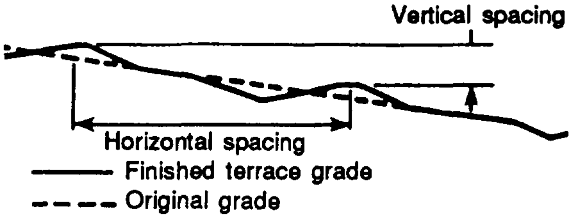
A terrace is a low, broad-based earth levee constructed approximately parallel to the contours of the topography. A terrace either intercepts and holds the water until it infiltrates the soil or moves it as overland flow to a suitable discharge point. A hardy, vigorous turf should be planted to hold the disturbed soil in place. Vertical spacing and longitudinal gradients of terraces are given in Figure 6-71.

TURFING

Ditches are often protected by placing strips of sod (held in place by wooden boards or stakes) perpendicular to the flow path at intervals along the ditch, as shown in Figure 6-72.

PAVING AND GUNITE LINING

Ditches having grades in excess of 5 percent usually require paving or a Gunite lining. Where a slope equals or exceeds 5 percent, paving must be extended down slope at least to the point in the ditch at which the erosive energy of the water is controlled or absorbed without erosion damage.



Vertical spacing

Average land slope (percent)	Horizontal spacing (feet)	Vertical spacing (feet)
2	125	2.5
4	75	3.0
6	58	3.5
8	50	4.0
10	45	4.5
12	42	5.0
14	39	5.5

Longitudinal gradients

Length of terrace (feet)	Terrace channel grade (percent)
0 - 300	0.10
300 - 600	0.15
600 - 900	0.20
900 - 1,200	0.30
1,200 - 1,500	0.40

Figure 6-71. Terrace spacing and gradients

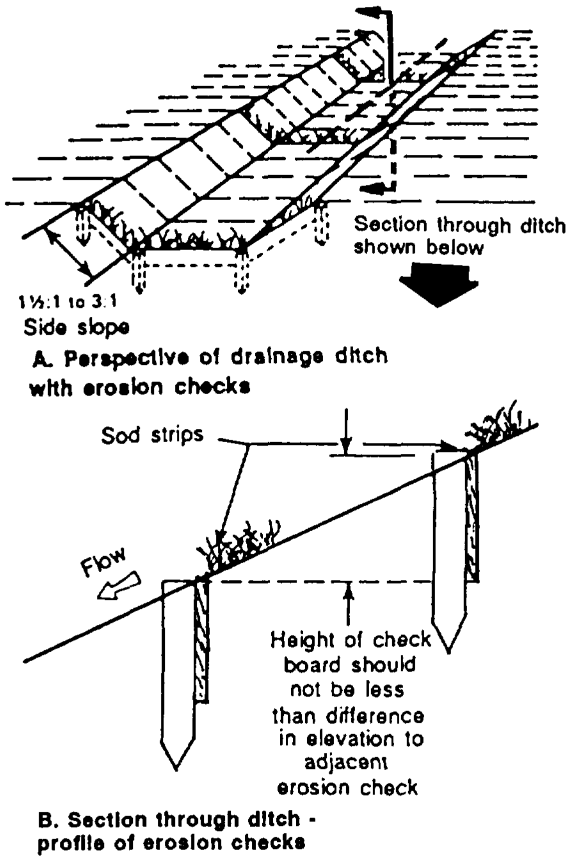


Figure 6-72. Erosion-control checks

Paving with either asphalt or portland-cement concrete provides superior erosion-resistant linings in gutters, ditches, and outfall structures.

Gunite lining of ditches controls erosion effectively. Gunite is a mixture of portland cement and sand with water added just before the mixture is sprayed from a high-pressure nozzle onto the surface being protected. The Gunite lining is formed over steel mesh placed over the bottom and sides of the ditch. Gunite is sprayed to a thickness of 1 to 1 1/2 inches, with the steel mesh located midway in the thickness. Human resources, time, material and equipment expenses usually limit the use of paving or Gunite linings to only the most demanding conditions in TO airfield construction.

PLACING RIPRAP

Riprap protection should be provided adjacent to all hydraulic structures. When placed on erodible surfaces, it prevents scour- al the ends of the structure. This protection is required on the bed and banks for a sufficient distance to establish velocity gradients and turbulence levels at the end of the riprap.

Riprap can also be used for lining the channel banks to prevent lateral erosion and undesirable meandering. Provide an expansion either horizontally or vertically (or both) immediately downstream from hydraulic structures such as drop structures or energy dissipators. The expansion allows the flow to expand and dissipate its excess energy in turbulence rather than directly on the channel bottom and sides. Riprap has been known to fail from-

- Movement of the individual stones from a combination of velocity and turbulence.
- Movement of the natural bed material through the riprap, resulting in slumping of the blanket.
- Undercutting and leveling of the riprap from scour at the end of the blanket.

Consideration must be given to the selection of an adequate size of stone, the use of adequately graded riprap, the provision of a filter blanket, and the proper treatment of the end of the riprap blanket.

Selection of Size

Curves for the selection of stone size required for protection, with Froude numbers and depths of flow in the channel shown, are shown in Figure 6-73.

Two curves are given. One is for riprap subjected to direct flow or adjacent to hydraulic structures such as side inlets, confluences, and energy dissipators, where turbulence levels are high. The other is for riprap on the banks of a straight channel where flows are relatively quiet and parallel to the banks.

With the depth of flow and average velocity in the channel known, the Froude number can be computed from the following equation:

$$F = \frac{0.716V}{\sqrt{d}}$$

D_{50} value can be determined from the appropriate curve.

Curves for determining the riprap size required to prevent scour downstream from culvert outlets with scour holes of various depths are shown in Figure 6-74, page 6-118. Make the thickness of the riprap blanket equal to the longest dimension of the maximum size of stone or 1.5 times D_{50}, whichever is greater.

When the use of large rock is desirable but impractical, substituting a grouted reach of smaller rock in areas of high velocities or turbulence may be appropriate. Grouted riprap should be followed by an ungrouted reach.

A well-graded mixture of stone sizes is preferred to a relatively uniform size of riprap. A recommended gradation is shown in Figure 6-75, page 6-119. In certain locations, the available material may dictate the gradation of riprap to be used. The

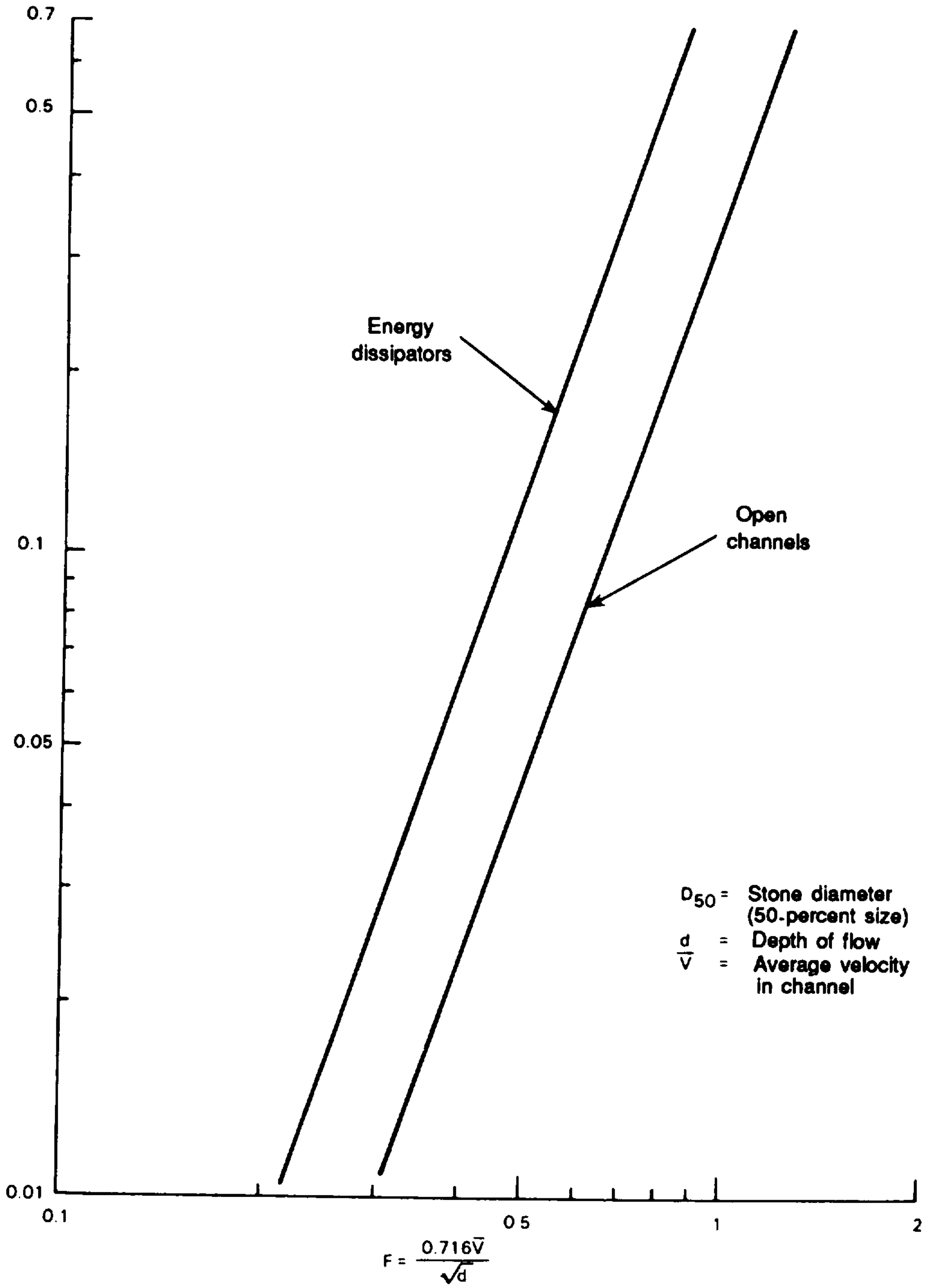


Figure 6-73. Recommended riprap sizes for open channels

gradation should resemble the recommended mixture as closely as possible. Consider increasing the thickness of the riprap blanket when locality dictates using gradations with a larger percentage of small stone than shown by the recommended plot. If the gradation of the available riprap is such that movement of natural material through the riprap blanket would be likely, place a filter blanket of sand, crushed rock, gravel, or synthetic cloth under the riprap. The usual blanket thickness is 6 inches,

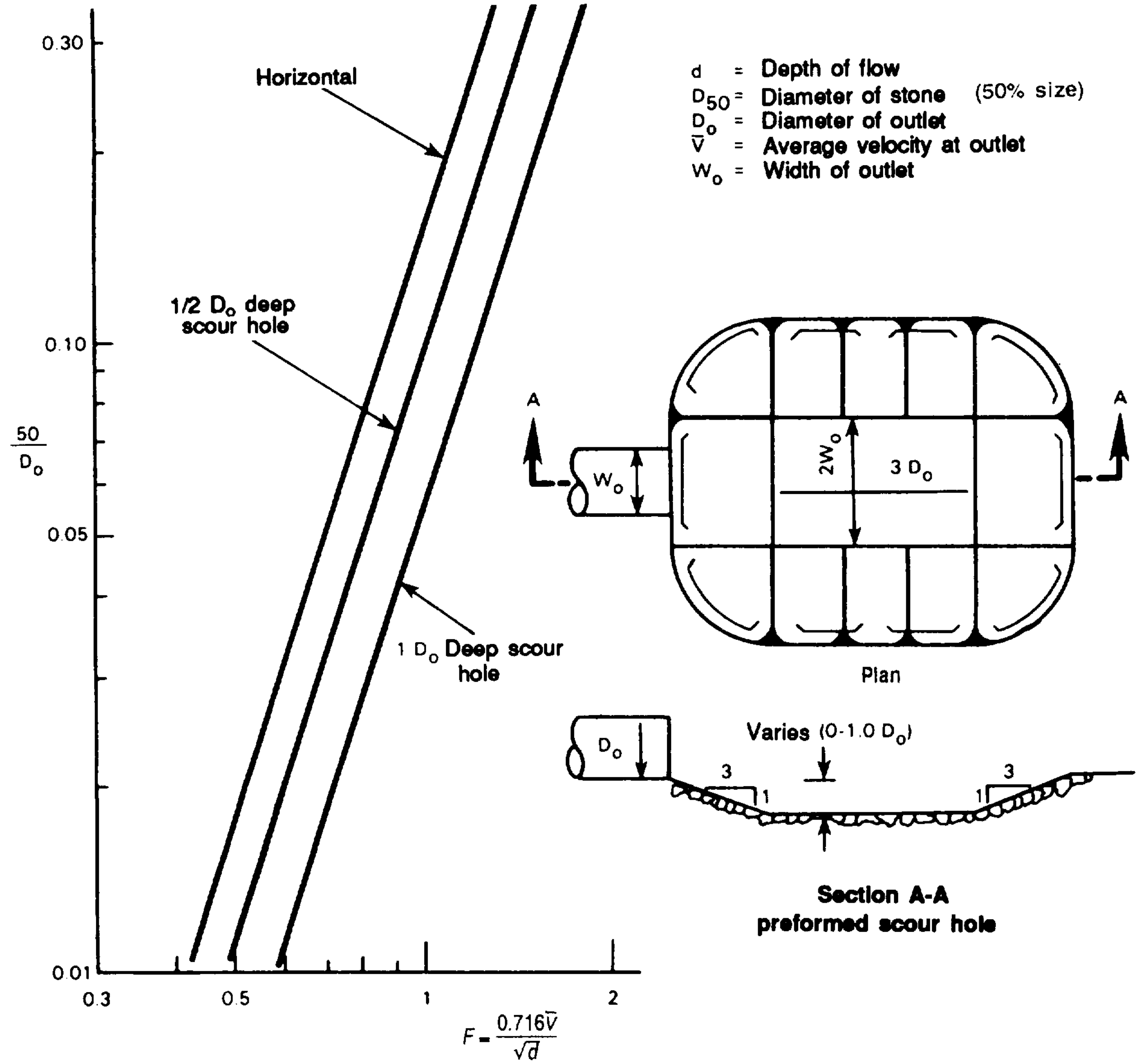


Figure 6-74. Recommended riprap sizes for culvert outlets

although a greater thickness is sometimes necessary.

Design

An ideal riprap design would provide a gradual reduction in riprap size until the downstream end of the blanket blends with the natural bed material. Unless this is done, turbulence caused by the riprap is likely to develop a scour hole at the end of the riprap blanket. However, the extra effort required to provide gradual reduction in riprap size is seldom justified. Double the thickness of the riprap blanket at the downstream end to protect against undercutting and unraveling. An alternative is a rubble blanket of constant thickness and suitable length, dipping below the natural streambed to the estimated depth of bottom scour.

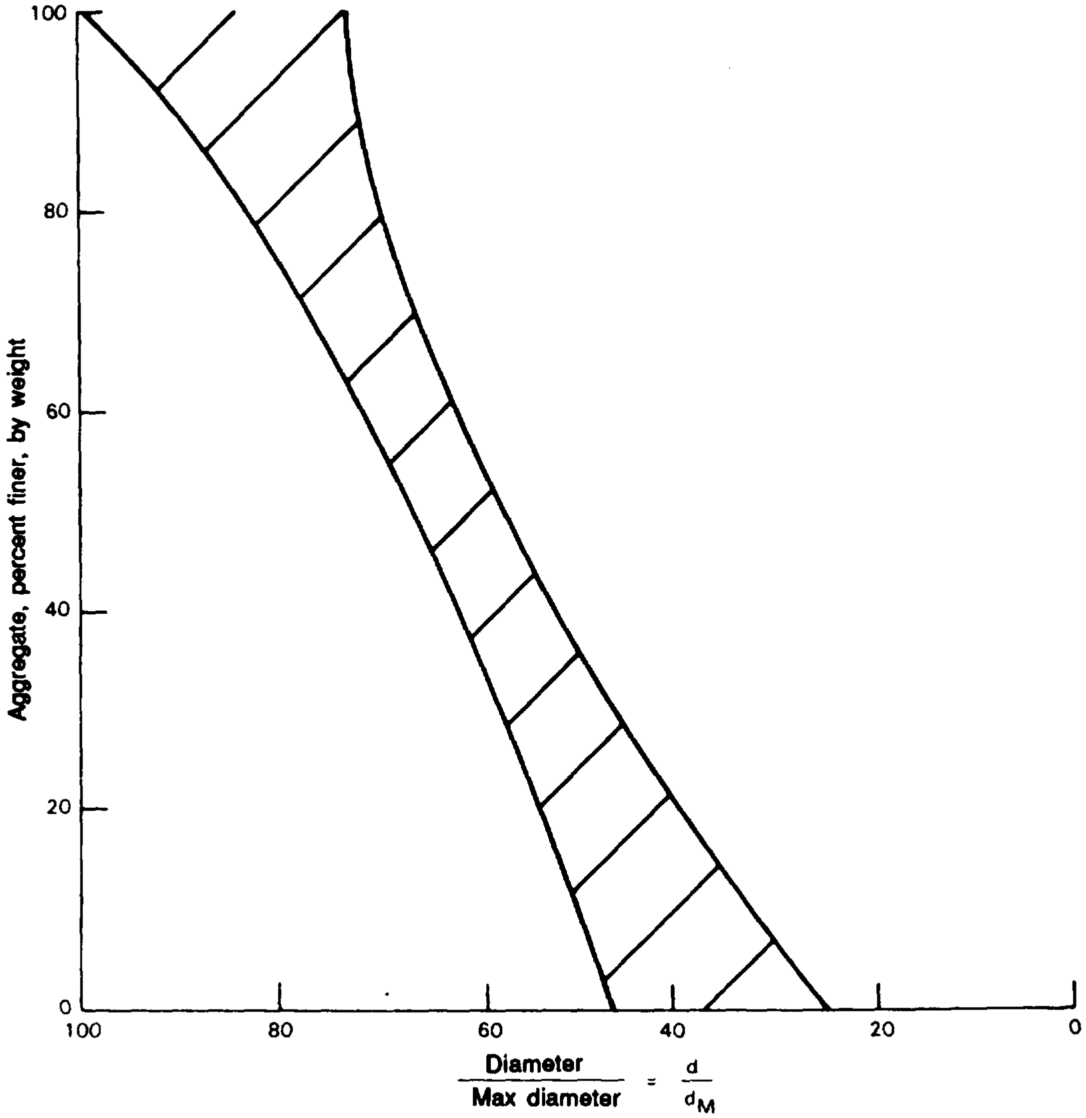


Figure 6-75. Recommended gradation of stones for riprap

GABIONS

Gabions are large, steel, wire-mesh baskets, usually rectangular and variable in size, designed to solve erosion problems at a low cost, Widely used in Europe, gabions are now accepted in the United States as a valuable and practical construction and maintenance tool. They can be used in place of sheet piling, masonry construction, or cribbing.

Description and Assembly

Gabions are supplied from manufacturers in flat, folded bundles. For ease in handling and shipping, the number of gabions per bundle varies according to the size of the gabions. The box gabion is a rectangular cage or basket formed of woven, hexagonal, galvanized steel, wire mesh with 4- to 8-inch openings and divided by diaphragms into cells. To assemble, remove a single gabion from the bundle and unfold it on a hard, flat surface to straighten unnecessary creases and kinks. Fold the front, back, and end panels to a right angle to form a box, as shown in Figure 6-76. Securely lace the vertical edge and diaphragms with binding wire,

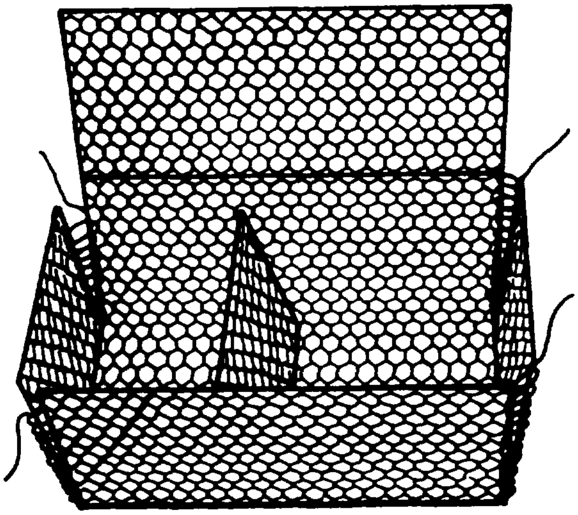


Figure 6-76. Assembly of a gabion

Installation

Before placing the gabions, make the ground surface relatively smooth and even. Place the assembled gabions in position singly or wired together in groups suitable for handling. It is convenient to place the gabions front-to-front and back-to-back to expedite the stone-filling and lid-lacing operation. Lace the basket along the perimeter of all contact surfaces. Where there is more than one course of gabions, the base of the empty gabions placed on top of a completed row must be tightly wired to the latter as shown in Figure 6-77.

When using 3-foot-high gabions, place them empty and lace for approximately 100 linear feet. Anchor the first gabion firmly and apply tension to the other end with a come-along or by other means to achieve the proper alignment. Anchoring can be accomplished by partially filling the first gabion with stone. While the gabions are being stretched, inspect all corners to make sure the lacing is secure and the corners are closed. Keep gabions taut while they are being filled with stone.

Where water, soil, and atmospheric conditions allow, galvanized wire mesh can have a life of 40 years or more, For soils and water showing a pH factor of less than 7 or more than 12, plastic-coated wire must be used to form gabions.

Filling Procedures

The best filling material is one that allows flexibility in the structure and, at the same time, fills the gabion compartments with a minimum of voids and maximum weight. Ideally, the stone should be small, just slightly larger than the size of the mesh (usually 4 to 8 inches). The stone should be clean, hard, durable, and resistant to weathering and frost action.

Fill the gabions to a depth of 1 foot. Then place one connecting wire in each direction and loop around two meshes of the gabion wall, as shown in Figure 6-77. Repeat this operation twice or until the gabion is filled; then fold the top shut and wire it to the ends, sides, and diaphragms.

Pack the stone inside the compartment as tight as practical, To protect the vertical diaphragm during the filling operation, temporarily place rebars and lace them along the upper edges.

Some manual stone adjustment is required during the filling operation to prevent undue voids. Fill the gabion slightly overfull and allow for subsequent settlement; then lace the lid down with binding wire to the tops of all the sides and to the tops of the diaphragm panels. Since it is necessary to stretch the lid to fit the sides exactly, use a short crowbar or special tool designed for this purpose.

The strong interconnection of all units in a gabion structure is an important feature. It is essential that the lacing be done properly. Adjoining gabions are wired together by their vertical edges. Empty gabions, stacked on filled gabions, arc wired to the filled gabions at front and back.

Gabions may be filled by almost any type of earth-handling equipment such as a payloader, crane, conveyor, or modified concrete bucket. The use of rounded stone, if it is available, reduces the possibility of damage to the galvanized wire during mechanical filling.

When the depth of the water is too great for the gabions to be filled on site, fill them at a dry location nearby and place them underwater by crane or barge,

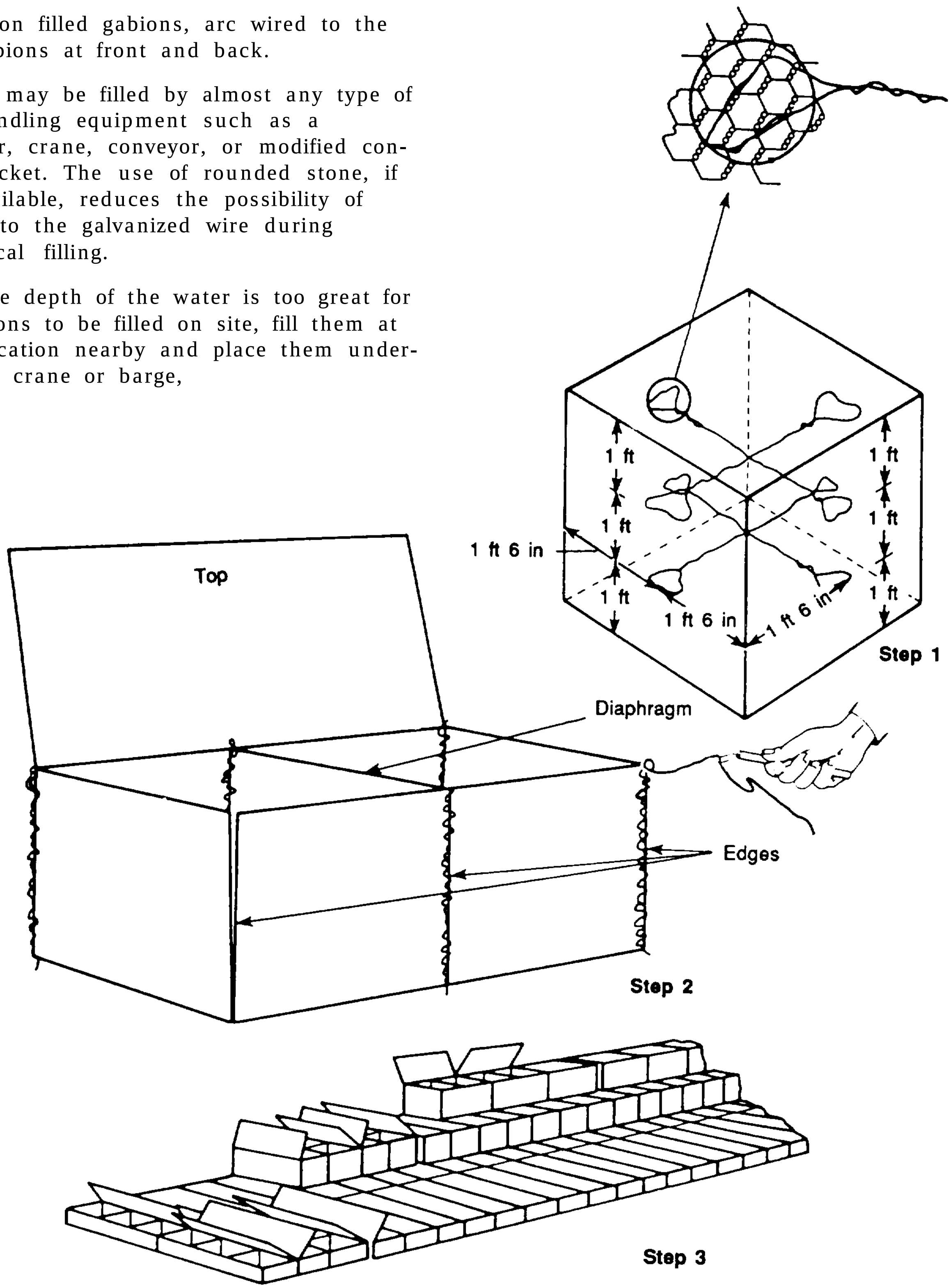


Figure 6-77. Assembly and construction of gabions

Maintenance and Repair

Maintenance and repair are simple procedures; therefore, gabions are inspected at least once a year. Holes can be patched with small panels of mesh, and broken wires can be repaired by using the method shown in Figure 6-78.

Thickness

The thickness of gabions may be calculated by considering the gradient of the channel, the steepness of its slope, the type of material forming the banks and bed, and the curvature of its course. A 12-inch-deep lining is suggested for channels having reasonably straight alignment, side slopes of less than 35 degrees, and a flow velocity of about 10 fps, as shown in Figure 6-79. Use an 18-inch gaion lining for curved channel sections with a side slope of 45 degrees. Use 36-inch stepped-back gabion protection for sharper side slopes. For a steep channel slope, a combination of lining and weirs may be required.

In the case of easily erodible soil, a layer of filter material or permeable membrane of cloth woven of synthetic fibers is required. The gabion should be filled with stone small enough to allow at least two overlapping layers,

In designing a gabion-lined channel, the roughness factor or coefficient (n) in Manning's formula may be assumed to be between 0.025 and 0.030, If the gabions are grouted, the roughness factor can be assumed to be between 0.012 and 0.018.

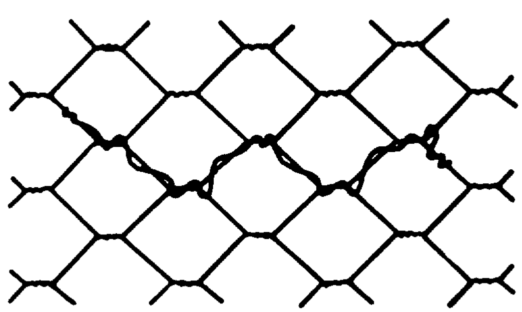


Figure 6-78. Method of repairing a broken wire

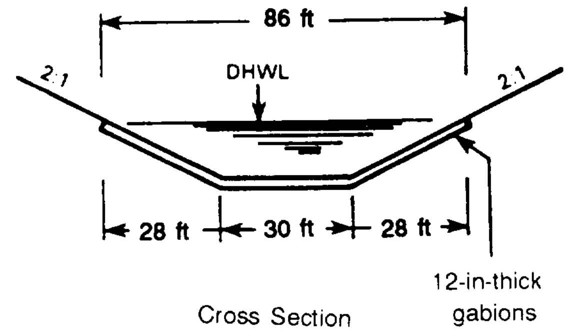


Figure 6-79. Typical channel lining using gabions

Gabion-lined channels may be designed using Manning's equation and the procedures for open -channel design.

Uses

Gabions can be used in the following ways:

- Protective and antierosion structures on rivers (for example, revetments, groins, or spurs).
- Other antierosion structures (for example, weirs, drop structures, and check dams).
- Channel linings.
- Seashore protection.
- Low-water bridges or fords.
- Culvert headwall and outlet structures.
- Bridge abutments and wing walls.

It is often necessary to modify the jnlet and outlet of a culvert by using transition structures to reduce entrance losses and to inhibit erosion. Therefore, the two most common devices for which gabions are used are headwalls and outlet aprons.

Headwalls or wing walls are designed to protect the slopes of an embankment against scour, to increase culvert efficiency by providing a flush inlet as opposed to projecting one, to prevent disjointing of sectional-pipe culverts by anchoring the inlet and outlet, and to retain the embankment slope. These structures are built in a variety of shapes: straight, L-shaped, flared, and warped. A typical plan using a headwall and an outlet apron with a culvert is shown in Figure 6-79. Straight headwalls arc generally used on small, roadside culverts under driveways and in small channels having a low approach veloc ly. They arc also recommended where there is a ten dency for lateral erosion to develop at the outlet.

An apron is often required at the outlet of a culvert to reduce the outlet velocity and thereby inhibit scour. Gabions are well adapted for use here because of their roughness, flexibility, and durability. See Figure 6-80.)

Table 6-24 indicates the type of gabion protection required for various ranges of outlet velocities

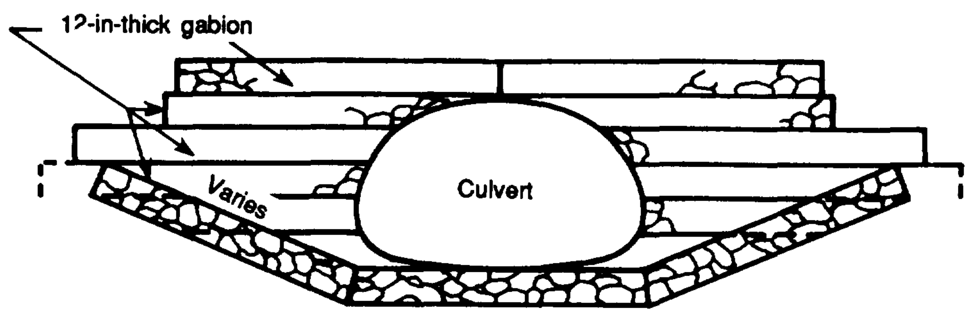


Figure 6-80. Culvert inlet or outlet using gabion headwall and channel lining

Table 6-24. Requirements for gabion protection

Calculated outlet velocity (fps)	Length of apron (ft)	Thickness of apron (in)
Less than 7	No apron is needed; good grass cover is adequate	—
7 to 10	10	12
10 to 15	13 to 23	18
More than 15	Energy dissipator or stilling basin required	—

CULVERT OUTLETS

Most culverts operate under free outfall conditions (that is, there is no control of tailwater), and the discharge possesses kinetic energy in excess of that occurring naturally in the waterway. This excess kinetic energy must often be dissipated to control damaging erosion. The extent to which protective works are required for energy dissipation depends on the amount of excess kinetic energy and the characteristics of the material in the outlet channel. In general, scour occurs at average velocities in excess of about 1.5 fps in uniform-graded sand and cohesionless silts, 2.5 fps in well-graded sand, 3.0 fps in silty sand, 4.0 fps in clay, and 6.0 fps in gravel. These velocities should be used only as a general guide.

If possible, make a study of local material to determine its erosion tendencies prior to a decision on the degree of protection required. The study should consider three types of outfalls offering three degrees of protection: plain outlets, transitions, and stilling basins. Plain outlets provide no protective works and depend on natural material to resist harmful erosion. Transitions provide little or no dissipation on the works themselves but result in a spreading of the effluent jet to approximate the cross-section flow of the natural channel, thus reducing the concentration of energy prior to releasing the flow to the outlet channel. Stilling basins result in dissipation of energy on the protective works.

PLAIN OUTLETS

If the discharge channel is in rock or a material highly resistant to erosion, special erosion protection is not required. This type of outlet should be used only if the material in the outlet channel can withstand velocities about 1,5 times the velocity in the culvert, At such an outlet, side erosion from eddy action or turbulence is more likely to prove troublesome than bottom scour.

Can tilevered culvert outlets may be used to discharge a free-falling jet onto the bed of the outlet channel. As a result, a plunge pool will be developed. The depth and size of the plunge pool depend on the energy of the falling jet at the tailwater and the erodibility of the bed material.

TRANSITIONS

Outlet headwalls and wing walls serve the dual purpose of retaining the embankment and limiting the outlet transition boundary. Erosion of embankment toes can be traced to eddy attack at the ends of such walls. A flared transition is effective if it is proportioned so that eddies induced by the effluent jet do not continue beyond the end of the wall or overtop a sloped wall.

As a guideline, it is suggested that the product of velocity and flare angle not exceed 150 degrees. For example, if effluent velocity is 5 fps, each wing wall may flare 30 degrees; but if velocity is 15 fps, the flare should not exceed 10 degrees. Unless wing walls can be anchored on a stable foundation, a paved apron between the wing walls is required. Special care must be taken in the structure design to preclude undermining.

A newly excavated channel may be expected to degrade, Proper allowance for this action should be included in establishing the apron elevation and depth of cutoff wall. Warped end walls provide excellent transitions that result in the release of flow in a trapezoidal cross section which approximates the cross section of the outlet channel. A warped transition is made at the end of the curved section to reduce the possibility of overtopping as a result of su - perelevation of the waler surface. A paved apron is required with warped end walls. Riprap is usually required at the end of a transition-type outlet.

STILLING BASINS

At culvert outlets where a high concentration of energy or easily eroded materials make excessive erosion likely, a stilling basin or other energy-dissipating device is required. For TO construction, riprap or simple, concrete stilling basins are usually required. There are many types of energy-dissipating devices such as hydraulic-jump basins, roller buckets, flip buckets, impact-energy-dissipating devices, and stilling wells. In unusual cases involving major structures, the use of a special type of device should be considered. Three types of dissipators which may offer a solution are the hydraulic-jump stilling basin, with details developed at the St, Anthony Falls Hydraulic Laboratory; the impact-energy dissipator, with details developed at the hydraulic laboratory of the Bureau of Reclamation; and stilling wells, These dissipa tors are beyond the scope of TO construction, Design procedures are not ineluded in this manual.

SOILS TRAFFICABILITY

CHAPTER 7

Soils trafficability is the capacity of soils to support military vehicles. This chapter includes information on the following topics:

- *Operating and maintaining the soil-trafficability test set.*
- *Measuring trafficability with the results of the tests performed by the cone penetrometer and remolding equipment.*
- *Making trafficability estimates from terrain data (topography and soil data) and weather conditions. The procedures in this chapter are conservative estimates for field use. Engineers must be cautious as the calculated results can vary by 20% or more from changes in tire pressure and deflection. Plan for the unexpected!*

This chapter discusses the trafficability of fine- and coarse-grained soils. Organic soils [muskeg.) and snow are not discussed.

The trafficability of fine-grained soils (silts and clays) and sands that contain enough fine-grained material to behave like fine-grained soils when wet is more difficult to assess than trafficability in coarse-grained soils (clean sands). Relationships that describe the soil-vehicle interactions are based on soil shearing-resistance measurements made with the cone penetrometer and corrected for soil remolding under vehicle traffic by the remolding index [RI) procedures.

The information presented in this chapter is limited to problems associated with soils. It does not include problems associated with natural or man-made obstacles (such as forests or ditches) nor Information on vehicle characteristics (such as the maximum tilt or side angle at which a vehicle can climb without power stall or overturning), The basic principles for the procedures presented are sound for temperate and tropical climates and for soils that have been subjected to freeze-thaw cycles, if they are not frozen at the time of testing and passage of traffic.

Originally, this chapter was designed to permit calculation ns of trafficability by field personnel with only a hand-held calculator. Performances were estimated for a minimum number of vehicle passes (1) or a maximum of 50 vehicles in the same ruts, Today most relations are used for one pass and the combined effects on vehicle performance of terrain features such as soil, vegetation, and slope can only accurately be determined through the use of the computerized Army mobility prediction system contained in the NATO Reference Mobility Model (NRMM). The engineering relationships which produce vehicle speed predictions or GO/NO GO performance based on measured terrain and vehicle characteristics are contained in the NRMM. This chapter only introduces fundamental relationships, terminologies, and illustrations of this computerized, comprehensive mobility evaluation tool. Most military units have access to NRMM relationships through personal computer-based NRMM versions of mobility predictions such as the Comprehensive Army Mobility Modeling System (CAMMS).

BASIC TRAFFICABILITY FACTORS

The following factors impact soil trafficability:

SOIL STRENGTH

Bearing and traction capacities of soils are functions of their shearing resistance. Shearing resistance is measured by the cone penetrometer and is expressed in terms of cone index (CI). Because the strength of fine-grained soils (silts and clays) may increase or decrease when loaded or disturbed, remolding tests are necessary to measure any loss of soil strength expected after traffic. The fine-grained soil CI multiplied by the RI produces the rating cone index (RCI) used to denote soil strength corrected for remolding. A comparison of the RCI with the vehicle cone index (VCI) indicates whether the vehicle can negotiate the given soil condition for a given number of passes. For example, if a soil has a CI of 120 and an RI of 0.60 in its critical layer, the soil strength may be expected to fall to 120 times 0.60, or an RCI of 72, under traffic. Therefore, such soil is not trafficable for vehicles with VCIs greater than 72. If a vehicle has a minimum soil-strength requirement of 72 for one pass, its VCI_1 is 72 and an RCI of 72 is required for the vehicle to complete one pass without immobilization. Appendix D of this manual summarizes VCIs for military vehicles.

STICKINESS

Stickiness may seriously hamper vehicles operating in wet, fine-grained soil. Under extreme conditions, sticky soil can accumulate in a vehicle's running gears, making travel and steering difficult. Normally, stickiness is troublesome only when it occurs in soils of low-bearing capacity (normally, fine-grained soils).

SLIPPERINESS

Excess water or a layer of soft, plastic soil of low LL overlying a firm layer of soil can produce a slippery surface. Such a condition may make steering difficult or may immobilize rubber -tired vehicles. Vegetation, especially when wet and on a slope, may cause immobilization of rubber-tired vehicles. Slipperiness is troublesome, even when associated with soils with high-bearing capacities.

VARIATION OF TRAFFICABILITY WITH WEATHER

Weather changes produce changes in soil trafficability. Fine-grained soils increase in moisture during rainy periods. This results in slipperiness, stickiness, and decreased strength. Dry periods produce the opposite effects. Loose sands improve trafficability through an increase in cohesion during rainy periods and return to the loose, less trafficable state during dry periods. Trafficability characteristics measured on a given date cannot be applied later unless full allowance is made for the changes in soil strength caused by weather. Freezing and thawing conditions can cause extreme variations in the trafficability of soils. Several inches of frozen soil may carry a large number of extremely heavy vehicles. However, when this same material is thawing, it may be impassable to nearly all vehicles. Snow cover can have a significant t effect on the depth of freezing. The absence of snow allows frost to penetrate more deeply into the soil. Techniques have been developed for predicting the effects of weather on soil trafficability. These techniques are part of the comprehensive NRMM and are not included in this publication.

CRITICAL LAYER

The critical layer is the layer in the soil that supports the weight of the vehicle in question. The critical layer's depth varies with the soil type, the soil's strength profile, the vehicle type and weight, and the number of passes required. Table 7-1 summarizes these variations for common military vehicles.

Table 7-1. Critical-layer depth variations

Type of Vehicle	Depth of Normal Critical Layer (Inches)			
	1 Pass		50 Passes	
	F-G Soils*	C-G Soils**	F-G Soils*	C-G Soils**
Tracked vehicles with ground contact pressure less than 4 psi	3 to 9	0 to 6	3 to 9	0 to 6
Wheel load up to 2,000 lb	3 to 9	0 to 6	3 to 9	0 to 6
Wheel load, 2,000 to 10,000 lb	6 to 12	0 to 6	6 to 12	0 to 6
Wheel load over 10,000 lb	9 to 15	0 to 6	9 to 15	0 to 6
Tracked, up to 100,000 lb	6 to 12	0 to 6	6 to 12	0 to 6
Tracked, over 100,000 lb	9 to 15	0 to 6	9 to 15	0 to 6

*Fine-grained soils and remoldable sands
**Coarse-grained soils

NOTE: Vehicle weights are located in Appendix D, along with VCI, and VCI_{50} values for military vehicles.

INSTRUMENTS AND TESTS FOR TRAFFICABILITY

This section contains general information regarding the soil-trafficability test set. The specific use, operating instructions, and test procedures for the soil-trafficability test set are described in detail in Appendix E of this manual. Sieve-analysis tests, plasticity tests, and other field identification tests arc described in Chapter 2 of FM 5-530.

Trafficability measurements are made with the soil-trafficability test set. This set consists of one canvas carrying case, one cone penetrometer with 3/8-inch steel and 5/8-inch aluminum shafts and a 0.5-square-inch cone, one soil sampler, remolding equipment (which includes a 3/8-inch steel shaft and a 0.2-square-inch cone, a 5/8-inch steel shaft with foot and handle, a 2 1/2-pound hammer, a cylinder and base with pin), and a bag of hand tools. The items are shown in Figure 7-1 in their proper places in the carrying case. The set is carried on the back as shown in Figure 7 2, page 7-4. The complete set weighs 19 pounds.

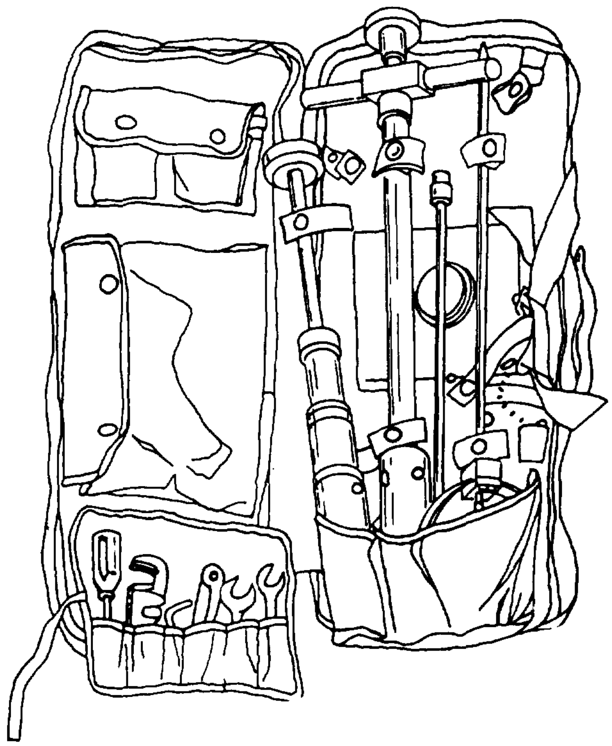


Figure 7-1. Soil-trafficability test set

The primary instrument of the soil-trafficability test set is the cone penetrometer. It is shown in Figures 7-3 and 7-4 and Figure 7-5, page 7-5. It is used to determine the shearing strength of low-strength soils. There is also a dynamic cone penetrometer, but this instrument is used to determine shear strength of high-strength soils such as those found in the base courses of roads and airfields. (The dynamic cone penetrometer is currently developmental and has only limited fielding.) The dynamic cone penetrometer is described in detail in Appendix E. The cone penetrometer consists of a 30-degree cone with a 1/2-inch-square base area, a steel shaft 19 inches long and 3/8 inch in diameter, a proving ring, a micrometer dial, and a handle. When the cone is forced into the ground, the proving ring is deformed in proportion to the force applied.

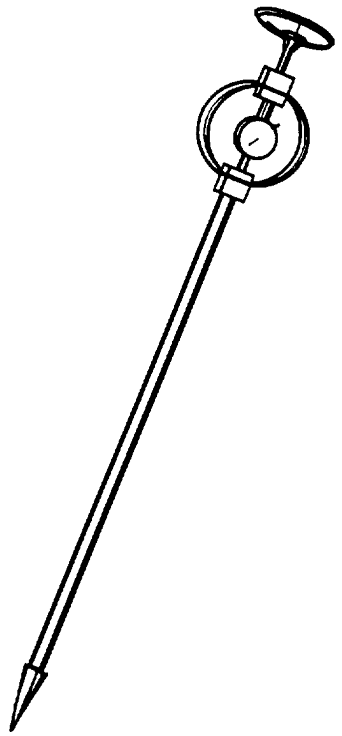


Figure 7-3. Cone penetrometer

Figure 7-2. Carrying a soil-trafficability test set

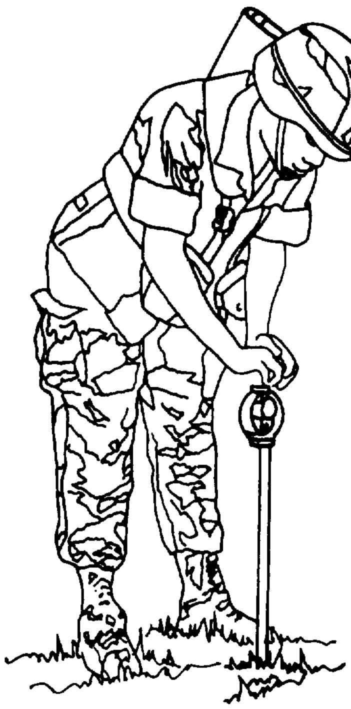


Figure 7-4. Using a cone penetrometer in the upright position

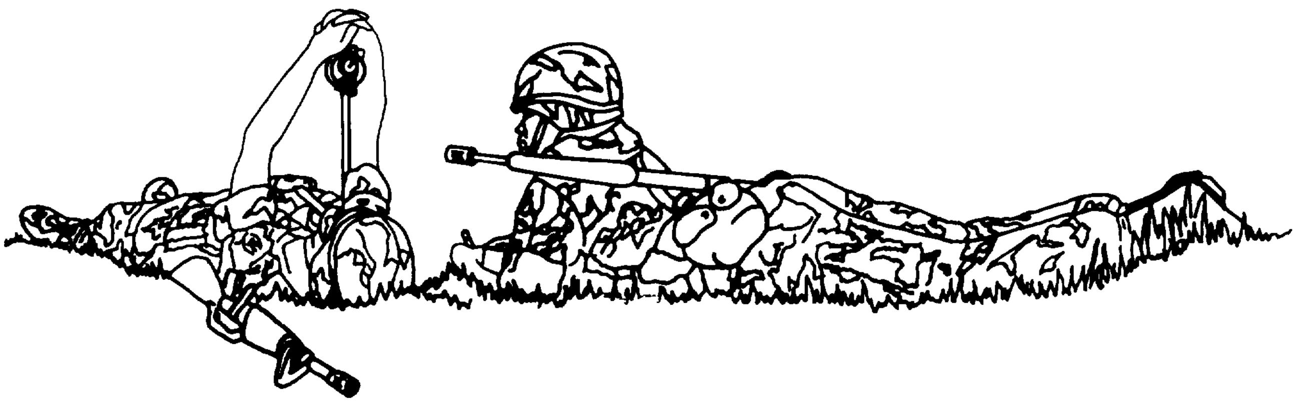


Figure 7-5. Using a cone penetrometer in the prone position

The amount of force required to move the cone slowly through a given plane is indicated on the dial inside the ring. This force is an index of the soil's shearing resistance and is called the soil's CI in that plane. The dial's range is 0 to 300 pounds per square inch (psi). (The actual load applied to the cone penetrometer is 0 to 150 pounds, since the instrument uses a 1/2-square-inch base.) The proving ring and handle are used with a 3/8-inch-diameter steel shaft and the 0.2-square-inch cone for remolding tests in remoldable sands. The cone penetrometer cannot be Used to measure gravels. Gravels arc considered excellent for 50 passes, and any problems can be determined by visual observation.

The specific use, operating instructions, and test procedures for the cone penetrometer as well as the remainder of the soil-trafficability test set are described in detail in Appendix E.

MEASURING TRAFFICABILITY

Whenever reconnaissance parties have time to take trafficability measurements, they should obtain data to determine the number and type of vehicles that can cross the area and the slopes they can climb. The procedures for measuring trafficability are described in this section. Remember that measurements are valid only for the time of the measurement and short periods thereafter, provided no weather changes occur.

RANGE OF CONE INDEXES

A CI ranging between 10 and 300 in the critical layer is required to support most military vehicles. Except for a few vehicles, a CI below 10 is considered to be a nontrafficable area and a CI above 300 is considered trafficable to all but a few vehicles for 50 passes. These limits usually make it possible, while gathering data for trafficability evaluation, to classify large areas as above or below the critical range without extensive testing.

NUMBER OF MEASUREMENTS

The number of measurements taken is determined by the time available, the judgment of the range of soil strengths, and the general uniformity of the area. Trafficability-measuring instruments are designed for rapid observations. The accuracy of the average of any series of readings increases with the number taken. Variations in soft soils require that at least 15 readings be taken to establish a true average CI at any spot at a given depth. The 15 readings should be distributed throughout a uniform area, If time is not available to take a large number of measurements, use judgment to reduce the number according to the following instructions:

- If CIs are between 0 and 150, enough readings should be taken to assure accurate coverage of the area. Readings should be made at enough locations to establish the area boundaries and the average CI within close limits. Four to six sets of readings should be made at each location. Remolding tests (in the case of fine-grained soil and remoldable sand) should be run at a sufficient number of locations to establish the range of RIs If a tentative route can be selected in the field, penetrometer and remolding measurements should be made at closely spaced intervals to locate any soft spots. Where CIs are less than 10, readings should be limited to the number needed to establish the nontrafficable-area limits. No remolding tests are required.

- If the CI ranges from 150 to 200, select enough locations to verify the area limits. Three or four sets of readings should be made at each location. For fine-grained soils and remoldable sands, remolding tests should be made at the first two or three locations. If these show an RI of 0.90 or more, additional remolding tests are not needed. If the RI is below 0.90, sufficient remolding tests should be made to establish the range for the area. This can be established with tests at approximately six locations.

- If the CIs are above 200, a few penetrometer readings will usually verify the extent of the area. Two sets of profile readings taken at each location should be adequate. Remolding tests on soil from the critical layer (fine-grained soils and remoldable sands) should be made at the first two or three locations. If these show an RI of 0.80 or more, no additional remolding tests are needed. Sufficient tests should be made to establish the range for the area if the RI is below 0.80. This can be established with tests at approximately four locations.

Example:

Using the work sheet in Figure 7-6, five tests down to 24 inches were completed at a selected site. The corresponding penetrometer readings are listed in the blocks for the corresponding depth and test. For example, in test number 1 the 0-inch reading is 58, the 6-inch reading is 63, and so on. The individual depth readings are then added and averaged, as in the 0-inch layer. (Always round down.)

Solution:

$$\frac{58 + 63 + 65 + 72 + 75}{5} = 66.6 = 66$$

The numbers in the numerator are the individual readings. The number in the denominator represents the number of tests conducted. The resulting quotient is the average CI for that depth.

After all individual readings are added together, they are averaged with the reading above and below to obtain the average CI for that layer. In the case of the 0- to 6-inch layer, the 66 and 71 are added and then averaged [68). The 68 is the CI for the 0- to 6-inch layer, Readings are then averaged for the 6- to 12-inch layer and so on.

NOTE: Intermediate values for the 3-, 9-, and 15-inch depths (fine grains and remoldable sands) can be interpolated when the vehicle types under consideration require them.

Continuing with the example above, the M929 dump critical layer for one vehicle and for 50 vehicles is 9-15 inches. (To determine the critical layer, refer to the section on critical layers in this chapter, page 7-3.) Because the readings on the cone penetrometer are taken at the 0-, 6-, 12-, 18-, and 24-inch depths, the 3-, 9-, 15-, and 21-inch readings must be interpolated where necessary.

TRAFFICABILITY TEST DATA

1. TEST LOCATION/DATE/TIME	2. TYPE OF VEHICLE/WEIGHT
WM765362/130945ZOCT93	32,700 lbs TRUCK, DUMP, 5 TON, 6X6, M929

SAMPLE

3. CONE INDEX (CI) VALUES

a. TEST NUMBERS	b. DIAL READINGS AT DEPTH 0"	6"	12"	18"	24"
1	58	63	69	73	79
2	63	69	73	75	80
3	65	71	75	77	80
4	72	80	82	87	90
5	75	76	78	79	82
AVERAGE	66	71	75	78	82
NORMAL ~~ABNORMAL~~	68	73	76	80	
c. CI_1 = 74 / d. CI_{50} = 74		3-9" 70	9-15" 74	15-21" 78	

e. CRITICAL LAYER FOR 1 VEHICLE	f. CRITICAL LAYER FOR 50 VEHICLES
9-15"	9-15"

4. REMOLDING INDEX (RI) VALUES

	a. LAYER 9-15"		LAYER 9-15"		LAYER 9-15"		LAYER	
b. DEPTH	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER	BEFORE	AFTER
0"	58	71	63	67	71	83		
1"	60	73	63	68	75	85		
2"	63	75	65	71	79	90		
3"	65	77	66	73	80	94		
4"	70	81	67	78	85	97		
c. AVERAGE	63	75	64	71	78	89		
d. RI = After / Before	1.19		1.10		1.14			

e. RI FOR 1 VEHICLE 1.0	f. RI FOR 50 VEHICLES 1.0
g. RATING CONE INDEX (RCI) $RCI_1 = CI_1 \times RI_1 =$ 74 X 1 = 74	h. SOIL CONDITION *(Describe)* MOIST / SOD COVERED
$RCI_{50} = CI_{50} \times RI_{50} =$ 74X1 = 74	i. WEATHER CONDITIONS *(Describe)* CLOUDY AND COOL TEMP. 43°F RAINED WITHIN LAST 24 HOURS
j. VEHICLE CONE INDEX (VCI) VCI_1 = 30 /CG CI ≥ VCI 74 ≥ 30	
VCI_{50} = 68 /FG & RS RCI ≥ VCI 74 ≥ 68	k. TYPE OF SOIL *(Check one)* ✓ FG ☐ CG
WILL 1 VEHICLE PASS? *(Check one)* ✓ YES ☐ NO	l. SOIL STRENGTH PROFILE *(Check one)* ✓ NORMAL ☐ ABNORMAL
WILL 50 VEHICLES PASS? *(Check one)* ✓ YES ☐ NO	

5. TESTED BY	6. COMPUTED BY	7. CHECKED BY
DONALD H. PURINTON SFC	DONALD H. PURINTON	STEVEN R PASSEY

DD Form 2641, AUG 93

Figure 7-6. Trafficability test data form

To find the 3- to 9-inch layer, the]CI readings for the 0- to 6-inch and the 6- to 12-inch layers are added together and then averaged:

$$\frac{68 + 73}{2} = 70$$

70 becomes the CI for the 3- to 9-inch layer. For the 9- to 15-inch layer, the 6- to 12-inch and 12- to 18-inch layers will be interpolated:

$$\frac{73 + 76}{2} = 74$$

The CI for the 9- to 15-inch layer is 74.

STRENGTH PROFILE

Normal Strength Profile in Fine-Grained soils and Remoldable Sands

In a soil with a normal strength profile, the CI readings either increase or remain constant with each increment of depth. An area with a normal strength profile is shown in Table 7-2. CIs should be measured at 6-inch increments down to 18 inches in the early stages of area reconnaissance. If these measurements consistently reveal that the profile is normal, only readings in the critical layer need to be recorded.

For a tracked vehicle weighing less than 100.000 pounds, such as the M113A3 armored personnel carrier (APC), readings are recorded for the 6- and 12-inch depths. In a normal profile, remolding tests should be run only on samples taken from the normal critical depth for the vehicle in question, since a decrease in RI with increasing depth is not common. The RCI for this layer is used as the criterion of traffic ability for this particular vehicle.

Abnormal Strength Profile in Fine-Grained Soils and Remoldable Sands

An abnormal strength profile has at least one CI reading that is lower than the reading immediately preceding it. An area with

Table 7-2. Examples of normal- and abnormal-soil strength profiles

Depth (inches)	CI Penetration Test Number 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	AV
Area A (Normal Strength Profiles)																
Surface	28	33	32	27	39	31	30	27	29	30	28	31	33	30	32	31
6	47	48	50	51	49	52	53	49	47	48	50	50	53	51	52	50
12	69	67	71	70	69	67	72	73	68	70	71	68	70	73	72	70
18	80	82	83	83	81	70	80	79	77	78	81	79	77	82	83	80
The results from 3 remolding tests made in the 6- to 12-in layer are: 0.86, 0.91, and 0.93. The average remolding index for the layer is 0.90.																
Area B (Abnormal Strength Profiles)																
Surface	32	30	33	31	28	30	29	27	30	31	29	27	32	33	28	30
6	77	78	75	73	76	75	73	78	77	72	74	75	76	72	74	75
12	42	43	45	46	44	47	48	44	42	43	45	46	47	47	46	45
18	38	37	34	36	35	33	32	36	37	37	36	35	33	32	34	35
The results from 3 remolding tests made in the 6- to 12-in layer are: 0.87, 0.90, and 0.93. The average remolding index for the 6- to 12-in layer is 0.90. The results from 3 remolding tests made in the 12- to 18-in layer are: 0.88, 0.91, and 0.91. The average remolding index for the 12- to 18-in layer is 0.90.																
Coarse-Grained Soils																
Surface	15	20	30	40	15	20	25	30	20	20						24
3	80	60	80	70	50	75	70	60	65	75						68
6	120	110	130	110	120	115	125	115	120	125						119
9	180	175	190	175	180	185	190	180	175	180						181
12	200	220	195	205	210	205	200	210	215	220						208
15	250	230	245	230	250	245	240	235	250	240						242
18	300+	250	280	295	300+	275	280	300+	290	290						284+

NOTE: The "+" is normally added to the average to indicate several of the readings exceeded 300. Do not add if less than 20% do not exceed 300.

an abnormal strength profile is shown in Table 7-2.

When an abnormal strength profile exists, CI readings should be made and recorded at 6-inch increments from the top of the normal critical layer (6-inch depth for the M113A3 APC) to 6 inches below the bottom of the normal critical layer (18 inches for the M113A3 APC).

Remolding tests must be run on samples from the normal critical layer and also from the 6-inch layer below it. The lower RCI is used as the trafficability measurement. Low-ground-pressure tracks are an exception to this rule. The 3- to 9-inch layer is always used as the critical layer for these vehicles.

Strength Profile in Coarse-Grained Soils

As indicated in Table 7-1, page 7-3, the critical layer for most vehicles in coarse-grained soils is the 0- to 6-inch layer. Most coarse-grained soils have a normal strength profile with a large increase in strength with depth when compared to fine-grained soils. For this reason, CI measurements should be taken at 3-inch increments to 18 inches or until the maximum capacity (300 CI) of the penetrometer has been reached.

Usually, fewer penetrations are required to establish an average because coarse -grained soil areas generally are more uniform than fine-grained soils and remoldable sands. The RI tests are not required. The strength measurements in a coarse-grained soil area are shown in Table 7-2.

RATING CONE INDEX

The RCI defined earlier is the CI that will result under traffic. This value is compared to the VCI to determine the trafficability of the area for a specific vehicle,

Example:

The following fine-grained soil areas are to be investigated for trafficability for vehicles with a normal critical layer of 6 to 12 inches. Because area A in Table 7-2 has a normal profile, a remolding test was run only for the 6- to 12-inch layer. The RCI for area A is 60 (the average of 50 and 70) x 0.90 = 54. In area B, remolding tests were necessary for both the 6- to 12-inch and 12-to 18-inch layers. In this area, the RCI of the 6- to 12-inch layer is 60 (the average of 75 and 45) x 0.90 = 54, and the RCI of the 12-to 18-inch layer is 40 x 0.90 = 36. The RCI of the 12- to 18-inch layer, 36, is the governing value for the trafficability in area B.

Example:

Using the work sheet in Figure 7-6, page 7-7, the critical layer for one or 50 M929 dumps is 9 to 15 inches.

Samples are removed from the critical layer. For the 9- to 15-inch layer, three tests were conducted, each yielding different results. The average of these results is determined, and this number becomes the RI for that layer.

$$\frac{1.19 + 1.10 + 1.14}{3} = 1.14 \text{ or } 1.0$$

(1.14 exceeds 1.0, so use 1.0.) The RCI_1 is 74 and the RCI_{50} is 74. The VCI_1 is 30, and the VCI_{50} is 68. Comparing the RCI_1 to the VCI_1 and the RCI_{50} to the VCI_{50}, it is determined that one M929 dump can cross the area and 50 M929 dumps also can cross the area because the RCI values are greater than or equal to the VCI values.

Usually a mixture of vehicles will pass through an area, not a column of one vehicle type. Therefore, the VCI and critical layer will be determined for the critical vehicle.

To estimate how many vehicles will cross an area when the RCI is less than the VCI_{50} or to see what the VCI for less than 50 vehicles will be, use the following formula:

$$VCI_{50} - VCI_1 = \Delta VCI$$

$\frac{\Delta VCI}{50}$ will give an increment for one vehicle that, when added to the VCI_1, will give the AVCI for any amount of vehicles up to 50.

Example:

Estimate how many M1A1 tanks can cross a level area with fine-grained soil where the CI is 65 and the RI is 0.80 in the critical layer. For simplicity, we have used this approach on trafficability research. In actuality, the strength increment decreases as passes increase; for example, more strength is required for lower passes than for higher passes, so that more remolding occurs at lower passes than at higher passes. The differences are not linear but can be estimated in the manner shown here.

From Appendix D—

$VCI_{50} = 58$ and $VCI_1 = 25$

$RCI_{50} = 65 \times 0.80 = 52$

To determine the VCI increment per vehicle—

$VCI_{50} - VCI_1 = 58 - 25 = 33$

$$\frac{VCI}{50} = \frac{33}{50} = 0.66$$

Each vehicle adds 0.66 to the VCI_1.

To estimate the number of vehicles that can pass, add 0.66 to VCI_1 until the number is equal to the RCI or one more increase will exceed the RCI. (Remember, this is only an estimate.)

OTHER TRAFFICABILITY EVALUATION FACTORS

In addition to the CI of an area, consider the factors that follow when evaluating traffixability.

Slope

The steepest slope, or ruling grade, that must be negotiated should be determined by studying a contour map. For travel over slopes, the CI requirements must be increased over those required for level terrain.

Stickiness

Stickiness occurs in all fine-grained soils when they are wet. The greater the plasticity of the soil, the more severe the effects of stickiness. Stickiness adversely affects the speed and control of all vehicles but will not cause immobilization except for the smallest tracked vehicles. The worst stickiness is nothing more than a nuisance to larger, more powerful military vehicles. Removing fenders will reduce stickiness effects on some vehicles. Instruments for measuring the effects of stickiness on the performance of vehicles have not yet been devised.

Slipperiness

Like stickiness, the effects of slipperiness cannot be measured. Soils that are covered with water or a layer of soft, plastic soil usually are slippery and often cause steering difficulty, especially in rubber-tired vehicles. Immobilization can occur when slipperiness is associated with low-bearing capacity. The adverse effects of slipperiness are more severe on slopes. Sometimes slopes with adequate soil strength will not be passable because of slipperiness. Chains on rubber-tired vehicles usually improve mobility in slippery conditions. The following categories are used to rate slipperiness:

Condition	Symbol
Not slippery under any conditions	N
Slippery when wet	P
Slippery at all times	S

Vegetation

The effects of vegetation on trafficability are not within the scope of this manual, but some points are worthy of mention, Dense grass, especially if wet, may provide slippery conditions. Additionally, soil strength requirements will be greater than normal if small trees or thick brush must be pushed down by the vehicle.

Organic-Soil Areas

Much of the terrain in northern latitudes is blanketed with a layer of organic material composed of roots, mosses, and other vegetation in various stages of decomposition, Limited testing with military vehicles reveals that low-ground-pressure, tracked vehicles, such as the M973 small-unit support vehicle (SUSV), can travel 50 passes over organic mats that are more than 6 inches thick.

Usually, high-ground-pressure vehicles can travel only a few passes before they break through and become immobilized. Wheeled vehicles usually cannot travel on most of these organic-soil areas. Cone indices denote the relative strength of organic soils. However, the soil-strength vehicle performance relations for organic soils are not as well defined as for fine-grained and coarse-grained soils.

Other Obstacles

A complete assessment of the traffic ability of a given area must include an evaluation of obstacles such as forests, rivers, boulder fields, ditches, and hedgerows. Exact effects of such obstacles on the performance of vehicles are determined by the comprehensive NRMM but are not within the scope of this manual.

APPLICATION OF TRAFFICABILITY PROCEDURES IN FINE-GRAINED SOILS AND REMOLDABLE SANDS

The procedures presented in earlier sections of this chapter are intended for use in tactical operations. Criteria have been established so that when a given area's RCI is equal to or higher than the VCI for 1 or 50 passes (VCI_1 or VCI_{50}) of the selected vehicle, sufficient strength will be available in the soil to withstand 1 or 50 passes of the same vehicle (or vehicles with smaller VCI_1 or VCI_{50}) operating at a slow speed in the same ruts (in the case of 50 passes) and to permit stopping and resumption of movement, if necessary.

SELF-PROPELLED, TRACKED VEHICLES AND ALL-WHEEL-DRIVE VEHICLES NEGOTIATING SLOPES

The maximum slope negotiable and the maximum towing force or gross vehicle weight for the RCI are essentially equal. Therefore, when the RCI is known, the maximum slope negotiable by a given vehicle for 50 passes (or by 50 similar vehicles in straight- line formation) can be estimated from Figure 7-7, page 7-12. The differences in the properties of various soils produce some differences in vehicle performance, so NRMM actually predicts performance in fine-grained soils based on specific soil type.

Example:

Estimate the maximum slope an M1A1 tank can climb for 50 passes where the slope consists of a fine-gralned soil with a CI of 100 and an RI of 0.85 in the critical layers.

Solution:

RCI = 100 x 0.85 = 85
RCI. = RCI - VCI_{50} = 85-58 = 27

Using Figure 7-7, the maximum slope equals 50 percent. The maximum slope the M1A1 can negotiate under the given conditions is 50 percent.

Example:

Estimate the maximum slope an M923 5-ton cargo truck can climb for 50 passes where the slope consists of a remoldable sand whose CI is 93 and RI is 1.00 in the critical layer.

Solution:

RC = 93 x 1.00 = 93
RCI_x = RCI - VCI_{50} = 93 - 68 = 25

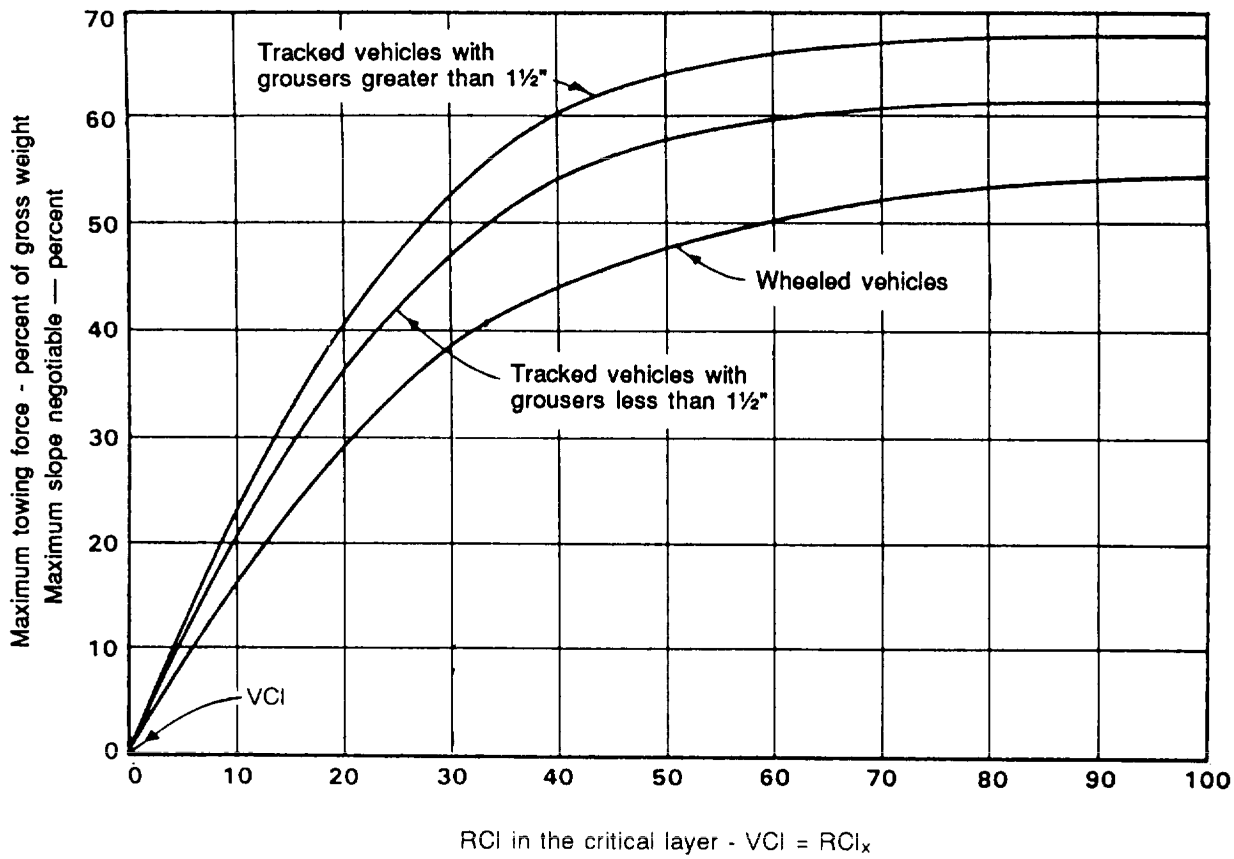


Figure 7-7. Fifty-pass performance curves for self-propelled vehicles operating in fine-grained soils or remoldable sands

In Figure 7-7, at RCI_x = 25, the maximum slope is 35 percent. The maximum slope the M923 truck can climb under the stated conditions is 35 percent.

ONE-PASS PERFORMANCE

The following information is used to determine if various vehicles can make a single pass over different types of terrain:

Self-Propelled, Tracked Vehicles and All-Wheel-Drive Vehicles on Level Terrain

The ability of a given vehicle to make one pass on a straight line on level terrain is assured if the RCI of the area is greater than the VCI for one pass VCI_1). The VCI_1s of most military vehicles are listed in Appendix D.

Example:

Estimate if an M1A1 tank can complete one pass on a level, fine-grained soil with a CI of 50 and an RI of 0.70 in the critical layer. (Use Appendix D to determine the VCI.)

Solution:

$$VCI_1 = 25$$
$$RCI = 50 \times 0.70 = 35$$

Because the RCI is greater than the VCI_1 (35 is greater than 25), the M1A1 tank can complete one pass. Immobilization of a vehicle probably will occur when the RCI is less than the VCI_1. Immobilization may occur even when the RCI is slightly greater than the VCI_1, if water on the soil surface causes excessive sinkage or slipperiness.

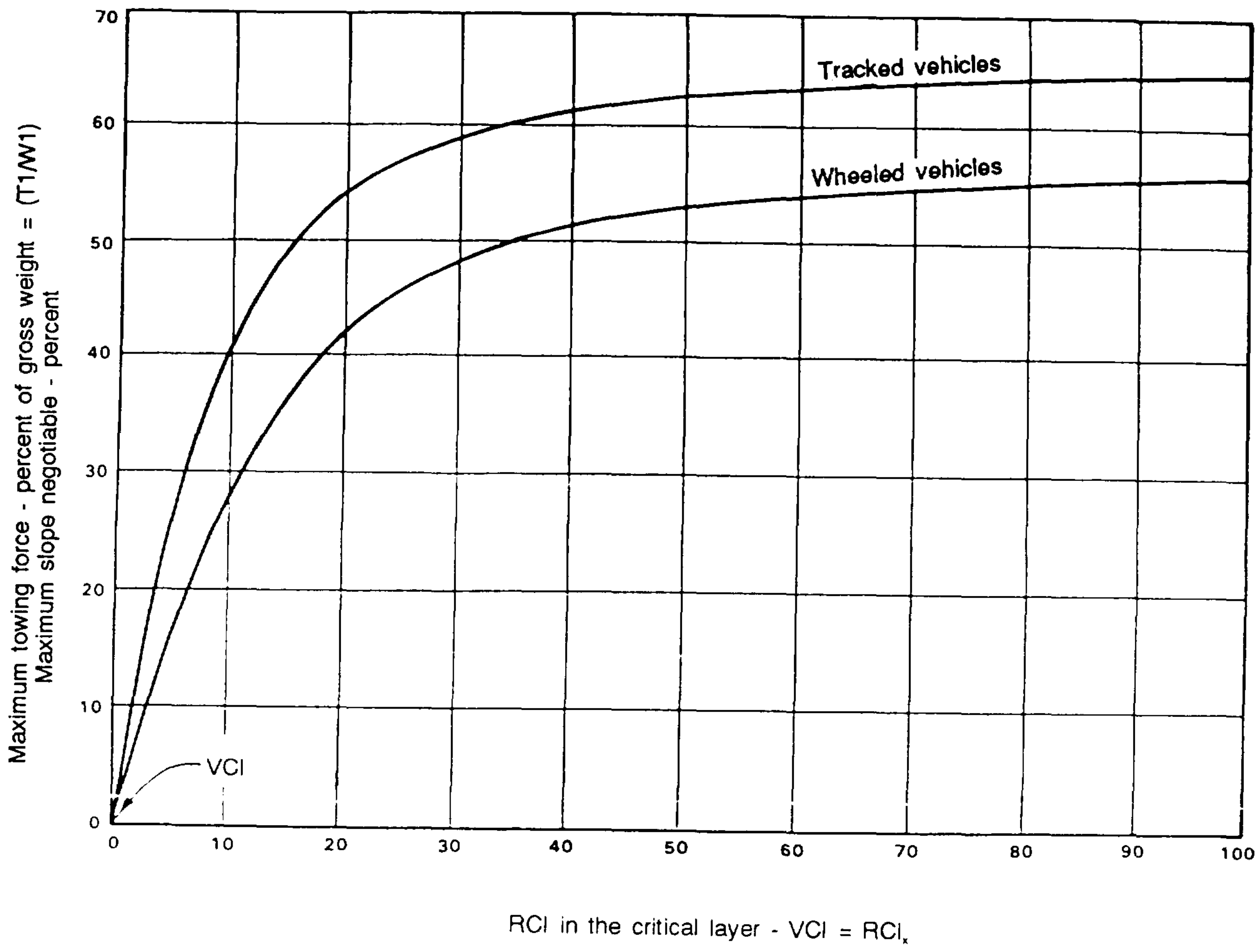


Figure 7-8. One-pass performance curves for self-propelled vehicles operating in fine-grained soils or remoldable sands

Self-Propelled, Tracked Vehicles and All-Wheel-Drive Vehicles Up Slopes

The maximum slope negotiable and the max. imum towing force (as a percentage of gross vehicle weight) for the same RCI_x are essentially equal. Therefore, when the RCI is known, the maximum slope negotiable by a given vehicle for one pass in a straight line up a slope can be determined by using the information in Figure 7-8.

Example:

Determine the maximum slope an M1A1 tank can climb on one pass where the slope consists of fine-grained soil with a CI of 100 and an RI of 0.85 in the critical layer.

Solution:

VCI_1 = 25 (Appendix D)
RCI = 100 x 0.85 = 85
RCI_x = RCI - VCI_1 = 85 - 25 = 60

Using Figure 7-8, at RCI_x = 60, the maximum slope = 63 percent. Under the stated conditions, the maximum slope the M1Al tank can negotiate is 63 percent.

Example:

Determine the maximum slope an M923 truck can climb on one pass where the slope consists of a remoldable sand with a CI of 93 and an RI of 0.40 in the critical layer.

Solution:

VCI_1 = 30 (Appendix D)
RCI = 93 x 0.40 = 37
RCI_X = RCI - VCI_1 = 37 - 30

In Figure 7-8, page 7-13, at RCI_X = 7, the maximum slope = 21 percent. Under the stated conditions, the M923 truck can climb a slope less than or equal to 21 percent.

Vehicles Towing Trailers on Level Terrain and Up Slopes

One-pass performance of vehicles towing trailers is predicted using the comprehensive NRMM and is beyond the scope of this manual. The prediction system is not as well validated as that for single, self-propelled vehicles. Although the procedure for determining the VCI for combinations of trucks or tractor-trailers is not discussed, the VCIs of commonly used combination vehicles are listed in Appendix D.

Vehicles Towing Other Vehicles on Level Terrain

When the RCI is equal to the VCI, the soil has just enough shear strength for the vehicle to overcome its motion resistance. If the vehicle must tow another vehicle, additional shear strength is required to produce the thrust needed to overcome the motion resistance (or required towing force) of the towed vehicle. Thus, RCI - VCI, or RCI_X, is a measure of additional shear strength that allows the vehicle to develop a towing force.

Curves that predict the maximum towing force that can be developed by three types of self-propelled vehicles on level terrain are presented in Figure 7-7, page 7-12. The maximum towing force (expressed as a percentage of vehicle gross weight) is related to RCI_X. Curves that predict the force required to tow vehicles of various weights and types on level terrain are shown in Figure 7-8, where required towing force (expressed as a percentage of vehicle gross weight) is related to RCI. When a vehicle is required to develop a given towing force, the necessary RCI can be determined.

When the RC1 is known, the ability of one vehicle to tow another can be determined.

The determination of VCI for towed tractors and self-propelled vehicles with nonpowered wheels requires calculations on an axle-by-axle basis and is beyond the scope of this manual; therefore, in examples involving vehicles towing other vehicles always refer to the towed vehicles as "inoperable, powered vehicles." The following paragraphs give examples of the application of vehicle performance criteria for both 1 and 50 passes, using Appendix D and Figures 7-7 through 7-10, pages 7-12 through 7-17:

Procedures used in the examples should not be extended to the development of a single VCI for a tractor-trailer combination vehicle. Such development can be reliably made only through the integration of complex considerations which are beyond the scope of this manual. However, some commonly used truck-trailer combination vehicles are listed in Appendix D, where their VCIs are used in the same way the VCIs for other vehicles are used to predict their performance on level terrain.

Example:

Estimate if an M1A1 tank can tow an M923, 5-ton cargo truck for 50 passes on a level, fine-grained soil where the CI is 100 and the RI is 0.80 in the critical layer for the tank, and the CI is 60 and the RI is 0.80 in the critical layer for the truck.

Solution:

From Appendix D:

For the M1A1 tank—

VCI_{50} = 58
Gross weight = 125,000 lb
Grousers < 1 1/2 in

For the M923 truck—

VCI_{50} = 68
Gross weight = 32,500 lb

RCI for the tank = 100 x 0.80 = 80
RCI for the truck = 60 x 0.80 = 48

The maximum towing force (T1) of the tank is read from the curve in Figure 7-7, page 7-12, labeled "Tracked vehicles with grousers less than 1 1 /2 inches." Using this curve, where the RCI_x = 80 - 58 = 22, it is estimated that the tank can tow 25 percent of its weight. Thus, 25 percent of 125.000 = 0.25 x 125.000 = 31.250 lb.

The required towing force (T2) of the M923 truck is read from the curve in Figure 7-8, page 7-13, for 30,000 lb for wheeled vehicles. On this curve, at RCI = 48, T2 = 49 percent of 32,500 = 0.49 x 32,500 = 15,925 lb,

Because the available towing force (31,250 lb) of the M1A1 tank is greater than the required towing force (15,925 lb) for the M923 truck, the tank can tow the truck under the stated conditions.

Example:

Estimate if an M923, 5-ton cargo truck can tow an M1A1 tank for 50 passes on a level, fine-grained soil whose shear strength (CI = 95 and RI = 1.00) is the same for the critical layers for both vehicles. The vehicles are the same as those in the previous example.

Solution:

RCI = 95X 1,00=95
RCI_x = 95 - 68 = 27 for the M923 truck

The maximum towing force (Tl1 of the M923 truck is read from the curve labeled "Wheeled vehicles" in Figure 7-7. On this curve, RCI_x = 27, T1 = 37 percent of 32,500 = 0.37 x 32,500 = 12,025 lb.

The required towing force (T2) of the M1A1 tank is read from the curve in Figure 7-8 that is labeled "75,000 lb" for tracked vehicles. On this curve, at RCI = 95, T2 = 18 percent of 125,000 lb = 0.18 x 125,000 = 22,500 lb.

Because the available towing force (12,025 lb) of the M923 truck is less than the required towing force (22,500 lb) of the M1A1 tank, the truck cannot tow the tank.

Vehicles Towing Other Vehicles Up Slopes

The maximum slope a vehicle towing another vehicle can negotiate is estimated using the following formula:

$$\frac{T1 - T2}{W1 + W2}$$

Where—

T1 = the maximum towing force (in lb) of the towing vehicle

T2 = the force (in lb) required to tow the towed vehicle on level terrain

W1 = weight (in lb) of the towing vehicle

W2 = weight (in lb) of the towed vehicle

NOTE: This formula does not apply to slippery surfaces.

Example:

Estimate the maximum slope that can be negotiated by an M1A1 tank towing an M923 truck for 50 passes, where the slope consists of fine -grained soil whose shear strength is such that the CI is 100 and the RI is 0.85 in the critical layer for the tank, and the CI is 80 and the RI is 0.80 in the critical layer for the truck.

Solution:

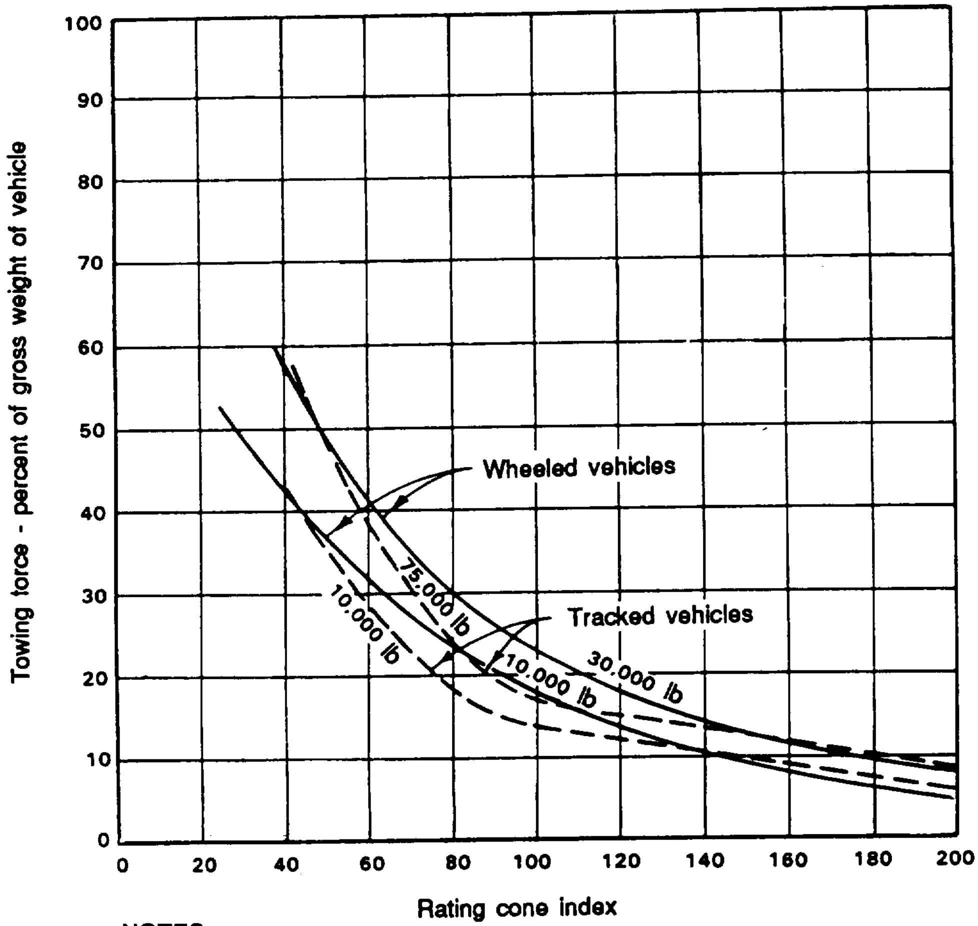
RCI = 100 x 0.85 = 85
RCI_x = RCI - VCI_{50}
= 85 - 58 = 27 for the tank
RCI = 80 x 0.80 = 64 for the truck

Using Figure 7-7, the maximum towing force (Tl) of the MlA1 tank at RCI_x = 27 is 45 percent of 125,000 = 56,250 lb.

In Figure 7-9, page 7-16, the required towing force (T2) of the M923 truck at RCI = 64 is 38 percent of 32,500 = 0.38 x 32,500 = 12,350 lb.

$$\frac{T1 - T2}{W1 + W2} = \frac{56,250 - 12,350}{125,000 + 32,500}$$

$$= 0.279 = 28 \text{ percent}$$



NOTES:

1. The towing force in soft areas where vehicles are bogged down may equal or exceed the weight of the vehicle.

2. These curves also apply to inoperable powered vehicles.

Figure 7-9. Fifty-pass performance curves for vehicles towed in level, fine-grained soils or remoldable sands

Thus, the maximum slope negotiable by the M1A1 tank towing the M923 truck under the given conditions is 28 percent.

Example:

Estimate the maximum slope negotiable by an M923, 5-ton cargo truck towing an M998 high mobility, multipurpose wheeled vehicle (HMMWV) for 50 passes, where the slope consists of fine-grained soil with a CI of 120 and an RI of 1.00 in the critical layer. The M998 is a wheeled vehicle with a gross weight of 7,500 lb.

Solution:

$$RCI = 120 \times 1.00 = 120$$
$$RCI_x = RCI - VCI_{50}$$
$$= 120 - 68$$
$$= 52 \text{ for the M923 truck}$$

Using Figure 7-7, page 7-12, where the RCI_x is 52, the maximum towing force (T1) for the truck is 47 percent, 0.47 x 32,500 = 15,275 lb.

Using Figure 7-9, the required towing force (T2) of the M998 at RCI = 120 is 13 percent of 7,500 = 0.13 x 7,500 = 975 lb.

$$\frac{T1 - T2}{W1 + W2} = \frac{15{,}275 - 975}{32{,}500 + 7{,}500}$$

$$= 0.358 = 36 \text{ percent}$$

Thus, the maximum slope negotiable by the M923 truck towing an M998 HMMWV under the given conditions is 36 percent.

Vehicles Towing Inoperable, Powered Vehicles on Level Terrain [One Pass]

When the RCI is equal to the VCI_1, the soil has enough shear strength for a given vehicle to overcome its motion resistance. If the vehicle is required to tow another vehicle, additional shear strength is required to produce the necessary thrust to overcome the motion resistance (or required towing force) of the towed vehicle. Thus, $RCI - VCI_1 = RCI_x$, which is the additional shear strength that allows a vehicle to develop a towing force when required (for one pass).

Two performance curves, one for self-propelled, tracked vehicles and one for self-propelled, wheeled vehicles, are shown in Figure 7-8, page 7-13. The maximum towing force (expressed as a percentage of the vehicle's gross weight) that can be

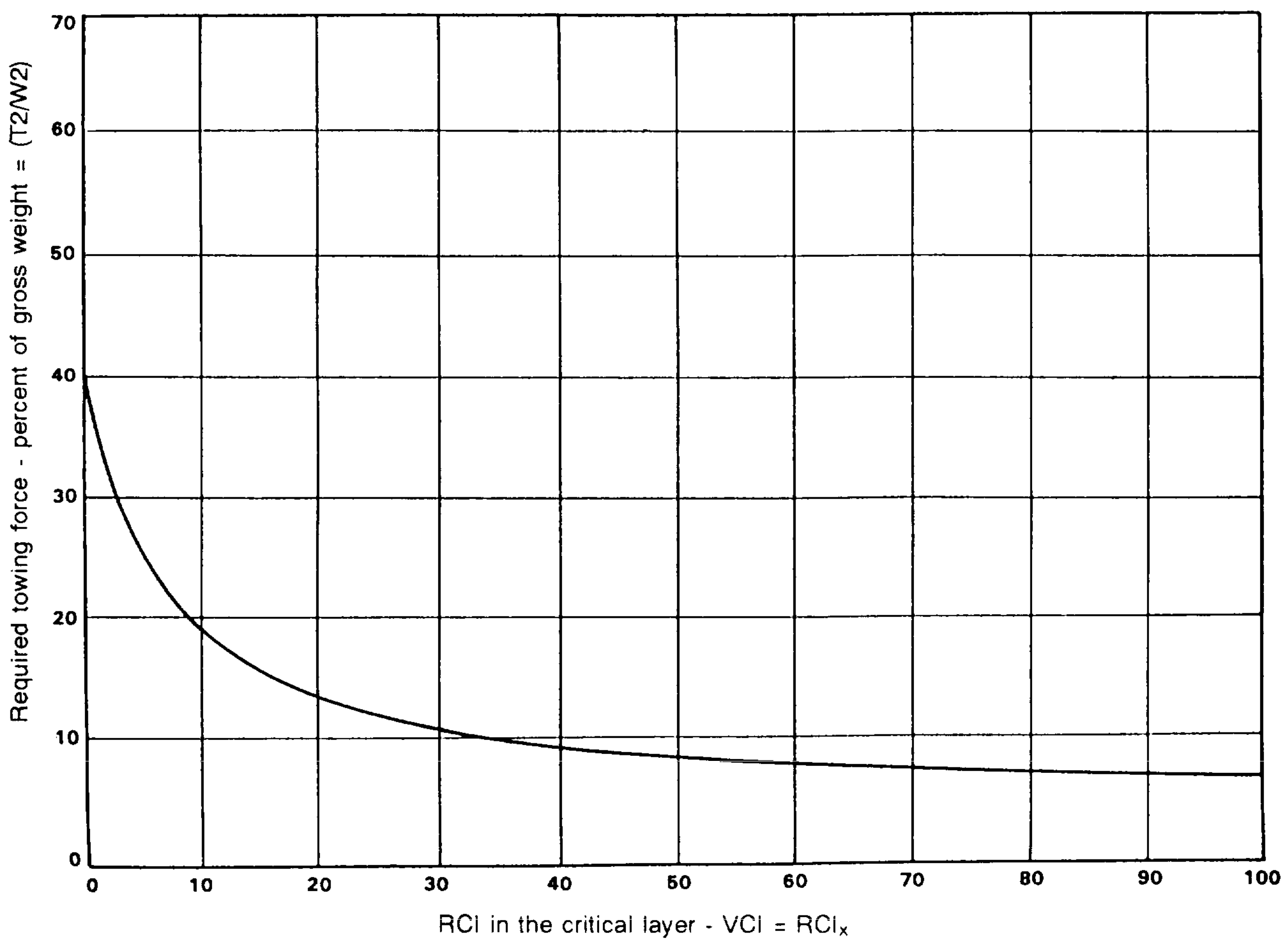


Figure 7-10. One-pass performance curves for vehicles towed in level, fine-grained soils or remoldable sands

developed by a vehicle on level terrain is related to RCI_X. The performance curve for all vehicles when towed on level terrain is shown in Figure 7-10, page 7-17, where the required towing force (expressed as a percentage of the vehicle's gross weight) is related to RCI_X. When the RCI is known, the ability of one vehicle to tow another can be estimated,

Example:

Estimate if an M1A1 tank can tow an M923, 5-ton cargo truck for one pass on level, fine-grained soil whose shear strength is such that the CI is 100 and the RI is 0.70 in the critical layer for the tank, and the CI is 50 and the RI is 0.70 in the critical layer for the truck.

Solution:

For the M1A1 tank, VCI_1 = 25, gross weight = 125,000 lb, and grousers are less than 1 1 /2 inches. For the M923 truck, VCI_1 = 30 and gross weight = 32,500 lb. (See Appendix D.)

For the M1A1 tank, RCI = 100 x 0.70 = 70 and RCI_X = 70 - 25 = 45. In Figure 7-9, page 7-16, at RCI_X = 45, the maximum towing force (T1) = 63 percent of 125,000 = 0.63 X 125,000 = 78,750 lb.

For the M923 truck, RCI = 50 x 0.70 = 35 and RCI_X = 35 - 30 = 5. In Figure 7-10, at RCI_X = 5, the required towing force (T2) = 25 percent of 32,500 = 0.25 x 32,500 = 8,125 lb.

Because the available towing force (78,750 lb] of the tank exceeds the required towing force (8,125 lb) of the truck, the tank can tow the truck under the stated conditions.

Example:

Estimate if an M923, 5-ton cargo truck can tow an M1A1 tank for one pass on level, fine-grained soil whose CI is 95 and RI is 1.00 in the critical layer for each vehicle.

Solution:

RCI = 95 x 1.00 = 95

For the M923 truck, RCI_X = RCI - VCI_1 = 95 - 30 = 65. In Figure 7-8, page 7-13, at RCI_X = 65 the towing force (T1) = 54.6 percent of 32,500 = 0.546 x 32,500 = 17,745 lb.

For the M1A1 tank, RCI_X = RCI -VCI_1 = 95 - 25 = 70. In Figure 7-10, at RCIX = 70 the required towing force (T2) = 8 percent of 125,000 = 0.08 X 125,000 = 10,000 lb.

Because the available towing force of the truck (17,745 lb) exceeds the force (10,000 lb) required to tow the tank, the truck can tow the tank under the stated conditions.

Vehicles Towing Inoperable, Powered Vehicles Up Slopes

The maximum slope a vehicle towing an inoperable, powered vehicle can climb is estimated using the following formula:

$$\frac{T1 - T2}{W1 + W2}$$

Where—

T1 = the maximum towing force (in lb) of the towing vehicle

T2 = the force (in lb) required to tow the inoperable, powered vehicle on level terrain

W1 = weight (in lb) of the towing vehicle

W2 = weights (in lb) of the towed vehicles

NOTE: The relation does not apply to slippery surfaces.

Example:

Estimate the maximum slope that can be negotiated by an M1A1 tank towing an M923 truck on one pass, where the slope consists of fine-grained soil with a CI of 100 and an RI of 0.85 in the critical layer for each vehicle

Solution:

RCI = 100 X 0.85 = 85

For the M1A1 tank, RCI_X = RC1 -VCI_1 = 85 - 25 = 60. In Figure 7-8, at RCI_X = 60 the maximum towing force (T1) = 63 percent of 125,000 = 0.63 x 125,000 = 78,750 lb.

For the M923 truck, RCI_X = RCI - VCI_1 = 85 - 30 = 55, In Figure 7-10, page 7-17, at RCI_X = 55, the required towing force (T2). 8.3 percent of 32,500 = 0.083 x 32,500. 2,698 lb.

$$\frac{T1 - T2}{W1 + W2} = \frac{78{,}750 - 2{,}698}{125{,}000 + 32{,}500}$$

$$= 0.48 = 48 \text{ percent}$$

Thus, the maximum slope negotiable by the M1A1 tank towing the M923 truck under the given conditions is 48 percent.

CLASSES OF VEHICLES

Appendix D contains a list of vehicles divided into four classes: self-propelled, tracked vehicles; self-propelled, wheeled vehicles: construction equipment; and truck-trailer combinations. Each vehicle is identified by its standard nomenclature. Appendix D also includes performance categories for each vehicle and each vehicle's VCI for 1- and 50-pass performance.

PERFORMANCE CATEGORIES

Military vehicles can be divided into seven arbitrary categories according to the minimum CI requirements (VCI_1 and VCI_{50}). The range of VCI_1s and VCI_{50}s for each category (exceptions are numerous) are shown in Table 7-3.

Determination of VCIs for New or Unlisted Vehicles

For conventional-type vehicles not shown in Appendix D, the following procedure can be used to calculate the VCI: First, a mobility index (MI) is calculated for each vehicle.

Table 7-3. Military vehicles and VCI and category of each vehicle

Category	Range VCI_1	Range VCI_{50}	Vehicles
1	12 or less	29 or less	Lightweight vehicles with low contact pressures (less than 2.0 psi)
2	12-21	30-49	Engineer and high-speed tractors with comparatively wide tracks and low contact pressures
3	21-26	50-59	Tractors with average contact pressures, tanks with comparatively low contact pressures, and some trailed vehicles with very low contact pressures
4	26-30	60-69	Most medium tanks, tractors with high contact pressures, and all-wheel-drive trucks and trailed vehicles with low contact pressures
5	31-35	70-79	Most all-wheel-drive trucks, a great number of trailed vehicles, and heavy tanks
6	35-44	80-99	A great number of all-wheel-drive and rear-wheel-drive trucks and trailed vehicles intended primarily for highway use
7	45 or greater	100 or greater	Rear-wheel-drive vehicles and others that generally are not expected to operate off roads, especially in wet soils

Although the NRMM calculates actual VCIs based on an axle-by-axle basis, the VCI can be **estimated** by using the following steps to calculate the MI and VCI for each type of vehicle, assuming equal wheel or track loads and all wheel drive: (The NRMM adjusts for uneven loads and differences in tire pressures. With the newer trucks the VCI_1 may vary 20% with tire pressure changes ONLY.)

NOTE: These formulas could be used to determine estimates of VCI and adjusted by 20% to reflect that drivers of trucks with central tire inflation will reduce as required.

Self-Propelled, Tracked Vehicles.

Step 1. Determine the MI.

$$\text{Mobility index} = \left[\frac{\text{contact pressure factor} \times \text{weight factor}}{\text{track factor} \times \text{grouser factor}} + \text{bogie factor} - \text{clearance factor} \right] \times \text{engine factor} \times \text{transmission factor}$$

wherein

$$\text{contact pressure factor} = \frac{\text{gross weight in lbs}}{\text{area of tracks in contact with ground in square inches}}$$

weight factor:

less than 50,000 lb	= 1.0
50,000 to 69,999 lb	= 1.2
70,000 to 99,999 lb	= 1.4
100,000 lb or greater	= 1.8

$$\text{track factor} = \frac{\text{track width in inches}}{100}$$

grouser factor: grousers less than 1.5 inches high = 1.0
grousers more than 1.5 inches high = 1.1

$$\text{bogie factor} = \frac{\text{gross weight, in lbs, divided by } 10}{(\text{total number of bogies on tracks in contact with ground}) \times (\text{area of } 1 \text{ track shoe in square inches})}$$

$$\text{clearance factor} = \frac{\text{clearance in inches}}{10}$$

engine factor: ≥ 10 horsepower/ton of vehicle weight = 1.00
<10 horsepower/ton of vehicle weight = 1.05

transmission factor: hydraulic = 1.00
mechanical = 1.05

Step 2. Use Figure 7-11 to convert the MI to VCI. [For MIs above 40, the VCI_{50} can be obtained from the equation VCI_{50} = 25.2 + (0.454 x MI).]

Self-Propelled, Wheeled Vehicles.

(1) All-wheel-drive vehicles.

Step 1. Determine the MI.

$$\text{Mobility index} = \left[\frac{\text{contact pressure factor} \times \text{weight factor}}{\text{tire factor} \times \text{grouser factor}} + \text{wheel load} - \text{clearance factor} \right] \times \text{engine factor} \times \text{transmission factor}$$

wherein

$$\text{contact pressure factor} = \frac{\textit{gross weight in lbs}}{\textit{tire width in inches} \times \dfrac{\textit{outside diameter of tires in inches}}{2} \times \textit{number of tires}}$$

weight factor:

Weight range (lbs)*	Weight factor equations
less than 2,000	Y = 0.553X
2,000 to 13,500	Y = 0.033X + 1.050
13,501 to 20,000	Y = 0.142X - 0.420
greater than 20,000	Y = 0.278X - 3.115

$$* \frac{\textit{gross vehicle weight (lbs)}}{\textit{number of axles}}$$

where $X = \dfrac{\textit{gross vehicle weight (kips)}}{\textit{number of axles}}$

Y = *weight factor*

$$\textit{tire factor} = \frac{10 + \textit{tire width in inches}}{100}$$

grouser factor: with chains = 1.05
without chains = 1.00

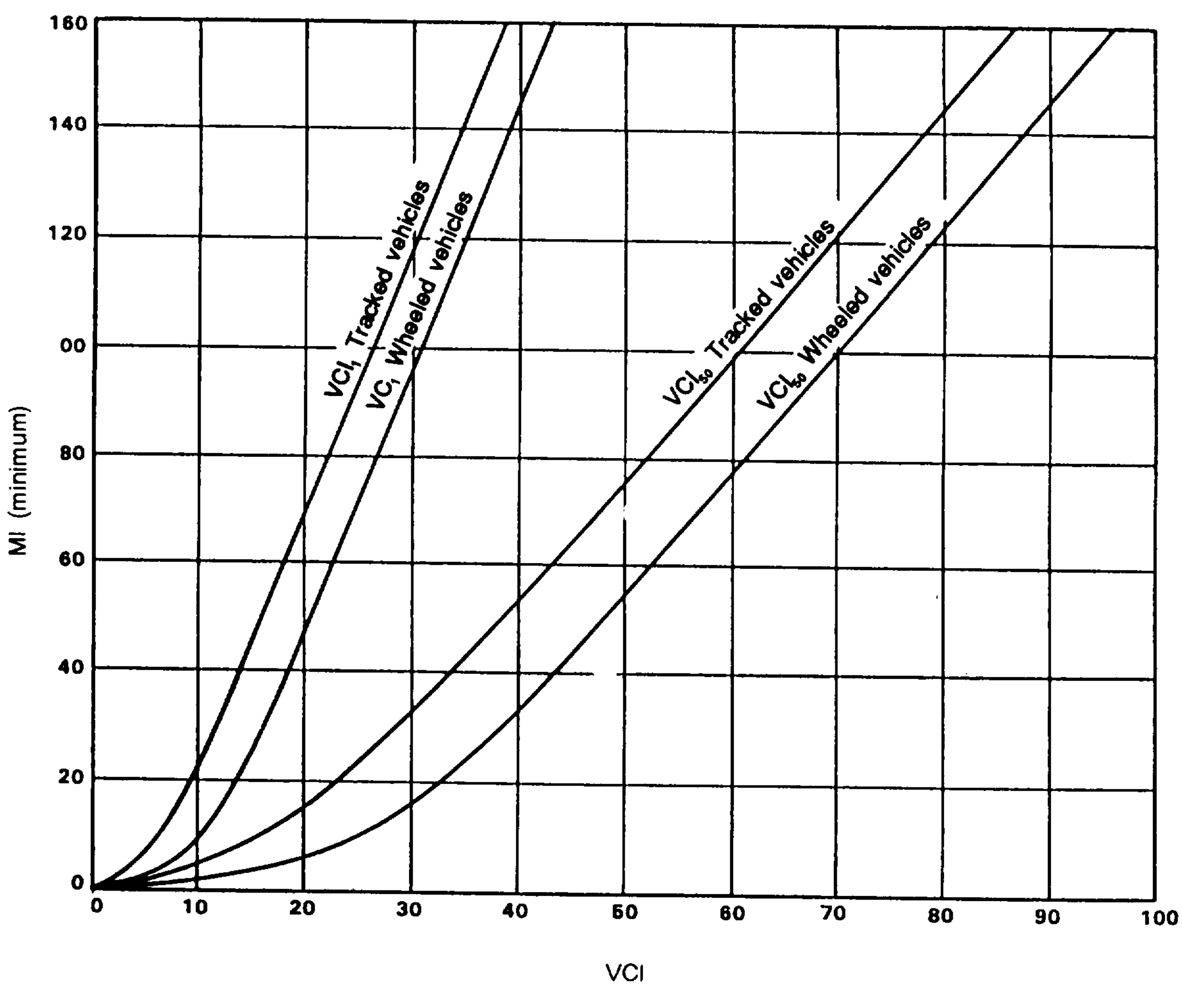


Figure 7-11. Estimated relation of a MI to a VCI

$$\text{wheel load factor} = \frac{\text{gross weight in kips}}{\text{number of wheels}} \quad \text{(wheels may be single or dual)}$$

$$\text{clearance factor} = \frac{\text{clearance in inches}}{10}$$

engine factor : $\geq$ 10 horsepower/ton of vehicle weight = 1.00
$<$ 10 horsepower/ton of vehicle weight = 1.05

transmission factor: hydraulic = 1.00
mechanical = 1.05

Step 2. Enter Figure 7-11, page 7-21, to convert the MI to VCI. [For MIs above 40, the VCI_{50} can be obtained from the equation $VCI_{50} = 25.2 + (0.454 \times MI)$].

(2) Rear-wheel drive vehicles only. If the vehicle being considered is not equipped with an all-wheel drive, the MI is computed according to the formula for all-wheel-drive vehicles, then multiplied by 1.4 to obtain the VCI.

(3) Half-tracked vehicles. The all-wheel-drive formula is used to obtain the VCI of half-tracked vehicles by assuming that the vehicle has wheels instead of tracks on the rear end. The wheels are assumed to be of the same size and have the same load as the front wheels, A grouser factor of 1.1 is used (to account for increased traction provided by the rear tracks).

Towed, Tracked Vehicles.

Step 1. Determine the MI.

$$\text{Mobility index} = \left[\frac{\text{contact pressure factor} \times \text{weight factor}}{\text{track factor}} + \text{bogie factor} - \text{clearance} \right] + 30$$

$$\text{wherein} \quad \text{contact pressure factor} = \frac{\text{gross weight in lbs}}{\text{area of tracks in contact with ground in square inches}}$$

weight factor: 15,000 lb or greater = 1.0
below 15,000 lb = 0.8

$$\text{track factor} = \frac{\text{track width in inches}}{100}$$

$$\text{bogie factor} = \frac{\text{gross weight, in lbs, divided by } 10}{(\text{total number of bogies on track in contact with ground}) \times (\text{area of 1 track shoe in square inches})}$$

clearance = clearance in inches

Step 2. Use Table 7-4, page 7-24, to convert the MI to VCI. [For MIs above 40, the VCI can be obtained from the equation *$VCI_{50} = 25.2 + (0.454 \times MI)$].*

Towed. Wheeled Vehicles,

Step 1. Determine the MI.

$$\text{Mobility index} = 0.64\left[\frac{\text{contact pressure factor} \times \text{weight factor}}{\text{tire factor}} + \text{axle load} - \text{clearance}\right] + 10$$

wherein

$$\text{contact pressure factor} = \frac{\text{normal tire pressure, in lb, per square inch}}{2}$$

weight factor:

15,000 lb per axle or greater	= 1.0
12,500 to 14,999 lb	= 0.9
10,000 to 12,499 lb	= 0.8
7,500 to 9,999 lb	= 0.7
less than 7,500 lb	= 0.6

$$\textit{tire factor: } \text{single tire} = \frac{\text{width in inches}}{100}$$

$$\text{dual tire} = \frac{1.5 \times \text{width in inches}}{100}$$

$$\text{axle load} = \frac{\text{axle load in lb}}{1{,}000}$$

clearance = clearance in inches

Step 2. Use Table 7-5, page 7-25, to convert the MI to VCI. [For MIs above 40, the VCI_{50} can be obtained from the equation $VCI_{50} = 25.2 + (0.454 \times MI)$].

....

Limitations. MIs and resultant VCIs from trailers (towed, tracked and towed, and wheeled vehicles) may be used only for determination of probable RCI that will permit a trailer to complete 25 to 40 passes without the axle or undercarriage dragging.

Table 7-4. Tracked vehicles

Mobility Index vs Vehicle Cone Index For One and Fifty Passes														
MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}
0	0.0	1.5	33	12.6	30.3	66	19.7	45.9	99	26.4	60.6	132	33.1	75.1
1	1.3	4.1	34	12.8	30.8	67	19.9	46.4	100	26.6	61.1	133	33.3	75.6
2	2.2	6.3	35	13.0	31.3	68	20.1	46.8	101	26.8	61.5	134	33.5	76.0
3	3.0	8.1	36	13.3	31.8	69	20.3	47.3	102	27.0	62.0	135	33.7	76.4
4	3.7	9.6	37	13.5	32.3	70	20.5	47.7	103	27.2	62.4	136	33.9	76.9
5	4.3	11.0	38	13.7	32.8	71	20.7	48.2	104	27.4	62.8	137	34.1	77.3
6	4.8	12.2	39	13.9	33.3	72	20.9	48.6	105	27.6	63.3	138	34.3	77.7
7	5.3	13.4	40	14.1	33.8	73	21.1	49.1	106	27.8	63.7	139	34.5	78.2
8	5.7	14.4	41	14.4	34.3	74	21.3	49.5	107	28.0	64.2	140	34.7	78.6
9	6.1	15.3	42	14.6	34.8	75	21.5	50.0	108	28.2	64.6	141	34.9	79.0
10	6.5	16.2	43	14.8	35.2	76	21.7	50.4	109	28.5	65.1	142	35.1	79.5
11	6.8	17.0	44	15.0	35.7	77	21.9	50.9	110	28.7	65.5	143	35.3	79.9
12	7.2	17.8	45	15.2	36.2	78	22.1	51.3	111	28.9	65.9	144	35.5	80.4
13	7.5	18.6	46	15.4	36.7	79	22.3	51.8	112	29.1	66.4	145	35.7	80.8
14	7.8	19.3	47	15.7	37.2	80	22.5	52.2	113	29.3	66.8	146	35.9	81.2
15	8.1	20.0	48	15.9	37.6	81	22.7	52.7	114	29.5	67.2	147	36.1	81.7
16	8.4	20.7	49	16.1	38.1	82	23.0	53.1	115	29.7	67.7	148	36.3	82.1
17	8.7	21.4	50	16.3	38.6	83	23.2	53.6	116	29.9	68.1	149	36.6	82.5
18	8.9	22.0	51	16.5	39.0	84	23.4	54.0	117	30.1	68.6	150	36.8	83.0
19	9.2	22.6	52	16.7	39.5	85	23.6	54.4	118	30.3	69.0	151	37.0	83.4
20	9.5	23.2	53	16.9	40.0	86	23.8	54.9	119	30.5	69.4	152	37.2	83.8
21	9.7	23.8	54	17.1	40.4	87	24.0	55.3	120	30.7	69.9	153	37.4	84.3
22	10.0	24.4	55	17.4	40.9	88	24.2	55.8	121	30.9	70.3	154	37.6	84.7
23	10.2	25.0	56	17.6	41.4	89	24.4	56.2	122	31.1	70.8	155	37.8	85.1
24	10.5	25.5	57	17.8	41.8	90	24.6	56.7	123	31.3	71.2	156	38.0	85.6
25	10.7	26.1	58	18.0	42.3	91	24.8	57.1	124	31.5	71.6	157	38.2	86.0
26	11.0	26.6	59	18.2	42.7	92	25.0	57.6	125	31.7	72.1	158	38.4	86.4
27	11.2	27.2	60	18.4	43.2	93	25.2	58.0	126	31.9	72.5	159	38.6	86.9
28	11.4	27.7	61	18.6	43.6	94	25.4	58.4	127	32.1	72.9	160	38.8	87.3
29	11.7	28.2	62	18.8	44.1	95	25.6	58.9	128	32.3	73.4			
30	11.9	28.8	63	19.0	44.6	96	25.8	59.3	129	32.5	73.8			
31	12.1	29.3	64	19.2	45.0	97	26.0	59.8	130	32.7	74.2			
32	12.4	29.8	65	19.4	45.5	98	26.2	60.2	131	32.9	74.7			

NOTE: For MIs above 180, the VCI is obtained from the following equations:
(1 Pass) $VCI_1 = 11.48 + 0.2\ MI - \{39.2/(MI + 3.74)\}$
(50 Passes) $VCI_{50} = 28.23 + .43\ MI - \{92.67/(MI + 3.67)\}$

Table 7-5. Wheeled vehicle

Mobility Index vs Vehicle Cone Index For One and Fifty Passes

MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}	MI	VCI_1	VCI_{50}
0	1.0	3.0	33	17.0	39.9	66	24.1	55.3	99	30.9	69.9	132	37.6	84.3
1	3.4	8.8	34	17.2	40.4	67	24.3	55.7	100	31.1	70.3	133	37.8	84.7
2	5.0	12.8	35	17.5	40.9	68	24.5	56.2	101	31.3	70.8	134	38.0	85.2
3	6.3	15.6	36	17.7	41.4	69	24.7	56.6	102	31.5	71.2	135	38.2	85.6
4	7.2	17.9	37	17.9	41.9	70	25.0	57.1	103	31.7	71.6	136	38.4	86.0
5	8.0	19.7	38	18.1	42.4	71	25.2	57.5	104	31.9	72.1	137	38.6	86.5
6	8.7	21.2	39	18.4	42.8	72	25.4	58.0	105	32.1	72.5	138	38.8	86.9
7	9.2	22.6	40	18.6	43.3	73	25.6	58.4	106	32.3	73.0	139	39.0	87.4
8	9.7	23.7	41	18.8	43.8	74	25.8	58.9	107	32.5	73.4	140	39.2	87.7
9	10.2	24.8	42	19.0	44.3	75	26.0	59.3	108	32.7	73.8	141	39.4	88.2
10	10.6	25.8	43	19.2	44.7	76	26.2	59.8	109	32.9	74.3	142	39.6	88.6
11	11.0	26.6	44	19.5	45.2	77	26.4	60.2	110	33.1	74.7	143	39.8	89.1
12	11.4	27.5	45	19.7	45.7	78	26.6	60.6	111	33.3	75.2	144	40.0	89.5
13	11.7	28.3	46	19.9	46.1	79	26.8	61.1	112	33.5	75.6	145	40.2	90.0
14	12.0	29.0	47	20.1	46.6	80	27.0	61.5	113	33.7	76.0	146	40.4	90.4
15	12.4	29.7	48	20.3	47.1	81	27.2	62.0	114	34.0	76.5	147	40.6	90.8
16	12.7	30.4	49	20.5	47.5	82	27.4	62.4	115	34.2	76.9	148	40.8	91.3
17	13.0	31.1	50	20.8	48.0	83	27.6	62.8	116	34.4	77.3	149	41.0	91.7
18	13.3	31.7	51	21.0	48.5	84	27.8	63.3	117	34.6	77.8	150	41.2	92.1
19	13.6	32.3	52	21.2	48.9	85	28.0	63.7	118	34.8	78.2	151	41.4	92.6
20	13.8	32.9	53	21.4	49.4	86	28.2	64.2	119	35.0	78.6	152	41.6	93.0
21	14.1	33.5	54	21.6	49.8	87	28.4	64.6	120	35.2	79.1	153	41.8	93.4
22	14.4	34.1	55	21.8	50.3	88	28.6	65.1	121	35.4	79.5	154	42.0	93.9
23	14.6	34.6	56	22.0	50.8	89	28.9	65.5	122	35.6	80.0	155	42.2	94.3
24	14.9	35.2	57	22.2	51.2	90	29.1	65.9	123	35.8	80.4	156	42.4	94.7
25	15.1	35.8	58	22.4	51.7	91	29.3	66.4	124	36.0	80.8	157	42.6	95.2
26	15.4	36.3	59	22.7	52.1	92	29.5	66.8	125	36.2	81.3	158	42.8	95.6
27	15.6	36.8	60	22.9	52.6	93	29.7	67.3	126	36.4	81.7	159	43.0	96.0
28	15.8	37.3	61	23.1	53.0	94	29.9	67.7	127	36.6	82.1	160	43.2	96.5
29	16.1	37.9	62	23.3	53.5	95	30.1	68.1	128	36.8	82.6			
30	16.3	38.4	63	23.5	53.9	96	30.3	68.6	129	37.0	83.0			
31	16.6	38.9	64	23.7	54.4	97	30.5	69.0	130	37.2	83.4			
32	16.8	39.4	65	23.9	54.8	98	30.7	69.5	131	37.4	83.9			

NOTE: For MIs above 160, the VCI is obtained from the following equations:

(1 Pass) $VCI_1 = 11.48 + 0.2\ MI - \{39.2/(MI + 3.74)\}$

(50 Passes) $VCI_{50} = 28.23 + .43\ MI - \{92.67/(MI + 3.67)\}$

OPERATION IN COARSE-GRAINED SOILS

Coarse-grained soils present trafficability problems different from those encountered in fine-grained soils. Some important differences are—

- Coarse-grained soils do not respond to the remolding test (except for highly saturated sands).
- Wheeled-vehicle performance is affected more by tire-inflation-pressure changes on coarse-grained soils than on fine-grained soils.
- Level, coarse-grained soils seldom cause immobilization of tracked vehicles or all-wheel-drive wheeled vehicles when operating at low tire-inflation pressures.
- The first pass over a coarse-grained soil area is the most critical, and subsequent passes are usually assured if they are made in the first-pass ruts.

Coarse-grained soil in the dry state is easily recognizable. It is the round, granular material found on most beaches and in sand dunes. When wet, however, it may be confused with remoldable sand or even fine-grained soil. Because coarse-grained soils do not remold there is no need to conduct the remolding test. Use RCI for fine-grained soils and CI for coarse-grained soils (assume RI = 1.0.)

Use the following procedure to ensure the soil in question is coarse-grained: Push the penetrometer into the soil. If the color of the soil near the penetrometer immediately becomes lighter, the internal drainage is good, which signifies a coarse-grained soil, Another test is to confine a soil sample in the remolding cylinder and attempt to penetrate it with the cone penetrometer. If the soil is coarse-grained, it will be difficult or impossible to penetrate.

The presence of vegetation in coarse-grained-soil areas indicates the soil is stabilized and is of high trafficability. This effect is reflected in high CI readings. Testing to date has not permitted development of CI performance relations for tracked vehicles because all tracked vehicles have been able to travel on all level, coarse-grained soils encountered. Furthermore, the effects of soil strength on the performance of a given tracked vehicle are minimal.

One-pass performance of all-wheel-drive vehicles has been determined. In most cases, the first pass is the most critical, and subsequent passes are assured if the first pass is successful,

ALL-WHEEL-DRIVE VEHICLES ON LEVEL TERRAIN

The ability of a given vehicle to travel one pass in a straight line over level terrain is generally assured if the CI of the area is greater than the VCI. The prediction of the performance of a wheeled vehicle in sands is a complex interplay among many vehicle characteristics including tire size, tire pressure, the number of tires, tire construction, vehicle characteristics, and the soil condition (in terms of soil strength and moisture content). Since most natural, sandy soils contain fine-grained materials that cause the soil to behave like a fine-grained soil, only dry-to-moist, poorly-graded sands (SP) are evaluated using coarse-grained vehicle performance relationships in NRMM or CAMMS.

Other sandy soils with appreciable quantities of fine material (SM, SC, SM-SC) are treated in the NRMM as fine-grained soils. The need to evaluate the interplay among terrain and vehicle characteristics requires that the coarse-grained soil predictive relationships be computerized into the NRMM. Therefore, these relationships cannot be simplified to a point where they could be displayed as figures in this chapter. The relationships do follow the trends of the fine-grained relationships, and they compute a minimum soil strength requirement for traffic (VCI_1) based on measured CI (RI is considered 1.0 or greater for these soils). Speed predictions can then be made for these areas.

In coarse-grained soils, the performance of wheeled vehicles is generally affected most by tire pressure. To optimize vehicle performance, tire pressures should be reduced to mud, sand, and snow pressure, at a minimum, or to the emergency tire pressures if speed is not a requirement. In this manner, vehicle ground pressure is decreased from on-road, and vehicle traction is maximized for the terrain conditions encountered.

TRACKED VEHICLES ON LEVEL TERRAIN

As a general rule, tracked vehicles are able to travel on all level, coarse-grained soils regardless of soil strength. Performance predictions in the NRMM are made for two categories of tracked vehicles; flexible, found on most military vehicles, or girderized, found on most bulldozer-type vehicles.

Vehicles with girderized tracks generally produce higher tractive forces on SP soils than do flexible-tracked vehicles.

CALCULATIONS OF VEHICLE CONE INDEX

Calculation of a coarse-grained VCI for a vehicle configuration is considered beyond the scope of this manual. These predictions are made using the comprehensive NRMM. In general, wheeled vehicles operating in sands should use the lowest tire pressures possible and all-wheel-drive for maximum off-road performance. The user should be aware that immobilization can easily occur in these soils, especially if the soils are dry and loose, and should prepare for such emergencies. Tracked vehicles generally do not suffer immobilization on level SP soils.

TRAFFICABILITY DATA

The objective of mapping trafficability data is to provide commanding officers with an estimate of an area's trafficability prior to actual operation. The estimate consists of placing symbols that describe the trafficability of a small area at strategic points on existing maps as shown in Figure 7-12, page 7-28. The maps produced by the techniques described in the following paragraphs are elementary compared with the complicated and comprehensive maps now in production for use with the NRMM.

ESTIMATING

Trafficability can be estimated if weather conditions, soils, and area topography are generally known. Weather and climatic information usually are available, even for remote areas, from meteorological records, climatology textbooks, or personnel interrogation. Soils and topography data may be obtained from topographic, soils, and geologic maps; aerial photos; or interrogation.

The accuracy of the trafficability estimate depends on the type, quantity, and accuracy of the available data. The analyst's ability to interpret the data is also important, especially if soil types must be deduced from geological maps and air photos.

Weather Conditions

For estimating trafficability, consider only two general weather conditions—the "dry season" and the "wet season."

Dry Season. A dry season is defined as a time when climatic and vegetation factors combine to produce, in general, low soil moistures. For temperate, humid climates, such as the United States east of the Mississippi River, the dry season is from about the first of May to the first of November. In this season, evaporation of water from the

Figure 7-12. Photomap with trafficability data

soil is high because of long days, high temperatures, and few clouds, and water is rapidly extracted from the soil and transpired to the atmosphere by growing plants. A dry season may also occur at other times of the year as a result of long periods of fair weather. Areas of arid climates may be considered to be constantly in a dry season.

During the dry season, fine-grained soils and remoldable sands of any type usually are trafficable and, in general, are of higher trafficability than dry, coarse-grained soils. The trafficability of dry, coarse-grained soils is poorer than that of all wet, coarse-grained soils except quicksand. Even in the dry season, trafficability of any type of soils is affected by a high water table that results from underground springs, low-lying and poorly drained soils, or any other cause.

Wet Season. A wet season is defined as a time in which weather conditions combine to produce high soil moistures. In temperate, humid climates, the wet season extends from about the first of November to the first of May. During the wet season, frequent rains, low temperatures, heavy cloud cover, and the absence of growing plants tend to keep soil moisture near a maximum value. Melting of snow and thawing of previously frozen soils may also produce wet soil conditions. Wet seasons may occur at any time as a result of prolonged rains, floods, or irrigation. Adding moisture to a soil affects the strength of that soil; the effect differs with soil types.

Topography and Classification of Soils

The configuration of the soil surface and soil types in a given area are determined from the sources that follow.

Aerial Photos. Elevations and slopes can be estimated by personnel who are properly instructed to read (with the aid of stereopairs) and interpret information in aerial photos. Accurate elevations and slopes can be obtained with mechanical equipment by operators trained to use such equipment. Locations of rivers, forests, escarpments, and embankments can also be obtained from aerial photos.

The techniques for identifying soils from aerial photos are so complex that only well-trained personnel can fully use aerial photos for this purpose. However, some general information can be obtained by personnel with a minimum of training. For example, orchards are usually planted in well-drained, sandy soils; vertical cuts are evidence of deep loessial (silty) soils; and tile drains in agricultural areas indicate the presence of poorly drained soils (probably silts and clays).

In a given photo, light color tones generally indicate higher elevations, sandier soils, and lower soil moisture than those signified by dark color tones. However, the same color tone may not signify the same conditions in the same photo and may signify an entirely different condition in another aerial photo. In addition, natural soil tones may be obscured and modified by tones created by vegetation (natural and cultivated), plowed fields, and cloud shadows.

Geologic Maps. Geologic maps show parent material and age data. With that information and a general knowledge of climate, topography, and vegetation, trained analysts can estimate the soil types likely to be found in the area.

Soils Maps. Trafficability can be estimated rapidly from maps that delineate surface soils according to the USCS, although these maps are scarce. The more common types of soils maps are those using an agricultural system of soil classification. In formation from agricultural maps must be translated into engineering terms before a trafficability estimate can be made. No exact method exists for doing this, but analysts familiar with the classification systems can usually make good translations. For example, the term "loam" in the agricultural classification system usually includes CL and ML soils in the USCS.

Topographic Maps. Physical features such as rivers, streams, cultivation, forests, and roads can usually be identified from a topographic map. Estimates of surface slopes can be made from the contour lines (lines passing through points of equal elevation).

For trafficability classification purposes, topography has been divided into two classes: low topography and high topography, Low-topography areas are those at comparatively low elevations with respect to surrounding terrain, and high-topography areas are those at comparatively high elevations. Absolute elevation has no significance in identifying the topography class. Low-topography areas are usually poorly drained and have water tables occurring within 4 feet of the surface at some time during the year. High-topography areas are usually medium-well to well-drained and do not have water tables within 4 feet of the surface at any time during the year.

TRAFFICABILITY MAPS

A wide variety of mobility-related products can be obtained from computerized mobility models such as the NRMM or CAMMS. Input terrain data includes land use: terrain slope; obstacles; soil types; vegetation type, density, and spacing; surface geometry; linear and hydrologic feature data; and road and trail data. This data is used by the models with input vehicle data to make speed or GO/NO GO predictions for each individual terrain unit (on a quad sheet) formed by the complex interplay of the input variables. Visual displays or hard copies of the video displays can be used for such tasks as vehicle and terrain analysis, operational planning, route selection, convoy planning, or unit-movement preparation.

Comparison visual products can also be obtained to show quad-sheet-sized differences in the mobility performance levels of red or blue vehicles to contrast the performances of a vehicle in a variety of configurations, such as with different tire pressures, with and without towed loads, or with different load configurations. The derived displays can then be enlarged via "zoom" techniques to precisely plan the optimum route for the configuration.

Cross-country traverse or route movements can be configured to show the additive effects on vehicle speeds of mined areas, obstacles emplaced, choke points, or gap crossings, together with changes in vehicle configurations which may result as a consequence of offensive or defensive actions during tactical or combat operations. The models may also be used by materiel and hardware developers to determine the effects of proposed designs or changes on manual vehicle performances. Thus, the uses and displays achievable through the NRMM or CAMMS computer models are basically limited only by the imagination or requirements of the user.

As an example of hard-copy outputs from the mobility models, three graphical products are presented in Figures 7-13 through 7-15, pages 7-31 through 7-33, as they would appear on the console of a CAMMS computer. Figure 7-13 depicts the cross-country speed performance of a US Army M1A1 tank in Germany. The different cross-hatched areas are used to depict cross-country mobility rates of 0-10, 10-20, 20-30, and greater than 30 kilometers per hour (kph) for the tank. The shading and the speed increments are arbitrary. The initial assessment of the display in Figure 7-13 would indicate the platoon can move across this quad at greater than 30 kph except for scattered areas where speeds will drop to 20-30 kph. River and stream crossings will be required in west-east movement across the quad, and these crossings will require 30 minutes except for a few crossing points which would require "zoom"' techniques to locate. Scattered NO GO areas should be avoided, especially those concentrated in the upper portion of the quad. On-road mobility in most areas should exceed 10 kph,

Figure 7-14 is a display of potential landing zones from CAMMS for the same areas as in Figure 7-13. The potential landing-zone map indicates primarily unfavorable landing-zone sites, with favorable sites located in the northwest third and southeast corner of

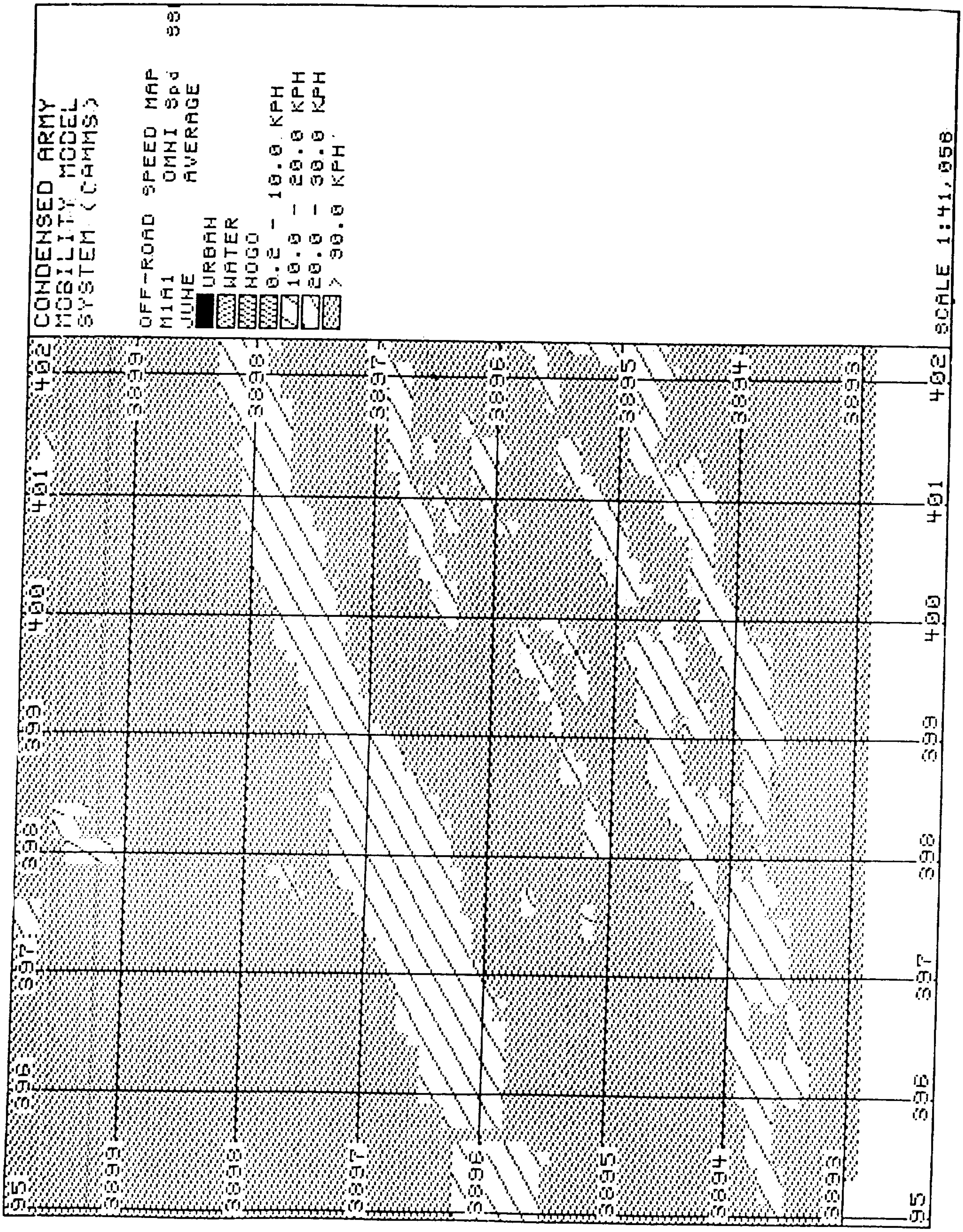


Figure 7-13. Off-road speed map

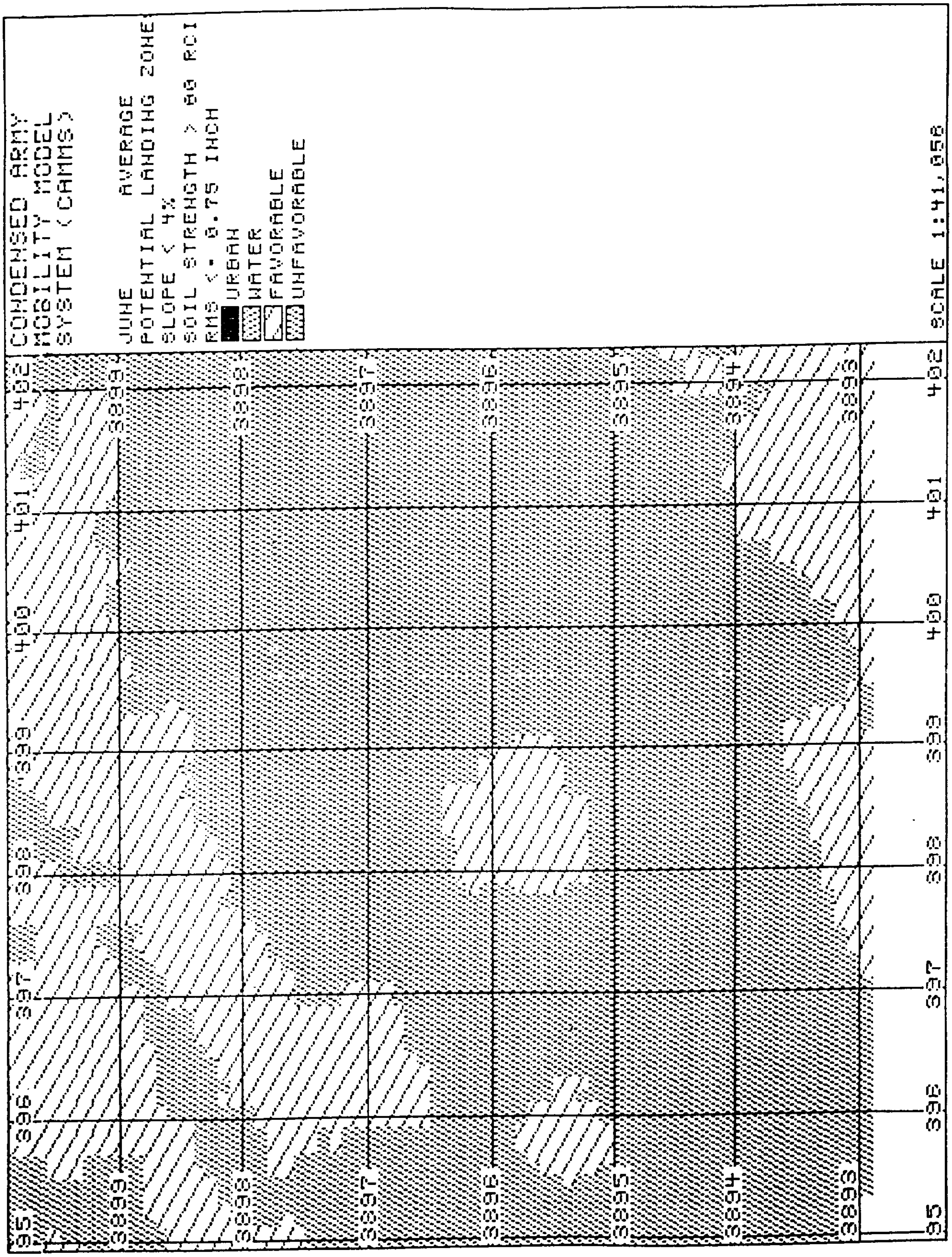


Figure 7-14. Potential landing zones

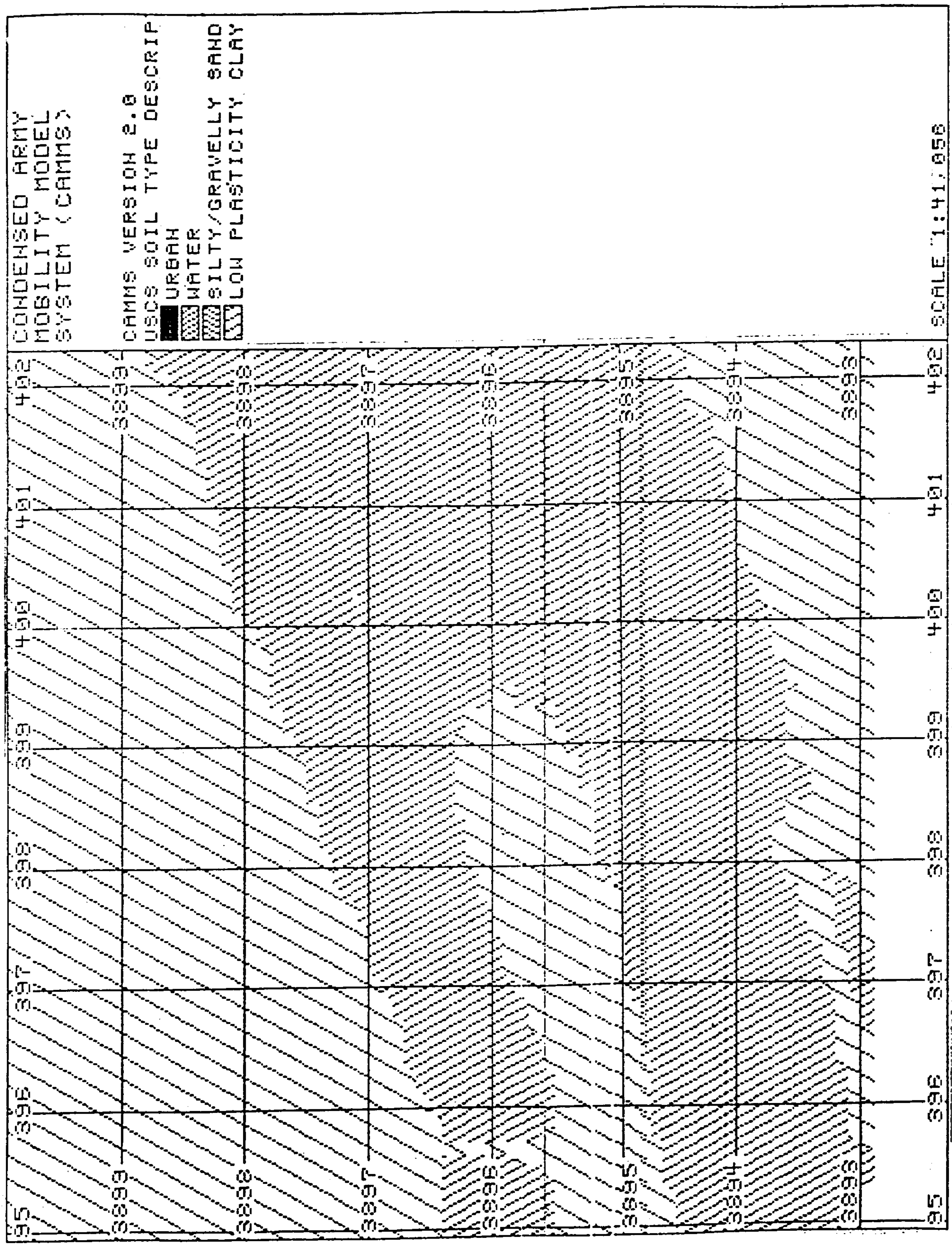


Figure 7-15. USCS soil-type description

the quad. This display helps guide the user to the most suitable sites for landing-zone construction.

Figure 7-15, page 7-33, is a display of USCS soil-type descriptions from CAMMS for the same area as in Figures 7-13 and 7-14, pages 7-31 and 32. This display indicates the locality of soil types that may be useful in scouting for construction materials or sites more suitable for road and airfield construction.

Thus, the mobility analysis is now a very powerful tool for the military planner. It integrates natural features of the landscape with vehicle parameters to produce mobility products which can be used by materiel developers, planners, war garners, or combat soldiers in the field to plan real-time red and blue actions in terrains around the world.

MANUALLY MAPPING SOIL CONDITIONS AND TRAFFICABILITY

Although mapping soil conditions and trafficability through manual means is rarely done since the development of computerized mobility models, it is important that the procedure be presented should the need ever arise. The following paragraphs describe the proper procedure in detail:

VCIs vary widely, and it is desirable to present basic terrain data that can be compared directly with VCIs. The four basic terms describing trafficability are: soil type, RCI, slope, and slipperiness.

The soil-type condition is shown by A, B, C, or D (as defined in Table 7-6); RCI is shown by a single number; slope, in percent, is shown by a single number; and slipperiness is shown by N, P, or S. (Stickiness effects are not considered significant enough to include on maps.) The four factors may be presented (as in Figure 7-12, page 7-28) in fractional form with two items in the numerator and two in the denominator.

Example:

In the fraction $\frac{\text{B - 80}}{\text{25 - S}}$

B is the soil type condition (from Table 7-5, page 7-25), 80 is the RCI, 25 is a 25-percent slope, and S is a slippery surface.

Solution:

To interpret the meaning of $\frac{\text{B - 80}}{\text{25 - S}}$,

first find in Figure 7-7, page 7-12, the RCI_X for the three vehicle types on 25-percent slopes. For wheeled vehicles, RCI_X = 17; for conventional, tracked vehicles, RCI_X = 13; and for long-grousered, tracked vehicles, RCI_X = 11. Then, for each type of vehicle, find the VCI_{50}. Thus, the area is trafficable for wheeled vehicles with VCI_{50} less than 63 (VCI_{50} = RCI - RCI_X or VCI_{50} = 80-17), for conventional, tracked vehicles with VCI_{50} less than 67, and for long-grousered, tracked vehicles with VCI_{50} less than 69.

Since the slope may be slippery, the operations officer should order all wheeled vehicles to be equipped with traction devices and should expect some sliding and steering difficulty. The photomap in Figure 7-12 shows how areas can be delineated in this manner.

Example:

Fifty M60 tanks (102,000 lb) and 50 M923 trucks (32,500 lb) are to be moved from point X to point Y in the area shown in Figure 7-12. Movement must be cross-country because the roadnet is heavily mined.

Solution:

Step 1, From Appendix D:

Vehicle	VCI_1	VCI_{50}
Tank	20	48
Truck	30	68

Step 2. Examine the possibility of single-file travel through flat terrain. All vehicles can negotiate areas 1 and 6. The RCI of 50 for area 3 will allow passage of all 50 tanks but not all 50 trucks. The tanks can proceed in single file from X through areas 1, 3, and 6, consecutively to Y. However,

Table 7-6. Wet-season trafficability characteristics of fine-grained soils and remoldable soils

Group	Soils	USCS	Probable CI Range	Probable RI Range	Probable RCI Range	Slipperiness Effects	Stickiness Effects	Concepts
A	Well-graded gravels Poorly graded gravels Well-graded sands Poorly graded sands	GW CP SW SP	35 to 100	Not applicable	Not applicable	Slight to none	None	Will support continuous traffic of tracked military vehicles or all-wheel-drive trucks with high-flotation tires. Performance will increase with a decrease in tire pressure. Moist sands are good, dry sands only fair. Wheeled vehicles with standard tires may be immobilized in dry sands.
B	Inorganic clays of high plasticity (heavy clays)	CH	55 to 165	0.75 to 1.35	65 to 140	Severe to slight	Severe to slight	Usually will support more than 50 passes of military vehicles. Going will be difficult at times.
C	Clayey gravels, gravel-sand-clay mixtures Clayey sands, sand-clay mixtures Gravelly clays, sandy clays, inorganic clays of low to medium plasticity, lean clays, silty clays	GC SC CL	85 to 175	0.45 to 0.75	45 to 125	Severe to slight	Moderate to slight	Often will not support 40 to 50 passes of military vehicles but usually will support limited traffic. Going will be difficult in most cases.
D	Silty gravels, gravel-sand silt mixtures Silty sands, sand-silt mixtures Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts Organic silts and organic silty clays of low plasticity Organic clays of medium to high plasticity, organic silts	GM SM ML, CL-ML MI OL OM	85 to 180	0.25 to 0.85	25 to 120	Moderate to slight	Slight	Usually will not support 40 to 50 passes of military vehicles. Often will not permit even a single pass. Going will be difficult in most cases.

the trucks will have to fan out and use more lanes with less than 50 vehicles in single file to ensure passage through area 3 (VCI_1 to 30, RCI less than 50). The slipperiness of area 3 may present an insurmountable problem for the trucks unless traction devices are available.

An alternative route that would be safe for all vehicles would be through areas 1, 5, and 6, consecutively, provided the slope of area 5 can be negotiated. For this example, it is assumed that the combination of 60-percent slope and other terrain obstacles in areas 2 and 4 would not allow travel through them.

Step 3. Check the slope-climbing ability (using Figure 7-7 and Figure 7-8, pages 7-12 and 7-13) of both vehicle types in area 5:

1 Pass

Vehicle	Slope	RCI_X	$RCI_X + VCI_1$	=	RCI_{Reqd}
Tank	30%	6	6 + 20	=	26
Truck	30%	11	11 + 30	=	41

50 Passes

Vehicle	Slope	RCI_X	$RCI_X + VCI_1$	=	RCI_{Reqd}
Tank	30%	15	15 + 48	=	63
Truck	30%	20	20 + 68	=	88

The tanks can negotiate the 30-percent slope in single file (available RCI of 80 is greater than the required RCI of 63). All trucks cannot negotiate the slope in single file (available RCI of 80 is less than the required RCI of 88), but they can fan out and negotiate the slope on a one-pass basis (available RCI of 80 is greater than the required RCI of 41). The conclusion is that all vehicles could travel from X to Y through areas 1, 5, and 6, respectively, provided caution is used with the trucks. This route is shown as a dashed line.

This example indicates the usefulness of mapped trafficability data in planning operational exercises.

The presentation of trafficability data for strategic purposes is most effective when one vehicle is used as a standard, reference vehicle. For example, if the vehicle selected has a VCI of 49 and the information on that specific vehicle is presented, trafficability data for that vehicle can be generalized and considered applicable to all vehicles with a VCI of 49 or less. Recommended techniques of mapping trafficability data follow:

- The base of the map should be a standard topographic map printed in a gray monochrome with streams in a strong blue color.
- The four soil-slope combinations should be shown as trafficability symbols, as indicated in Table 7-7. (See previous weather conditions.)
- Obstacles should be indicated by red added numbers circled in red.
- Forests should be indicated by appropriate open-type patterns in strong green.
- The reverse of the trafficability map should contain an inset map of the principle physiographic provinces, landforms, geologic areas, and related data used in making the analysis. The inset map should show in detail all important data on soils, topography, and obstacles that cannot readily be shown on the face of the map.

SOIL-TRAFFICABILITY CLASSIFICATION

Soil classification of a specific area can be accomplished rapidly for seasonal (high-moisture) conditions when the soil has been classified in terms of the USCS, the topography (high or low) has been identified, and the VCI for vehicle category has been determined (from Table 7-3, page 7-19, or Appendix D) or computed, when necessary, as previously described.

Table 7-7. Trafficability symbols

Dry Period	Wet Period	Letter Symbol	Color
Passable	Passable	A	Dark Green
Passable	Doubtful/Impassable	B	Light Green or Green Stripes
Doubtful	Doubtful/Impassable	C	Orange or Yellow
Impassable	Impassable	D	Red
Generally impassable due to steep slope or rough terrain. Soil not evaluated.		E	Red Stripes

FINE-GRAINED SOILS

The trafficability classification of fine-grained soils is shown in Table 7-8, page 7-38.

The interpretation of the example shown in the "Low Topography, High Moisture Condition" graph of Table 7-8 for a level area of MH soil follows:

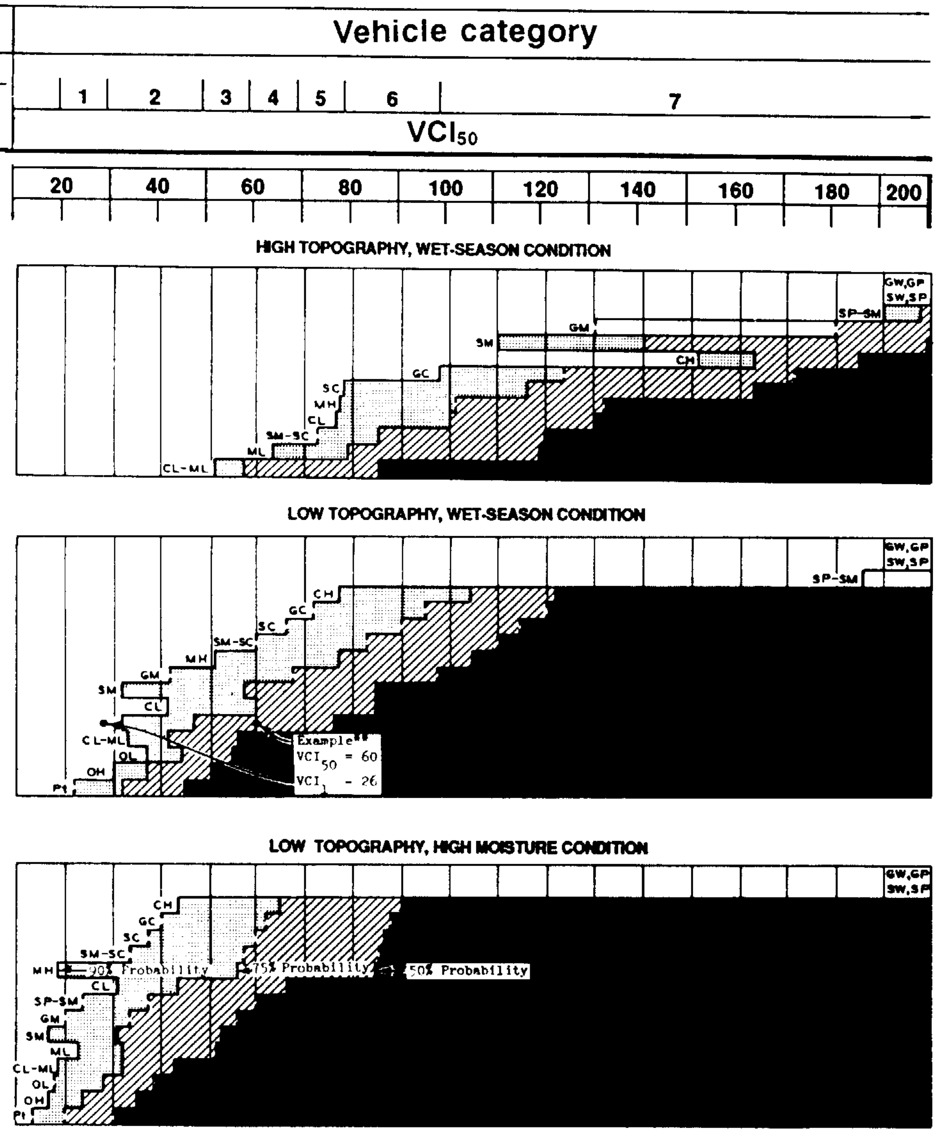
- Vehicles with a VCI_{50} or VCI_1 equal to or greater than 84 will have a less than 50 percent probability of traversing the area.
- Vehicles with a VCI_{50} or VCI_1 equal to or greater than 56, but less than 84, will have a probability equal to or greater than 50 percent, but less than 5 percent, of traversing the area.
- Vehicles with a VCI_{50} or VCI_1 equal to or greater than 18, but less than 56, will have a probability equal to or greater than 75 percent, but less than 90 percent, of traversing the area.
- Vehicles with a VCI_{50} or VCI_1 less than 18 will have a probability equal to or greater than 90 percent, but no more than 100 percent, of traversing the area.

COARSE-GRAINED SOILS

The trafficability classification of coarse-grained soils can be obtained from Figure 7-16, page 7-39. The classification interpretation is the same as for the trafficability of fine-grained soils from Table 7-8. To use Figure 7-16, identify only the coarse-graincd soils (location and origin) and determine the VCIs from the equation presented earlier in this chapter. Figure 7-16 applies to wheeled vehicles only. The effect of the strength of coarse-graincd soils on tracked-vehicle performance is negligible.

Table 7-8. Soil-trafficability classification in USCS terms

Soil type symbol	Strength measurements: Probable range[1] CI	RI	RCI	Mean RCI	Effects of: Slipperiness[2]	Stickiness
High topography, wet-season condition						
GGW, GP	—	—	—	—	NONE	NONE
SW,SP	—	—	—	—	NONE	NONE
SP-SM	125 - 241	1.19 - 2.17	196 - 316	256	NONE	NONE
GM	—	—	—	230	NONE	NONE
SM	130 - 224	0.77 - 1.83	137 -287	212[3]	NONE	NONE
CH	167 - 217	0.84 - 1.10	158 - 210	184	SLIGHT	MODERATE
GC	—	—	—	165[3]	SLIGHT	SLIGHT
SC	127 - 231	0.72 - 0.98	104 - 208	156	SLIGHT	SLIGHT
MH	151 - 211[4]	0.32 - 1.00[4]	64 - 160[4]	112[4]	SLIGHT	SLIGHT
CL	123 - 211	0.59 - 0.95	82 - 180	131	SLIGHT	SLIGHT
SM -SC	147 -185	0.47 - 1.13	65 - 211	138	SLIGHT	NONE
ML	118 - 224	0.46 - 1.02	67 - 189	128	SLIGHT	NONE
CL - ML	111 - 209	0.44 - 072	54 - 136	95	SLIGHT	NONE
Low topography, wet-season condition						
GW, GP	—	—	—	—	NONE	NONE
SW, SP	—	—	—	—	NONE	NONE
SP - SM	300	0.94	282	282[3]	NONE	NONE
CH	98 - 194	0.74 - 1.14	81 - 193	137	SEVERE	SEVERE
GC	—	—	—	130[3]	MODERATE	MODERATE
SC	97 - 257[4]	0.59 - 1.21[4]	61 - 255[4]	158[4]	MODERATE	MODERATE
SM - SC	160 - 216[4]	0.45 - 1.31[4]	72 - 208[4]	140[4]	SLIGHT	NONE
MH	94 - 170	0.51 - 0.99	48 - 162	105	SEVERE	MODERATE
GM	—	—	—	125[3]	SLIGHT	NONE
SM	109 - 217	0.29 - 1.03	34 - 188	111	SLIGHT	NONE
CL	90 - 188	0.46 - 0.88	46 - 146	96	MODERATE	MODERATE
ML	102 - 200	0.27 - 0.81	34 - 134	84	MODERATE	SLIGHT
CL ML	85 - 165	0.31 - 0.69	34 - 96	65	MODERATE	SLIGHT
OL	95 - 135	0.38 - 0.74	41 - 89	65	MODERATE	SLIGHT
OH	64 - 164	0.32 - 0.78	14 - 110	62	MODERATE	SLIGHT
Pt	76 - 90[4]	0.45 - 0.67[4]	41 - 51[4]	46[4]	MODERATE	SLIGHT
Low topography, high moisture condition						
GW, GP	—	—	—	—	NONE	NONE
SW, SP	—	—	—	—	NONE	NONE
CH	69 - 167	0.64 - 1.02	48 - 146	97	SEVERE	SEVERE
GC	—	—	—	90[3]	SEVERE	MODERATE
SC	—	—	—	88[3]	SEVERE	MODERATE
SM - SC	150 - 182[4]	0.45 - 0.63[4]	66 - 98[4]	82[4]	MODERATE	SLIGHT
MH	83 - 151	0.46 - 0.92	43 - 123	83	SEVERE	SEVERE
CL	71 - 165	0.42 - 0.80	39 - 117	78	SEVERE	MODERATE
SP - SM	—	—	—	74[3]	SLIGHT	NONE
GM	—	—	—	72[3]	SLIGHT	SLIGHT
SM	81 - 183	0.15 - 0.87	12 - 126	69	SLIGHT	SLIGHT
ML	81 - 171	0.26 - 0.60	22 - 88	55	SEVERE	SLIGHT
CL - ML	71 - 155	0.27 - 0.53	26 - 66	46	SEVERE	SLIGHT
OL	87[4]	0.56[4]	49[4]	49[4]	MODERATE	SLIGHT
OH	42 - 132	0.26 - 0.66	21 - 49	35	SEVERE	SLIGHT
Pt	76 - 90[4]	0.45 - 0.61[4]	41 - 51[4]	46[4]	SEVERE	SLIGHT



Probability (P_r) of a vehicle (or 50 vehicles) traversing level terrain

Excellent	90% ≤ 100% ≤ 100%	
Good	75% ≤ Pr < 90%	
Fair	50% ≤ Pr < 75%	
Poor	0% ≤ Pr < 50%	

Good reliability, based on analysis of data

Fair reliability, based on judgement

NOTE: Vehicle category and CI range are given in Table 7-3, page 7-19.

[1]Based on +1 and -1 standard deviation from the mean.
[2]Applies only to wheeled vehicles without traction devices.
[3]Estimated from textural, plasticity, and organic properties of soil under given moisture condition.
[4]Based on analysis of less than five samples.

**A vehicle with a VCI of 60 would have a 50 to 75% chance of "GO" on an ML soil of low topography, wet-season condition.

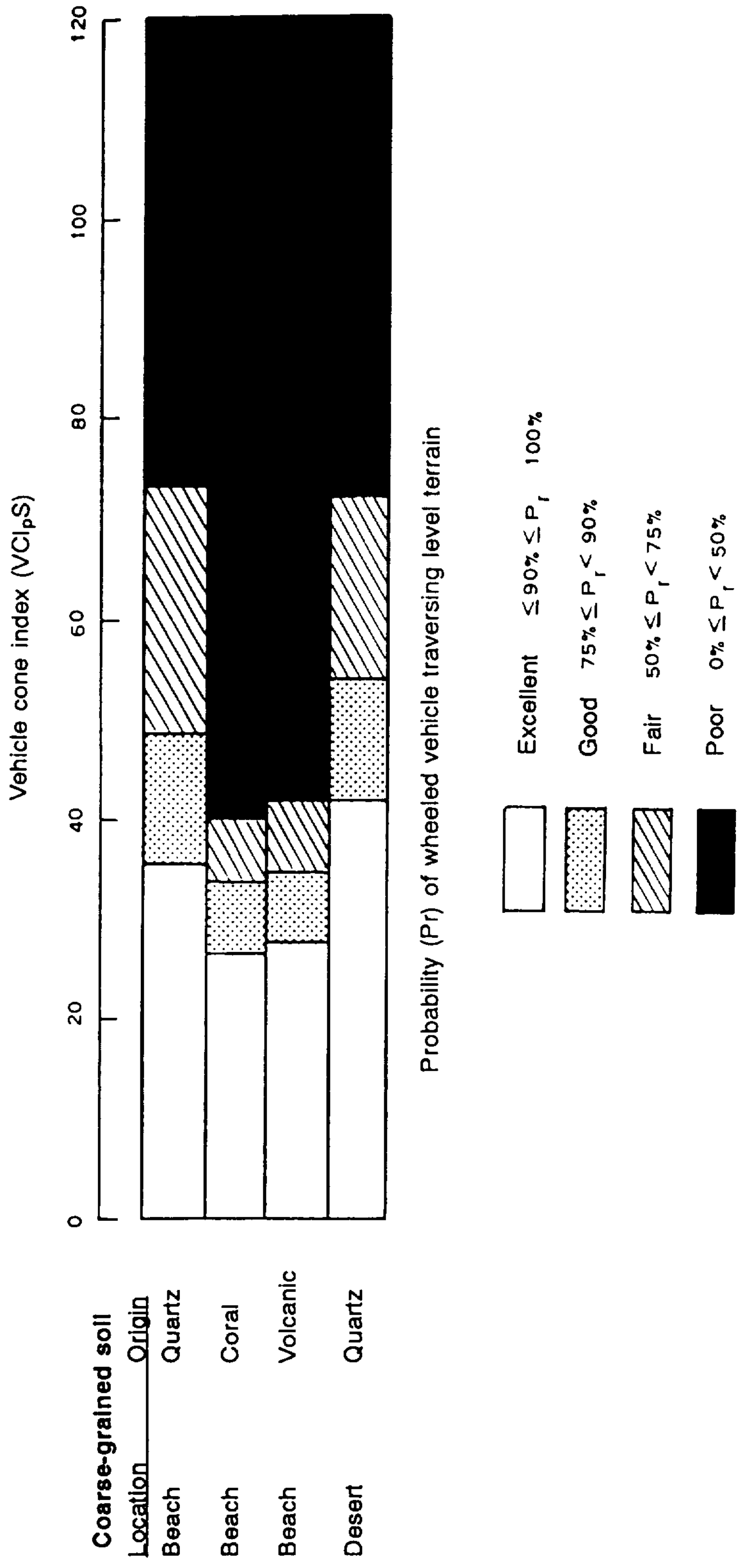


Figure 7-16. Trafficability classification of dry-to-moist, coarse-grained soils

MAINTENANCE, REPAIR, AND REHABILITATION OF ROADS, AIRFIELDS, AND HELIPORTS

CHAPTER 8

Maintenance is the routine prevention and correction of normal damage and deterioration (from use and the elements) to keep road and airfield surfaces and facilities in usable condition. Repair is that work necessary (other than maintenance) to correct damage caused by abnormal use, accidents, hostile forces, and severe weather. Repair includes the resurfacing of a road or runway when maintenance can no longer accomplish its purpose. Based on this manual, rehabilitation is the restoration of captured airfields and heliports to usable condition. Rehabilitation resembles war-damage repair except that it is accomplished before occupancy.

MAINTENANCE AND REPAIR CONSIDERATIONS

The purpose of all maintenance and repair activities is to keep roads, airfields, or other installation surfaces in as usable and as safe a condition as the situation permits. Prompt and adequate maintenance is important. Once surface deterioration or destruction has started, it can proceed very rapidly. Postponing minor maintenance jobs can result in the development of major repair jobs involving the subgrade, base course, and surface.

Use the following guidelines when performing maintenance and repair work:

- Ensure that maintenance and repair activities interfere as little as possible with the normal flow of traffic. Whenever feasible, plan and perform maintenance and repair activities to permit at least partial use of the facility. When it is necessary to close the facility to all traffic, select alternative facilities or perform repair work at night or during periods of reduced activity. Reopen the facility as soon as practicable.

- Remedy the cause before repairing the problem. For example, surface repairs made on a defective subgrade are wasted. All maintenance and repair jobs should include an investigation to find the cause of the damage or deterioration. To ignore the cause is to invite the prompt reappearance of damage. Ignore the cause only when making temporary repairs to meet immediate, minimum needs under combat or other urgent conditions.

- Maintain and repair existing surfaces as closely as possible to the original construction in strength, appearance, and texture. Spot strengthening may create differences in wear and traffic impact that can harm adjoining surfaces. Also, uniformity simplifies maintenance and repair operations.

- Prioritize the needed repairs based on the tactical requirements, the traffic volume, and the hazards that result from complete failure of the facility. For example, roads used for tactical-operations support take priority over less essential facilities. One pothole in a heavily used road that is in otherwise excellent condition takes priority over repairs to less heavily used roads in poor condition.

MAINTENANCE AND REPAIR OPERATIONS

Maintenance and repair operations include many tasks besides improving the pavement condition. To ensure a comprehensive maintenance and repair operation, incorporate the following tasks:

- Routine inspections.
- Material stockpiling.
- Maintenance and repair of all related drainage systems.
- Maintenance and repair of the actual pavement, including dust and mud control and snow and ice removal.
- Miscellaneous tasks, including the maintenance and repair of necessary buildings, structures, and utilities, and the operation of necessary utilities.

MAINTENANCE INSPECTIONS

The purpose of maintenance inspections is to detect early evidence of defects before actual failure occurs. Frequent inspections and effective follow-up procedures prevent minor defects from becoming major repair jobs. Inspect surface and drainage systems carefully during rainy seasons and spring thaws and after heavy storms.

Surface Inspection

Surface defects can usually be attributed to excessive loads, inferior surfacing material, poor subgrade or base conditions, inadequate drainage, or a combination of these conditions. Surface inspections should include a complete inventory of the current pavement defects. Careful investigation of the causes of the defects will allow for timely maintenance to prevent the pavement defects from requiring repair.

Drainage Inspection

Ensure that all drainage channels and structures arc unobstructed. Check culverts and drainage lines for structural damage. Inspect check dams for debris and excessive erosion. Investigate water ponding on or adjacent to surfaced areas. Inspect the system drainage during or after every storm. Also, thoroughly inspect the system in late fall to prepare for winter and in early spring to ensure minimum spring breakup difficulties. Inspect subsurface drains at least twice a year.

MATERIALS FOR MAINTENANCE

Generally, materials required in the maintenance and repair of roads and airfields arc the same as those used in new construction. Open pits and prepare stockpiles of sand and gravel; base material; and premixed, cold patching materials at convenient places and in sufficient quantities for emergencey maintenance and repair. Arrange stockpiles for quick loading and transporting to the road or runway. Build onc of the several types of trap-and-chute combinations described in Chapter 5 of TM 5-332 for sand, gravel, and base materials, Premixed. cold patching material may be prepared as explained in Chapter 9 of TM 5-337. Maintain small quantities of aggregate in dry storage for concrete patching.

DRAINAGE MAINTENANCE

Defective or inadequate drainage causes most pavement failures and deterioration. Pending or delayed runoff of surface water allows seepage into the pavement structure unless the surface is tightly sealed.

Surface Drainage

Mark areas where pending occurs on sur - faced areas. Correct such problems by filling or raising depressions and by providing outlets for water blocked by high shoulders. Control penetration of storm water through pavement by scaling joints and cracks. Keep unpaved roads and airfields crowned to prevent water from remaining on the road or airfield where it will saturate and weather the surface. Maintain crowns and superelevations with graders or drags.

Shoulders

Keep shoulders smooth and graded so water will drain from the surfaced area toward the ditch. Replace eroded shoulder material on paved surfaces with new material. Material cleaned out of ditches can often be used 10 rebuild shoulders. Shoulders should be kept bladed flush to, or slightly below, the edges of the pavement and should slope away from the pavement to prevent water seepage into the subgrade.

Drainage Ditches

Keep drainage ditches clear of weeds. brush. sediment. and other debris that obstruct water flow. Maintain ditches as to line and grade. Correct sags and minor washouts as they occur. Side ditches can usually be maintained with graders.

When cleaning and shaping, avoid unnecessary blading or cutting that destroys natural ground cover. Where possible, develop dense sod to stabilize open ditches. Where vegetation is not effective because of soil or moisture conditions, erosion may be corrected by lining the ditch with riprap, asphalt-coated membrane, or concrete.

Inspect check dams in side ditches and clean them regularly. The weir notch of a check clam must be kept clean or water will cut into the surfaced area at the edge of the dam (Figure 8-1). The aprons of check dams must also be maintained, and paving material must be replaced when washed out. Dikes or berms may be required along the tops of high-fill slopes to prevent gullies and washes.

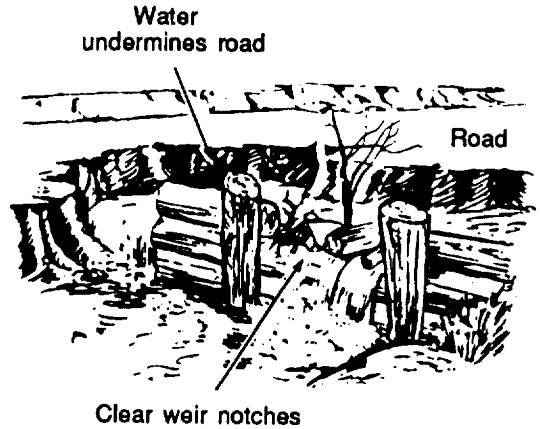


Figure 8-1. Check dam

Cut-slope interceptor ditches must be considered for all side-hill and through cuts to prevent gully washing and erosion from the top of the cuts. If benching or terracing has been used in the design of the cut, ensure that the top of each bench is sloped back into the cut to provide for proper drainage. Also ensure that each bench top is wide enough to maintain that drainage with earthmoving equipment. A good rule of thumb is to make benches at least as wide as a dozer blade. Figure 8-2, page 8-4, illustrates proper terraced side-hill-cut drainage.

For proper design considerations of cut slopes, refer to Chapter 10 of FM 5-410.

Culverts

Keep culverts clear of debris and sediment (Figure 8-3, page 8-4). This prevents water from cutting around or undermining the culverts. Inspect culverts frequently to determine whether they arc func-tioning properly. Cleaning by hand is usually necessary after heavy rains.

NONPAVED SURFACES

Basic maintenance of nonpaved surfaces includes shaping the cross section to maintain adequate drainage anti a smooth, compacted surface.

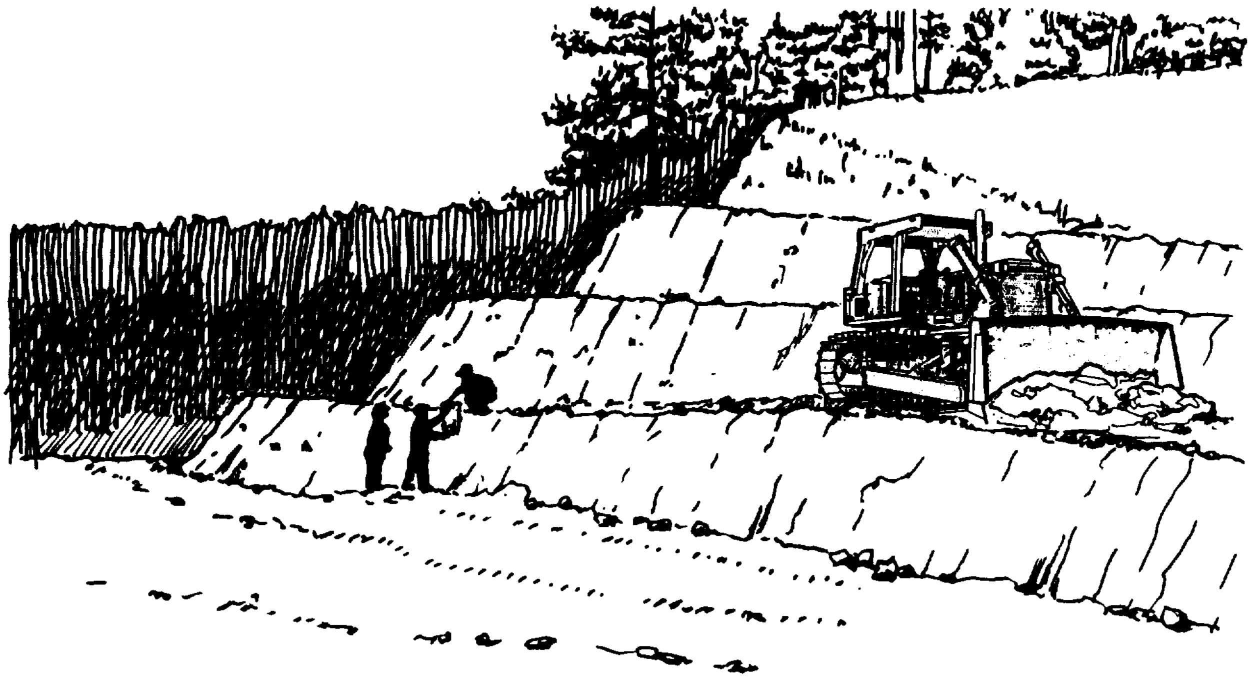


Figure 8-2. Side-hill terracing

Equipment

Most surface maintenance consists of light scraping with grader blades and drags. Multiple-blade drags of iron or iron-shod, heavy timber may be used for normal scraping. Drags are frequently used to float mud and water off a road and to prevent corrugation or washboarding of the surface. One type of drag is shown in Figure 8-4.

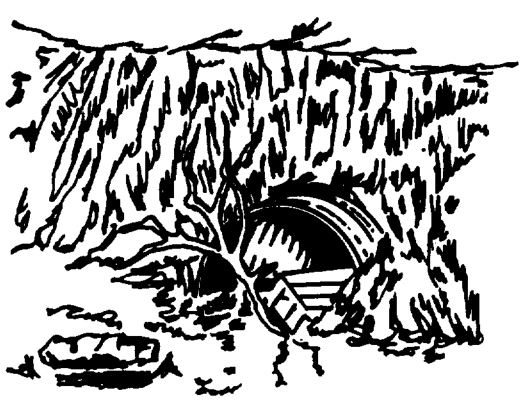


Figure 8-3. Culvert entrance

Materials

Materials removed from ditches, other than silt, some clays, and all organic soils (OL, OH, Pt), may be used on shoulders and traveled ways. Dispose of silt deposits when removed; they are not suitable for construction. After heavy storms and spring thaws, additional material may need to be hauled in.

Procedures

Keep traffic areas and shoulders free of potholes, ruts, and irregularities. Light blading will prevent corrugation or washboarding. Work from the ditches to the center of the road to ensure good drainage and proper road crowning. Since loosened, dry material cannot be compacted, blading or dragging should be done during or soon after rains. For prolonged dry spells or when surface material will not compact, add water or moist subsurface material

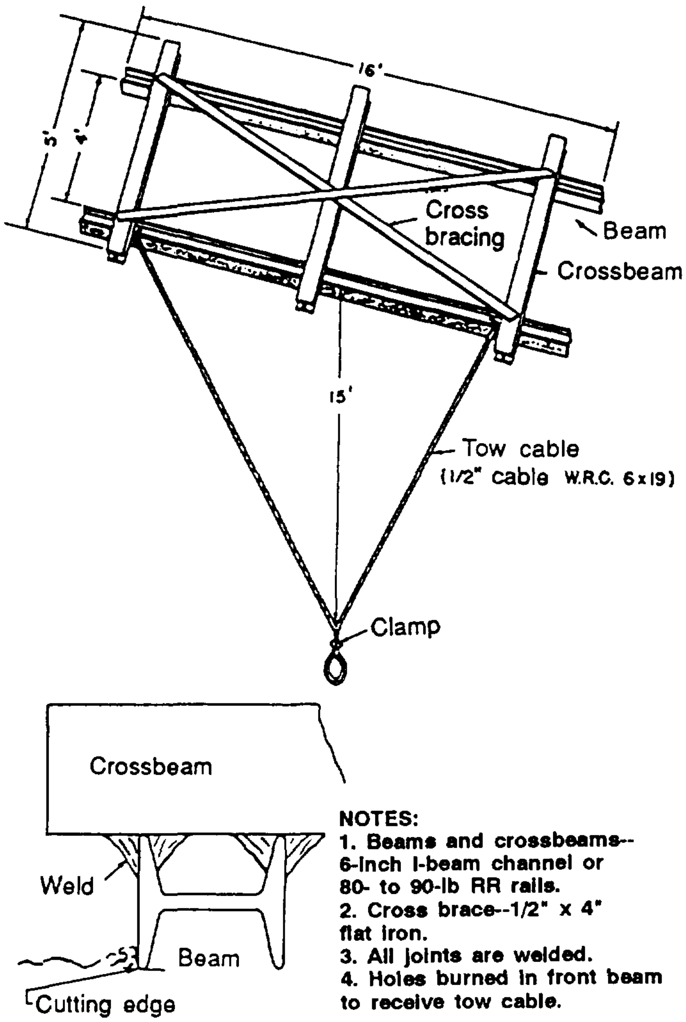


Figure 8-4. Improvised road drag

with a rototiller, plow, or scarifier before compaction.

Compaction of the graded surface material will reduce maintenance and repair for non-paved surfaces. The type of roller used will vary with the material compacted. The correct moisture content will result in the most economical compaction.

Soft spots, indicated by rutting or shoving of the surface, are generally caused by excess moisture, poor subsurface drainage, or unstable material. Determine the source of excess moisture and correct the drainage. On traffic areas, cobbles may be used to stabilize small areas of failure. Where surface failures are caused by pockets of mulch or peat, it may be necessary to remove the objectionable material and replace it with a more stable material. When adequate repairs cannot be made, soft spots can be temporarily reinforced by adding crushed rock or clean gravel.

Keep non paved surfaces crowned to prevent water from remaining on the surface and saturating the soil. Maintain the crown and superelevation with drags or graders.

Dust control may be a problem under some conditions. Spraying with water or a bituminous stabilizer is the most commonly used method of controlling dust. Dust control is discussed further in Chapter 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2.

Examples of road repairs are shown in Figure 8-5, page 8-6.

OILED SURFACES

The routine maintenance of oiled surfaces consists of shaping and patching. Shaping is done with graders or drags. Patching may be done with a mixture of the soil and oil. Oiling is necessary each year because oiled surfaces frequently break up in the spring and become very rough. Thorough scarifying. blading (or dragging), and reshaping is necessary before oiling each year.

GRAVEL SURFACES

Maintenance procedures for gravel surfaces are much the same as for nonpaved surfaces. Continual shaping is needed to maintain a smooth surface and a uniform crown, and the drainage system must be kept functioning.

Surface Maintenance

Heavily-traveled, graveled surfaces require constant attention by maintenance patrols. Intensive maintenance is required when the surface is first open to travel. Bumps compacted at this time remain in the surface and can be corrected only by scarifying or adding more material. Blade or drag the surface soon after rain until all ruts and holes are filled. Do not work on a dry

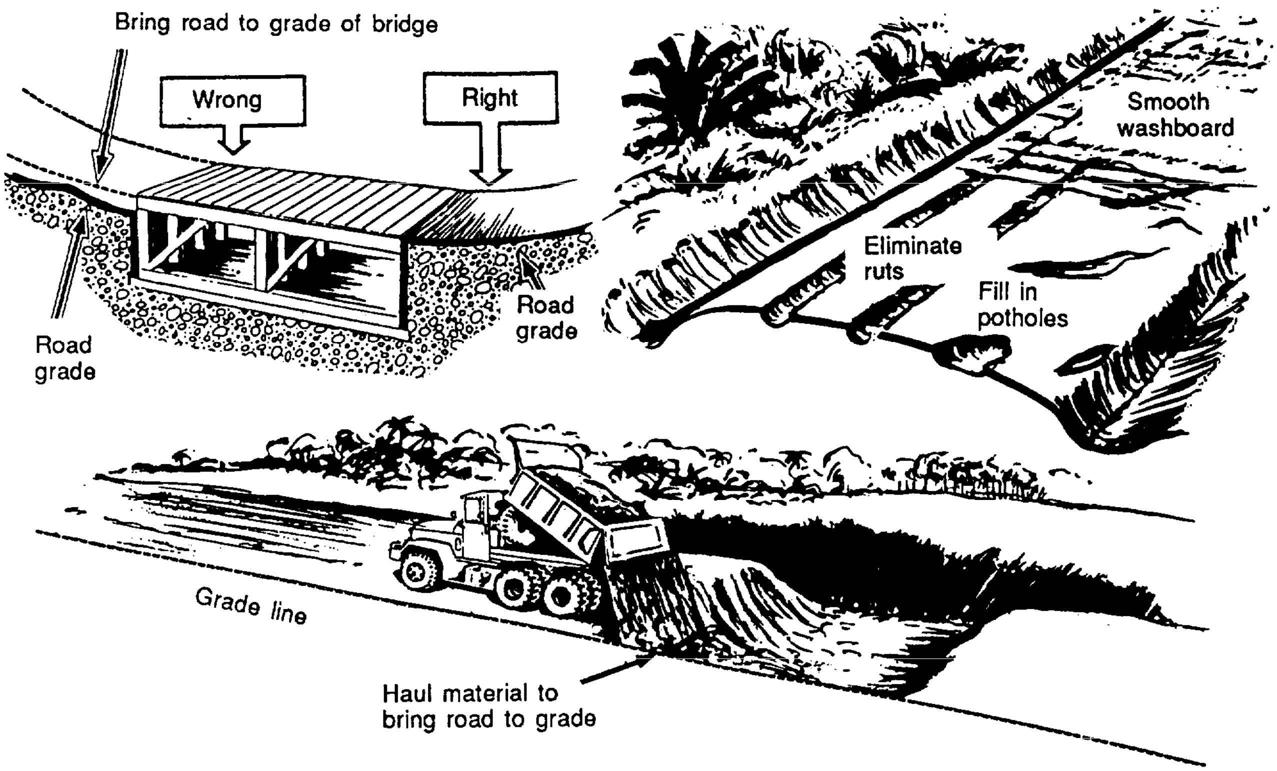


Figure 8-5. Road maintenance problems and proper corrective action

surface. Maintain a crown of at least 1/2 inch per foot. Multiple blade drags or sled drags can be used for routine maintenance, but graders are necessary for heavy reshaping work.

Keep a slight excess of gravel available at the edges of the roadway and blade it uniformly over the surface in wet weather. Stockpile additional materials in advance of fall and winter and prolonged wet periods. Material added or spread on the surface during warm, dry weather is of little value.

Repair of Potholes

Most potholes are caused by material displaced by traffic. Initially they are shallow and are readily filled by blading when the surface is moist. Deep holes require filling with additional material. New material should be moistened and compacted.

Treatment of Corrugations

All gravel surfaces tend to develop transverse or nearly transverse waves, called corrugations, which may progress into ruts as deep as 4 inches and from 1 1/2 to 3 feet apart. After this stage, they become major grooves needing extensive repair.

Some blade equipment inherently chatters and starts slight irregularities, especially when the operator attempts to move a heavy cut. Corrugations often appear to start from small holes or depressions made when the road is wet or from an obstruction such as a stick or rock. Other contributing factors are a soft subgrade, poor grading of the gravel, poor binder, and an insufficient amount of binder.

Corrugations can be prevented to a considerable extent by frequent maintenance and by the careful use of maintenance equipment. Once corrugations form, they can be removed only by thoroughly scarifying, reshaping, and compacting. Regrading, as this is called, should begin with a thorough cleaning and reshaping of the shoulders and ditches, continuing across the entire roadway.

Use of Calcium Chloride

Calcium chloride may be applied to a gravel surface to control or eliminate dust, prevent the loss of material under the whipping action of traffic, and aid in maintaining a dense surface. The usual method is to apply 1 pound of calcium chloride per square yard in the late spring and 1/2 pound per square yard twice during the summer season. However, the amount of rain, the volume of traffic, and the character of the gravel affect the quantities required. Calcium chloride is corrosive to metal surfaces and may require more maintenance for aircraft and vehicles.

The best time to apply calcium chloride is following a rain and after necessary blading or dragging is completed. If the application cannot be deferred until rain occurs, water the surface before applying the calcium chloride. For best results, apply calcium chloride before the traffic area becomes dry and dusty.

PROCESSED MATERIAL SURFACES

Traffic areas composed of processed materials (crushed and screened rock, gravel, or slag) are maintained by methods similar to those used on gravel surfaces. When coarse, processed materials are used, surface failures are usually in the form of sharp-edged holes caused by poor drainage. Repairs require cleaning the holes down to the solid subgrade and ensuring that no silt, mud, or water remains. Subgrade repairs are then made with a well-graded soil aggregate. Surface repairs should consist of a well-tamped or rolled-in-place, coarse-graded aggregate of the same gradation as the original surrounding surface.

BITUMINOUS SURFACES

The maintenance and repair of bituminous surfaces are discussed in Chapter 9 of TM 5-337. Some considerations applicable to bituminous-surfaced traffic areas follow.

Inspection

Maintenance patrols should frequently inspect bituminous pavements for early detection of failures. Small defects quickly develop into large ones, resulting in pavement failure unless promptly corrected. Small crews using hand tools can quickly make minor repairs with a minimum interruption of traffic. Large, bituminous repairs require more time, personnel, and equipment. Such repairs also interfere with traffic. In extreme cases, detours may be required to avoid complete traffic stoppage.

Patches

All patches should be trimmed square or oblong with straight, vertical sides running parallel and perpendicular to the centerline of the traffic area, as explained in Chapter 9 of TM 5-337.

Temporary Repairs

Any stable material may be used for temporary repairs in combat areas or where suitable material is not available and the traffic area must be patched to keep traffic moving, Good-quality soil and masonry, such as concrete rubble, are suitable for this purpose. All such patches must be thoroughly compacted and constantly maintained with replacement material. More permanent patching should be accomplished as soon as possible.

Maintenance of Shoulders

Blade shoulders so water drains from the surface and all ruts and washouts are filled. Grade shoulder material flush against or slightly below pavement edges to restrict water seepage to the subgrade and to prevent breaking of the pavement edge caused by traffic driving off the pavement onto the shoulder. Replace material displaced from shoulders with material hauled in, as required.

STABILIZED SOIL SURFACES

Maintenance of mechanically-stabilized soil surfaces and sand-clay surfaces is essentially the same as that for nonpaved and gravel surfaces. Procedures described for gravel surfaces arc applicable to surfaces that contain considerable coarse aggregate. Procedures described for nonpaved surfaces are applicable to surfaces that contain little or no coarse aggregate. Bituminous surfaces, soils, and soil-cement may require additional maintenance as described in the paragraphs that follow.

Potholes

Clean potholes and trim them rectangularly with straight, vertical sides running parallel and perpendicular to the centerline. This provides a shoulder against the movement of the patch. Fill the potholes with a stabilized mix of the same character as the adjacent sound area. This material should be thoroughly compacted in place, one thin layer at a time.

Ravels

Edge raveling is caused by water softening the foundation material. Before proceeding with the patching operations, reconstruct the shoulder or lower the subdrainage so this condition will not recur. Then build up the foundation. The patching mixture should conform 10 the surrounding area, as for pothole repair.

CORAL SURFACES

The maintenance of well-built coral traffic areas is relatively simple. Use fresh, raw coral of the proper moisture content for the repair material. Maintenance is best done during or after a rain while the coral is wet. Fill low spots, ruts, and potholes by shoveling or dumping coral directly from a truck onto the low spots. Such patches, if rolled while wet, will bond onto the original material almost without a mark. Salt water is usually available where coral is available, and salt water makes a better bond than fresh water. Occasional blading and rolling are necessary to maintain a proper crown and a smooth surface. In dry seasons, sprinkling is necessary to maintain a proper crown, a smooth surface, and high stability and to minimize dust. The traffic areas hold up well in wet seasons. An asphalt treatment may be justified in prolonged dry seasons, if dust and raveling become serious. Careful attention to shoulders and to the drainage system is essential.

RIGID PAVEMENTS

Maintenance of rigid pavements is covered in Chapter 15 of TM 5-337. Frequently inspect concrete pavement to detect early signs of failure, and make prompt repairs to prevent minor defects from spreading.

CRATER REPAIR

Bombs, shells, land mines, and cratering charges can produce extensive craters in traffic areas. Surface damage does not present any unusual repair problem, but explosions may displace or destabilize large areas of the subgrade. Drainage may also be disrupted, allowing water penetrating the broken surface to accumulate and further soften the subgrade. Satisfactory repairs require the restoration of subgrade stability to support traffic and prevent undue surface settling after repairs have been completed.

Use the following procedures for crater repairs:

1. Remove, from around the edge of the crater, all surfacing that is damaged or not firmly bonded to the base course.

2. Trim the surface and base course to a sound, vertical edge.

3. Remove water, mud, and debris from the crater.

4. Fill the crater with successive 6- to 8-inch layers of suitable material to the original level of the subgrade. Compaction is essential; each layer must be thoroughly

tamped with hand or pneumatic tamping tools. After the material has reached a suitable level, compaction equipment can be pulled through or driven across the crater.

5. Repair the base course and wearing surface.

Gravel, rock, masonry debris, sandy soil, or other suitable, stable materials can be used to fill craters, as shown in Figure 8-6. Material blown from craters can be used for much of this fill. In an emergency. material from the shoulders of roads or airfields may be borrowed and replaced later. When the situation permits and where enemy action may be anticipated, stockpiles or material pits should be prepared at convenient sites. Alternate layers of sandbags and tamped earth allow good subgrade compaction where other equipment or materials are not available.

For a detailed discussion of specific crater repair techniques used in air-base damage repair, refer to Training Circular (TC) 5-340.

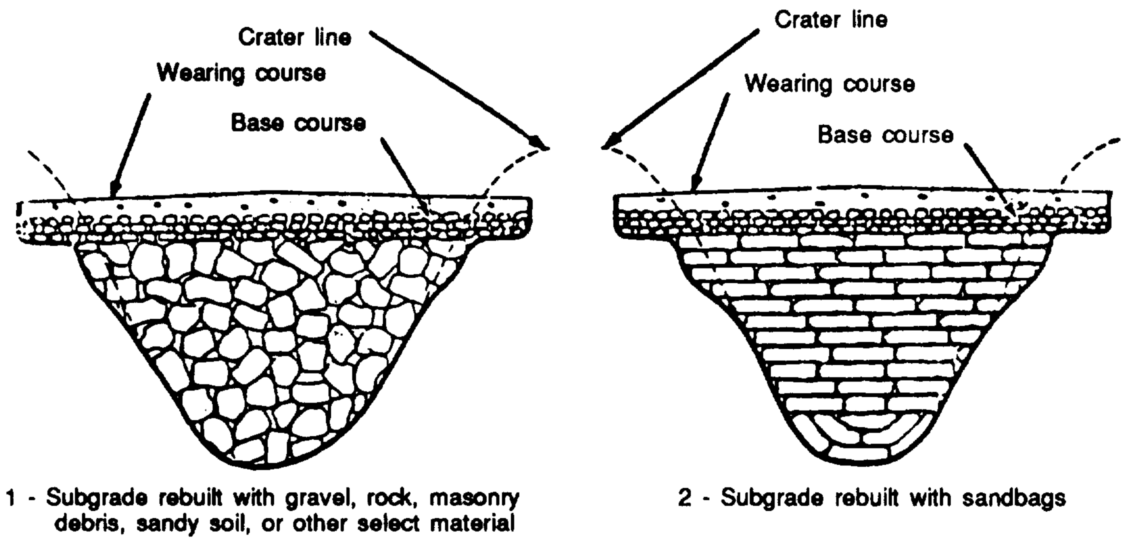


Figure 8-6. Crater backfill materials

ROAD MAINTENANCE

The importance of preventive maintenance and the necessity for prompt maintenance of all types cannot be overemphasized. Neglect and delay permit the traffic and weather to turn minor defects in 10 major problems. Progressive failure of roads is a serious matter. The more serious a failure is, the more quickly it deteriorates.

In forward areas, extensive repairs are often necessary before roads can be used. Expedient work is usually done by combat engineer units. Under the pressure of combat conditions, temporary repairs are made hurriedly using the materials most readily available. Such repairs are intended only to meet immediate minimum needs. As the advance units move forward, other engineer units lake over the work of additional repair and maintenance. Epedient repairs previously made arc supplemen ted or replaced by more permanent work. Surfaces are brought to a standard that will withstand the required use and maintenance becomes routine.

MAINTENANCE PATROLS

Adequate maintenance requires a workable maintenance organization. Usually engineer units establish a patrol system to handle the roadnet for which the unit is responsible. It is desirable to use squads as patrols and thus retain unit integrity, with each squad commanded by its squad leader and using its regular table(s) of organization and equipment (TOE). The platoon furnishes reinforcements (personnel or equipment) as needed.

Assign each patrol to a specific area. Organize as many patrols as necessary to adequately cover the total area of responsibility. It is sometimes necessary in forward or heavy-traffic areas to provide enough patrols to put the maintenance function on a 24-hour-a-day basis.

Personnel and Equipment

Two plans are presented for the organization of maintenance patrols. Consider the merits of each plan with respect to the maintenance problems of the situation and the personnel and equipment available for the patrols.

One patrol consists of a normal squad equipped with a dump truck, a grader, and hand tools. This patrol can handle all the maintenance and minor repairs normally en countered on a 5- to 15-mile stretch of road. The number of people in a squad “can be decreased and more miles can be assigned to a patrol expected to cover a stretch of permanent pavement in good condition and not heavily traveled. If a patrol is to cover a poor dirt road accommodating heavy traffic, more personnel will be furnished by the platoon and fewer miles will be assigned.

Another plan calls for the assignment of a patrol of one to three people, a grader, and hand tools. This crew can handle the routine maintenance on a 12-mile stretch of average earth, gravel, or water-bound macadam road. However, the crew must be supplemented with a truck and repair crew whenever material must be hauled or surface patching accomplished.

Winter weather; severe storms; heavy, destructive enemy action; and other conditions demand that the patrols be reinforced with additional personnel and equipment or that the assigned areas be reduced and the number of patrols increased. Special conditions often call for special equipment.

Duties

The duties of maintenance patrols are as follows:

- Clean out drainage facilities.
- Mow grass and weeds.
- Repair minor washouts and potholes.
- Maintain the road surface; for example, eliminate ruts, potholes, and washboards.
- Maintain road shoulders and ditches.
- Make frequent, thorough inspections of road conditions, and report to higher headquarters any need for repair work more significant than the patrol is equipped or manned to handle.

Inspection and Supervision

An officer or senior noncommissioned officer (NCO) is assigned several patrols to supervise, assist, and inspect. Normal unit organization should be retained as much as possible. A platoon leader should be responsible for the patrols composed of personnel from his command.

REPAIR CREWS

The engineer sending out maintenance patrols should keep sufficient equipment and personnel available to send out repair crews to handle those situations reported by the patrols. The repair crew composition will be dictated by the needs of the particular job in terms of equipment and personnel. Frequently, the regular maintenance patrol can work with the repair crew, but other maintenance and inspection of the patrol's area must not be neglected.

MAINTENANCE WITH TRAFFIC

Give full consideration to the importance of keeping traffic flowing with a minimum of interference or delay. Maintenance of shoulders and ditches can normally be performed without interference from, or hindrance to, traffic. Repairs to other drainage structures may delay traffic or slow other repair work. Permanent repairs are often postponed so that temporary and emergency repairs can be made in order to maintain traffic flow.

Traffic Control

Repairs should be made on one-half the surface at a time when surface repairs will deny traffic the use of sections of the pavement. Block off short sections and post guides to regulate traffic and avoid delay.

When single-lane traffic must be used, control traffic by the baton method. With this method, a flagman is placed at each end of the single-lane traffic section. The flagman at one end of the section has a baton or some other distinctive marker. Working under a prepared plan, all vehicles traveling in one direction are stopped, while those traveling in the opposite direction are permitted to go through. After a suitable time interval, the flagman on the open end of the section gives the baton to the driver of the last vehicle permitted to go through. Upon arrival at the other end of the section, the driver of the last vehicle gives the baton to the flagman. Vehicles are then permitted to travel in the opposite direction until all waiting vehicles have passed through and the driver of the last vehicle carries the baton to the other end of the section. This process continues as long as necessary.

Sometimes two-way traffic can be maintained through blocked-off sections by diverting one stream of traffic to the shoulder of the road. Grading and stabilization of the shoulder with gravel or bituminous material may be justified in such instances. Repairs to be performed during traffic flow should be carefully planned; proper procedures should be selected; and all labor, material, and equipment should be on hand to complete the work as rapidly as possible.

Bypasses and Detours

Bridge or pavement failures or the destruction of part of a roadway by floods or combat action may make part or all of the roadway impassable to traffic. In such cases, a bypass or detour around the damaged or obstructed area is necessary. The construction of a short bypass around an obstruction may be preferable to a longer detour on existing roads. A detour may also be used while a bypass is being constructed. Base the decision upon traffic interference, the work involved, and the time available.

Conduct a reconnaissance to determine the best possible route when establishing a detour. The road should be as short as possible and must be in condition, or be put in condition, to handle traffic for the period when it will be used. Use existing roads when possible. Construct short sections of connecting roads, if necessary.

Check and repair bridges or reinforce them with timber or planking. Clean and repair culverts. If the need for a detour is anticipated, complete this work beforehand. Detour roads are usually subjected to heavier loads and more traffic than their design specifications. Because increased maintenance is usually required to keep detours passable, stockpile surfacing material along the route, carefully plan maintenance operations, and keep labor and material constantly available.

Place signs at detour entrances, road intersections, and turns to direct traffic. Post warning signs at dangerous points. Place other signs or markings, as required, to ensure minimum traffic delay. Install barricades at each end of the road section under repair. Refer to Chapter 8 of FM 5-36 for the types and posting of road signs.

Safety of Maintenance Personnel

Give special attention to the safety of maintenance personnel working where traffic moves past or around them. Use restrictive speed and warning signs, barricades, and flagmen to control traffic and lessen the danger to maintenance personnel. Instruct crew members to avoid stepping into the traveled way and to be continually alert to passing traffic. Conspicuously mark maintenance vehicles operating in or on the edge of the roadway with red flags, flashing red lights, or similar devices.

WINTER MAINTENANCE

Winter weather may present special problems in TO maintenance. Regions of heavy snowfall require special equipment and material to keep pavement and traffic areas in usable condition. Low temperatures cause icing of pavements and frost on subgrade structures. Alternate freezing and thawing may damage surfaces and tend to block drainage systems with ice. Spring thaws may result in both surface and subgrade failure. Winter maintenance consists chiefly of removing snow and ice, sanding icy surfaces, erecting and maintaining snow fences, and keeping drainage systems free from obstruction.

Preparation for Winter

Organize snow-removal crews and place equipment in readiness. Stockpile abrasives and chemicals in locations where they will be required. Perform late fall maintenance before the winter freeze. Continue with routine maintenance of ditches and shoulders as far into the winter as possible, so that the drainage system will be in the best possible condition for the spring runoff.

Keep earth and gravel surfaces smooth and shaped to prevent moisture from entering the subgrade. Smoothing and shaping also prevent snowplow blades from being obstructed by rough, frozen shoulder and surface material. In areas of heavy snowfall, outline bridges, culverts, and narrow places in the road with poles that extend above the snow, and mark these locations for maintenance crews.

Snow Fences

Conduct reconnaissance before winter to determine where snow fences will be needed to control drifting snow. Because it is fine and compacts into a dense mass, drifted snow obstructs traffic more than an equal depth of freshly fallen snow. Drifts form when wind-borne snow is picked up in open spaces, loses velocity, and is deposited in sheltered places. Danger spots, therefore, are roads at ground level or in cuts adjacent to large, open areas. Drifts also form in the lee (down-wind side) of buildings, signboards, and similar wind barriers. Similarly, high snowbanks left close to the road by snowplows furnish both the conditions and the material for extensive drifting. Snow fences are not normally required near high fills, in wooded or brushy areas, or where vegetation prevents snow from drifting on the road.

Placement. Place snow fences on the windward side of roads according to prevailing winds (Figure 8-7). The height of the fence determines the distance it is to be placed from the traveled way. Generally, the proper distance is 20 times the height of the fence. This distance is increased where winds are of high velocity. According to the above ratio, a 4-foot fence erected 1 foot above the ground should be placed at least 80 feet beyond the point where drifting is to be prevented. In extreme cases, a distance as great as 300 feet may be necessary.

Fences should be as long as possible without any holes or openings. Openings provide for dispersion of snow on the back side of the fencing. The effect of a snow fence in controlling drifts caused by a road cut is shown in Figure 8-8. Two or more parallel fences may be required, but one tall fence is generally better than two short ones. If fences are set too far away, they have little or no effect in reducing drifts. If fences are placed too close to the road, drifts to the leeward side of the fence fall on the road.

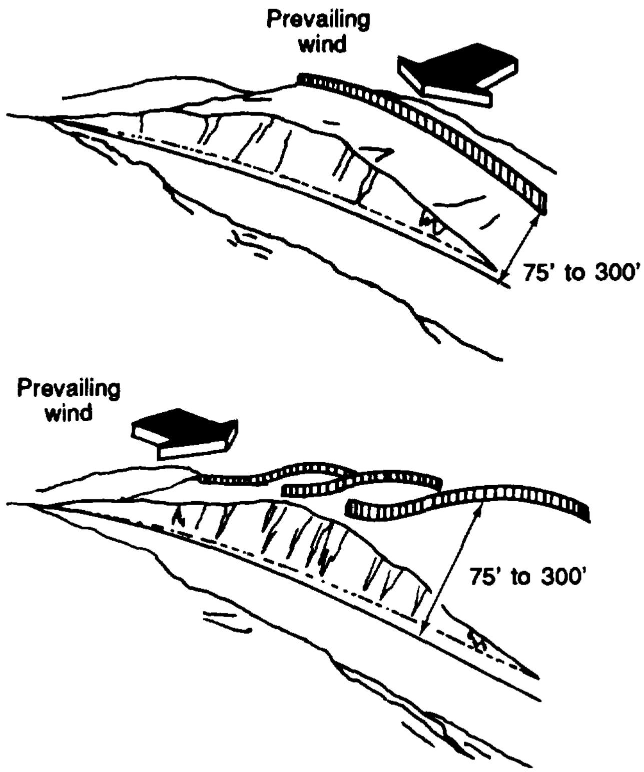


Figure 8-7. Location of snow fences

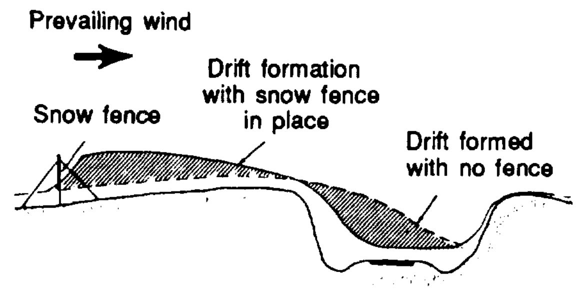


Figure 8-8. Snow fence control of road cut drifts

Types of Snow Fences. Commercial snow fences are commonly used. They consist of metal posts and wooden laths or metal pickets about 5 feet long, woven together with wire. Such fencing is portable, easily erected and dismantled, and may be rolled up and stored in the summer. Permanent snow fences include open-board fences on posts and evergreen or deciduous shrub hedges. Plastic snow fencing is lighter and more efficient than wooden fencing.

Other types of snow fencing include wood slats or pressed-steel slats mounted on collapsible A-frames, worm fences, and brush or branches suspended on wire. Local materials, such as corn stalks, brush, and coarse grass anchored in place by wire or wood, may be used. Figure 8-9, page 8-14, shows three types of snow fences.

Erection. Erect snow fences before the ground is frozen. Drive metal posts into the ground and mount wire fencing on the windward side. In regions of heavy snow, use long posts so that fencing may be raised on the posts as the season progresses. This will increase snow storage to the leeward side. Install fencing with the bottom about 6 inches above ground level to prevent the ends of the pickets from freezing fast and to prevent the fence from choking with snow. Frozen ends make it difficult to raise the fence and may cause the pickets to break when swayed by the wind. Brace the posts according to the anticipated wind velocities.

Maintenance. Inspect snow fences after heavy storms. Repair broken ties and braces, and straighten blown-down sections. Raise fences to exceed the height of accumulated snow on the leeward side. Lowering of fences may be required after midseason thaws or long periods of settling.

Removal and Storage. Remove snow fences in the spring and repair damaged sections. The fences are frequently stored on dunnage at the drift location for use the following winter.

Snow Removal

Prompt snow removal is essential to prevent traffic interference and ice formation on the road. Begin removal operations when the snow starts. The amount of equipment necessary depends on the intensity of the storm. If possible, store equipment at intervals along road sections or roadnets that are to receive early attention, and have operators ready to move promptly when a snowstorm arrives.

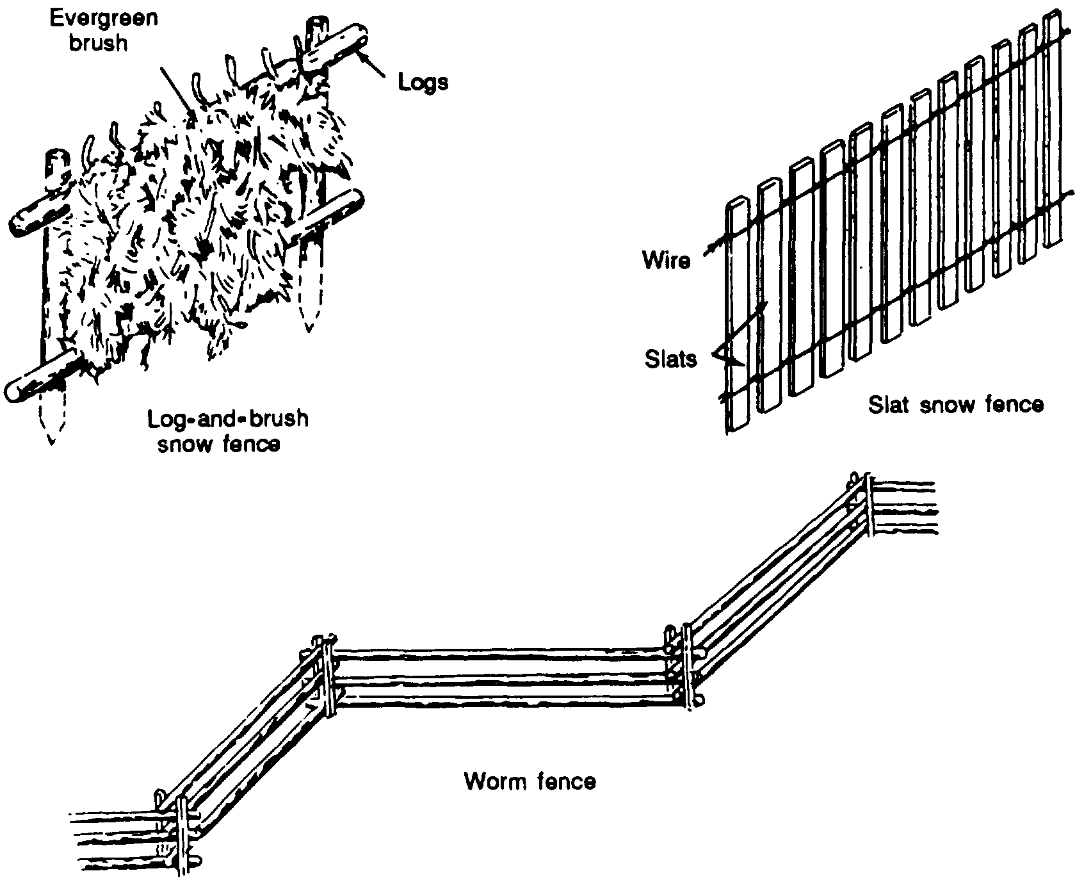


Figure 8-9. Types of snow fences

Provide for relief crews, since snow-removal equipment must frequently be operated on a 24-hour basis during periods of heavy storms. Use snowplows to patrol areas or sections of roads that are subject to drifting in windy weather. Standard snow-removal equipment consists of various attachments mounted on trucks or similar prime movers. These include one-way, reversible, and rotary-type straight blades; straight under-body blades; V-type plows; and rotary snowplows. These items are described in Chapter 11 of TM 5-624. Graders, dozers, and loaders are also useful in snow removal. Improvised equipment, such as drags, may be used in expedient situations.

Use comparatively light and fast snow-removal equipment for light snowfalls and at the beginning of severe storms. Use heavier equipment to widen traffic lanes and for heavy snowfalls. Continue snow-removal operations until the snow has been pushed back, leveled, or hauled to a disposal point. When snow removal is delayed or interrupted, the snow may become too deep for available equipment to handle, drifting may worsen, or wet snow may freeze.

Trucks with 2 1/2- to 5-ton capacities and equipped with one-way blades are best adapted for long stretches of roads. This equipment travels at speeds of 15 to 25 miles per hour (mph), removes snow from the road before it is compacted, and provides an open track for traffic.

Use 5- to 10-ton trucks equipped with either straight or V-shaped blades for heavier snowfalls and to widen traffic lanes. This equipment operates at about 15 mph. Use V-shaped plows to break through heavy drifts. Use either straight or V-shaped plows equipped with side wings to push the snow beyond the shoulder line, to prevent drifting, and to provide room for additional snow storage.

Trucks used for plowing generally carry abrasives as a ballast for better traction. Tractors are sometimes required for heavy snows and drifts. Graders are satisfactory for light snow. In areas of very heavy snowfall, rotary plows and blowers may be needed.

Surface Ice Control

The formation of road ice resulting from packed snow, slush, or melting snowbanks is prevented if snow removal is effectively performed and drainage is provided. When possible, push snowbanks away from pavements so that melted water will not run onto cleared surfaces.

Use plows and graders to remove slush from the road to prevent freezing and icing or to remove ice that has previously formed. Use care to avoid damaging the pavement. Dry snow compacts under traffic but can usually be loosened and bladed off without difficulty. Wet snow or slush, if it is allowed to freeze in place, sticks tightly to the pavement and cannot be easily removed without a period of warm weather or the use of salts.

Various conditions cause icing of road surfaces. Dangerous icing is more likely to occur during the late fall and early spring when frequent temperature changes occur. Midday thawing and night freezing is a common cause of icing. Rain; sleet; or light, wet snow falling on cold pavement can form ice films too thin to be removed by mechanical means and can make long sections of the road hazardous. Curves and grades are critical points under icing conditions, and high-crowned roads can become difficult to travel at any point when iced over.

Use of Salts. At the beginning of a storm, apply sodium chloride or calcium chloride to wet snow and sleet to keep it in a slushy condition and prevent it from sticking to the surface. Limit the use of salts on concrete pavements to one or two applications per year or the pavement will pit and scale. Salts do not damage bituminous surfaces. To prevent blockage of the drains by freezing. place bags of salts at drain inlets or catch basins so they do not obstruct flow.

Use of Abrasives. Treat icy pavements with abrasives and salts to reduce slipping and skidding. Sand, both treated and untreated; cinders; and crushed rock or slag screenings are commonly used. Other materials include pea gravel, coal stock, and coke screenings. The choice of materials is based on the availability and the length of the haul. Sharp, angular material embeds itself readily, and dark-colored material absorbs the most heat from the sun. Untreated materials are fairly effective on compacted snow but are easily blown or thrown from the traveled way.

For the most effective use, mix calcium chloride with abrasive material. The calcium chloride causes the abrasive to embed in the ice and improves surface traction. Treat abrasives with 40 to 75 pounds of calcium chloride per cubic yard for stockpiling, and add another 25 to 50 pounds per cubic yard of calcium chloride when applying it to a road. Sodium chloride may be used in place of calcium chloride but is not recommended for temperatures below 10° F.

Store abrasive materials where they are quickly available when needed. Establish stockpiles for hand application at critical locations such as steep grades and curves. Make wider distribution with trucks by either hand or mechanical spreading. Heating an abrasive material before placing it on the road will allow it to melt into the ice and prevent it from being forced out by traffic.

Mechanical Removal of Ice. Graders can sometimes remove ice that is not tightly bonded to the road surface. Extremely icy conditions can be reduced by using scarifiers or rotary tillers equipped with special teeth. Exercise extreme care not to damage the road surface.

Correction of Spring Breakup Problems

In regions subject to frost and snow, spring is a critical time in the maintenance of pavement and other traffic areas. Abnormal and repetitive traffic loads during the spring breakup period may cause subgrade pumping of concrete surfaces and breakthrough of bituminous surfaces. Melting ice and snow, spring rains, and frost leaving the ground all have a tendency to saturate permeable surfaces. Drainage obstructions may raise the water table and make subbases unstable.

Ditches. When the spring thaw begins, open ditches at critical points so that melted water will not flow onto the road. Outlets of cuts and road sections next to snowbanks require special attention. As the snow begins to melt, snowplows or grading equipment may be required to clear snow from the shoulders to avoid erosion. Handwork is usually required to clear the shoulders and open ditch outlets. Remove accumulations of ice in culverts and small drainage structures by hand or thaw them with truck-mounted steamers.

Frost Heaves. Frost heaves are indicated by the localized raising of road surfaces and pavements. Damage occurs as a result of the heaving of the subgrade soil due to the formation of ice lenses. This expanding subgrade causes an upheaval of the surface and subsequent reduction in overall strength. Frost heaves are most prevalent in silt and clay subgrades.

Frost Boils. Frost boils are indicated by the breaking up of a localized section of road surfaces and pavements when subjected to traffic. During thawing, the melted water produces a fluid subgrade condition with very limited or no supporting capacity. The traffic imposes a force on the pavement and thus to the excess water in the subgrade. This in turn exerts an equalizing pressure in all directions. This pressure is relieved through the point of least resistance (up through the pavement surface) and produces a small mound similar in appearance to an oversized boil.

Frost boils are often large and deep enough to make the road impassable until repaired. Repairs may be made by one or a combination of the following procedures:

- Bridge soft spots with timber, landing mats, sapling mats, corduroy, or other available material. Store materials in advance at or near load sections where frost boils may occur. Such repairs are temporary; therefore, remove the material when the road thaws and dries.

- Patch soft spots with crushed rock or gravel. Place large rocks in soft spots and cover them with smaller ones. It is better to remove soft, water-soaked material beforehand, although more time is required.

- For best results, provide adequate drainage along with the repair work to correct the cause of the problem. Remove soft material and dig an outlet ditch to one shoulder. A temporary bridge of planks or other material permits traffic to pass. The excavated soft spot and the ditch are backfilled with rock, gravel, or other suitable material. Drainage tile may be installed in the ditch before backfilling.

Preventive Maintenance

The best maintenance of any road is preventive maintenance. During the spring breakup, the best maintenance of a road subject to frost heaves and boils is to prohibit all traffic during the critical 2- or 3-day period. Methods to eliminate or minimize the damaging effects of frost action are discussed further in Chapter 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2.

FORDS AND BRIDGES

The approaches and bottoms of fords must be kept smooth and clear of large boulders and debris. Replace marking posts that have been knocked down or washed out. Refer to FM 5-446 for additional information.

Frequently inspect bridge abutments, trestles, piers, and trusses for damage and deterioration. Repair defects at the earliest opportunity. The maintenance crew normally obtains help to rebuild or repair the bridge.

AIRFIELD AND HELIPORT MAINTENANCE

Airfield and heliport maintenance is the responsibility of the primary user. For Air Force air bases, such maintenance is normally clone by an Air Force civil engineering squadron (CES) or similar unit. Army airfields and heliports arc normally maintained by Army engineer units. When the repair and rehabilitation requirements of Air Force bases exceed the immediate, emergency-damagce recovery capability of the air base, Army engineer units will be assigned to perform the work.

AIR BASE DAMAGE REPAIR

The immediate, emergency-damage recovery of air bases generally is considered to be the minimum work required to permit aircraft to land and take off.

The Air Force is primarily responsible for the emergency repair of the air base. This includes the emergency repair of the air base paved surfaces, which is called rapid runway repair (RRR). This is accomplished through the employment of Air Force base civil engineering troop assets; prime base engineer emergency forces (Prime BEEF), and rapid engineering deployable heavy operational repair squadrons, engineering (RED HORSE) units. The Army is responsible for semipermanent construction, the byond-emergency repair of the air base and, upon request, emergency repairs which exceed the Air Force's capability. Joint service regulation AR 4 15-30/ AFR 93-10 specifics these repair responsibilities for each service.

Army Responsibilities for Air Base Damage Repair (ADR)

The Army provides engineer support to the Air Force overseas. It ensures that units are equipped, manned, and trained to support Air Force needs. This support includes—

- Assisting in emergency repair of war-damaged air bases where requiremen ts exceed the Air Force's organic repair capability.
- Repairing and restoring damaged air bases with beyond-emergency repairs.
- Developing engineer designs, plans, and materials to meet Air Force needs as agreed upon by the Air Force. Where practicable, designs will be based on the Army Facilities Component System (AFCS).
- Supplying construction materials and equipment, except for that provided by the Air Force.
- Upon request, assisting within their capabilities in the removal of UXO declared safe by EOD personnel and limited damage assessment operations.
- Managing and supervising the repair and restoration of war damage per - formed by Army personnel. The Air Force base comma nder sets the priorities for air base repair.

Air Force Responsibilities for ADR

The Air Force provides military troop engineering support from its resources. The Air Force ensures that units arc equipped, manned, and trained adequately to support its needs. This support includes—

- Emergency repair of war damage to air bases.
- Organizing host-nation support (overseas).
- Force beddown of units and weapon systems, excluding Army base development responsibilities.
- Operation and maintenance of facilities and installations.
- Crash recue and fire suppression.
- Managing force beddown and the emergency repair of war damage.
- Supplying material and equipment to perform its engineering mission.

- Providing logistical support to the Army for all classes of supply except II, V, VII, and IX.
- Conducting damage assessment and removal of UXO.
- Providing nuclear, biological, chemical (NBC) collective shelters and establishing and operating personnel and equipment decontamination sites for the air base and the Army. There are shortages of these assets on air bases, and support to army units may be limited.

Air base support agreements may be established in some theaters between the Air Force and the host nation where ADR support capability exists. These host-nation support agreements may include equipment, materials, and manpower assets.

For a detailed description of personnel, equipment, and material requirements and critical path schedules for repair of runways cratered by high-explosive bombs, refer to AFR 93-2. For a further, detailed discussion of general ADR, refer to TC 5-340.

TURF SURFACES

Plant grass to provide a turf surface on shoulders and all graded areas. Turf aids in camouflage, reduces dust, and minimizes erosion. Turf surfaces are limited to areas where the climate and soil are favorable. Table 8-1 gives the characteristics of many native grasses to aid in selecting proper grass seed or sod.

MUD CONTROL

Mud on the runway creates slippery surfaces that impede takeoff and increase the difficulties and dangers of landing. Muddy taxiway and runway surfaces decrease tire life and increase the wear and maintenance of brakes. Flying mud particles may damage propellers, rotors, and jet engines. Removing mud from wheels, struts, and fuselage is an additional maintenance burden.

Mud on airfield and heliport surfaces is either deposited by vehicular traffic from adjacent muddy areas or caused by subgrade failure because of excess moisture and the pumping of mud to the surface under traffic.

Enforce mud discipline by limiting access to taxiways only to required service vehicles. Also, remove mud from the wheels and undercarriages of vehicles before they enter the taxiway. The most satisfactory solution is to provide surfaced service roads to all hardstands.

Repairs

Localized soft or muddy spots in an otherwise satisfactory surface are repaired by replacing the unsatisfactory subgrade material with a more suitable one. If the muddy areas are widespread, it may be necessary to stop all traffic until the surface dries. In extreme conditions, resurfacing may be necessary.

Mud Removal

In some instances, surfaces may be kept in operational condition by removing the surface mud. Remove mud by hand shoveling, blading, or dragging. Light mud or slush is sometimes removed by hand or with rotary brooms.

When a grader is used on a landing-mat surface, take precautions to prevent the blade from tearing the surfacing. A satisfactory method is to bolt a 4- by 12-inch hardwood moldboard over the cutting edge and extending 2 inches below it. This provides a scraping edge with sufficient spring to remove the mud from irregularities in the landing mat, yet soft enough to protect the mat. A piece of 1/2-inch rubber belting (or the cap from a worn tire) bolted between the blade and the moldboard and extending an inch below the cutting edge makes an effective squeegee for removing light mud and slush. Grader operations for removing mud and slush from the runway are similar to those employed for snow removal. Start in the

Table 8-1. Characteristics of grasses

Name of grass grouped by region of suitability	Resistance to traffic wear and mowing	Preferred soil texture	Drought resistance	Acid tolerance	Rate of establishment	Method of establishment	Best season for work	Use in mixture with–
TURF GRASSES								
COOL, HUMID REGION								
1. Kentucky bluegrass (Poapratensis)	Good	Loam, clayey loam	Good	Fair	Slow	Seed, sod	Fall or spring	Number 2
2. Creeping red fescue and chewing fescue (Festuca rubra and festuca rubra fallax)	Traffic, good Mowing, fair	Sandy to gravelly loam	Good	Good	Fast	Seed	Early fall	White clover and number 1
3. Smooth brome (Bromus inermis)	Good	Various	Good	------	Medium, fast	Seed	Spring	Mix with native grass
WARM, HUMID REGION								
4. Bermuda grass (Cynodon dactylon)	Excellent	Sandy to clayey loam	Excellent	------	Fast	Sprigs, sod, or seed	Spring or summer	Seed alone or with carpet grasses
5. Common carpet grass (Axonopus affinis)	Good	Moist clays, clayey loam	Fair	------	Medium, fast	Seed, sod	Spring or summer	Seed alone or with carpet grasses
6. St. Augustine grass (Stenotaphrum secundatum)	Fair	Moist, various	Fair	------	Fast	Sprigs	Spring or summer	
DRY REGION								
7. Buffalo grass (Buchloe dactyloides)	Excellent	Clayey loam to loam	Excellent	------	Medium, fast	Block, sod	Spring	Seed blue grama between buffalo-grass-sod blocks
ROUGH TURF AND BUNCH GRASSES								
COOL, HUMID REGION								
8. Common ryegrass (Lolium multiflorum and L. perenne)	Good	Various	Good	Good	Fast	Seed	Fall or spring	Number 9
9. Orchard grass (Dactylis glomerata)	Fair	Loam to clay	Good	Excellent	Fast	Seed	Fall or spring	Number 8
WARM, HUMID REGION								
10. Hairy crabgrass (Digitaria sanguinalis)	Excellent	Loam to clay	Excellent	------	Fast	Hayseeding	Summer	
11. Bluestem (broomsedge) (Andropogan)	Fair	Loam to clay	Excellent	Excellent	Medium	Hayseeding	Fall	
12. Korean lespedeza (Lespedeza stipulacea)	Excellent	Loam to clay	Excellent	------	Medium	Seed	Spring	
DRY REGION								
13. Little bluestem (Andropogon scoporius)	------	Moist, sandy to loamy	------	------	------	Seed, hayseeding	Spring	Other grasses suited to soil
14. Blue grama (Bouteloug gracilis)	------	Loam to clay	------	------	------	Seed, hayseeding	Spring	Seed between blocks of buffalo grass
15. Crested wheatgrass (Agropyron cristatum)	------	Clayey loam to loam	------	------	Slow	Seed	Spring	Other native grasses

center of the runway and proceed progressively to the edge, overlapping several feel on each pass.

SNOW REMOVAL AND ICE CONTROL

Snow removal methods, the order of operations, and the assignment of equipment arc established in advance of the winter season. Factors to be considered in planning the snow-handling program are climatic conditions the average snowfall, the aircraft to be accommodated, the equipment available, and the camouflage requirements. Aircraft may be equipped with either wheels or landing skis. Ski-equipped aircraft operate successfully on packed snow. Aircraft with landing wheels cannot operate in more than 3 inches of loose snow. This limitation applies to fresh snow on a clear runway, fresh snow on previously packed snow, or melting snow previously packed on a runway. For camoflage it is undesirable to remove all snow from a runwav when the surrounding terrain is blanketed with snow.

Controlling with Equipment

Equipment useful for handling snow includes rubber-tired tractors, scoop loaders, graders rotary brooms, and band brooms. Supplementary equipment may include single-wing one-way, and reversible snow plows: V type plows; rotary plows; blowers; rollers; and other snow-removal equipment.

Packing

In regions of heavy snowfalls with prolonged cold weather relatively free from sudden thaws. snow may be handled by packing. The runway, shoulders, and as much adjacent area as practical arc packed. Rolling begins as soon as 3 inches of snow have fallen and continues during the snowfall. Snow is packed by rollers drawn behind a tractor with snow treads.

Smoothing is done with a drag equipped with metal cutting edges on the front and rear or with a grader. Usually one tractor is used to pull both the drag and the rollers, with the drag ahead of the rollers.

Rollers can be made to any desirable diameter and length with a shell of 10-gage corrugated steel. The shell is supported on an axle by two structural frames, or spiders, at the third points, and two steel-plated bulkheads at each end. One plate has a hole to permit filling the roller with sand to increase its weight.

Clearing and Removing

Snow clearing and removal are required where climatic conditions will not permit packing or where snowfalls arc in excess of that which can be packed on the runway. Remove light snowfalls with a grader or rotary broom. Very light snowfalls can be blown off the runway by the prop wash of Several aircraft lined up along one edge. Remove heavy snowfalls with truck-mounted plows, rotary snowplows, rubber-tired tractors, or scoop loaders. Drifts may be opened by a truck or tractor with a V-type blade or by a rotary snowplow.

Equip trucks with tire chains and carry ballast for traction while plowing. Keep all blades about 1/2 inch above the runway surface. especially if the surface is a landing mat. This clearance is accomplished by runners mounted at each end of the blade.

The assignment of plows varies with the condition at each air field and the type of equipment available. Arranging the plows into units simplifies coordination of snow removal with the control tower. Ordinarily, on c- way, truck -mounted snowplows operate in echelon to expedite snow removal. Remove snow near landing lights and other obstructions with a blower, if available, or by hand.

The rapid removal of snow requires a rotary blower, snow loader, or other special equipment. Trucks used for hauling snow arc equipped with nigh sideboards. Tractor-drawn sleighs built of lumber may be used as an expedient hauling device or to supplement snow-handling trucks.

Abating Ice Conditions

Sprinkle ice coatings on runways, taxiwavs, and hardstands with urea, coarse sand, or cinders, spread by hand or by mechanical spreaders. If practical, heat abrasives before spreading. Remove accumulated abrasives in the spring by brooming, ice conditions on airfields used by jet aircraft arc a very serious problem because abrasives cannot be used. Do not use sodium chloride and calcium chloride for ice control without approval because these salts may promote corrosion of metal aircraft parts.

MAINTENANCE DURING FLYING OPERATIONS

Coordinate maintenance and repair work during flying operations, and plan the work for minimum interference with air and ground traffic. Much of the maintenance work may need to be done at night or during inclement weather in order not to interfere with flying operations.

Do not leave equipment hazardous to aircraft on the runway or other areas. Clearly mark construction or repair areas on the runway so that they are visible from the air. Mark repairs on taxiways so that they are visible to pilots while taxiing.

REHABILITATION OF CAPTURED AIRFIELDS

The decision to rehabilitate a captured enemy airfield and the decision as to the type and construction standard of the rehabilitated field are Air Force and Army responsibilities. The work is ordinarily accompi shed by a combat-heavy engineer battalion. The engineer mission is to convert the existing facilities, which arc usually damaged, to the standard decided upon by the Air Force and Army, with a minimum outlay of labor, equipment, and materials. Considerable discretion must be exercised in applying standard specifications to captured airfields. No large-scale relocation of facilities should be undertaken merely to conform to standard patterns, if the existing patterns will serve the same purpose in a satisfactory manner. Sensible, existing substitutions and deviations from specified arrangements must be recognized and accepted.

An appraisal of the damage done to a captured field precedes the decision to rehabilitate it. Occasionally, it is necessary to expend more effort to restore a badly damaged airfield than to construct a new one. The damage to the installation includes war damage by our forces in any battle for the airfield and the deliberate damage that the enemy did before yielding the field to our forces. Complete destruction of an airfield is a major undertaking; therefore, the enemy will likely resort to onc or more of the following less destructive measures:

- Placing delayed-action bombs, mines, and booby traps.
- Demolishing drainage systems and pavements.
- Placing obstacles and debris in the runway.
- Plowing turfed areas.
- Flooding surfaced areas.
- Blowing craters in runways, taxiways, and handstands.
- Demolishing buildings, utilities, and similar installations.

Assume these damages were inflicted when conducting such reconnaissance.

Use the criteria that follow to prioritize rehabilitation operations:

- When restoring a captured airfield, the first priority is to establish minimum facilities and utilities to include the establishment of a minimum operating strip for immediate operation of friendly aircraft. This also includes removing UXO, delayed-action bombs, mines, and booby traps from the traffic areas; clearing debris from those areas; and repairing craters on the runway and taxiway surfaces. Promptly repair the drainage system. Concentrate runway work first on a minimum operating strip; second, on an access route; and finally, on other traffic areas. Give early attention to the provision of suitable sanitary and water facilities. Chapter 7 of TC 5-340 gives detailed information regarding these areas.

- The second priority is improvements to the minimum operational facilities. Restore remaining runways, taxiways, hardstands, parking aprons, access and service roads, and fuel and bomb storage areas before rehabilitating other, less vital facilities.

- The third priority is the repair of buildings such as the control tower, operational buildings, crew shelters, communication centers, and other maintenance facilities.

- The fourth priority is the camouflage of installations; the restoration of utilities (making use of any utility map and any available citizen labor familiar with the installation's utilities); and the repair or establishment of bathing, dining, and recreational facilities. A complete cleanup of the grounds, including the removal of debris and seeding and sodding, is the last phase of a rehabilitation project.

ROAD DESIGN

CHAPTER 9

Road selection and design depend on the nature of the subgrade; the traffic and drainage conditions; the construction time available; the supply of local and imported materials; and the engineer equipment, personnel, and expertise available. The completed design must then meet the requirements for the given load class and allow safe and efficient traffic movement.

The load-carrying capacity of a road surface depends on continuous, stable support furnished by the subgrade. Subgrade stability requires adequate drainage and proper load distribution by the surface and base courses. Surface and base courses of sufficient thickness and quality to spread the wheel loads over the subgrade are necessary so that the applied stress is less than the unit load capacity of the subgrade. in areas where seasonal freezing and thawing occur. the load-carrying capacity of inadequately designed or improperly constructed roads can be dramatically decreased to the extent that failure may occur.

For safe and speedy traffic movement, the geometric design requirement for given road classes must be met. In a combat zone, military urgency dictates rough, hasty work designed to meet pressing needs. An improved network of well-surfaced, high-quality roads may be required in rear areas and near major airfields, ports, and supply installations. Road design uses stage construction for the progressive improvement of the road to meet increased traffic demands. Road design also uses many technical terms. Figures 9-1 and 9-2, page 9-2, show terms used to designate road features and components. In addition to this chapter TM 5-337 provides additional detailed information on the design of bituminous and concrete-surfaced roads.

GEOMETRIC DESIGN

The geometric design process begins with good-quality topographic surveys. In most cases, a minimum 5-foot contour interval is required to clearly describe the terrain. The design process can be described in the following steps:

1. Draw the proposed centerline on the topographic survey.

2. Plot the centerline on plan-and-profile paper.

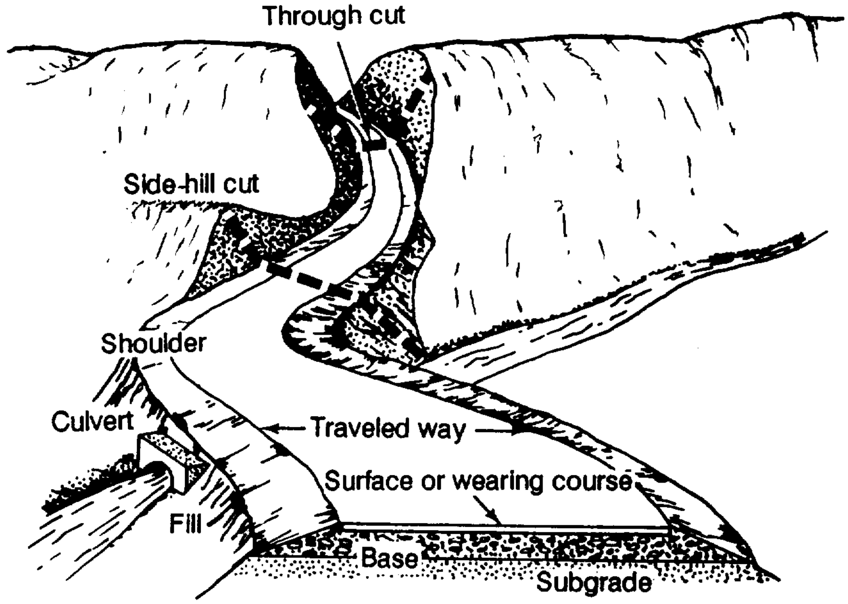


Figure 9-1. Road nomenclature

3. Calculate grades, the degree of curvature of horizontal curves, and curve lengths of vertical curves.

4. Compare the values of step 3 with the military road specifications stated in Table 9-1.

5. Adjust the centerline, if possible, to reduce any calculated grades and limit horizontal and vertical curves that exceed the specifications.

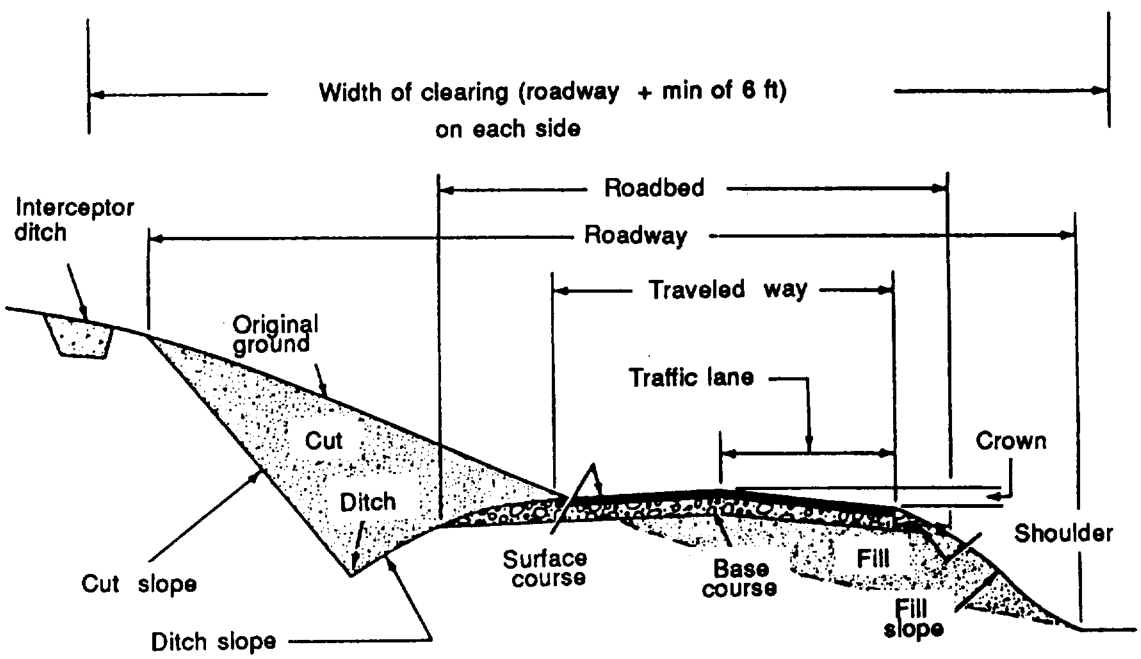


Figure 9-2. Road cross section and nomenclature

Table 9-1. Geometric design data for military roads

Design Controls and Elements		Class A (4 Lane)	Class B (2 Lane)	Class C (2 Lane)	Class D (1 Lane)
Design Controls					
1. Traffic composition	(1)				
Average daily traffic (ADT) (45% trucks)		3,400-6,700	935-3,400	200-935	Under 200
Design hourly volume (DHV)		510-1,000	140-510	30-140	Under 30
Sight distance restriction, %	(2)	40-0	80-0	80-40	100
2. Design speed (V), mph (kph)		60 (97)	60 (97)	40 (64)	30 (48)
Average running speed, mph (kph)		45 (72)	45 (72)	35 (56)	25 (40)
Cross-Section Elements					
3. Pavements	(3)				
Minimum width of traffic lane, ft (m)					
with barrier curb		12 (3.658)	12 (3.658)	10 (3.048)	10 (3.048)
without barrier curb		12 (3.658)	12 (3.658)	10 (3.048)	10 (3.048)
Minimum distance between curb faces, ft (m)		53 (16.154)	29 (8.839)	25 (7.620)	15 (4.572)
Lateral clearance from edge of traffic lane to obstructions, ft (m)		6 (1.829)	6 (1.829)	6 (1.829)	4 (1.219)
Normal cross slope (crown slope) rate		0.0104-0.0108	0.0104-0.0208	0.0208-0.0417	0.0208-0.0417
4. Shoulders	(4)				
Minimum width w/o barrier curbs, ft (m)		10 (3.048)	10 (3.048)	6 (1.829)	4 (1.219)
Normal cross slope, rate		0.0417-0.0625	0.0417-0.0625	0.0417-0.0625	0.0417-0.0625
Type, (perm road)		Dustless	Stable	Compacted soil	Compacted soil
5. Bridge clearance (perm)*					
6. Curb offset for barrier curb, ft (m)		2.5 (0.762)	2.5 (0.762)	2.0 (0.610)	2.0 (0.610)
Alignment Elements					
7. Sight distance					
Minimum stop sight distance, ft (m)		475 (144.780)	475 (144.780)	275 (83.820)	200 (60.960)
Minimum pass sight distance, ft (m)		N/A	2,100 (640.081)	1,500 (457.201)	N/A
8. Horizontal alignment					
Maximum horizontal curvature	(5)	5.5°	5.5°	14.5°	26.7°
Pavement widening, ft (m)	(6)	None	None	2-4 (0.610-1.1219)	2-5.5 (0.610-1.676)
9. Vertical alignment					
Grade					
Maximum grade, %		6	6	10	15
Critical length, ft (m)	(7)	700 (213.360)	700 (213.360)	450 (137.160)	250 (76.200)
Minimum grade, %		0.3	0.3	0.3	0.3
Vertical curves	(8)				
Overt (crest) vertical curve k, ft (m)		160 (48.768)	160 (48.768)	55 (16.764)	35 (10.668)
Invert (sag) vertical curve k, ft (m)		105 (32.004)	105 (32.004)	55 (16.764)	28 (8.534)
Absolute minimum length, ft (m)		180 (54.864)	180 (54.864)	120 (36.576)	80 (24.384)

Remarks:

(1) The DHV shown for all roads is in total vehicles per hour for all lanes in both directions. The DHV is approximately 15 percent of the ADT.

(2) The values shown for this term indicate the combined effects of horizontal (curves) and vertical (grade) alignment on capacity. A value of zero percent indicates an absolutely straight, flat alignment with no restriction on sight distance. A value of 100 percent indicates a road with numerous sharp curves and grade changes on which the sight distance is less than 1,500 ft(457.201 m) at any point on the road.

(3) If the anticipated traffic includes a significant number of vehicles having widths in excess of 8.5 ft (2.591 m), the traffic lanes should be widened in the amount by which the vehicle width exceeds 8.5 ft (2.591 m).

(4) There should be a color or texture contrast between traffic lane and shoulder surfaces.

(5) Values shown are calculated on basis of maximum rate of superelevation of 0.100.

(6) Pavement widening for a class C or class D road varies 2 to 5.5 ft (0.610 to 1.676 m) as the curvature varies from 2 to 26.7°. Values obtained may be rounded off to the nearest 0.5 ft (0.152 m).

(7) The term *critical length* is used to indicate the maximum length of a designated upgrade upon which a loaded truck can operate without an unreasonable reduction in speed. Critical lengths may be increased at an approximate rate of 50 ft (15.240 m) per percent decrease in grade from the values shown.

(8) The minimum lengths of vertical curves are determined by multiplying k by the algebraic differences in grades (in percent).

Notes:

1. As can be seen, capacities are shown as a range of values. If maximum (or minimum) design values shown are rigidly adhered to, then the resultant capacity of the road will be on the lower side of the capacity range. Therefore, discretion should be used in selecting design values by avoiding maximums or minimums whenever possible.

2. Turnouts should be provided at 1/4-mile (402.250 m) intervals on class-D roads.

3. Curbs will generally not be provided in open areas.

*Bridge clearance (permanent) width of the traveled way should be equal to the width of the lanes plus 5 ft (1.524 m) [2.5 ft (0.762 m)] on each side; 14.75 ft (4.498 m) vertical clearance.

6. Plot new tangents (straight sections of road) on the plan and profile in those locations where horizontal and vertical curves exceed the military road specifications.

7. Design horizontal and vertical curves for all tangent intersections.

8. Plot newly designed curves on the plan and profile.

9. Develop a mass diagram for the project. Balance the cuts and fills and optimize ruling grade and earthwork volumes.

10. Design superelevations (curve *banking)* and widening for all horizontal curves.

11. Draw typical cross sections.

12. Design the required drainage structures and bridges.

SELECTION OF ROAD TYPE

Structural characteristics should accommodate traffic volumes throughout the road's design life. Table 9-1, page 9-3, shows four possible road types. They are based on expected traffic volumes and show the values for the design control elements for each road class. The capacities arc shown as a range of values. Only road classes B, C, and D apply to TO construction. If the maximum (or minimum) design value for the various criteria is always adhered to, the resulting vehicle capacity of the road will be on the lower side of the range. Use discretion by designing the road to the best possible standard in a given road class.

DESIGN CALCULATION

The values in Table 9-1 for each geometric feature must be attained to ensure that the desired road will have a capacity equal to or greater than either the average daily traffic (ADT) or design hourly volume (DHV) shown. The first step in the design of a road is to estimate the daily or hourly number of vehicles in a military organization. Where this cannot be done, the number of vehicles organic to the units that will use the road, multiplied by a factor of two, is suggested as a reasonable estimate. This conservatively assumes that each vehicle uses the road twice (one round-trip) per day.

Figure 9-3 shows the relationship between DHV and sight distance restriction. If either anticipated DHV or ADT is known and the sight distance restriction can be estimated from preliminary plans, the necessary road type can be determined from Figure 9-3. If ADT or DHV and the road type desired are known, sight-distance-restriction requirements can be determined from Figure 9-3.

A range of possible DHV values is given for each road classification in Table 9-1. The actual DHV for a road is a function of the sight-distance-restriction factor, which also has an allowable range for each type of road. The DHV varies directly with a change in the sight-distance-restriction factor. Figure 9-3 shows this straight-line relationship.

After the sight-distance-restriction factor is determined from the design plans of an assumed road class, the actual DHV is determined to ensure that the capacity is adequate.

Example:

A road is to be designed for a military organization having approximately 250 vehicles.

ADT = 250 x 2 = 500 and DHV = 0.15 x 500 = 75. The 0.15 factor clusters the traffic into rush hours. Otherwise, the hourly volume = 500 vehicles (per day)/24 hours (per day) - 21 vehicles per hour.

Solution:

The calculated DHV of 75 could be met by a class C road. Therefore, assuming a class C road is used, plan-and-profile designs could be drawn and a sight-restriction factor can be determined from the design.

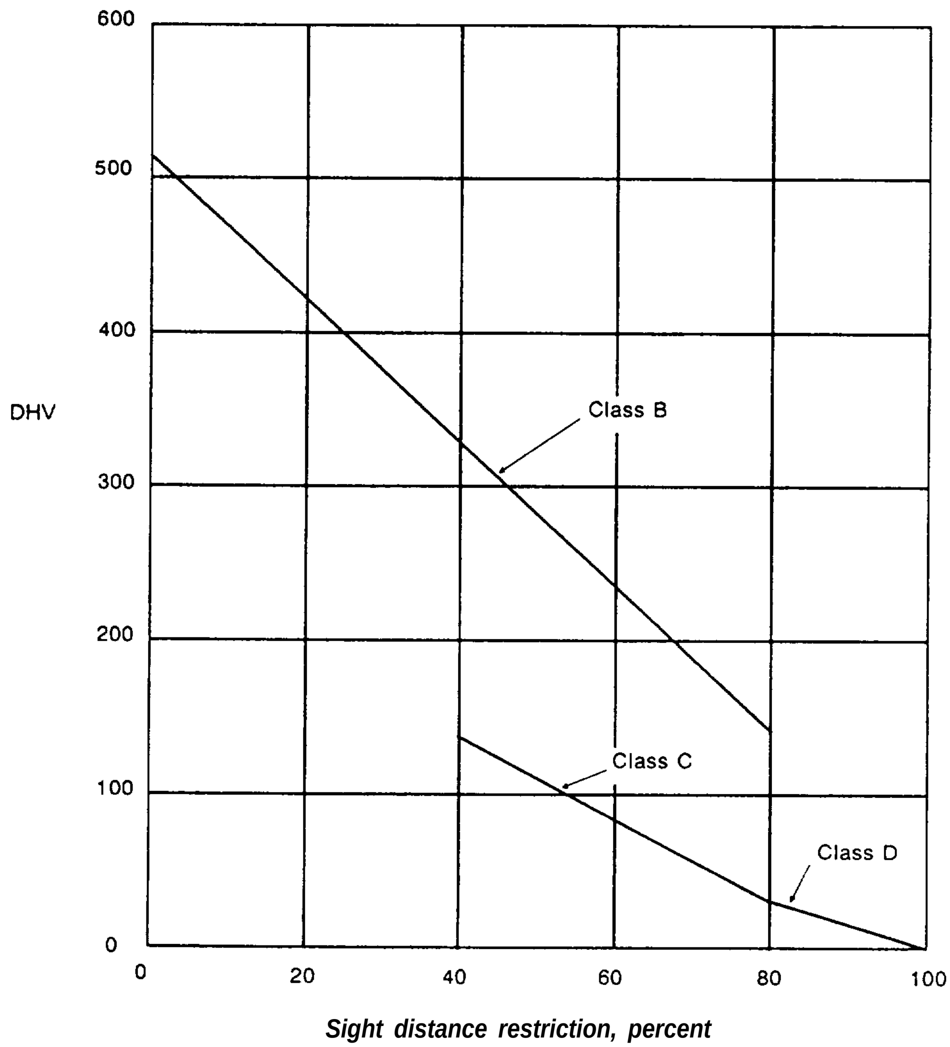


Figure 9-3. Interpolation of DHV for selection of road class (not to scale)

From Figure 9-3, a sight-distance-restriction factor of 62 percent is determined (based on a class C road and a DHV of 75). Using Figure 9-3, the maximum sight-distance-restriction factor for a class C road is 80 percent. This provides a DHV for a class C road of only 30. Since the sight-distance-restriction factor for the example (62 percent) is less than the maximum of 80, this meets the initial requirement of the DHV being greater than or equal to 75. Therefore, the class C road assumption is adequate. If a DHV of 75 could not be handled by the class C road, it would be necessary to construct a class B road.

ESTIMATING CAPACITY

The information in Figure 9-3 and Table 9-1, page 9-3, is adequate for the geometric design of military roads. However, additional information is available in TM 5-822-2. The information can be used to evaluate the capacity of existing roads by obtaining pertinent characteristics and comparing them to the values in Table 9-1. If the data does not conform to that shown for a given road type, use discretion in estimating the road type and ADT or DHV to which the data best conforms. The volume and capacity values in Table 9-1 are for roads of a given class when they are new or in good condition. As the road surface deteriorates, the road is less able to accommodate the traffic for which it was designed. Plan and carry out a maintenance program to keep the road in good condition.

GRADE AND ALIGNMENT

Before building a road or an airfield, the engineer must determine the best vertical and horizontal alignment of the facility concerned. Design both horizontal and vertical alignment to keep sight distance restrictions to a minimum. Define the route by a series of straight lines and curves to meet the stated mission and capacity. This provides the shortest, most efficient route that requires the least construction effort. Define the route vertically in a series of grades and curves that fall within acceptable specifications and requirements. Horizontal and vertical alignment are interrelated and must be considered concurrently. However, the principles on each are best studied separately. Horizontal and vertical curves of all types are discussed in FM 5-233.

HORIZONTAL ALIGNMENT AND HORIZONTAL CURVES

The principles of horizontal alignment are summarized as follows:

Tangents (straight sections of road) should be as long as possible, because the shortest distance between two points is the connecting straight line. Terrain conditions, however, seldom permit the construction of a route between two points in one tangent line. Therefore, the engineer should make each tangent as long as possible, limit the number of curves, and provide long straight stretches, thereby improving the route capacity.

Make curves as gentle as possible. Long, gentle curves increase the capacity of the roadway by permitting higher speeds. They also provide a safer path of travel for the vehicle. Making gentle, horizontal curves will increase the curve length, thereby decreasing the tangent length. However, this reduction in tangent length is minor compared to the benefits gained by reducing the total number of curves.

Tangents should intersect other roads and railroads at right angles. Military roads normally supplement existing roadnets and have intersections at one or both ends of the military road. Operating efficiency usually is improved when these intersections approach right angles.

Frequently used horizontal curves are shown in Figure 9-4. The most common are the simple curve, the reverse curve, the compound curve, and the spiral curve.

- A simple curve uses the arc of a circle to provide a smooth transition between two tangents. This curve is used frequently in the TO because it fills the needs of the low-speed design roads normally used and is easy to construct. A reverse or compound curve can be designed using the same basic equations.

- A reverse curve uses two simple curves tangent to a common line at a common point. Their centers are on opposite sides of the common line. The radii of the curves may or may not be equal in length.

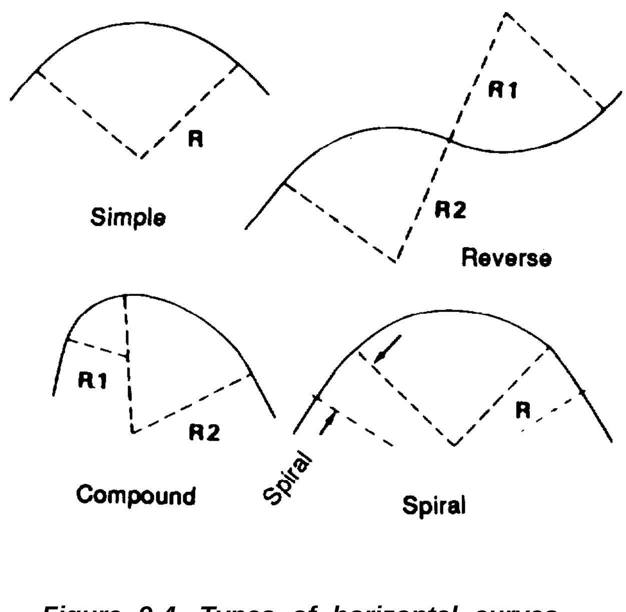


Figure 9-4. Types of horizontal curves

- A compound curve has two simple curves tangent to a common line at a common point. The centers of these curves are on the same side of the common line, and the curves have radii of different lengths.

- A spiral curve is a simple curve in the center with parts of a spiral on each end to smooth transition to the tangent. The spiral is used only on high-speed roads (classes A and B). Detailed steps for the design and layout of spiral transition curves are in FM 5-233. Low design speeds of class-C and -D roads do not require spiral transition sections.

ELEMENTS OF A HORIZONTAL CURVE

The following are elements of a simple, horizontal curve as shown in Figure 9-5:

- The PC is the point where the curve begins or leaves tangent A—the tangent nearest the origin of stationing (station 0 + 00) or start of the project.

- The PT is the point where the curve ends or joins tangent B.

- The PI is the intersecting point of two tangents that must be connected by a horizontal curve.

- The tangent distance (T) is the distance from the PI to the PC or from the PI to the PT.

- The radius (R) of curvature is the radius of the circle whose arc forms the curve from the PC to the PT.

- The length of curve (L) is the distance from the PC to the PT along the curve, measured as an arc or as a series of 100-foot arcs. Railroad engineers measure L as a series of 100-foot chords.

- The angle of intersection (I) is the exterior angle at the PI formed by tangents A and B. The central angle, between the radius points at O, is equal to the exterior angle.

- The external distance (E) is the distance from the PI to the midpoint of the curve.

- The long chord (C) is the straight-line distance from the PC to the PT.

- The middIe ordinate (M) is the distance from the midpoint of the curve to the midpoint of the long chord.

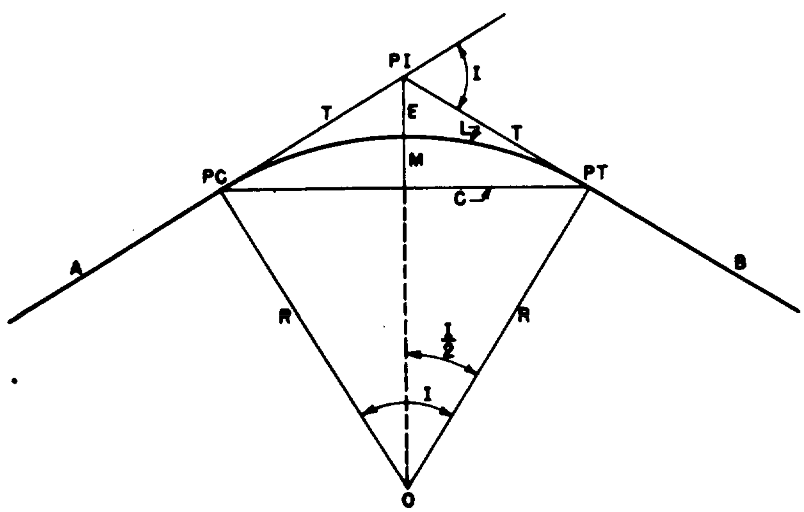


Figure 9-5. Elements of 8 simple, horizontal curve

DEGREE OF CURVATURE

The connecting curve between two tangents may be short and sharp or long and gentle, depending on the properties of the circle chosen. Sharpness is defined by the radius of the circle. For example, a curve may be called a 150-foot curve. However, a curve is seldom referred to by its radius because the center of the curve is often inaccessible on the long, gentle curves used on modern highways. The more practical and common reference term for defining curve sharpness is the degree of curvature (D). The degree of curvature is established as a whole or half degree. The degree of curvature may be stated in terms of either the arc or the chord.

Arc Definition

The degree of curvature, D, is that angle which subtends a 100-foot arc along the curve. (See Figure 9-6.) This definition is used by state highway departments and the Corps of Engineers in road design.

Chord Definition

The degree of curvature, D, is the angle which subtends a 100-foot chord on the curve. (See Figure 9-7.) This definition results in a slightly larger angle than the arc method, and it is used by the railroad industry and the Corps of Engineers in railroad design.

The difference between the arc and chord definitions is very slight and nearly insignificant (frequently well below 1 percent) for TO construction. However, because the arc definition is the most widely used procedure in road design, only its definition will be used throughout the rest of the chapter.

EQUATIONS FOR SIMPLE, HORIZONTAL-CURVE DESIGN

The two methods commonly used to solve horizontal curve problems are the 1-degree-curve method and the trigonometric method. Both methods may be used with the same degree of accuracy. The 1-degree-curve method requires the Functions of a 1-Degree Curve table shown in Appendix F of this manual.

Appendix F is based on the trigonometric relationships for a curve of D = 1°. Curves of different degrees of curvature can be readily designed because of the proportionality between all curves and the l-degree curve. For example, a curve of D = 15° has one-fifteenth the L, E, T, and M values as for a 1-degree curve (D = 10). The only information needed to obtain the L, E, T, and M values for a 1-degree curve is the angle of intersection (I), and I is always known at the onset of the design process. The trigonometric method requires a calculator with trigonometric functions or trigonometric tables found in TM 5-236 or any surveying manual.

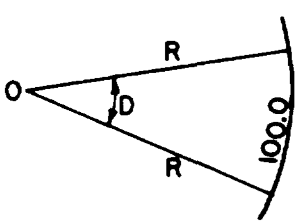


Figure 9-6. Arc definition for degree of curvature

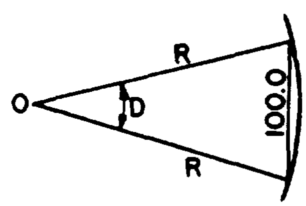


Figure 9-7. Chord definition for degree of curvature

Radius of Curvature

As previously described in the arc definition, D is that angle subtended by a 100-foot arc on a circle. By comparing the 100-foot arc and the total circumference of the circle, an equation for R is developed in terms of D.

$$\frac{D_{arc}}{100'} = \frac{360^\circ}{2\pi R}$$

Solving for R—

$$R = \frac{(100)(360)}{2\pi D} = \frac{5{,}729.58}{D}$$

Tangent Distance

In the right triangle shown in Figure 9-5, page 9-7, the vertices are at PC (or PT), PI, and O. The tangent distance (T) is found using the 1-degree-curve method, as follows:

$$T = \frac{T_{1^\circ}}{D} \text{ (arc definition)}$$

T_{1° is found in Appendix F, Table F-1, for a given I. Use Table F-2 to determine the chord correction.

External Distance

Using the 1-degree-curve method (refer to Figure 9-8), the external distance (E) is found as follows:

$$E = \frac{E_{1^\circ}}{D} \text{ (arc definition)}$$

Middle Ordinate

Using the 1-degree-curve method (refer to Figure 9-8), the middle ordinate (M) is found as follows:

$$M = \frac{M_{1^\circ}}{D} \text{ (arc definition)}$$

Length of Curve (L)

Measure the length of the curve in 100-foot arcs. Because D subtends a 100-foot arc, the total number of such arcs in a horizontal curve must be the number of times that D can be included in the central angle I.

$$L = \frac{I}{D} \times 100$$

The central angle subtended by the entire horizontal curve has sides that are radii to the PC and PT. Both of these radii are perpendicular to the tangents that form the

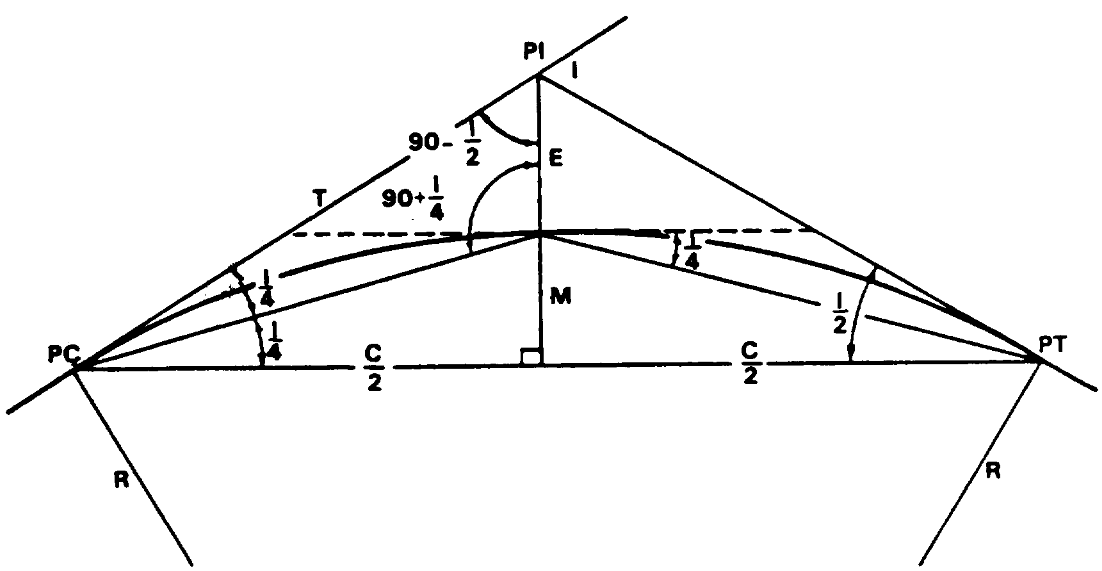


Figure 9-8. Derivation of external distance

intersection angle I. The quadrilateral formed by the four points of PI (180°-I), PC (900), O (I), and PT (90°) must total 360°. 1(180-I) + 90 + I + 90 = 360.] Hence, the central angle is equal to the angle of intersection I.

DESIGNING HORIZONTAL CURVES

The engineer designing horizontal curves must know two facts about the curve from the preliminary survey: the location and station of the PI and the angle between intersecting tangent lines (I). The curves can be designed after this information is obtained.

The first step is to determine the desired sharpness of the curve. This is defined by the radius or the degree of curvature. Topographic conditions govern the final location of the centerline and sharpness of the curve. A maximum or minimum tangent distance may fit the terrain conditions, or there may be a limit on the external distance or the middle ordinate. If a restriction exists, solve for the degree of curvature by transposing the equations previously given. Where no terrain condition dictates the sharpness of the curve, choose a degree of curvature within allowable specifications.

When choosing a degree of curvature, remember that gentle curves are more desirable. However, these long curves may increase surveying and construction time, materials, and effort required. There is no restriction on the length of the curve with respect to a minimum degree of curvature. However, the maximum allowable degree of curvature is specified by the road classification. Table 9-1, page 9-3, specifies the maximum degree of curvature for each class of road as stated in the row titled "Maximum horizontal curvature."

After the degree of curvature is selected, determine the stations of the PC and the PT. Next, design the curve except for the calculations needed to locate stationing points of the curve between the PC and PT. The following steps show the design of horizontal curves using the 1-degree-curve method:

1. Find the degree of curvature, D, by one of three methods:

- If the curve is unrestricted,

$$D = \frac{5{,}729.58}{R}$$

where R = the radius of the curve

- If the curve is restricted by the tangent distance,

$$D = \frac{T_{1^\circ}}{T_{(restricted)}}$$

where—

T_{1° = tangent distance for a 1-degree curve (found in Appendix F, based on the angle of intersection)

$T_{(restricted)}$ = restricted tangent distance for a horizontal curve

- If the curve is restricted by the external distance,

$$= \frac{E_{1^\circ}}{E_{(restricted)}}$$

where—

E_{1° = external distance for one-degree curve (found in Appendix F, based on the angle of intersection)

$E_{(restricted)}$ = restricted external distance for a horizontal curve

2. Round up the degree of curvature to the next half degree when possible.

3. Determine the length of the tangent.

$$T = \frac{T_{1^\circ}}{D}$$

4. Find the stationing of PC.

PC = P I - T

5. Calculate the length of the curve.

$$L = \left(\frac{I}{D}\right) 100$$

6. Find the stationing of PT.

$$PT = PC + L$$

Horizontal-Curve Design Examples

This section describes the horizontal-curve design procedures for three common situations:

- No terrain restriction which limits T or E.
- Terrain restriction of the tangent distance.
- Terrain restriction of the external distance.

Example:

Degree of Curvature with No Terrain Restriction. Figure 9-9 illustrates the following computations:

Given: I = 50°, PI at 14 + 28

Find the station and location of PC and PT for a class C road.

Solution:

A degree of curvature, D, of 6° is selected as a flat, gentle curve. D = 6° is far below the maximum allowable of D = 14.5° for class-C roads and is slightly sharper than the maximum allowable of D = 5.5° for class-B roads.

Use D = 6°

$$R = \frac{5{,}729.58}{D} = \frac{5{,}729.58}{6} = 954.93'$$

$$T = \frac{T_1°}{D} = \frac{2{,}671.58}{6} = 445.30' \text{ (arc}$$

definition)

$$L = \frac{I}{D} \times 100 = \frac{50}{6} \times 100 = 833.33'$$

PC = PI - T = (14 + 28) - (445.29')
= (9 + 82.71)

PT = PC + L = (9 + 82.71) + (833.33')
= (18 + 16.04)

The station of the PT is determined by adding the curve length to the station of the PC, not by adding T to the station of the PI. However, the actual point of the PT is found by measuring a distance T (at angle I) from the PI. The station is the distance from the point of origin at station (0+ 00), as measured along the centerline.

Example:

Terrain Restriction of the Tangent Distance. Figure 9-10 illustrates the following computations:

Given: I = 32°, PI at 25 + 87, T_{Res} is 282' (due to bridge)

Find the station and location of PC and PT.

Solution:

Knowing that T must not exceed 282 feet and that the D that will give this value is

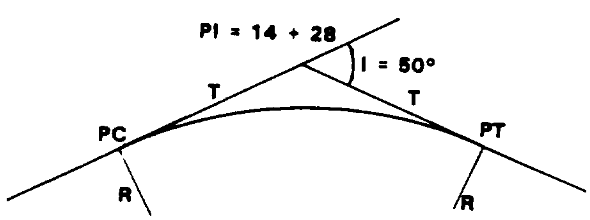


Figure 9-9. Horizontal curve with no sharpness restriction

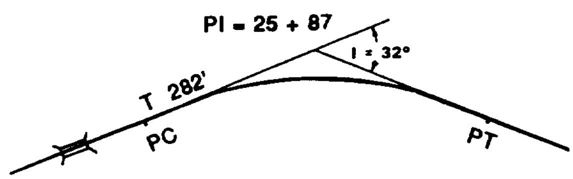


Figure 9-10. Horizontal curve with restriction on tangent

probably not equal to a whole or half degree, it is necessary to first find which D gives 282 feet for T_{Res} and then round it off, as shown.

$$D = \frac{T_{1°}}{T_{Res}}$$

$$D = \frac{1,643}{282} = 5.83° = 5°50'$$

If the value of T was specified as exactly 282 feet (as opposed to a maximum or restricted value), the value for D of 5°50' must be used. Rounding D up to the next half degree (D = 6°) will slightly sharpen the curve and will reduce T slightly below the 282-feet maximum.

Use D = 6°

$$R = \frac{5,729.58}{D} = \frac{5,729.58}{6} = 954.93'$$

$$T = \frac{T_{1°}}{D} \text{ (arc definition)}$$

$$T = \frac{1,642.9300}{6} = 273.82'$$

$$L = \frac{I}{D} \times 100 = \frac{32}{6} \times 100 = 533.33'$$

$$PC = PI - T = (25 + 87) - (273.82') = (23 + 13.18)$$

$$PT = PC + L = (23 + 13.18) + (533.33') = (28 + 46.51)$$

Increasing the degree of curvature decreases the values of the radius and tangent distance and vice versa. When the degree of curvature was changed from 5°50' to 6°00', it caused the radius and the tangent distance to decrease from 983.5 feet to 954.9 feet and 282 feet to 273.8 feet, respectively. Therefore, if the maximum value of the tangent distance is used to determine a trial value of D, the rounding must be to the next higher half degree. If minimum value of the tangent is given, the rounding is to the next lower half degree.

Example:

Terrain Restriction on the External Distance. Figure 9-11 illustrates the following computations:

Given: E $\leq$ 85 feet, I = 80°, Sta PI at 43 + 32.75

If E exceeds 85 feet, the road centerline will be closer than 25 feet to the building.

Find the station and location of PC and PT.

Solution:

$$D = \frac{E_{1°}}{E_{Res}}$$

$$D = \frac{1,749.9}{85} = 20.59° = 20°35'$$

Because the limiting value for the external distance and the value used to get this trial value of D are maximums, it is necessary to round to the next higher half degree, thereby decreasing T and E.

Use D = 21°

$$R = \frac{5,729.58}{21} = 272.83'$$

$$T = \frac{T_{1°}}{D} = \frac{4,807.6867}{21} = 228.94'$$

$$E = \frac{E_{1°}}{D} = \frac{1,749.8548}{21} = 83.33 \leq 85' \text{ (check)}$$

$$L = \frac{I}{D} \times 100 = \frac{80}{21} \times 100 = 380.95'$$

$$PC = PI - T = (43 + 32.75) - (228.94') = (41 + 03.81)$$

$$PT = PC + L = (41 + 03.81) + (380.95') = (44 + 84.76)$$

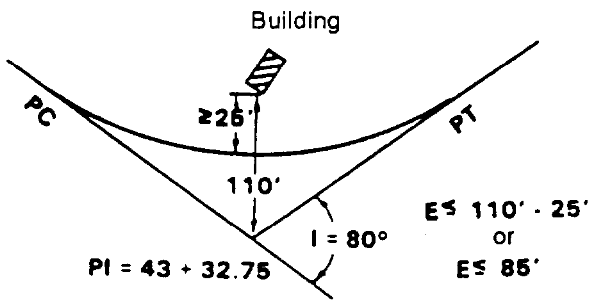


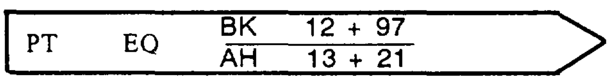
Figure 9-11. Horizontal curve with restriction on the external distance

Station Adjustments Due to Curve Installation

Horizontal curves occasionally are designed at the site by the surveying team. When this is done, the route is staked out and stationed progressively along the centerline from the point of origin of the project. It is not necessary to calculate station adjustments required by the shortening of the overall centerline length by a distance of 2T-L. However, when horizontal curves are designed in the office with data supplied by the preliminary survey, the adjustments must be calculated.

When the preliminary tangent alignment of a route is first determined and stationing along the tangent lines is accomplished, the station of any point represents its distance from the point of origin as measured along straight lines only. When a horizontal curve is installed and becomes the centerline of the route, the stationing distance from the PC to the PT is shortened by 2T-L for each horizontal curve.

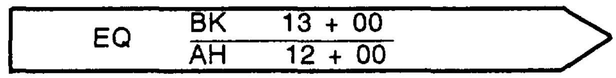
In Figure 9-5, page 9-7, the initial centerline distance from the PC to the PT is measured along the two tangents and is equal to 2T. When the curve is installed and the new centerline is created, the final centerline distance from the PC to the PT becomes L. At this point, the centerline stationed ahead would need to be restationed or adjusted in some manner. To prevent restaking the rest of the project centerline, an adjustment is made to the construction stake at the PT. The method of adjustment will produce a stationing equation at the point of adjustment that will satisfy both the stationing back and the stationing ahead. The equation will have a station which corresponds with the correct station to the rear (or back) and the correct station forward (or ahead). The equation will be written on the construction stake as follows:



where—

PT = point of tangent
EQ = equation
BK = correct station back
AH = correct station ahead

Equations most often will occur at the PT but may be used anywhere an adjustment to the centerline stationing is required. The equation indicates that an adjustment to the centerline stationing has occurred for some reason. For example, if the survey crew accidently placed two centerline stakes with the same station number, say 13 + 00, the equation stake would look like this:



The adjustments shown in the preceding equation indicate that the total length of the road has been shortened by the difference of 24 feet in the first example and lengthened by 100 feet (or one station) in the second example.

Equations normally are shown in the profile section of the plans as a gap in the grade with the back and ahead stations written out.

Field Methods of Curve Layout

The location and station of the PC and PT of a horizontal curve constitute only two points on the curve. They do not adequately define the necessary construction. The following methods are applicable to military construction for locating points on the curve:

Arc Method. When the radius of a curve is less than 100 feet and topographic conditions permit, locate the center of the circle and swing an arc to locate a curve or fillet, Curves with a small radius are seldom used except at street intersections and for fillets between hardstands, taxiways, or other operational features of an airfield.

External-Distance Method. For short curves where three points are adequate for the construction standard desired, calculate the external distance and use it to locate the center of the curve. This method is not recommended for precise construction or for long curves. It is impractical when the terrain on the curve side of the PI is difficult to negotiate and measure.

Deflection-Angle Method. This method of curve layout usually is the fastest and most exact method, particularly for curves with a long radius. In the arc definition, a deflection angle is the angle formed between a tangent line and a chord from the same point. (See Figure 9-12.)

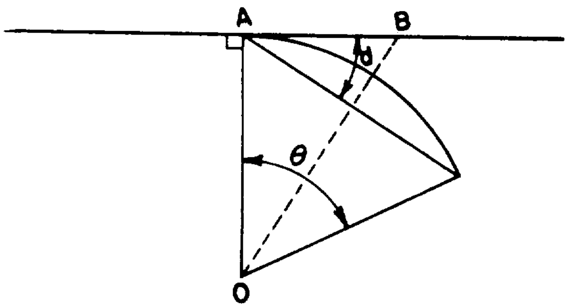


Figure 9-12. Deflection angle

In triangle OAB—

$$\text{Angle ABO (at B)} = 90 - \frac{\theta}{2}$$

$$d = 180 - (90 + B)$$

$$d = 180 - 90 - 90 + \frac{\theta}{2}$$

$$d = \frac{\theta}{2}$$

NOTE: In other words, the deflection angle is always one-half the intercepted central angle.

If the initial arc is 100 feet long, the central angle will be the degree of curvature D, and the deflection angle will be one-half the degree of curvature D, or D/2, as shown in Figure 9-13. With the addition of each 100-foot arc, the total central angle increases by D and the total deflection angle increases by D/2.

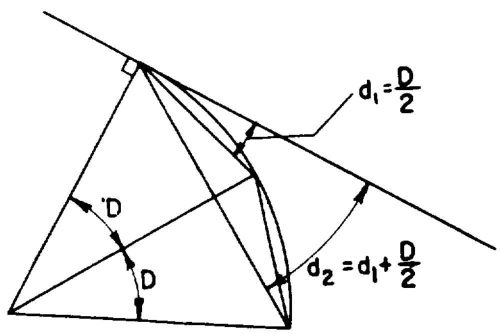


Figure 9-13. Deflection angles for 100-foot arcs

When laying out a curve, it is common practice to locate stakes at every full station. In view of this and because the PC of any curve rarely falls on an even station, the first arc will be something less than 100 feet in length (called a subarc). The deflection angle for the subarc to the first full station is the same proportion of D/2 that the subarc is to 100 feet (Figure 9-14).

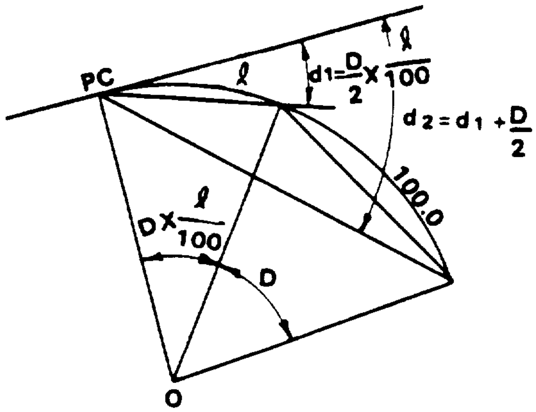


Figure 9-14. Subarc deflection angles

Once located at a full station, the curve continues by 100-foot arcs. Therefore, the central angle increases by D and the deflection angle increases by D/2 (Figure 9-15).

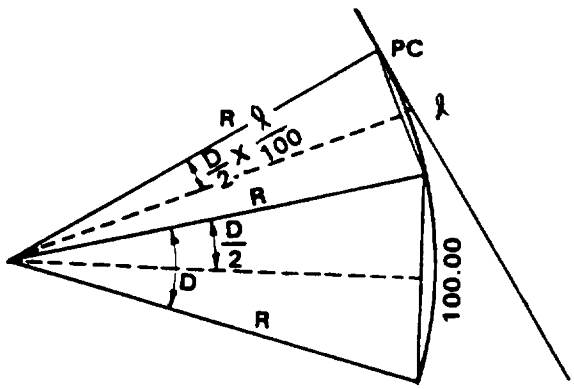


Figure 9-15. Calculation of chord lengths

The length of the chord for a 100-foot arc on the curve is equal to 2R Sin (D/2). For the first and last arcs, which are almost always less than 100 feet, the measured chord Is equal to *2R Sin [(D/2)(L/100)]* where L is the distance from the PC or PT to the closest full station.

A summary of the deflection-angle and chord-length calculations is shown in Figure 9-16.

LOCATION ON ARC	DEFLECTION ANGLE	CHORD LENGTH
PC to first full station	$d_1 = \frac{D}{2} \times \frac{\ell}{100}$	$C_1 = 2RSin\left(\frac{D}{2} \times \frac{\ell}{100}\right)$
Full station to full station	$d_n = \frac{D}{2} + d_{n-1}$	$C_n = 2RSin\left(\frac{D}{2}\right)$
Last full station to PT	$d_f = \left(\frac{D}{2} \times \frac{\ell}{100}\right) + d_{f-1}$	$C_f = 2RSin\left(\frac{D}{2} \times \frac{\ell}{100}\right)$

Figure 9-16. Deflection-angle and chord-length determinations

Example:

In the example of a horizontal curve with no terrain restrictions, the station of PC is 9 + 82.71, the station of PT is 18 + 16.04, and the deflection angles and chord distances are shown in Table 9-2. A useful check on the long series of computations is that the final deflection angle from the PC to the PT must always equal I/2. This is based on the previously stated principle that the deflection angle (from PC to PT) is one-half the total angle subtended (I). This check is illustrated in Table 9-2 for d = 25° = 1/2 = 500/2 for station, (18 + 16.04), which is the PT.

Layout Techniques

When using the deflection-angle method, set the transit up at the PC. Set zero on the vernier, sight the PI (or take a backsight down the centerline), and turn the first deflection angle. Measure the subarc distance along the instrument's line of sight. To locate the second point on the

Table 9-2. Deflection angles and chord distances

Station	Deflection computation	d	Chord computation	chord length
PC 9+82.71				
10 + 00	$d_1 = \left(\frac{D}{2} \times \frac{\ell}{100}\right) = \frac{6}{2} \times \frac{17.29}{100} = 0.519°$. 0.519x60=31.1	0° 31' 6"	$C = 2R\ Sin\left(\frac{D}{2}\frac{\ell}{100}\right) = 2(954.93)(0.52336)\left(\frac{17.20}{100}\right)$	17.28'
11 + 00	$d_2 = d_1 + \frac{D}{2} = .519 + \frac{6}{2} = 3.519°$	3° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
12 + 00	$d_3 = d_2 + \frac{D}{2} = 3.519 + \frac{6}{2} = 6.519°$	6° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
13 + 00	$d_4 = d_3 + \frac{D}{2} = 6.519 + \frac{6}{2} = 9.519°$	9° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
14 + 00	$d_5 = d_4 + \frac{D}{2} = 9.519 + \frac{6}{2} = 12.519°$	12° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
15 + 00	$d_6 = d_5 + \frac{D}{2} = 12.519 + \frac{6}{2} = 15.519°$	15° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
16 + 00	$d_7 = d_6 + \frac{D}{2} = 15.519 + \frac{6}{2} = 18.519°$	18° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
17 + 00	$d_8 = d_7 + \frac{D}{2} = 18.519 + \frac{6}{2} = 21.519°$	21° 31' 6"	$C = 2R\ Sin\frac{D}{2} + 2(954.93)(0.52336)$	99.954'
18 + 00	$d_9 = d_8 + \frac{D}{2} = 21.519 + \frac{6}{2} = 24.519°$	24° 31' 6"	$C = 2R\ Sin\frac{D}{2} = 2(954.93)(0.52336)$	99.954'
PT 18+16.04	$d_{10} = d_9 + \left(\frac{D}{2} \times \frac{\ell}{100}\right) = 24.519 + \frac{6}{2} \times \frac{16.04}{100} = 24.519 + .481 = 25.00$	25° 00' 00" check $\frac{I}{2} = 25°\ 00'$	$C = 2R\ Sin\left(\frac{D}{2}\frac{\ell}{100}\right) = 99.954\left(\frac{16.04}{100}\right)$	16.032'

curve (station 11 + 00), turn the second deflection angle (the angle is measured turning from the PI to station 11 + 00) with the transit still at the PC. Measure the intersection of this line of sight and the 100-foot arc from the preceding station to locate station 11 + 00. Lay out the rest of the curve in this manner.

If the curve is long so that the transit must be moved or if an obstruction prevents a clear line of sight, move the transit to an intermediate station. The same deflection angles previously calculated may be used to locate the rest of the curve. (See Figure 9-17.)

If station 17 + 00 cannot be sighted, move the transit to station 16 + 00. Set zero on the vernier, backsight the PC, and turn the angle 21°31'6" to sight station 17 + 00. Figure 9-17 shows that angle D must be turned for the transit at station 16 + 00 to become tangent to the curve at that point. Once the transit is tangent to the curve, angle D/2 must be turned to locate the next station because the arc is 100 feet long. The total angle turned is d_7 + D/2, which is d_8 as originally calculated.

Frequency of Placing Survey Stakes (In Feet)

Horizontal curves should be staked at a minimum interval of 100 feet. The staking interval on horizontal curves should be based on the degree of curvature and can be determined from the following table:

Degree of Curvature (100')	Radius (meters)	Staking interval
0 to 3°	> 1,910'	100'
> 3° to 8°	1,910' to 721'	50'
> 8° to 16°	720' to 360'	25'
> 16"	< 360°	10'

Horizontal Curve Design Using Metric Units

The design of horizontal curves using metric units is essentially the same as in English units. The only difference lies in the relationships of arc length to the degree of curvature as shown in Figures 9-18a, 9-18b, and 9-18c.

NOTE: The functions of a 1° curve table are also applicable to metric design based on the relationship shown in Figure 9-18b. However, if designing using metric units, the lengths of T, E, M, and R are in meters.

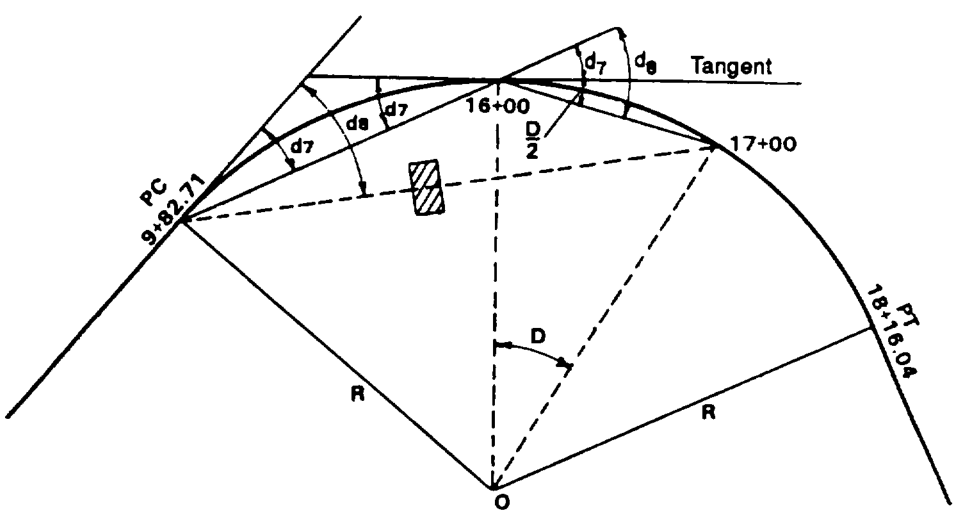
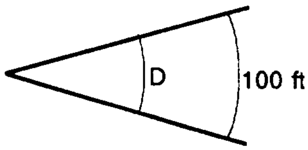


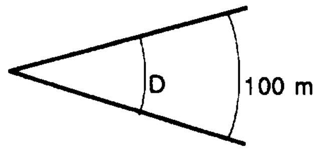
Figure 9-17. Obstruction on a curve



$$\frac{D_{100\,ft}}{100\,ft} = \frac{360}{2\pi R}$$

$$D_{100\,ft} = \frac{5729.58}{R(ft)}$$

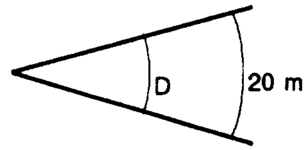
Figure 9-18a. D based on a 100-ft arc



$$\frac{D_{100\,m}}{100\,m} = \frac{360}{2\pi R}$$

$$D_{100\,m} = \frac{5729.58}{R(m)}$$

Figure 9-18b. D based on a 100-m arc



$$\frac{D_{20\,m}}{20\,m} = \frac{360}{2\pi R}$$

$$D_{20\,m} = \frac{5729.58}{R(m)}$$

Figure 9-18c. D based on a 20-m arc

If you design the curve based on D_{100} m, but intend on staking at an interval of 20 m, you must determine the degree of curvature based on 20 m ($D_{20\,m}$) to determine the correct deflection angles. A summary of the deflection-angle and chord-length calculations based on $D_{20\,m}$ is shown in Figure 9-18d.

Frequency of Placing Survey Stakes (In Meters)

Horizontal curves should be staked at a maximum interval of 20 meters (m). The staking internal on horizontal curves should be based on the degree of curvature and can be determined from the following information:

Degree of Curvature (100')	Radius (meters)	Cord Lengths (meters)
0 to 3°	>585	20
>3° to 8°	585 to 221	10
>8° to 16°	220 to 110	5
>16°	<110	5

LOCATION ON ARC	DEFLECTION ANGLE	CHORD LENGTH
PC to first full station	$D_1 = \frac{D_{20\,m}}{2} \times \frac{\ell}{20\,m}$	$C_1 = 2R\,Sin\left(\frac{D_{20\,m}}{2} \times \frac{\ell}{100}\right)$
Full station to full station	$d_n = d_{n-1} + \frac{D_{20\,m}}{2}$	$C_n = 2R\,Sin\left(\frac{D_{20\,m}}{2}\right)$
Last full station to PT	$d_l = d_{l-1} + \left(\frac{D_{20\,m}}{2} \times \frac{\ell}{20\,m}\right)$	$C_l = 2R\,Sin\left(\frac{D_{20\,m}}{2} \times \frac{\ell}{100}\right)$

NOTE: ℓ is the distance from the PC or PT to the closest full station.

Figure 9-18d. Deflection-angle and chord length selection based on $D_{20\,m}$

VERTICAL ALIGNMENT

The capabilities of vehicles or aircraft using any particular road or airfield determine the maximum allowable grades that should be established. However, other factors may be considered. Excessive grades can be installed where speed and capacity are not essential. Whenever possible, grades should be less than the prescribed maximum values stated in Table 9-1, page 9-3.

Within limitations imposed by various other criteria, place tangent grade lines so that earthwork is minimized. The earthwork required in most road-construction projects is usually the largest, single work item. Anything that reduces earthwork will improve job efficiency and economy. Attempt to balance the earthwork operations between cut and fill in any area, within the capabilities of available equipment. When drawing the grade lines, the engineer can usually do this balancing by inspection, keeping the profile area of cut equal to the profile area of fill. These areas are not necessarily proportional to the actual volumes involved, but they serve as a basis for comparison. It is impractical to baIance a volume of cut with an equivalent volume of fill at a distance beyond the hauling capabilities of the available equipment.

Along any proposed route will be points at which the elevation is already fixed. Intersections with existing roads and railroad crossings present predetermined elevations that the engineer must meet when locating the tangent grade lines.

In addition to the controlling specifications for grades, other criteria may control the placement of grade lines. These criteria include the minimum allowable gradients, the maximum allowable change in grade at any Intersection point, the permissible depth of cut or fill, and the maximum gradients in approaching bridges or points of intersection.

PLOTTING A PROFILE VIEW

The profile of a road or airfield is a side view of the project. It represents the horizontal distance, or stations, as abscissa (x axis) against the elevations at these stations, which are plotted as ordinates (y axis). When a horizontal alignment is set and the project stationed, determine the elevation of critical points along the centerline. Engineers usually calculate the elevation at all half and full stations, PVC, PVT, and HP and LP elevations.

A break point where the prevailing grade makes an appreciable change should be stationed and the elevation ascertained. The most common procedure for determining existing terrain elevations is by a ground survey. However, it is possible to obtain elevations for specific points from a contour map on which the proposed horizontal alignment has been plotted. Unless the scale of the contour map is large, this method is inaccurate and should be used only for preliminary planning and initial location. The centerline profile may not represent the typical or prevailing condition across the entire section at any particular point. This error may be noticeable when the section is wide, as for an airfield. In such cases, additional profiles may be needed along the shoulder line. It is also possible to make a typical profile that represents the average elevation across the entire section.

PLOTTING TRIAL GRADE LINES

After studying the profile, determine the tangent grade lines. These grade lines serve as the proposed final profile of the project. It is possible for rough, pioneer construction to follow existing contours with a little smoothing of rough spots. Such a route provides a rough and relatively unsafe roadbed that is not capable of carrying a large volume of traffic. A well-designed route has a series of tangent grades with a smooth transition between them. These tangent grade lines can be determined with a good profile.

GRADE DETERMINATION

The degree of steepness measured longitudinally is normally defined as the percentage of grade. It is established as a relationship between vertical rise or fall for each 100-foot horizontal distance and is expressed as a percentage (per 100).

The equation for determining grades is—

$$G = \frac{V}{H}(100)$$

where—

G = percentage of grade
V = rise or fall between the two points
H = horizontal distance between the two points

NOTE: V and H must be in the same units.

To differentiate between rising and falling grades along the centerline, a plus sign is used to denote rising grades in the direction of increasing stations and a minus sign is used 10 denote falling grades.

VERTICAL CURVES

After grade lines are placed, define the route vertically in a series of grade lines (straight segments of constant grade) between points of vertical intersection. Design a transition that provides smooth, easy movement from one grade line to another at these intersections. The vertical curves used for this transition and its pertinent dimensions are easily calculated.

Types of Vertical Curves

Two types of vertical curves must be considered: overt and invert. (See Figure 9-19.) Overt curves are commonly called crest curves, and invert curves are referred to as sag curves. Both types are designed the same way but different specifications govern their dimensions.

Elements of Vertical Curves

Figure 9-20 shows a typical vertical curve installed between two grade lines. The parts of a vertical curve include the following:

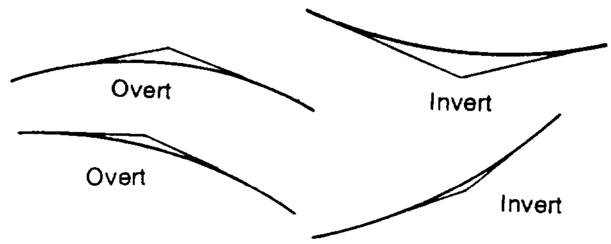


Figure 9-19. Types of vertical curves

- The PVI is the intersection of two grade lines. This station is always read from the profile view.
- The PVC is the point along the first grade line at which the vertical curve begins. The grade line is tangent to the parabolic curve at this point. By convention. the PVC is always one-half the length of the vertical curve from the PVI, measured horizontally.
- The PVT is the point along the second grade line at which the vertical curve ends. It has the same properties as the PVC.
- The percentage of grade (G) on the grade line nearest the point of origin is called G1, and the other grade line percentage is called G2. These two grade lines, which are tangent to the parabolic curve at the PVC and PVT, intersect at the PVI.

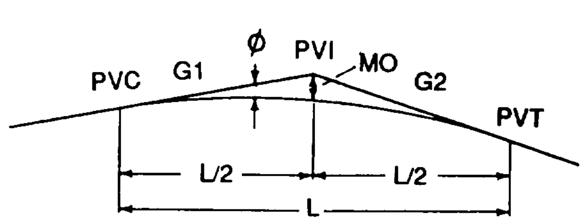


Figure 9-20. Elements of a vertical curve

- The length of vertical curve (L) is the horizontal distance from the PVC to the PVT. The walking distance along the actual curve has no significance. The PVI is horizontally midway between the PVC and PVT. Therefore, the distance from the PVC to the PVI is L/2, and the distance from the PVI to the PVT is also L/2.

- Offsets (Ø) are the vertical distances from the grade lines to the vertical curve. The heights of offsets are computed for selected points along the length of the vertical curve. The selected points are usually at every station and half station.

- The maximum offset (MO) is the offset at the PVI. It is always the greatest offset along the vertical curve.

Design of Vertical Curves

The design of vertical curves includes two tasks-determining the curve length and calculating the heights of a sufficient number of offsets to adequately define or locate the final grade line.

Length Determination. It is possible to design vertical curves to be long and gentle (flat) or short and abrupt. This is done by varying the curve length. Depending on the facility to be constructed and the standards of construction desired, there are certain limitations on curve length. Minimum lengths usually are specified.

Change of Grade (ΔG). The difference in grade between the two grade lines is called the change of grade. This difference, which Is symbolized as ΔG, is computed as $|G_1 - G_2|$ which represents the absolute value of $G_1 - G_2$. The curve installed between two grade lines with a large ΔG which might occur at the top of a steep hill, is longer than the curve required between two grades with a smaller ΔG.

Allowable Rate of Change of Grade [r). Criteria have been established to ensure that the rate at which the change in gradient is made throughout a vertical curve is consistent with the operating characteristics of the vehicles or aircraft using the facility. One criterion (called "r") is expressed as an allowable rate of change of grade for a specific horizontal distance. For example, with a criterion of 0.5 percent change in grade over 500 feet, a ΔG of 1.0 percent requires a curve length of 1,000 feet. However, r is usually expressed as the allowable gradient change in 100 feet of length. The term "r" is frequently used for airfield vertical curve design.

Sight Distance (S). When an overt curve is traversed, the ability of the driver to see down the road or airfield is curtailed. If a vertical curve is quite short, the distance that can be seen ahead becomes critically short. Reduced speed is required to reduce the safety hazard. Sight distance depends upon the design speed permitted.

Vertical-Curve-Length Factor (k). This factor is used when determining road vertical curve lengths. It is equal to the horizontal distance, in feet, required to effect a 1-percent change in gradient while providing the minimum stopping distance.

Determination of the Vertical Curve Length. Determine the vertical curve length by using the vertical-curve-length factor (see Table 9-1, page 9-3) for the given class of road (A, B, C, or D) and for the type of curve. The factor "k" is used in the following equation:

$$L = \frac{k\Delta G}{100}$$

where—

L = length of vertical curve in 100-foot stations
k = vertical-curve-length factor (Table 9-1)
ΔG = change of grade

If length L, as computed, is not in whole stations, round up to the next full 100-foot station. The length derived by this procedure is compared to the absolute minimum length found in Table 9-1.

Knowing the curve length and the station of the PVI, compute the station of the PVC and the PVT.

PVC = PVI - L/2
PVT = PVI + L/2

Offset Determinations. For the curve to be defined, the engineer must determine elevations at various locations along the curve. In order to do so, the engineer must determine the offset (Ø) which is the vertical distance from the original grade line to the designed curve.

Maximum Offset. As previously stated, the MO will always be located at the PVI. The following formula is used to calculate MO:

$$MO = \frac{L\Delta G}{8}$$

where—

MO = vertical height of the maximum offset in feet
L = length of vertical curve in sections
ΔG = change of grade in percent

Intermediate Offsets. Offsets at locations along the curve other than the PVI are referred to as intermediate offsets. Since it is common practice to stake vertical curves at whole and half stations, intermediate offsets are determined at every whole and half station along the curve. Use the following formula to calculate the intermediate offsets:

$$\varnothing = \frac{\Delta G}{2L} d^2$$

where—

Ø = offset at a distance, d, from the PVC or PVT in feet
d = selected distance (in stations) from the PVC or PVT at which an offset distance is to be calculated
L = curve length in stations
ΔG = change of grade in percent

Since ΔG and L have been determined prior to using this equation, the term $\Delta G/2L$ is a constant. Finding the offsets is a simple matter of varying the value of d.

NOTE: The offsets at equal distances from the PVC or PVT are equal. In other words, the offsets are symmetric about the PVI. (This does not mean that the resulting curve is symmetric (unless $G_1 = G_2$).) Thus, the offset for only one side of a vertical curve needs to be calculated. Therefore, the above equation can be used to calculate the offset at any point within the vertical curve.

Elevations Along Vertical Curves. Vertical curves have the shape of a parabola. Use the following equation to calculate elevations along the vertical curve:

Invert curve:

y = elev PVC - change in elevation + offset

$$y = elev\ PVC - G_1 d + \frac{\Delta G}{2L} d^2$$

y = elev PVT ± change in elevation + offset

$$y = elev\ PVT \pm G_2 d + \frac{\Delta G}{2L} d^2$$

Overt curve:

Y = elev PVC + change in elevation - offset

$$y = elev\ PVC + G_1 - \frac{\Delta G}{2L} d^2$$

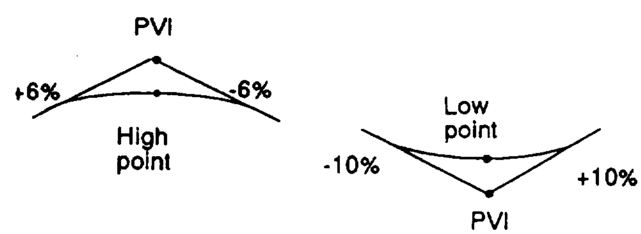
y = elev PVT ± change in elevation - offset

$$y = elev\ PVT \pm G_2 d - \frac{\Delta G}{2L} d^2$$

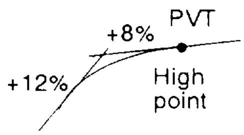
where—

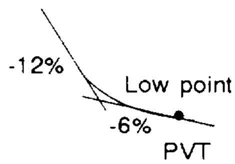
y = elevation of point on curve in feet
d = horizontal distance of point on curve from PVC or PVT in stations
ΔG = change of grade in percent
G_1 = percent slope of first grade line
G_2 = percent slope of second grade line
Elev PVC = elevation at point of vertical curve in feet

High or Low Point of a Vertical Curve. When the tangent grades (G_1 and G_2) are equal, the high or low point of the curve occurs at the PVI. (See the following example:)

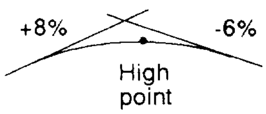


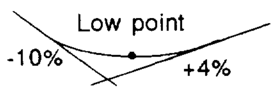
When the tangent grades are the same sign (both positive or both negative), the high and low points correspond with either the PVC or PVT. (See the following example:)





When the tangent grades are unequal, the high or low point of the curve always falls on the flatter of the two grades. (See the following example:)





To determine the location along the curve of the maximum (or minimum) elevation, use the following equation:

$$d = \frac{GL}{\Delta G}$$

where—

d = horizontal distance along curve from PVC (or PVT) in stations
L = length of vertical curve in stations
G = percent slope of flattest grade
ΔG = change of grade in percent

Cut or Fill Values. By using the calculated curve elevations and the existing ground elevations, we can determine the cut or fill values to place on the grade stakes for construction operations. The ground elevations can be determined from the profile view established from the initial survey. The difference in elevation between the curve and the ground elevation constitutes the cut or fill value. For example, if the ground elevation at a point on a curve was 86 feet and the curve elevation at that point was 82 feet, the construction stake would indicate a cut of 4 feet (86 feet - 82 feet).

Design Steps.

The following steps show the design procedure for vertical curve:

1. Compute the change of grade.

$$\Delta G = |G_1 - G_2|$$

2. Compute the vertical curve length (L).

$$L = \frac{k\Delta G}{100}$$

Round up to the next higher full station (if possible).

3. Determine the PVC.

$$PVC = PVI - (L/2)$$

4. Determine the PVT.

$$PVT = PVI + (L/2)$$

5. Determine the elevation of PVC.

$$EL_{PVC} = EL_{PVI} \pm |G_1| \times (L/2)$$

6. Determine the elevation of PVT.

$$EL_{PVT} = EL_{PVI} \pm |G_2| \times (L/2)$$

7. Determine the maximum offset (MO).

$$MO = \frac{L\Delta G}{8}$$

8. Determine final curve elevations. Compute at every half station and full station along the curve.

a. Determine grade-line elevations (GLEs).

$$GLE = EL_{PVC} \pm |G_1| \times d \text{ from PVC}$$
$$GLE = EL_{PVT} \pm |G_2| \times d \text{ from PVT}$$

b. Determine intermediate offsets ($\varnothing$).

$$\varnothing = MO \times \left(\frac{d}{L/2}\right)^2 \text{ or } \left(\frac{\Delta G}{2L}\right) d^2$$

c. Determine curve elevations.

$$\text{Elev curve} = GLE \pm \varnothing$$

9. Determine the location of maximum (or minimum) elevation, if required.

$$d = \frac{GL}{\Delta G}$$

10. Determine the highest (lowest) elevation, if required.

$$Y = \frac{-\Delta G x^2}{2L} + G_1 x + elev\ PVC$$

Example:

Complete the design of a vertical curve to include PVC, PVT, and offsets, with cuts and fills determined every 50 feet (one-half station).

Given:

The unit survey section has completed a centerline survey for a proposed vertical curve. The road is class D.

Solution:

PVI = 5 + 00, elevation = 73.00'

G_1 = +3.1%, G_2 = -5.75%

(1) Determine ΔG.

$$\Delta G = |G_1 - G_2| = |(+3.1) - (-5.75)| = 8.85\%$$

(2) Determine L.

From Table 9-1, page 9-3, k for crest vertical curve on a class-D road is 35.

$$L = \frac{k\Delta G}{100} = \frac{35(8.85)}{100} = 3.10 \text{ stations}$$

Use L = 4, stations = 400'

(3) Determine the PVC.

$$PVC = PVI - L/2 = (5 + 00) - (2 + 00) = (3 + 00)$$

(4) Determine the PVT.

$$PVT = PVI + L/2 = (5 + 00) + (2 + 00) = (7 + 00)$$

(5) Determine the elevation of PVC.

$$EL_{PVC} = EL_{PVI} \pm |G_1|\ (L/2) = 73.00 - |0.031|\ (400/2) \text{ or } 73 - (3.1)(4/2) = 66.80'$$

(6) Determine the elevation of PVT.

$$EL_{PVT} = EL_{PVI} \pm |G_2|\ (L/2) = 73.00 - |-0.0575|\ (400/2) \text{ or } 73 - 5.75\ (4/2) = 61.50'$$

(7) Determine the maximum offset (MO).

$$MO = \frac{L\Delta G}{8} = \frac{4(8.85)}{8} = 4.43'$$

(8) Determine final curve elevations.

(a) Determine GLEs (Figure 9-21).

$$GLE = EL_{PVC} \pm |G_1|\ x\ d \text{ from } PVC$$

$$GLE = EL_{PVT} \pm |G_2|\ x\ d \text{ from } PVT$$

(b) Determine intermediate offsets (Ø)

$$Ø = \left(\frac{\Delta G}{2L}\right)d^2 = \left(\frac{8.85}{2(4)}\right)d^2 = 1.106\,d^2$$

NOTE: The term ΔG/2L becomes a constant (1.106).

Station	GLE	d	Ø	Curve elevation
3 + 00	66.80	0	0	66.80
3 + 50	68.35	0.5	0.28	68.07
4 + 00	69.90	1.0	1.11	68.79
4 + 50	71.45	1.5	2.49	68.96
5 + 00	73.00	2.0	4.43	68.57
5 + 50	70.13	1.5	2.49	67.64
6 + 00	67.25	1.0	1.11	66.14
6 + 50	64.38	0.5	0.28	64.18
7 + 00	61.50	0	0	61.50

Figure 9-21. Compiling vertical curve design data

Where—

$\emptyset$ = offset from the GLE to the curve
d = distance, from the PVC or PVT in stations
$\emptyset_{(zero\ stations)}$ = $1.106(0)^2 = 0$ (There is no offset at the PVC and PVT.)
$\emptyset_{(0.5\ station)}$ = $1.106(0.5)^2 = 0.28'$ (at stations 3 + 50 and 6 + 50)
$\emptyset_{(1.0\ station)}$ = $1.106(1.0)^2 = 1.11'$ (at stations 4 + 00 and 6 + 00)
$\emptyset_{(1.5\ station)}$ = $1.106(1.5)^2 = 2.49'$ (at station 4 + 50 and 5 + 50)
$\emptyset_{(2.0\ station)}$ = $1.106(2.0)^2 = 4.43'$ (at station 5 + 00)

(9) Determine the location of the maximum or minimum elevation, if required.

$$D = \frac{GL}{\Delta G} = \frac{(3.1)(4)}{8.85} = 1.40 \text{ stations or } 140'$$

(The maximum or minimum elevation is always on the flattest grade when grades have opposite signs.)

Station maximum elevation point = PVC + d (or PVT - d)

Station maximum elevation point = (3+00)+(1+40)=4+40

(10) Determine the highest elevation, if required.

$$y = \text{elev PVC (or PVT)} + Gd \pm \emptyset$$

NOTE: + or - is determined by inspection.

$$y = PVC_{elev} + Gd - \left(\frac{\Delta G}{2L}\right)d^2$$

$$y = 66.80 + 3.1(1.4) - \frac{8.85}{8}(1.4)^2$$

$$y = 66.80 + 4.34 - 2.17 = 68.97$$

Vertical Curve Through a Known Point. When the vertical curve must go through a known point and the known point is at the PVI, the length of the curve can be determined using the following formula:

$$L = \frac{MO(8)}{\Delta G}$$

When the known point is at a location other than the PVI, the length of the curve can be determined using the following formula:

$$L = 2\left[A + \frac{2(\emptyset)}{\Delta G}\right] + 4\sqrt{\left(\frac{A(\emptyset)}{\Delta G}\right) + \left(\frac{\emptyset}{\Delta G}\right)^2}$$

Where—

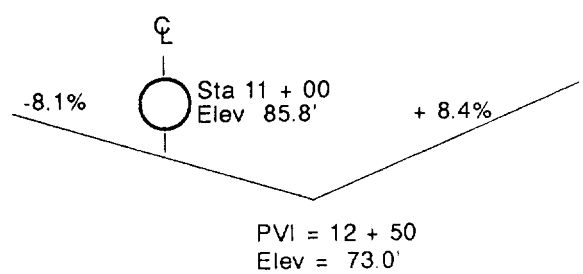
A = horizontal distance from PVI to known point
$\emptyset$ = offset between elevation of known point and GLE of known point

NOTE: ΔG must be entered as a decimal.

Once the length of the curve has been determined, the remainder of the design must be completed according to the vertical curve design procedures previously outlined.

Example:

A new road with a PVI at station 12 + 50 and an elevation at 73.00 feet is to pass over a 24-inch culvert at station 11 + 00. The invert of the culvert is at elevation 85.8 feet.



Solution:

Determine the length of the vertical curve required to clear this culvert with 1 foot of cover.

1. Determine the horizontal distance (A) from the PVI to the known point.

A = 1,250' - 1,100' = 150'

2. Determine ΔG (as a decimal).

$\Delta G = |G_1 - G_2| = |-0.081 - 0.084|$
$\Delta G = 0.165$

3. Determine the elevation of the vertical curve required to clear the culvert with 1 foot of cover (at the known point).

EL_{CURVE} = 85.8' + 2.0' (pipe diameter) + 1.0' (cover required)

EL_{CURVE} = 88.8'

4. Determine the GLE at the known point (directly below the culvert).

GLE = EL_{PVI} + $|G_1|$ x d (from PVI to known point)

GLE = 73.0' + |0.081| x 150'

GLE = 85.15'

5. Determine the offset (Ø) between the elevation of known point and the GLE of known point.

Ø = 88.8'- 85.15' = 3,65'

6. Determine the length of the vertical curve.

$$L = 2\left[A + \frac{2(\varnothing)}{\Delta G}\right] + 4\sqrt{\left(\frac{A(\varnothing)}{\Delta G}\right) + \left(\frac{\varnothing}{\Delta G}\right)^2}$$

$$L = 2\left[150' + \frac{2(3.65')}{0.165}\right] + 4\sqrt{\left(\frac{150(3.65)}{0.165}\right) + \left(\frac{3.65}{0.165}\right)^2}$$

= 388.48' + 246,82'
= 635.30'

Vertical-Curve Design Using Metric Units. The design of vertical curves using metric units is the same as in English units. The only difference lies in the use of meters as units of measurement for elevations and distances.

Frequency of Placing Survey Stakes. Vertical curves should normally be staked at 50-foot (every half station) or 10-meter intervals. On extremely rugged terrain, the interval should be reduced.

THE CROWNED SECTION

The typical cross section of a road is the crowned cross section. The amount of crown provided depends on the type of surface used. Normal crown slopes are provided in Table 9-1, page 9-3. Figure 9-22 shows a typical cross section for a class-A road. If the road is to be surfaced, the subgrade and the finished surface till have the same crown.

SUPERELEVATION

The outer edge of a road is elevated to balance the overturning forces experienced by a vehicle rounding a horizontal curve. The amount of superelevation is governed by the degree of curvature (or the curve radius) and the design speed. Detailed information for designing superelevation on

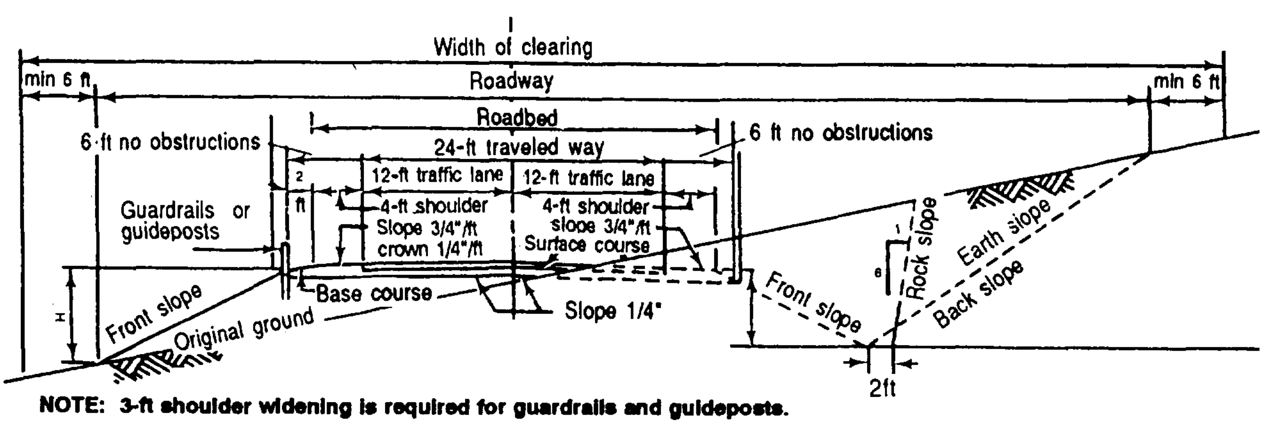


Figure 9-22. Normal crown cross section for a class-A road

curves is in FM 5-233. Table 9-3 lists superelevation rates and appropriate transition lengths to develop the superelevated section as a function of the design speed and degree of curvature. Figure 9-23 shows a class-A road section with a superelevated curve.

As a safely factor, the pavement width on the inside lane of a curve is increased. The amount of increase is governed by the degree of curvature and the design speed.

Table 9-4 lists pavement widening requirements. As shown in Figure 9-24, the transition from a normal cross section on a tangent (A-A) to a fully superelevated, widened cross section on a curve (D-D) is a uniform, gradual change. The length of highway needed to accomplish this transition is given in Table 9-3. Two-thirds of the specified transition length is affected on the tangent and one-third is affected on the curve.

Table 9-3. Superelevation lengths and transition lengths

D	R		V = 30 mph (48 kph)	L		V = 40 mph (64 kph)	L		V = 60 mph (97 kph)	L	
	ft	m	E	ft	m	E	ft	m	E	ft	m
0° 15'	22918	6985.420	NC	0	0	NC	0	0	NC	0	0
0° 30'	11459	3492.710	NC	0	0	NC	0	0	RC	175	53.340
0° 45'	7639	2328.372	NC	0	0	NC	0	0	.024	175	53.340
1° 00'	5730	1746.507	NC	0	0	RC	125	38.100	.032	175	53.340
1° 30'	3820	1164.338	RC	100	30.480	.021	125	38.100	.048	175	53.340
2° 00'	2865	873.254	RC	100	30.480	.028	125	38.100	.058	175	53.340
2° 30'	2292	698.603	.021	100	30.480	.034	125	38.100	.069	190	57.912
3° 00'	1910	582.169	.025	100	30.480	.040	125	38.100	.079	210	64.008
3° 30'	1637	498.959	.029	100	30.480	.046	125	38.100	.087	230	70.104
4° 00'	1432	436.474	.033	100	30.480	.051	125	38.100	.093	250	76.200
5° 00'	1146	349.301	.040	100	30.480	.061	130	39.524	.099	270	82.296
6° 00'	955	291.085	.046	100	30.480	.070	160	45.720	.100	270	82.296
7° 00'	819	249.832	.053	100	30.480	.077	160	48.768	D MAX = 5.5°		
8° 00'	716	218.237	.059	110	33.528	.084	180	54.864			
9° 00'	637	194.158	.064	120	36.576	.089	190	57.912			
10° 00'	573	174.651	.068	120	36.576	.093	200	60.960			
11° 00'	521	158.801	.073	130	39.624	.097	200	60.960			
12° 00'	477	145.390	.077	140	42.672	.099	210	64.008			
13° 00'	441	134.417	.080	140	42.672	.100	210	64.008			
14° 00'	409	124.663	.083	150	45.720	.100	210	64.008			
16° 00'	358	109.119	.089	160	48.768	.100	210	64.008			
18° 00'	318	96.927	.093	170	51.816	D MAX = 14.5°					
20° 00'	286	87.173	.097	170	51.816						
22° 00'	260	79.248	.099	180	54.864						
24° 00'	239	72.847	.100	180	54.864						
28° 00'	205	62.484	.100	180	54.864						
			D MAX = 26.7°								

D - Degree of curvature
R - Radius of curvature
V - Design speed
E - Rate of superelevation
L - Minimum length of transition
NC - Normal crown section
RC - Remove adverse crown, superelevate at normal crown slope
Transitions desirable but not as essential above heavy line

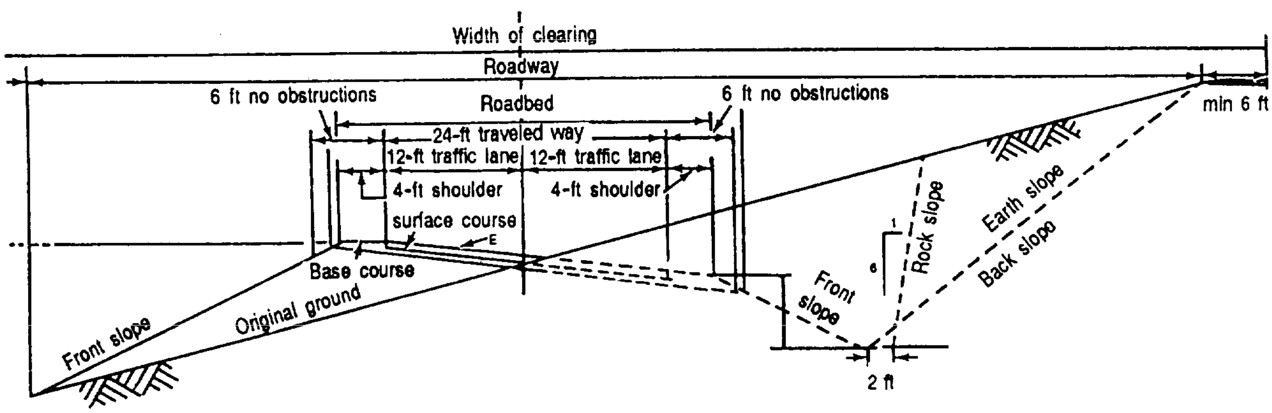


Figure 9-23. Superelevated cross section

Table 9-4. Pavement widening

	Widening for 2-lane pavements on curves for width of pavement on tangent of 20 ft		
Degree of Curve	Design Speed ·30 mph	40 mph	50 mph
1	0.0'	0.0'	2.0'
2	2.0'	2.0'	2.5'
3	2.0'	2.0'	2.5'
4	2.0'	2.5'	3.0'
5	2.5'	2.5'	3.0'
6	2.5'	3.0'	3.5'
7	2.5'	3.0'	
8	3.0'	3.0'	
9	3.0'	3.5'	
10-11	3.0'	3.5'	
12-14.5	3.5'	4.0'	
15-18	4.0'		
19-21	4.5'		
22-25	5.0'		
26-26.7	5.5'		

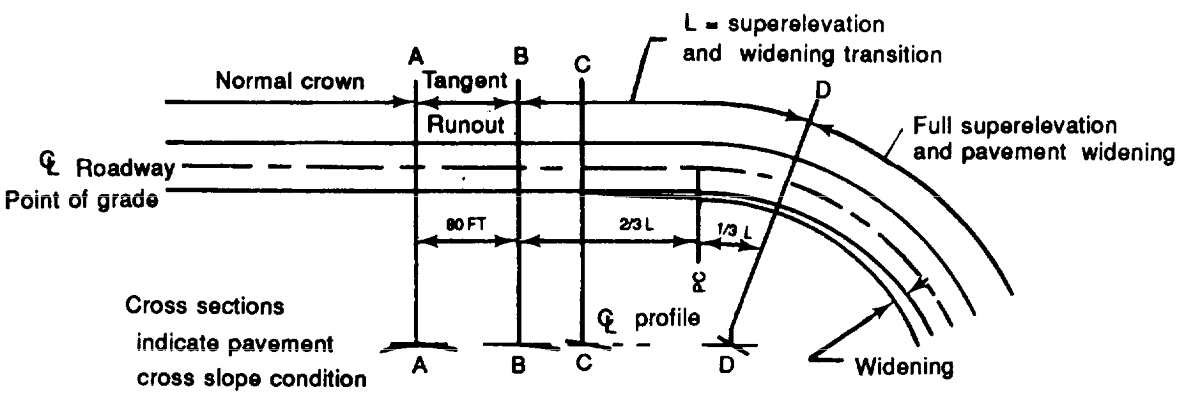


Figure 9-24. Method for transition (not to scale)

STRUCTURAL DESIGN

In the TO, few roads receive a bituminous or portland-cement concrete surface. Most two-lane roads are surfaced with sand, gravel, crushed rock, or the best locally available material. Expedient surfacing methods are used when required.

This section describes the procedures to improve natural earth surfaces and to resurface them with sand, gravel, or other materials. Included are some common methods of expedient surfacing, guidance, and criteria to determine thickness requirements for bituminous pavements in the TO. The design of mixes and aggregates and the procedures for placing bituminous and concrete surfaces are in TM 5-337. For subgrade and base-course requirements, refer to Chapter 5 of this manual. Additional information in Chapter 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2, supplements the frost-design procedures in this chapter.

Road surfaces in the TO consist of earth in the most expedient circumstances. When in-place soil is not strong enough, it may be chemically or mechanically stabilized or covered with a bituminous surface treatment. As time becomes available, earthen roads can be improved for increased traffic loads by covering them with material from a borrow pit or with processed material.

EARTH

Earthen roads consist of native soils graded and drained to form a surface for carrying traffic. They are designed to satisfy immediate traffic needs and provide a subgrade for surfaces of better quality. Their use is generally limited to dry weather and light traffic. For continued use, periodic maintenance by graders and drags is necessary to maintain a high crown and smooth surface for draining surface water. Dust control must also be provided in dry climates or during dry weather.

Earthen roads become impassable in wet weather because of the rutting action of heavy traffic. Generally, they are used in combat areas where speed of construction is required with limited equipment and personnel. They are also used as haul roads in construction areas and as service roads for military installations.

TREATED SURFACE

Earthen roads may be treated with bituminous materials to control dust and to waterproof the surface. This helps prevent softening of the surface in wet weather. Treated surfaces are most successful with silt or clay soils. The bituminous material should be of low viscosity and should contain a wide range of volatile materials of light fractions. Slow-curing liquid asphalts are frequently used, particularly grades SC-70 and -250. Medium-curing cutback asphalts, grades MC-30, -70, and -250, have also been used successfully. Road tars have been used to some extent, especially RT-1.

Many residual oils from oil refineries have been used in this work. In only expedient situations, waste military oils (such as crankcase oils) can be used. The amount of oil ranges from about 1/2 to 1 gallon per square yard and is applied in two or three increments, depending on the type and condition of the oil. The serious environmental, ecological implications of these methods must be considered. Also, using these methods will greatly impair the ability of bituminous admixtures and surface applications to properly cure, if applied later.

STABILIZED SOIL

Bituminous, stabilized soil mixtures and soil-cement are used as road surfaces to carry light traffic in expedient situations for relatively brief periods. Mechanically stabilized soil mixtures are widely used as surfaces for military roads under favorable conditions. Requirements for mechanically stabilized surfaces are discussed below:

Gradation requirements for mechanically stabilized soils used directly as surfaces are shown in Table 9-5. Mixtures that have a maximum size of aggregate of 1 to 1 1/2 inches are preferred because the large partitles tend to work to the surface under traffic. Somewhat finer soil is desirable in a mixture that will serve as a surface compared with one used for a base. The finer soil makes the surface resistant to the abrasive effects of traffic and to the penetration of precipitation. Such a surface will

Table 9-5. Suggested grading requirements for gravel and composite-type surface course of processed materials

Sieve Designation	Percent Passing by Weight
1 in	100
3/4 in	85-100
3/8 in	65-100
No. 4	55-85
No. 10	40-70
No. 40	25-45
No. 200	10-25

also more easily replace (by capillary action) moisture that is lost by evaporation.

Road surfaces require an LL of 35 or less and a PI ranging between 4 and 9. For best results, the PI of a stabilized soil that will function first as a wearing surface and then as a base, with a bituminous surface to be provided later, should be 5 or less. The LL should be less than 25. Compaction, bearing value, and frost action are important considerations for surfaces of this type.

SAND CLAY

One type of mechanically stabilized soil surface is called a sand-clay road. It consists of a natural or artificial mixture of sand and clay that is graded and drained to form a road surface. Although difficult to obtain, the PI should be less than 5 and LL less than 25, in case this layer becomes a subbase after placing additional layers above the sand clay. The gradation requirements for a typical sand-clay surface are in Table 5-4, page 5-12, under the column for 1 -inch sand-clay. The addition of fine gravel (slightly larger than the No. 4 sieve) usually adds stability.

Sand-clay roads will carry light traffic reasonably well and heavy traffic except under bad weather conditions. The amount of moisture these roads absorb determines their stability under traffic loads. Dust control, blading, and dragging are needed. Sand-clay roads withstand traffic better than ordinary earthen roads, but their use is limited to areas where a suitable mixture of sand and clay occurs naturally or where a deficiency of either is readily corrected. As a base course for future surfacing, sand-clay roads produce poor results, unless the plasticity can be reduced by adding a chemical stabilization agent such as lime.

GRAVEL

Gravel roads consist of a compacted layer of gravelly soil that meets the plasticity requirements for mechanically stabilized soil mixtures. The gravel is graded from coarse to fine, with a maximum allowable size of 1 inch. Recommended gradation requirements for a gravel surface are given in Table 9-6. A natural pit- or bank-run gravel may meet these requirements without further processing other than screening. Some pit- or bank-run gravels may require both screening and washing to meet the requirements. River-run gravels normally require the addition of binder to the soil, as do mechanically stabilized soil mixtures. River-run gravels may also require crushing to provide a rough, angular surface rather than the natural, smooth surface characteristic of river-run materials. The ability to carry heavy, sustained traffic depends on the strength and hardness of the gravel, the cohesiveness of the clay binder, the thickness of the layer, and the stability of the subgrade. These roads can be built rapidly, even in cold weather. Organizational equipment of combat engineer units is readily adapted to hauling and placing a gravel surface.

Like other untreated surfaces, gravel roads require considerable maintenance such as blading and dust control in dry weather. During wet weather, proper maintenance is difficult, especially under heavy traffic. Gravel road surfaces with low plasticity make excellent base courses for later-stage pavements.

Table 9-6. Suggested grading requirements for coarse-graded type surface course of processed materials

Sieve Designation	Percent Passing by Weight
3/4 in	100
No. 4	70-100
No. 10	35-80
No. 40	25-50
No. 200	8-25

PROCESSED MATERIALS

Processed materials are prepared by crushing and screening rock, gravel, or slag. A composite-type surface material should meet the gradation requirements of Table 9-5, page 9-28. A coarse-graded type of surface material should meet the gradation requirements in Table 9-6, page 9-29. The information presented here about gravel roads generally applies to roads of processed materials. When gravel or sand-clay is available, processed materials should not be used except when their use will save time and effort.

EXPEDIENT-SURFACED ROADS

Several types of roads are considered expedient surfaced. These are unsurfaced roads and roads where some material has been placed on the natural soil to improve the roadway. Types of expedient-surfaced roads include corduroy, chespaling, landing mats, Army track, plank tread, wire mesh, snow and ice, and sand grid.

CORDUROY-SURFACED ROADS

A corduroy road is an expedient road which uses logs or small trees as the road surface (decking). This method of construction is used in extremely muddy terrain when there is a sufficient supply of natural material. There are three types of corduroy construction: standard corduroy, corduroy with stringers, and heavy corduroy.

Standard Corduroy

The most frequently used corduroy road, shown in Figures 9-25 and 9-26, is built of 6- to 8-inch diameter logs about 13 feet long. The logs are placed across the road surface adjacent to each other from butt to tip. Along the edges of the roadway, place 6- to 8-inch-diameter logs as curbs and attach them in place with driftpins. Drive pickets about 4 feet long into the ground at regular intervals along the outside edge of the road to hold the road in place. To give this surface greater smoothness, fill the gaps between logs with brush, rubble, or twigs. Cover the whole surface with a layer of gravel or dirt. Construct side ditches and culverts as for normal roads.

Corduroy with Stringers

A more substantial corduroy road is made by placing log stringers, as shown in Figure 9-27, parallel to the centerline on about 3-foot centers. Lay a standard corduroy over them, Securely pin the corduroy decking to the stringers, and prepare the surface as described in the preceding paragraph.

Heavy Corduroy

Sleepers (heavy logs 8 to 10 inches in diameter) are used for heavy corduroy roads. The sleepers must be long enough to span the entire road. Place the sleepers at right angles to the centerline on 4-foot centers. Build a corduroy with stringers, as shown in Figure 9-28, on top of the sleepers.

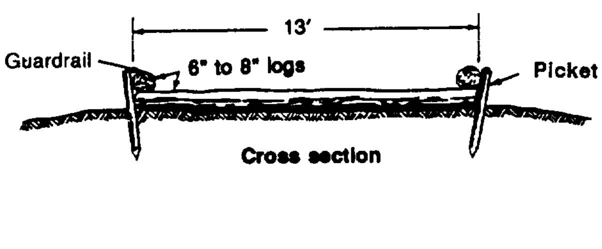


Figure 9-25. Standard corduroy

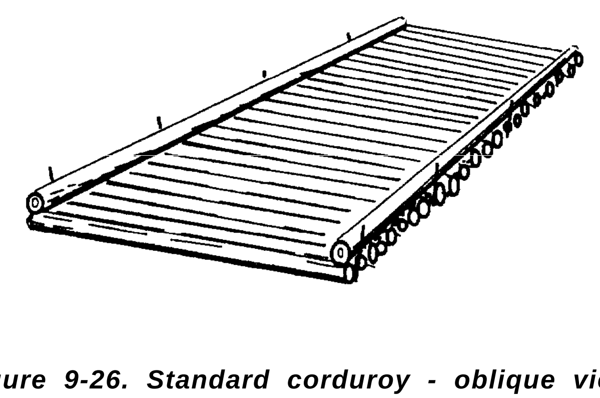


Figure 9-26. Standard corduroy - oblique view

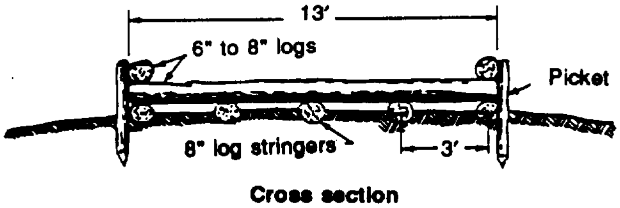


Figure 9-27. Corduroy with stringers

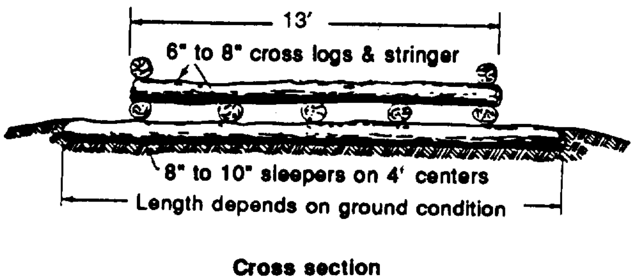


Figure 9-28. Heavy corduroy

Choice of Corduroy Type

Generally, softer ground requires a heavier type of corduroy. The stringers and sleepers do not increase the bearing capacity of the decking. They serve as a crib, keeping the road surface above the level of the surrounding mud. They sink into the ground until a stratum capable of supporting the load is reached. On fairly firm ground, the standard corduroy may be adequate; on softer ground, stringers are needed, Portable corduroy mats can be prefabricated and put down quickly when needed, They are made by wiring 4-inch-diameter logs together.

Diagonal corduroy is preferred for heavy traffic. It is made by placing the decking at an angle of 45 degrees to the centerline. This modified construction is applicable to all three corduroy types. The angled decking decreases the impact load because each log supports only one wheel at a time and there is longitudinal and lateral weight distribution.

CHESPALING

Chespaling is a hasty expedient used in either mud or sand. It is made from small, green saplings, preferably about 1 1/2 inches in diameter and 6 1/2 feet long. They are wired together to form a 12-foot-long mat as shown in Figure 9-29. Chespaling is often rolled into bundles and carried on each wheeled vehicle. The mats are used to

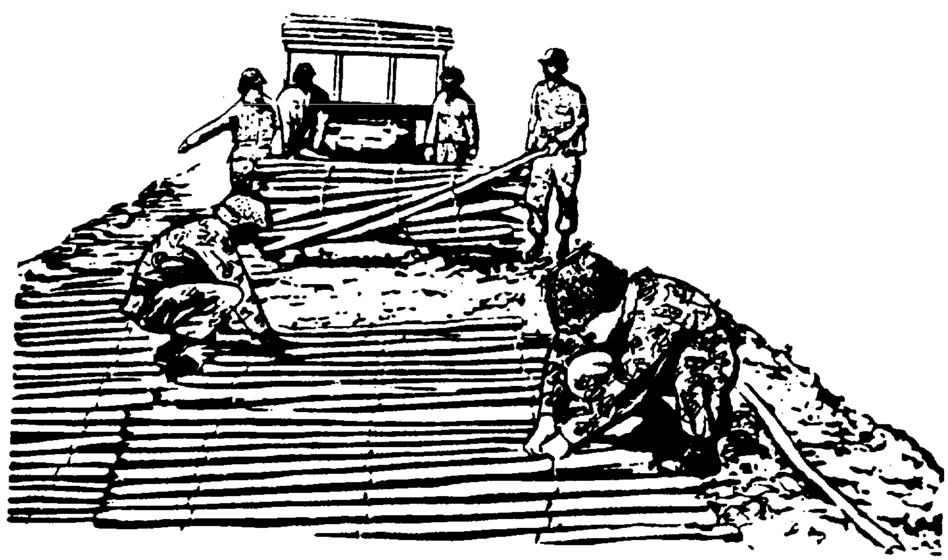


Figure 9-29. Chespaling

cross sandy terrain or to get out of mud. Some mats are constructed from dimensioned timbers wired together to resemble a picket fence. A variation slightly more effective for crossing sand is made by attaching chicken-wire netting to the bottom of the mats.

To build a chespaling road, lay a double row of mats, each mat having its long axis parallel to the centerline with a 1-foot overlay at the centerline. Wire the mats together. Keep the road wet to prevent the saplings from becoming brittle and breaking.

Bamboo mats are an excellent chespaling-type expedient for beach roadways. These mats are light and comparatively strong. They are made by splitting 2-inch bamboo rods and weaving them into a mat in a manner similar to rug weaving. Soak the rods before weaving, and keep the mats moist while they are in use. An 11- by 4-foot mat takes about 15 man-hours to construct. The mats are placed with the long dimension parallel to the centerline. The mats remain serviceable for three or four months on firm ground or sand. Bamboo mats can also be used over mud.

LANDING MATS

The demand for rapidly constructed airfields led to the development of several poriable, metal landing mats. When metal airfield landing mats became a standard supply item in the TO, they were quickly put to use on beaches as well as on airfields. They are still the foremost expedient for crossing sandy terrain. Landing-mat designs fabricated from aluminum alloys can support heavier loads. They also provide smoother surfaces and have a lower weight per square foot.

When used on sand, place the metal landing mats directly on the sand to the length and width desired. If pierced steel mats are used, place an impervious membrane under the mat to smooth and firm the subgrade, thus improving the road.

Place the mat so that its long axis is perpendicular to the flow of traffic. If a width greater than the effective length of one plank is required, use half sections to stagger the joints. A second layer of the steel mat, laid as a treadway over the initial layer, increases its effectiveness.

Landing mats tend to curl at the edges. This problem can be overcome by anchoring the edges properly. Screw-type earth anchors furnished with the mat sets provide the best means of anchoring. Another method of securing the edges is to use a curb of timber on the outside edge of the road and either wire it tightly to buried logs laid parallel to the road or stake it. One type of landing-mat surface is shown in Figure 9-30. If MO-MAT (a reinforced plastic material) is available, it may be used as a roadway surface for vehicular traffic.

ARMY TRACK

A portable timber expedient called Army track, shown in Figure 9-31, can be used to pass vehicles across sandy terrain. The track consists of 4- by 4-inch or larger timbers threaded at each end onto a 1/2-inch wire rope or a 3/4-inch hemp rope. The timbers resemble railroad ties, and a cable runs through them on each side. Space the timbers so that the smallest-wheeled vehicle using the road can obtain traction. Drill cable holes at a 45-degree angle to the centerline so the cable will bend. This practice will prevent individual timbers from moving together. Anchor the cables securely at both ends. Fill spaces between the timbers with select material to smooth out the surface.

PLANK-TREAD ROAD

The plank-tread road is shown in Figure 9-32, page 9-34. To construct a plank-tread road, first place sleepers 12 to 16 feet long, perpendicular to the centerline on 3- to 4-foot centers, depending on the loads to be carried and subgrade conditions. (If finished timber is not available, logs may be used as sleepers.) Then place 4- by 10-inch planks parallel to the line of traffic to

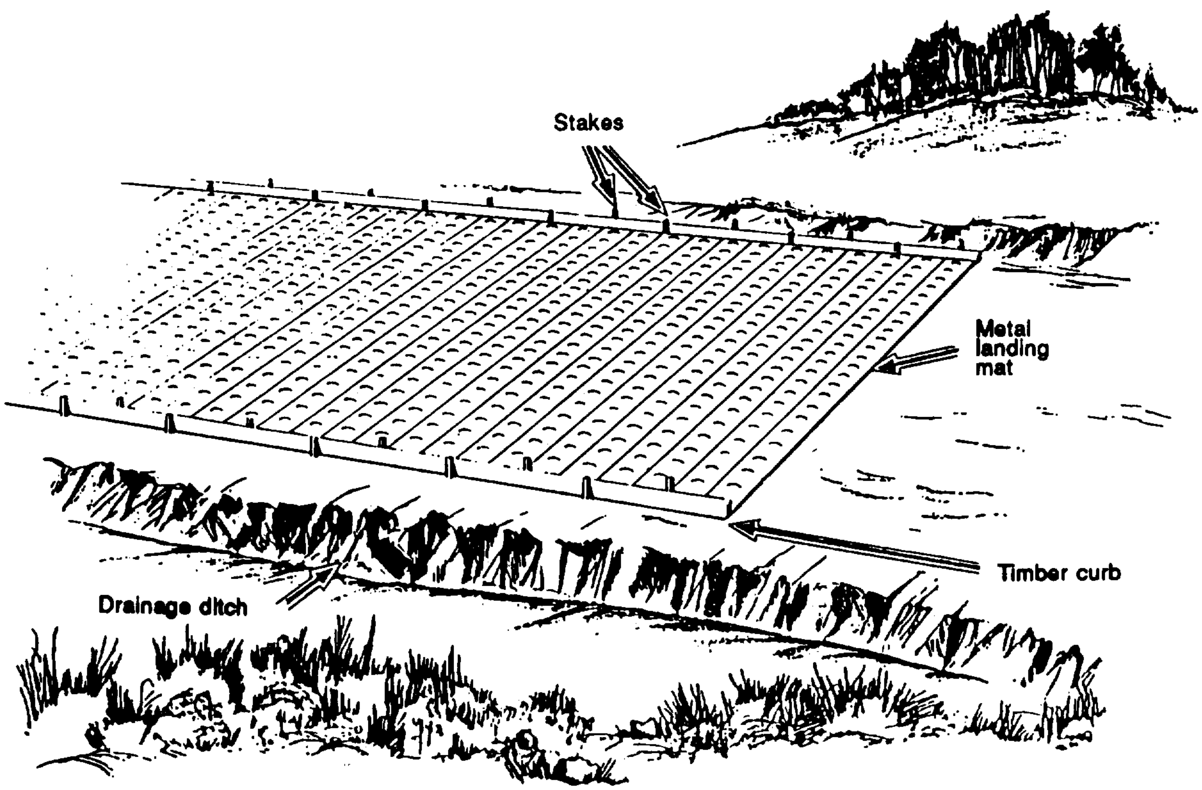


Figure 9-30. Landing-mat road

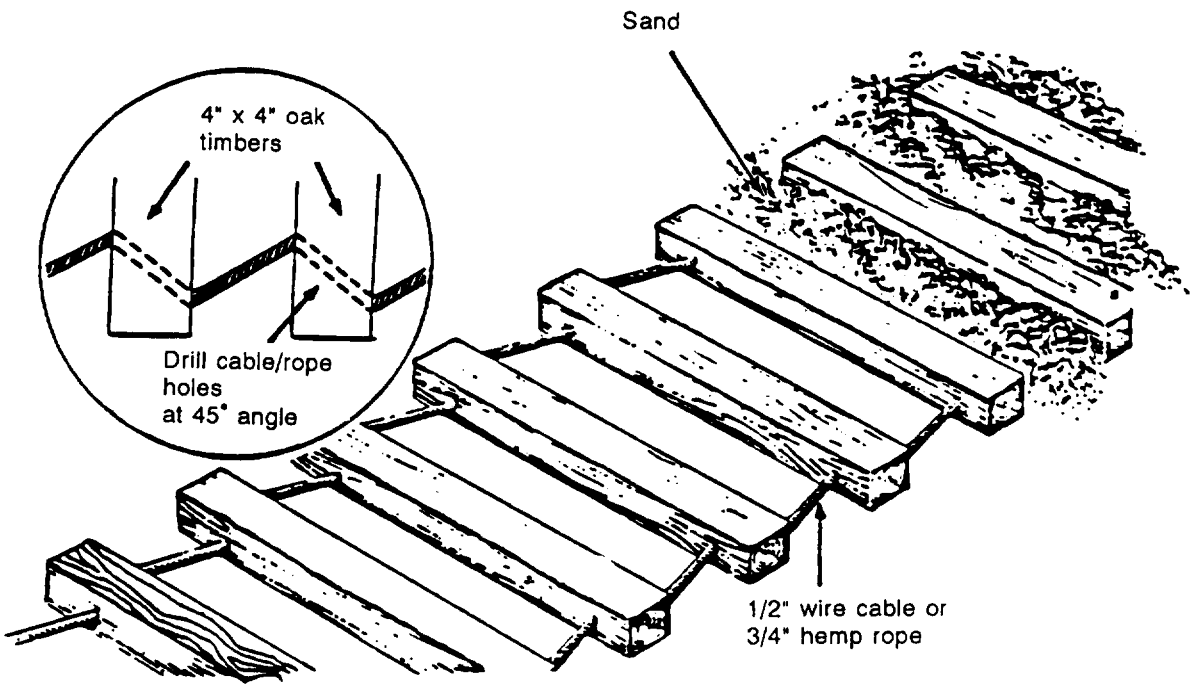


Figure 9-31. Army track

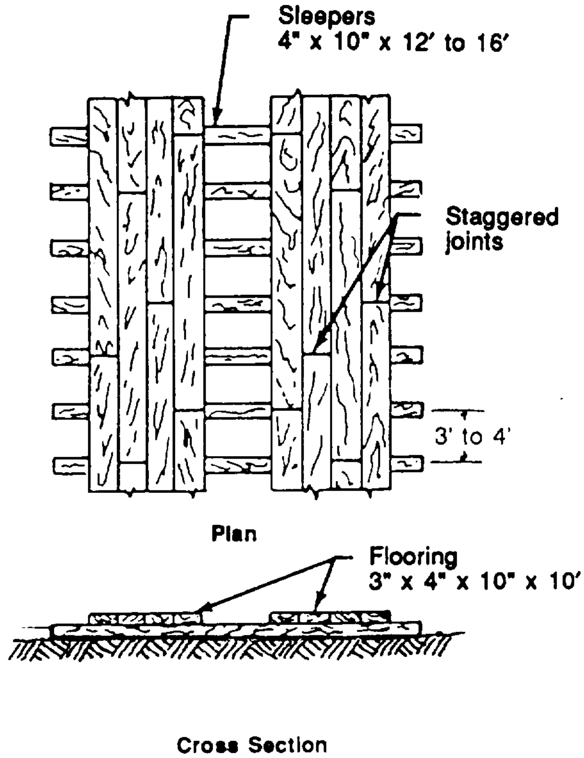


Figure 9-32. Plank-tread road

form two treads about 36 inches apart. Stagger the joints to prevent forming weak spots. If desired, 6-inch curbs may be installed on the inside of the treads.

Use plank roads for crossing short sections of loose sand or wet, soft ground. When built with an adequate base, plank roads last for several months. Planks 3 to 4 inches thick, 8 to 12 inches wide, and at least 13 feet long arc desirable for flooring, stringers, and sleepers. When desired, 3- by 10-inch planks (rough, not finished) can replace the 4- by 10-inch timbers shown in Figure 9-33. Rough 3- by 8-inch and 3- by 10-inch planks can be cut to order.

Position stringers in regular rows parallel to the centerline, on 3-foot centers, with staggered joints. Lay floor planks across the stringers with about 1-inch gaps when seasoned lumber is used. The gaps allow for swell when the lumber absorbs moisture. Spike the planks to every stringer.

Place 6-inch-deep guardrails on each side, with a 12-inch gap left between successive lengths of the guardrail for surface-water drainage. Place pickets along each side at 15-foot intervals to hold the roadway in line. Where necessary, use corduroy or other expedient cross sleepers spaced on

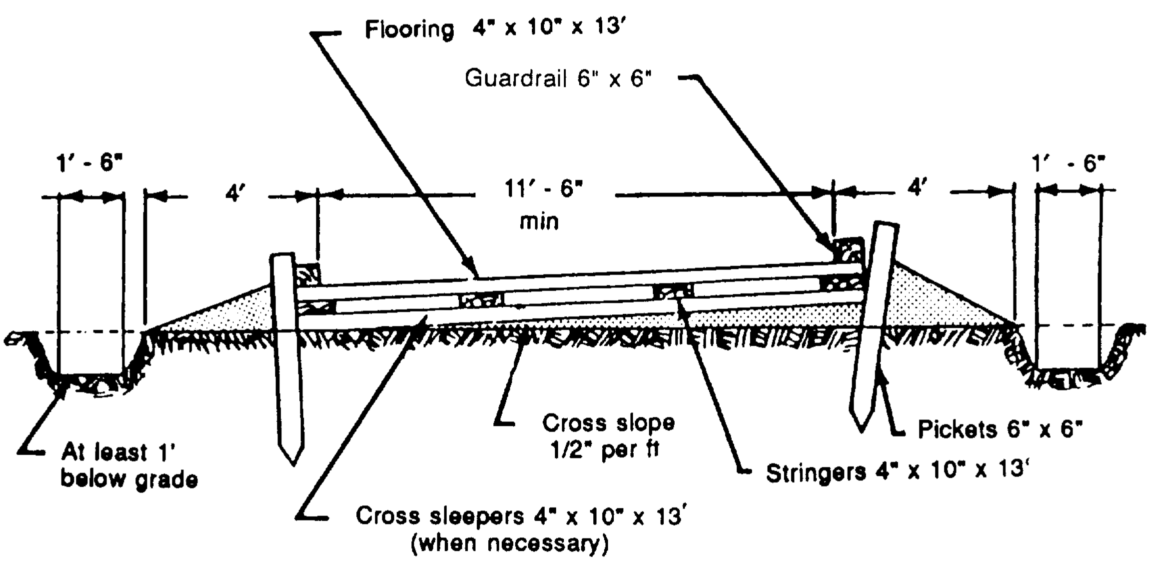


Figure 9-33. Construction details for a plank road

3- to 5-foot centers to hold the stringers in place and to gain depth for the structure.

For drainage, construct the base for a plank road with a transverse slope instead of a center crown. To provide a smoother-riding surface, place treads parallel to the line of traffic over the floor planks.

WIRE-MESH ROAD

Most wire-mesh surfaces are expedient measures. Applied directly to the subgrade, they provide passage for a limited number of vehicles for a short time. Longer life can be obtained by proper subgrade preparation, multilayer or sandwich construction, and frequent staking. Wire-mesh roads should never be crossed by other roads unless planking or some such material is placed over the mesh to protect it.

Chicken wire, expanded metal lath (used for plastering walls), and chain-link wire mesh may be used as road expedients in sand. Mesh surfaces should not be used on muddy roads because they prevent grading and reshaping of the surface when ruts appear.

Any wire-mesh surface is much more effective if a layer of burlap or similar membrane material is placed underneath it to help confine the sand. (See Figure 9-34.) Lighter forms of wire mesh, such as chicken wire or cyclone fencing, require an extra layer. Often a sandwich type of construction is used-one layer of wire mesh followed by one layer of burlap, then a second layer of wire mesh.

Wire mesh must be kept taut. Anchor the edges of a wire-mesh road at 3- to 4-foot intervals. Diagonal wires crossing the centerline at a 45-degree angle and attached securely to buried pickets reinforce the light mesh.

SNOW AND ICE ROADS

In regions with heavy snowfall and where temperatures are below freezing for extended periods, expedient roads can be constructed over the snow. When the road is

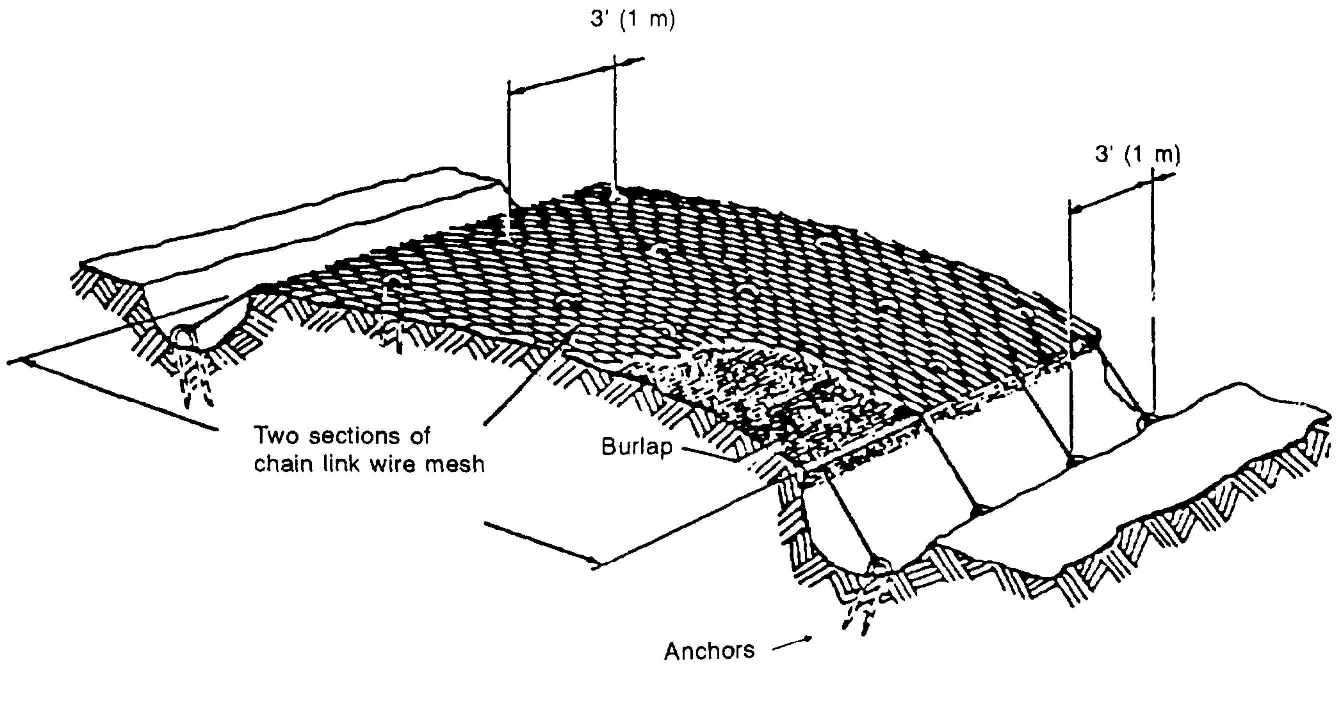


Figure 9-34. Construction details for a wire-mesh road

laid out, make grades and curves as gentle as possible. Compact the snow until it is capable of supporting the weight of vehicles. Add water on the compacted snow and allow it to freeze to produce a hard surface. Frozen lakes or streams can be used to move traffic, but first carefully reconnoiter the route for quality of ice, thickness, cracks, and shore conditions. Determine the load-bearing capacity either by an actual test or by consulting Table 9-7.

Table 9-7. Load capacity of ice

Ice Thickness (inches)	Capacity	Maximum Spacing
1 1/2	Individual soldiers	20 paces
2	Individual soldiers	5 paces
4	Single infantry columns	65 feet
8	Administrative vehicle, artillery, up to 2 1/2 tons, or 4-ton vehicles with maximum axle load of 2.7 tons	65 feet
10 to 13	8-ton (gross) vehicles, including loaded 2 1/2-ton trucks	65 feet
12 to 15	10-ton vehicle (gross)	65 feet
14 to 18	20-ton vehicle (gross)	65 feet
20 to 36	40-ton vehicle (gross)	100 feet

USE OF POLYMER CELLS (SAND GRID) TO BUILD ROADS IN SANDY SOILS

Trafficability over sandy soils is difficult to maintain. The soil strength is adequate, but the soil will displace under a load, due to its cohesionless nature. Wheeled vehicles arc particularly affected. In order to improve trafficability, sand-grid base layers can be used.

Sand grid involves the confinement and compaction of sand or sandy soils in interconnected cellular elements called grids to produce a load-distributing base layer. Uses of the grid include road and airfield pavements, airfield crater repair, erosion control, field fortifications, and expedient dike repair.

Plastic grids (national stock number (NSN) 5680-01-198-7955) are manufactured and shipped in collapsed 4-inch thick, 110-pound sections. (See Figure 9-35.) Each expanded grid section is 8 by 20 feet and contains a honeycomb arrangement of cells. Each cell has a surface area of 39 square inches and a depth of 8 inches. Grids are delivered in 3,000-pound pallets, each containing 25 collapsed 8- by 20-foot sections.

A sand-asphalt surfacing is incorporated within the top portion of the sand-grid cells. Its function is to seal the sand into cells and provide a wearing surface for moderate amounts of rubber-tired traffic. The sand-asphalt surfacing is formed by spraying a suitable liquid-asphalt cement, emulsion, or cutback (rapid-curing (RC) 250 at 165+°F is preferred) on the surface of

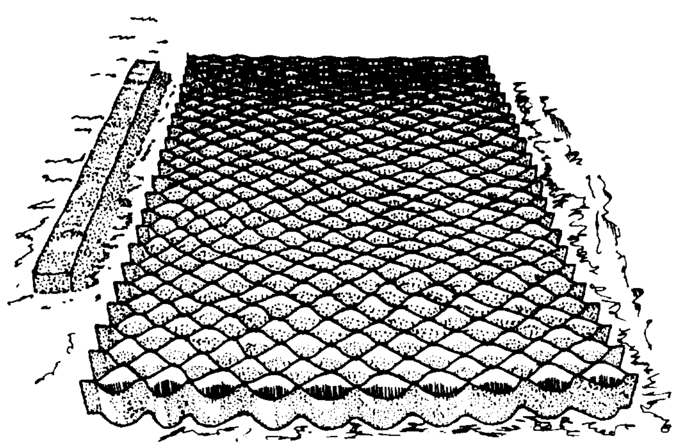


Figure 9-35. Plastic grids

the sand-grid layer. The asphalt used should penetrate into the top 1/2 to 1 inch of sand in the cells. Over a sand subgrade, such a sand-grid road is capable of handling over 10,000 passes of heavy truck traffic, including tandem-axle loads of up to 53,000 pounds. **Avoid tracked vehicles** traveling over this road, as their tracks will easily damage the grid cells.

The following are the procedures for emplacing sand grid.

EQUIPMENT RECOMMENDED

Equipment recommended for the emplacement of sand grid includes bulldozers; smooth-bucket (no teeth) scoop loaders; rough-terrain forklifts; vibratory rollers; water distributors; bituminous distributors; long-handled, round-pointed shovels; and 3/8-inch by 8-foot by 4-foot plywood sheets.

SITE PREPARATION

Site preparation includes the following steps:

1. Perform normal cut or fill operations to reach desircd road grade.

2. Back blade the surface for smoothness.

3. Compact the sand subgrade at a moisture content approaching saturation using the vibratory roller.

GRID INSTALLATION

Grid installation includes the following Steps:

1. Set up stakes and string lines in 8-foot by 20-foot boxes. See Figure 9-36 for layout patterns.

2. Deliver grid pallets with the forklift.

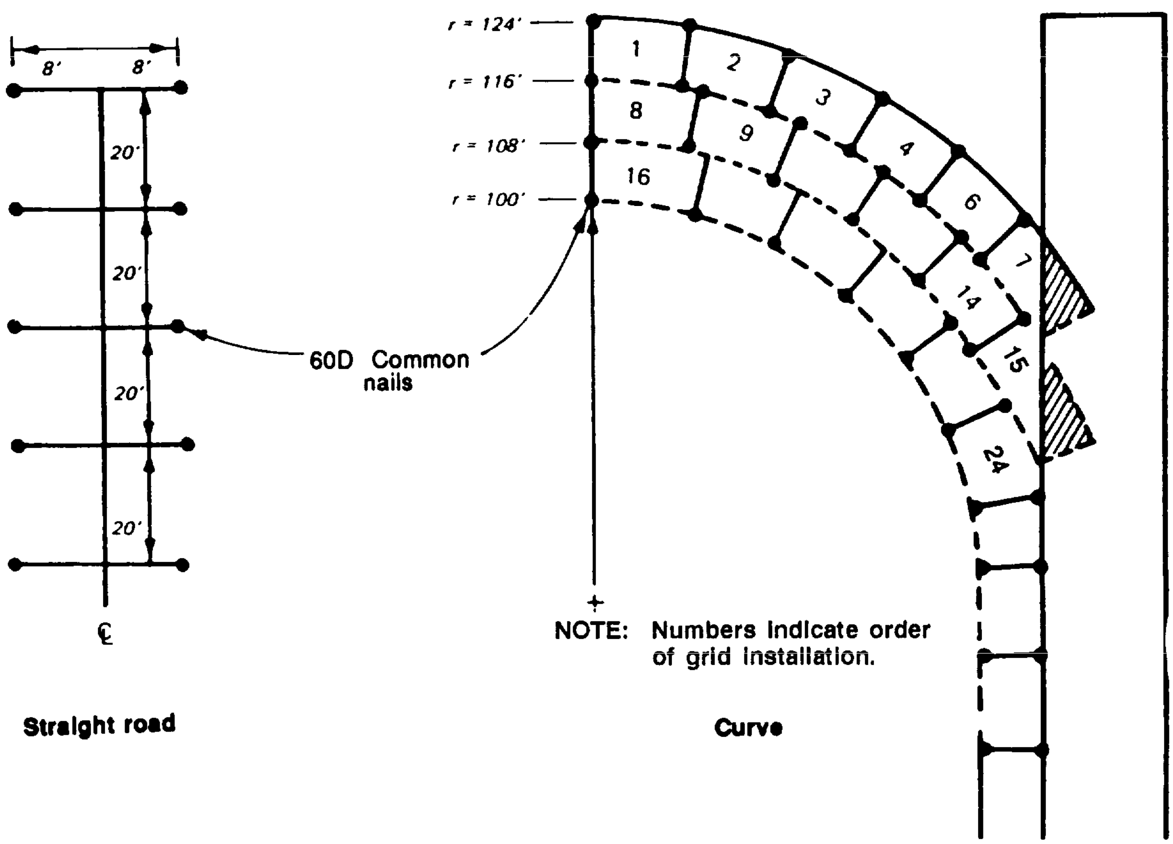


Figure 9-36. Sand-grid layout patterns

3. Expand each grid section using three people on each end, pulling outward to slightly over 20 feet, then shaking the section in midair to obtain uniform cell openings. (See Figure 9-37.) Place the section where string lines dictate, (8-foot by 20-fool steel frames can be ordered.)

4. Shove sand from road shoulders into each end cell and approximately every fifth sirtc-edge cell to anchor the grid in place.

5. To construct joints between grids, usc small, 3/8-inch plywood sheets 10 allow soldiers to stand on top of the unfilled grid, allowing access to the joint cells. For end joints, the rounded end cells from different sections should touch each other. For longitudinal (side-by-side) joints, interlock the "welded" cell portions of each section, as if fitting a puzzle. Fill the jointed cells with sand.

6. Level the joints by placing plywood over the joints and having soldiers walk on top of the plywood.

7. Completely fill each grid using scoop loaders. (See Figure 9-38.) Drop the sand vertically into the cells from a height of at least 2 feet. Do not push the sand forward or the cells will be displaced. Overfill grids by 2 to 4 inches so scoop loaders can operate on the sand-filled grid layer without damaging any cells. Have scoop loaders vary wheel paths to achieve uniform initial compaction of the road.

8. If water is readily available, wet the sand using a water distributor. This will significantly aid in the compaction process.

9. Compact the road surface with one or two passes of the vibratory roller.

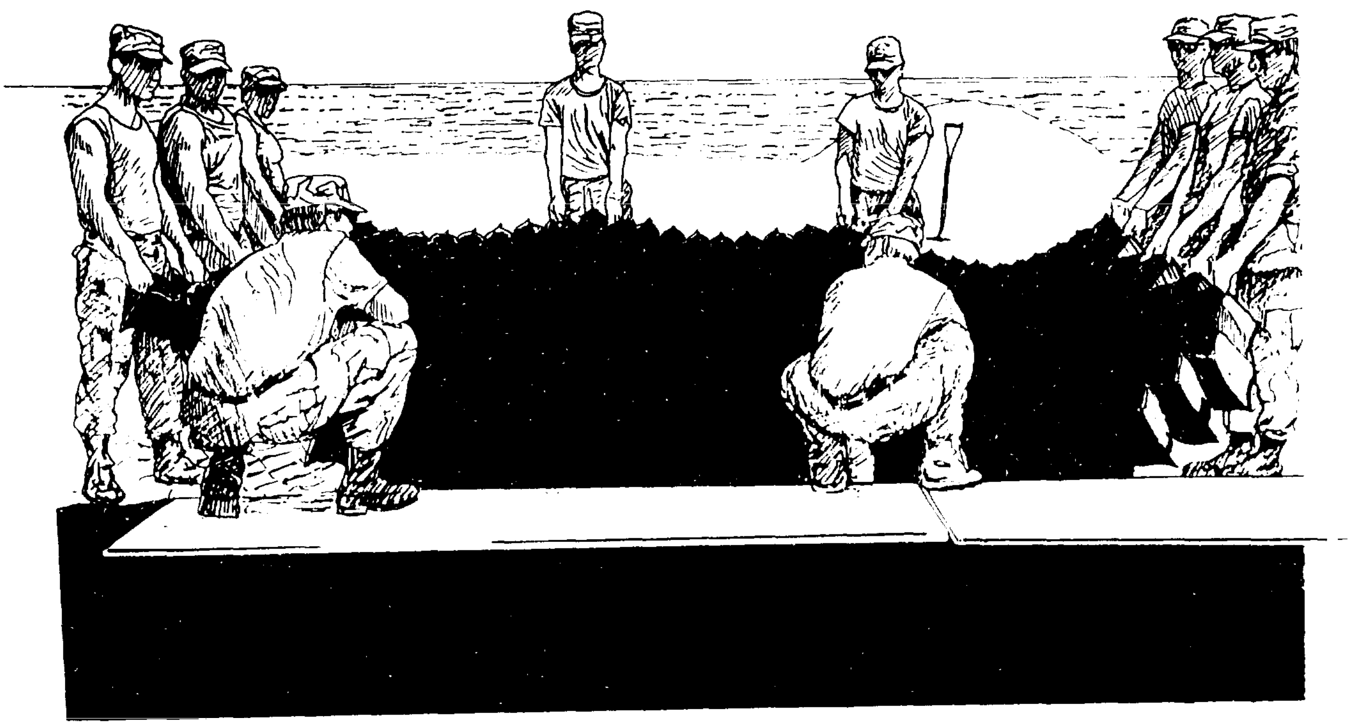


Figure 9-37. Plastic sand-grid section emplacement

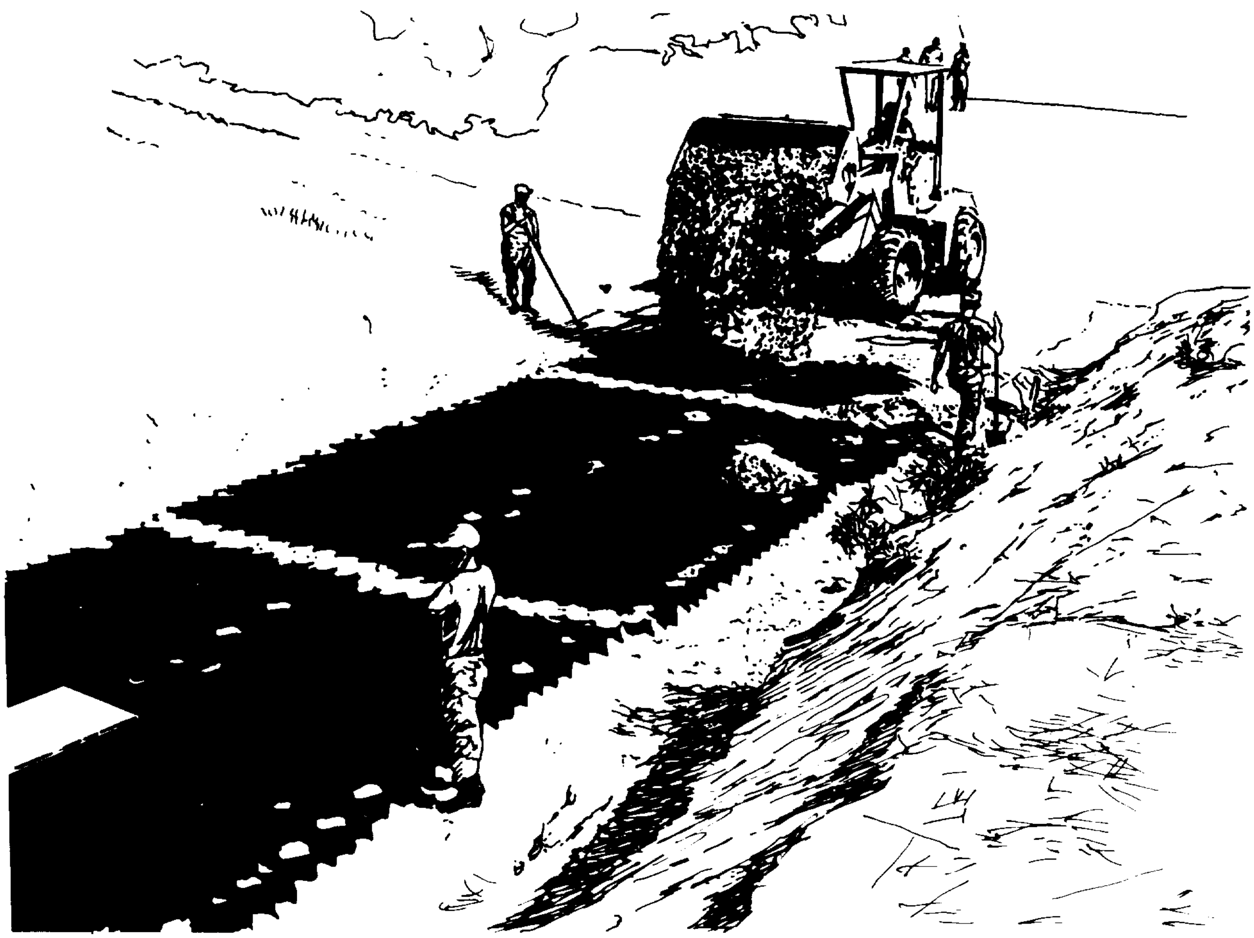


Figure 9-38. Sand-grid road - filling cells with sand

10. Remove excess sand from the grid surface with a grader or by back blading with a smooth-bucket scoop loader. Do not use a bucket with teeth, as cell damage can result.

11. Recompact the road with one pass of the vibratory roller. (See Figure 9-39, page 9-40.)

12. Spray the asphalt product on the road surface and allow enough time for the asphalt product to completely soak into the grid structure (usually about 10 hours). (See Figure 9-40, page 9-40.)

13. Apply a very light coat (1/4 inch) of blotter sand using shovels.

14. Compact the road using one pass of a **nonvibrating roller.** Vibrating the road at this point will break the asphaltic bonds,

15. Apply and compact a 3-inch surface coat of 1-inch (maximum) gravel, if available. This layer will add a protective cover material over the sand-grid road, significantly increasing the road life.

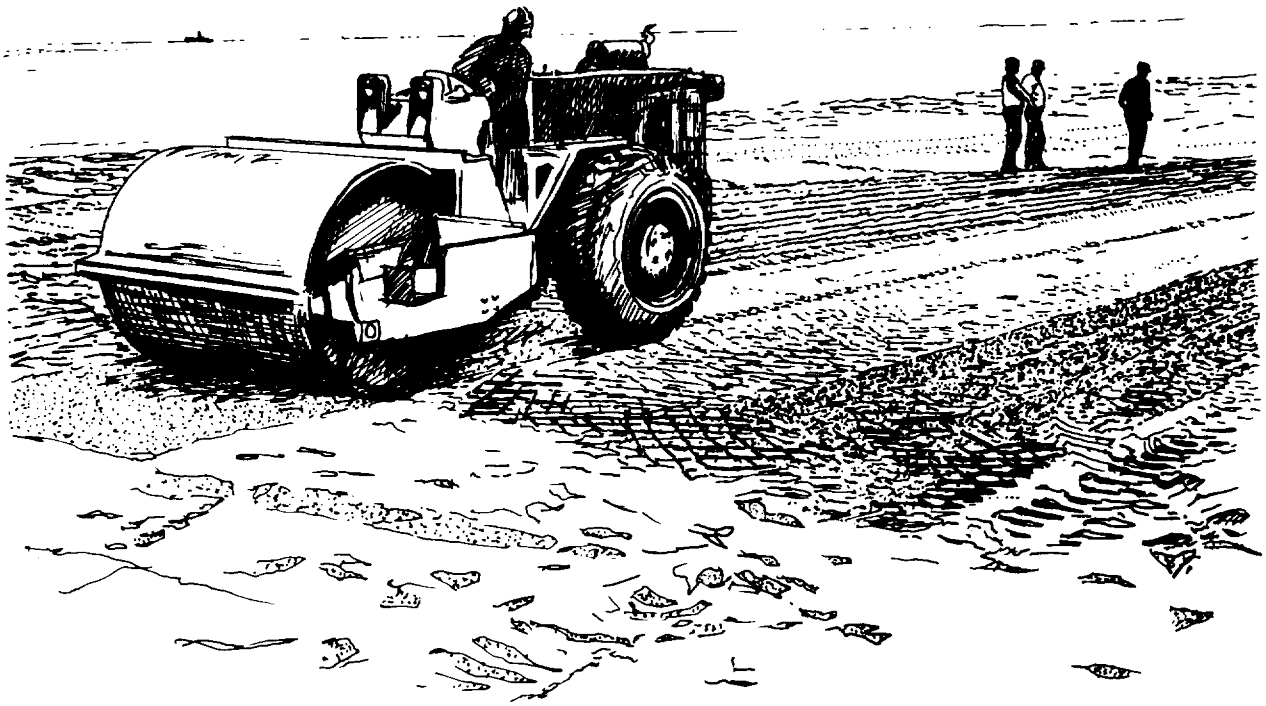


Figure 9-39. Sand-grid road - compacting sand with roller

Figure 9-40. Sand-grid road - spraying asphalt coating

SPRAY APPLICATIONS AND SURFACE TREATMENTS

Surface treatments are the most economical troop-constructed surfaces. They require the fewest resources and minimal quality control effort, and they are placed in the shortest period of time. Surface treatments range from single, light applications of bituminous material to multiple surface courses made up of bitumen and aggregates. Surface treatments can be divided into two categories: sprayed treatments and sprayed bitumen with an aggregate surface.

Spray applications provide soil or aggregate surfaces with the following:

- Prime coat (waterproofing).
- Tack coat (binding bituminous pavement to surface).
- Dustproofing.

Sprayed bitumen with an aggregate surface provides a waterproof, abrasive, wear -resistant surface with no structural strength. Bitumen with aggregate surface pavements will be either of the following:

- Single surface treatment.
- Multiple surface treatment.

Bituminous materials are either tars, road-tar cutbacks, asphalt cement, asphalt cutbacks, or asphalt emulsions. Uses of bituminous materials are shown in Figure 9-41, page 9-42. Asphalt cement is the heavy material left at the end of the petroleum distillation process. Crude oil is refined into gasoline, kerosene, diesel, motor oil, asphalt, and many other products as shown in Figure 9-42, page 9-43. Asphalt cement may then be further modified by cutting back the asphalt with some petroleum product, specifically naph - tha, gasoline, kerosene, or diesel, to form an asphalt cutback. Asphalt may also become an emulsion by mixing asphalt cement, water, and an emulsifying agent together with variable-speed pumps to form an asphalt water suspension. Emulsions are either anionic (carrying a negative charge) or cationic (carrying a positive charge).

Tars are the residue from the high-temperature conversion of coal to coke. (See Figure 9-43, page 9-44.) They may be modified by cutting them with a light to medium coal oil to form a road-tar cutback. Tars do not dissolve in petroleum products. They become soft and bleed at high temperatures and are brittle at cold temperatures. Because of their susceptibility to temperature change, tars normally arc used only in areas where fuel spills are commonplace, such as tank farms, fuel depots, and aircraft refuel points.

FIELD IDENTIFICATION

Field identification consists of a series of simple tests designed to identify an unknown bitumen product in the field. The purpose of these tests is to determine the uses of a bituminous material and how to use it safely, rather than the exact specifications to which it conforms. Field-identification procedures are applicable to both tars and asphalts.

Bituminous materials are often found stockpiled in unmarked or incorrectly marked containers. This leads to confusion and delay in construction since the various types and grades of bituminous material are manufactured for a specific purpose.

Field identification is important to the military engineer because—

- Once the type and grade of material are known, the type of surface that can be constructed may be determined. The safety procedures to be followed also depend on the material identification.
- Once the surface type is known, the construction procedure may be outlined and scheduled for a specific target date.
- Once established, the procedure will determine the proper equipment for the job.

TYPE OF CONSTRUCTION	ASPHALT CEMENTS															CUTBACK ASPHALTS													EMULSIFIED ASPHALTS													TARS													
	VISCOSITY GRADED -ORIGINAL					VISCOSITY GRADED -RESIDUE					PENETRATION GRADED					RAPID CURING (RC)				MEDIUM CURING (mc)					SLOW CURING (SC)				ANIONIC							CATIONIC																			
	AC-40	AC-20	AC-10	AC-5	AC-2.5	AR-16000	AR-8000	AR-4000	AR-2000	AR-1000	40-50	60-70	85-100	120-150	200-300	70	250	800	3000	30	70	250	800	3000	70	250	800	3000	RS-1	RS-2	MS-1	MS-2	MS-2h	SS-1	SS-1h	CRS-1	CRS-2	CMS-2	CMS-2h	CSS-1	CSS-1h	1 RT	2 RT	3 RT	4 RT	5 RT	6 RT	7 RT	8 RT	9 RT	10 RT	11 RT	12 RT	RTCB	RTCB 6
ASPHALT - AGGREGATE MIXTURES CONCRETE AND HOT LAID PLANT MIX																																																							
PAVEMENT BASE AND SURFACES HIGHWAYS		X	X	X	1	X	X	X	A	1		X	X	X	1																																								
AIRPORTS		X	X	X			X	X				X	X																																						X	X		
PARKING AREAS		X	X			X	X	X				X	X																																						X	X		
DRIVEWAYS		X	X				X	X				X	X																																						X	X		
CURBS	X	X	X			X	X				X	X	X																																						X	X		
INDUSTRIAL FLOORS	X	X				X	X				X	X																																										
BLOCKS	X										X																																											
GROINS	X	X									X																																											
DAM FACINGS	X	X									X	X																																										
CANAL AND RESERVOIR LININGS	X	X									X	X																																										
COLD - LAID PLANT MIX																																																							
PAVEMENT BASE AND SURFACES																																																							
OPEN-GRADED AGGREGATE*																							X								X	X	X					X	X										X	X	X				
WELL-GRADED AGGREGATE*																	X					X	X	X		X	X	X						X	X			X		X	X							X	X	X	X				
PATCHING, IMMEDIATE USE																	X	X				X	X	X			X	X						X	X					X	X													X	X
PATCHING, STOCKPILE																						X	X			X	X																												
MIXED - IN - PLACE (ROAD MIX)																																																							
PAVEMENT BASE AND SURFACES																																																							
OPEN - GRADED AGGREGATE*																	X	X	X				X	X		X	X				X	X						X	X								X	X	X	X	X				
WELL - GRADED AGGREGATE*																	X					X	X			X	X					X		X	X			X		X	X					X	X	X	X	X					
SAND				X	X									X	X		X	X			X	X	X											X	X					X	X														
SANDY SOIL				X	X									X	X	X	X	X				X	X											X	X			X		X	X														
PATCHING, IMMEDIATE USE																	X	X				X	X	X			X	X						X	X					X	X													X	X
PATCHING, STOCKPILE																						X	X			X	X																												
ASPHALT - AGGREGATE APPLICATIONS (SURFACE TREATMENT)																																																							
SINGLE SURFACE TREATMENT				X	X									X	X		X	X	X				X	X					X	X						X	X							X	X	X	X	X	X	X	X	X			
MULTIPLE SURFACE TREATMENT				X	X									X	X		X	X	X					X					X	X						X	X							X	X	X	X	X	X	X	X	X			
AGGREGATE SEAL				X	X									X	X		X	X	X				X	X					X	X						X	X							X	X	X	X	X	X	X	X	X			
SAND SEAL									X								X					X	X						X	X						X	X							X	X	X	X	X	X						
SLURRY SEAL																																		X	X					X	X														
(PENETRATION MACADAM)																																																							
PAVEMENT BASES																																																							
LARGE VOIDS			X										X					X	X											X						X	X															X	X		
SMALL VOIDS				X										X			X												X	X						X	X															X	X		
ASPHALT APPLICATIONS (SURFACE TREATMENT)																																																							
FOG SEAL																																		2	2					2	2			X	X	X	X								
PRIME COAT, OPEN SURFACES																X	X				X	X																					X	X	X										
PRIME COAT, TIGHT SURFACES																X				X	X													X									X	X											
TACK COAT																X													X					2	2	X				2	2			X	X	X	X	X	X	X	X				
DUST LAYING																					X													2	2					2	2	X	X												
MULCH																																		2	2										X	X	X	X							
(MEMBRANE)																																																							
CANAL AND RESERVOIR LININGS	X										X																																											
EMBANKMENT ENVELOPES	X	X									X	X																																										
(CRACK FILLING)																																																							
ASPHALT PAVEMENT																																		3	3					3	3	X	X	X	X										
PORTLAND CEMENT CONCRETE PAVEMENT	4										4																															X	X	X	X										

* Evaluation of emulsified asphalt-aggregate system required to determine the proper grade of emulsified asphalt to use.

1 FOR USE IN COLD CLIMATES A FOR USE IN BASES IN COLD CLIMATES 2 DILUTED WITH WATER 3 SLURRY MIX

4 RUBBER ASPHALT COMPOUNDS # Emulsified asphalts shown are AASHTO and ASTM grades and may not include all grades produced in all geographic areas.

Figure 9-41. Use of bituminous materials

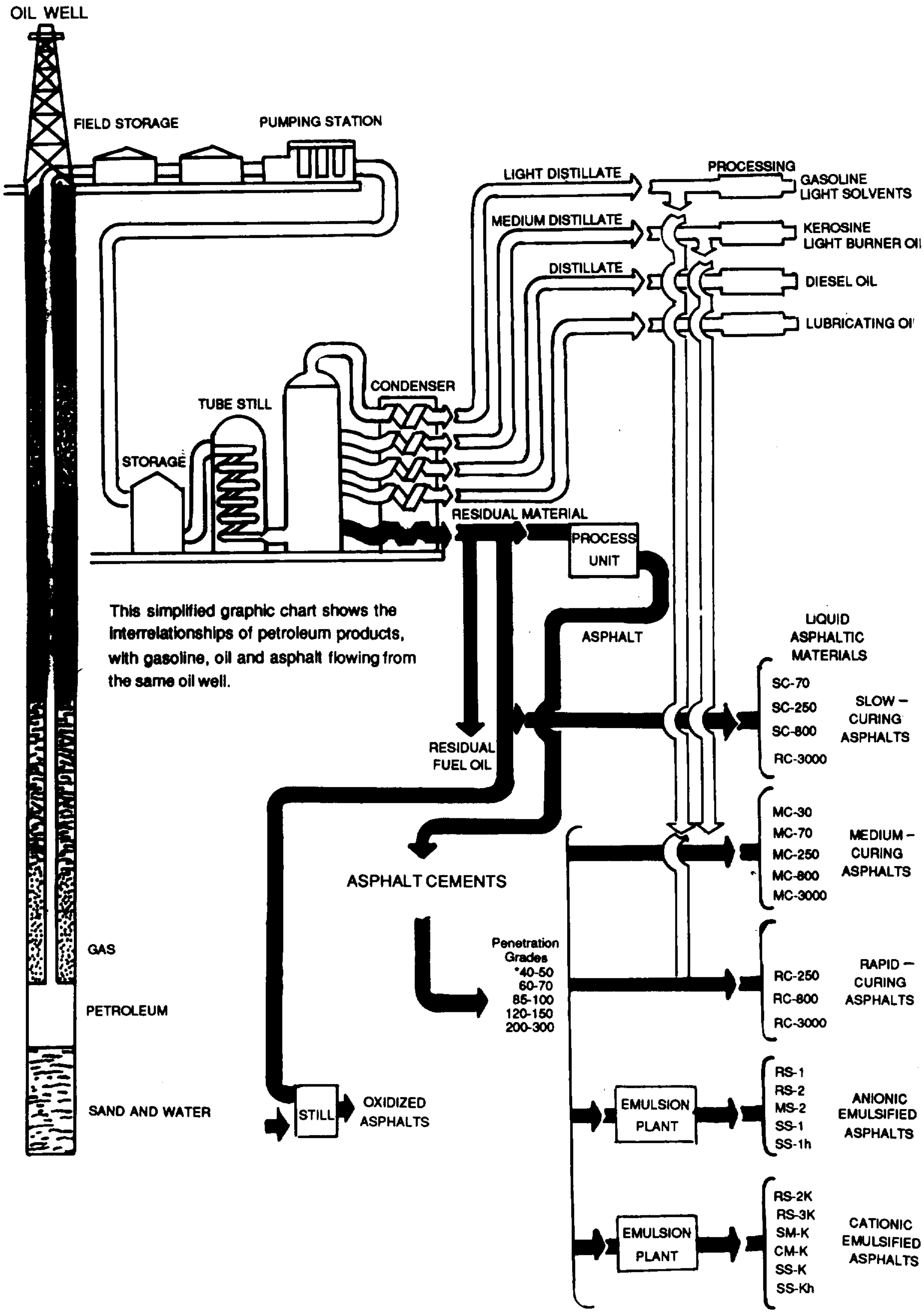


Figure 9-42. Petroleum-asphalt flowchart

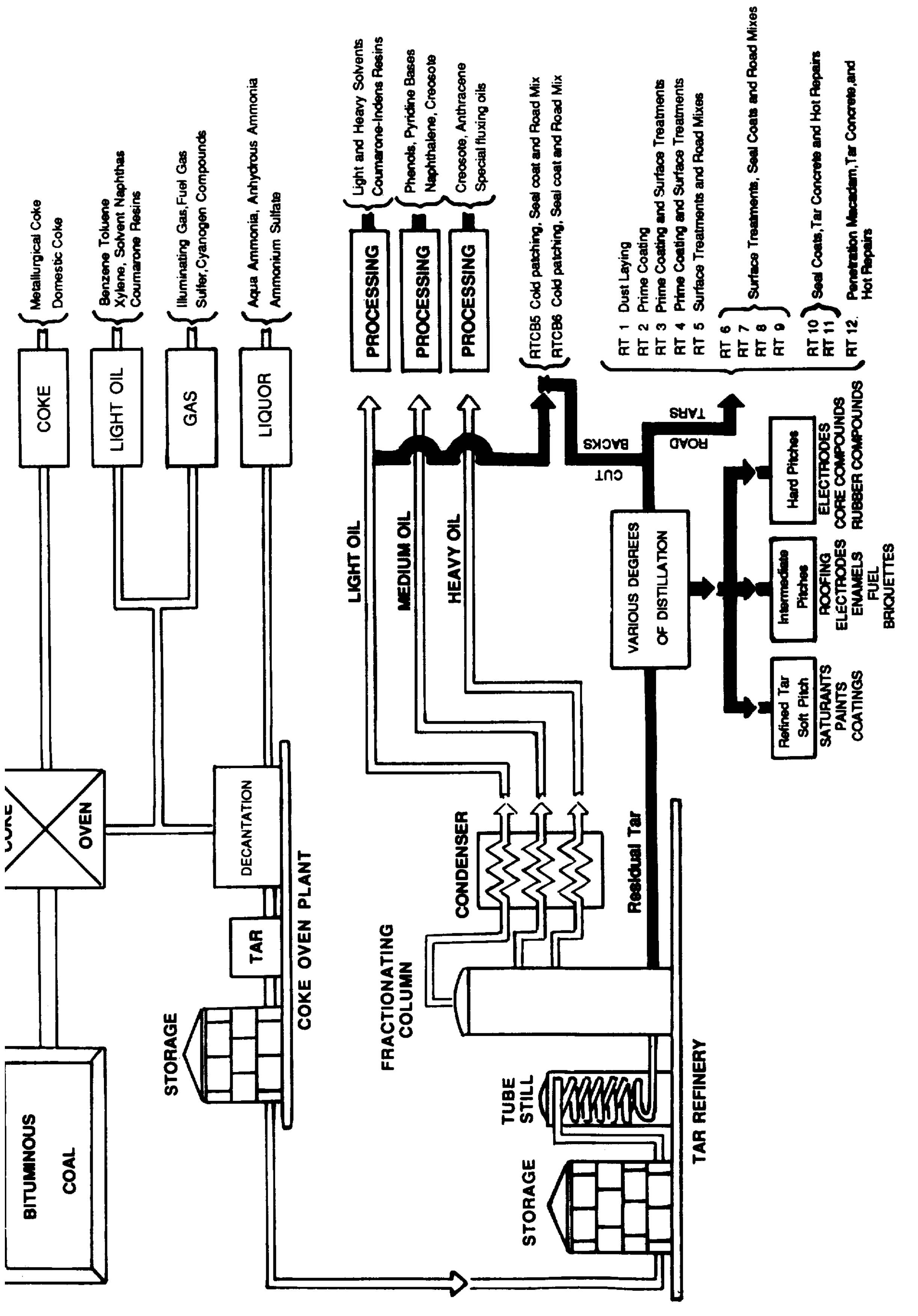


Figure 9-43. Tar simplified flowchart

Field tests may be performed to identify the bituminous paving material as asphalt cement, asphalt cutback, asphalt emulsion, road tar, or road-tar cutback. In addition, it is necessary to identify as closely as possible the viscosity grade of the bitumen. In order to distinguish among the several asphaltic and tar products, it is necessary to know something of their origin, their physical properties, and the manner in which they are normally used. The identification procedure outlined in Figure 9-44, page 9-46, is based upon the physical properties of these materials.

Asphalts and Tars

The first procedure in the identification of an unknown bituminous material is to determine whether it is an asphalt or a tar. Asphalts and tars may be differentiated by a simple volubility test. To perform the test, simply attempt to dissolve an unknown sample (a few drops, if liquid, or enough to cover the head of a nail, if solid) in any petroleum distillate. Kerosene, gasoline, diesel oil, or jet fuel is suitable for this test. Since asphalt is derived from petroleum, it will dissolve in the petroleum distillate. Road tar will not dissolve. If the sample is an asphalt, the sample distillate mix will consist of a dark, uniform liquid. If it is a road tar, the sample will be a dark, stringy, undissolved mass in the distil late. A check can be made by spotting a piece of paper or cloth with the mix. The volubility test provides a positive method of identification.

Grades of Asphalt Cement

The various grades of asphalt cement are distinguished principally by their hardness, as measured by a field penetration test explained in TM 5-337. This information may be sufficient for planning or, in some cases, actually starting emergency construction.

Asphaltic Cutbacks

The pouring or nonpouring quality is one way to distinguish between an asphalt cement and a cutback or emulsion. Asphalt cement is *cutback* with a petroleum distillate to make it more fluid. If the material does not *pour,* it is an asphalt cement. Note that at 77° F, even the softest asphalt cement will not pour or deform noticeably if the container is tilted. If it pours, it is a cutback or emulsion. If it is soluble or dilutable in water, it is an emulsion. In addition, the manner in which it pours will furnish a clue to its grade.

To determine whether a cutback is a RC asphaltic cutback or not, the *smear test* is performed. This is done by making a uniform smear of the substance on a piece of glazed paper or other convenient, nonab - sorbent surface. This will give the volatile materials, if present, a chance to evaporate. Since RCs are cut back with a very volatile substance, most of the volatiles will evaporate within 10 minutes. The surface of the smear will then become tacky. This is not true of the lighter grades of medium- and slow-curing asphaltic cutbacks (MCs and SCs), which remain fluid and smooth for some time. An MC will not result in a tacky surface for a matter of hours; for SC materials, several days may be required.

To identify an 800- or 3,000-grade MC or SC cutback, a *prolonged* smear test is used. This process is necessary because these grades of MCs and SCs contain such small quantities of cutterstock that they, too, may become tacky in the 10-minute period specified above. A thin smear of the material is made on a nonabsorbent surface and left to cure for at least 2 hours. By the end of that time, if the material being tested is an RC, the smear will have cured and will be hard or just slightly sticky. However, if the material being tested is an MC or SC, the smear will not be cured and will still be quite sticky. If the material is an RC 3,000, it will cure completely in 3 hours, whereas an RC 800 will take about 6 hours. Even after 24 hours, an MC or SC will still be sticky.

Since MCs are cutback with kerosene and SCs with oil, this fact may be employed to differentiate between them. Heat is used to drive off the kerosene, if present, and make it show up in the form of an odor. It is best to heat the unknown sample in a

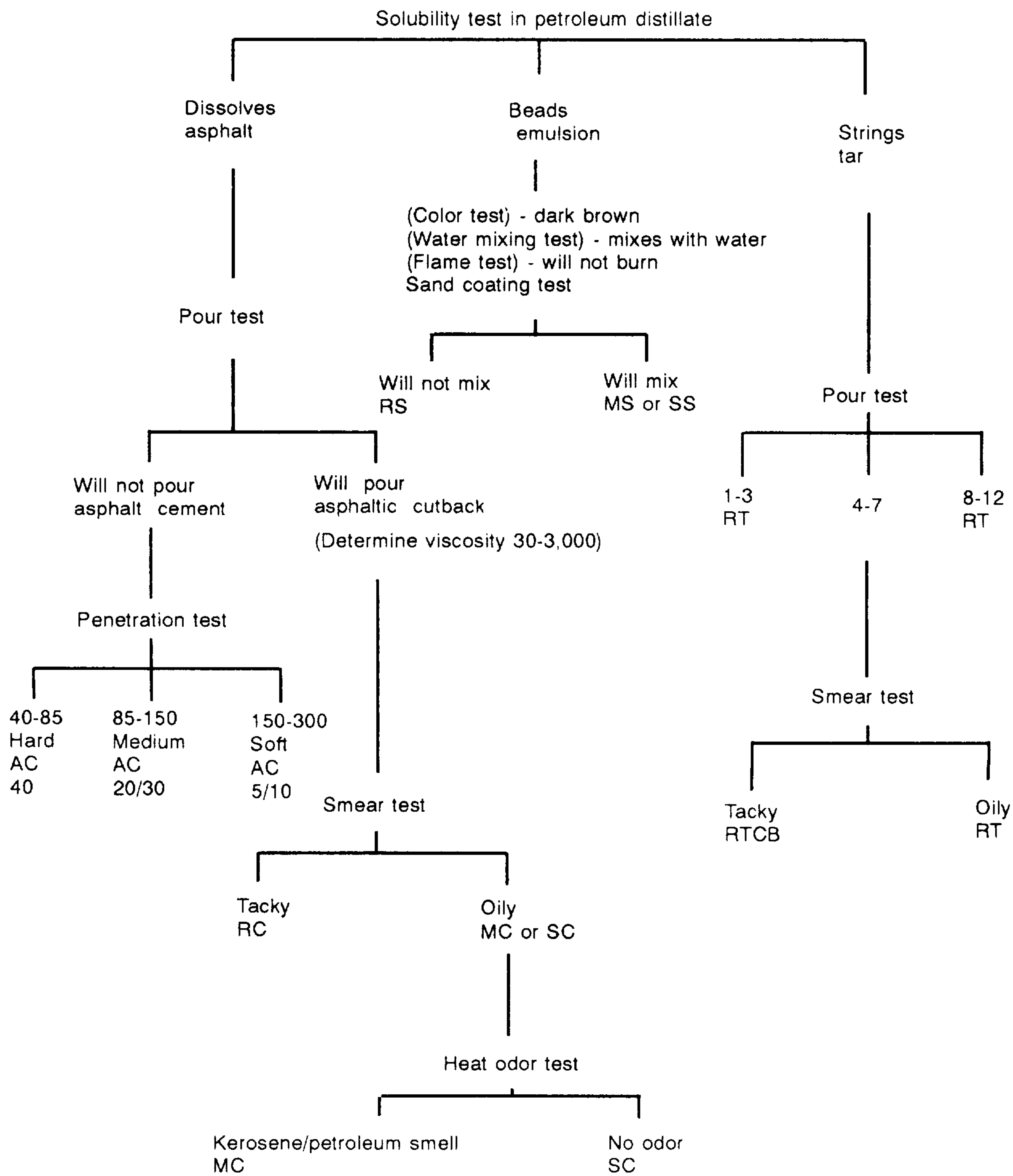


Figure 9-44. Identification of unknown bituminous materials

closed container in order to capture the escaping vapors, being careful not to apply too much heat. If the sample is an MC, it will have a strong petroleum or kerosene odor. On the other hand, if the sample is an SC, no kerosene or petroleum odor will be detected. It might smell somewhat like hot motor oil. The ability to distinguish an RC from an MC and an SC from either, is perhaps as important as any other part of field identification.

Asphalt Emulsions

Another asphaltic material used in paving work is an asphalt emulsion, which is a mixture of asphalt, water, and an emulsifying agent. It is easy to identify, since it is usually distinguished by its dark brown color, while the other bitumens are black, If mixed with kerosene or some other petroleum distillate, the emulsion can be detected by the appearance of small black globules or beads which fall to the bottom of the container. If mixed with water, an emulsion will accept the extra water and still remain a uniform liquid. The other bitumens will not mix with water. Since an emulsion contains water, a small piece of cloth saturated with it will not burn if a flame is applied. The other bitumens will burn or flame. After it has been established that the material is an emulsion, it is still important to know whether or not the emulsion is a mixing grade. The best way to tell if the emulsion is a mixing grade (slow-setting (SS) or medium-setting (MS)) is to try to mix a small amount (6 to 8 percent, by weight) with damp sand using a metal spoon. A rapid-setting (RS) emulsion cannot be mixed; it breaks immediately, gumming up the spoon with the relatively hard original asphalt cement. A SS or MS emulsion mixes nicely, coating the sand. Be careful not to add too much emulsion to the sand. This will saturate the sand and not give conclusive results. No further identification is necessary, since both MS and SS grades are largely used for the same jobs.

Road Tars

If the unknown bituminous material did not dissolve in the volubility test but formed a stringy mass, as shown in Figure 9-44, the material is a tar. The next step is to determine its viscosity grade by the pour test. By comparing the flow to that of common materials, such as water or honey, the viscosity of the tar may be closely estimated. The grades vary from road tar (RT)-1 to RT-12. If the identified tar has a viscosity in the range of RT-4 to RT-7 material, a smear test must be performed to determine whether it is a road tar or a road-tar cutback. The smear test is performed in the manner previously described for cutback asphalt. A great increase in stickiness in about 10 minutes identifies a road-tar cutback. No apparent change in consistency after 10 minutes indicates a road tar. It does not matter which grade of cutback is available, since both are used under approximately the same conditions.

AGGREGATE IDENTIFICATION

The aggregate must also be tested to determine its suitability for bituminous construction. The desirable characteristics of an aggregate used in bituminous construction include—

- Angular and rough.
- Tough, hard, and durable.
- Clean and dry.
- Hydrophobic.

Available aggregate may not always have all desirable characteristics. An aggregate meeting most of the requirements is usually selected, unless rejected for reasons such as availability, length of haul, or difficulty in conducting borrow-pit or quarry operations.

Angular and Rough

The aggregate in a pavement must transmit the traffic load to the base, usually by the interlocking and surface friction of the different particles. Angular particles with a rough texture arc best for this purpose since they do not tend to slide past each other. More binder may be required since the angular shape has a greater surface-area-per-unit volume than a round particle.

Tough, Hard, and Durable

The aggregate must withstand loads without cracking or being crushed. Resistance to weathering is also a function of the durability. The resistance-to-wear of an aggregate can be determined by the Los Angeles Abrasion Test, if the equipment is available. The equipment and procedures are detailed in the American Association of State Highway and Transportation Officials (AASHTO) method T96. The equipment for the above test is usually not available for field testing. The Mob's hardness scale may be used to determine the hardness of the aggregate. The Mob's scale ranges from 1 for talc or mica to 10 for diamond. By trying to scratch the aggregate or the common materials and vice versa, it is possible to establish which is harder; from this analysis the hardness of the aggregate can be determined. If both materials scratch each other, the hardness of each is the same. Be sure to rub the "scratch" mark to see that it is really a scratch and not a powdering of the softer material. Some common materials and their hardness are: fingernail, about 2: copper coin, between 3 and 4; and knife blade, nail, and window glass, about 5.5. If the material can be scratched with one of these common items, it is considered to be soft. If it cannot be scratched, it is considered to be hard.

Clean and Dry

The bituminous binder must penetrate into the pores of the aggregate and adhere to the surface of the particles. Coated (with clay or dust) or water-filled aggregate will prevent the penetration or the adherence of the binder and result in stripping of the binder. For hot mixes, the aggregate must be hot as well as dry. If the aggregate is not clean, it should be washed either as part of the crushing operation or by spreading it on a hard surface and hosing it with water. When washing is impractical, dry screening may remove a great deal of dust and clay. Hand picking may be necessary if no other method can be used. The aggregate should be made as clean as possible with the equipment and manpower available.

Hydrophobic

Affinity for water can make an aggregate undesirable. If the aggregate is porous and absorbs water easily (hydrophilic), the binder can be forced out of the pores. When this happens, the bond between the aggregate and binder weakens and breaks and *stripping* occurs. Stripping is the loss of the bituminous coating from the aggregate particles due to the action of water, leaving exposed aggregate surfaces. One of the following three tests can be used to determine the detrimental effect of water on a bituminous mix:

Stripping Test. A test sample is prepared by coating a specific amount of aggregate with bituminous material at the applicable temperature for the grade of bitumen to be used. The mixture is spread in a loose, thin layer and air-cured for 24 hours. A representative sample is placed in a jar (up to no more than one-half of its capacity) and covered with water. The jar is closed tightly and allowed to stand 24 hours. At the end of 24 hours, the jar with the sample is vigorously shaken for 15 minutes. A visual examination is made to determine the percentage of exposed aggregate surface which is reported as percent stripping.

Swell Test. Asphaltic mixtures containing fines of doubtful quality are sometimes measured for swell as a basis for judging the possible effects on a pavement. This test is more frequently used with dense-graded mixtures using liquid asphalts. A sample of the mix is compacted in a metal cylinder and cooled to room temperature. The specimen and mold are placed in a pan of water and a dial gage is mounted above the sample in contact with the surface. An initial reading is taken. The specimen is allowed to soak for a specified period (usually 24 hours) or until there is no further swelling. Another reading of the dial is taken. The difference in reading is the swell of the mixture. Experience has shown that bituminous pavement made with clear, sound stone; slag; or gravel aggregate and mineral filler produced from limestone will show test values of less than 1.5 percent. Aggregates of doubtful character should be tested for conformance to ASTM tests.

CONSTRUCTION METHODS

PRIME COAT

A prime coat is used when a surface treatment or pavement is placed on a soil or aggregate base. The prime coat should penetrate the base about 1/4 inch, filling the voids. The prime coat acts as a waterproof barrier to prevent moisture that may penetrate the wearing surface from reaching the base. Also, the bitumen acts as a bonding agent, binding the particles of the base to the wearing surface. Plan priming operations so that there will always be an adequate amount of cured, primed base ahead of the surfacing operations; but not so far ahead that the base will become dirty or completely cured *(dead).* To preserve the base, a prime coat should be applied as soon as the base is ready; however, the prime coat will lose its effectiveness as a bonding agent if the wearing surface is not placed soon after curing.

Base Preparation

The base should be well-graded, shaped to the desired cross section, compacted to the specified density, well-drained, free from excessive moisture but not completely dry, and swept clean. The surface of the base should be broomed if it contains an appreciable amount of loose material, either fine or coarse, or if it is excessively dusty. When brooming is omitted, apply a prime coat to the base and lightly roll it with a pneumatic roller, or use a light sprinkling of water to settle the dust. Sprinkling is usually undesirable; but when it is necessary, lightly apply a spray of water at the rate of approximately 0.2 gallon to 0.3 gallon per square yard, depending on the condition of the base, the temperature, and the humidity. Completely cover the base with a minimum amount of water and allow it to become dry or almost dry before applying the prime coat so that it will absorb the prime material. If the base is too wet, it will not take the prime properly and the moisture will tend to come out, particularly in hot weather, and strip the prime from the base during construction. Rains also tend to strip the prime from a base that was too wet when primed. Heavy rains may also strip a properly primed base to some extent, but less than an improperly cured base. In general, the lowest acceptable moisture content for the upper portion of the base course prior to priming should not exceed one-half of the optimum moisture content. On the other hand, if the base dries out completely, cracks may develop and a heavy rain may then cause swelling and loss of density. See Chapter 5 of this manual for subgrade, subbase, and base-course preparation.

Materials

Bituminous materials used for prime coats will depend on the condition of the soil base and the climate. In moderate and warm climates, RT-2, RT-3, RT-4, MC-30, MC-70, SS-1, SS-1h, cationic slow-setting emulsified asphalt (CSS)-1, and CSS-1h are satisfactory. In cold climates, rapid-setting asphalt cutbacks, such as RC-70 and RC-250, have proved more satisfactory. If the climate is very cold, the prime coat may be eliminated because it is likely to be extremely slow in curing. RT-2 and MC-30 are satisfactory for a prime coat used on a densely graded base course. MC-70 is generally used on loosely bonded, fine-grained soils, such as well-graded sand. MC-250 is usually satisfactory for coarse-grained sandy soils.

The formula used to determine the quantity of prime coat material required is—

$$\frac{L \times (W + 2) \times AR \times LF}{9\ (ft^2/yd^2)} = Q_p \text{ Gallons}$$

where—

L = length of untreated surface in feet
W = width of untreated surface in feet
("+2" in the formula is to include for overspray of shoulders 1 foot on both side of the road.)
AR = application rate of prime coat in gallons per square yard

NOTE: AR for dense soils = 0.2, AR for soils with a lot of cracks = 0.5

LF = handing loss factor for prime coat (usually 1.05 - 1.10)
Q_p = quantity of prime-coat material in gallons

Example:

Compute the quantity (in gallons) of prime-coat material (Q_p) required to prime an untreated surface with dense soil. The surface is 1,000 feet long and 12 feet wide. Use a loss factor of 1.05.

Solution:

$$Q_p = \frac{L \times (W + 2) \times AR \times LF}{9\ (ft^2/yd^2)}$$

$$= \frac{1{,}000 \times (12+2) \times 0.2 \times 1.05}{9\ (ft^2/yd^2)}$$

$$= 326.67 \text{ or } 327 \text{ gallons}$$

TACK COAT

A tack coat is a sprayed application of a bituminous material that is applied to an existing wearing surface of concrete, brick, bituminous material, or binder course before a new bituminous pavement is placed over the existing surface. The purpose of the tack coat is to provide a bond between the existing pavement and the new surface. The tack coat should become tacky within a few hours. A tack coat is not required on a primed base unless the prime coat has completely cured and become coated with dust. Figure 9-45 shows the sequence of operations for the application of a tack coat. Operation of asphalt-surface-treatment equipment is explained in FM 5-434.

The procedure for estimating the bitumen required for a tack coat is similar to that described for a prime coat except that the tack coat is generally applied only over the proposed width of the pavement. The formula used to determine the quantity of tack coat required is--

$$\frac{L \times W \times AR \times LF}{9\ (ft^2/yd^2)} = Q \text{ gallons}$$

where—

L = length of treated section in feet
W = width of treated section in feet
AR = rate of application of bitumen in galions per square yard
LF = handling loss factor for bitumen (usually 1.05)
Q_t = quantity of tack coat material in gallons

The tack coat is generally applied only over the width of the existing area that is to be surfaced. A tachometer chart may be used to establish the rate of application, The usual rate of application vatries between 0.05 and 0.25 gallon per square yard. On a smooth, dense, existing surface, the minimum rate of 0.05 gallon per square yard should produce a satisfactory bond. If the surface is worn, rough, and cracked, the maximum rate of 0.25 will probably be required. An extremely heavy tack coat may be absorbed into the surface mixture resulting in a bleeding and flushing action and loss of stability. Roll the surface lightly with a rubber-tired roller or truck tires for uniform distribution of the bituminous material.

Example:

Compute the quantity (in gallons) of tack-coat material (Q_t) required to cover a worn, rough, and cracked surface. The surface is 1,000 feet long and 12 feet wide. Use a loss factor of 1.05.

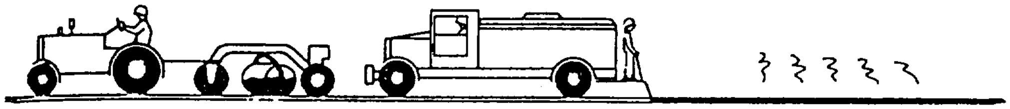


Figure 9-45. Tack-coat sequence of operations

Solution:

$$Q_t = \frac{L \times W \times AR \times LF}{9\ (ft^2/yd^2)}$$

$$Q_t = \frac{1{,}000 \times 12 \times 0.25 \times 1.05}{9\ (ft^2/yd^2)}$$

$= 350.0$ gallons (gal)

DUSTPROOFING

Dustproofing consists of spraying an untreated surface with a diluted, slow-setting asphalt emulsion or a low-viscosity cutback The asphalt and dilutent penetrate the fine soil particles and adhere to the dust particles.

An asphalt cutback is usually sprayed at a rate of 0.1 to 0.5 gallon per square yard (gal/yd^2).. When using an emulsion, dilute it with up to five parts of water by volume. Diluted-emulsion rustproofing treatments usually require several treatments. The dust stirred by traffic between applications eventually conglomerates and no longer rises. This is an effective treatment in very dusty areas where one application of cutback asphalt is insufficient. In all cases, lay a test strip to determine what application rate will be the most effective. Apply with either an asphalt distributor or something as simple as a common watering can. Rustproofing is usually a short-lived solution and project plans should include regular inspections and maintenance, as required.

SPRAYING ASPHALT WITH COVERED-AGGREGATE AND SINGLE AND MULTIPLE SURFACE TREATMENTS

A sprayed asphalt with a cover-aggregate surface treatment consists of an application of asphalt followed by an application of aggregate. If the process is repeated, the resulting surfaces arc referred to as double, triple, quadruple, and so on, surface treatments, depending on the number of applications.

Apply these surface treatments on a primed, nonasphaltic base; an asphalt base course; or any type of existing pavement. This type of surface treatment, with a good prime coat (see preceding paragraph), provides the lowest-cost waterproof covering for a road surface. With good aggregate, this type of surface treatment will economically provide a wearing surface to meet the needs of medium and low volumes of traffic.

This type of surface treatment is very useful as a wearing surface on base courses in the staged construction of highways pending placement of asphalt-concrete surface courses.

Limitations in the use of sprayed asphalt with cover-aggregate surface treatments are—

- Weather conditions must be favorable.
- The surface on which the asphalt is sprayed must be hard, clean, and dry for the surface treatment to adhere properly.
- The amount and viscosity of the asphalt must be carefully balanced with the size and amount of cover-aggregate to assure proper retention of the aggregate.
- Heavy, high-speed traffic tends to dislodge the aggregate from the asphalt.

Because of these limitations, consider using plant-mix surface treatments when the above conditions are anticipated.

Single Surface Treatment

A single surface treatment usually consists of a sprayed application of a bitumen and an aggregate cover one stone thick. Surface treatment may be referred to as a seal coat, armor coat, or carpet coat. A single surface treatment, shown in Figure 9-46, is usually less than 1 inch thick. Surface treatments serve only as an abrasive and weather-resisting medium that waterproofs the base. They are not as durable as bituminous concrete and may require frequent maintenance. Although they are not recommended for airfields, they may be used as an expedient measure. They are particularly suitable for TO construction because they can be laid quickly with a minimum of materials and equipment, constructed in multiple layers with little interference to traffic, and used as the first step in stage construction. Surface treatment will not withstand the action of metal wheels on vehicles, tracked vehicles, or nonskid chains on vehicle wheels. Do not attempt surface treatments when the temperature is below 50° F.

The three requirements for a surface treatment arc as follows:

- The quality of the bitumen must be sufficient to bold the stone without submerging il.
- Sufficient aggregate must be used to cover the bitumen.
- The base course on which the surface treatment is laid must be sufficiently strong to support the anticipated traffic load.

Uniformly graded sand or crushed stone, gravel, or slag may be used for surface treatments. The purpose of the surface treatment dictates the size of aggregate to be selected. For example, coarse sand may be used for scaling a smooth, existing surface. For a badly broken surface, the maximum size of the aggregate should be about 3/4 inch; the minimum size should pass the No. 4 sieve. Surface treatments include the following: dust palliatives, prime coats, and sprayed asphalt with a cover aggregate (single and multiple surface treatments).

Material

RC and MC cutbacks, road tars, rapid-setting emulsions, and asphalt cements may be used for surface treatment. RC cutbacks are most widely used because they evaporate rapidly and the road can be opened to traffic almost immediately after applying the surface treatment. Viscosity grades of the bitumen depend on the size of aggregate used as cover stone. The larger particles of aggregate require a bitumen of higher viscosity so that the bitumen will hold the aggregate. For example, RC-70 or RC-250 may be used with coarse sand for a surface treatment to seal cracks in an otherwise satisfactory surface. For resurfacing a badly cracked or rough surface, RC-800 or RC-3,000 may be used with 3/4-inch aggregate.

To assure uniform distribution, the bitumen should be applied with a bituminous

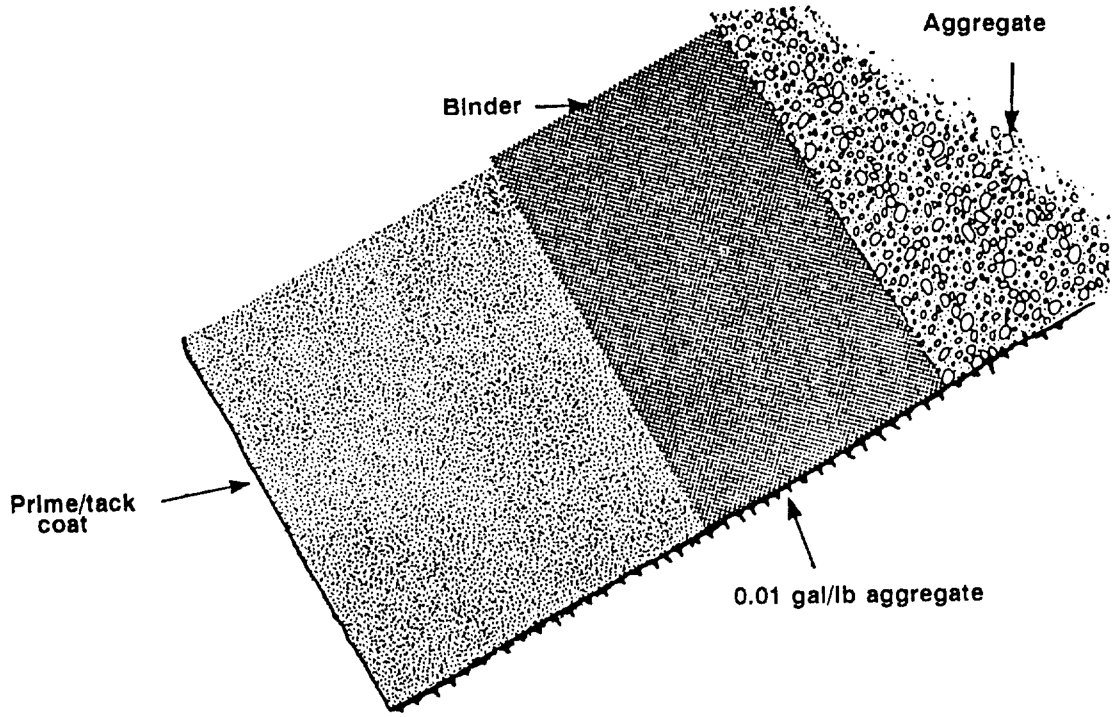


Figure 9-46. Single surface treatment

distributor. The quantity of the bitumen required is based on the average particle size of the cover stone. The bitumen must be sufficient to hold the aggregate in place without leaving a sticky surface. The aggregate must not be completely submerged in the bitumen. One-quarter-inch aggregate should be submerged approximately 30 percent: 3/8-inch aggregate, 32 percent; 1/2-inch aggregate, 35 percent; and 3/4-inch aggregate, 43 percent. Approximately 1 gallon of bitumen is usually used for 100 pounds of aggregate. The recommended rate of bitumen application is given by the following formula:

$$\frac{\text{wt of aggregate (lb)}}{\text{area (yd}^2)} \times \frac{\text{1 gal bitumen}}{\text{100 lb aggregate}}$$
$$= \text{quantity of bitumen in gal/yd}^2$$

Example:

Compute the recommended rate of bitumen application, in gallons per square yard, if 30 pounds of aggregate are required to cover an area of 1.0 square yard.

Solution:

$$\frac{\text{30 lb agg}}{\text{1 yd}^2} \times \frac{\text{1 gal bitumen}}{\text{100 lb agg}} = 0.30 \text{ gal/yd}^2$$

Requirements for a Surface Treatment

In bituminous surface treatments, the unit quantities of bitumen and aggregate can be determined by a test strip, by the specifications of the job, or by adding approximately 1 gallon of bitumen for every 100 pounds of aggregate or 0.1 gallon of bitumen for every 10 pounds of aggregate. The weight of the aggregate, one stone deep, required to cover 1 square yard is determined by spreading the aggregate to be used a depth of one stone over a measured surface, weighing it, and computing the amount in pounds per square yard.

The formula used to determine the quantity of binder material required for a surface treatment is—

$$\frac{L \times W \times AR_B \times AR_A \times LF}{9(ft^2/yd^2)} = Q_b \text{ Gallons}$$

where—

L = length of treated surface in feet
W = width of treated surface in feet
AR_B = application of bitumen in gallons per lb of aggregate (usually 1 gal per 100 lb aggregate or 0.01 gal per lb)
AR_A application of aggregate in lb per square yard
LF = handling loss factor for bitumen
Q_b = quantity of binder material required in gallons
9 = square feet per square yard (ft^2/yd^2) conversion factor

Example:

Compute the required quantity of binder material needed for a single surface treatment. The surface is 1,000 feet long and 12 feet wide. Use a bitumen application rate of 0.01 gal per lb of aggregate and a loss factor of 1.05. The aggregate is 3/8-inch crushed stone with a unit weight of 100 lb/ft^3.

Solution:

Determine the application rate of the aggregate in pounds per square yard.

$$AR_A = 100 \text{ lb/ft}^3 \times (3/8 \text{ in}) \times (1 \text{ ft}/12 \text{ in}) \times (9 \text{ ft}^2/1 \text{ yd}^2)$$

$$= 28.125 \text{ lb/yd}^2$$

Now determine the binder quantity.

$$Q_b = \frac{L \times W \times AR_B \times AR_A \times LF}{9 \text{ ft}^2/\text{yd}^2}$$

$$= \frac{\frac{1{,}000}{\text{ft}} \times \frac{12}{\text{ft}} \times \frac{0.01}{\text{gal/lb}} \times \frac{28.125}{\text{lb/yd}^2} \times 1.05}{9 \text{ ft}^2/\text{yd}^2}$$

$$= 393.75 \text{ or } 394 \text{ gallons}$$

Multiple Surface Treatment

When a tougher, more resistant surface is desired than that obtained with a single surface treatment, a multiple surface treatment may be used. A multiple surface treatment is two or more successive layers of a single surface treatment (as shown in Figure 9-47). Smaller particles of aggregate and correspondingly less bitumen are used for each successive layer. Although multiple surface treatments are usually more than 1 inch thick, they are still considered surface treatments because each layer is usually less than 1 inch and the total surface treatment does not add to the load-carrying capacity of the base.

The first layer of a multiple surface treatment is laid according to instructions previously given for a single surface treatment. Loose aggregate remain ing on the first layer must be swept from the surface so that the layers may be bonded together. As stated previously, the size of the aggregate and the amount of the bitumen will decrease for each successive layer. For the second layer, the bitumen will usually be reduced to one-third or one-half the amount of the first application. The aggregate used in the second application should be approximately one-half the diameter of that used in the first application. The final application of aggregate should be swept clean, if necessary, so that an even layer of aggregate will remain. It should also be rolled with a pneumatic roller so that the aggregate will become embedded in the bitumen. After the surface is rolled and cured, it is ready for traffic. If the multiple surface treatment has been laid on an airfield, loose aggregate must be swept from the surface so that it will not damage the aircraft. Final sweeping is also recommended for roads.

CONSTRUCTION OF SURFACE TREATMENTS USING SPRAYED ASPHALT WITH COVERED AGGREGATE

Weather

Weather conditions are an important factor for success in the construction of sprayed asphalt with covered-aggregate surface treat-

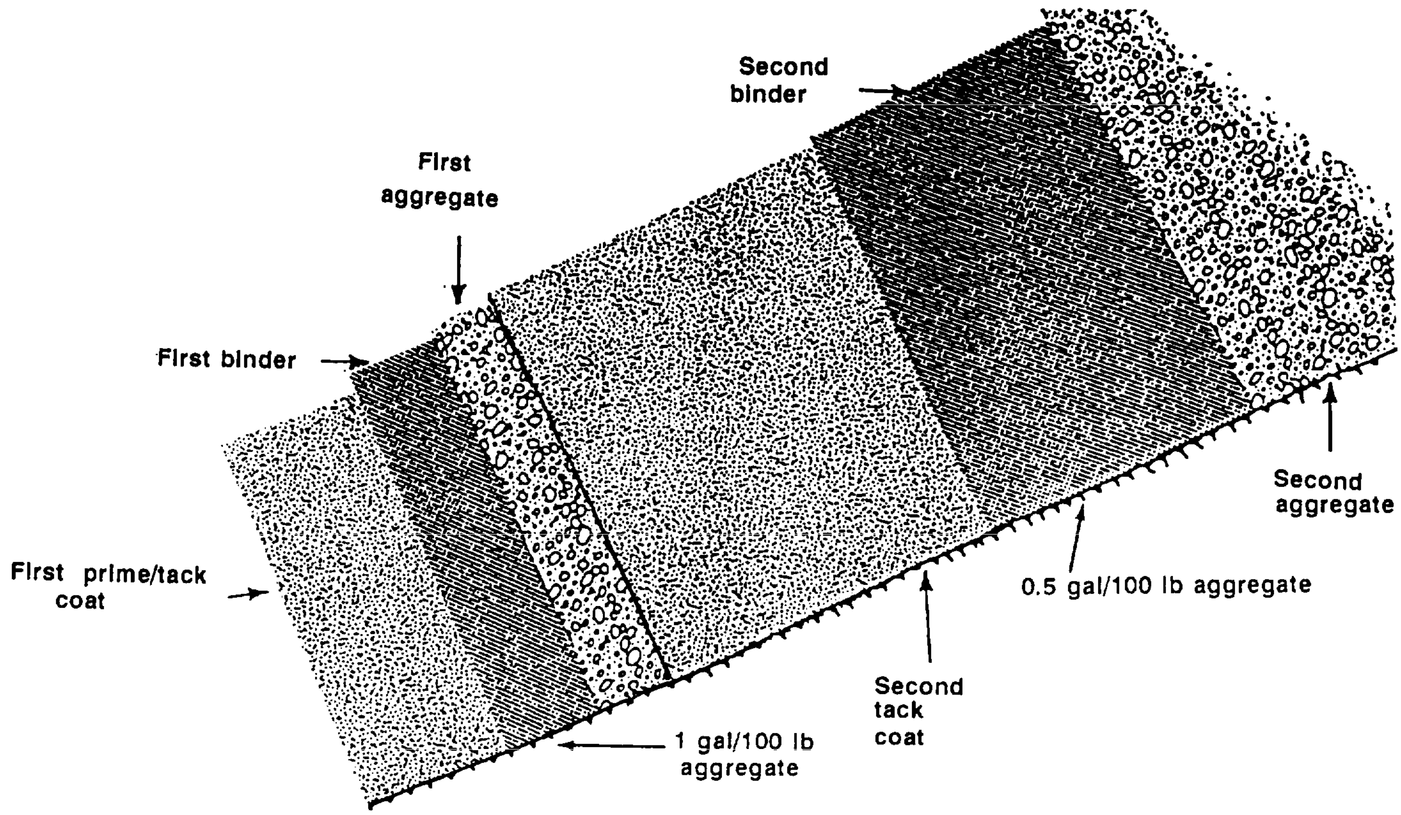


Figure 9-47. Multiple surface treatment

ments and seal coats. For best results in aggregate retention, the pavement temperature should be relatively high during the application of the seal coat and considerably lower before fast traffic is allowed to use the new seal coat. A certain amount of curing, or setting, is required even with the heaviest liquid-asphaltic materials. This curing takes place best when the air temperature is well above 50° F and the relative humidity is low, A survey of surface treatments rated excellent shows more than 85 percent were placed in the hot summer months. Every effort should be made to plan the work for placement in summer weather. After completion of the surface treatment, traffic should be controlled until the surface has cured.

Aggregate

Once an aggregate has been selected for use based upon the desirable characteristics, it is then necessary to determine what quantity of the aggregate will be required for a specific job. When placing a surface treatment with an aggregate cover, the quantity of aggregate required can be determined from the following formula:

$$\frac{L \times W \times AR_A \times LF}{9 \times 2{,}000} = Q_A \text{ Tons}$$

where—

L = length of treated surface in feet
W = width of treated surface in feet
LF = handling loss factor for aggregate (10 percent or 1.10).
AR_A = application rate for aggregate
Q_A = quantity of aggregate in tons

The materials for a multiple surface treatment are determined by the same method as above except that the results are multiplied by the number of treatment passes, The aggregate size (not quantity) must be cut in half for the second layer and each layer thereafter.

Spreading Aggregate. Before the application of asphalt begins, an adequate aggregate spreader should be available and properly adjusted for the aggregate actually to be used. The spray-bar width of the bituminous distributor should be equal to the width of the aggregate being spread in one pass. Normally, this is the width of one traffic lane. An adequate supply of aggregate should be on hand to cover the asphalt that has been spread, without interruption, in the shortest practical time after the asphalt hits the surface. In addition, the aggregate spreader should be filled, in place, and ready to spread aggregate before commencing the asphalt spray. A common fault is to operate the distributor too far ahead of the aggregate spreader.

Aggregate spreaders vary from a simple, controllable gate box attached to the dump truck, to very efficient, self-propelled units which apply the larger-size aggregate on the bottom and the finer on top. The more efficient, self-propelled units are most desirable.

Standard, Hopper-Type Aggregate Spreaders. The standard aggregate spreader shown in Figure 9-48 can handle aggregate which ranges from sand to 1 1/2-inch gravel. The rate and depth of application depend upon the gate opening. The width of spread may be varied from 4 to 8 feet in 1-foot increments. Depending upon the manufacturer, the spreader has either two or four wheels. It hooks on the rear of a 5-ton dump truck and the truck backs up. This allows the aggregate to be spread on the bitumen ahead of the truck tires, thus preventing pickup of the bitumen. As a safety precaution, men should not be allowed to stand on the aggregate, either in the truck or in the spreader, at any time.

Rolling. Pneumatic-tire rollers should be used for surface-treatment construction. Steel-wheeled rollers are not recommended for rolling, but if they are all that is avail-

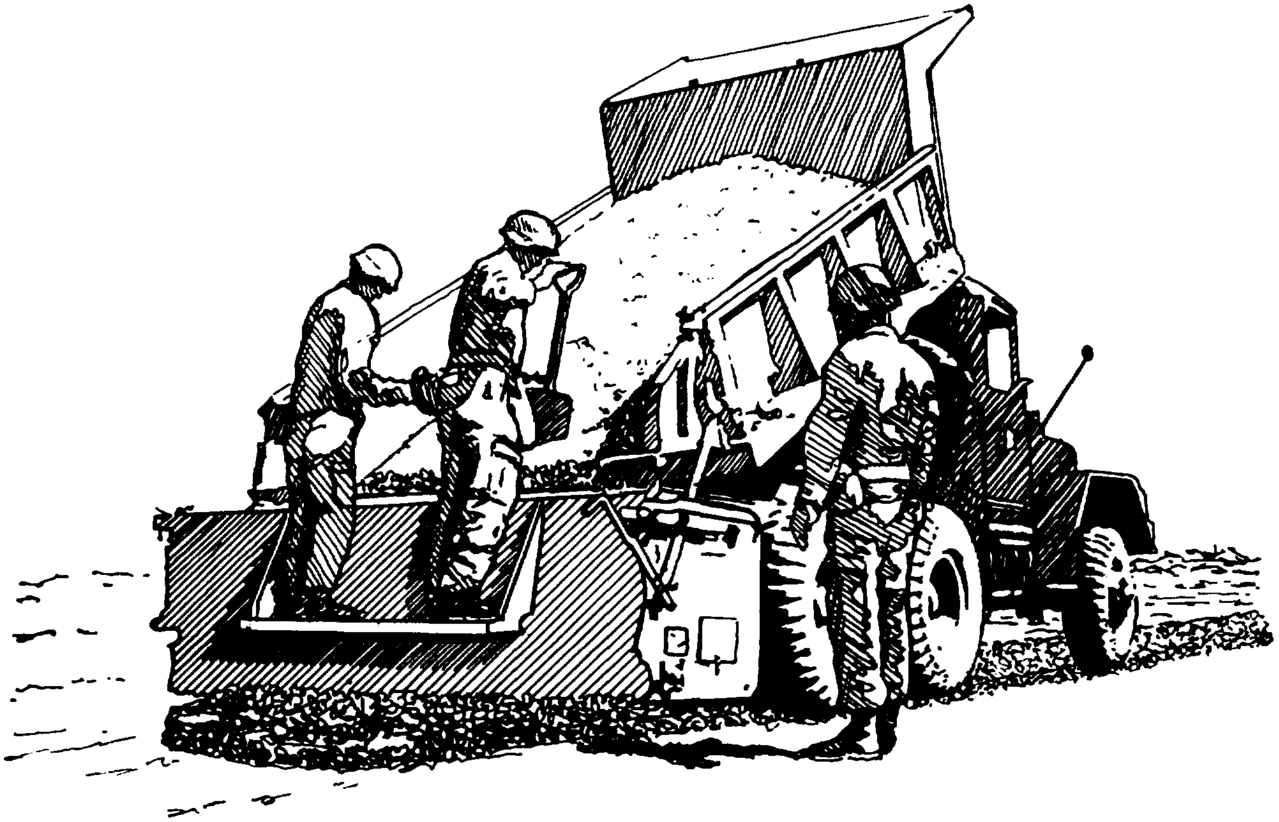


Figure 9-48. Typical hopper-type aggregate spreader

able, rollers should not be so heavy as to crush the aggregate particles. Pneumatic-tire rollers are essential to firmly embed the aggregate into low areas or graded deformities that would normally be bridged over by use of steel-wheel rollers and to produce conformity across the width of the roadway, particularly over the outer quarters of the surface where there is the least traffic. The rollers should be heavy enough to properly imbed the aggregate, but rolling should be stopped as soon as crushing becomes evident. When double or triple surface treatments are used, each course should be rolled before subsequent applications of asphalt. When an asphalt fog seal is used, it should be applied after the rolling is completed.

Traffic Control. It is extremely important that traffic be controlled to prevent loss of aggregate. One method of controlling traffic is to form a single line of traffic behind a pilot vehicle with a red flag between stops at each end of the work area.

The Asphalt Distributor

The asphalt distributor is the key piece of equipment in the construction of surface treatments. It consists of a truck (or a trailer) equipped with a mounted, insulated tank with a heating system, usually oil-burning, with direct heat from the flue passing through the tank. It is further supplied with a power-driven pump, suitable to handle products ranging from light, cold-application liquid asphalt to heavy asphalt cements heated to spraying viscosity. Attached to the back end of the tank is a system of spray bars and nozzles through which the asphalt is forced under pressure onto the construction surface. The construction of the spray bars should be such that there will be full circulation of the asphalt through the bar when not spraying. These spray bars should have a minimum application width of 8 feet. On larger equipment, the spray bars will cover as much as a 24-foot width in one pass when equipped with a suitable capacity pump. The height of the spray bar determines the type of coverage: single lap, double lap, or triple lap, as shown in Figure 9-49. A suitable

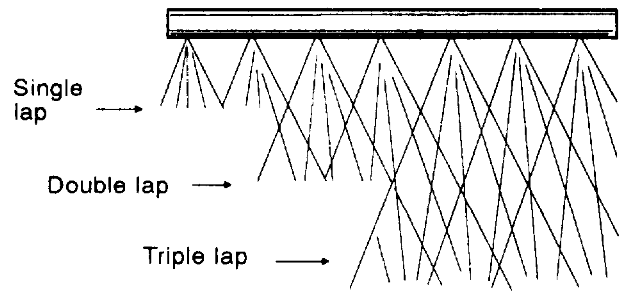


Figure 9-49. Spray bar coverage

thermometer should be installed in the tank to readily ascertain the temperature of the contents. A connection should be available to attach a hose for a single- or double-nozzle outlet to cover areas not reached by the spray bars or as a means of forcing a stream of asphalt to a desired point as in subsealing rigid, slab pavements. Distributors are made in sizes ranging from 800- to 4,000-gallon capacity. Some maintenance distributors as small as 400 gallons are available.

It is essential that the distributor be capable of distributing the asphalt uniformly over the surface to be treated. For best results in surface treatments, observe the following points:

- Maintain uniform pressure and temperature on all spray nozzles. The fan of the spray from each nozzle must be uniform and set at the proper angle with the spray bar (according to the manufacturer's instructions) so that the spray fans do not interfere with each other.
- Maintain the spray bar at the proper height above the road surface (according to the manufacturer's instructions) 10 provide complete and uniform overlap of the spray fans.
- Ensure that the distributor road speed is uniform.
- Before beginning work, check the spread of the distributor spray bar.

Valve action should be instantaneous, both in opening and closing. The spraying operation should be inspected frequently to ensure that the nozzles are the proper height from the road surface and working fully. An otherwise good job may be spoiled if one or more spray nozzles are clogged.

MAINTENANCE AND REPAIR OF BITUMINOUS SURFACES

Inspection

Maintenance patrols frequently inspect bituminous pavements for early detection of failures. Small defects quickly develop into larger ones under the effects of weather and traffic and may result in pavement failure unless promptly corrected. Minor repairs are quickly made with small crews and hand-tools, with a minimum interruption of traffic. Larger bituminous repairs require more time, personnel, and equipment, and may result in interference with traffic or, in extreme cases, require construction of detours to avoid complete stoppage.

Patches

All patches should be trimmed square or oblong with straight, vertical sides running parallel and perpendicular to the centerline of the traffic area.

Temporary Repairs

Any stable material may be used for temporary repairs in combat areas or where suitable material is not available and the traffic area must be patched to keep traffic moving. Good-quality soil and masonry or concrete rubble are suitable for this purpose. All such patches must be thoroughly compacted and constantly maintained with replacement material. More permanent patching should be accomplished as soon as possible.

Maintenance of Shoulders

Shoulders are bladed to facilitate drainage of rainwater from the surface. Ruts and washouts are filled. Shoulder material is kept graded flush against pavement edges to restrict seepage of water to the subgrade and to prevent breaking of the pavement edge by traffic driving off the pavement onto the shoulder. Material displaced from shoulders is replaced with new material as required.

GENERAL ROAD STRUCTURAL DESIGN

TO roads will normally be designed as un-*surfaced, aggregate, or flexible-pavement* systems. The design procedure for each type first involves assigning a class (A - G) designation to the road based upon the number of vehicle passes per day. A *design category* (I - VII) is then assigned to the traffic based upon the composition of the traffic. A *design index* (1-10) is determined from the design category and road class. This design index is used to determine either the CBR strength requirements of the unsurfaced roads or the thickness of the aggregate surface or flexible-pavement system required above a soil with a given CBR strength.

NOTE: As mission requirements change (a forward-area road becomes a rear-area road), the road class and design index will change. The design procedures outlined in this section allow for the easy upgrading of roads as the mission changes. This ensures the ability to easily convert an unsurfaced road to an aggregate-surfaced road to a flexible-pavement road without major changes in the design procedure.

CLASSES OF ROADS

The classes of roads vary from A to G. Selection of the proper class depends upon the traffic intensity and is determined from Table 9-8.

Table 9-8. Road-class selection criteria

Road Class	Number of Vehicles Per Day
A	10,000
B	8,400-10,000
C	6,300-8,400
D	2,100-6,300
E	210-2,100
F	70-210
G	Under 70

Table 9-9. Pneumatic-tired traffic categories based on traffic composition

Traffic Category	Percentage of total traffic for vehicle groups		
	Group 1	Group 2	Group 3
Category I	≥ 99%	≤ 1%	
Category II	≥ 90%	≤ 10%	
Category III	≥ 84%	≤ 15%	≤ 1%
Category IV	≥ 65%	≤ 25%	≤ 10%
Category IVA	Any amount	> 25%	> 10%

DESIGN INDEX

The design of roads will be based on a design index representing all traffic expected to use the road during its life. The design index is based on typical magnitudes and compositions of traffic reduced to equivalents in terms of repetitions of an 18,000-pound, single-axle, dual-wheel load. For designs involving pneumatic-tired vehicles, traffic is classified into three groups, as follows:

- Group 1. Passenger cars and panel and pickup trucks.
- Group 2. Two-axle trucks (excluding pickup trucks).
- Group 3. Three-, four-, and five-axle trucks.

Traffic composition will then be grouped into the following categories (summarized for easy reference in Table 9-9):

Ž Category I. Traffic composed primarily of passenger cars and panel and pickup trucks (Group 1 vehicles) but containing not more than 1 percent two-axle trucks (Group 2 vehicles).

- Category II. Traffic composed primarily of passenger cars and panel and pickup trucks (Group 1 vehicles), and containing as much as 10 percent two-axle trucks (Group 2 vehicles). No trucks having three or more axles (Group 3 vehicles) are permitted in this category.

Ž Category III. Traffic containing as much as 15 percent Group 2 but with not more than 1 percent of the total traffic composed of trucks having three or more axles (Group 3 vehicles).

- Category IV. Traffic containing as much as 25 percent Group 2 but with not more than 10 percent of the total traffic composed of trucks having three or more axles (Group 3 vehicles).
- Category IVA. Traffic containing more than 25 percent Group 2 or more than 10 percent trucks having three or more axles (Group 3 vehicles).

The design index to be used, if designing a road for the usual pneumatic-tired vehicles, will be selected from Table 9-10 based on

Table 9-10. Design index for pneumatic-tired vehicles

Class	Design Index: Category I	Category II	Category III	Category IV	Category IVA
A	2	3	4	5	6
B	2	2	4	5	6
C	2	2	4	5	6
D	1	2	3	4	5
E	1	2	3	4	5
F	1	1	2	3	4
G	1	1	1	2	2

the road class (A to G) and category (I to IVA) .

Where tracked vehicles or forklift trucks are involved in the traffic composition, the following three considerations apply:

- Tracked vehicles not exceeding 15,000 lb and forklift trucks not exceeding 6,000 lb are treated as two-axle trucks (Group 2 vehicles) in determining the design index.

- Tracked vehicles exceeding 15,000 pounds but not 40,000 pounds and forklift trucks exceeding 6,000 pounds but not 10,000 pounds are treated as three-axle trucks (Group 3 vehicles) in determining the design index.

- Traffic composed of tracked vehicles exceeding 40,000 pounds and forklift trucks exceeding 10,000 pounds has been divided into the three categories shown in Table 9-11.

Table 9-11. Tracked-vehicle and forklift traffic categories

Category	Vehicle Weight, Pounds: Tracked Vehicles	Forklift Trucks
V	40,001-60,000	10,001-15,000
VI	60,001-90,000	15,001-25,000
VII	Over 90,000	Over 25,000

Roads sustaining traffic of tracked vehicles weighing less than 40,000 pounds and forklift trucks weighing less than 10,000 pounds will be designed according to the pertinent class and category from Table 9-10, page 9-59. Roads sustaining traffic of tracked vehicles heavier than 40,000 pounds and forklifts heavier than 10,000 pounds will be designed according to the traffic intensity and category from Table 9-12.

NOTE: DO NOT include any wheeled vehicles in the total number of tracked vehicles and forklifts when using Table 9-12.

Design Life

The life assumed for design is less than or equal to 5 years. For a design life of more than 5 years, the design indexes in Tables 9-10 and 9-12 must be increased by one. Design indexes below 3 need not be increased.

Entrances, Exits, and Segments

Regardless of the design class selected for hardstands, special consideration should be given to the design of approach roads, exit roads, and other heavy-traffic areas. Failure or poor performance in these channelized traffic areas often has greater impact than localized failure on the hardstand itself.

Since these areas will almost certainly be subjected to more frequent and heavier loads than the hardstand, the design index used for the primary road should be used for entrances and exits to the hard stand.

Table 9-12. Design index for tracked vehicles and forklifts

Traffic Category	Number of Vehicles per Day (or Week as Indicated): 500	200	100	40	10	4	1	1 Per Week
V	6	6	6	6	5	5	5	--
VI	9	8	7	6	6	6	5	5
VII	10	10	9	9	8	7	6	5

NOTE: If number of vehicles is between values, round up to the next higher number.

In the case of large hardstands having multiple uses and multiple entrances and exits, consideration should be given to partitioning and using different classes of design. The immediate benefits that would accrue include economy through elimination of excessive design in some areas and better organization of vehicles and equipment.

UNSURFACED ROADS

An unsurfaced road is one in which the in-place natural soil or borrow soil is used as the road surface. Typically, the construction effort required includes only clearing and grubbing followed by scarifying grading, and compacting.

Designing unsurfaced roads consists of the following steps:

1. Estimate the number of passes of each type of vehicle expected to use a road on a daily basis.

2. Select the proper road class based upon the traffic intensity from Table 9-8, page 9-59.

3. Determine the traffic category based upon the traffic composition criteria shown in Table 9-9, page 9-59.

4. Determine the design index from Table 9-10, page 9-59, or Table 9-12.

5. Read the soil-surface strength required to support the design index from Figure 9-50.

6, Check whether the design (compacted) CBR value of in-place soil exceeds the CBR value required. If the in-place design CBR value is less than the CBR required, the engineer must decide whether 10 decrease the design life or improve the in-place soil to meet the CBR required by one of the following methods: soil stabilization, soil treatment, or placing aggregate.

7. Determine the required unsurfaced-soil thickness. Given the required CBR from step 6 and the design index from step 4, the required unsurfaced-soil thickness or depth of compaction can be obtained from Figure 9-51, page 9-62.

Example (Unsurfaced-Road Design):

To illustrate the procedure for determining soil-surface strength requirements, assume that an unsurfaced road is to be used one year. The road will be subjected to the fol - lowing average daily traffic:

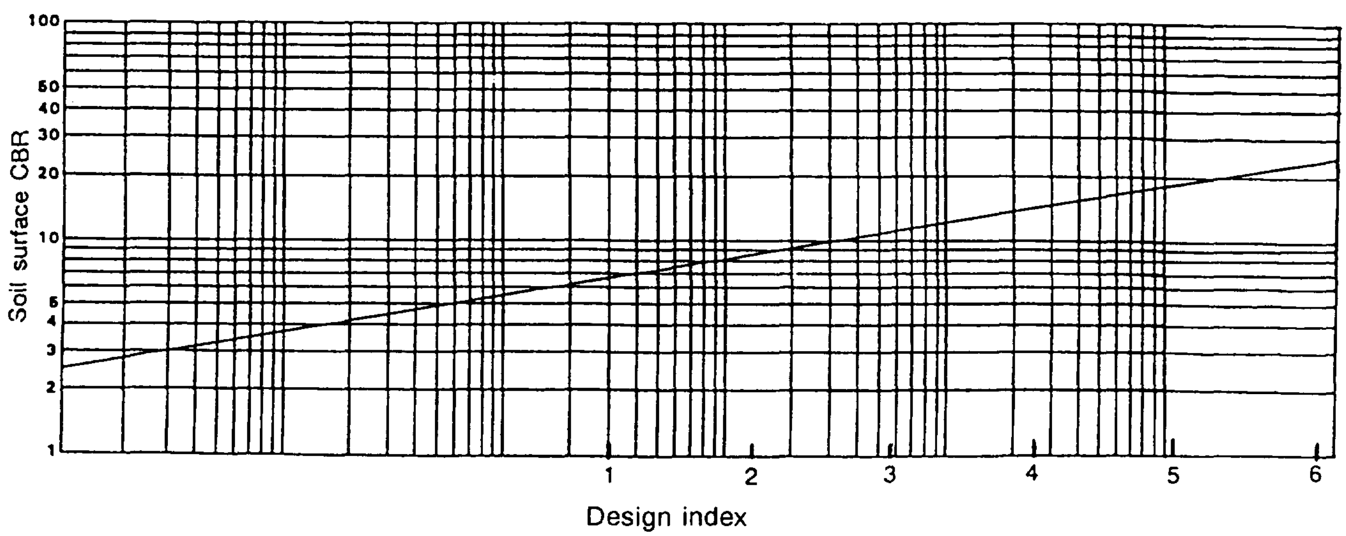


Figure 9-50. Unsurfaced-soil strength requirements

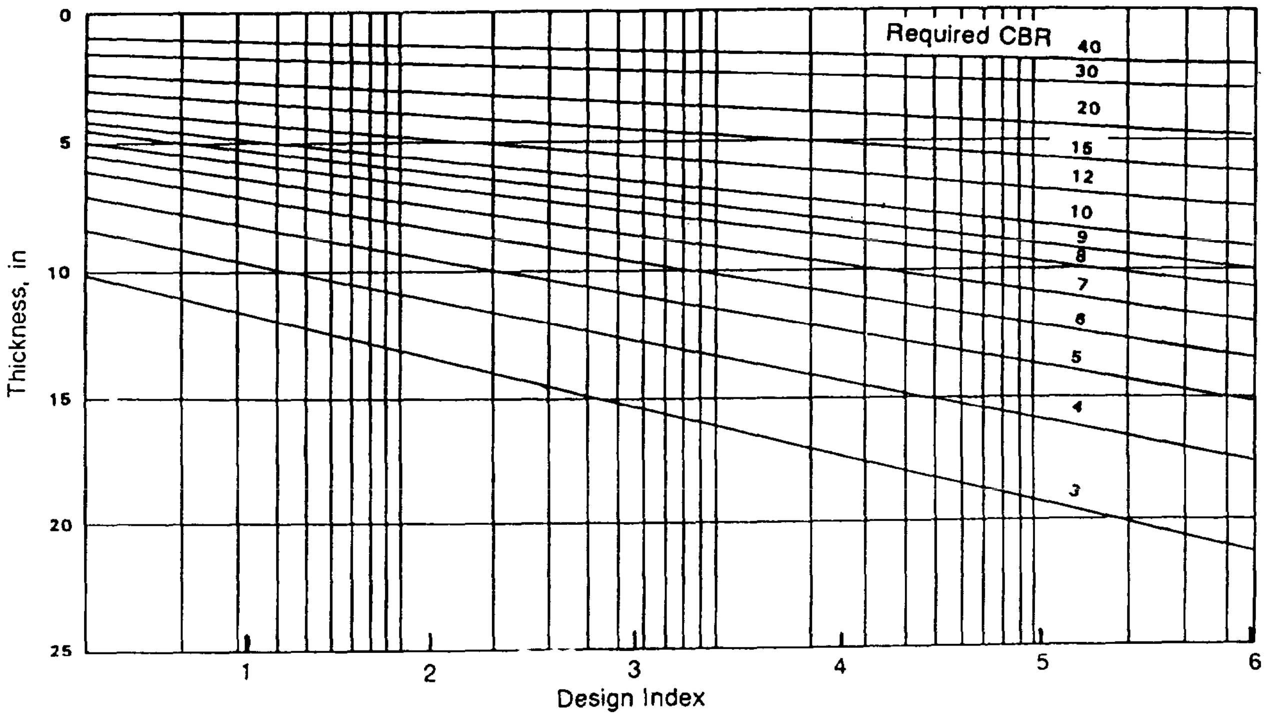


Figure 9-51. Unsurfaced-soil thickness requirements

Vehicle	Average Daily Traffic
M998 HMMWV (two axle)	180
M929 5-ton dump truck (three axle)	50

Solution:

1. Determine the average daily traffic (given).

2. Select road class E from Table 9-8, page 9-59, based upon 230 vehicles per day.

3. Select traffic category IVA, based upon the percentage of Group 3 vehicles.

4. The design index is 3 from Table 9-10, page 9-59.

5. The soil-surface strength reequirement for a design index of 3 is 10.8 CBR.

6. Check to ensure the design CBR value of the in-place soil exceeds the 10.8 CBR required. If not, consider using either soil stabilization or an aggregate road.

7. Determine the required unsurfaced-soil thickness from Figure 9-51. Given a design index of 3 and a required CBR of 10.8, the required thickness from Figure 9-51 is 6 inches.

AGGREGATE-SURFACED ROADS

The design of aggregate-surfaced roads is similar to the design of unsurfaced roads. However, in aggregate-surfaced roads. Layers of high-quality material are placed on the natural subgrade to improve its strength.

Materials

Materials used in aggregate roads must meet the requirements as stated in Chapter 5 of this manual and in the following paragraphs. The materials should have greater strength than the subgrade and should be placed so that the higher-quality material is placed on top of the lower-quality material.

Table 9-13. Compaction criteria and CBR requirements for an aggregate road structure

CBR requirements	Layer	Compaction requirements
50, 80, 100	Base course	100 - 105%
20 - 50	Subbase course	100 - 105%
0 - 20	Select material	Cohesive: 90 - 95% Cohesionless: 95 - 100%
	Design subgrade (SCIP)	Cohesive: 90 - 95% Cohesionless: 95 - 100%
	Uncompacted subgrade	

NOTES:
1. All lifts in a road design must be at least 4 inches.
2. A cohesive soil is one with a PI above 5.
3. A cohesionless soil is one with a PI of 5 or less.
4. Percent compaction Is compared to the CE 55 curve according to ASTM D1557.

Select and Subbase Materials

Select and subbase materials used in aggregate and flexible-pavement roads must meet the requirements of Table 9-13.

Base Course

Only good-quality materials should be used in base courses of heavy-duty aggregate roads. Specifications for graded, crushed aggregate; lime rock; and stabilized aggregate may be used without qualification for design of roads, streets, and parking areas.

Specifications for dry and water-bound macadam base courses may be used for design of heavy-duty roads only when the following two conditions arc satisfied:

- The cost of the dry or water-bound macadam base does not exceed the cost of a stabilized, aggregate base course.
- The construction unit has the equipment and expertise to place a macadam surface (wet or dry) to acceptable standards of smoothness and grade.

Design CBR of Base Course. Where subbase material is used for base-course construction, the base course CBR must be at least 50 and the material must conform to the gradation and Atterberg limit requirement for a 50-CBR subbase as shown in Table 9-14. Otherwise, the design CBR of the base course must meet the requirements of Table 9-15, page 9-64.

Gradation Requirements. Gradation requ irements for aggregate-surfaced roads and for macadam base courses are given in Chapter 5.

Table 9-14. Maximum permissible values for subbases and select materials

		Maximum Permissable Values for Gradation and Atterberg Limits				
		Gradation Requirements Percent Passing				
Material	Maximum Design CBR	Size In	No 10 Sieve	No 200 Sieve	Liquid Limit	Plasticity Index
Subbase	50	2	50	15	25	5
Subbase	40	2	80	15	25	5
Subbase	30	2	100	15	25	5
Select material	20	3	--	--	35	12

Table 9-15. Assigned CBR ratings for base-course materials - aggregate-surfaced road

Number	Type	Design CBR
1	Graded crushed aggregate	100
2	Water-bound macadam	100
3	Dry-bound macadam	100
4	Lime rock	80
5	Stabilized aggregate	80
6	Soil cement	80
7	Sand shell or shell	80

NOTE: It is recommended that stabilized-aggregate base-course material not be used for tire pressures in excess of 100 psi.

Thickness Requirements. Thickness requirements for aggregate-surfaced roads are determined from Figure 9-52, page 9-65, for a given soil strength and design index. The minimum thickness requirement will be 4 inches.

Figure 9-52 provides the thickness of aggregate based on CBR and design index. The thickness determined from the figure may be constructed of compacted granular fill for the total depth over the compacted subgrade or in a layered system of granular fill with subbases for the same total depth. The layered section must be checked to ensure that an adequate thickness of material is used to protect the underlying layer based on the CBR of the underlying layer. The granular fill may consist of base, subbase, and select material, provided the top 4 inches meet the gradation requirement ts

Compaction Requirements

Cornpaction requirements for the subgrade and granular layers are expressed as a percent of maximum CE 55 density as determined by using MIL-STD-621 Test Method 100.

Normal Subgrades. Compact the subgrade to 90-percent CE 55 density for cohesive soils (PI>5; LL>25) and 95-percent for cohesionless soils (PI≤5; LL≤25).

NOTE: It may be possible to compact the subgrade material to the required density in its natural state. However, in cases where the moisture content is out of the specification range, it may be necessary to scarify the soil (thereby aerating the soil to adjust the moisture content) and then compact. This process is called scarify and compact in place (SCIP).

Special Subgrades. The procedures for compacting subgrades of clays that lose strength when remolded, silts that become quick when remolded, and soils with expansive characteristics are described in Chapter 5 of this manual.

Subgrade in cuts and fills must have densities equal to or greater than the values shown in Table 9-13, page 9-63, except that fills will be placed at no less than 95 percent density for cohesionless soils or 90 percent for cohesive soils.

Where this is not the case for cuts, the subgrade must (1) be compacted from the surface to meet the densities shown; (2) be removed and replaced, in which case the requirements given above for fills apply; or (3) be covered with sufficient select material, subbase, and base so that the uncompacted subgrade is at a depth where the in-place densities are satisfactory.

Depth of Compaction. Compact the subgrade to the depth specified in Table 9-16, page 9-66, for cohesive soils (PI > 5) and Table 9-17, page 9-66, for cohesionless soils (PI ≤ 5).

NOTE: When depth of compaction from Table 9-16 or 9-17 is not feasible or attainable in cut sections, perform a 6-inch SCIP and continue design based on the uncompacted subgrade CBR.

Select Materials. The procedure is the same as for the subgrade.

Subbase. Compact the subbase to not less than 100-percent CE 55 density.

Base Course. Compact the base course to the maximum degree practicable but not less than 100-percent CE 55 density.

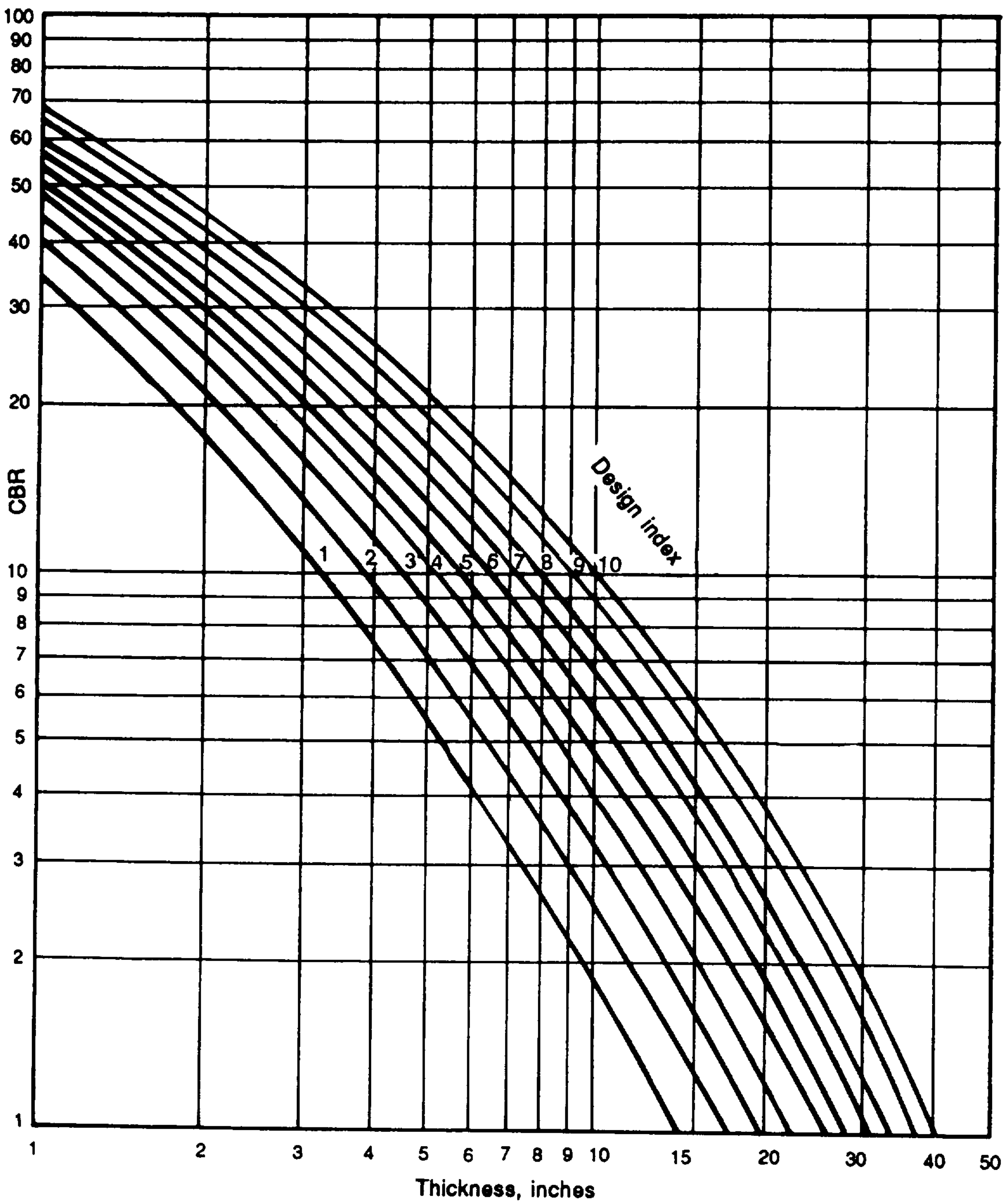


Figure 9-52. Design curves for aggregate-surfaced roads.

Table 9-16. Required depth of subgrade compaction for roads, cohesionless soils

Percent Compaction	Depth of Compaction (in inches) for Indicated Design Index									
	1	2	3	4	5	6	7	8	9	10
95-100[1]	7	8	10	11	12	14	15	17	19	21
90-95[2]	10	12	14	16	18	20	22	24	28	30

[1]Normally used.
[2]Use if on-site test strip results show the 95-100 range is not attainable.

Table 9-17. Required depth of subgrade compaction for roads, cohesive soils (PI>5)

Percent Compaction	Depth of Compaction (in inches) for Indicated Design Index									
	1	2	3	4	5	6	7	8	9	10
90-95[1]	6	7	8	9	10	11	12	13	15	17
95-100[2]	6	6	6	6	7	7	8	9	10	11

[1]Normally used.
[2]Use if on-site test strip results show these ranges are attainable, and shear failure is unlikely..

Design Steps for Aggregate-Surfaced Roads

1. Estimate the number of passes of each type of vehicle expected to use the road on a daily basis.
2. Select the proper road class based upon the traffic intensity from Table 9-8, page 9-59.

3. Determine the traffic category based upon the traffic-composition criteria given in Table 9-9, page 9-59.

4. Determine the design index from Table 9-10, page 9-59, or Table 9-12, page 9-60.

5. Check soils and construction aggregates using standard criteria in Tables 5-4, page 5-12; 9-14, page 9-63; and 9-15, page 9-64.

6. Determine the depth of compaction for the subgrade soil from Table 9-16 or 9-17,

7. Determine the total road-structure thickness and cover requirements.

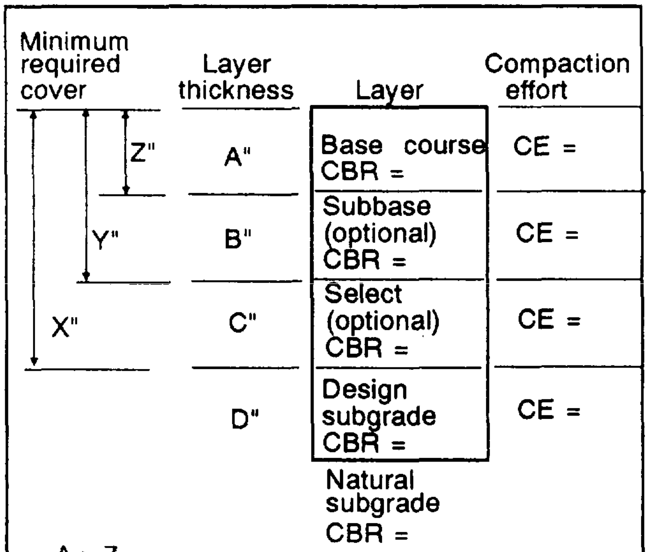
a. Enter Figure 9-52, page 9-65, for each layer of soil or aggregate with the following information:

—Design index.
—Design CBR values for subgrade, select, and subbase materials.

b. Determine the minimum cover thickness, in inches, for each layer of the aggregate road structure.

8. Determine the required percent compaction in terms of CE 55 for each layer from Table 9-13, page 9-63.

9. Draw the section of the aggregate road structure, as shown following.



A ≥ Z
B = Y - A or 4", whichever is greater
C = X - Y or 4", whichever is greater
D = Depth of compacted SCIP
X = minimum cover over compacted subgrade (Figure 9-52, page 9-65)
Y = minimum cover over select (Figure 9-52)
Z = minimum cover over subbase (Figure 9-52)

Total thickness above compacted subgrade = A + B + C

NOTES:

1. All layer thicknesses should be rounded up to the next full inch for construction purposes.

2. After all possible design sections are determined, the final section used should be determined on the basis of an economic analysis.

Example (Aggregate-Road Design):

An aggregate-surfaced road is to be used for two years. The road will be subjected to—

Vehicle	Average Daily Traffic
M998 HMMWV	10
M929 5-ton dump truck (dual axle)	25
M729 combat engineer vehicle (CEV) (60-ton tracked vehicle)	35

Available material CBR:
Natural subgrade = 5 (clay, PI = 15)
Design (compacted) subgrade = 8
Clean sand subbase = 30
Lime rock = 80; meets gradation requirement for maximum size aggregate of 1".

Solution:

1. Number of daily passes = 70 (given).

2. Select road class F from Table 9-8, page 9-59, based upon average daily traffic of 70.

3. Select traffic category VII from information previously given, based upon the presence of the 60-ton tracked vehicle.

4. Select design index of 9 from Table 9-12, page 9-60. **NOTE: You must round the average daily tracked-traffic value of 35 to the next higher value (40) in Table 9-12.**

5. Clean sand CBR 30. (Suitable for subbase CBR 30.) Crushed rock CBR 80. (Suitable for base course.)

6. Depth of compaction based on CBR 5 for compacted subgrade is 15 inches. (See Table 9-17.

7. Determine the road-structure thickness required to support a design index of 9.

a. First, look at the required road thickness if the subgrade was not compacted to the design CBR. In this case, the natural subgrade CBR 5 is used in Figure 9-52. This results in a required total thickness of 16 inches, as shown.

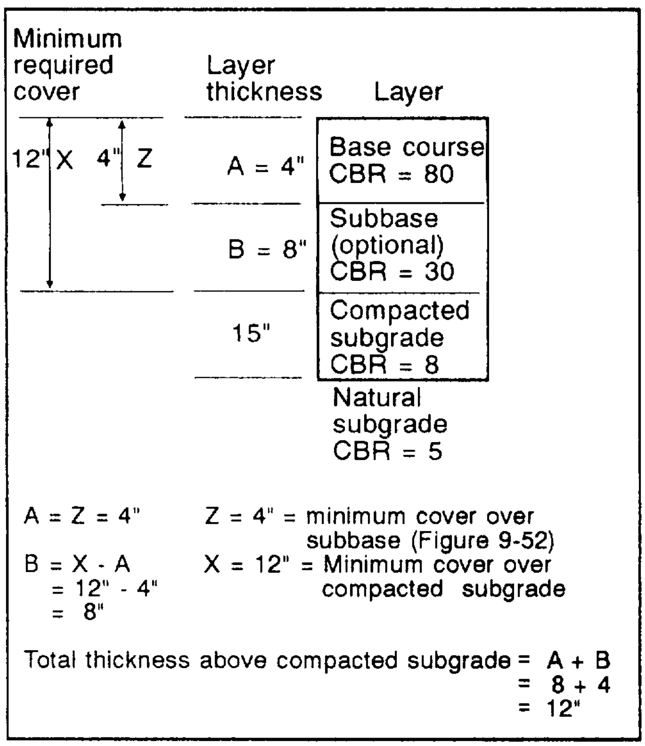
16 inches of cover
Natural subgrade CBR=5

b. Now, look at the required thickness when the subgrade is compacted, In this case, the design subgrade CBR = 8 is used in Figure 9-52, resulting in a required total thickness of 12 inches, as shown.

12 inches of cover
15 inches design subgrade CBR = 8
Natural subgrade CBR = 5

Notice how compacting the subgrade greatly reduces the required thickness of the cover material. This is why the subgrade is **always** compacted.

c. Finally, look at the total thickness and required cover for each layer when the subgrade is compacted and a clean sand subbase with CBR 30 is used. First, the design subgrade CBR 8 is used in Figure 9-52, page 9-65, to determine the 12-inch total thickness required above the compacted subgrade. Next, the clean sand CBR 30 is used in Figure 9-52 to determine the required cover of 4 inches above the subbase. This results in the section, as shown.



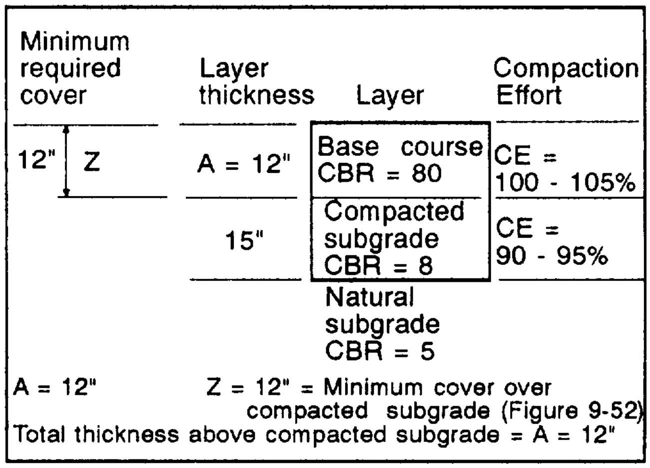
Notice how the addition of the clean sand subbase reduces the required thickness of the mom expensive lime rock.

8. The required percent compaction of each layer is determined from Table 9-13, page 9-63, as follows:

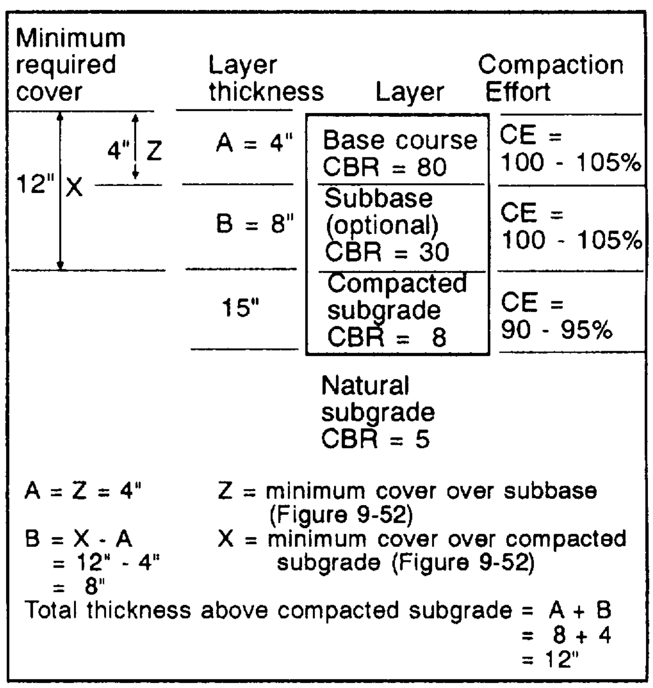
Crushed rock base course: 100-105 percent
Clean sand subbase course: 100-105 percent
Compacted subgrade-since the PI = 15, it is a cohesive soil: 90-95 percent

9. Draw the section of the aggregate road structure. Since two sections were designed, one with a subbase and one without, both should be drawn.

a. First, design the section without the subbase layer.



b. Now, the design the section with the subbase layer.



Given that the base-course material is more expensive than the clean sand subbase, section b would be the most economical design. Note, however, that all possible design sections for the available materials must be evaluated economically. There may be rare instances where the subbase material may be more expensive than the base course. In that case, only the base course would be used.

BITUMINOUS PAVEMENTS

Bituminous-, or flexible-, pavement designs permit the maximum use of readily available local construction materials. They are easier to construct and upgrade than rigid pavement designs. Thus, they permit greater flexibility in responding to changes in the tactical situation.

Flexible-pavement design procedures are different from airfield design procedures. This chapter is limited to flexible-pavement designs for roads. Chapter 12 of FM 5-430-00-2/AFPAM 32-8013, Vol 2 covers airfield flexible-pavement designs. TM 5-822-6 covers rigid-pavement designs.

Pavement Types and Uses

The descriptions, uses, advantages, and disadvantages of bituminous pavements and surfacing presented in TM 5-337 are applicable to TO construction except as modified in the following paragraphs. However, when surfacing for steel treads is necessary, use an asphalt cement with a penetration grade of 50-60 or 60-70, depending on the climate or season.

Special consideration must also be given to the design and construction of bituminous pavements that will be subjected to traffic of tanks and solid, rubber-tired vehicles. Most often the number of passes of tanks and solid, rubber-tired vehicles governs the bituminous-pavement design.

Hot-Mix, Bituminous-Concrete Pavements. Dense-graded, hot-mix. bituminous-concrete mixtures are well-suited for paving heavy-duty traffic roads with volumes of 3,000 vehicles or more per day. Where conditions warrant, use these mixtures to pave roads having traffic volumes of less than 3,000 vehicles per day. Select exact percentages of bituminous materials on the basis of design tests described in TM 5-337.

Cold-Laid, Bituminous-Concrete Plant Mix. Where hot-mix, bituminous-concrete mixtures are not available, use cold-plant, bituminous concrete to pave areas subject to pneumatic-tired traffic only.

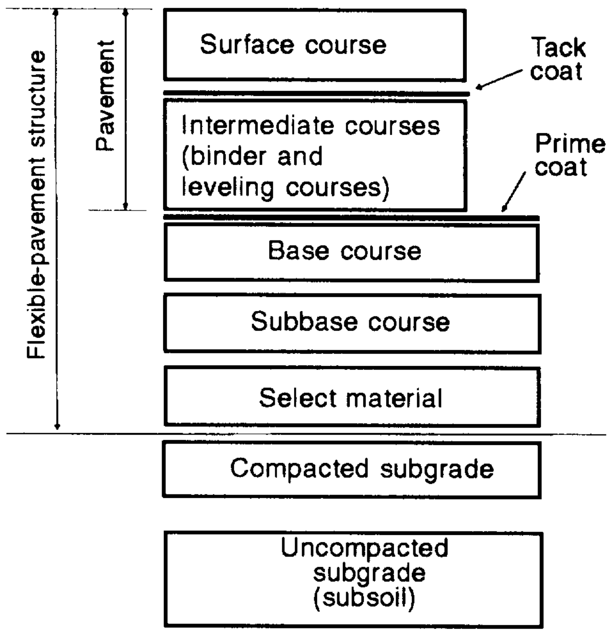
Sheet Asphalt, Stone-Filled Sheet Asphalt, Sand Asphalt, or Sand-Tar Mixes. Fine-aggregate mixes may be used for binder and surface courses of roads with traffic volumes of 2,000 or fewer vehicles per day when sand or other suitable fine aggregates are the only aggregates available. These mixes should not be used as surface or binder courses for roads or industrial-use pavements designed for solid, rubber-tired or steel wheels. In all cases, mixtures made with these aggregates should conform to the criteria for low-pressure tires (100 psi or less), based on laboratory tests.

Penetration Macadams. Do not use penetration macadam for paving any areas subject to traffic from tracked vehicles.

Bituminous Road Mix. Use road mix as a wearing course for TO roads or as the first step in stage construction for more permanent roads. When the existing subgrade soil is suitable or satisfactory aggregates are nearby, road mixing saves time in handling and transporting aggregates as compared with plant mixing. When properly designed and constructed, the quality of road mix approaches that of cold-laid plant mix. Road mix is used for binder and surface courses. It is generally considered inferior to plant-mix pavements manufactured in standard plants because of the less accurate control.

Flexible-Pavement Structure

A typical flexible-pavement structure is shown in Figure 9-53, page 9-70, and illustrates the terms used to refer to the various layers.



NOTE: Not all layers and coats are present In every flexible-pavement structure. Intermediate courses may be placed In one or more lifts. Tack coats maybe required on the surface of each intermediate course.

Figure 9-53. Flexible-pavement design

A bituminous pavement may consist of one or more courses depending on stage construction features, job conditions, and the economical use of materials. The pavement should consist of a surface course, an intermediate (binder) course, and a leveling course, when needed. These should be thick enough to prevent displacement of the base course because of shear deformation, to provide long life by resisting the effects of wear and traffic abrasion, to be waterproof, and to minimize differential settlements.

Sources of Supply

If time and conditions permit, investigate subgrade conditions: borrow areas; and all sources of select materials, subbase, base, and paving aggregates before designing the pavement. In determining subgrade conditions in cut sections of roads, conduct test borings deeper than the frost penetration depth. The minimum boring should never be less than 4 feet below the final grade.

Materials

Materials used in flexible pavements must meet the requirements as stated in Chapter 5 and in the following paragraphs:

Select Materials and Subbase

Select materials and subbases used in bituminous pavements must meet the same requirements as for aggregate-surfaced roads as indicated in Table 9-14, page 9-63.

Base Course

The base course used in bituminous pavements must meet the same requirements as for aggregate-surfaced roads as indicated previously. except as noted below.

Design CBR of Base Course. Where subbase material is used for base construction, the base course CBR must be at least 50 and the material must conform to the Atterberg limit requirement for a 50-CBR subbase as shown in Table 9-14. Otherwise, the design CBR of the base course must meet the requirements of Table 9-18.

Base Course Gradation Requirements. The gradation requirements of the base course are as indicated in Chapter 5 of this manual. The base course for a flexible pavement must meet the same gradation requirements of Table 5-4, page 5-12, since the flexible pavement will transfer most of the shear stress caused by the load directly to the base course.

Table 9-18. Assigned CBR ratings for base course materials - bituminous-surfaced road

Number	Type	Design CBR
1	Graded crushed aggregate	100
2	Water-bound macadam	100
3	Dry-bound macadam	100
4	Bituminous base course, central plant, hot mix	100
5	Lime rock	80
6	Bituminous macadam	80
7	Stabilized aggregate	80
8	Soil cement	80
9	Sand shell or shell	80

NOTE: It is recommended that stabilized-aggregate base-course material not be used for tire pressures in excess of 100 psi.

Minimum Base-Course Thickness. The minimum allowable thickness of the base course will be as shown in Table 9-19; except that in no case will the total thickness of pavement plus base for classes A through D roads be less than 6 inches.

Bituminous-Pavement Mix. Bituminous-pavement-mix design consists of selecting the bitumen and aggregate gradation, blending aggregates to conform to the selected gradation, determining the optimum bitumen (asphalt cement) content, and calculating the job mix formula. Bituminous-mix design is beyond the scope of this manual and is described in detail in Chapter 4 of TM 5-337.

Bitumininous-Pavement Thickness Requirement. Thickness design requirements are given in Figure 9-54, page 9-72, in terms of CBR and the design index determined. Minimum thickness requirements are shown in Table 9-19.

Note that each layered section must be checked to ensure that an adequate thickness of material is used to protect the underlying layer based on the CBR of the underlying layer.

Compaction Requirements

Compaction of the subgrade, subbase, and base course must meet the same requirements as for aggregate-surfaced roads. In addition, an asphalt base course and pavement must be compacted to CE-55 density of 98-100 percent. The compaction criteria and CBR requirements for a bituminous pavement are summarized in Table 9-20, page 9-73.

Bituminous-Pavement Design

Design Requirements. Flexible-pavement design must provide the following:

- Sufficient compaction and testing of the subgrade and each layer during construction to prevent objectionable settlement under traffic.
- Adequate drainage of the base course, when frost conditions are a factor, to provide for drainage of the base course during spring thaw.

Table 9-19 Minimum thickness, in inches, of pavement and base for conventional pavements

CBR	100			80			50[a]		
Design Index	Pavement	Base	Total	Pavement	Base	Total	Pavement	Base	Total
1	ST[b]	4	4 1/2[c]	MST[d]	4	4 1/2[c]	2	4	6
2	MST[d]	4	5[c]	1 1/2	4	5 1/2[c]	2 1/2	4	6 1/2
3	1 1/2	4	5 1/2[c]	1 1/2	4	5 1/2[c]	2 1/2	4	6 1/2
4	1 1/2	4	5 1/2[c]	2	4	6	3	4	7
5	2	4	6	2 1/2	4	6 1/2	3 1/2	4	7 1/2
6	2	4	6 1/2	3	4	7	4	4	8
7	2 1/2	4	6 1/2	3	4	7	4	4	8
8	3	4	7	3 1/2	4	7 1/2	4 1/2	4	8 1/2
9	3	4	7	3 1/2	4	7 1/2	4 1/2	4	8 1/2
10	3 1/2	4	7 1/2	4	4	8	5	4	9

[a]In general, 50-CBR base course will only be used for classes E and F roads and streets.
[b]Bituminous surface treatment (spray application).
[c]Minimum total thickness of pavement plus base for classes A through D roads and streets will be 6 inches.
[d]Multiple bituminous surface treatment (spray application).

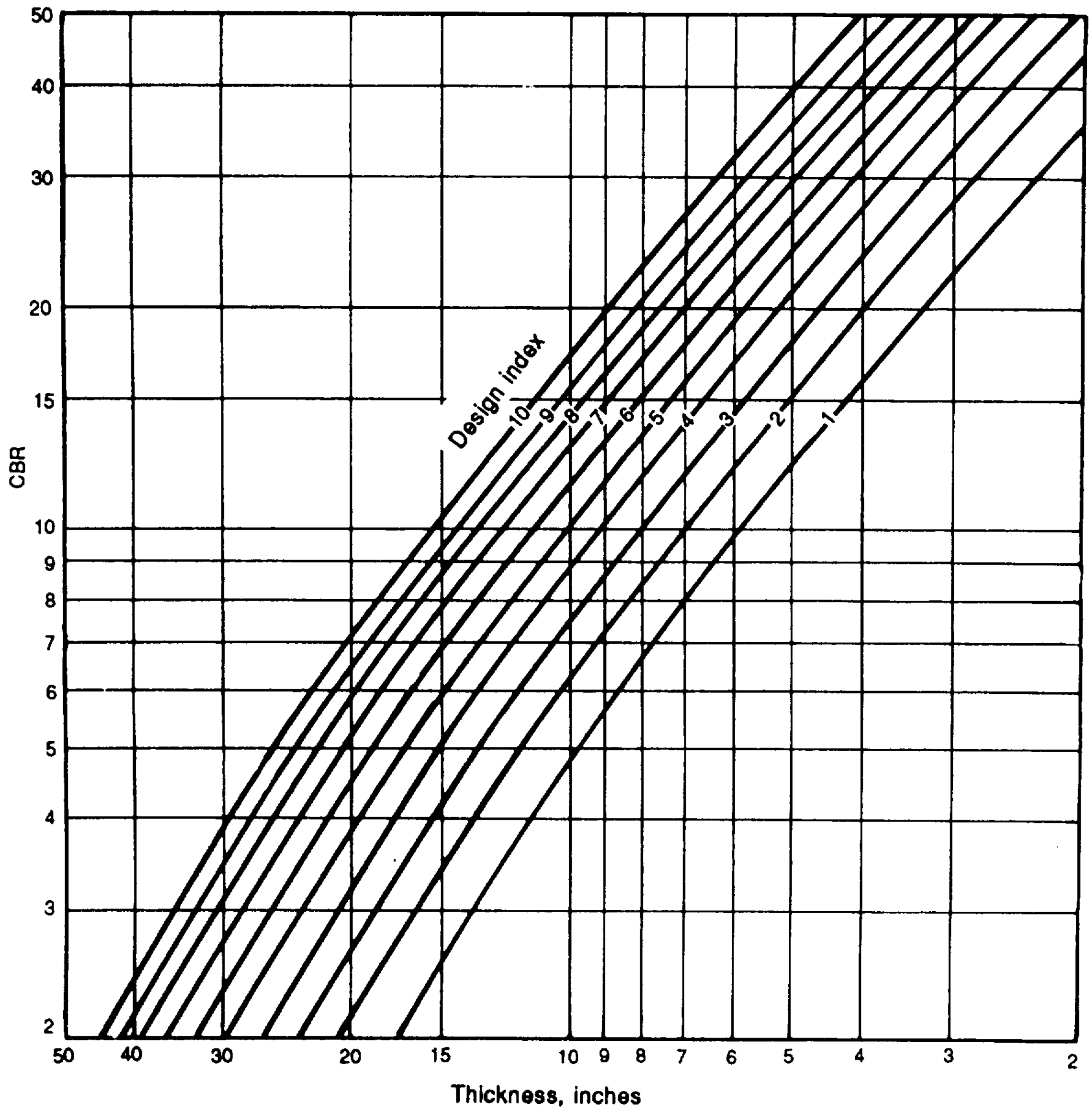


Figure 9-54. Thickness design requirements for flexible pavements

Table 9-20. Compaction criteria and CBR requirements for a flexiblew-pavement structure

CBR requirements	Layer	Compaction requirements
	Pavement	98 - 100%
50, 80, 100	Base course	Asphalt: 98 - 100% Soil: 100 - 105%
20 - 50	Subbase course	100 - 105%
0 - 20	Select material	Cohesive: 90 - 95% Cohesionless: 95 - 100%
	Design subgrade (SCIP)	Cohesive: 90 - 95% Cohesionless: 95 - 100%
	Uncompacted subgrade	

NOTES:
1. All lifts (excluding the pavement) in an Army flexible pavement must be at least 4 inches.
2. A cohesive soil is one with a PI above 5.
3. A cohesionless soil is one with a PI of 5 or less.
4. Percent Compaction is compared to CE 55 compactive effort.

- Adequate thickness above the subgrade and above each layer together with adequate quality of the select material, subbase, and base courses to prevent detrimental shear deformation under traffic and, when frost conditions are a factor, to control or reduce the effects of frost heave or permafrost degradation.
- A stable, weather-resistant, wear-resistant, waterproof, nonslippery pavement.

Design Steps.

1. Estimate the number of passes of each type of vehicle expected to use the road on a daily basis.

2. Select the proper road class based upon the traffic intensity from Table 9-8, page 9-59.

3. Determine the traffic category based upon the traffic-composition criteria given previously.

4. Determine the design index from Table 9-10, page 9-59, or 9-12, page 9-60.

5. Check soils and construction aggregates using standard criteria in Tables 5-9, page 5-12; 9-14, page 9-63; and 9-18, page 9-70.

6. Use Table 9-16 or 9-17, page 9-66, to determine compaction depth of the subgrade.

7. Determine the total road-structure thickness and cover requirements.

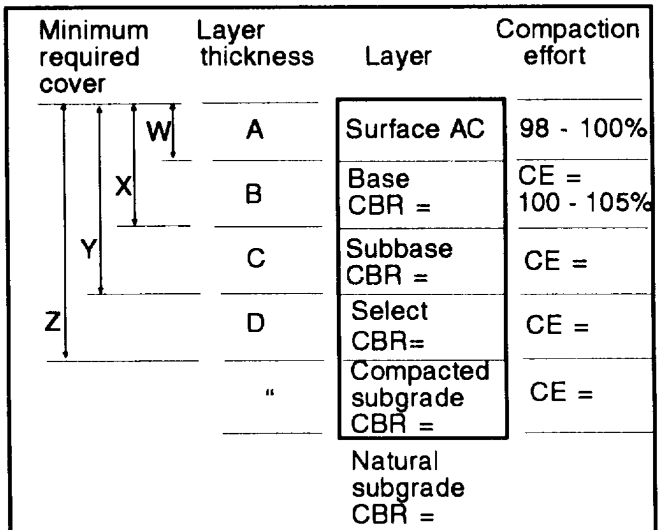
a. Enter Figure 9-54 for each layer of soil or aggregate with the following information:

- Design index.
- Design CBR values for subgrade, select, and subbase materials.

b. Determine the minimum cover thickness, in inches, for each layer of the road structure through Figure 9-54 and Table 9-19, page 9-71.

8. Determine the required percent compaction in terms of CE 55 for each layer from Table 9-20.

9. Draw the section of the bituminous-pavement road structure. (See below.)



A = W — W = Minimum cover over base (Table 9-18)
B = X - A — X = Minimum cover over subbase (Figure 9-54)
C = Y - A - B — Y = Minimum cover over select (Figure 9-54)
D = Z - A - B - C — Z = Minimum cover over compacted subgrade (Figure 9-54)

Total thickness above subgrade = A + B + C + D

NOTES:
1. All layer depths, <u>except</u> for the surface AC, should be rounded up to the next full inch for construction purposes.
2. After all possible design sections are determined, the final section used should be determined on the basis of an economic analysis.

Example (Bituminous-Pavement Design):

Design the most economical bituminous pavement for a 3-year design life capable of sustaining the following traffic:

Vehicle	Average Daily Traffic
M998 HMMWV	1,000
M35A2 2 1/2-ton truck (dual axle)	500
M113A3 (13 tons)	40
M1A1	20

The soils data are—

Subgrade:
CL material, PI = 12, W = 14 percent
Natural CBR = 5
Compaction at 90- to 95-percent CE 55, CBR = 7

Borrow:
CBR 30 at 90- to 95-percent CE 55
W = 8 percent, LL = 15, PI = 5
40 percent passing No. 10 sieve, 12 percent passing No. 20 sieve

CBR 35 at 100- to 105-percent CE 55
W = 8 PERCENT, LL - 15, PI = 5
40 percent passing No. 10 sieve, 10 percent passing No. 20 sieve

Base: GP material at CBR 50 (meets gradation)

Solution:

1. Total average daily traffic = 1,560 (given).

2. Select road class E from Table 9-8, page 9-59, based upon average daily traffic of 1,560.

3. Select traffic category VII based upon the presence of the M1A1 tank.

4. Select design index 9 from Table 9-12, page 9-60. Notice that the number of vehicles per day (20) in this table refers to the M1A1 only, since the M113A3 is considered as a Group 3 vehicle because of its weight.

5. Check soils and construction aggregate.

a. Where can the CBR 30 material be used? Table 9-14, page 9-63, indicates this material can be used only as a select material (CBR 20) because the PI exceeds 5.

b. Where can the CBR 35 material be used? Table 9-14 indicates that this material can be used as a subbase with a CBR 30 design because of the percent passing the No. 10 sieve.

6. The required depth of subgrade compaction = 15 inches.

7. Determine the total thickness and cover requirements.

a. From Figure 9-54, page 9-72, the required cover for each layer is determined for design index of 9.

Layer	Required Cover	After Rounding
Compacted subgrade, CBR 7	18"	18"
Select, CBR 20	8 1/4"	9"
Subbase, CBR 30	5 3/4"	6"
Base, CBR = 50	4"	4"

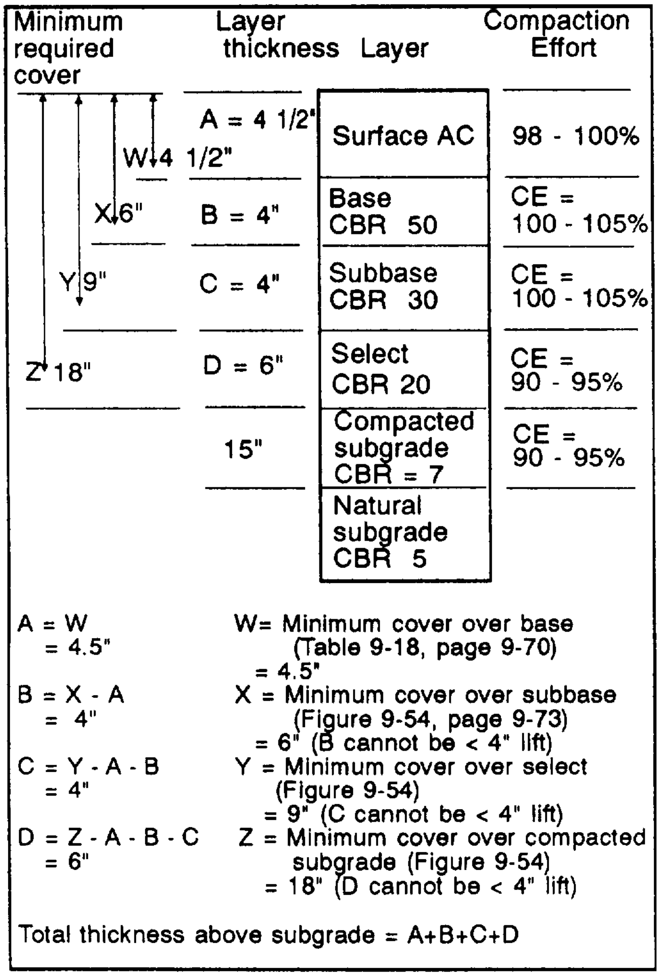
b. From Table 9-19, page 9-71, the required minimum thickness for the base course and surface asphalt is determined for design index of 9.

Layer	Minimum thickness
Base, CBR 50	4"
Surface AC	4 1/2"

8. From Table 9-20, page 9-73, the required compaction is determined for each layer.

Layer	Compaction Effort
Compacted subgrade PI > 5 (cohesive)	90 - 95 percent
Select, CBR 20	90 - 95 percent
Subbase, CBR 30	100 - 105 percent
Base, CBR 50	100 - 105 percent
Surface AC	98 - 100 percent

9. Draw the section of the bituminous-pavement road structure.



Shoulders and Similar Areas. These areas are provided only for the purpose of minimizing damage to vehicles using them accidentally or in emergencies; therefore, they are not considered normal, vehicular traffic areas. Normally only shoulders for class A roads will be paved. Others will be surfaced with soils selected for their stability in wet weather and will be compacted as required. Dust and erosion control will be provided by means of vegetative cover, anchored mulch, coarse-graded aggregate, or liquid palliative. Shoulders will not block base-course drainage, particularly where frost conditions are a factor. Where paving of shoulders is deemed necessary, the shoulders will be designed as a class F road or street.

Special Considerations for Open Storage Areas. In the design of open storage areas, consideration will be given to any special requirements necessary because of the use of a particular area. In repair yards, for instance, the final-surface texture will be one that will promote quick drying and will not contribute to the easy loss of nuts, bolts, and tiny parts. Mixtures in such areas will contain approximately 50 percent coarse aggregate. Areas subject to an appreciable amount of foot traffic will be designed to avoid the occurrence of free bituminous material on the surface.

SPECIAL DESIGN CONSIDERATIONS

Special design considerations include frost design, stabilized-base design, and geotextile design.

Frost Design

Normally, frost effects are not considered as part of the design in TO road construction. However, in the event that extremely severe host conditions exist or that frost design is directed, an outline of the frost-design procedure is included in Appendix G of this manual.

Stabilized-Soil Design

The use of stabilized-soil layers (as described in Chapter 5 of this manual and in FM 5-410) within a road structure provides the opportunity to reduce the overall thickness required to support a given load. To design a road containing stabilized-soil layers requires the application of equivalency factors to a layer or layers of a conventionally designed pavement.

To qualify for application of equivalency factors, the stabilized layer must meet appropriate strength and durability requirements. An equivalency factor represents the number of inches of a conventional base or subbase which can be replaced by 1 inch of stabilized material. Equivalency factors are determined as shown on Table 9-21, page 9-76, for bituminous-stabilized materials and from Figures 9-55 and 9-56, page 9-76, for materials stabilized with

Table 9-21. Thickness criteria

	Equivalency Factors	
Material	Base	Subbase
All-bituminous concrete	1.15	2.30
GW, GP, GM, GC	1.00	2.00
SW, SP, SM, SC	--	1.50

cement, lime, or a combination of fly ash mixed with cement or lime. The selection of an equivalency factor from the tabulation is dependent upon the classification of the soil to be stabilized. The selection of an equivalency factor from Figures 9-55 and 9-56 requires that the unconfined compressive strength (as determined according to the American Society of Testing and Materials Standatd (ASTM) D1633) be known. The equivalency factors from Figure 9-55 are for subbase materials, and those from Figure 9-56 are for base materials.

Minimum Thickness. The minimum thickness requirement for a stabilized base or subbase is 4.0 inches. The minimum thickness requirements for an asphalt pavement are the same as shown for convention pavements in Table 9-19, page 9-71.

Application of Equivalency Factors. The use of equivalency factors requires that a road be designed to support the design-load conditions. If using a stabilized base or subbase course, the thickness of a conventional base or subbase is divided by the equivalency factor for the applicable stabilized soil. The following are examples for the application of the equivalency factors:

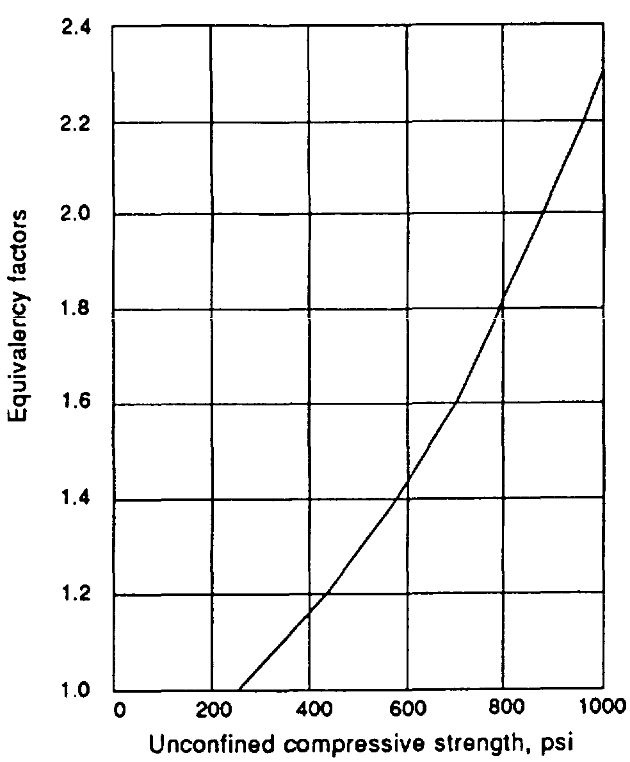


Figure 9-55. Equivalency factor for subbase soils stabilized with cement, lime, or cement and lime mixed with fly ash

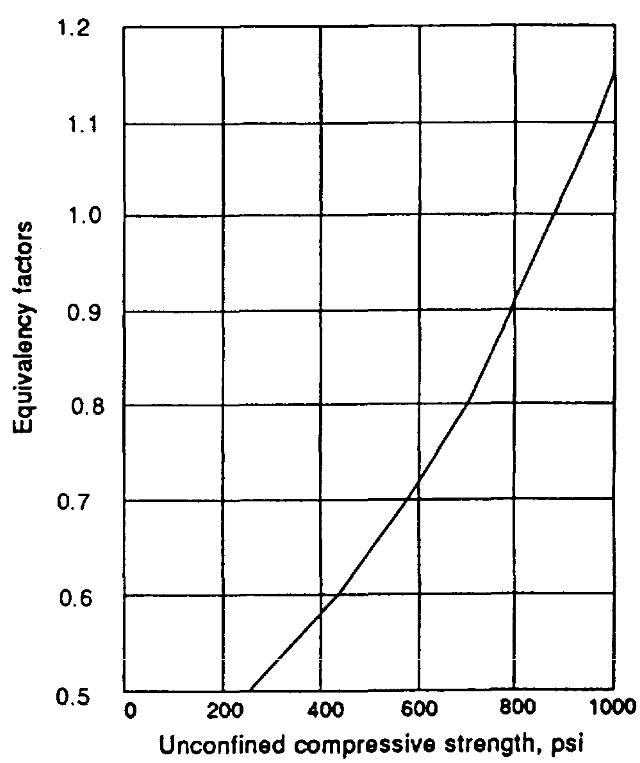


Figure 9-56. Equivalency factors for base soils stabilized with cement, lime, or cement and lime mixed with fly ash

Example 1:

Assume an aggregate-surfaced road has been designed which requires a total thickness of 14 inches above the CBR 6 subgrade. The minimum thickness of the 80 CBR base is 7 inches and the 15 CBR subbase is 7 inches. It is desired to replace the base and subbase with a lime-stabilized gravelly soil having an unconfined compression strength of 950 psi.

Solution:

From Figure 9-55 the equivalency factor for the subbase is 2.20. From Figure 9-56, the equivalency factor for the base is 1.10. Therefore, the thickness of the stabilized subbase is 7.0 inches/2.20 = 3.18 inches, and the thickness of the stabilized base is 7.0 inches/1.10 = 6.36 inches. However, since the minimum lift thickness is 4 inches, the stabilized subbase must be 4.0 inches instead of 3.18 inches. In addition, the stabilized base lift must be rounded up to the nearest full inch, so 6.36 inches is rounded up to 7 inches. Therefore, the final thickness is 7.0 inches of base + 4.0 inches of subbase = 11.0 inches of lime-stabilized gravel.

Example 2:

Assume a conventional flexible pavement has been designed which requires a total thickness of 16 inches above the subgrade. The minimum thicknesses of the asphalt concrete and the base are 2 and 4 inches, respectively, and the thickness of the subbase is 10 inches. It is desired to replace the base and subbase with a cement-stabilized, gravelly soil having an unconfined compressive strength of 890 psi.

Solution:

From Figure 9-55, the equivalency factor for a subbase having an unconfined compressive strength of 890 is 2.0: and from Figure 9-56, the equivalency factor for the base is 1.0. Therefore, the thickness of the stabilized subbase is 10 inches/2.0 = 5.0 inches, and the thickness of the stabilized base course is 4 inches/1.0 = 4.0 inches. The final section would be 2 inches of asphalt concrete and 9 inches of cement-stabilized, gravelly soil. The base-course thickness of 4.0 inches would also have been required due to the minimum thickness of the stabilized base. The subgrade still has an equivalent cover of 16 inches within the newly designed 2 inches of asphalt concrete and 9 inches of cement-stabilized, gravelly soil.

Example 3:

Assume a conventional flexible pavement has been designed which requires 2 inches of asphalt-concrete surface, 4 inches of crushed stone base, and 6 inches of subbase. It is desired to construct an all-bituminous pavement.

Solution:

The equivalency factor from data in Table 9-21, for a base course is 1.15 and for a subbase is 2.30. The thickness of asphalt concrete required to replace the base is 4 inches/ 1.15 = 3.5 inches and the thickness of asphalt concrete required to replace the subbase is 6 inches/2.30 = 2.6 inches. Therefore, the total thickness of the all-bituminous pavement is 2 + 3.5 + 2.6 or 8.1 inches, which would be reduced to 8.0 inches.

Geotextiles

The term geotextile refers to any permeable textile used with foundation, soil. rock, earth, or any other geotechnical, engineering-related material as an integral part of a human-made project, structure, or system. Geotextiles are commonly referred to as geofabrics, engineering fabrics, or just fabrics. They serve four prlmary functions:

- Reinforcement.
- Separation.
- Drainage.
- Filtration.

In many situations, the use of these fabrics can replace soil, saving time, materials, and equipment costs. In TO construction, the primary concern is with separating and reinforcing low load-bearing soils to reduce construction time.

Geotextile design is an emerging technology. As such, each geotextile manufacturer uses its own design procedure, and a general design procedure using the design criteria established in previous sections has yet to be established. Nonetheless, Appendix H of this manual outlines a typical geotextile design procedure. Note that Appendix H describes only one design procedure, and the particular geotextile used in construction may require alterations to this procedure. Additional details on geotextiles and their use are in Chapter 11 of FM 5-410.

APPENDIX A - METRIC CONVERSIONS

MULTIPLY	BY	TO OBTAIN
acre feet	43,560	cubic feet
acres	43,560	square feet
acres	4,047	square meters
acres	1.562 x 10^{-3}	square miles
acres	5,645.38	square varas
acres	4,840	square yards
atmospheres	76.0	cm of mercury
atmospheres	29.92	inches of mercury
atmospheres	33.90	feet of water
atmospheres	14.70	pounds per sq in
barrels	31.5	gallons
board feet	144 sq in x 1 in	cubic inches
BTU	0.2520	kilogram calories
BTU	778.2	foot pounds
BTU	2.928 x 10^{-4}	kilowatt hours
BTU per min	0.02356	horsepower
BTU per min	0.01757	kilowatts
BTU per min	17.57	watts
bushels	1.244	cubic feet
centares	1	square meters
centigrams	0.01	grams
centiliters	0.01	liters
centimeters	0.3937	inches
centimeters	0.01	meters
centimeters	393.7	mils
centimeters	10	millimeters
centimeter grams	1 x 10^{-5}	meter kilograms
centimeter grams	7.233 x 10^{-5}	pound feet
cm of mercury	0.01316	atmospheres
cm of mercury	0.4461	feet of water
cm of mercury	136.0	kg of sq meters
cm of mercury	27.85	pounds per sq ft
cm of mercury	0.1934	pounds per sq in
cm per second	0.6	meters per mm
circular mils	0.7854	square mils
cord feet	4 ft x 4 ft x 1 ft	cubic feet
cords	8 ft x 4 ft x 4 ft	cubic feet
cubic cm	6.102 x 10^{-2}	cubic inches
cubic cm	1 x 10^{-6}	cubic meters
cubic cm	2.642 x 10^{-4}	gallons
cubic cm	10^{-3}	liters
cubic feet	2.832 x 10^{-4}	cubic cm
cubic feet	1,728	cubic inches
cubic feet	0.02832	cubic meters
cubic feet	0.03704	cubic yards
cubic feet	7.481	gallons
cubic feet	28.32	liters
cubic feet per min	472.0	cubic cm per sec
cubic feet per min	0.1247	gallons per sec
cubic feet per min	0.4720	liters per sec
cubic feet per min	62.4	lb of water per min
cubic inches	16.39	cubic cm
cubic inches	5.787 x 10^{-4}	cubic feet
cubic inches	0.01732	quarts (liquid)
cubic meters	1 x 10^{6}	cubic cm
cubic meters	35.31	cubic feet
cubic meters	1.308	cubic yards
cubic meters	264.2	gallons
cubic yards	27	cubic feet

MULTIPLY	BY	TO OBTAIN
cubic yards	0.7646	cubic meters
cubic yards	202.0	gallons
cubic yards per min	0.45	cubic feet per sec
cubic yards per min	3.367	gallons per sec
decigrams	0.1	grams
deciliters	0.1	liters
decimeters	0.1	meters
degrees (angle)	60	minutes
degrees (angle)	0.01745	radians
degrees (angle)	3,600	seconds
dekagrams	10	grams
dekaliters	10	liters
dekameters	10	meters
drams	1.772	grams
drams	0.0625	ounces
ergs	9.486 x 10^{-11}	BTU
fathoms	6	feet
feet	03048	meters
feet	0.36	varas
feet	0333	yards
feet of water	04335	lb per sq in
feet per min	0.5080	cm per sq in
feet per min	0.01667	feet per sec
feet per min	0.01136	miles per hour
feet per sec	1.097	km per hour
feet per sec	0.5921	knots per hour
feet per sec	18.29	meters per min
feet per sec	0.6818	miles per hour
feet per 100 feet	1	percent grade
foot pounds	1.286 x 10^{-3}	BTU
foot pounds	1.356 x 10^{7}	ergs
foot pounds	5.050 x 10^{-7}	horsepower hours
foot pounds	3.241 x 10^{-4}	kilogram calories
foot pounds	3.766 x 10^{-7}	kilowatt hours
foot pounds per min	1.286 x 10^{-3}	BTU per min
foot pounds per min	3.030 x 10^{-5}	horsepower
foot pounds per min	3.241 x 10^{-4}	kg calories per min
foot pounds per min	2.260 x 10^{-5}	kilowatts
furlongs	40	rods
gallons	3,785	cubic cm
gallons	0.1337	cubic feet
gallons	231	cubic inches
gallons	3.785 x 10^{-3}	cubic meters
gallons	4,951 x 10^{-3}	cubic yards
gallons per min	2.228 x 10^{-3}	cubic feet per sec
gills	0.1183	liters
grains (troy)	1	grains (av)
grains (troy)	0.06480	grams
grains (troy)	0.04167	penny weights (troy)
grams	980.7	dynes
grams	15.43	grains (troy)
grams	10^{-3}	kilograms
grams	10^{3}	milligrams
grams	0.03527	ounces
grams	0.03215	ounces (troy)
grams	2.205 x 10^{-3}	pounds
gram calories	3.968 x 10^{3}	BTU

METRIC CONVERSIONS (continued)

MULTIPLY	BY	TO OBTAIN
gram centimeters	2.344×10^{-8}	kilogram calories
gram centimeters	10^{-5}	kilogram meters
grams per cm	5.600×10^{-3}	pounds per inch
grams per cu cm	62.43	pounds per cu ft
hectares	2.471	acres
hectares	1.076×10^{5}	square feet
hectograms	100	grams
hectoliters	100	liters
hectometers	100	meters
hectowatts	100	watts
horsepwer	42.44	BTU per min
horsepower	33,000	ft pounds per min
horsepower	550	ft pounds per sec
horsepower	1.014	horsepower (metric)
horsepower	10.70	kg calories per min
horsepower	0.7457	kilowatts
horsepower	745.7	watts
inches	2.540	centimeters
inches	10^{3}	mils
inches	0.03	varas
inches	0.03342	atmospheres
inches of mercury	1.133	feet of water
inches of mercury	70.73	pounds per sq ft
inches of water	0.002458	atmospheres
inches of water	0.07355	inches of mercury
inches of water	0.5781	ounces per sq in
inches of water	5.204	pounds per sq ft
inches of water	0.03613	pounds per sq in
joules	9.486×10^{-4}	BTU
joules	10^{7}	ergs
joules	0.7376	foot pounds
joules	2.390×10^{-4}	kilogram calories
joules	0.1020	kilogram meters
joules	2.778×10^{-4}	watt hours
kilograms	980,665	dynes
kilograms	1×10^{3}	grams
kilograms	2.2046	pounds
kilograms	1.102×10^{3}	tons (short)
kilogram calories	3.968	BTU
kilogram calories	3,088	foot pounds
kilogram calories	1.588×10^{3}	horsepower hours
kilogram calories	1.162×10^{3}	kilowatt hours
kilogram calories per min	0.06972	kilowatts
kilogram meters	9.302×10^{3}	BTU
kilogram meters	9.807×10^{7}	ergs
kilograms per cubic meter	1×10^{3}	grams per cu cm
kilograms per cubic meter	0.06243	pounds per cu ft
kilograms per sq meter	9.678×10^{-5}	atmospheres
kilograms per sq meter	3.281×10^{-3}	feet of water
kilograms per sq meter	2.886×10^{-3}	inches of mercury
kilograms per sq meter	0.2048	pounds per sq ft
kilograms per sq meter	1.422×10^{-3}	pounds per sq in
kiloliters	10^{3}	liters
kilometers	10^{5}	centimeters
kilometers	3,281	feet
kilometers	10^{3}	meters
kilometers	0.6214	miles

MULTIPLY	BY	TO OBTAIN
kilometers per hour	0.5396	knots per hour
kilowatts	56.92	BTU per min
kilowatts	4.425×10^{4}	ft pounds per min
kilowatts	1.341	horsepower
kilowatt hours	3,415	BTU
kilowatt hours	2.655×10^{6}	foot pounds
knots	1.853	kilometers per hour
knots	1.152	miles per hour
links (engineer's)	12	inches
links (surveyor's)	7.92	inches
liters	10^{3}	cubic centimeters
liters	0.2642	gallons
liters	1.057	quarts (liquid)
liters per minute	5.885×10^{-4}	cubic feet per sec
liters per minute	4.403×10^{-3}	gallons per sec
meters	100	centimeters
meters	3.2808	feet
meters	39.37	inches
meters	10^{-3}	kilometers
meters	10^{3}	millimeters
meters	1.0936	yards
microns	10^{6}	meters
miles	5,280	feet
miles	1.6093	kilometers
miles	1,760	yards
miles per hour	1.467	feet per second
miles per hour	1.6093	kilometers per hour
miles per hour	0.8684	knots per hour
milliers	10^{3}	kilograms
milligrams	10^{-3}	grams
milliliters	10^{-3}	liters
millimeters	0.1	centimeters
millimeters	0.03937	inches
millimeters	39.37	mils
mils	0.002540	centimeters
mils	10^{-3}	inches
minutes (angle)	2.909×10^{-4}	radians
minutes (angle)	60	seconds (angle)
myriagrams	10	kilograms
myriameters	10	kilometers
myriawatts	10	kilowatts
nautical miles	1.152	miles
nautical miles	2.027	yards
ounces	8	drams
ounces	437.5	grains
ounces	28.35	grams
ounces	0.0625	pounds
ounces (fluid)	1.805	cubic inches
ounces (troy)	480	grams (troy)
ounces (troy)	31.10	grams
ounces (troy)	20	pennyweights (troy)
ounces (troy)	0.08333	pounds (troy)
perches (masonry)	24.75	cubic feet
pints (dry)	33.60	cubic inches
pints (liquid)	28.87	cubic inches
pounds	444,823	dynes

METRIC CONVERSIONS (continued)

MULTIPLY	BY	TO OBTAIN
pounds	453.6	grams
pounds	16	ounces
pounds	32.17	poundals
pound feet	1.356×10^{7}	centimeter dynes
pound feet	13,825	centimeter grams
pounds feet	0.1383	meter kilograms
pounds of water	0.01602	cubic feet
pounds of water	27.68	cubic inches
pounds of water	0.1198	gallons
pounds per cubic foot	16.02	kg per cubic meter
pounds per cubic inch	27.68	grams per cu cm
pounds per foot	1.488	kilograms per meter
pounds per square foot	0.01602	feet of water
pounds per square foot	4.882	kilograms per sq meter
pounds per square inch	0.06804	atmospheres
pounds per square inch	2.307	feet of water
pounds per square inch	2.038	inches of mercury
pounds per square inch	703.1	kg per square meter
pounds per square inch	144	pounds per sq ft
quadrants (angle)	90	degrees
quadrants (angle)	5,400	minutes
quadrants (angle)	1.571	radians
quarts (dry)	67.20	cubic inches
quarts (liquid)	57.75	cubic inches
radians	57.30	degrees
radians	3,438	minutes
radians	0.0637	quadrants
reams	500	sheets
revolutions	360	degrees
revolutions	4	quadrants
revolutions	6,283	radians
revolutions per minute	6	degrees per second
revolutions per minute	0.1047	radians per sec
revolutions per minute	0.01667	rev per second
revolutions per min per min	1.745×10^{3}	radians per sec per sec
revolutions per min per min	0.01667	rev per min per sec
revolutions per min per min	2.778×10^{4}	rev per sec per sec
revolutions per second	360	degrees per second
revolutions per second	6.283	radians per second
rods	16.5	feet
seconds (angle)	4.848×10^{-6}	radians
square centimeters	0.1550	square inches
square centimeters	100	square millimeters
square feet	2.296×10^{5}	acres
square feel	0.09290	square meters
square feet	3.587×10^{-8}	square miles
square feet	0.1296	square varas
square feet	0.111	square yards
square inches	6.452	square centimeters
square inches	$6{,}944 \times 10^{3}$	square feet
square kilometers	247.1	acres
square kilometers	10.76×10^{6}	square feet
square kilometers	1×10^{6}	square meters
square kilometers	0.3861	square miles
square kilometers	1.196×10^{6}	square yards
square meters	2.47×10^{-4}	acres
square meters	10.764	square feet
square meters	3.861×10^{-7}	square miles
square meters	1.196	square yards
square miles	640	acres
square miles	27.88×10^{6}	square feet
square miles	2.590	square kilometers
square miles	3,613,040.45	square varas
square miles	3.098×10^{6}	square yards
square yards	2.066×10^{4}	acres
square yards	9	square feet
square yards	0.8361	square meters
square yards	3.228×10^{-7}	square miles
square yards	1.1664	square varas
steradians	0.1592	hemispheres
steres	10^{3}	liters
temperture (deg C) + 273	1	absolute temp (deg C
temperature (deg C) + 17.8	1.8	temperature (deg F)
temperature (deg F) + 460	1	absolute temp (deg F)
temperature (deg F) - 32	0.555	temperature (deg C)
tons (long)	1,016	kilograms
tons (long)	2,240	pounds
tons (metric)	10^{3}	kilograms
tons (metric)	2,205	pounds
term (short)	907.2	kilograms
tons (short)	2,000	pounds
tons (short) per sq ft	9,765	kg per sq meter
tons (short) per sq ft	13.89	pounds per sq in
tons (short) per sq ft	1.406×10^{6}	kg per sq meter
tons (short) per sq in	2,000	pounds per sq in
varas	2.7777	feet
watts	0.05692	BTU per min
watts	10^{7}	ergs per second
watts	44.26	foot pounds per min
watts	1.341×10^{-3}	horsepower
watts	10^{2}	kilowatts
watt hours	3.415	BTU
weeks	168	hours
yards	91.44	centimeters
yards	3	feet
yards	36	inches
yards	0.9144	meters

METRIC CONVERSIONS (continued)

One unit (below) equals →	mm	cm	m	km
Millimeters (mm)	1	0.1	0.001	0.000001
Centimeters (cm)	10	1	0.01	0.00001
Meters (m)	1,000	100	1	0.001
Kilometers (km)	1,000,000	100,000	1,000	1

One unit (below) equals →	g	kg	Metric ton
Grams (g)	1	0.001	0.000001
Kilograms (kg)	1,000	1	0.001
Metric tons	1,000,000	1,000	1

Units of centimeters

Cm	0.10	0.20	0.30	0.40	0.50	0.60	0.70	0.80	0.90	1.00
Inch	0.04	0.08	0.12	0.16	0.20	0.24	0.28	0.31	0.35	0.39

Fractions of an inch

Inch	1/16	1/8	3/16	1/4	5/16	3/8	7/16	1/2
Cm	0.16	0.32	0.48	0.64	0.79	0.95	1.11	1.27
Inch	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1
Cm	1.43	1.59	1.75	1.91	2.06	2.22	2.38	2.54

Weight

Qty	Metric ton → Short ton	Short ton → Metric ton	kg → Pounds	Pounds → kg	g → Ounces	Ounces → g
1	1.10	0.91	2.20	0.45	0.04	28.4
2	2.20	1.81	4.41	0.91	0.07	56.7
3	3.31	2.72	6.61	1.36	0.11	85.0
4	4.41	3.63	8.82	1.81	0.14	113.4
5	5.51	4.54	11.02	2.67	0.18	141.8
6	6.61	5.44	13.23	2.72	0.21	170.1
7	7.72	6.35	15.43	3.18	0.25	198.4
8	8.82	7.26	17.64	3.63	0.28	226.8
9	9.92	8.16	19.84	4.08	0.32	255.2
10	11.02	9.07	22.05	4.54	0.35	283.5
20	22.05	18.14	44.09	9.07	0.71	567.0
30	33.07	27.22	66.14	13.61	1.06	850.5
40	44.09	36.29	88.18	18.14	1.41	1,134.0
50	55.12	45.36	110.23	22.68	1.76	1,417.5
60	66.14	54.43	132.28	27.22	2.12	1,701.0
70	77.16	63.50	154.32	31.75	2.47	1,984.5
80	88.18	72.57	176.37	36.29	2.82	2,268.0
90	99.21	81.65	198.42	40.82	3.17	2,551.5
100	110.20	90.72	220.46	45.36	3.53	2,835.0

Length

Qty	km → Miles	Miles → km	m → Yards	Yards → m	m → Feet	Feet → m	cm → Inches	Inches → cm
1	0.62	1.61	1.09	0.91	3.28	0.30	0.39	2.54
2	1.24	3.22	2.19	1.83	6.56	0.61	0.79	5.08
3	1.86	4.83	3.28	2.74	9.84	0.91	1.18	7.62
4	2.49	6.44	4.37	3.66	13.12	1.22	1.57	10.16
5	3.11	8.05	5.47	4.57	16.40	1.52	1.97	12.70
6	3.73	9.66	6.56	5.49	19.68	1.83	2.36	15.24
7	4.35	11.27	7.66	6.40	22.97	2.13	2.76	17.78
8	4.97	12.87	8.75	7.32	26.25	2.44	3.15	20.32
9	5.59	14.48	9.84	8.23	29.53	2.74	3.54	22.86
10	6.21	16.09	10.94	9.14	32.81	3.05	3.93	25.40
20	12.43	32.19	21.87	18.29	65.62	6.10	7.87	50.80
30	18.64	48.28	32.81	27.43	98.42	9.14	11.81	76.20
40	24.85	64.37	43.74	36.58	131.23	12.19	15.75	101.60
50	31.07	80.47	54.68	45.72	164.04	15.24	19.68	127.00
60	37.28	96.56	65.62	54.86	196.85	18.29	23.62	152.40
70	43.50	112.65	76.55	64.00	229.66	21.34	27.56	177.80
80	49.71	128.75	87.49	73.15	262.47	24.38	31.50	203.20
90	55.92	144.84	98.42	82.30	295.28	27.43	35.43	228.60
100	62.14	160.94	109.36	91.44	328.08	30.48	39.37	254.00

Volume

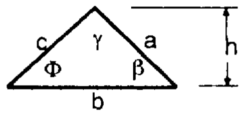
Qty	cu ft → cu yd	cu ft → cu meters	cu yd → cu ft	cu yd → cu meters	Cu meters → cu ft	Cu meters → cu yd
1	0.037	0.028	27.0	0.76	35.3	1.31
2	0.074	0.057	54.0	1.53	70.6	2.62
3	0.111	0.085	81.0	2.29	105.9	3.92
4	0.148	0.113	108.0	3.06	141.3	5.23
5	0.185	0.142	135.0	3.82	176.6	6.54
6	0.212	0.170	162.0	4.59	211.9	7.85
7	0.259	0.198	189.0	5.35	247.2	9.16
8	0.296	0.227	216.0	6.12	282.5	10.46
9	0.333	0.255	243.0	6.88	317.8	11.77
10	0.370	0.283	270.0	7.65	353.1	13.07
20	0.741	0.566	540.0	15.29	706.3	26.16
30	1.111	0.850	810.0	22.94	1,059.4	39.24
40	1.481	1.133	1,080.0	30.58	1,412.6	52.32
50	1.852	1.416	1,350.0	38.23	1,765.7	65.40
60	2.222	1.700	1,620.0	45.87	2,118.9	78.48
70	2.592	1.982	1,890.0	53.52	2,472.0	91.56
80	2.962	2.265	2,160.0	61.16	2,825.2	104.63
90	3.333	2.548	2,430.0	68.81	3,178.3	117.71
100	3.703	2.832	2,700.0	76.46	3,531.4	130.76

APPENDIX B - GEOMETRIC FORMULAS

(1) Any triangle:

$A = 1/2bh$

or: $Sin\gamma = \frac{c\ Sin\,\Phi}{a}$

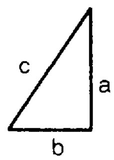


(2) Right triangle:

$a = \sqrt{c^2 - b^2}$

$b = \sqrt{c^2 - a^2}$

$c = \sqrt{a^2 + b^2}$

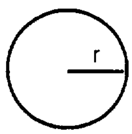


(3) Circle:

$A = \pi r^2$

$A = 0.7854\ D^2$

$C = \pi D$

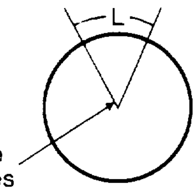


(4) Segment of circle:

$A = \frac{\pi r^2 a}{360} - \frac{r^2\ Sin\ a}{2}$

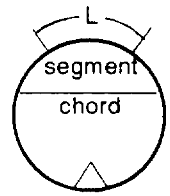
$L = \frac{2\pi\ ra}{360}$

a = angle in degrees



(5) Segment of circle:

$A = \frac{rL}{2} = \frac{\pi r^2 a}{360}$



A = area
h = height
b = length of base
c = hypotenuse
C = circumference

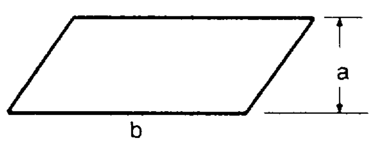
V = volume
r = radius
D = diameter
π = 3.1416
L = length of arc
K = length of chord

(6) Regular polygons. The area of any regular polygon (all sides equal, all angles equal) is equal to the product of the square of the lengths of one side and the factors. Example problem: Area of a regular octagon having 6-inch sides is 6 x 6 x 4.828, or 173.81 square inches. See factors in table.

POLYGON FACTORS			
No. of sides	Factor	No. of sides	Factor
3	0.433	8	4.828
4	1.000	9	6.182
5	1.720	10	7.694
6	2.598	11	9.366
7	3.634	12	11.196

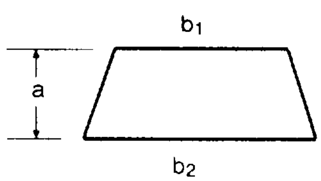
(7) Rectangle and parallelogram:

$A = ab$



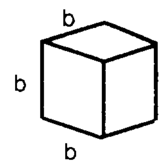
(8) Trapezoid:

$A = 1/2a(b_1 + b_2)$



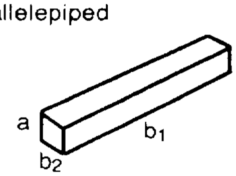
(9) Cube:

$V = b^3$



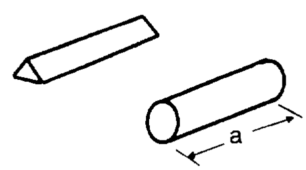
(10) Rectangular parallelepiped

$V = ab_1b_2$



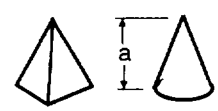
(11) Prism or cylinder:

V = a x area of base



(12) Pyramid or cone:

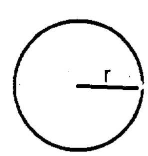
V = (1/3)a x area of base



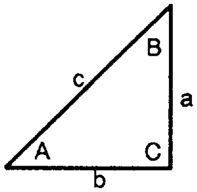
(13) Sphere:

$V = (4/3)\pi r^3 = \frac{\pi D^3}{6}$

$A = 4\ \pi r^2$



GEOMETRIC FORMULAS (continued)

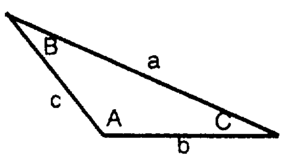


$a^2 = c^2 - b^2$ $\qquad Sin\,A = \frac{a}{c}$

$b^2 = c^2 - a^2$ $\qquad Cos\,A = \frac{b}{c}$

$c^2 = a^2 + b^2$ $\qquad Tan\,A = \frac{a}{b}$

Right triangle							
To find							
Given	A	B	C	a	b	c	area
a,b	$\tan A = \frac{a}{b}$	$\tan B = \frac{b}{a}$	90			$\sqrt{a^2 + b^2}$	$\frac{ab}{2}$
a,c	$\sin A = \frac{a}{c}$	$\cos B = \frac{a}{c}$	90		$\sqrt{c^2 - a^2}$		$\frac{a}{2}\sqrt{c^2\ a^2}$
A,a		90 - A	90		a cot A	$\frac{a}{\sin A}$	$\frac{a^2 \cot A}{2}$
A,b		90 - A	90	b tan A		$\frac{b}{\cos A}$	$\frac{b^2 \tan A}{2}$
A,c		90 - A	90	c sin A	c cos A		$\frac{c^2 \sin 2A}{2}$



$$\frac{a}{Sin\,A} = \frac{b}{Sin\,B} = \frac{c}{Sin\,C}$$

$$S = \frac{a + b + c}{2}$$

$$a^2 = b^2 + c^2 - 2bc\,Cos\,A$$

$$b^2 = a^2 + c^2 - 2ac\,Cos\,B$$

$$c^2 = a^2 + b^2 - 2ab\,Cos\,C$$

Oblique triangle						
Given	To find					
	A	B	C	b	c	area
a,b,c	$\cos\frac{A}{2} = \sqrt{\frac{s(s-a)}{bc}}$	$\cos\frac{B}{2} = \sqrt{\frac{s(s-b)}{ac}}$	$\cos\frac{C}{2} = \sqrt{\frac{s(s-c)}{ab}}$			$\sqrt{s(s-a)(s-b)(s-c)}$
a,A,B			180 - (A+B)	$\frac{a \sin B}{\sin A}$	$\frac{a \sin C}{\sin A}$	$\frac{a^2 \sin B \sin C}{2 \sin A}$
a,b,A		$\sin B = \frac{b \sin A}{a}$			$\frac{b \sin C}{\sin B}$	
a,b,c		$\tan A = \frac{a \sin C}{b - a \cos C}$			$\sqrt{a^2 + b^2 - 2ab \cos C}$	$\frac{ab \sin C}{2}$

GEOMETRIC FORMULAS (continued)

Degree of Angle	Sine	Cosecant	Tangent	Cotangent	Secant	Cosine	Degree of Angle
0	0.000		0.000		1.000	1.000	90
1	0.017	57.30	0.017	57.29	1.000	1.000	89
2	0.035	28.65	0.035	28.64	1.001	0.999	88
3	0.052	19.11	0.052	19.08	1.001	0.999	87
4	0.070	14.34	0.070	14.30	1.002	0.998	86
5	0.087	11.47	0.087	11.43	1.004	0.996	85
6	0.105	9.567	0.105	9.514	1.006	0.995	84
7	0.122	8.206	0.123	8.144	1.008	0.993	83
8	0.139	7.185	0.141	7.115	1.010	0.990	82
9	0.156	6.392	0.158	6.314	1.012	0.988	81
10	0.174	5.759	0.176	5.671	1.015	0.985	80
11	0.191	5.241	0.194	5.145	1.019	0.982	79
12	0.208	4.810	0.213	4.705	1.022	0.978	78
13	0.225	4.445	0.231	4.331	1.026	0.974	77
14	0.242	4.134	0.249	4.011	1.031	0.970	76
15	0.259	3.864	0.268	3.732	1.035	0.966	75
16	0.276	3.628	0.287	3.487	1.040	0.961	74
17	0.292	3.420	0.306	3.271	1.046	0.956	73
18	0.309	3.236	0.325	3.078	1.051	0.951	72
19	0.326	3.072	0.344	2.904	1.058	0.946	71
20	0.342	2.924	0.364	2.747	1.064	0.940	70
21	0.358	2.790	0.384	2.605	1.071	0.934	69
22	0.375	2.669	0.404	2.475	1.079	0.927	68
23	0.391	2.559	0.424	2.356	1.086	0.921	67
24	0.407	2.459	0.445	2.246	1.095	0.914	66
25	0.423	2.366	0.466	2.145	1.103	0.906	65
26	0.438	2.281	0.488	2.050	1.113	0.899	64
27	0.454	2.203	0.510	1.963	1.122	0.901	63
28	0.469	2.130	0.532	1.881	1.133	0.883	62
29	0.485	2.063	0.554	1.804	1.143	0.875	61
30	0.500	2.000	0.577	1.732	1.155	0.866	60
Degree of Angle	Cosine	Secant	Cotangent	Tangent	Cosecant	Sine	Degree of Angle

TRIGONOMETRIC FUNCTIONS

GEOMETRIC FORMULAS (continued)

Degree of Angle	Sine	Cosecant	Tangent	Cotangent	Secant	Cosine	Degree of Angle
31	0.515	1.942	0.601	1.664	1.167	0.857	59
32	0.530	1.887	0.625	1.600	1.179	0.848	58
33	0.545	1.836	0.649	1.540	1.192	0.839	57
34	0.559	1.788	0.675	1.483	1.206	0.829	56
35	0.574	1.743	0.700	1.428	1.221	0.829	55
36	0.588	1.701	0.727	1.376	1.236	0.809	54
37	0.602	1.662	0.754	1.327	1.252	0.799	53
38	0.616	1.624	0.781	1.280	1.269	0.788	52
39	0.629	1.589	0.810	1.235	1.287	0.777	51
40	0.643	1.556	0.839	1.192	1.305	0.766	50
41	0.656	1.542	0.869	1.150	1.325	0.755	49
42	0.669	1.494	0.900	1.111	1.346	0.743	48
43	0.682	1.466	0.933	1.072	1.367	0.731	47
44	0.695	1.440	0.966	1.036	1.390	0.719	46
45	0.707	1.414	1.000	1.100	1.414	0.707	45
Degree of Angle	Cosine	Secant	Cotangent	Tangent	Cosecant	Sine	Degree of Angle

TRIGONOMETRIC FUNCTIONS

APPENDIX C

HYDROLOGIC AND HYDRAULIC TABLES AND CURVES

PRECIPITATION TABLES

Table C-1, page C-2, provides local data for use in designing drainage systems as discussed in Chapter 6.

HYDROLOGIC SUPPLY CURVES FOR OVERLAND FLOW

Figures C-1 through C-7, pages C-7 through C-13, represent the peak runoff rates from individual storm events of various durations, all of which have the same average frequency of occurrence.

These curves are computed from Horton's equation given in Chapter 6, using a retardance coefficient in n = 0.04 and hydraulic gradient S = 1 percent.

HYDRAULIC CHARACTERISTICS OF SELECTED DRAINAGE CHANNELS

Tables C-2 through C-10, pages C-14 through C-22, present commonly used ditch sections, with cross-section areas and hydraulic radius, to facilitate selection of ditch size and shape.

Table C-1. Mean monthly, maximum monthly, and maximum 24-hour precipitation for selected stations throughout the world

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Years Recorded
ALASKA														
Barrow:														
Mean	0.15	0.20	0.13	0.12	0.14	0.26	0.89	0.73	0.49	0.56	0.20	0.36	4.23	26
24-hour maximum	0.70	0.23	0.28	0.20	0.30	0.36	0.84	0.43	1.00	0.41	0.26	0.26	1.00	18-19
Mean snowfall	2.60	2.70	2.00	2.70	1.80	0.40	1.00	0.70	0.30	7.90	4.30	3.90	33.00	21-23
Ketchikan:														
Mean	13.62	11.16	12.13	10.88	8.39	6.60	8.07	11.61	12.22	20.18	19.96	15.86	150.68	30-32
24-hour maximum	5.87	5.36	5.70	4.44	5.87	4.25	3.34	8.07	6.06	7.04	7.14	5.33	8.07	29-30
Mean snowfall	10.00	7.10	4.70	0.60	0.40	0	0	0	0	0.20	0.90	7.40	32.30	-----
Teller (65° 16' N, 166° 20' W):														
Mean	1.13	0.53	0.34	0.69	0.51	0.62	1.88	1.48	1.74	0.71	0.79	0.52	10.94	8-13
24-hour maximum	2.10	0.58	0.35	0.70	0.80	0.50	1.00	1.10	2.20	0.50	0.72	0.55	2.20	6-10
Mean snowfall	12.00	7.10	6.70	6.50	2.40	0	0	0	0.20	2.20	7.50	8.20	52.90	5-9
Dutch Harbor:														
Mean	6.32	6.12	4.92	4.19	4.33	2.82	1.89	2.45	5.45	7.39	5.81	6.92	58.61	19-23
24-hour maximum	3.73	3.75	2.47	3.46	2.74	3.00	1.60	2.08	2.53	3.08	3.22	2.31	3.75	19-23
Mean snowfall	15.60	19.70	10.50	6.70	0.10	T	0	0	T	0.50	5.80	10.80	69.70	19-23
CANADA														
Edmonton, Alberta:														
Mean	0.76	0.67	0.67	0.80	1.86	3.26	3.57	2.47	1.40	0.74	0.73	0.75	17.67	-----
24-hour maximum	0.70	0.80	0.60	0.83	2.80	2.20	2.16	2.68	1.37	1.03	0.80	0.70	2.80	15
St. Johns, Newfoundland:														
Mean	5.39	5.08	4.53	4.25	3.54	3.54	3.74	3.58	3.78	5.39	6.06	4.92	53.80	52
Maximum	11.38	12.84	8.84	8.79	7.11	7.83	7.72	9.76	11.42	13.11	12.27	14.05	69.05	57
24-hour maximum	2.44	3.31	1.81	2.99	3.62	2.01	2.28	1.54	2.36	2.80	3.35	3.15	3.62	73
CENTRAL AND SOUTH AMERICA														
Puerto Barrios, Guatemala (15° 35' N, 88° 35' W):														
Mean	11.46	8.86	3.82	3.70	7.99	10.63	16.50	18.62	9.06	7.20	15.47	11.46	124.77	7
Maximum	25.94	26.97	12.43	14.65	18.15	18.15	25.08	29.58	18.54	23.95	32.69	19.93	158.10	19
24-hour maximum	6.75	6.55	5.30	4.69	3.61	4.25	4.94	5.20	6.59	5.55	7.80	11.18	11.18	13
Fortaleza, Brazil (3° 42' S, 38° 30' W):														
Mean	3.35	6.93	11.77	13.46	9.61	4.72	2.13	1.10	0.67	0.51	0.55	1.54	56.34	72
Maximum	18.27	25.12	26.61	32.40	26.14	15.90	10.43	6.61	2.99	3.58	3.78	8.98	109.41	72
24-hour maximum	7.40	7.01	9.61	9.45	5.75	3.50	2.80	2.76	1.57	2.17	1.81	5.20	9.61	59

Table C-1. Mean monthly, maximum monthly, and maximum 24-hour precipitation for selected stations throughout the world (in inches) (continued)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Years Recorded
EUROPE AND ICELAND														
London, England:														
Mean	1.88	1.57	1.56	1.60	1.75	2.03	2.24	2.23	1.95	2.61	2.28	2.29	23.99	62
Maximum	4.88	4.13	3.94	3.98	4.09	7.20	4.88	6.50	5.71	5.91	3.98	6.38	38.19	68
24-hour maximum	1.60	1.10	0.90	1.20	1.80	2.40	2.30	1.80	1.60	1.40	1.30	1.50	2.40	65
Berlin, Germany:														
Mean	1.69	1.38	1.57	1.54	1.89	2.36	3.07	2.32	1.69	1.77	1.69	1.89	22.86	80
Maximum	3.94	4.88	5.28	4.17	5.71	5.59	9.06	6.57	4.25	5.28	4.65	4.49	31.61	80
24-hour maximum	0.97	0.85	0.50	1.55	1.27	1.73	2.59	2.66	1.77	0.99	1.28	0.62	2.66	15
Prague, Czechoslavakia:														
Mean	0.87	0.83	1.10	1.54	2.36	2.76	2.56	2.24	1.65	1.22	1.18	0.94	19.25	70
Maximum	2.36	2.01	2.40	4.33	6.50	6.18	5.67	4.80	5.35	4.17	3.70	3.27	27.52	70
24-hour maximum	0.53	0.56	0.73	1.30	1.28	1.98	2.17	1.57	1.60	1.22	1.13	0.80	2.17	15
Budapest, Hungary:														
Mean	1.46	1.22	1.77	2.28	2.91	2.91	2.09	1.97	2.01	2.60	2.09	1.89	25.20	35
Maximum	3.94	3.11	4.33	3.82	5.51	8.31	7.05	5.04	4.88	5.16	5.79	4.29	37.05	31
24-hour maximum	1.34	1.38	1.26	1.69	1.69	2.20	2.52	3.31	2.44	1.61	1.57	1.50	3.31	30
Helsinki, Finland:														
Mean	1.77	1.46	1.38	1.42	1.77	1.81	2.24	2.91	2.52	2.60	2.48	2.01	24.37	71
Maximum	4.61	3.62	4.45	3.94	4.16	4.61	7.56	6.50	5.91	5.79	5.94	4.76	33.90	87
24-hour maximum	0.73	0.83	0.96	1.46	1.48	1.87	1.82	1.64	2.12	2.12	1.13	1.56	2.12	36
Trondheim, Norway:														
Mean	3.43	2.87	2.24	1.81	1.50	1.73	2.24	2.95	3.27	3.39	3.11	2.56	31.10	50
Maximum	11.45	8.80	8.31	6.37	4.07	4.54	4.64	6.40	6.82	8.78	9.95	8.64	56.67	44
24-hour maximum	1.68	1.91	1.96	0.99	1.30	1.00	1.66	2.23	1.42	2.24	1.85	2.58	2.58	31
Corfu, Greece:														
Mean	7.72	6.11	4.18	3.70	2.45	1.05	0.28	0.56	2.82	5.02	7.05	9.54	50.48	38
Maximum	17.13	11.26	9.22	13.35	4.09	4.75	1.14	4.29	6.65	21.50	15.39	13.95	66.75	26
24-hour maximum	5.47	3.23	2.17	2.83	1.85	1.61	1.02	3.35	3.23	5.91	5.08	3.90	5.91	26
Stockholm, Sweden:														
Mean	0.98	0.91	1.08	1.07	1.44	1.63	2.37	2.59	1.75	1.98	1.53	1.31	18.64	51
Maximum	3.35	3.90	3.03	3.54	3.31	5.08	7.28	4.84	5.67	6.85	3.15	3.07	28.27	51
24-hour maximum	0.53	0.84	0.81	1.46	1.82	1.26	2.62	2.69	1.66	1.29	1.59	1.14	2.69	38
Reykjavik, Iceland:														
Mean	3.86	3.31	2.72	2.44	1.89	1.93	1.89	2.01	3.54	3.43	3.74	3.50	34.20	50
Maximum	6.21	9.54	7.21	5.90	2.95	3.64	4.63	6.49	5.99	7.12	6.87	7.35	50.83	19
24-hour maximum	1.30	1.36	2.23	0.87	0.78	1.16	1.07	1.18	1.07	1.30	1.73	2.17	2.23	19

Table C-1. Mean monthly, maximum monthly, and maximum 24-hour precipitation for selected stations throughout the world (continued)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Years Recorded
ASIA														
Sapporo, Hokkaido (Japan):														
Mean	3.50	2.50	2.40	2.20	2.70	2.80	3.30	3.70	5.00	4.60	4.40	4.00	40.09	30
Maximum	8.20	4.60	5.50	5.00	4.90	7.30	8.20	11.20	11.00	11.80	8.30	7.80	53.50	22
24-hour maximum	2.80	1.90	1.80	4.20	2.80	4.70	4.90	4.30	4.60	4.00	1.70	2.90	4.90	44
Urakawa, Hokkaido (Japan):														
Mean	1.60	0.90	2.10	3.10	3.80	3.40	5.10	6.30	4.60	4.40	4.20	2.10	41.80	6
Maximum	2.60	1.60	2.60	5.50	5.30	7.40	15.80	9.10	6.10	5.70	7.00	2.70	55.20	6
24-hour maximum	1.00	0.60	1.10	1.40	1.80	2.00	5.80	3.50	2.40	2.60	2.00	1.10	5.80	6
Kobe, Honshu (Japan):														
Mean	1.90	2.20	3.60	5.00	4.90	8.20	6.00	4.60	7.70	4.80	2.60	1.80	53.30	....
Maximum	3.40	5.90	6.50	8.56	9.80	14.80	11.80	9.60	6.60	10.80	6.00	3.90	65.60	....
24-hour maximum	1.60	2.30	2.50	2.50	3.40	5.90	4.60	4.60	7.80	3.90	2.70	1.80	7.80	....
Hamada, Honshu (Japan):														
Mean	4.50	3.90	4.60	5.00	4.90	7.70	7.50	4.60	8.20	5.10	4.20	4.60	64.80	....
Maximum	6.90	7.30	7.10	8.60	9.60	22.30	18.60	12.00	20.00	10.90	7.50	9.10	92.50	....
24-hour maximum	2.00	2.60	2.00	2.50	3.90	4.30	8.00	8.90	5.40	3.30	1.90	2.20	8.90	....
Gensan, Korea:														
Mean	1.20	1.47	1.78	2.76	3.41	4.74	10.46	12.09	7.02	3.33	2.50	1.39	52.15	21
Maximum	3.26	5.11	5.33	7.08	5.90	12.26	16.33	21.33	16.43	17.94	5.65	7.07	79.89	21
24-hour maximum	2.90	2.90	3.80	4.60	5.00	4.80	6.90	7.70	9.60	8.80	3.40	3.30	9.60	29
Yunnan, China (25° 07' N, 102° 54' E):														
Mean	0.35	0.55	0.67	0.75	4.21	6.22	8.74	8.58	4.96	2.91	1.81	0.43	40.18	32
Maximum	2.17	2.32	2.91	2.80	12.68	22.56	18.11	15.67	11.61	7.56	6.85	1.81	61.02	32
24-hour maximum	1.22	1.06	1.26	1.14	2.60	9.61	4.29	4.49	4.53	2.80	2.87	1.30	9.61	32
Hongkong, China:														
Mean	1.27	1.75	2.93	5.44	11.50	15.52	15.01	14.22	10.11	4.55	1.70	1.15	85.16	53
Maximum	8.43	7.94	11.48	17.16	48.84	34.38	30.08	34.31	30.60	23.98	8.82	4.90	119.72	47
24-hour maximum	3.92	2.18	3.78	6.22	20.50	12.63	21.02	11.14	7.96	11.50	5.88	3.58	21.02	53
Tientsin, China:														
Mean	0.16	0.09	0.37	0.63	1.13	2.46	7.00	5.50	1.76	0.59	0.39	0.15	20.23	44
Maximum	0.91	0.62	1.73	1.89	2.28	6.96	14.46	11.39	5.26	1.64	1.89	0.46	28.70	20
24-hour maximum	0.64	0.50	1.37	1.22	1.90	3.09	4.77	4.89	2.31	1.26	1.13	0.28	4.89	22
Urga, Mongolia (47° 55' N, 106° 50' E):														
Mean	0.04	0.03	0.06	0.22	0.34	0.96	2.91	1.91	0.76	0.20	0.15	0.10	7.68	15
Maximum	0.16	0.13	0.37	0.79	1.35	3.40	6.47	4.33	2.09	0.66	1.38	0.89	13.69	15
24-hour maximum	0.09	0.06	0.17	0.31	0.67	1.07	2.86	2.38	1.61	0.33	0.69	0.69	2.86	10

Table C-1. Mean monthly, maximum monthly, and maximum 24-hour precipitation for selected stations throughout the world (continued)

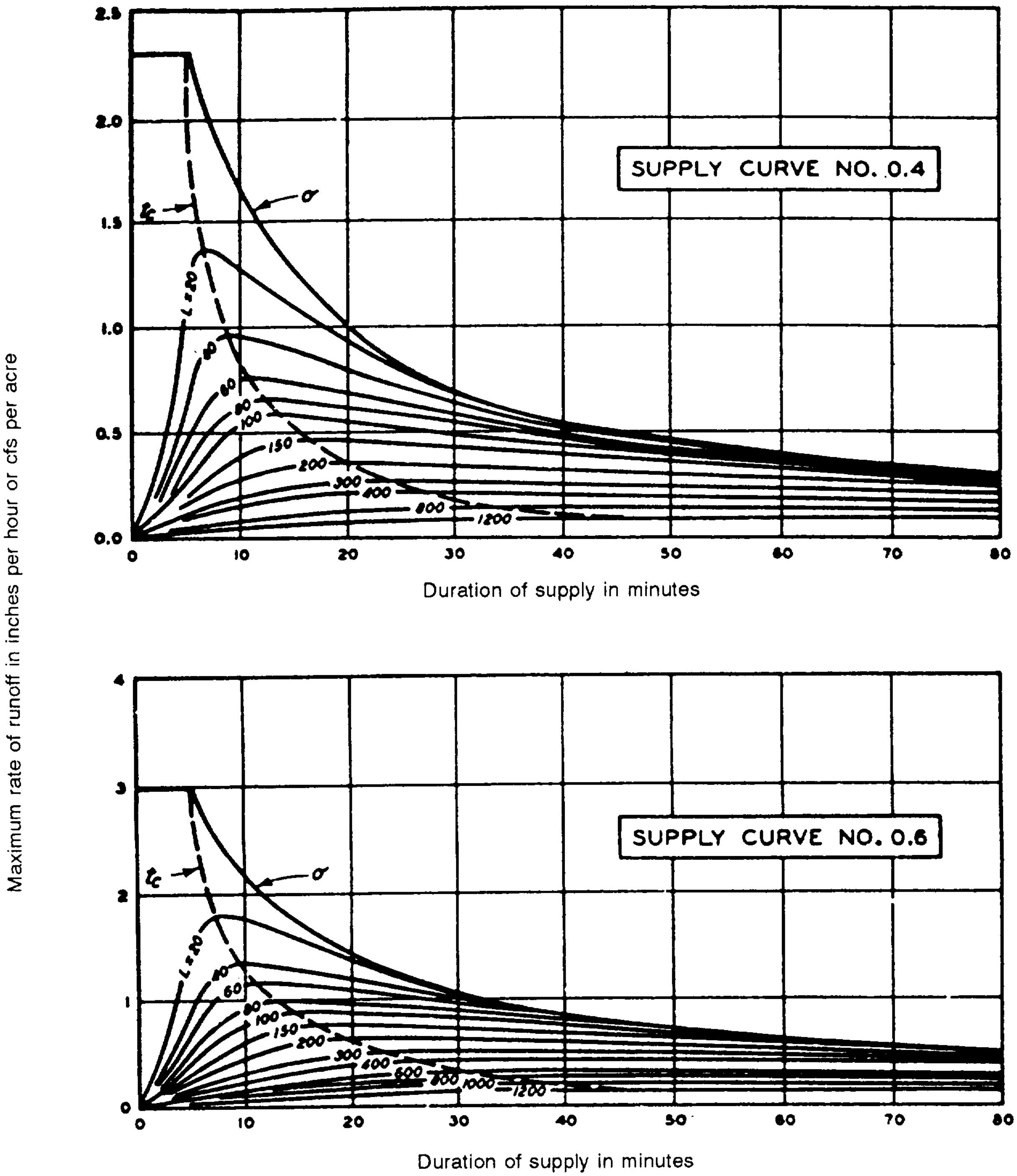
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Years Recorded
ASIA (continued)														
Singapore, China:														
Mean	9.88	6.62	7.40	7.64	6.65	6.85	6.77	7.95	6.77	8.07	9.92	10.55	95.07	52
Maximum	14.13	24.98	19.74	14.92	10.79	14.08	13.56	10.81	17.26	12.06	15.15	16.91	14.03	21
24-hour maximum	6.93	4.30	4.88	2.97	4.92	3.46	3.91	4.17	5.20	6.02	4.02	4.35	6.93	21
Dairen, Manchuria:														
Mean	0.50	0.30	0.70	1.00	1.70	1.80	6.40	5.10	4.00	1.10	1.00	0.50	24.10	25
Maximum	1.90	1.10	3.50	2.60	4.30	3.30	16.00	15.30	12.70	2.30	2.70	0.90	44.30	19
24-hour maximum	1.40	1.10	1.80	1.90	2.20	2.70	7.50	6.40	4.10	1.50	1.80	0.60	7.50	25
Manchouli:														
Mean	0.10	0.10	0.10	0.10	0.60	1.70	2.90	2.40	1.30	0.30	0.20	0.10	9.90	20
Maximum	0.40	0.20	0.50	0.70	2.30	4.30	6.10	5.60	3.30	1.50	0.60	0.40	14.40	20
24-hour maximum	0.10	0.10	0.30	0.40	1.30	1.90	1.10	2.30	1.40	1.00	0.50	0.20	2.30	7
Aden, Arabia:														
Mean	0.30	0.20	0.40	0.20	0.10	0.10	0.03	0.10	0.10	0.10	0.10	0.10	1.80	....
Maximum	3.31	1.58	6.57	3.87	1.40	1.34	0.62	1.97	1.36	2.23	1.28	1.55	8.57	....
24-hour maximum	2.20	1.40	3.00	2.60	1.40	1.00	0.60	1.40	1.30	2.20	0.80	1.10	3.00	....
Cherrapunji, India:														
Mean	0.45	2.72	9.38	28.19	46.28	95.92	98.51	79.84	37.98	21.26	3.23	0.31	424.07	20
Maximum	8.07	5.39	19.65	52.05	128.27	169.92	147.44	97.83	99.41	51.73	14.02	9.61	560.27	20
24-hour maximum	1.12	1.32	2.80	10.20	16.90	36.40	21.08	22.61	24.66	21.62	3.23	0.15	36.40	5
Karachi, Pakistan:														
Mean	0.48	0.36	0.21	0.11	0.06	0.57	3.10	1.69	0.65	0.02	0.13	0.20	7.58	83
Maximum	3.38	2.94	3.83	4.75	1.85	10.59	18.63	14.15	15.35	1.56	4.66	2.58	28.00	83
24-hour maximum	0.39	1.15	0.21	0.70	0.15	2.04	7.86	5.41	8.11	0.52	0.86	1.83	8.11	18
Trichinopoly, India:														
Mean	0.96	0.48	0.40	1.79	3.18	1.48	1.47	3.88	4.65	7.08	5.54	2.77	33.68	61
Maximum	8.02	3.06	1.26	7.88	10.76	3.66	3.30	9.61	10.80	27.43	13.68	12.98	51.11	18
24-hour maximum	4.32	1.86	0.67	3.74	7.21	1.87	2.12	3.81	3.26	12.56	6.21	5.34	12.56	18
AFRICA														
Casablanca, Morocco														
Mean	2.03	1.98	2.37	1.24	0.78	0.22	0	0.01	0.42	1.21	2.76	2.59	15.61	22
Maximum	4.71	4.92	7.02	3.96	2.62	0.89	0.01	0.16	2.19	4.68	7.26	7.69	21.28	28
24-hour maximum	1.14	1.13	0.76	1.24	3.01	0.26	0.11	0.04	1.80	1.27	2.54	2.58	3.01	10
Huambo, Angola:														
Mean	8.71	8.87	8.42	6.06	1.08	0	0	0.09	0.82	5.14	8.94	9.48	57.61	6
Maximum	17.40	19.87	12.02	10.93	6.93	0	0	0.36	2.48	10.53	12.38	14.93	87.00	6
24-hour maximum	1.31	1.04	1.50	1.20	0.28	....	....	....	0.49	3.04	2.43	2.63	....	2
Cairo, Egypt:														
Mean	0.24	0.16	0.20	0.08	0.08	0.04	0	0	0	0.08	0.08	0.20	1.16	26
Maximum	1.14	0.63	1.02	0.47	0.51	0.08	0	0	0	0.55	0.59	1.06	2.48	21
24-hour maximum	1.70	0.61	0.85	0.81	1.11	0.52	0	0	0	0.57	0.38	1.15	1.70	26

Table C-1. Mean monthly, maximum monthly, and maximum 24-hour precipitation for selected stations throughout the world (continued)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Years Recorded
FORMER SOVIET UNION														
Vladivostok:														
Mean	0.50	0.60	0.80	1.30	2.50	3.50	3.60	5.40	4.80	2.00	1.10	0.70	27.80	----
Maximum	1.50	1.90	4.90	3.00	5.20	7.60	9.20	16.70	13.30	4.70	7.70	1.90	----	38
24-hour maximum	0.90	1.30	2.90	1.00	1.70	2.80	3.10	3.60	4.40	2.80	5.20	1.20	5.20	10
Gizhiga (62° 02' N, 160° 40' E):														
Mean	0.30	0.30	0.40	0.30	0.40	0.70	1.70	1.80	1.20	1.00	0.60	0.40	9.10	16
Maximum	1.60	1.00	1.20	0.90	1.20	1.50	3.90	3.40	3.00	1.80	1.80	1.60	----	16
24-hour maximum	0.40	0.40	0.80	0.60	0.40	0.60	1.80	0.90	0.80	1.10	0.50	0.60	1.80	14
Irkutsk:														
Meak	0.39	0.29	0.28	0.24	1.50	2.17	3.46	3.11	1.69	0.75	0.67	0.63	15.51	----
Maximum	0.94	0.87	0.71	1.38	4.29	6.10	8.90	5.32	4.13	1.85	1.50	2.68	22.68	35
24-hour maximum	0.34	0.22	0.31	0.54	1.71	1.94	2.39	2.68	1.35	0.81	0.51	0.31	2.68	30
Odessa:														
Mean	1.00	1.00	0.95	0.94	1.05	1.98	1.59	0.96	0.82	1.12	0.85	1.11	13.37	18
Maximum	3.98	3.74	2.52	4.02	4.80	6.57	4.65	5.95	5.71	4.21	3.58	3.74	24.88	35
24-hour maximum	1.20	0.99	1.36	0.85	0.85	1.58	2.98	1.28	1.94	2.22	0.78	0.62	2.98	18
Kola:														
Mean	0.50	0.84	0.49	0.62	1.25	1.24	2.27	1.94	1.73	1.26	1.31	0.90	14.35	18
Maximum	1.28	2.41	1.11	2.14	2.35	3.13	4.45	4.37	3.52	1.92	2.37	2.48	20.33	18
24-hour maximum	0.41	0.77	0.28	0.58	0.75	0.78	1.07	1.12	0.73	0.76	0.63	0.65	1.12	18
Batum:														
Mean	10.07	7.33	5.90	5.84	3.41	6.75	6.30	9.24	11.94	9.47	12.88	10.43	99.56	----
24-hour maximum	2.66	2.14	2.89	2.90	3.87	5.82	5.79	4.07	4.64	4.34	3.36	3.01	5.82	10
Stavropol:														
Mean	1.26	1.14	1.34	2.24	2.91	4.06	3.23	1.69	2.32	1.46	2.13	1.73	25.51	26
Maximum	4.21	2.49	3.66	5.08	6.73	7.80	6.38	4.17	4.65	2.91	4.45	5.63	32.99	26
24-Hour maximum	1.43	0.58	1.30	1.50	1.29	3.75	3.11	2.35	2.31	1.19	1.17	1.04	3.75	18
Tiflis:														
Mean	0.59	0.87	1.06	2.32	3.15	2.91	1.77	1.46	2.05	1.57	1.38	0.87	20.00	18
24-hour maximum	0.54	0.61	0.75	1.18	2.11	2.65	1.30	1.43	1.91	1.24	1.14	1.80	2.65	18

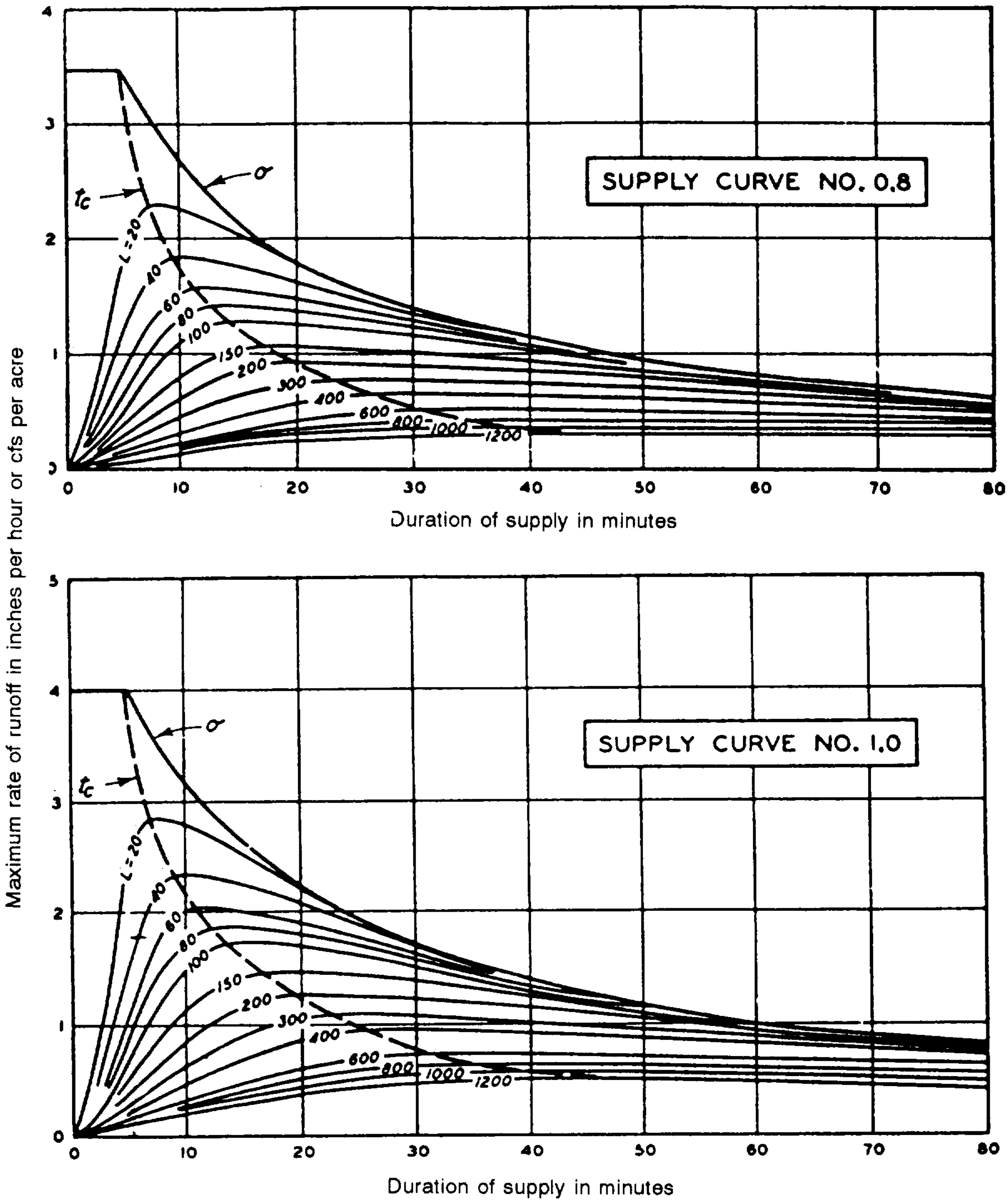
Table C-1. Mean monthly, maximum monthly, and maximum 24-hour precipitation for selected stations throughout the world (in inches) (continued)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual	Years Recorded
NEW ZEALAND, AUSTRALIA, AND NEW GUINEA														
Finschafen, New Guinea (6° 03' S, 147° 52' E):														
Mean	3.13	4.02	3.94	8.56	13.60	17.44	17.62	19.82	13.43	14.70	9.36	4.61	130.23	15
Maximum	7.56	10.59	8.31	17.66	29.64	29.09	29.61	35.43	31.58	41.61	23.23	12.36	181.22	15
24-hour maximum	2.11	4.17	2.64	8.82	6.88	5.83	9.57	6.73	7.44	19.65	7.83	4.49	19.65	----
Darwin, Australia:														
Mean	15.83	12.87	9.88	4.17	0.67	0.16	0.08	0.12	0.51	2.17	4.88	10.39	61.73	52
Maximum	27.86	22.65	21.88	23.74	10.27	1.53	2.56	3.00	2.31	6.28	14.57	22.38	87.22	58
24-hour maximum	11.65	5.12	7.17	4.25	1.18	1.34	1.69	1.06	1.97	3.58	2.76	7.87	11.65	28
Daly Waters, Australia:														
Mean	6.21	6.45	4.79	1.00	0.16	0.32	0.06	0.14	0.27	0.83	2.06	4.08	26.37	43
Maximum	23.16	13.22	14.50	4.39	0.28	0.46	0.56	0	0.93	1.42	10.42	15.76	43.25	12
24-hour maximum	1.74	3.25	2.80	2.78	0	0.11	0.28	0	0.79	0.30	1.34	3.60	3.60	3
Brisbane, Australia:														
Mean	6.50	5.40	5.70	3.90	2.80	2.70	2.20	2.00	2.00	2.50	3.80	4.90	45.40	83
Maximum	27.72	40.39	34.04	15.28	13.85	14.03	8.46	14.67	5.43	9.99	12.40	13.99	88.26	91
24-hour maximum	18.30	10.60	11.20	5.50	5.60	6.00	3.50	4.90	2.50	3.70	4.50	6.60	18.30	63
Wellington, New Zealand:														
Mean	3.30	3.19	3.29	3.80	4.76	4.87	5.55	4.43	3.99	4.19	3.44	3.30	48.11	69
Maximum	10.13	8.89	9.94	12.15	11.80	9.53	12.17	9.88	11.05	12.94	9.99	12.46	67.68	69
24-hour maximum	4.50	6.30	5.70	4.90	5.70	3.00	3.10	3.70	3.80	3.50	2.70	3.50	6.30	63
ARCTIC (not included previously)														
Jan Mayen (70° 59' N, 8° 20' W):														
Mean	1.54	1.69	1.10	0.94	0.51	0.59	0.79	1.06	2.52	2.20	1.42	0.98	15.34	7
Maximum	3.41	4.09	3.03	3.07	1.41	1.82	4.05	4.05	4.75	4.80	3.66	4.14	----	16
Coppermine, Canada:														
Mean	0.44	0.56	0.73	0.82	0.52	0.86	1.34	1.76	1.19	1.01	0.80	0.53	10.46	5
Maximum	0.74	2.10	1.95	3.45	0.89	2.34	1.84	3.20	1.95	2.23	2.20	0.90	----	7
Mean snowfall	3.50	5.60	7.30	6.80	3.90	2.90	0.10	0.40	4.70	9.10	8.60	5.30	58.20	----
Uelen, Siberia:														
Mean	0.63	0.56	0.39	0.49	0.43	0.50	1.61	1.79	2.09	1.85	0.54	0.96	11.84	8
Maximum	2.28	1.85	0.67	0.98	1.08	0.94	2.11	2.64	4.70	6.18	0.88	2.38	----	8
Average thickness of snow cover at Anadyr	12.80	17.60	20.30	21.50	17.70	1.80	----	----	0.10	1.70	4.60	8.50	----	----
ANTARCTIC														
Little America:														
1929 snowfall (unmelted depth in inches)	7.30	12.70	24.20	5.60	7.60	16.20	8.90	11.30	2.40	2.00	2.90	2.30	103.50	1



Note: L = effective length of overland or channel flow, in feet.
tc = critical duration of supply, in minutes, assuming surface storage as negligible.
σ = rate of supply, in inches per hour.

Figure C-1. Rates of overland flow corresponding to standard supply curves, supply curve number 0.4 and 0.6; n = 0.40, S = 1 percent

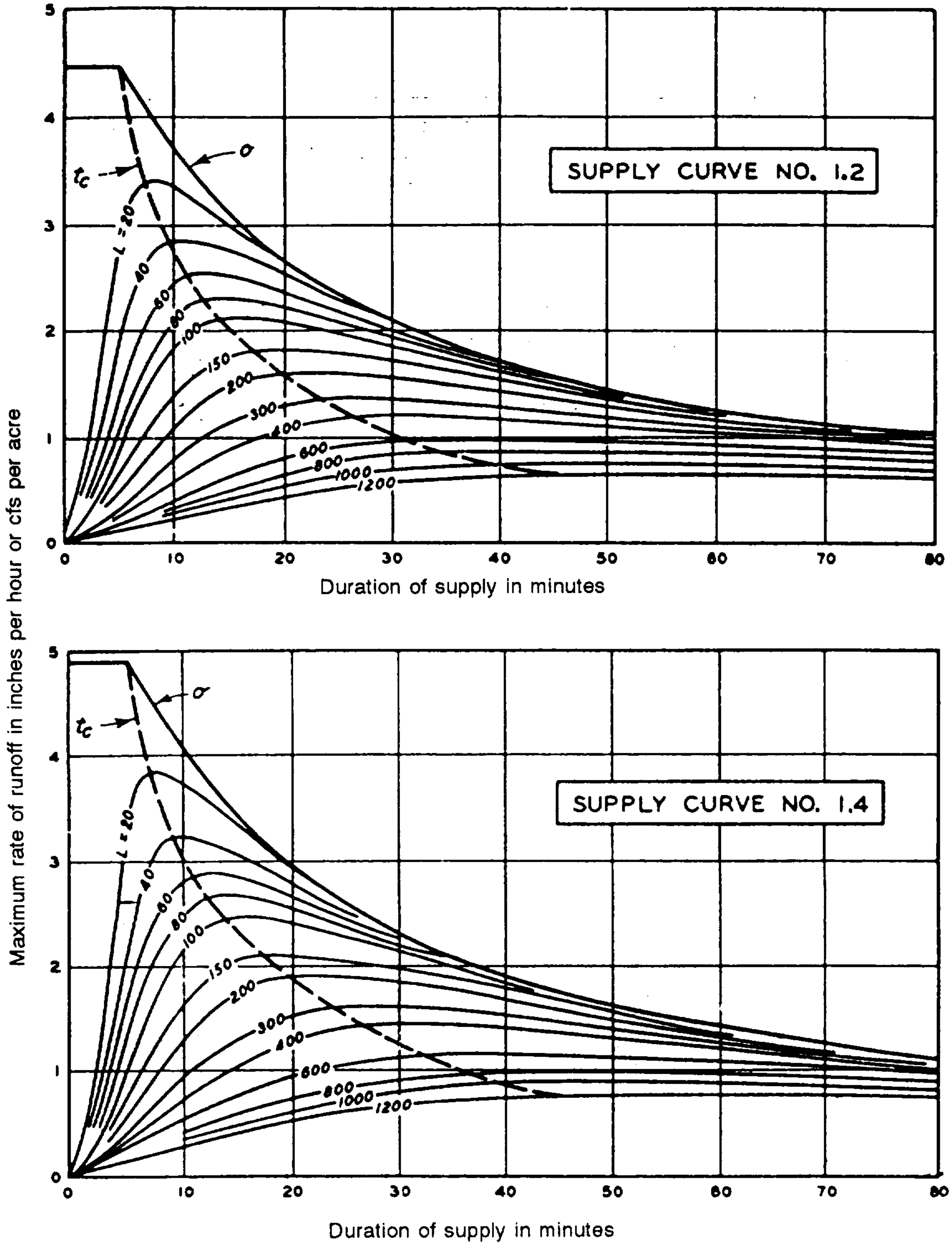


Note: L = effective length of overland or channel flow, in feet.

tc = critical duration of supply, in minutes, assuming surface storage as negligible.

σ = rate of supply, in inches per hour.

Figure C-2. Rates of overland flow corresponding to standard supply curves, supply curve numbers 0.8 and 1.0; n = 0.40, S = 1 percent

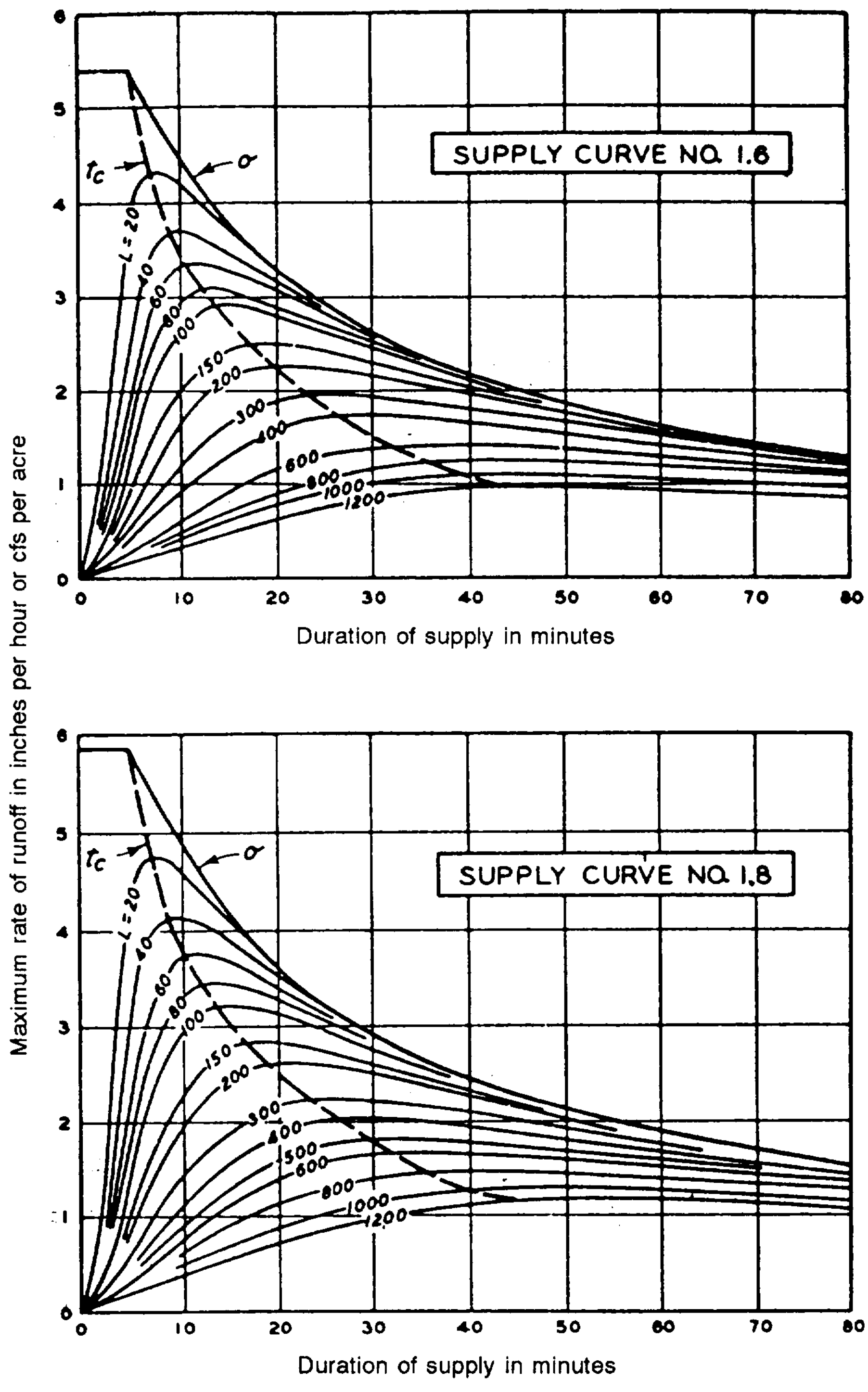


Note: L = effective length of overland or channel flow, in feet.

tc = critical duration of supply, in minutes, assuming surface storage as negligible.

σ = rate of supply, in inches per hour.

Figure C-3. Rates of overland flow corresponding to standard supply curves, supply curve numbers 1.2 and 1.4; n = 0.40, S = 1 percent

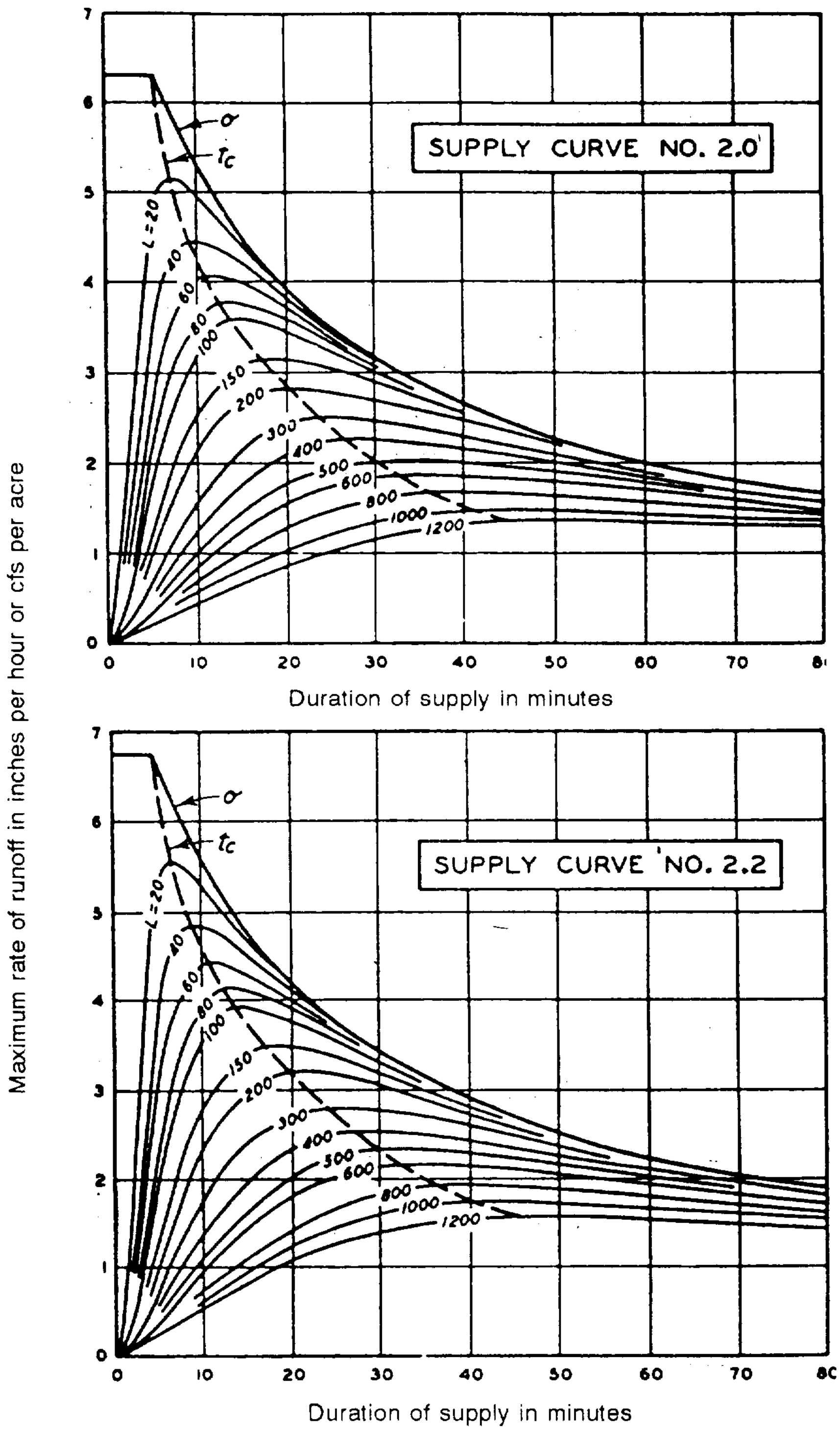


Note: L = effective length of overland or channel flow, in feet.

tc = critical duration of supply, in minutes, assuming surface storage as negligible.

σ = rate of supply, in inches per hour.

Figure C-4. Rates of overland flow corresponding to standard supply curves, supply curve numbers 1.6 and 1.8; n = 0.40, S = 1 percent

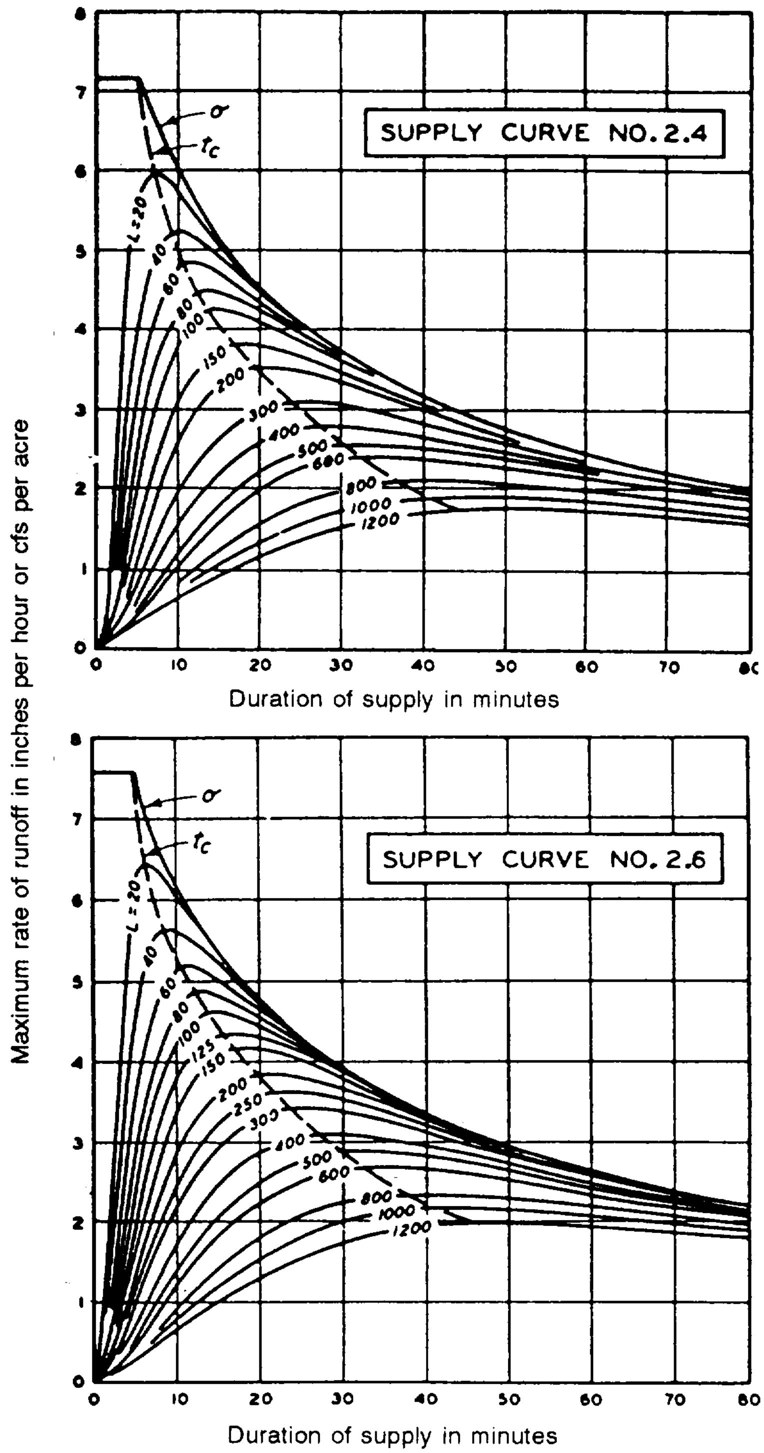


Note: L = effective length of overland or channel flow, in feet.

tc = critical duration of supply, in minutes, assuming surface storage as negligible.

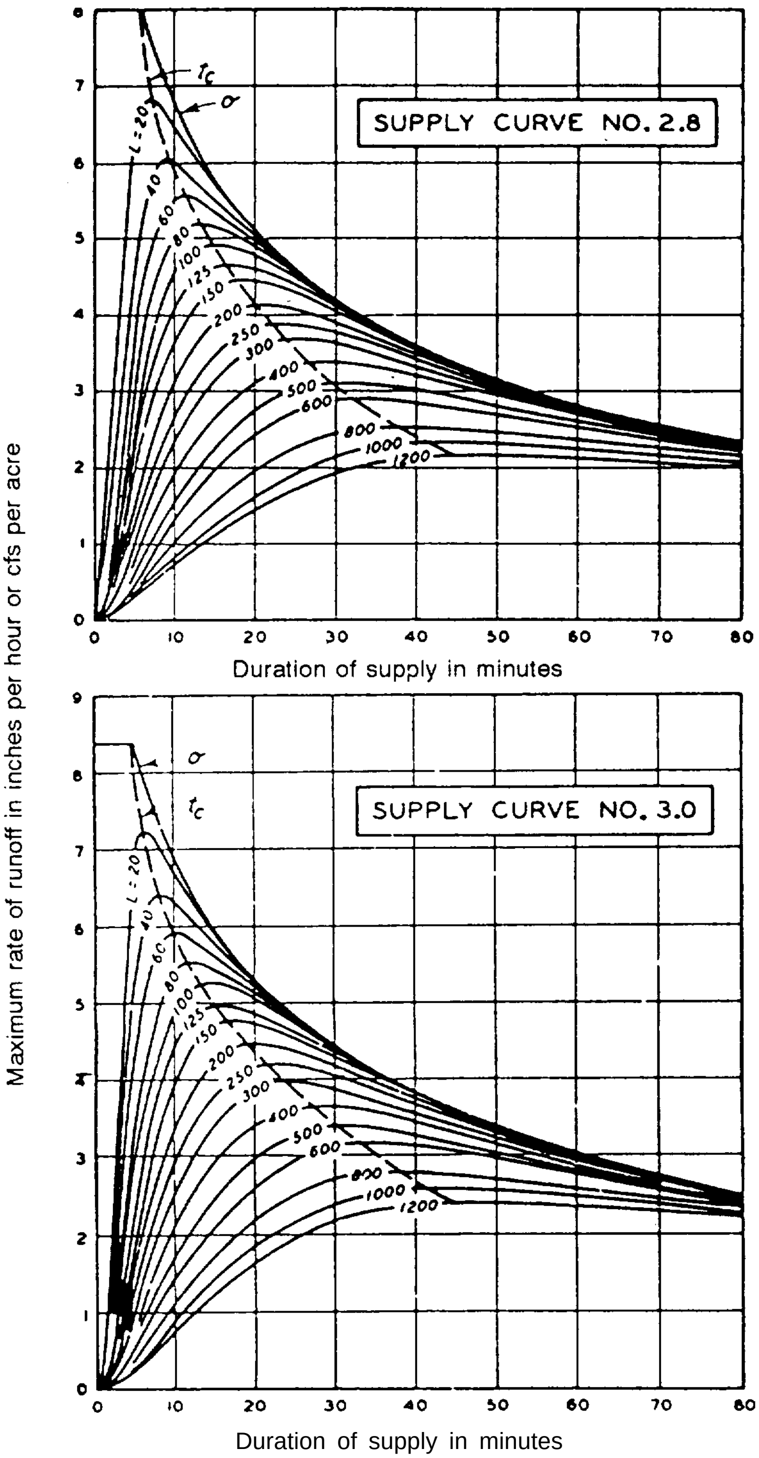
σ = rate of supply, in inches per hour.

Figure C-5. Rates of overland flow corresponding to standard supply curves, supply curve numbers 2.0 and 2.2; n = 0.40, S = 1 percent



Note: L = effective length of overland or channel flow, in feet.
tc = critical duration of supply, in minutes, assuming surface storage as negligible.
σ = rate of supply, in inches per hour.

Figure C-6. Rates of overland flo w corresponding to standard supply curves, supply curve numbers 2.4 and 2.6; n = 0.40, S = 1 percent



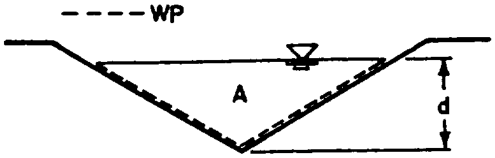
Note: L = effective length of overland or channel flow, in feet.

tc = critical duration of supply, in minutes, assuming surface storage as negligible.

σ = rate of supply, in inches per hour.

Figure C-7,* Rates *of overland flow corresponding to standard supply curves, supply curve numbers 2.8 and 3.0; n = 0.40, S = 1 percent

Table C-2. Hydraulic radius (R) and area (A) of symmetrical triangular channels

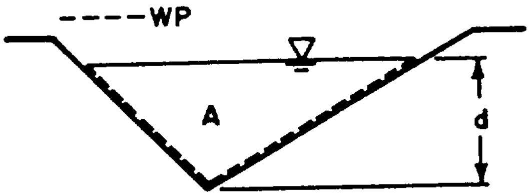


R= A/WP

Depth, d (feet)	Slope ratio 1:1 A	R	1½:1 A	R	2:1 A	R	2½:1 A	R	3:1 A	R	4:1 A	R
0.5	0.25	0.18	0.38	0.21	0.50	0.22	0.63	0.23	0.75	0.24	1.00	0.24
0.6	0.36	0.21	0.54	0.25	0.72	0.27	0.90	0.28	1.08	0.28	1.44	0.29
0.7	0.49	0.25	0.74	0.29	0.98	0.31	1.23	0.32	1.47	0.33	1.96	0.34
0.8	0.64	0.28	0.96	0.33	1.28	0.36	1.60	0.37	1.92	0.38	2.56	0.39
0.9	0.81	0.32	1.21	0.37	1.62	0.40	2.03	0.42	2.43	0.43	3.24	0.44
1.0	1.00	0.35	1.50	0.42	2.00	0.45	2.50	0.46	3.00	0.47	4.00	0.49
1.1	1.21	0.39	1.82	0.46	2.42	0.49	3.03	0.51	3.63	0.52	4.84	0.53
1.2	1.44	0.42	2.16	0.50	2.88	0.54	3.60	0.56	4.32	0.57	5.76	0.58
1.3	1.69	0.46	2.54	0.54	3.38	0.58	4.23	0.60	5.07	0.62	6.76	0.63
1.4	1.96	0.50	2.94	0.58	3.92	0.63	4.90	0.65	5.88	0.66	7.84	0.68
1.5	2.25	0.53	3.38	0.62	4.50	0.67	5.63	0.70	6.75	0.71	9.00	0.73
1.6	2.56	0.57	3.84	0.67	5.12	0.72	6.40	0.74	7.68	0.76	10.24	0.78
1.7	2.89	0.60	4.34	0.71	5.78	0.76	7.23	0.79	8.67	0.80	11.56	0.83
1.8	3.24	0.64	4.86	0.75	6.48	0.80	8.10	0.84	9.72	0.85	12.96	0.87
1.9	3.61	0.67	5.42	0.79	7.22	0.85	9.03	0.88	10.83	0.90	14.44	0.92
2.0	4.00	0.71	6.00	0.83	8.00	0.90	10.00	0.93	12.00	0.95	16.00	0.97
2.5	6.25	0.88	9.38	1.04	12.50	1.12	15.63	1.16	18.75	1.19	25.00	1.21
3.0	9.00	1.06	13.50	1.25	18.00	1.34	22.50	1.39	27.00	1.42	36.00	1.46
3.5	12.25	1.24	18.38	1.45	24.50	1.56	30.62	1.62	36.75	1.66	49.00	1.70
4.0	16.00	1.41	24.00	1.66	32.00	1.78	40.00	1.85	48.00	1.90	64.00	1.94

	5:1 A	R	6:1 A	R	7:1 A	R	8:1 A	R	9:1 A	R	10:1 A	R
0.5	1.25	0.25	1.50	0.25	1.75	0.25	2.00	0.25	2.25	0.25	2.50	0.25
0.6	1.80	0.29	2.16	0.30	2.52	0.30	2.88	0.30	3.24	0.30	3.60	0.30
0.7	2.45	0.34	2.94	0.35	3.43	0.35	3.92	0.35	4.41	0.35	4.90	0.35
0.8	3.20	0.39	3.84	0.39	4.48	0.40	5.12	0.40	5.76	0.40	6.40	0.40
0.9	4.05	0.44	4.86	0.44	5.67	0.45	6.48	0.45	7.29	0.45	8.10	0.45
1.0	5.00	0.49	6.00	0.49	7.00	0.49	8.00	0.50	9.00	0.50	10.00	0.50
1.1	6.05	0.54	7.26	0.54	8.47	0.55	9.68	0.55	10.89	0.55	12.10	0.55
1.2	7.20	0.59	8.64	0.59	10.08	0.59	11.52	0.60	12.96	0.60	14.40	0.60
1.3	8.45	0.64	10.14	0.64	11.83	0.64	13.52	0.64	15.21	0.65	16.90	0.65
1.4	9.80	0.69	11.76	0.69	13.72	0.69	15.68	0.69	17.64	0.70	19.60	0.70
1.5	11.25	0.74	13.50	0.74	15.75	0.74	18.00	0.74	20.25	0.75	22.50	0.75
1.6	12.80	0.78	15.36	0.79	17.92	0.79	20.48	0.79	23.04	0.80	25.60	0.80
1.7	14.45	0.83	17.34	0.84	20.23	0.84	23.12	0.84	26.01	0.84	28.90	0.85
1.8	16.20	0.88	19.44	0.89	22.68	0.89	25.92	0.89	29.16	0.89	32.40	0.90
1.9	18.05	0.93	21.66	0.94	25.27	0.94	28.88	0.94	32.49	0.94	36.10	0.95
2.0	20.00	0.98	24.00	0.99	28.00	0.99	32.00	0.99	36.00	0.99	40.00	1.00
2.5	31.25	1.23	37.50	1.23	43.75	1.24	50.00	1.24	56.25	1.24	62.50	1.24
3.0	45.00	1.47	54.00	1.48	63.00	1.48	72.00	1.49	81.00	1.49	90.00	1.49
3.5	61.25	1.72	73.50	1.72	85.75	1.73	98.00	1.74	110.25	1.74	122.50	1.74
4.0	80.00	1.96	96.00	1.97	112.00	1.98	128.00	1.98	144.00	1.98	160.00	1.99

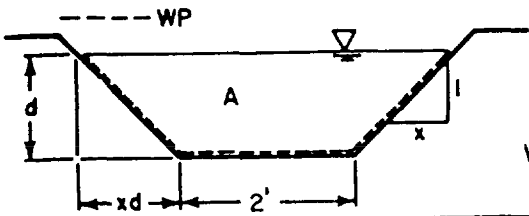
Table C-3. Hydraulic radius (R) and area (A) of nonsymmetrical triangular channels



R = A/WP

Depth, d (feet)	Slope ratio											
	1:1 - 3:1		1½:1 - 3:1		2:1 - 3:1		2½:1 - 3:1		4:1 - 3:1		5:1 - 3:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	0.50	0.22	0.56	0.23	0.63	0.23	0.69	0.23	0.88	0.24	1.00	0.24
0.6	0.72	0.26	0.81	0.27	0.90	0.28	0.99	0.28	1.26	0.29	1.44	0.29
0.7	0.98	0.31	1.10	0.32	1.23	0.32	1.35	0.33	1.72	0.34	1.96	0.34
0.8	1.28	0.35	1.44	0.36	1.60	0.37	1.76	0.38	2.24	0.38	2.56	0.39
0.9	1.62	0.39	1.82	0.41	2.03	0.42	2.23	0.42	2.84	0.43	3.24	0.44
1.0	2.00	0.44	2.25	0.45	2.50	0.46	2.75	0.47	3.50	0.48	4.00	0.48
1.1	2.42	0.48	2.72	0.50	3.03	0.51	3.33	0.52	4.24	0.53	4.84	0.53
1.2	2.88	0.52	3.24	0.54	3.60	0.56	3.96	0.56	5.04	0.58	5.76	0.58
1.3	3.38	0.57	3.80	0.59	4.23	0.60	4.65	0.61	5.92	0.63	6.76	0.63
1.4	3.92	0.61	4.41	0.63	4.90	0.65	5.39	0.66	6.86	0.67	7.84	0.68
1.5	4.50	0.66	5.06	0.68	5.63	0.69	6.19	0.70	7.88	0.72	9.00	0.73
1.6	5.12	0.70	5.76	0.73	6.40	0.74	7.04	0.75	8.96	0.77	10.24	0.77
1.7	5.78	0.74	6.50	0.77	7.23	0.79	7.95	0.80	10.12	0.82	11.56	0.82
1.8	6.48	0.79	7.29	0.82	8.10	0.83	8.91	0.85	11.34	0.86	12.96	0.87
1.9	7.22	0.83	8.12	0.86	9.03	0.88	9.93	0.89	12.64	0.91	14.44	0.92
2.0	8.00	0.87	9.00	0.91	10.00	0.93	11.00	0.94	14.00	0.96	16.00	0.97
2.1	8.82	0.92	9.92	0.95	11.03	0.97	12.13	0.99	15.44	1.00	17.64	1.02
2.2	9.68	0.96	10.89	1.00	12.10	1.02	13.31	1.03	16.94	1.06	19.36	1.07
2.3	10.58	1.01	11.90	1.04	13.23	1.07	14.55	1.08	18.52	1.10	21.16	1.11
2.4	11.52	1.05	12.96	1.09	14.40	1.11	15.84	1.13	20.16	1.15	23.04	1.16
2.5	12.50	1.09	14.06	1.13	15.63	1.16	17.19	1.17	21.87	1.20	25.00	1.21
2.6	13.52	1.14	15.21	1.18	16.90	1.20	18.59	1.22	23.66	1.25	27.04	1.26
2.7	14.58	1.18	16.40	1.22	18.23	1.25	20.05	1.27	25.52	1.30	27.16	1.31
2.8	15.68	1.22	17.64	1.27	19.60	1.30	21.56	1.32	27.44	1.35	31.36	1.36
2.9	16.82	1.27	18.92	1.31	21.03	1.34	23.13	1.36	29.44	1.39	33.64	1.40
3.0	18.00	1.31	20.25	1.36	22.50	1.39	24.75	1.41	31.50	1.44	36.00	1.45

Table C-4. Hydraulic radius (R) and area (A) of symmetrical trapezoidal channels (2-foot bottom width)



$$A = xd^2 + 2d$$

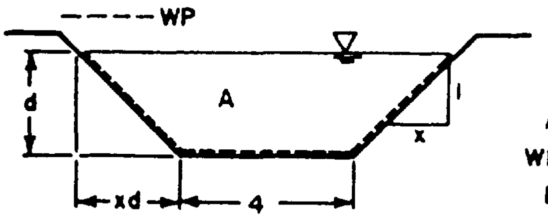
$$WP = 2d\sqrt{1 + x^2} + 2$$

$$R = A/WP$$

Depth, d (feet)	Slope ratio 1:1		1½:1		2:1		2½:1		3:1		4:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	1.25	0.37	1.38	0.36	1.50	0.35	1.63	0.35	1.75	0.34	2.00	0.33
0.6	1.56	0.42	1.74	0.42	1.92	0.41	2.10	0.40	2.28	0.39	2.64	0.38
0.7	1.89	0.47	2.14	0.47	2.28	0.44	2.63	0.46	2.87	0.45	3.36	0.43
0.8	2.24	0.53	2.56	0.52	2.88	0.52	3.20	0.51	3.52	0.50	4.16	0.48
0.9	2.61	0.51	3.01	0.57	3.42	0.57	3.83	0.56	4.23	0.55	5.04	0.54
1.0	3.00	0.62	3.50	0.62	4.00	0.62	4.50	0.61	5.00	0.60	6.00	0.59
1.1	3.41	0.67	4.02	0.67	4.63	0.67	5.23	0.66	5.84	0.65	7.05	0.64
1.2	3.84	0.71	4.56	0.72	5.28	0.72	6.00	0.71	6.72	0.70	8.16	0.69
1.3	4.29	0.76	5.14	0.77	5.98	0.77	6.83	0.76	7.67	0.75	9.36	0.74
1.4	4.76	0.80	5.74	0.81	6.72	0.81	7.70	0.81	8.68	0.80	10.64	0.79
1.5	5.25	0.84	6.38	0.86	7.50	0.86	8.63	0.86	9.75	0.85	12.00	0.84
1.6	5.76	0.88	7.04	0.91	8.32	0.91	9.60	0.90	10.88	0.90	13.44	0.88
1.7	6.29	0.92	7.74	0.95	9.18	0.96	10.63	0.95	12.07	0.95	14.96	0.93
1.8	6.84	0.96	8.46	1.00	10.08	1.00	11.70	1.00	13.32	1.00	16.56	0.98
1.9	7.41	1.00	9.22	1.04	11.02	1.05	12.83	1.05	14.63	1.04	18.24	1.03
2.0	8.00	1.04	10.00	1.09	12.00	1.10	14.00	1.10	16.00	1.09	20.00	1.08
2.5	11.25	1.24	14.38	1.30	17.50	1.33	20.63	1.33	23.75	1.33	30.00	1.33
3.0	15.00	1.43	19.50	1.52	4.00	1.56	28.30	1.57	33.00	1.57	42.00	1.57

	5:1		6:1		7:1		8:1		9:1		10:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	2.25	0.32	2.50	0.31	2.75	0.30	3.00	0.30	3.25	0.29	3.50	0.29
0.6	3.00	0.37	3.36	0.36	3.72	0.35	4.08	0.35	4.44	0.34	4.80	0.34
0.7	3.85	0.42	4.34	0.41	4.83	0.41	5.32	0.40	5.81	0.39	6.30	0.39
0.8	4.80	0.47	5.44	0.46	6.08	0.46	6.72	0.45	7.36	0.45	8.00	0.44
0.9	5.85	0.52	6.66	0.51	7.47	0.51	8.28	0.50	9.09	0.50	9.90	0.49
1.0	7.00	0.51	8.00	0.56	9.00	0.56	10.00	0.55	11.00	0.55	12.00	0.54
1.1	8.25	0.62	9.47	0.62	10.68	0.61	11.89	0.60	13.10	0.60	14.31	0.59
1.2	9.60	0.67	11.04	0.67	12.48	0.66	13.92	0.65	15.36	0.65	16.80	0.64
1.3	11.05	0.72	12.74	0.72	14.43	0.71	16.12	0.70	17.81	0.70	19.50	0.69
1.4	12.60	0.77	14.50	0.77	16.52	0.76	18.48	0.75	20.44	0.75	22.40	0.74
1.5	14.25	0.82	16.50	0.81	18.75	0.81	21.00	0.80	23.25	0.80	25.50	0.79
1.6	16.00	0.87	18.56	0.86	21.12	0.86	23.68	0.85	26.24	0.85	28.80	0.84
1.7	17.85	0.92	20.74	0.91	23.63	0.91	26.52	0.90	29.41	0.90	32.30	0.89
1.8	19.80	0.97	23.04	0.96	26.28	0.96	29.52	0.95	32.76	0.95	36.00	0.94
1.9	21.85	1.02	25.46	1.01	29.07	1.01	32.68	1.00	36.29	1.00	39.90	0.99
2.0	24.00	1.07	28.00	1.06	32.00	1.06	36.00	1.05	40.00	1.05	44.00	1.04
2.5	36.25	1.32	42.50	1.31	48.75	1.30	55.00	1.30	61.25	1.30	67.50	1.29
3.0	51.00	1.56	60.00	1.56	69.00	1.55	78.00	1.55	87.00	1.54	96.00	1.54

Table C-5. Hydraulic radius (R) and area (A) of symmetrical trapezoidal channels (4-foot bottom width)



$A = xd^2 + 4d$

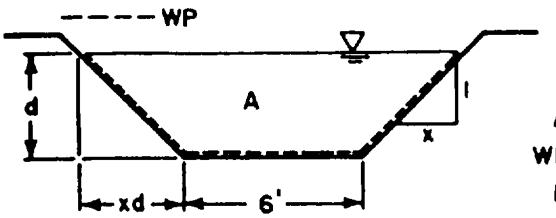
$WP = 2d\sqrt{1 + x^2} + 4$

$R = A/WP$

Depth, d (feet)	Slope ratio											
	1:1		1½:1		2:1		2½:1		3:1		4:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	2.25	0.41	2.38	0.41	2.50	0.40	2.63	0.39	2.75	0.39	3.00	0.37
0.6	2.76	0.48	2.94	0.48	3.12	0.47	3.30	0.46	3.48	0.45	3.84	0.43
0.7	3.29	0.55	3.54	0.54	3.78	0.53	4.03	0.52	4.27	0.50	4.76	0.49
0.8	3.84	0.61	4.16	0.60	4.48	0.59	4.80	0.58	5.12	0.57	5.76	0.54
0.9	4.41	0.67	4.82	0.66	5.22	0.65	5.63	0.64	6.03	0.62	6.84	0.60
1.0	5.00	0.73	5.50	0.72	6.00	0.71	6.50	0.69	7.00	0.68	8.00	0.65
1.1	5.61	0.79	6.22	0.78	6.82	0.76	7.43	0.75	8.03	0.73	9.24	0.71
1.2	6.24	0.84	6.96	0.84	7.68	0.82	8.40	0.80	9.12	0.79	10.56	0.76
1.3	6.89	0.90	7.74	0.89	8.58	0.87	9.43	0.86	10.27	0.84	11.96	0.81
1.4	7.56	0.95	8.54	0.94	9.52	0.93	10.50	0.91	1148	0.89	13.44	0.86
1.5	8.25	1.00	9.38	1.00	10.50	0.98	11.63	0.96	12.75	0.94	15.00	0.92
1.6	8.96	1.05	10.24	1.05	11.52	1.03	12.80	1.01	14.08	1.00	16.64	0.97
1.7	9.69	1.10	11.14	1.10	12.58	1.08	14.03	1.07	15.47	1.05	18.36	1.02
1.8	10.44	1.15	12.06	1.15	13.68	1.14	15.30	1.12	16.92	1.10	20.16	1.02
1.9	11.21	1.20	13.02	1.20	14.82	1.19	16.63	1.17	18.43	1.15	22.04	1.12
2.0	12.00	1.24	14.00	1.25	16.00	1.24	18.00	1.22	20.00	1.20	24.00	1.17
2.5	16.25	1.47	19.38	1.48	22.50	1.48	25.63	1.47	28.75	1.45	35.00	1.42
3.0	21.00	1.68	25.50	1.72	30.00	1.72	34.50	1.71	39.00	1.70	48.00	1.67

	5:1		6:1		7:1		8:1		9:1		10:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	3.25	0.36	3.50	0.35	3.75	0.34	4.00	0.33	4.25	0.32	4.50	0.32
0.6	4.20	0.42	4.56	0.40	4.92	0.39	5.28	0.38	5.64	0.38	6.00	0.37
0.7	5.25	0.47	5.74	0.46	6.23	0.45	6.72	0.44	7.21	0.43	7.70	0.43
0.8	6.40	0.53	7.04	0.51	7.68	0.50	8.32	0.49	8.96	0.49	9.60	0.48
0.9	7.65	0.58	8.46	0.56	9.27	0.55	10.08	0.55	10.89	0.54	11.70	0.53
1.0	9.00	0.64	10.00	0.62	11.00	0.61	12.00	0.60	13.00	0.59	14.00	0.58
1.1	10.45	0.69	11.66	0.67	12.87	0.66	14.08	0.65	15.29	0.64	16.50	0.63
1.2	12.00	0.74	13.44	0.72	14.88	0.71	16.32	0.70	17.76	0.69	19.20	0.68
1.3	13.65	0.79	15.34	0.77	17.03	0.76	18.72	0.75	20.41	0.74	22.10	0.73
1.4	15.40	0.84	17.36	0.83	19.32	0.81	21.28	0.80	23.24	0.79	25.20	0.78
1.5	17.25	0.89	19.50	0.88	21.75	0.86	24.00	0.85	26.25	0.84	28.50	0.83
1.6	19.20	0.94	21.76	0.93	24.32	0.91	26.88	0.90	29.44	0.89	32.00	0.89
1.7	21.25	1.00	24.14	0.98	27.03	0.96	29.92	0.95	32.81	0.94	35.70	0.94
1.8	23.40	1.05	26.64	1.03	29.88	1.01	33.12	1.00	36.36	0.99	39.60	0.99
1.9	25.65	1.10	29.26	1.08	32.87	1.06	36.48	1.05	40.09	1.04	43.70	1.04
2.0	28.00	1.15	32.00	1.14	36.00	1.12	40.00	1.10	44.00	1.09	48.00	1.09
2.5	41.25	1.40	47.50	1.38	53.75	1.37	60.00	1.35	66.25	1.34	72.50	1.34
3.0	57.00	1.65	66.00	1.64	75.00	1.63	84.00	1.62	93.00	1.62	102.00	1.61

Table C-6. Hydraulic radius (R) and area (A) of symmetrical trapezoidal channels (6-foot bottom width)



$A = xd^2 + 6d$

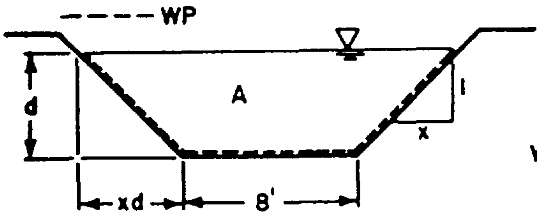
$WP = 2d\sqrt{1 + x^2} + 6$

$R = A/WP$

Depth, d (feet)	Slope ratio 1:1 A	R	1½:1 A	R	2:1 A	R	2½:1 A	R	3:1 A	R	4:1 A	R
0.5	3.25	0.44	3.38	0.43	3.50	0.42	3.63	0.42	3.50	0.41	4.00	0.40
0.6	3.96	0.51	4.14	0.51	4.32	0.50	4.50	0.49	4.68	0.48	5.04	0.46
0.7	4.69	0.59	4.94	0.58	5.18	0.57	5.43	0.56	5.67	0.54	6.16	0.52
0.8	5.44	0.66	5.76	0.65	6.08	0.63	6.40	0.62	6.72	0.61	7.36	0.58
0.9	6.21	0.73	6.62	0.72	7.02	0.70	7.43	0.68	7.83	0.67	8.64	0.64
1.0	7.00	0.79	7.50	0.78	8.00	0.76	8.50	0.75	9.00	0.73	10.00	0.70
1.1	7.81	0.86	8.42	0.85	9.02	0.83	9.63	0.80	10.23	0.79	11.44	0.76
1.2	8.64	0.92	9.36	0.91	10.08	0.89	10.80	0.87	11.52	0.85	12.96	0.82
1.3	9.49	0.98	10.34	0.97	11.18	0.95	12.03	0.93	12.87	0.91	14.56	0.87
1.4	10.36	1.04	11.34	1.03	12.32	1.00	13.30	0.98	14.28	0.96	16.24	0.93
1.5	11.25	1.10	12.38	1.08	13.50	1.06	14.63	1.04	15.75	1.01	18.00	0.98
1.6	12.16	1.16	13.44	1.14	14.72	1.12	16.00	1.09	17.28	1.07	19.84	1.03
1.7	13.09	1.22	14.54	1.20	15.98	1.17	17.43	1.15	18.87	1.13	21.76	1.09
1.8	14.04	1.27	15.66	1.25	17.28	1.23	18.90	1.20	20.52	1.18	23.76	1.14
1.9	15.01	1.32	16.82	1.30	18.62	1.28	20.43	1.25	22.23	1.24	25.84	1.19
2.0	16.00	1.37	18.00	1.36	20.00	1.34	22.00	1.31	24.00	1.29	28.00	1.24
2.5	21.25	1.61	24.38	1.61	27.50	1.60	30.63	1.58	33.75	1.55	40.00	1.50
3.0	27.00	1.86	31.50	1.87	36.00	1.85	40.50	1.83	45.00	1.80	54.00	1.76

	5:1 A	R	6:1 A	R	7:1 A	R	8:1 A	R	9:1 A	R	10:1 A	R
0.5	4.25	0.38	4.50	0.37	4.75	0.36	5.00	0.36	5.25	0.35	5.50	0.34
0.6	5.90	0.45	5.76	0.43	6.12	0.42	6.48	0.41	6.84	0.41	7.20	0.40
0.7	6.65	0.51	7.14	0.49	7.63	0.48	8.12	0.47	8.61	0.46	9.10	0.45
0.8	8.00	0.56	8.64	0.55	9.28	0.54	9.92	0.53	10.56	0.49	11.20	0.51
0.9	9.45	0.62	10.26	0.61	11.07	0.59	11.88	0.58	12.69	0.57	13.50	0.55
1.0	11.00	0.68	12.00	0.66	13.00	0.65	22.12	0.63	15.00	0.62	16.00	0.61
1.1	12.65	0.73	13.86	0.72	15.07	0.70	16.28	0.69	17.49	0.67	18.70	0.67
1.2	14.40	0.79	15.84	0.77	17.28	0.75	18.72	0.74	20.16	0.75	21.60	0.72
1.3	16.25	0.85	17.94	0.82	19.63	0.80	21.32	0.79	23.01	0.78	24.70	0.77
1.4	18.20	0.90	20.16	0.87	22.12	0.85	24.08	0.84	26.04	0.83	28.00	0.82
1.5	20.25	0.95	22.50	0.92	24.75	0.91	27.00	0.90	29.25	0.88	31.50	0.87
1.6	22.40	1.00	24.96	0.98	27.52	0.96	39.08	0.95	32.64	0.93	35.20	0.92
1.7	24.45	1.06	27.54	1.03	30.43	1.01	33.32	1.00	36.21	0.97	39.10	0.97
1.8	27.00	1.11	30.24	1.08	33.48	1.06	36.72	1.08	39.96	1.04	43.20	1.02
1.9	29.45	1.16	33.06	1.14	36.67	1.12	40.28	1.10	43.89	1.09	47.50	1.07
2.0	32.00	1.21	36.00	1.19	40.00	1.17	44.00	1.15	48.00	1.13	52.00	1.12
2.5	46.25	1.47	52.50	1.45	58.75	1.46	65.00	1.40	71.25	1.39	77.50	1.38
3.0	63.00	1.72	72.00	1.70	81.00	1.71	90.00	1.65	99.00	1.66	108.00	1.65

Table C-7. Hydraulic radius (R) and area (A) of symmetrical trapezoidal channels (8-foot bottom width)

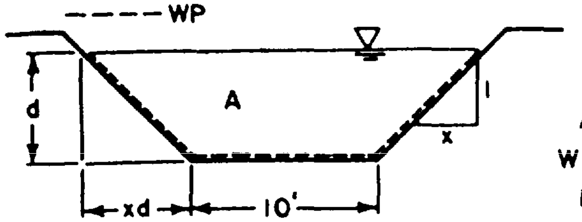


$$A = xd^2 + 8d$$
$$WP = 2d\sqrt{1 + x^2} + 8$$
$$R = A/WP$$

Depth, d (feet)	Slope ratio 1:1		1½:1		2:1		2½:1		3:1		4:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	4.25	0.45	4.38	0.45	4.50	0.44	4.63	0.43	4.75	0.43	5.00	0.41
0.6	5.16	0.53	5.34	0.53	5.52	0.52	5.70	0.51	5.88	0.50	6.24	0.48
0.7	6.09	0.61	6.34	0.60	6.58	0.59	6.83	0.58	7.07	0.57	7.56	0.55
0.8	7.04	0.69	7.36	0.68	7.68	0.66	8.00	0.65	8.32	0.64	8.96	0.61
0.9	8.01	0.76	8.42	0.75	8.82	0.73	9.22	0.72	9.63	0.70	10.44	0.68
1.0	9.00	0.83	9.50	0.82	10.00	0.80	10.50	0.78	11.00	0.77	12.00	0.74
1.1	10.01	0.90	10.62	0.89	11.22	0.87	11.83	0.85	12.43	0.83	13.64	0.80
1.2	11.04	0.97	11.76	0.95	12.48	0.93	13.20	0.91	13.92	0.89	15.36	0.86
1.3	12.09	1.04	12.94	1.02	13.78	1.00	14.63	0.98	15.97	0.95	17.16	0.92
1.4	13.16	1.10	14.14	1.08	15.12	1.06	16.10	1.04	17.08	1.01	19.04	0.97
1.5	14.25	1.16	15.38	1.14	16.50	1.12	17.63	1.10	18.75	1.07	21.00	1.03
1.6	15.36	1.23	16.64	1.21	17.92	1.18	19.20	1.16	20.48	1.13	23.04	1.09
1.7	16.49	1.29	17.44	1.27	19.38	1.24	20.83	1.22	22.27	1.19	25.16	1.14
1.8	17.64	1.35	19.26	1.33	20.88	1.30	22.50	1.27	29.12	1.24	27.36	1.20
1.9	18.81	1.41	20.63	1.40	22.42	1.36	24.23	1.33	26.03	1.30	29.64	1.25
2.0	20.00	1.46	22.00	1.45	24.00	1.42	26.00	1.39	28.00	1.36	32.00	1.31
2.5	26.25	1.76	29.38	1.72	32.50	1.69	35.63	1.66	38.75	1.63	45.00	1.57
3.0	33.00	2.00	37.50	1.99	42.00	1.96	46.50	1.93	51.00	1.89	60.00	1.83

	5:1		6:1		7:1		8:1		9:1		10:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	5.25	0.40	5.50	0.39	5.75	0.38	6.00	0.37	6.25	0.36	6.50	0.36
0.6	6.00	0.47	6.96	0.44	7.32	0.44	7.68	0.43	8.04	0.43	8.40	0.42
0.7	8.05	0.53	8.54	0.52	9.03	0.50	9.52	0.49	10.01	0.48	10.50	0.48
0.8	9.60	0.59	10.24	0.58	10.88	0.56	11.20	0.54	12.16	0.54	12.80	0.53
0.9	11.25	0.65	12.06	0.64	12.87	0.63	13.68	0.61	14.49	0.60	15.30	0.59
1.0	13.00	0.71	14.00	0.70	15.00	0.68	16.00	0.66	17.00	0.65	18.00	0.64
1.1	14.85	0.77	16.06	0.75	17.27	0.73	18.48	0.72	19.69	0.71	20.90	0.69
1.2	16.80	0.83	18.24	0.81	19.68	0.79	21.12	0.77	22.56	0.76	24.00	0.74
1.3	18.85	0.88	20.54	0.86	22.23	0.84	23.92	0.83	25.61	0.81	27.30	0.79
1.4	21.00	0:92	22.96	0.91	24.92	0.90	26.88	0.88	28.84	0.86	30.80	0.84
1.5	23.25	1.00	25.50	0.97	27.75	0.95	30.00	0.93	32.25	0.92	34.50	0.90
1.6	25.60	1.05	28.16	1.03	30.72	1.00	33.28	0.98	35.84	0.97	38.40	0.96
1.7	28.25	1.11	30.94	1.08	33.85	1.06	36.72	1.04	39.61	1.02	42.50	1.01
1.8	30.60	1.16	33.84	1.13	37.08	1.11	40.32	1.08	43.56	1.07	46.80	1.06
1.9	33.25	1.22	36.86	1.18	40.47	1.16	44.08	1.14	47.69	1.12	51.30	1.11
2.0	36.00	1.28	40.00	1.24	44.00	1.21	48.00	1.19	52.00	1.18	56.00	1.16
2.5	57.25	1.54	57.50	1.50	63.75	1.48	70.00	1.45	76.25	1.43	82.50	1.42
3.0	69.00	1.80	78.00	1.77	87.00	1.74	95.00	1.70	105.00	1.70	114.00	1.69

Table C-8. Hydraulic radius (R) and area (A) of symmetrical trapezoidal channels (10-foot bottom width)



$A = xd^2 + 10d$

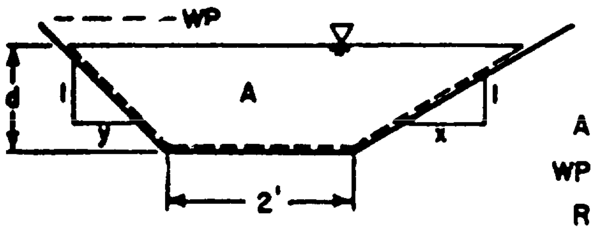
$WP = 2d\sqrt{1 + x^2} + 10$

$R = A/WP$

Depth, d (feet)	Slope ratio											
	1:1		1½:1		2:1		2½:1		3:1		4:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	5.25	0.46	5.38	0.46	5.50	0.45	5.63	0.44	5.75	0.44	6.00	0.42
0.6	6.36	0.54	6.54	0.54	6.72	0.53	6.90	0.52	7.08	0.51	7.44	0.50
0.7	7.49	0.63	7.74	0.62	7.98	0.61	8.23	0.60	8.47	0.59	8.96	0.57
0.8	8.64	0.71	8.96	0.70	9.28	0.68	9.60	0.67	9.92	0.66	10.56	0.64
0.9	9.81	0.78	10.22	0.77	10.62	0.76	11.03	0.74	11.43	0.73	12.24	0.70
1.0	11.00	0.86	11.50	0.85	12.00	0.83	12.50	0.81	13.00	0.80	14.00	0.77
1.1	12.21	0.93	12.82	0.92	13.42	0.90	14.03	0.88	14.63	0.86	15.84	0.83
1.2	13.44	1.00	14.16	0.99	14.88	0.97	15.60	0.95	16.32	0.93	17.76	0.89
1.3	14.69	1.07	15.54	1.06	16.38	1.04	17.23	1.01	18.07	0.99	19.76	0.95
1.4	15.96	1.14	16.94	1.13	17.92	1.10	18.90	1.08	19.88	1.05	21.84	1.01
1.5	17.25	1.21	18.38	1.19	19.50	1.17	20.63	1.14	21.75	1.12	24.00	1.07
1.6	18.56	1.28	19.84	1.26	21.12	1.23	22.40	1.20	23.68	1.18	26.24	1.13
1.7	19.89	1.34	21.34	1.32	22.78	1.29	24.23	1.26	25.67	1.24	28.56	1.19
1.8	21.24	1.41	22.86	1.39	24.48	1.36	26.10	1.33	27.72	1.30	30.96	1.25
1.9	22.61	1.47	24.42	1.45	26.22	1.42	28.03	1.39	29.83	1.35	33.44	1.30
2.0	24.00	1.53	26.00	1.51	28.00	1.48	30.00	1.44	32.00	1.41	36.00	1.36
2.5	31.25	1.83	34.38	1.81	37.50	1.77	40.63	1.73	43.75	1.69	50.00	1.63
3.0	39.00	2.11	43.50	2.09	48.00	2.05	52.50	2.01	57.00	1.97	66.00	1.90

	5:1		6:1		7:1		8:1		9:1		10:1	
	A	R	A	R	A	R	A	R	A	R	A	R
0.5	6.25	0.41	6.50	0.40	6.75	0.40	7.00	0.39	7.25	0.38	7.50	0.37
0.6	7.80	0.48	8.16	0.47	8.52	0.46	8.88	0.45	9.24	0.44	9.60	0.44
0.7	9.45	0.55	9.94	0.54	10.43	0.52	10.92	0.51	11.41	0.50	11.90	0.49
0.8	11.20	0.62	11.84	0.60	12.48	0.59	13.12	0.57	13.76	0.56	14.40	0.55
0.9	13.05	0.68	13.86	0.66	14.67	0.65	15.48	0.63	16.29	0.62	17.10	0.61
1.0	15.00	0.74	16.00	0.72	17.00	0.70	18.00	0.69	19.00	0.68	20.00	0.66
1.1	17.05	0.80	18.26	0.78	19.47	0.76	20.68	0.75	21.89	0.73	23.10	0.72
1.2	19.20	0.86	20.64	0.84	22.08	0.82	23.52	0.80	24.96	0.79	26.40	0.77
1.3	21.45	0.92	23.14	0.90	24.83	0.87	26.52	0.86	28.21	0.84	29.90	0.83
1.4	23.80	0.98	25.76	0.95	27.72	0.93	29.68	0.91	31.64	0.89	33.60	0.88
1.5	26.25	1.04	28.50	1.01	30.75	0.99	33.00	0.97	35.25	0.95	37.50	0.93
1.6	28.80	1.10	31.36	1.06	33.92	1.04	36.48	1.02	39.04	1.00	41.60	0.99
1.7	31.45	1.15	34.34	1.12	37.23	1.09	40.12	1.07	43.01	1.05	45.90	1.04
1.8	34.20	1.21	37.44	1.17	40.68	1.15	43.92	1.13	47.16	1.11	50.40	1.09
1.9	37.05	1.26	40.66	1.23	44.27	1.20	47.88	1.18	51.49	1.16	55.10	1.14
2.0	40.00	1.32	44.00	1.28	48.00	1.25	52.00	1.23	56.00	1.21	60.00	1.20
2.5	56.25	1.58	62.50	1.55	68.75	1.52	75.00	1.49	81.25	1.47	87.50	1.45
3.0	75.00	1.85	84.00	1.81	93.00	1.77	102.00	1.75	111.00	1.73	120.00	1.71

Table C-9. Hydraulic radius (R) and area (A) of nonsymmetrical trapezoidal channels (2-foot bottom width)



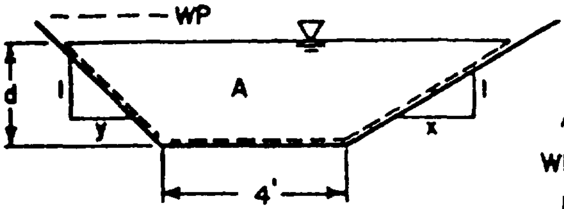
$$A = \frac{1}{2} d^2 (x + y) + 2d$$

$$WP = d\left(\sqrt{1+y^2} + \sqrt{1+x^2}\right) + 2$$

$$R = A/WP$$

Depth, d (feet)	Slope ratio											
	1:1 - 3:1		1½:1 - 3:1		2:1 - 3:1		2½:1 - 3:1		4:1 - 3:1		5:1 - 3:1	
	A	R	A	R	A	R	A	R	A	R	A	R
1.0	4.00	0.61	4.25	0.61	4.50	0.61	4.75	0.61	5.50	0.59	6.00	0.58
1.1	4.62	0.66	4.92	0.66	5.23	0.66	5.53	0.66	6.44	0.64	7.04	0.63
1.2	5.28	0.70	5.64	0.71	6.00	0.71	6.36	0.70	7.44	0.68	8.16	0.68
1.3	5.98	0.75	6.40	0.76	6.83	0.76	7.25	0.75	8.52	0.73	9.36	0.74
1.4	6.72	0.80	7.21	0.80	7.70	0.81	8.19	0.81	9.66	0.79	10.64	0.79
1.5	7.50	0.85	8.06	0.85	8.63	0.85	9.19	0.85	10.88	0.84	12.00	0.84
1.6	8.32	0.89	8.96	0.91	9.60	0.91	10.24	0.90	12.16	0.90	13.44	0.88
1.7	9.18	0.94	9.90	0.95	10.63	0.95	11.35	0.95	13.52	0.94	14.96	0.93
1.8	10.08	0.99	10.89	1.00	11.90	1.01	12.51	1.00	14.94	0.98	16.56	0.98
1.9	11.02	1.03	11.92	1.04	12.83	1.05	13.73	1.05	16.44	1.03	18.24	1.03
2.0	12.00	1.07	13.00	1.09	14.00	1.10	15.00	1.10	18.00	1.09	20.00	1.08
2.2	14.08	1.17	15.29	1.19	16.50	1.19	17.71	1.19	21.34	1.19	23.76	1.18
2.4	16.32	1.26	17.76	1.28	19.20	1.28	20.64	1.29	24.96	1.28	27.84	1.27
2.6	18.72	1.35	20.41	1.37	22.10	1.37	23.79	1.38	28.86	1.38	32.24	1.37
2.8	21.28	1.43	23.24	1.46	25.20	1.48	27.16	1.48	33.04	1.48	36.76	1.48
3.0	24.00	1.52	26.25	1.54	28.50	1.57	30.75	1.57	37.50	1.57	42.00	1.57
3.5	31.50	1.76	34.57	1.78	37.63	1.80	40.70	1.81	49.88	1.81	56.01	1.81
4.0	40.00	1.97	44.00	2.00	48.00	2.02	52.00	2.03	64.00	2.04	72.00	2.04

Table C-10. Hydraulic radius (R) and area (A) of nonsymmetrical trapezoidal channels (4-foot bottom width)



$$A = \frac{1}{2} d^2 (x + y) + 2d$$

$$WP = d\left(\sqrt{1+y^2} + \sqrt{1+x^2}\right) + 4$$

$$R = A/WP$$

Depth, d (feet)	Slope ratio											
	1:1 - 3:1		1½:1 - 3:1		2:1 - 3:1		2½:1 - 3:1		4:1 - 3:1		5:1 - 3:1	
	A	R	A	R	A	R	A	R	A	R	A	R
1.0	6.00	0.70	6.25	0.69	6.50	0.69	6.75	0.68	7.50	0.66	8.00	0.65
1.1	6.82	0.75	7.12	0.75	7.43	0.75	7.73	0.74	8.64	0.72	9.24	0.70
1.2	7.68	0.80	8.04	0.80	8.40	0.81	8.76	0.79	9.84	0.78	10.56	0.76
1.3	8.58	0.86	9.00	0.86	9.43	0.85	9.85	0.85	11.12	0.81	11.96	0.81
1.4	9.52	0.91	10.01	0.91	10.59	0.92	10.99	0.90	12.46	0.88	13.44	0.87
1.5	10.50	0.97	11.06	0.97	11.63	0.96	12.19	0.95	13.88	0.93	15.00	0.92
1.6	11.52	1.02	12.16	1.02	12.80	1.01	13.44	1.00	15.36	0.98	16.64	0.96
1.7	12.58	1.06	13.30	1.07	14.03	1.07	14.75	1.06	16.92	1.04	18.36	1.01
1.8	13.38	1.10	14.49	1.12	15.50	1.13	16.11	1.11	18.54	1.08	20.16	1.07
1.9	14.82	1.17	15.72	1.17	16.63	1.17	17.53	1.12	20.24	1.13	22.04	1.12
2.0	16.00	1.21	17.00	1.22	18.00	1.22	19.00	1.21	22.00	1.18	24.00	1.17
2.2	18.48	1.31	19.69	1.32	20.90	1.32	22.11	1.31	25.74	1.29	28.16	1.28
2.4	21.12	1.41	22.56	1.42	24.00	1.41	25.44	1.41	29.76	1.38	32.64	1.37
2.6	23.92	1.51	25.61	1.51	27.30	1.51	28.99	1.51	34.06	1.49	37.44	1.47
2.8	26.88	1.60	28.84	1.61	30.80	1.62	32.76	1.61	38.64	1.59	42.36	1.57
3.0	30.00	1.69	32.25	1.71	34.50	1.71	36.75	1.71	43.50	1.68	48.00	1.66
3.5	38.50	1.93	41.57	1.94	44.63	1.95	47.70	1.95	56.88	1.93	63.07	1.92
4.0	48.00	2.15	52.00	2.17	56.00	2.18	60.00	2.18	72.00	2.16	80.00	2.15

APPENDIX D

CONE INDEX REQUIREMENTS

Fine-Grained Soils

Tracked Vehicles

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}
Amphibious vehicles			
Carrier, cargo, amphibious, tracked, M116	10.9	7	18
Landing vehicle, tracked, command, M5(LVTP5A1(CMD))	97.5	20	49
Landing vehicle, tracked, personnel, M5 (LVTP5A1)	87.8	19	45
Armored bulldozers			
Bulldozer, earthmoving tank, tank-mtd, M9 (tank, combat, 105-mm gun, M60, and M60A1)	116.0	23	53
Tractor, armored, combat earthmover (ACE), M9	35.6	18	41

NOTES:

1. Items listed include selected self-propelled vehicles as of January 1993. Certain items in final development or undergoing field testing have been included where the type of classification is pending.

2. The VCIs have been calculated from the formulas, curves, and tables in NRMM. The vehicle weight was based on normal design loads or combat weights, equipment, cross-country tire pressures, and crews as the conditions would be under full operational deployment in typical off-road movements. Trucks which could operate at lower tire pressures would generate slightly lower VCI values with increased tire deflection.

3. Amphibious vehicles and engineer tractors have grousers greater than 1 1/2 inches; all other tracked vehicles have grousers less than 1 1/2 inches.

4. One kip equals 1,000 pounds (US customary).

Tracked Vehicles (continued)

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}
Combat vehicles			
Armored reconnaissance airborne assault vehicle (General Sheridan), M551	35.8	15	35
Howitzer, heavy, self-propelled, full- tracked, 8-in, M55 (T108)	98.0	20	47
Howitzer, heavy, self-propelled, 8-in, M110 (T236E1)	58.5	20	47
Howitzer, medium, self-propelled, 155-mm, M109 (T196E1)	53.2	25	57
Howitzer, light, self-propelled, full-tracked, 105-mm, M37	46.0	N/A	58
Howitzer, light, self-propelled, full-tracked, 105-mm,			
M52	53.0	N/A	46
M52A1	53.0	N/A	46
Howitzer, light, self-propelled, 105-mm, M108	46.9	N/A	54
LAV-25, 8x8, light, armored vehicle	27.7	32	72
Mortar, infantry, self-propelled, full-tracked, 107-mm (4.2-in), M84	47.1	N/A	46
Tank, combat, full-tracked, 90-mm gun,			
M48	99.0	20	47
M48C	99.0	20	47
M48A1	104.0	21	49
M48A2	105.0	21	49
M48A2C	105.0	21	49
M48A3 (M48A1E2)	104.0	21	49
M48A5	106.0	22	50
Tank, combat, full-tracked, 105-mm gun,			
M60	110.0	21	48
M60A1	116.0	22	51
M60A3	110.0	20	46

Tracked Vehicles (continued)

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}
Combat vehicles continued			
Tank, combat, full-tracked, 120-mm gun,			
M1	115.0	23	54
M1A1	125.0	25	58
M 1A2	140.0	28	64
Vehicle, combat engineer, full-tracked, 165-mm gun, M729 (basic M60A1 tank)	115.0	N/A	54
Armored vehicle bridges			
Launcher, M48 tank chassis, transporting	96.0	N/A	49
Launcher, M48 tank chassis, transporting, with bridge, armored vehicle launched, scissoring type, class 60, 60-ft	128.0	N/A	65
Launcher, M60A1 chassis, transporting	86.3	15	36
Launcher, M60A1 chassis, transporting with bridge, armored vehicle launched, scissoring type, class 60, 60-ft	115.9	22	51
Carriers			
Carrier, cargo, tracked, 6-on, M548	28.0	N/A	37
Carrier, command post, light, tracked,			
M577	23.9	17	40
M577A1	24.4	17	40
Carrier, personnel, full-tracked, armored,			
M113	22.6	N/A	48
M113A1	23.4	N/A	49
M113A2	23.4	N/A	49
M113A3	23.6	N/A	49

Tracked Vehicles (continued)

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}
Carriers (continued)			
Infantry fighting vehicle,			
M2A1	50.2	15	35
M2A2	66.0	16	37
Multiple Launch Rocket System	54,2	15	35
Recovery vehicles			
Recovery vehicle, full-tracked, medium, M88	112.0	21	50
Recovery vehicle, full-tracked, light, armored, M578	54.0	21	49
Wheeled Vehicles			
Trucks			
Truck, utility, 1/4-ton, 4x4, M151	3.1	19	44
Truck, utility, 1 1/4-ton, 4x4 M998 (HMMWV)	7.5	20	47
Truck, cargo, 1 1/4-ton, 6x6, M561	9.6	19	44
Truck, cargo, 1 3/4-ton, 4x4, M 1028 commercial utility cargo vehicle (CUCV)	9.3	31	70
Truck, cargo, 2 1/2-ton, 6x6, M34	17.2	27	61
Truck, cargo, 2 1/2-ton, 6x6, M35A 1	19.2	26	59
Truck, cargo, 5-ton, 6x6 M923	32.5	30	68
Truck, cargo, 8-ton, 4x4 M520	43.4	43	97
Truck, cargo, 10-ton, 6x6			
M125	49.5	37	84
M125A1	49.5	37	84
Truck, cargo, 10-ton, 8x8 M977	60.4	36	79
Truck, cargo, 5-ton, 6x6 M 1084	35.8	25	57

Wheeled Vehicles (continued)

Vehicle Description	Vehicle Weight (kips)	VCI_1	VCI_{50}
Trucks (continued)			
Truck, cargo, 2 1/2-ton, 4x4, M1078	21.8	25	57
Truck, dump, 2 1/2-ton, 6x6, M47	19.2	28	64
Truck, dump, 5-ton, 6x6,			
M51	32.7	32	72
M51A2	32.7	32	72
M929	32.7	30	68
Truck, tractor, 5-ton, 6x6			
M52 (w/o payload)	17.8	21	48
M52A1 (w/o payload)	17.8	21	48
Truck, tractor, 10-ton, 6x6			
M123 (w/o payload)	28.9	21	48
M123C (w/o payload)	30.2	22	50
M123D (w/o payload)	30.2	22	50
Truck, van, expansible, 2 1/2-ton, 6x6, M292	25.1	N/A	76
Truck, van, shop, 2 1/2-ton, 6x6, M109A1	21.0	N/A	65
Truck, van, shop, 2 1/2-ton, 6x6, M220	20.4	N/A	62
Truck, wrecker, crane, 2 1/2-ton, 6x6, M108	19.8	N/A	63
Truck, tractor, wrecker, medium, 5-ton, 6x6, M246 (w/ payload)	44.8	32	73
Truck, wrecker, medium, 5-ton, 6x6, M543	34.4	N/A	76
Palletized Loading System, 10 x 10	86.6	34	77

APPENDIX E

SOIL-TRAFFICABILITY TEST SET

Trafficability measurements are made with the soil-trafficability test set. This set consists of one canvas carrying case, one cone penetrometer with 3/8-inch steel and 5/8-inch aluminum shafts and a 0.5-square-inch cone, one soil sampler, remolding equipment (which includes a 3/8-inch steel shaft and a 0.2-square-inch cone, a 5/8-inch steel shaft with foot and handle, a 2 1/2-pound hammer, and a cylinder and base with pin), and a bag of hand tools (which includes one combination spanner wrench and 1 /4-inch screwdriver; two open-end wrenches, 1/2 by 9/16; one 6-inch Stillson wrench; one 3/16-inch Allen wrench; and one 2-inch screwdriver with a 1/8-inch bit). The items are shown in their proper places in the carrying case in Figure E-1, SC-6635-98-CL-E02-HR gives component listings and stock numbers.

CONE PENETROMETER

The cone penetrometer is shown in Figure E-2, page E-2. It consists of a 30-degree cone with a 1/2-inch-square base area, a steel shaft 19 inches long and 3/8 inch in diameter, a proving ring, a micrometer dial, and a handle.

Use of the Cone Penetrometer

Inspect and adjust the cone penetrometer prior to use. Using an operator's assistant

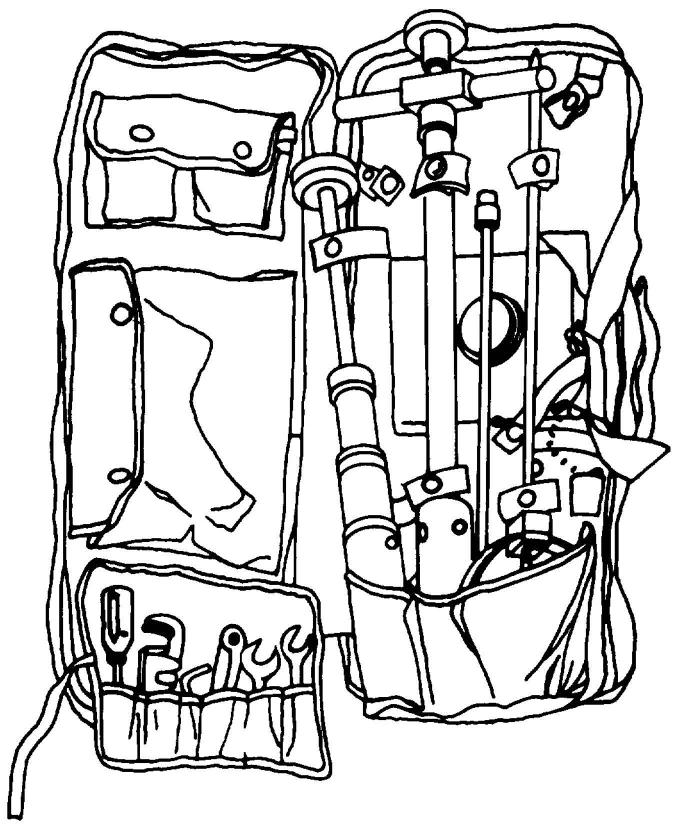


Figure E-1. Soil Trafficability test set

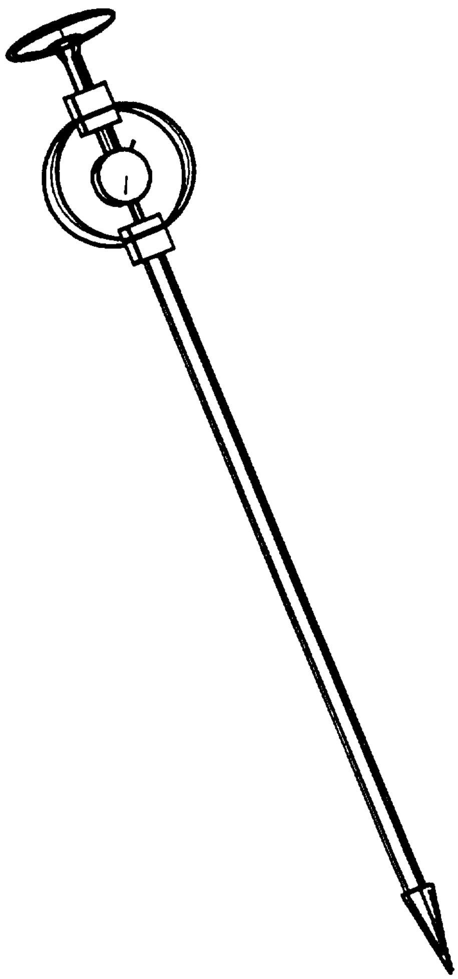


Figure E-2. Cone penetrometer

Operation. Operate the penetrometer as follows:

1. Place one hand over the other on the handle, palms down, and approximately at right angles as shown in Figure E-3. This minimizes eccentric loading of the proving ring and helps keep the shaft vertical.

2. Apply force until slow, steady downward movement occurs.

3. Take a dial reading just as the base of the cone becomes flush with the ground surface. To do this, watch the cone descend until an instant before the cone base is expected to be flush with the ground surface, then immediately shift the vision to the dial face. Continue the slow, steady, downward movement and take successive dial readings which measurements can be made and recorded and usually diminishes the likelihood of errors.

Inspection. Inspect the penetrometer before using it to make sure that all nuts, bolts, and joints are tight and that the dial-gage stem contacts the proving-ring bearing block.

Zeroing. Allow the penetrometer to hang vertically from its handle, and rotate the dial face until 0 is under the needle. When the instrument is kept vertical between the fingertips and allowed to rest on its cone, the dial will register about 2 to 4 pounds—the total weight of the instrument—or 4 to 8 on the dial.

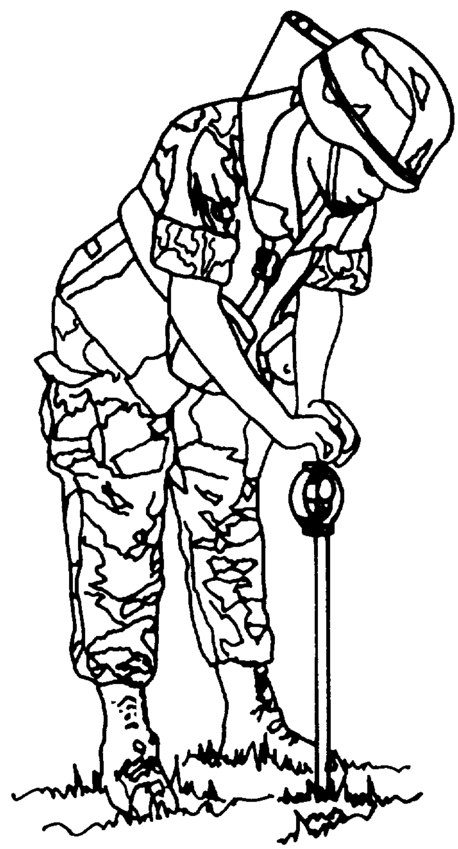


Figure E-3. Using a cone penetrometer in the upright position at appropriate 6-inch intervals to a depth of 18 inches, If it is necessary to stop the downward progression of the cone penetrometer for any reason, progress may be resumed with no adverse effects on the cone penetrometer readings. Progression should be stopped between depths so that the next reading is taken only after downward progression has resumed. For example, when only one person is on the trafficability reconnaissance, it may be convenient to make two cone penetrometer readings, stop the penetration to record the readings, resume the penetration to obtain two additional readings, then stop to record.

Precautions. Observe the following precautions when operating a penetrometer:

- Keep the instrument vertical.
- Do not attempt to take readings higher than the capacity of the dial (300). This may overstress the proving ring.
- If dial capacity is exceeded at less than 18 inches of penetration, make another penetration nearby. The cone may be striking an isolated rock fragment or other object.
- Withdraw the instrument by the shaft, never by the ring or the handle. Pulling the handle may stretch the proving ring.
- Read the CI only at the proper depth. If readings are made as little as 1/4 inch from the proper depth and recorded as being at the proper depth, an average of such readings will not accurately reflect the average strength. Carelessness in making proper depth determinations is probably the greatest source of error in using the penetrometer.

Training Penetrometer Operators

Train operators in areas that have uniform soil conditions. The instructor should take approximately 50 sets of readings equally spaced over the area. The average CI for 6-inch layers should be computed and used as standards or references. The trainee should be instructed in all proper techniques of operation. He should practice penetration, observed by a qualified instructor, until he becomes familiar with the techniques of operation. The trainee should then make 50 sets of readings, using an assistant to record them. The average CIs obtained by a trainee should be compared to the standard. If the trainee's readings deviate widely, the causes for the deviations should be sought and corrected.

In a uniform area, a 5-percent deviation is wide. The most probable cause of error is carelessness in determining the proper depth. The rate of progression recommended is such that four readings (surface, 6, 12, and 18 inches) can be measured in 15 seconds during a continuous penetration in soft soil. Slower or faster penetration rates will reflect lower or higher values, respectively, but the discrepancies will not be large. The CI is also insignificantly affected by the variation in the rate of penetration for the same operator or between experienced operators. However, if deviations persist, check the possibility of cone-penetrometer mechanical imperfections, Inspect the dial face to ensure that its position has not shifted around the dial's shaft and that the needle is not sticking or has not slipped on its shaft. Any of these conditions could cause an improper zero setting. Secondly, inspect the proving ring. A damaged or overstressed ring might require recalibration. Finally, check to ensure the Instrument was properly zeroed. The micrometer dial stem may not have been in good contact with the proving-ring bearing block when the instrument was zeroed.

Care and Adjustment of the Penetrometer

Keep the penetrometer free from dirt and rust and keep all parts tight. Frequently check the instrument and rezero, if necessary. Ensure no grit is caught between the stem of the dial and the lower mounting block.

Dial Care. The micrometer dial is a sensitive instrument that should be protected against water and rough use. Never immerse it in water, and wipe it dry as soon as possible after use in rainy weather. When the dial is transported by truck, wrap it in paper or cloth.

Bearing-Block Adjustment. If either or both bearing blocks become loosened and moved, adjust them so that they lie on the same diameter of the ring. Retighten them and recalibrate the proving ring. Do not calibrate while on reconnaissance. Instead, note all readings made in the field after bearing blocks have been removed and correct them according to the calibration made later.

Cone Replacement. Considerable use of the same cone may result in a rounding of its point, but it will not affect the accuracy of the instrument, However, if the base of the cone has had excessive wear or is deformed by hard use, replace the cone.

Proving-Ring Recalibration. The calibration will remain true for the life of the instrument unless the bearing blocks are moved or the ring is overstressed (deformed by a hard knock or subjected to extreme changes in temperature or other unusual strains). If the ring needs recalibration, complete the following steps:

1. Remove the handle and shaft,

2. Place the lower mounting block of the ring assembly on a smooth, horizontal surface.

3. Check the bearing-block alignment and tightness. Both blocks should be on the same diameter of the ring. Use a drafting triangle or a carpenter's square for this operation. The bolts should be snug.

4. Ensure that the stem of the dial bears firmly on the lower bearing block. Ensure that the dial arm has sufficient travel available for the full range of motion (approximately 1/10-inch deflection) of the proving ring. The dial can be moved up or down by adjusting the two nuts on the threaded stud that holds the gage in position. Both nuts should be tight when in final position.

5. Zero the dial by rotating its face so that 0 is under the needle.

6. Add the load in 10-pound increments up to 150 pounds. Mark or note the needle's position on the dial after the addition of each load increment. Any of the following loading methods may be used:

- Add deadweights to the top of the ring assembly. If a plate is used to hold the weights, its weight should be considered in the first 10-pound load.
- Use any of the load machines commonly used in laboratory work to apply the load.
- Place the ring assembly on a set of platform scales. Apply the load increments with a jack and measure them with the platform scales.

7. Remove the load in 10-pound increments, noting the position of the needle after the removal of each increment.

8. Make the load run at least twice, using the average of the needle position for each increment as the final point.

9. Expect some variation in needle position; it will not be significant.

10. Establish 10-pound intervals on the dial face and mark them 20, 30, 40, so on, to 300. Each interval should be subdivided separately because the arcs for various 10-pound intervals are not necessarily the same.

NOTE: If the instrument cannot be calibrated or the proving ring is severely damaged, the instrument will need to be turned in for repair.

SOIL SAMPLER

A piston-type soil sampler, as shown in Figure E-4, is used to extract soil samples for remolding tests.

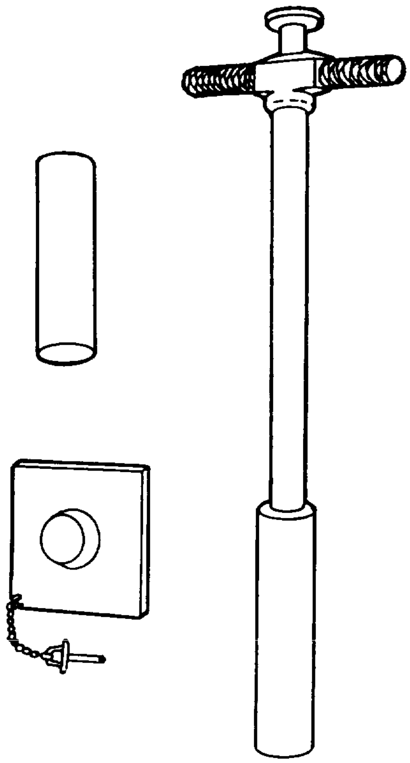


Figure E-4. Soil sampler and sampling cylinder

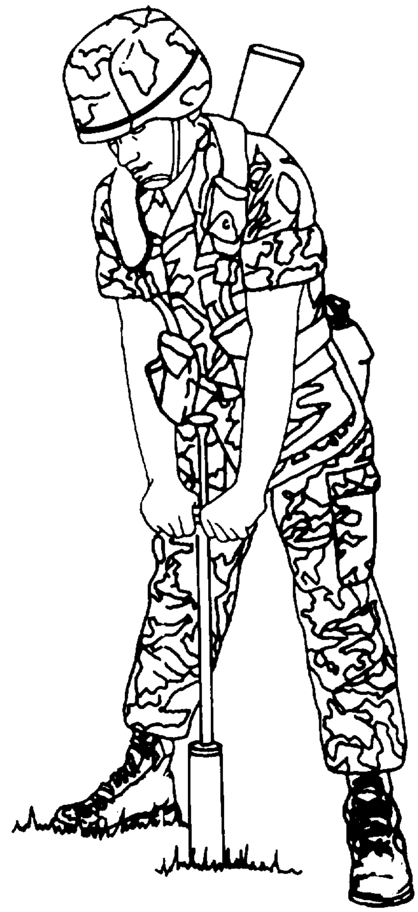


Figure E-5. Operating the sampler in the upright position

Operation

The soil sampler is used in the following way:

1. Loosen the knurled handle of the soil sampler so the piston rod will move freely. Hold the sampler firmly in both hands and force it into the soil vertically (Figure E-5). Do not twist the sampler while pushing it into the soil. Sometimes two people are needed to force the sampler into firm soils.

2. After locking the piston rod by turning the knurled handle, twist the sampler slightly and withdraw.

3. Deposit the sample directly into the remolding cylinder.

Figure E-6, page E-6, shows the technique for using the sampler in a prone position.

Care

It is essential to keep the inside of the sampling tube, the piston ring, and the leather washer clean. After 5 to 25 samplings, depending upon the type of soil, complete the following cleaning procedures:

1. Immerse the tube first in water and then in fuel oil. Work the piston up and down five or six times in each liquid.

2. Wipe off the excess fuel oil, and squirt light machine oil into the tube.

3. If the sampler becomes stiff and hard to work, remove the tube, disassemble and thoroughly clean the piston, and oil the leather washer. Tube walls and cutting edges are soft and should be handled with

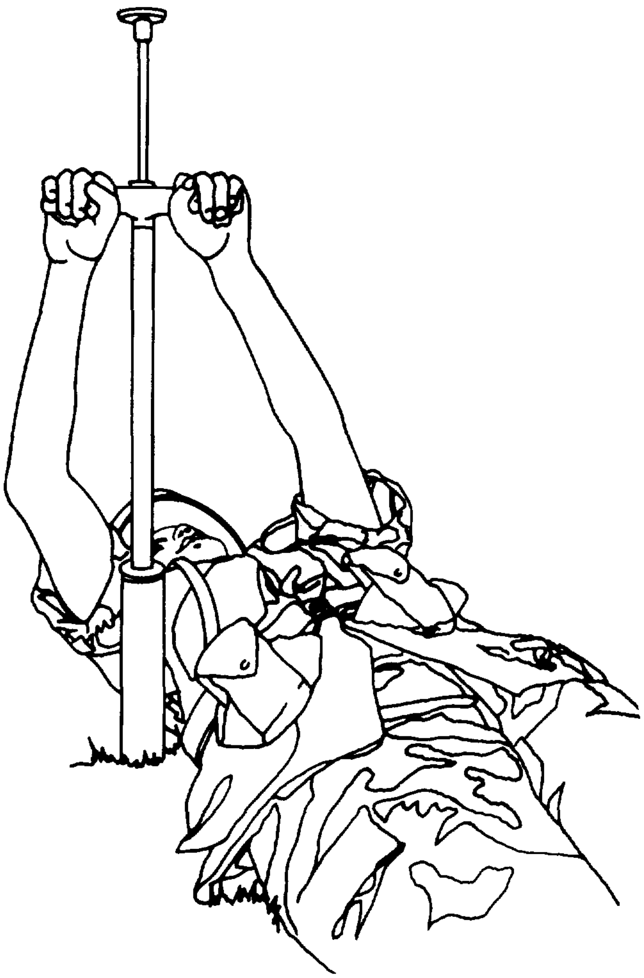


Figure E-6. Using the sampler in a prone position

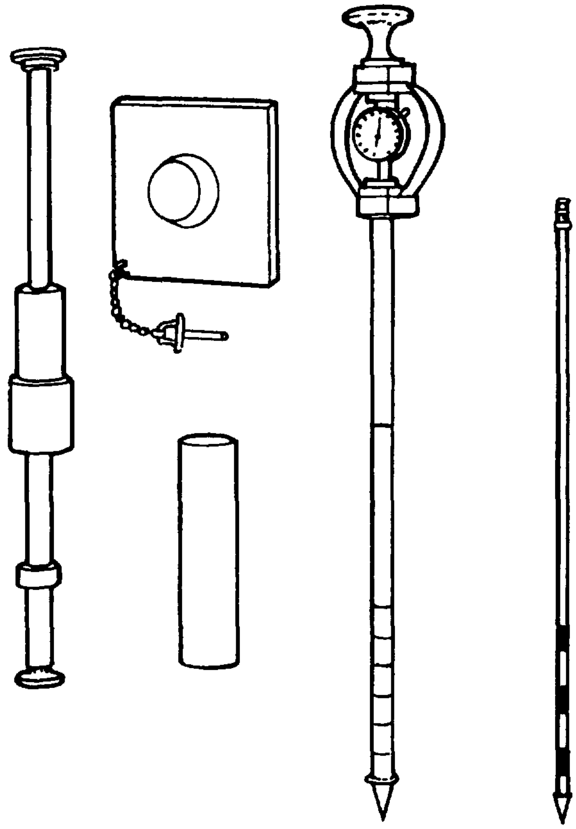


Figure E-7. Remolding test equipment

care. The cutting edges will require sharpening from time to time.

Adjustment

Adjust the piston-rod length to keep the face of the piston flush with the cutting edge of the tube when the piston-rod handle (disk) is fully depressed. To do this, loosen the setscrew on the handle, screw the handle up or down to the correct position, and retighten the setscrew.

Remolding test

The equipment for the remolding test, shown in detail in Figure E-7 and in use in Figures E-8 through E-11, pages E-7 through E-9, consists of the following:

Ž A steel cylinder approximately 2 inches in diameter and 8 inches long, mounted on an aluminum base.

• A 2 1/2-pound steel drop hammer sliding on an 18-inch steel shaft with a handle.

Ž A cone penetrometer

A cone penetrometer may be equipped with an aluminum shaft (5/8-inch in diameter) or a steel shaft (3/8-inch in diameter) with a 0.5-square-inch cone (for fine-grained soils) or a more slender steel shaft with a 0.2-square-inch cone (for remoldable sands). The penetrometer is used to measure soil strength in the cylinder before and after remolding. The sampler (Figure

Figure E-8. Operating the remolding test equipment to obtain a soil sample

E-4, page E-5) is used to obtain the soil sample from the critical layer and place it in the remolding cylinder.

Test Procedure for Fine-Grained Soils

The following remolding test is performed for fine-grained soils:

1. Take a sample from the critical layer with the sampler as shown in Figure E-8, eject it directly into the remolding cylinder as shown in Figure E-9, page E-8, and push it to the bottom of the cylinder with the foot of the drop-hammer shaft.

2. Measure the strength with the penetrometer (steel shaft) by taking CI readings as the base of the cone enters the surface of the soil sample and at each successive inch, to a depth of 4 inches, as shown in Figure E-10, page E-9,

3. Apply 100 blows with the drop hammer falling 12 inches as shown in Figure E-11, page E-9.

4. Measure the remolded strength from the surface to the 4-inch depth at 1 -inch increments, as was done before remolding as shown in Figure E-10.

NOTE: Some samples are so hard they cannot be penetrated the full 4 inches. In such cases, the full dial capacity (300) is recorded for each inch below the last reading obtained.

To find the remolding index, take the sum of the five CI readings after remolding and divide by the sum of the five readings before remolding.

Test Procedure for Remoldable Sands

The test procedure for remoldable sands is generally the same as that for fine-grained soils. However, the CI measurements are made with the slender shaft and 0.2-square-inch cone, and the sample is remolded by placing a rubber stopper in the top of the remolding tube and dropping it (along with the cylinder and base) 25 times from a height of 6 inches onto a firm surface, such as a piece of timber. Some remoldable sands with a large amount of fines (more than 12 but less than 50 percent) react very much like fine-grained soil, When testing a remoldable sand with a large amount of fines, run both the fine grains and remoldable sands tests, and use the lower remolding index. Continue to use the more critical test throughout the area.

Figure E-9. Operating test equipment to load the cylinder

Figure E-10. Measuring cone index in a remolding cylinder

Figure E-11. Applying hammer blows with remolding equipment

APPENDIX F

CURVE TABLES

FUNCTIONS OF A1-DEGREE CURVE

The long chords (C), middle ordinates (M), externals (E), and tangent distances (T) in Table F-1, pages F-2 through F-30, are for a 1-degree curve, based on the arc definition (5,729.578-foot radius). To find the corresponding functions of any other curve, divide the tabular values by the degree of curvature.

CORRECTIONS FOR TANGENTS AND DISTANCES

Complete the following steps to determine the degree of curvature for all curves other than the 1-degree curve:

1. Determine the value corresponding to the given intersection angle from Table F-1.

2. Divide this value by the given degree of curvature.

3. Add the correction derived from Table F-2, pages F-30 and F-31.

Table F-1. Functions of a 1-degree curve

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
0	0	0.000	0.0000	0.0000	0.0000
	10	16.666	0.0060	0.0060	8.3333
	20	33.333	0.0242	0.0242	16.6667
	30	49.999	0.0545	0.0545	25.0001
	40	66.666	0.0969	0:0969	33.3337
	50	83.332	0.1515	0.1515	41.6674
1	0	99.998	0.2181	0.2181	50.0012
	10	116.664	0.2969	0.2969	58.3353
	20	133.330	0.3878	0.3878	66.6696
	30	149.995	0.4908	0.4908	75.0042
	40	166.660	0.6060	0.6060	83.3392
	50	183.325	0.7332	0.7333	91.6744
2	0	199.989	0.8726	0.8727	100.0101
	10	216.653	1.0241	1.0243	108.3462
	20	233.317	1.1877	1.1879	116.6827
	30	249.980	1.3634	1.3638	125.0198
	40	266.642	1.5513	1.5517	133.3574
	50	283.304	1.7513	1.7518	141.6955
3	0	299.965	1.9633	1.9640	150.0342
	10	316.626	2.1875	2.1884	158.3736
	20	333.286	2.4238	2.4249	166.7136
	30	349.945	2.6723	2.6735	175.0544
	40	366.604	2.9328	2.9343	183.3959
	50	383.261	3.2055	3.2073	191.7381
4	0	399.918	3.4903	3.4924	200.0812
	10	416.574	3.7871	3.7896	208.4251
	20	433.230	4.0961	4.0991	216.7700
	30	449.884	4.4172	4.4207	225.1157
	40	466.537	4.7505	4.7544	233.4624
	50	483.190	5.0958	5.1003	241.8100

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
5	0	499.841	5.4532	5.4584	250.1587
	10	516.491	5.8228	5.8287	258.5085
	20	533.140	6.2044	6.2112	266.8593
	30	549.788	6.5982	6.6058	275.2113
	40	566.435	7.0041	7.0127	283.5645
	50	583.081	7.4221	7.4317	291.9188
6	0	599.725	7.8521	7.8629	300.2744
	10	616.369	8.2943	8.3063	308.6313
	20	633.010	8.7486	8.7620	316.9894
	30	649.651	9.2150	9.2298	325.3490
	40	666.290	9.6935	9.7099	333.7099
	50	682.928	10.1841	10.2022	342.0722
7	0	699.564	10.6868	10.7067	350.4360
	10	716.199	11.2016	11.2235	358.8012
	20	732.832	11.7284	11.7525	367.1680
	30	749.464	12.2674	12.2937	375.5363
	40	766.094	12.8185	12.8472	383.9063
	50	782.723	13.3817	13.4130	392.2778
8	0	799.350	13.9569	13.9910	400.6511
	10	815.975	14.5443	14.5813	409.0260
	20	832.599	15.1437	15.1838	417.4027
	30	849.220	15.7552	15.7987	425.7811
	40	865.840	16.3788	16.4258	434.1614
	50	882.458	17.0145	17.0652	442.5435
9	0	899.075	17.6623	17.7169	450.9275
	10	915.689	18.3222	18.3810	459.3134
	20	932.301	18.9941	19.0573	467.7013
	30	948.912	19.6782	19.7460	476.0912
	40	965.520	20.3743	20.4470	484.4831
	50	982.126	21.0825	21.1603	492.8770

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
10	0	998.731	21.8027	21.8860	501.2731
	10	1,015.333	22.5351	22.6240	509.6713
	20	1,031.933	23.2795	23.3744	518.0716
	30	1,048.531	24.0359	24.1372	526.4742
	40	1,065.126	24.8045	24.9123	534.8790
	50	1,081.720	25.5851	25.6999	543.2861
11	0	1,098.311	26.3778	26.4998	551.6956
	10	1,114.900	27.1825	27.3121	560.1073
	20	1,131.486	27.9993	28.1368	568.5215
	30	1,148.070	28.8282	28.9740	576.9381
	40	1,164.652	29.6691	29.8236	585.3572
	50	1,181.231	30.5221	30.6856	593.7788
12	0	1,197.807	31.3872	31.5601	602.2029
	10	1,214.382	32.2643	32.4470	610.6295
	20	1,230.953	33.1534	33.3464	619.0588
	30	1,247.522	34.0546	34.2582	627.4908
	40	1,264.088	34.9679	35.1826	635.9254
	50	1,280.652	35.8932	36.1194	644.3628
13	0	1,297.213	36.8305	37.0688	652.8029
	10	1,313.771	37.7799	38.0307	661.2458
	20	1,330.326	38.7413	39.0050	669.6916
	30	1,346.879	39.7148	39.9920	678.1402
	40	1,363.429	40.7003	40.9914	686.5917
	50	1,379.975	41.6978	42.0035	695.0462
14	0	1,396.519	42.7074	43.0281	703.5037
	10	1,413.060	43.7289	44.0653	711.9642
	20	1,429.598	44.7626	45.1150	720.4277
	30	1,446.133	45.8082	46.1774	728.8943
	40	1,462.665	46.8659	47.2524	737.3641
	50	1,479.194	47.9356	48.3400	745.8371

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
15	0	1,495.719	49.0173	49.4403	754.3132
	10	1,512.242	50.1110	50.5532	762.7926
	20	1,528.761	51.2168	51.6787	771.2753
	30	1,545.277	52.3345	52.8169	779.7613
	40	1,561.790	53.4643	53.9679	788.2507
	50	1,578.300	54.6060	55.1315	796.7434
16	0	1,594.806	55.7598	56.3078	805.2396
	10	1,611.309	56.9256	57.4968	813.7393
	20	1,627.808	58.1034	58.6986	822.2425
	30	1,644.304	59.2931	59.9132	830.7492
	40	1,660.796	60.4949	61.1404	839.2595
	50	1,677.285	61.7087	62.3805	847.7735
17	0	1,693.771	62.9344	63.6334	856.2911
	10	1,710.252	64.1722	64.8990	864.8124
	20	1,726.731	65.4219	66.1775	873.3375
	30	1,743.205	66.6836	67.4688	881.8663
	40	1,759.676	67.9573	68.7730	890.3990
	50	1,776.143	69.2429	70.0900	898.9355
18	0	1,792.606	70.5406	71.4199	907.4760
	10	1,809.066	71.8502	72.7626	916.0203
	20	1,825.522	73.1717	74.1183	924.5686
	30	1,841.974	74.5053	75.4869	933.1210
	40	1,858.422	75.8508	76.8684	941.6774
	50	1,874.866	77.2082	78.2629	950.2379
19	0	1,891.306	78.5777	79.6703	958.8025
	10	1,907.742	79.9590	81.0907	967.3713
	20	1,924.174	81.3524	82.5241	975.9443
	30	1,940.602	82.7576	83.9705	984.5215
	40	1,957.026	84.1749	85.4299	993.1030
	50	1,973.445	85.6040	86.9024	1,001.6889

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
20	0	1,989.861	87.0451	88.3879	1,010.2791
	10	2,006.272	88.4981	89.8865	1,018.8738
	20	2,022.680	89.9631	91.3982	1,027.4729
	30	2,039.082	91.4400	92.9230	1,036.0764
	40	2,055.481	92.9288	94.4609	1,044.6845
	50	2,071.875	94.4296	96.0120	1,053.2972
21	0	2,088.265	95.9423	97.5762	1,061.9145
	10	2,104.650	97.4669	99.1536	1,070.5364
	20	2,121.031	99.0034	100.7441	1,079.1630
	30	2,137.408	100.5518	102.3479	1,087.7943
	40	2,153.779	102.1121	103.9649	1,096.4304
	50	2,170.147	103.6843	105.5952	1,105.0713
22	0	2,186.510	105.2684	107.2387	1,113.7171
	10	2,202.868	106.8645	108.8955	1,122.3677
	20	2,219.221	108.4724	110.5656	1,131.0233
	30	2,235.570	110.0922	112.2490	1,139.6839
	40	2,251.914	111.7239	113.9458	1,148.3494
	50	2,268.253	113.3675	115.6559	1,157.0201
23	0	2,284.588	115.0229	117.3793	1,165.6958
	10	2,300.917	116.6903	119.1162	1,174.3766
	20	2,317.242	118.3695	120.8665	1,183.0626
	30	2,333.562	120.0605	122.6302	1,191.7539
	40	2,349.877	121.7635	124.4074	1,200.4504
	50	2,366.187	123.4783	126.1980	1,209.1522
24	0	2,382.492	125.2050	128.0021	1,217.8593
	10	2,398.792	126.9435	129.8198	1,226.5719
	20	2,415.087	128.6939	131.6509	1,235.2898
	30	2,431.377	130.4561	133.4957	1,244.0133
	40	2,447.661	132.2302	135.3539	1,252.7422
	50	2,463.941	134.0161	137.2258	1,261.4767

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
25	0	2,480.215	135.8138	139.1113	1,270.2168
	10	2,496.484	137.6234	141.0105	1,278.9625
	20	2,512.747	139.4448	142.9232	1,287.7140
	30	2,529.006	141.2780	144.8497	1,296.4711
	40	2,545.259	143.1231	146.7899	1,305.2340
	50	2,561.507	144.9800	148.7438	1,314.0027
26	0	2,577.749	146.8487	150.7114	1,322.7773
	10	2,593.985	148.7292	152.6928	1,331.5577
	20	2,610.217	150.6215	154.6880	1,340.3441
	30	2,626.443	152.5256	156.6970	1,349.1365
	40	2,642.663	154.4415	158.7198	1,357.9349
	50	2,658.877	156.3692	160.7565	1,366.7393
27	0	2,675.086	158.3086	162.8070	1,375.5499
	10	2,691.290	160.2599	164.8715	1,384.3667
	20	2,707.487	162.2230	166.9499	1,393.1896
	30	2,723.679	164.1978	169.0422	1,402.0188
	40	2,739.865	166.1844	171.1485	1,410.8542
	50	2,756.046	168.1828	173.2688	1,419.6960
28	0	2,772.220	170.1929	175.4031	1,428.5442
	10	2,788.389	172.2148	177.5515	1,437.3988
	20	2,804.552	174.2484	179.7139	1,446.2598
	30	2,820.708	176.2939	181.8905	1,455.1274
	40	2,836.859	178.3510	184.0811	1,464.0015
	50	2,853.004	180.4199	186.2859	1,472.8822
29	0	2,869.143	182.5005	188.5049	1,481.7696
	10	2,885.276	184.5929	190.7380	1,490.6636
	20	2,901.402	186.6970	192.9854	1,499.5644
	30	2,917.523	188.8128	195.2470	1,508.4719
	40	2,933.637	190.9404	197.5229	1,517.3863
	50	2,949.745	193.0796	199.8131	1,526.3076

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
30	0	2,965.847	195.2306	202.1176	1,535.2357
	10	2,981.943	197.3933	204.4364	1,544.1709
	20	2,998.032	199.5676	206.7697	1,553.1130
	30	3,014.115	201.7537	209.1173	1,562.0622
	40	3,030.192	203.9515	211.4794	1,571.0185
	50	3,046.262	206.1610	213.8559	1,579.9819
31	0	3,062.326	208.3821	216.2469	1,588.9526
	10	3,078.383	210.6149	218.6524	1,597.9304
	20	3,094.434	212.8594	221.0725	1,606.9156
	30	3,110.478	215.1156	223.5071	1,615.9081
	40	3,126.516	217.3834	225.9564	1,624.9079
	50	3,142.547	219.6629	228.4202	1,633.9152
32	0	3,158.571	221.9541	230.8987	1,642.9300
	10	3,174.589	224.2569	233.3919	1,651.9523
	20	3,190.600	226.5713	235.8998	1,660.9822
	30	3,206.604	228.8974	238.4224	1,670.0196
	40	3,222.601	231.2352	240.9598	1,679.0648
	50	3,238.592	233.5845	243.5121	1,688.1177
33	0	3,254.576	235.9455	246.0791	1,697.1783
	10	3,270.553	238.3181	248.6610	1,706.2467
	20	3,286.522	240.7023	251.2578	1,715.3230
	30	3,302.485	243.0981	253.8695	1,724.4072
	40	3,318.442	245.5056	256.4961	1,733.4994
	50	3,334.391	247.9246	259.1378	1,742.5995
34	0	3,350.332	250.3553	261.7944	1,751.7077
	10	3,366.267	252.7975	264.4661	1,760.8241
	20	3,382.195	255.2513	267.1529	1,769.9485
	30	3,398.116	257.7167	269.8548	1,779.0812
	40	3,414.029	260.1937	272.5718	1,788.2221
	50	3,429.935	262.6822	275.3040	1,797.3714

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1. Long Chord	M_1. Middle Ordinate	E_1. External Distance	T_1. Tangent Distance
35	0	3,445.834	265.1823	278.0514	1,806.5290
	10	3,461.726	267.6940	280.8140	1,815.6949
	20	3,477.610	270.2172	283.5919	1,824.8694
	30	3,493.487	272.7519	286.3851	1,834.0523
	40	3,509.357	275.2982	289.1936	1,843.2438
	50	3,525.219	277.8561	292.0175	1,852.4439
36	0	3,541.073	280.4255	294.8568	1,861.6527
	10	3,556.921	283.0064	297.7115	1,870.8702
	20	3,572.760	285.5988	300.5817	1,880.0964
	30	3,588.592	288.2027	303.4674	1,889.3314
	40	3,604.417	290.8182	306.3686	1,898.5753
	50	3,620.234	293.4451	309.2855	1,907.8281
37	0	3,636.043	296.0836	312.2179	1,917.0899
	10	3,651.845	298.7335	315.1659	1,926.3607
	20	3,667.638	301.3950	318.1297	1,935.6406
	30	3,683.424	304.0679	321.1091	1,944.9296
	40	3,699.203	306.7523	324.1043	1,954.2278
	50	3,714.973	309.4482	327.1153	1,963.5352
38	0	3,730.736	312.1555	330.1421	1,972.8519
	10	3,746.490	314.8743	333.1848	1,982.1779
	20	3,762.237	317.6046	336.2434	1,991.5133
	30	3,777.976	320.3463	339.3179	2,000.8582
	40	3,793.707	323.0994	342.4083	2,010.2125
	50	3,809.430	325.8640	345.5148	2,019.5764
39	0	3,825.144	328.6400	348.6373	2,028.9499
	10	3,840.851	331.4274	351.7759	2,038.3331
	20	3,856.550	334.2263	354.9306	2,047.7260
	30	3,872.240	337.0366	358.1015	2,057.1287
	40	3,887.922	339.8582	361.2886	2,066.5411
	50	3,903.596	342.6913	364.4919	2,075.9635

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1. Long Chord	M_1. Middle Ordinate	E_1. External Distance	T_1. Tangent Distance
40	0	3,919.262	345.5358	367.7115	2,085.3958
	10	3,934.919	348.3916	370.9474	2,094.8381
	20	3,950.568	351.2589	374.1997	2,104.2904
	30	3,966.209	354.1375	377.4683	2,113.7529
	40	3,981.841	357.0275	380.7534	2,123.2255
	50	3,997.465	359.9289	384.0550	2,132.7083
41	0	4,013.081	362.8416	387.3731	2,142.2014
	10	4,028.687	365.7657	390.7077	2,151.7048
	20	4,044.286	368.7011	394.0590	2,161.2186
	30	4,059.876	371.6478	397.4268	2,170.7429
	40	4,075.457	374.6059	400.8114	2,180.2776
	50	4,091.030	377.5753	404.2127	2,189.8229
42	0	4,106.594	380.5561	407.6307	2,199.3789
	10	4,122.149	383.5481	411.0656	2,208.9455
	20	4,137.696	386.5515	414.5173	2,218.5228
	30	4,153.234	389.5662	417.9859	2,228.1110
	40	4,168.763	392.5921	421.4715	2,237.7100
	50	4,184.283	395.6294	424.9740	2,247.3199
43	0	4,199.794	398.6779	428.4935	2,256.9407
	10	4,215.297	401.7377	432.0302	2,266.5726
	20	4,230.790	404.8088	435.5839	2,276.2156
	30	4,246.275	407.8912	439.1548	2,285.8698
	40	4,261.751	410.9848	442.7429	2,295.5351
	50	4,277.217	414.0896	446.3482	2,305.2118
44	0	4,292.675	417.2057	449.9709	2,314.8997
	10	4,308.123	420.3331	453.6109	2,324.5991
	20	4,323.563	423.4716	457.2682	2,334.3099
	30	4,338.993	426.6214	460.9430	2,344.0322
	40	4,354.414	429.7825	464.6353	2,553.7661
	50	4,369.826	432.9547	468.3451	2,363.5116

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_{1° Long Chord	M_{1° Middle Ordinate	E_{1° External Distance	T_{1° Tangent Distance
45	0	4,385.229	436.1381	472.0725	2,373.2689
	10	4,400.622	439.3327	475.8175	2,383.0379
	20	4,416.006	442.5385	479.5802	2,392.8187
	30	4,431.381	445.7556	483.3605	2,402.6114
	40	4,446.746	448.9837	487.1587	2,412.4161
	50	4,462.102	452.2231	490.9746	2,422.2328
46	0	4,477.448	455.4736	494.8085	2,432.0615
	10	4,492.785	458.7353	498.6602	2,441.9024
	20	4,508.113	462.0081	502.5299	2,451.7555
	30	4,523.431	465.2920	506.4176	2,461.6209
	40	4,538.739	468.5871	510.3234	2,471.4986
	50	4,554.038	471.8934	514.2473	2,481.3888
47	0	4,569.327	475.2107	518.1893	2,491.2914
	10	4,584.607	478.5392	522.1496	2,501.2065
	20	4,599.877	481.8786	526.1281	2,511.1342
	30	4,615.137	485.2294	530.1249	2,521.0746
	40	4,630.387	488.5912	534.1401	2,531.0277
	50	4,645.627	491.9641	538.1738	2,540.9936
48	0	4,660.858	495.3480	542.2259	2,550.9724
	10	4,676.079	498.7430	546.2965	2,560.9641
	20	4,691.290	502.1491	550.3857	2,570.9689
	30	4,706.491	505.5662	554.4936	2,580.9866
	40	4,721.682	508.9944	558.6201	2,591.0176
	50	4,736.863	512.4336	562.7654	2,601.0617
49	0	4,752.034	515.8839	566.9295	2,611.1191
	10	4,767.195	519.3452	571.1124	2,621.1898
	20	4,782.346	522.8175	575.3143	2,631.2740
	30	4,797.487	526.3008	579.5351	2,641.3716
	40	4,812.617	529.7951	583.7749	2,651.4828
	50	4,827.738	533.3005	588.0338	2,661.6076

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1. Long Chord	M_1. Middle Ordinate	E_1. External Distance	T_1. Tangent Distance
50	0	4,842.848	536.8168	592.3118	2,671.7460
	10	4,857.948	540.3441	596.6090	2,681.8983
	20	4,873.038	543.8824	600.9255	2,692.0644
	30	4,888.117	547.4316	605.2612	2,702.2444
	40	4,903.186	550.9919	609.6164	2,712.4383
	50	4,918.245	554.5630	613.9909	2,722.6463
51	0	4,933.293	558.1452	618.3850	2,732.8685
	10	4,948.331	561.7382	622.7985	2,743.1048
	20	4,963.359	565.3422	627.2317	2,753.3554
	30	4,978.375	568.9571	631.6845	2,763.6203
	40	4,993.382	572.5830	636.1571	2,773.8996
	50	5,008.378	576.2197	640.6494	2,784.1935
52	0	5,023.363	579.8674	645.1616	2,794.5019
	10	5,038.337	583.5259	649.6936	2,804.8249
	20	5,053.301	587.1953	654.2456	2,815.1626
	30	5,068.255	590.8756	658.8177	2,825.5151
	40	5,083.197	594.5668	663.4098	2,835.8825
	50	5,098.129	598.2688	668.0221	2,846.2648
53	0	5,113.050	601.9817	672.6546	2,856.6622
	10	5,127.960	605.7055	677.3074	2,867.0746
	20	5,142.859	609.4400	681.9805	2,877.5022
	30	5,157.748	613.1854	686.6740	2,887.9450
	40	5,172.625	616.9417	691.3880	2,898.4032
	50	5,187.492	620.7087	696.1225	2,908.8767
54	0	5,202.347	624.4866	700.8777	2,919.3658
	10	5,217.192	628.2752	705.6535	2,929.8703
	20	5,232.026	632.0747	710.4500	2,940.3906
	30	5,246.848	635.8849	715.2673	2,950.9265
	40	5,261.660	639.7059	720.1055	2,961.4783
	50	5,276.460	643.5377	724.9646	2,972.0459

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
55	0	5,291.249	647.3802	729.8448	2,982.6295
	10	5,306.027	651.2335	734.7459	2,993.2291
	20	5,320.793	655.0975	739.6683	3,003.8448
	30	5,335.549	658.9723	744.6118	3,014.4768
	40	5,350.293	662.8577	749.5766	3,025.1250
	50	5,365.026	666.7539	754.5628	3,035.7896
56	0	5,379.747	670.6608	759.5704	3,046.4706
	10	5,394.457	674.5785	764.5995	3,057.1682
	20	5,409.156	678.5068	769.6501	3,067.8824
	30	5,423.843	682.4458	774.7224	3,078.6133
	40	5,438.519	686.3954	779.8163	3,089.3609
	50	5,453.183	690.3558	784.9321	3,100.1255
57	0	5,467.836	694.3268	790.0697	3,110.9070
	10	5,482.477	698.3084	795.2292	3,121.7055
	20	5,497.107	702.3007	800.4107	3,132.5212
	30	5,511.725	706.3036	805.6143	3,143.3541
	40	5,526.331	710.3172	810.8401	3,154.2043
	50	5,540.926	714.3413	816.0880	3,165.0718
58	0	5,555.509	718.3761	821.3583	3,175.9569
	10	5,570.080	722.4215	826.6509	3,186.8595
	20	5,584.639	726.4775	831.9660	3,197.7798
	30	5,599.187	730.5440	837.3036	3,208.7178
	40	5,613.722	734.6212	842.6638	3,219.6737
	50	5,628.246	738.7088	848.0467	3,230.6474
59	0	5,642.758	742.8071	853.4523	3,241.6392
	10	5,657.258	746.9159	858.8808	3,252.6491
	20	5,671.746	751.0352	864.3323	3,263.6772
	30	5,686.222	755.1651	869.8067	3,274.7236
	40	5,700.686	759.3055	875.3042	3,285.7883
	50	5,715.138	763.4564	880.8248	3,296.8716

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1. Long Chord	M_1. Middle Ordinate	E_1. External Distance	T_1. Tangent Distance
60	0	5,729.578	767.6178	886.3688	3,307.9734
	10	5,744.005	771.7898	891.9360	3,319.0938
	20	5,758.421	775.9722	897.5266	3,330.2330
	30	5,772.824	780.1650	903.1408	3,341.3911
	40	5,787.215	784.3684	908.7785	3,352.5681
	50	5,801.594	788.5822	914.4398	3,363.7641
61	0	5,815.961	792.8065	920.1249	3,374.9793
	10	5,830.315	797.0412	925.8339	3,386.2137
	20	5,844.657	801.2863	931.5667	3,397.4675
	30	5,858.987	805.5419	937.3236	3,408.7407
	40	5,873.304	809.8079	943.1045	3,420.0334
	50	5,887.609	814.0843	948.9096	3,431.3457
62	0	5,901.901	818.3710	954.7390	3,442.6777
	10	5,916.181	822.6682	960.5927	3,454.0296
	20	5,930.448	826.9758	966.4709	3,465.4014
	30	5,944.703	831.2937	972.3736	3,476.7932
	40	5,958.945	835.6220	978.3009	3,488.2052
	50	5,973.175	839.9606	984.2529	3,499.6374
63	0	5,987.392	844.3096	990.2297	3,511.0899
	10	6,001.596	848.6689	996.2314	3,522.5628
	20	6,015.788	853.0386	1,002.2581	3,534.0563
	30	6,029.967	857.4185	1,008.3099	3,545.5704
	40	6,044.133	861.8088	1,014.3868	3,557.1053
	50	6,058.286	866.2094	1,020.4890	3,568.6610
64	0	6,072.427	870.6202	1,026.6166	3,580.2376
	10	6,086.555	875.0414	1,032.7696	3,581.8354
	20	6,100.670	879.4728	1,038.9482	3,603.4543
	30	6,114.771	883.9144	1,045.1524	3,615.0944
	40	6,128.860	888.3663	1,051.3823	3,626.7560
	50	6,142.936	892.8285	1,057.6381	3,638.4391

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1. Long Chord	M_1. Middle Ordinate	E_1. External Distance	T_1. Tangent Distance
65	0	6,157.000	897.3009	1,063.9198	3,650.1437
	10	6,171.050	901.7835	1,070.2276	3,661.8701
	20	6,185.086	906.2763	1,076.5615	3,673.6182
	30	6,199.110	910.7793	1,082.9216	3,685.3884
	40	6,213.121	915.2925	1,089.3081	3,697.1805
	50	6,227.119	919.8159	1,095.7210	3,708.9948
66	0	6,241.103	924.3495	1,102.1604	3,720.8314
	10	6,255.074	928.8933	1,108.6265	3,732.6904
	20	6,269.032	933.4471	1,115.1194	3,744.5718
	30	6,282.977	938.0112	1,121.6390	3,756.4759
	40	6,296.909	942.5854	1,128.1857	3,768.4027
	50	6,310.827	947.1697	1,134.7594	3,780.3523
67	0	6,324.732	951.7641	1,141.3602	3,792.3249
	10	6,338.623	956.3686	1,147.9883	3,804.3206
	20	6,352.501	960.9832	1,154.6438	3,816.3394
	30	6,366.365	965.6080	1,161.3268	3,828.3816
	40	6,380.217	970.2427	1,168.0374	3,840.4472
	50	6,394.054	974.8876	1,174.7757	3,852.5363
68	0	6,407.878	979.5425	1,181.5418	3,864.6491
	10	6,421.689	984.2075	1,188.3358	3,876.7857
	20	6,435.486	988.8825	1,195.1578	3,888.9462
	30	6,449.269	993.5675	1,202.0080	3,901.1308
	40	6,463.039	998.2626	1,208.8865	3,913.3395
	50	6,476.795	1,002.9676	1,215.7933	3,925.5724
69	0	6,490.537	1,007.6827	1,222.7286	3,937.8298
	10	6,504.265	1,012.4077	1,229.6925	3,950.1117
	20	6,517.980	1,017.1427	1,236.6852	3,962.4183
	30	6,531.681	1,021.8877	1,243.7066	3,974.7496
	40	6,545.369	1,026.6427	1,250.7570	3,987.1059
	50	6,559.042	1,031.4076	1,257.8365	3,999.4872

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
70	0	6,572.701	1,036.1824	1,264.9452	4,011.8937
	10	6,586.347	1,040.9672	1,272.0831	4,024.3254
	20	6,599.979	1,045.7619	1,279.2505	4,036.7826
	30	6,613.596	1,050.5665	1,286.4475	4,049.2653
	40	6,627.200	1,055.3810	1,293.6741	4,061.7738
	50	6,640.790	1,060.2053	1,300.9305	4,074.3080
71	0	6,654.365	1,065.0396	1,308.2168	4,086.8682
	10	6,667.927	1,069.8837	1,315.5331	4,099.4545
	20	6,681.474	1,074.7377	1,322.8797	4,112.0670
	30	6,695.008	1,079.6015	1,330.2565	4,124.7059
	40	6,708.527	1,084.4752	1,337.6637	4,137.3713
	50	6,722.032	1,089.3587	1,345.1014	4,150.0633
72	0	6,735.522	1,094.2520	1,352.5698	4,162.7820
	10	6,748.999	1,099.1551	1,360.0691	4,175.5277
	20	6,762.461	1,104.0680	1,367.5992	4,188.3004
	30	6,775.909	1,108.9907	1,375.1604	4,201.1004
	40	6,789.343	1,113.9232	1,382.7528	4,213.9276
	50	6,802.762	1,118.8654	1,390.3765	4,226.7823
73	0	6,816.167	1,123.8174	1,398.0317	4,239.6646
	10	6,829.557	1,128.7791	1,405.7185	4,252.5747
	20	6,842.933	1,133.7506	1,413.4370	4,265.5127
	30	6,856.294	1,138.7317	1,421.1873	4,278.4788
	40	6,869.641	1,143.7226	1,428.9696	4,291.4730
	50	6,882.974	1,148.7232	1,436.7841	4,304.4956
74	0	6,896.292	1,153.7335	1,444.6309	4,317.5467
	10	6,906.595	1,158.7535	1,452.5100	4,330.6264
	20	6,922.884	1,163.7831	1,460.4217	4,343.7349
	30	6,936.158	1,168.8224	1,468.3662	4,356.8723
	40	6,949.417	1,173.8713	1,476.3434	4,370.0388
	50	6,962.662	1,178.9299	1,484.3536	4,383.2346

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
75	0	6,975.892	1,183.9981	1,492.3970	4,396.4598
	10	6,989.107	1,189.0759	1,500.4736	4,409.7145
	20	7,002.307	1,194.1633	1,508.5836	4,422.9989
	30	7,015.493	1,199.2604	1,516.7272	4,436.3132
	40	7,028.664	1,204.3670	1,524.9045	4,449.6576
	50	7,041.819	1,209.4831	1,533.1156	4,463.0321
76	0	7,054.960	1,214.6089	1,541.3608	4,476.4369
	10	7,068.086	1,219.7442	1,549.6401	4,489.8722
	20	7,081.198	1,224.8890	1,557.9537	4,503.3382
	30	7,094.294	1,230.0433	1,566.3018	4,516.8350
	40	7,107.375	1,235.2072	1,574.6845	4,530.3628
	50	7,120.441	1,240.3806	1,583.1020	4,543.9218
77	0	7,133.492	1,245.5635	1,591.5544	4,557.5121
	10	7,146.528	1,250.7558	1,600.0419	4,571.1338
	20	7,159.549	1,255.9577	1,608.5647	4,584.7872
	30	7,172.554	1,261.1690	1,617.1228	4,598.4725
	40	7,185.545	1,266.3897	1,625.7165	4,612.1897
	50	7,198.520	1,271.6199	1,634.3459	4,625.9390
78	0	7,211.480	1,276.8595	1,643.0113	4,639.7207
	10	7,224.425	1,282.1086	1,651.7126	4,653.5349
	20	7,237.354	1,287.3670	1,660.4502	4,667.3818
	30	7,250.269	1,292.6349	1,669.2242	4,681.2615
	40	7,263.167	1,297.9121	1,678.0347	4,695.1743
	50	7,276.051	1,303.1987	1,686.8819	4,709.1202
79	0	7,288.919	1,308.4947	1,695.7660	4,723.0996
	10	7,301.772	1,313.8000	1,704.6072	4,737.1125
	20	7,314.609	1,319.1147	1,713.6455	4,751.1591
	30	7,327.431	1,324.4387	1,722.6413	4,765.2397
	40	7,340.237	1,329.7720	1,731.6747	4,779.3543
	50	7,353.028	1,335.1146	1,740.7458	4,793.5033

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1. Long Chord	M_1. Middle Ordinate	E_1. External Distance	T_1. Tangent Distance
80	0	7,365.803	1,340.4666	1,749.8548	4,807.6867
	10	7,378.563	1,345.8278	1,759.0020	4,821.9048
	20	7,391.307	1,351.1982	1,768.1874	4,836.1578
	30	7,404.035	1,356.5780	1,777.4113	4,850.4458
	40	7,416.748	1,361.9670	1,786.6738	4,864.7690
	50	7,429.445	1,367.3652	1,795.9751	4,879.1276
81	0	7,442.126	1,372.7727	1,805.3155	4,893.5219
	10	7,454.792	1,378.1893	1,814.6950	4,907.9519
	20	7,467.441	1,383.6152	1,824.1140	4,922.4180
	30	7,480.075	1,389.0503	1,833.5725	4,936.9202
	40	7,492.694	1,394.4945	1,843.0707	4,951.4589
	50	7,505.296	1,399.9480	1,852.6089	4,966.0341
82	0	7,517.882	1,402.4105	1,862.1872	4,980.6461
	10	7,530.453	1,410.8823	1,871.8059	4,995.2951
	20	7,543.007	1,416.3631	1,881.4651	5,009.9814
	30	7,555.546	1,421.8531	1,891.1650	5,024.7050
	40	7,568.069	1,427.3522	1,900.9058	5,039.4662
	50	7,580.575	1,432.8604	1,910.6878	5,054.2653
83	0	7,593.066	1,438.3777	1,920.5111	5,069.1024
	10	7,605.541	1,443.9041	1,930.3758	5,083.9777
	20	7,617.999	1,449.4395	1,940.2823	5,098.8914
	30	7,630.441	1,454.9840	1,950.2308	5,113.8438
	40	7,642.868	1,460.5376	1,960.2213	5,128.8351
	50	7,655.278	1,466.1001	1,970.2542	5,143.8655
84	0	7,667.672	1,471.6717	1,980.3296	5,158.9352
	10	7,680.049	1,477.2523	1,990.4478	5,174.0443
	20	7,692.441	1,481.8419	2,000.6090	5,189.1933
	30	7,704.756	1,488.4405	2,010.8133	5,204.3822
	40	7,717.084	1,494.0480	2,021.0610	5,219.6113
	50	7,729.397	1,499.6645	2,031.3524	5,234.8808

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
85	0	7,741.693	1,505.2899	2,041.6875	5,250.1909
	10	7,753.973	1,510.9243	2,052.0668	5,265.5419
	20	7,766.236	1,516.5676	2,062.4902	5,280.9340
	30	7,778.483	1,522.2199	2,072.9582	5,296.3674
	40	7,790.714	1,527.8810	2,083.4709	5,311.8423
	50	7,802.928	1,533.5510	2,094.0285	5,327.3591
86	0	7,815.125	1,539.2299	2,104.6313	5,342.9179
	10	7,827.306	1,544.9176	2,115.2795	5,358.5189
	20	7,839.470	1,550.6142	2,125.9733	5,374.1625
	30	7,851.618	1,556.3197	2,136.7129	5,389.8487
	40	7,863.749	1,562.0339	2,147.4987	5,405.5780
	50	7,875.864	1,567.7570	2,158.3308	5,421.3505
87	0	7,887.962	1,573.4889	2,169.2094	5,437.1665
	10	7,900.043	1,579.2296	2,180.1349	5,453.0261
	20	7,912.108	1,584.9791	2,191.1074	5,468.9298
	30	7,924.155	1,590.7373	2,202.1272	5,484.8777
	40	7,936.186	1,596.5043	2,213.1945	5,500.8700
	50	7,948.201	1,602.2800	2,224.3096	5,516.9071
88	0	7,960.198	1,608.0645	2,235.4727	5,532.9891
	10	7,972.179	1,613.8576	2,246.6841	5,549.1164
	20	7,984.142	1,619.6595	2,257.9440	5,565.2892
	30	7,996.089	1,625.4701	2,269.2527	5,581.5077
	40	8,008.019	1,631.2894	2,280.6104	5,597.7723
	50	8,019.932	1,637.1173	2,292.0174	5,614.0832
89	0	8,031.828	1,642.9539	2,303.4740	5,630.4406
	10	8,043.707	1,648.7991	2,314.9804	5,646.8448
	20	8,055.569	1,654.6530	2,326.5369	5,663.2961
	30	8,067.414	1,660.5154	2,338.1437	5,679.7949
	40	8,079.242	1,666.3865	2,349.8011	5,696.3412
	50	8,091.053	1,672.2662	2,361.5094	5,712.9355

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
90	0	8,102.846	1,678.1545	2,373.2689	5,729.5780
	10	8,114.623	1,684.0513	2,385.0797	5,746.2689
	20	8,126.382	1,689.9567	2,396.9423	5,763.0086
	30	8,138.125	1,695.8707	2,408.8568	5,779.7974
	40	8,149.849	1,701.7931	2,420.8236	5,796.6355
	50	8,161.557	1,707.7241	2,432.8430	5,813.5232
91	0	8,173.248	1,713.6636	2,444.9151	5,830.4609
	10	8,184.921	1,719.6116	2,457.0404	5,847.4487
	20	8,196.577	1,725.5681	2,469.2190	5,864.4871
	30	8,208.215	1,731.5331	2,481.4514	5,881.5763
	40	8,219.836	1,737.5065	2,493.7377	5,898.7166
	50	8,231.440	1,743.4883	2,506.0783	5,915.9083
92	0	8,243.026	1,749.4786	2,518.4734	5,933.1517
	10	8,254.595	1,755.4773	2,530.9235	5,950.4471
	20	8,266.147	1,761.4845	2,543.4286	5,967.7948
	30	8,277.681	1,767.5000	2,555.9893	5,985.1952
	40	8,289.197	1,773.5238	2,568.6057	6,002.6486
	50	8,300.696	1,779.5561	2,581.2783	6,020.1552
93	0	8,312.178	1,785.5967	2,594.0072	6,037.7154
	10	8,323.641	1,791.6457	2,606.7929	6,055.3295
	20	8,335.088	1,797.7030	2,619.6355	6,072.9978
	30	8,346.516	1,803.7686	2,632.5355	6,090.7207
	40	8,357.927	1,809.8425	2,645.4932	6,108.4985
	50	8,369.320	1,815.9247	2,658.5089	6,126.3316
94	0	8,380.696	1,822.0152	2,671.5829	6,144.2201
	10	8,392.053	1,828.1139	2,684.7156	6,162.1646
	20	8,403.393	1,834.2209	2,697.9072	6,180.1653
	30	8,414.716	1,840.3361	2,711.1581	6,198.2225
	40	8,426.020	1,846.4596	2,724.4687	6,216.3367
	50	8,437.307	1,852.5913	2,737.8393	6,234.5081

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
95	0	8,448.576	1,858.7312	2,751.2702	6,252.7371
	10	8,459.826	1,864.8792	2,764.7617	6,271.0241
	20	8,471.059	1,871.0355	2,778.3143	6,289.3694
	30	8,482.274	1,877.1999	2,791.9283	6,307.7734
	40	8,493.472	1,883.3724	2,805.6040	6,326.2364
	50	8,504.651	1,889.5531	2,819.3417	6,344.7588
96	0	8,515.812	1,895.7419	2,833.1419	6,363.3410
	10	8,526.955	1,901.9389	2,847.0049	6,381.9833
	20	8,538.080	1,908.1439	2,860.9311	6,400.6861
	30	8,549.187	1,914.3570	2,874.9208	6,419.4497
	40	8,560.276	1,920.5782	2,888.9743	6,438.2747
	50	8,571.347	1,926.8074	2,903.0922	6,457.1612
97	0	8,582.400	1,933.0447	2,917.2747	6,476.1098
	10	8,593.435	1,939.2900	2,931.5223	6,495.1208
	20	8,604.451	1,945.5433	2,945.8352	6,514.1945
	30	8,615.449	1,951.8047	2,960.2140	6,433.3315
	40	8,626.429	1,958.0740	2,974.6589	6,552.5320
	50	8,637.391	1,964.3513	2,989.1704	6,571.7966
98	0	8,648.334	1,970.6366	3,003.7489	6,591.1255
	10	8,659.259	1,976.9298	3,018.3948	6,610.5192
	20	8,670.166	1,983.2309	3,033.1084	6,629.9780
	30	8,681.055	1,989.5400	3,047.8902	6,649.5025
	40	8,691.925	1,995.8570	3,062.7406	6,669.0931
	50	8,702.777	2,002.1819	3,077.6600	6,688.7500
99	0	8,713.610	2,008.5147	3,092.6488	6,708.4739
	10	8,724.425	2,014.8554	3,107.7074	6,728.2650
	20	8,735.221	2,021.2039	3,122.8363	6,748.1238
	30	8,745.999	2,027.5602	3,138.0359	6,768.0508
	40	8,756.759	2,033.9244	3,153.3066	6,788.0464
	50	8,767.500	2,040.2964	3,168.6488	6,808.1110

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
100	0	8,778.222	2,046.6762	3,184.0630	6,828.2451
	10	8,788.926	2,053.0638	3,199.5496	6,848.4491
	20	8,799.611	2,059.4592	3,215.1090	6,868.7235
	30	8,810.278	2,065.8623	3,230.7418	6,889.0687
	40	8,820.926	2,072.2732	3,246.4484	6,909.4852
	50	8,831.555	2,078.6918	3,262.2291	6,929.9734
101	0	8,842.166	2,085.1182	3,278.0846	6,950.5339
	10	8,852.758	2,091.5522	3,294.0151	6,971.1670
	20	8,863.331	2,097.9940	3,310.0213	6,991.8733
	30	8,873.886	2,104.4434	3,326.1035	7,012.6532
	40	8,884.421	2,110.9005	3,342.2623	7,033.5072
	50	8,894.938	2,117.3653	3,358.4982	7,054.4358
102	0	8,905.436	2,123.8377	3,374.8115	7,075.4395
	10	8,915.916	2,130.3177	3,391.2029	7,096.5188
	20	8,926.376	2,136.8054	3,407.6727	7,117.6742
	30	8,936.817	2,143.3006	3,424.2215	7,138.9062
	40	8,947.240	2,149.8034	3,440.8498	7,160.2152
	50	8,957.644	2,156.3138	3,457.5581	7,181.6019
103	0	8,968.028	2,162.8318	3,474.3469	7,203.0667
	10	8,978.394	2,169.3573	3,491.2167	7,224.6101
	20	8,988.741	2,175.8903	3,508.1681	7,246.2327
	30	8,999.069	2,182.4309	3,525.2015	7,267.9350
	40	9,009.377	2,188.9789	3,542.3174	7,289.7175
	50	9,019.667	2,195.5345	3,559.5165	7,311.5808
104	0	9,029.938	2,202.0975	3,576.7993	7,333.5254
	10	9,040.189	2,208.6680	3,594.1662	7,355.5518
	20	9,050.421	2,215.2459	3,611.6179	7,377.6607
	30	9,060.635	2,221.8313	3,629.1548	7,399.8525
	40	9,070.829	2,228.4241	3,646.7777	7,422.1278
	50	9,081.004	2,235.0243	3,664.4869	7,444.4873

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
105	0	9,091.159	2,241.6319	3,682.2830	7,466.9314
	10	9,101.296	2,248.2468	3,700.1667	7,489.4607
	20	9,111.413	2,254.8692	3,718.1386	7,512.0759
	30	9,121.511	2,261.4988	3,736.1991	7,534.7775
	40	9,131.589	2,268.1358	3,754.3489	7,557.5661
	50	9,141.648	2,274.7802	3,772.5886	7,580.4423
106	0	9,151.688	2,281.4318	3,790.9187	7,603.4068
	10	9,161.709	2,288.0908	3,809.3400	7,626.4600
	20	9,171.710	2,294.7570	3,827.8529	7,649.6027
	30	9,181.692	2,301.4305	3,846.4581	7,672.8354
	40	9,191.654	2,308.1112	3,865.1562	7,696.1588
	50	9,201.597	2,314.7992	3,883.9479	7,719.5736
107	0	9,211.521	2,321.4944	3,902.8337	7,743.0802
	10	9,221.425	2,328.1968	3,921.8143	7,766.6795
	20	9,231.309	2,334.9064	3,940.8904	7,790.3720
	30	9,241.174	2,341.6232	3,960.0626	7,814.1584
	40	9,251.019	2,348.3472	3,979.3315	7,838.0393
	50	9,260.845	2,355.0783	3,998.6979	7,862.0155
108	0	9,270.651	2,361.8165	4,018.1623	7,886.0875
	10	9,280.438	2,368.5619	4,037.7254	7,910.2561
	20	9,290.205	2,375.3143	4,057.3880	7,934.5220
	30	9,299.952	2,382.0739	4,077.1507	7,958.8859
	40	9,309.680	2,388.8406	4,097.0142	7,983.3483
	50	9,319.388	2,395.6143	4,116.9792	8,007.9102
109	0	9,329.076	2,402.3951	4,137.0464	8,032.5721
	10	9,338.745	2,409.1829	4,157.2166	8,057.3347
	20	9,348.393	2,415.9777	4,177.4903	8,082.1989
	30	9,358.022	2,422.7796	4,197.8685	8,107.1653
	40	9,367.632	2,429.5884	4,218.3517	8,132.2347
	50	9,377.221	2,436.4042	4,238.9407	8,157.4078

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
110	0	9,386.791	2,443.2270	4,259.6364	8,182.6854
	10	9,396.340	2,450.0568	4,280.4393	8,208.0681
	20	9,405.870	2,456.8934	4,301.3504	8,233.5569
	30	9,415.380	2,463.7370	4,322.3703	8,259.1525
	40	9,424.870	2,470.5875	4,343.4998	8,284.8556
	50	9,434.340	2,477.4450	4,364.7398	8,310.6671
111	0	9,443.790	2,484.3092	4,386.0909	8,336.5877
	10	9,453.220	2,491.1804	4,407.5541	8,362.6182
	20	9,462.630	2,498.0584	4,429.1301	8,388.7595
	30	9,472.020	2,504.9432	4,450.8197	8,415.0123
	40	9,481.390	2,511.8349	4,472.6237	8,441.3776
	50	9,490.740	2,518.7333	4,494.5430	8,467.8561
112	0	9,500.070	2,525.6386	4,516.5784	8,494.4487
	10	9,509.380	2,532.5506	4,538.7308	8,521.1562
	20	9,518.670	2,539.4694	4,561.0010	8,547.9795
	30	9,527.939	2,546.3950	4,583.3899	8,574.9194
	40	9,537.189	2,553.3272	4,605.8983	8,601.9769
	50	9,546.418	2,560.2662	4,628.5271	8,629.1528
113	0	9,555.627	2,567.2119	4,651.2773	8,656.4480
	10	9,564.816	2,574.1643	4,674.1497	8,683.8635
	20	9,573.985	2,581.1234	4,697.1451	8,711.4001
	30	9,583.133	2,588.0891	4,720.2646	8,739.0587
	40	9,592.261	2,595.0615	4,743.5091	8,766.8404
	50	9,601.369	2,602.0405	4,766.8795	8,794.7459
114	0	9,610.456	2,609.0261	4,790.3767	8,822.7764
	10	9,619.524	2,616.0183	4,814.0017	8,850.9327
	20	9,628.570	2,623.0171	4,837.7554	8,879.2157
	30	9,637.597	2,630.0225	4,861.6389	8,907.6266
	40	9,646.603	2,637.0345	4,885.6531	8,936.1663
	50	9,655.588	2,644.0529	4,909.7990	8,964.8357

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
115	0	9,664.554	2,651.0779	4,934.0776	8,993.6359
	10	9,673.498	2,658.1095	4,958.4899	9,022.5679
	20	9,682.423	2,665.1475	4,983.0370	9,051.6328
	30	9,691.327	2,672.1920	5,007.7198	9,080.8315
	40	9,700.210	2,679.2429	5,032.5395	9,110.1651
	50	9,709.073	2,686.3003	5,057.4970	9,139.6348
116	0	9,717.915	2,693.3642	5,082.5935	9,169.2415
	10	9,726.737	2,700.4345	5,107.8300	9,198.9863
	20	9,735.538	2,707.5112	5,133.2077	9,228.8704
	30	9,744.318	2,714.5942	5,158.7275	9,258.8948
	40	9,753.078	2,721.6837	5,184.3907	9,289.0607
	50	9,761.818	2,728.7795	5,210.1984	9,319.3692
117	0	9,770.536	2,735.8817	5,236.1516	9,349.8215
	10	9,779.234	2,742.9902	5,262.2516	9,380.4186
	20	9,787.911	2,750.1050	5,288.4995	9,411.1618
	30	9,796.568	2,757.2261	5,314.8964	9,442.0523
	40	9,805.204	2,764.3535	5,341.4436	9,473.0912
	50	9,813.819	2,771.4872	5,368.1423	9,504.2797
118	0	9,822.413	2,778.6271	5,394.9937	9,535.6191
	10	9,830.987	2,785.7733	5,421.9989	9,567.1105
	20	9,839.540	2,792.9257	5,449.1593	9,598.7553
	30	9,848.072	2,800.0843	5,476.4761	9,630.5547
	40	9,856.583	2,807.2491	5,503.9505	9,662.5100
	50	9,865.073	2,814.4201	5,531.5839	9,694.6224
119	0	9,873.542	2,821.5973	5,559.3775	9,726.8932
	10	9,881.991	2,828.7806	5,587.3326	9,759.3238
	20	9,890.419	2,835.9701	5,615.4506	9,791.9155
	30	9,898.825	2,843.1657	5,643.7327	9,824.6696
	40	9,907.211	2,850.3673	5,672.1804	9,857.5875
	50	9,915.576	2,857.5751	5,700.7950	9,890.6706

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
120	0	9,923.920	2,864.7890	5,729.5780	9,923.9202
	10	9,932.243	2,872.0089	5,758.5305	9,957.3377
	20	9,940.544	2,879.2348	5,787.6542	9,990.9246
	30	9,948.825	2,886.4668	5,816.9504	10,024.6823
	40	9,957.085	2,893.7048	5,846.4206	10,058.6123
	50	9,965.324	2,900.9488	5,876.0663	10,092.7159
121	0	9,973.541	2,908.1988	5,905.8888	10,126.9948
	10	9,981.738	2,915.4547	5,935.8897	10,161.4503
	20	9,989.913	2,922.7166	5,966.0706	10,196.0841
	30	9,998.067	2,929.9844	5,996.4328	10,230.8976
	40	10,006.200	2,937.2582	6,026.9781	10,265.8925
	50	10,014.312	2,944.5379	6,057.7079	10,301.0701
122	0	10,022.403	2,951.8234	6,088.6239	10,336.4323
	10	10,030.473	2,959.1148	6,119.7276	10,371.9805
	20	10,038.521	2,966.4121	6,151.0206	10,407.7164
	30	10,046.548	2,973.7153	6,182.5047	10,443.6416
	40	10,054.554	2,981.0243	6,214.1813	10,479.7578
	50	10,062.539	2,988.3390	6,246.0523	10,516.0667
123	0	10,070.502	2,995.6596	6,278.1193	10,552.5699
	10	10,078.444	3,002.9860	6,310.3841	10,589.2692
	20	10,086.365	3,010.3181	6,342.8483	10,626.1664
	30	10,094.264	3,017.6560	6,375.5137	10,663.2632
	40	10,102.142	3,024.9996	6,408.3821	10,700.5613
	50	10,109.998	3,032.3490	6,441.4553	10,738.0626
124	0	10,117.834	3,039.7040	6,474.7352	10,775.7689
	10	10,125.648	3,047.0648	6,508.2235	10,813.6821
	20	10,133.440	3,054.4312	6,541.9221	10,851.8040
	30	10,141.211	3,061.8032	6,575.8329	10,890.1365
	40	10,148.960	3,069.1810	6,609.9578	10,928.6815
	50	10,156.688	3,076.5643	6,644.2987	10,967.4410

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
125	0	10,164.395	3,083.9533	6,678.8577	11,006.4169
	10	10,172.080	3,091.3478	6,713.6366	11,045.6112
	20	10,179.744	3,098.7479	6,748.6375	11,085.0259
	30	10,187.386	3,106.1536	6,783.8625	11,124.6631
	40	10,195.006	3,113.5649	6,819.3134	11,164.5247
	50	10,202.605	3,120.9817	6,854.9926	11,204.6130
126	0	10,210.182	3,128.4040	6,890.9019	11,244.9299
	10	10,217.738	3,135.8318	6,927.0436	11,285.4777
	20	10,225.272	3,143.2651	6,963.4199	11,326.2585
	30	10,232.785	3,150.7038	7,000.0328	11,367.2744
	40	10,240.275	3,158.1480	7,036.8847	11,408.5277
	50	10,247.744	3,165.5977	7,073.9777	11,450.0207
127	0	10,255.192	3,173.0528	7,111.3141	11,491.7556
	10	10,262.618	3,180.5133	7,148.8963	11,533.7347
	20	10,270.022	3,187.9791	7,186.7264	11,575.9603
	30	10,277.404	3,195.4504	7,224.8070	11,618.4348
	40	10,284.765	3,202.9270	7,263.1404	11,661.1606
	50	10,292.104	3,210.4090	7,301.7289	11,704.1401
128	0	10,299.421	3,217.8963	7,340.5750	11,747.3757
	10	10,306.716	3,225.3889	7,379.6813	11,790.8700
	20	10,313.989	3,232.8868	7,419.0502	11,834.6254
	30	10,321.241	3,240.3900	7,458.6842	11,878.6446
	40	10,328.471	3,247.8984	7,498.5860	11,922.9300
	50	10,335.679	3,255.4121	7,538.7581	11,967.4842
129	0	10,342.865	3,262.9310	7,579.2032	12,012.3100
	10	10,350.029	3,270.4552	7,619.9239	12,057.4100
	20	10,357.172	3,277.9845	7,660.9230	12,102.7870
	30	10,364.292	3,285.5191	7,702.2032	12,148.4436
	40	10,371.391	3,293.0588	7,743.7673	12,194.3827
	50	10,378.467	3,300.6037	7,785.6181	12,240.6071

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
130	0	10,385.522	3,308.1537	7,827.7585	12,287.1196
	10	10,392.554	3,315.7088	7,870.1913	12,333.9232
	20	10,399.565	3,323.2690	7,912.9196	12,381.0208
	30	10,406.554	3,330.8343	7,955.9462	12,428.4154
	40	10,413.520	3,338.4047	7,999.2743	12,476.1100
	50	10,420.465	3,345.9802	8,042.9067	12,524.1076
131	0	10,427.388	3,353.5607	8,086.8467	12,572.4114
	10	10,434.288	3,361.1462	8,131.0975	12,621.0245
	20	10,441.167	3,368.7367	8,175.6620	12,669.9501
	30	10,448.023	3,376.3322	8,220.5437	12,719.1915
	40	10,454.857	3,383.9328	8,265.7457	12,768.7518
	50	10,461.669	3,391.5382	8,311.2715	12,818.6345
132	0	10,468.459	3,399.1486	8,357.1242	12,868.8428
	10	10,475.227	3,406.7640	8,403.3075	12,919.3803
	20	10,481.973	3,414.3842	8,449.8246	12,970.2504
	30	10,488.697	3,422.0094	8,496.6792	13,021.4565
	40	10,495.398	3,429.6394	8,543.8747	13,073.0023
	50	10,502.077	3,437.2743	8,591.4148	13,124.8913
133	0	10,508.734	3,444.9141	8,639.3032	13,177.1272
	10	10,515.369	3,452.5586	8,687.5434	13,229.7138
	20	10,521.981	3,460.2080	8,736.1394	13,282.6548
	30	10,528.571	3,467.8622	8,785.0950	13,335.9539
	40	10,535.139	3,475.5212	8,834.4139	13,389.6152
	50	10,541.685	3,483.1850	8,884.1001	13,443.6425
134	0	10,548.208	3,490.8535	8,934.1577	13,498.0398
	10	10,554.709	3,498.5267	8,984.5905	13,552.8112
	20	10,561.188	3,506.2047	9,035.4029	13,607.9608
	30	10,567.644	3,513.8873	9,086.5988	13,663.4926
	40	10,574.078	3,521.5747	9,138.1826	13,719.4111
	50	10,580.490	3,529.2667	9,190.1585	13,775.7204

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	C_1, Long Chord	M_1, Middle Ordinate	E_1, External Distance	T_1, Tangent Distance
135	0	10,586.879	3,536.9634	9,242.5308	13,832.4249
	10	10,593.246	3,544.6647	9,295.3040	13,889.5290
	20	10,599.590	3,552.3706	9,348.4826	13,947.0372
	30	10,605.913	3,560.0812	9,402.0710	14,004.9541
	40	10,612.212	3,567.7963	9,456.0740	14,063.2842
	50	10,618.489	3,575.5160	9,510.4962	14,122.0323
136	0	10,624.744	3,583.2403	9,565.3423	14,181.2031
	10	10,630.976	3,590.9691	9,620.6172	14,240.8015
	20	10,637.186	3,598.7024	9,676.3257	14,300.8324
	30	10,643.373	3,606.4402	9,732.4730	14,361.3006
	40	10,649.538	3,614.1825	9,789.0639	14,422.2114
	50	10,655.680	3,621.9293	9,846.1036	14,483.5698
137	0	10,661.800	3,629.6806	9,903.5975	14,545.3811
	10	10,667.897	3,637.4363	9,961.5506	14,607.6505
	20	10,673.971	3,645.1964	10,019.9685	14,670.3834
	30	10,680.023	3,652.9609	10,078.8565	14,733.5853
	40	10,686.052	3,660.7299	10,138.2202	14,797.2618
	50	10,692.059	3,668.5032	10,198.0653	14,861.4184
138	0	10,698.043	3,676.2808	10,258.3975	14,926.0609
	10	10,704.005	3,684.0628	10,319.2225	14,991.1952
	20	10,709.944	3,691.8492	10,380.5463	15,056.8271
	30	10,715.860	3,699.6398	10,442.3749	15,122.9626
	40	10,721.753	3,707.4348	10,504.7144	15,189.6078
	50	10,727.624	3,715.2340	10,567.5710	15,256.7690
139	0	10,733.472	3,723.0374	10,630.9509	15,324.4524
	10	10,739.298	3,730.8452	10,694.8606	15,392.6645
	20	10,745.100	3,738.6571	10,759.3066	15,461.4117
	30	10,750.880	3,746.4733	10,824.2955	15,530.7007
	40	10,756.638	3,754.2936	10,889.8340	15,600.5382
	50	10,762.372	3,762.1182	10,955.9290	15,670.9310

Table F-1. Functions of a 1-degree curve (continued)

Degrees	Minutes	$C_{1°}$ Long Chord	$M_{1°}$ Middle Ordinate	$E_{1°}$ External Distance	$T_{1°}$ Tangent Distance
140	0	10,768.084	3,769.9469	11,022.5873	15,741.8861
	10	10,773.773	3,777.7797	11,089.8161	15,813.4106
	20	10,779.439	3,785.6167	11,157.6226	15,885.5116
	30	10,785.082	3,793.4578	11,226.0140	15,958.1965
	40	10,790.703	3,801.3029	11,294.9977	16,031.4726
	50	10,796.301	3,809.1522	11,364.5814	16,105.3476
141	0	10,801.875	3,817.0055	11,434.7727	16,179.8291

Table F-2.Corrections for Tangents and Externals

For tangents add--						
Angle in Degrees	5° curve	10° curve	15° curve	20° curve	25° curve	30°curve
10	0.03	0.06	0.09	0.13	0.16	0.19
20	0.06	0.13	0.19	0.26	0.32	0.39
30	0.10	0.19	0.29	0.39	0.49	0.59
40	0.13	0.26	0.40	0.53	0.67	0.80
50	0.17	0.34	0.51	0.68	0.85	1.02
60	0.21	0.42	0.63	0.84	1.05	1.27
70	0.25	0.51	0.76	1.02	1.28	1.54
80	0.30	0.61	0.91	1.22	1.53	1.84
90	0.36	0.72	1.09	1.45	1.83	2.20
100	0.43	0.86	1.30	1.74	2.18	2.62
110	0.51	1.03	1.56	2.08	2.61	3.14
120	0.62	1.25	1.93	2.52	3.16	3.81

Table F-2. Corrections for Tangents and Externals (continued)

For externals add--						
Angle in Degrees	5° curve	10° curve	15° curve	20° curve	25° curve	30° curve
10	0.001	0.003	0.004	0.006	0.007	0.008
20	0.006	0.011	0.017	0.022	0.028	0.034
30	0.013	0.025	0.038	0.051	0.065	0.078
40	0.023	0.046	0.070	0.093	0.117	0.141
50	0.037	0.075	0.116	0.151	0.189	0.227
60	0.056	0.112	0.168	0.225	0.283	0.340
70	0.080	0.159	0.240	0.321	0.403	0.485
80	0.110	0.220	0.332	0.445	0.558	0.671
90	0.149	0.299	0.450	0.603	0.756	0.910
100	0.200	0.401	0.604	0.809	1.015	1.221
110	0.268	0.536	0.806	1.082	1.355	1.633
120	0.380	0.721	1.086	1.456	1.825	2.197

APPENDIX G

FROST DESIGN FOR ROADS

FROST-AREA CONSIDERATIONS

In areas where frost effects have an impact on the design of roads, additional considerations concerning thicknesses and required layers in the road structure must be addressed. The specific areas where frost has an impact on the design are discussed in the following paragraphs: however, a more detailed discussion of frost effects is presented in Special Report 83-27. For frost-design purposes, soils have been divided into seven groups as shown in Table G-1. Only the NFS group is suitable for a base course. NFS, S1, S2, F1, or F2 soils may be used for a subbase course, and any of the six groups may be encountered as subgrade soils. Soils are listed in approximate order of decreasing bearing capability during periods of thaw.

REQUIRED THICKNESS

Where frost-susceptible subgrades are encountered, the section thickness required will be determined according to the reduced-subgrade-strength method. The reduced-subgrade-strength method requires the use of frost-area soil-support indexes listed in Table G-2, page G-2, and strength curves shown in Figure G-1, page G-2. The

Table G-1. Frost-design soil classification

Frost Group	Type of Soil	% By Weight < 0.02 mm	Typical Soil Types Under the USCS
NFS	(a) Gravels (e ≥ 0.25)	0 - 3	GW, GP
	Crushed stone	0 - 3	GW, GP
	Crushed rock	0 - 3	GW, GP
	(b) Sands (e ≤ 0.30)	0 - 3	SW, SP
	(c) Sands (e > 0.30)	3 - 10	SP
S1	(a) Gravels (e < 0.25)	0 - 3	GW, GP
	Crushed stone	0 - 3	GW, GP
	Crushed rock	0 - 3	GW, GP
	(b) Gravelly soils	3 - 6	GW, GP, GW-GM, GP-GM, GW-GC, GP-GC
S2	Sandy soils (e ≤ 0.30)	3 - 6	SW, SP, SW-SM, SP-SM, SW-SC, SP-SC
F1	Gravelly soils	6 - 10	GW-GM, GP-GM, GW-GC, GP-GC
F2	(a) Gravelly soils	10 - 20	GM, GC, GM-GC
	(b) Sands	6 - 15	SM, SC, SW-SM, SP-SM, SW-SC, SP-SC, SM-SC
F3	(a) Gravelly soils	> 20	GM, GC, GM-GC
	(b) Sands, except very fine silty sands	> 15	SM, SC, SM-SC
	(c) Clays (PI > 12)	-	CL, CH, ML-CL
F4	(a) Silts	-	ML, MH, ML-CL
	(b) Very fine sands	> 15	SM, SC, SM-SC
	(c) Clays (PI < 12)	-	CL, ML-CL
	(d) Varved clays and other fine-grained, banded sediments	-	CL or CH layered with ML, MH, SM, SC, SM-SC, or ML-CL

NOTE: e = void ratio.

Table G-2. Frost-area soil-support indexes of subgrade soils

Frost Group of Subgrade Soils	Frost-Area Soil-Support Index
F1 and S1	9.0
F2 and S2	6.5
F3 and F4	3.5

required thickness is determined by comparing the natural subgrade CBR to the frost-area soil-support index associated to the relevant first group. If the natural subgrade CBR is less than the frost-area soil-support index then the CBR value governs the design, and the thickness is determined from Figure G-2. If the natural subgrade CBR is greater than the soil-support index, then Figure G-1 is used. The required thickness is determined by entering Figure G-1 at the correct design index moving horizontally to intersect the relevant frost-group curve, and then moving vertically downward to determine the design thickness in inches.

REQUIRED LAYERS IN A ROAD SECTION

When frost is a consideration, the road section should consist of a series of layers that will ensure the stability of the system, particularly during thaw periods. The layered system in the aggregate fill may consist of a wearing surface of fine-crushed stone, a coarse-graded base course, and a well-graded subbase of sand or gravelly sand as shown in the following example:

A wearing surface of fine-graded crushed rock.
A base course of coarse-graded crushed rock.
A subbase of well-graded sand, frost group soils S1 and S2, or geotextile.

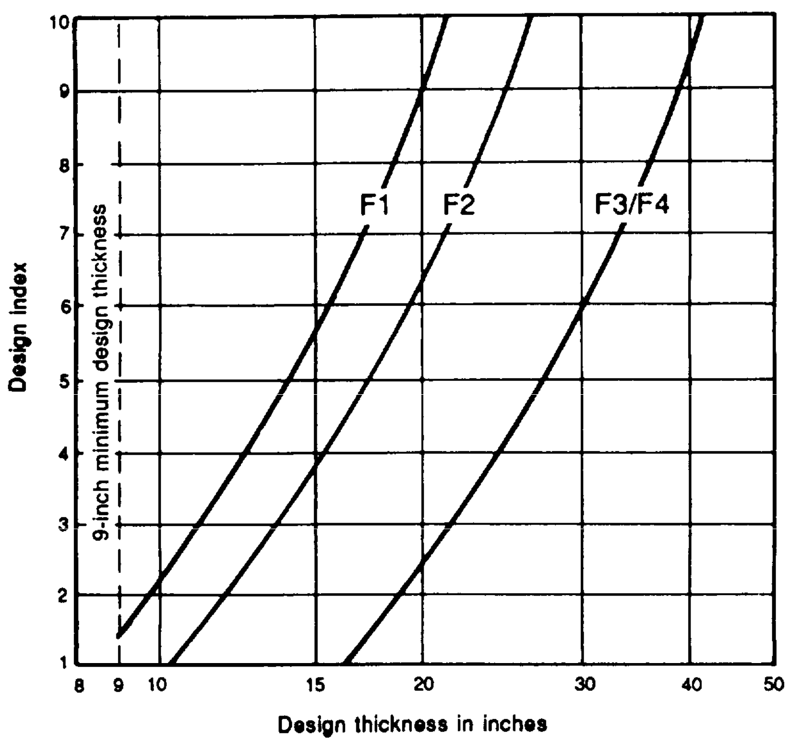


Figure G-1. Frost-design reduced-subgrade-strength curves

To ensure the stability of the wearing surface, the width of the base course and subbase should exceed the final desired surface width by a minimum of 1 foot on each side.

WEARING SURFACE

The wearing surface contains fines to provide stability in the aggregate surface. The presence of fines helps the layer's compaction characteristics and helps to provide a relatively smooth riding surface. Its thickness will vary between 4 and 6 inches.

BASE COURSE

The coarse-graded base course is important in providing drainage of the granular fill. It is also important that this material be NFS so that it retains its strength during spring thaw.

SUBBASE

The well-graded sand subbase is used for additional bearing capacity over the frost-susceptible subgrade and as a filter layer between the coarse-graded base course and the subgrade. This process prevents the migration of the subgrade into the voids in the coarser material during periods of reduced subgrade strength. The material must therefore meet standard filter criteria.

The sand subbase must be either NFS, S1, or S2. The filter layer may or may not be necesary depending upon the type of subgrade material. If the subgrade consists principally of gravel or sand, the filter layer may not be necessary and may be replaced by additional base course material, if the gradation of the base course is such that it meets filter criteria. However, for finer-grained soils, the filter layer will be necessary. If a geotextile is used, the sand subbase or filter layer may be omitted because the fabric will be placed directly on the subgrade and will act as a filter. If select materials are used, they must be either NFS, S1, S2, F1, or F2 from Table G-1, page G-1.

COMPACTION

The subgrade should be compacted to provide uniformity of conditions and a firm working platform for placement and compaction of the subbase. However, compaction of the subgrade will not change its frost-area soil-support index because frost action will cause the subgrade to revert to a weaker state.

THICKNESS OF BASE COURSE AND FILTER LAYER

The relative thicknesses of the base course and filter layer are variable and should be based on the required cover (minimum of 4 inches] and economic considerations.

FROST-AREA DESIGN STEPS

Steps 1 through 5 are the same as for regular aggregate-surfaced roads. (Refer to Chapter 9, page 9-66.)

6. Determine the applicable frost group for the subgrade type from Table G-1, page G-1.

7. Determine the frost-area soil-support index from Table G-2, page G-4, based on the applicable frost group.

8. Determine the required road-structure thickness. First, compare the natural subgrade CBR to the frost-area soil-support index.

a, If the natural subgrade CBR is less than the frost-area soil-support index, then the CBR value governs the design, and the thickness is determined from Figure G-2, page G-4, as in nonfrost design.

b. If the natural subgrade CBR is greater than the frost-area soil-support index, then Figure G-1, page G-2, is used. The required thickness is determined by entering Figure G-1 at the design index, moving horizontally to intersect the frost-group curve, and then moving vertically downward to determine the thickness in inches.

9. Determine the required compaction densities for each layer from Table 9-12, page 9-63.

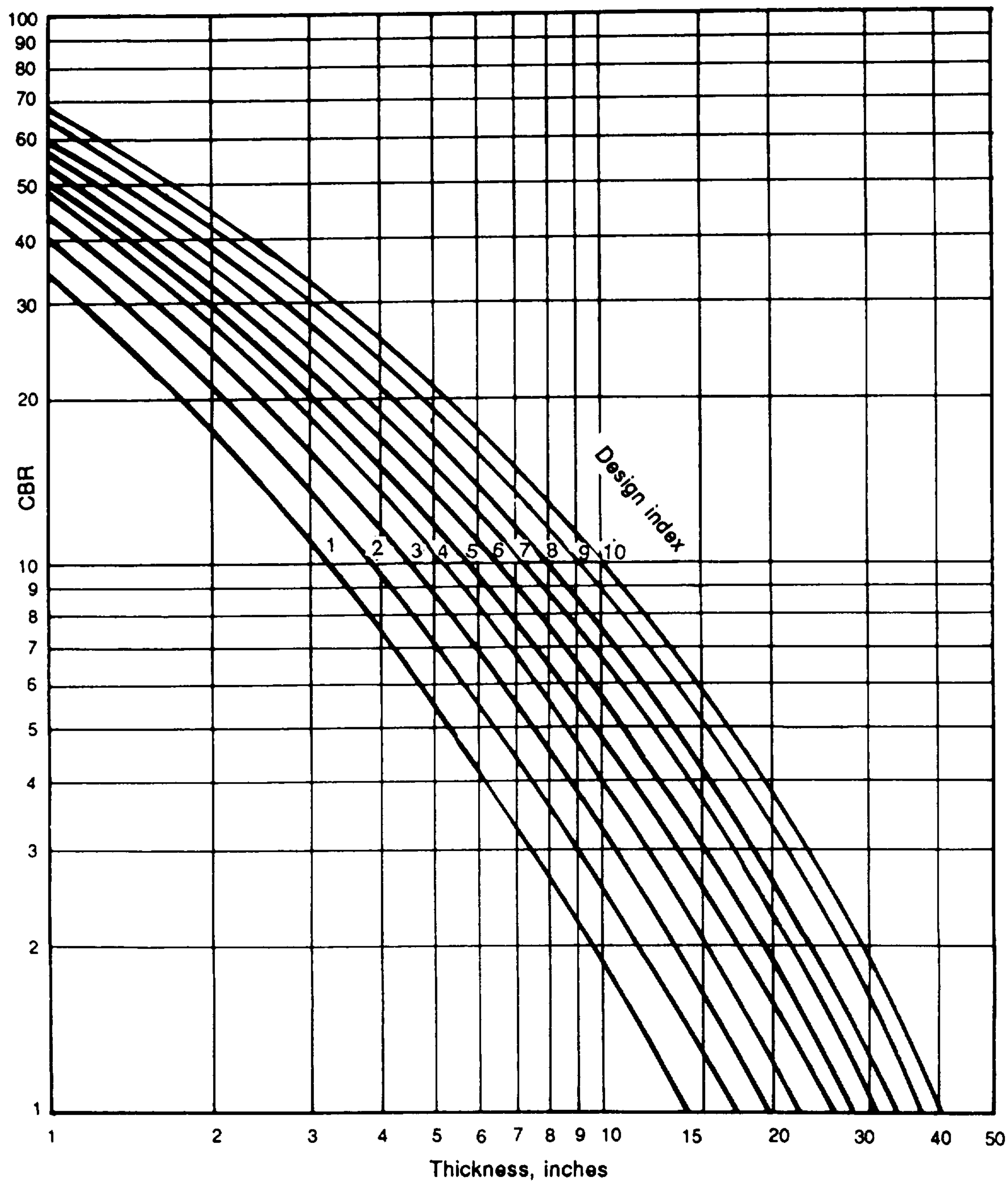


Figure G-2. Design curves for aggregate-surfaced roads

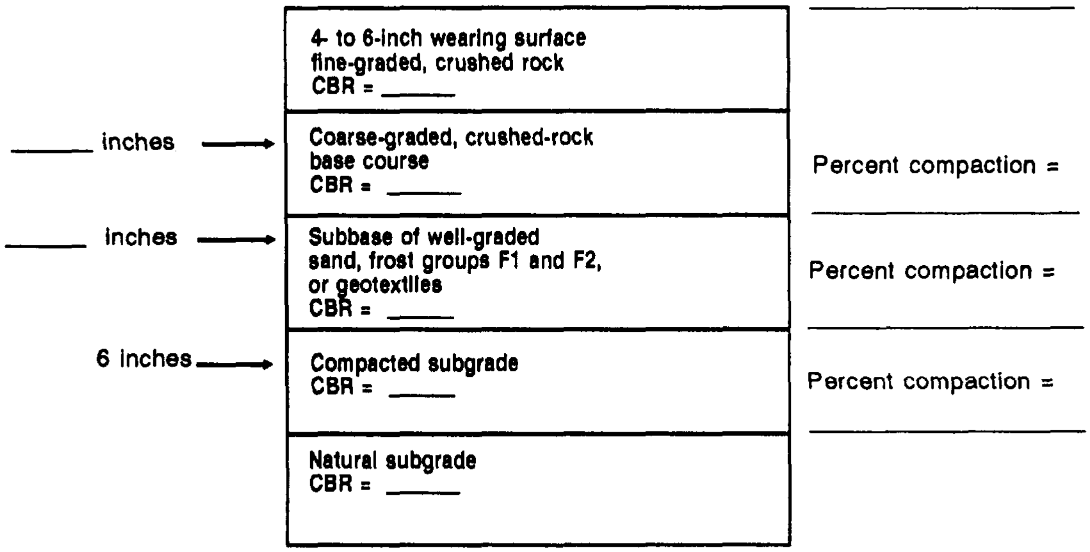
10. Draw the section of the aggregate road structure.

NOTES:

1. All layer depths should be rounded up to the next full inch for construction purposes.

2. The material should meet gradation requirements.

3. After all possible design sections are determined, the final section used should be determined on the basis of economic analysis.



Example (Frost-Area Design):

An aggregate-surfaced road in a frost area is to be used for one year. The road will be subject to—

Vehicles	Average Daily Traffic
M998 HMMWV	1,800
M929 5-ton dump (2 average trucks)	600

Available material CBR:
Natural subgrade = 4 (Clay PI = 14)
Compacted subgrade = 8
Fine-graded, crushed rock = 80
Coarse-graded, crushed rock = 80
Clean sand subbase = 15

Solution:

1. Number of daily passes = 2,400 (given).

2. Select road class D from Table 9-8, page 9-59, based on average daily traffic of 2,400.

3. Select traffic category IV from page 9-59, based upon the presence of the 25-percent truck traffic.

4. Select design index of 4 from Table 9-9, page 9-59.

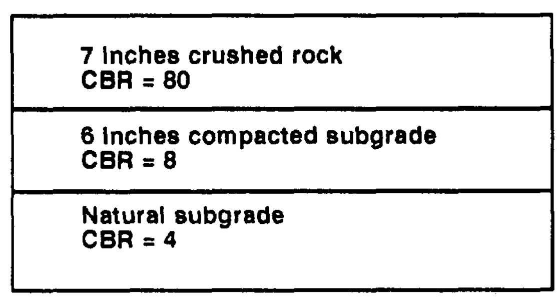
5. Natural subgrade CBR = 4.
Compacted subgrade CBR = 8

6. From Table G-1, page G-1, the subgrade frost group is F3 based on it being a clay (CL) material.

7. Select frost-area support-index of 3.5 from Table G-1, based on frost group F3.

8. Determine the acquired road-structure thickness.

a. First, look at the required road thickness if it was not designed for frost. In this case, the compact subgrade CBR = 8 is used in Figure G-2, page G-3. This results in a required total thickness of 6.25 inches, rounding up to 7 inches as shown below.



b. Now, design the road for frost. In this case, the natural subgrade CBR = 4, and the frost-area soil-support index = 3.5. Since the natural subgrade CBR is greater than the frost-area soil-support Index, Figure G-1 is used. A design index of 4 is entered into Figure G-1 resulting in a design thickness of 24.5 inches, rounding up to 25 inches. Notice the rather large difference in design thicknesses between frost design and non frost design.

9. Compaction densities for each layer are determined from Table 9-12, page 9-60.

Wearing course: at least 100 percent.
Base course: at least 100 percent.
Subbase course: 100 to 105 percent.
Subgrade: 90 to 95 percent for cohesion soil (PI>5).

10. Draw the section of the frost-area, aggregate road structure.

Layer	Compaction
4 inches wearing surface fine-graded, crushed rock CBR = 80	
12 inches coarse-graded, crushed rock CBR = 80	Percent compaction = at least 100%
9 inches clean sand subbase filter CBR = 15	Percent compaction = 100-105%
6 inches compacted subgrade CBR = 8	Percent compaction = 90-95%
Natural subgrade CBR = 4	

NOTES:

1. The function of the subbase as a filter layer is not always required, depending upon the subgrade material. In this case, the subgrade is a CL; therefore, it is required.

2. For economy, the thicknesses of the base and subbase courses can be adjusted, so long as the minimum thickness above the CBR=15 subbase is maintained at 4 inches, as determined from Figure G-2, page G-3.

3. An overall minimal thickness layer of 4 inches should be maintained.

When using a geotextile as a filter layer, the design above could be used by deducting 6 inches of the clean sand subbase and replacing it with a geotextile. The total thickness above the geotextile must be a minimum of 25 inches. Two alternate designs using geotextile are shown in the example at the bottom of the page.

Layer	Compaction
4 inches wearing surface fine-graded, crushed rock CBR = 80	Percent compaction = at least 100%
21 inches coarse-graded, crushed rock CBR = 80 **Geotextile**	
6 inches compacted subgrade CBR = 8	Percent compaction = 90-95%
Natural subgrade CBR = 4	

OR

Layer	Compaction
6 inches wearing surface fine-graded, crushed rock CBR = 80	Percent compaction = at least 100%
19 inches coarse-graded, crushed rock CBR = 80 **Geotextile**	
6 inches compacted subgrade CBR = 8	Percent compaction = 90-95%
Natural subgrade CBR = 4	

APPENDIX H

GEOTEXTILE DESIGN

DESIGN GUIDELINES

The widespread acceptance of geotextiles for use in engineering designs has led to a proliferation of geotextile manufacturers and a multitude of geofabrics, each with different engineering characteristics from which to choose. The design guidelines and methodology that follow help you select the right geofabric to meet your construction requirements.

UNPAVED-AGGREGATE DESIGN

Site Reconnaissance

As with any construction project, a site reconnaissance provides insight on construction requirements and potential problems.

Determine Subgrade Soil Type and Strength

Identify the subgrade soil and determine its strength as outlined in Chapter 9, FM 5-410. If possible, determine the soil's shear strength, C, in psi. If you are unable to determine C, use the nomograph in Figure H-1 to convert CBR value or CI to C.

Determine Permissible Load on the Subgrade Soil

The amount of loading that can be applied without causing the subgrade soil to fail is referred to as the *permissible stress, S.*

- Permissible subgrade stress **without** a geotextile:

$$S = (2.8)C$$

- Permissible subgrade stress **with** a geotextile:

$$S = (5.0)C$$

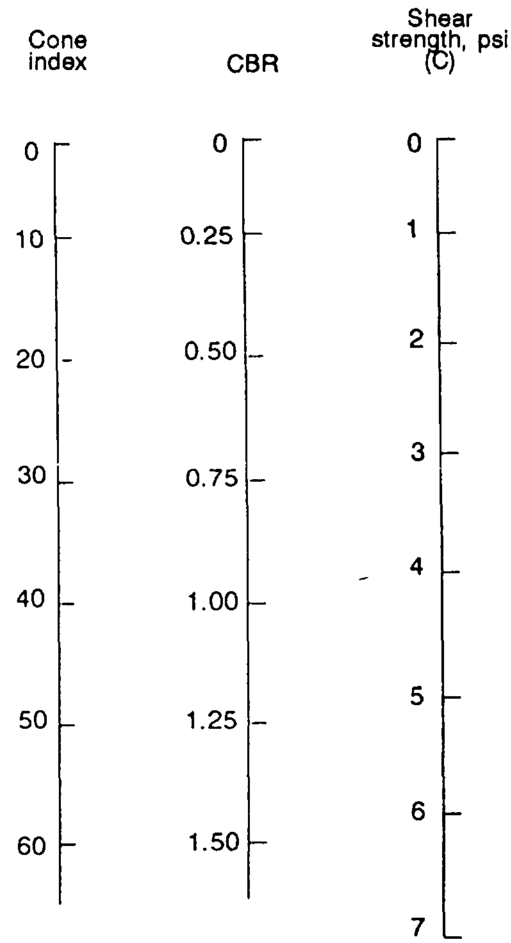


Figure H-1. Determining the soils's shear strength by converting CBR value or cone index

Determine Wheel Loads, Contact Pressure, and Contact Area

Estimate wheel loads, contact pressure, and contact-area dimensions from Table H-2. For geotextile design, single and dual wheels are represented as single-wheel loads (L) equal to one-half the axle load, The wheel load exerted by a single wheel is applied at a surface contact pressure (P) equal to the tire inflation pressure. Dual-wheel loads apply a P equal to 75 percent of the tire inflation pressure. Tandem axles exert 20 percent more than their actual weight to the subgrade soil due to overlapping stress from the adjacent axle in the tandem set,

- Estimate the area being loaded (B^2):

$$B^2 = \frac{L}{P}$$

where B^2 = length of one side of the square contact area

Determine Aggregate-Base Thickness

Assuming that wheel loads will be applied over a square area, we can use the Boussinesq theory of load distribution to determine the aggregate-section thickness required to support the design load. Boussinesq theory coefficients are found in Table H-2.

Table H-1. Vehicle input parameters

Vehicle Type (Choose Category Nearest the Actual Design Vehicles)	Axles S - Single T - Tandem	Wheels S - Single D - Dual	Axle Loads (lb)	Wheel Loads[1] L (lb)	Typical[2] Tire Inflation Pressure (psi)	Contact Pressure[3] P (psi)	Wheel Contact Area B^2 (in^2)	One Side of Square Contact Area B (in)
Highway Legal Vehicles								
Haul trucks[4] - F Axle (stone, concrete)	S	S	18,000	9,000	110	110	82	9.0
R Axle	T	D	18,000	10,800	110	83	130	11.4
Tractor trailer - F Axle	S	S	18,000	9,000	120	120	75	8.7
(18 wheeler) - R Axle	T	D	18,000	10,800	120	90	120	11.0
Off Highway Vehicles[5]								
35-ton trucks - F Axle	S	S	48,000	24,000	90	90	267	16.3
(CAT 769C) - R Axle	S	D	89,200	44,600	90	68	656	25.6
Wheel loader - F Axle	S	S	24,000	12,000	50	50	240	15.5
(CAT 910) - R Axle	S	S	10,000	5,000	50	50	100	10.0
Wheel loader - F Axle	S	S	37,000	18,500	60	60	308	17.6
(CAT 930) - R Axle	S	S	14,000	7,000	60	60	117	10.8
Wheel loader - F Axle	S	S	65,000	32,000	60	60	542	23.3
(CAT 966C) - R Axle	S	S	25,000	12,500	60	60	208	14.4
Wheel loader - F Axle	S	S	136,000	68,000	85	85	800	28.3
(CAT 988B) - R Axle	S	S	55,000	27,500	85	85	324	18.0
Wheel loader - F Axle	S	S	290,000	145,000	70	70	2071	45.5
(CAT 992) - R Axle	S	S	120,000	60,000	60	60	1000	31.6
Scraper - F Axle	S	S	88,600	44,300	80	80	554	23.5
(CAT 631D) - R Axle	S	S	75,400	37,700	75	75	503	22.4
Scraper - F Axle	S	S	120,000	60,000	85	85	706	26.6
(CAT 651B) - R Axle	S	S	110,800	55,400	80	80	692	26.3

NOTES:
1. Wheel load is one-half the axle load and increased by 20% if the wheel is on a tandem axle.
2. Maximum tire inflation pressure is given for each class of vehicle. Using tires with lower inflation pressures would lower the contact pressures and allow for less thickness of the aggregate structural section.
3. Same as tire inflation pressure except that a factor of 0.75 times the inflation pressure must be used for all dual wheels.
4. Trucks used on- and off-highway generally use lower inflation pressure tires requiring only 75 to 90 psi.
5. Manufacturers' specifications should be consulted for off-highway vehicles. Wide ranges of different inflation pressure tires are available for these vehicles.

Table H-2. Boussinesq theory coefficients

If X =	Then M =
0.005	0.10
0.011	0.15
0.018	0.20
0.026	0.25
0.037	0.30
0.048	0.35
0.060	0.40
0.072	0.45
0.084	0.50
0.096	0.55
0.107	0.60
0.118	0.65
0.128	0.70
0.138	0.75
0.146	0.80
0.155	0.85
0.162	0.90
0.169	0.95
0.175	1.00
0.186	1.10
0.196	1.20
0.207	1.35
0.215	1.50
0.224	1.75
0.232	2.00
0.237	2.25
0.240	2.50
0.242	2.75
0.244	3.00
0.247	4.00
0.249	5.00
0.249	7.50
0.250	10.00
0.250	∞

- First, solve for X.

  Without a geotextile: $X = \frac{S}{(4)P}$

  With a geotextile: $X_{geotextile} = \frac{S_{geotextile}}{(4)P}$

- Using the calculated values of X and $X_{geotextile}$, find the corresponding value of M and $M_{geotextile}$ from Table H-2.

- Then solve for aggregate-base thickness H and H geotextile.

  Without a geotextile: $H = \frac{B\,(inches)}{(2)M}$

  With a geotextile:

  $$H_{geotextile} = \frac{B}{(2)M_{geotextile}}$$

The difference between H and H geotextile is the aggregate savings due to the geotextile.

Adjust Aggregate-Section Thickness for Aggregate Quality

The design method is based on the assumption that good-quality aggregate (minimum CBR value of 80) is used. If lower-quality aggregate is used, the aggregate-section thickness must be adjusted.

Table H-3, page H-4, contains typical compacted strength properties of common structural materials. These values are approximations: use more specific data if it is available. Extract the appropriate thickness equivalent factor from Table H-3, then divide H by that factor to determine the adjusted aggregate-section thickness.

Adjust Aggregate-Base Thickness for Service Life

The design method assumes that the pavement will be subjected to 1,000 passes of the maximum design axle load. If the traffic is greater than 1,000 passes, increase H by the following percentages:

2,000 passes	8%
5,000 passes	19%
10,000 passes	27%

If you anticipate more than 10,000 passes, you need to increase the design thickness by 30 percent and monitor the performance of the road.

A second method of determining minimum required cover above a subgrade for wheeled vehicles with and without a geotextile requires fewer input parameters. Again, use Figure H-1 to correct CBR or CI values to a C value. Determine the permissible stress on the subgrade soil (S) by multiplying C by 2.8 without a geotextile and by 5.0 with a geotextile. Select the heaviest vehicle using the road and the design vehicle for each wheel-load configuration: single, dual, or tandem. Enter the appropriate graph (see Figures H-2, H-3, or H-4, pages H-5 through H-7) at S (with and without a geotextile). Round design-vehicle wheel loads to the next higher 5,000-pound increment. Determine the intersection between the appropriate wheel-load curve and S (with and without a geotextile), then read the minimum required thickness on the left axis. Use the greatest thickness values as

Table H-3. Typical compacted strength properties of common structural materials

Material	CBR Range	Thickness Equivalency Factor
Asphalt, concrete plant mix, high stability	>100	3.00
Crushed hard rock	80-100	1.00
Crushed medium-hard rock	60-80	0.85
Well-graded gravel	40-70	0.80
Shell	40-60	0.75
Sand-gravel mixtures	20-50	0.50
Soft rock	20-40	0.45
Clean sand	10-30	0.40
Lime-treated base[1]	>100	1.00-2.00
Cement-treated base[1,2]		
650 psi or more	>100	1.60
400 psi to 650 psi	>100	1.40
400 psi or less	>100	1.05

[1] The strength of lime-treated and cement-treated bases depends on soil properties and construction procedures. Treated bases are also subject to long-term failure due to continuing chemical reactions over time.

[2] Compressive strength at 7 days.

Note: The values listed above are general guidelines. More exact thickness equivalency factors can be determined by comparing the CBR of the available aggregate to the design CBR of 80. For example, an aggregate with a CBR of 55 would have an approximate thickness equivalency factor of 55/80 = 0.69.

the design thickness with and without a geotextile. Compare the cost of the material saved with the cost of the geotextile to determine if the use of the geotextile is cost effective.

Up to this point in the geotextile-design process, you have been concerned with general design properties for designing unpaved aggregate roads. Now you must decide which geotextile fabric best meets your project requirements.

TYPES OF GEOTEXTILES

There are two major types of geotextiles: woven and nonwoven. Woven fabrics have filaments woven into a regular, usually rectangular, pattern with fairly even opening spacing and size. Nonwoven fabrics have filaments connected in a method other than weaving, typically needle punching or head bonding at intersection points of the filaments. The pattern and opening spacing and size are irregular in nonwoven fabrics.

Woven fabrics are generally stronger than nonwoven fabrics of the same fabric weight. Woven geotextiles typically reach peak tensile strength at between 5 and 25 percent strain. Nonwoven fabrics have a high elongation of 50 percent or more at maximum strength.

Table H-4, page H-8, provides information on important criteria and principal properties useful when selecting or specifying a geotextile for a specific application. The type of equipment used to construct a road or airfield pavement structure on top of the geotextile must be considered. Equipment ground pressure (in psi) is an important factor in determining the geotextile fabric thickness; a thicker fabric is necessary to stand up to high equipment ground pressure (see Table H-5, page H-9).

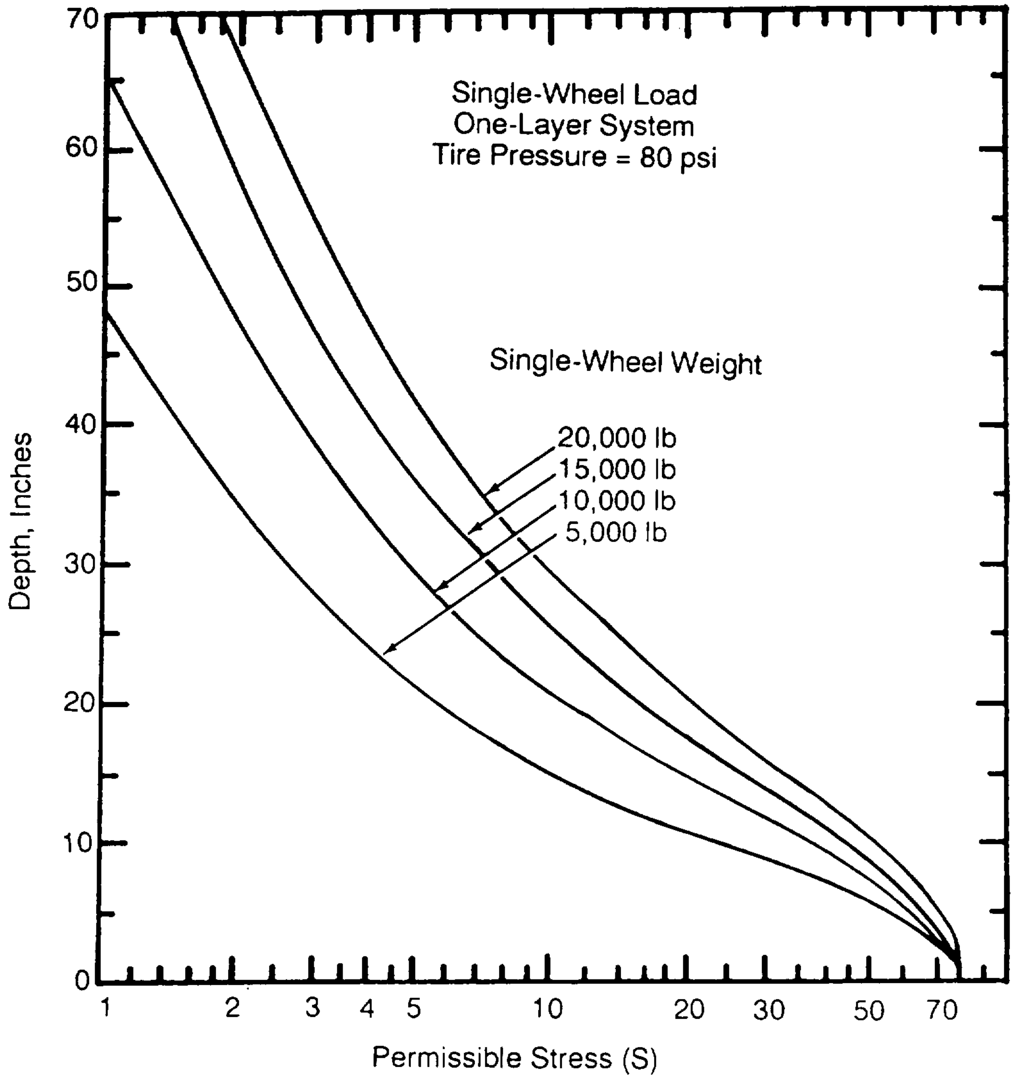


Figure H-2. Thickness design curve for slngle-wheel load on gravel-surfaced pavements

Once the required degree of geotextile surviv-ability is determined, minimum speification requirements can be established based on ASTM standards (see Table H-6, page H-10). When you have determined the set of testing standards, the geotextile will be required to withstand to meet use and construction requirements, you are ready to either specify a geotextile for ordering or evaluate on-hand stocks.

ROADWAY CONSTRUCTION

There is no singular way to construct mad-ways with geofabrics. However, there are several applications and general guidelines that can be used.

SITE PREPARATION

Clear, grub, and excavate the site to design grade: fill in ruts and surface irregularities deeper than 3 or 4 inches (see Figure H-5, page H-11). Compact the subgrade if the soil CBR is greater than 1. The compaction aids in locating unsuitable materials that may damage the fabric. Remove unsuitable materials where practical.

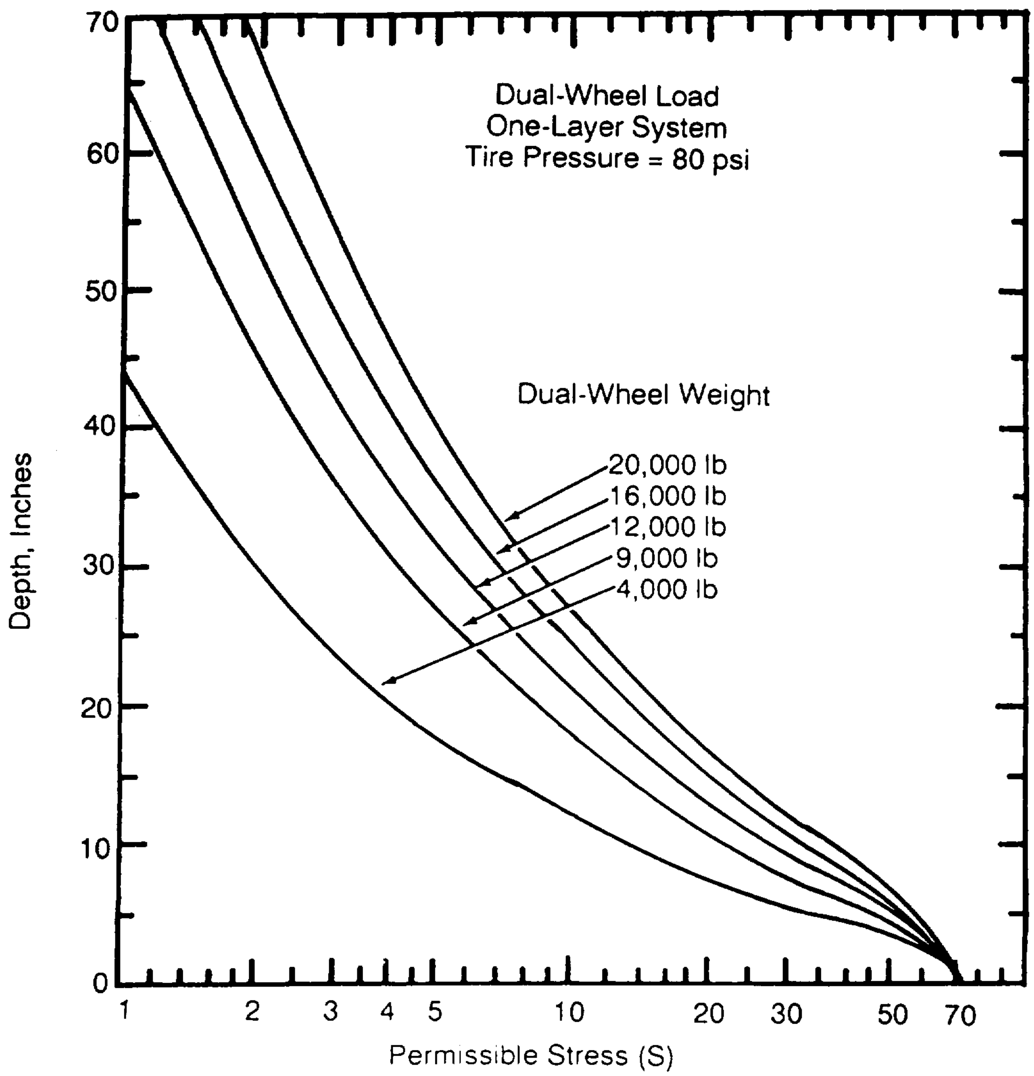


Figure H-3. Thickness design curve for dual-wheel load on gravel-surfaced pavements

When constructing over extremely soft soils (such as peat bogs), surface materials (such as the root mat) may be advantageous and should be disturbed as little as possible. Use sand or sawdust to cover roots, stumps, or stalks. This cushions the fabric and reduces the potential for fabric puncture. Nonwoven geotextiles are preferred when the soil surface is uneven.

LAYING OF FABRIC

The fabric should be rolled out by hand, ahead of backfilling, directly on the soil subgrade. The fabric is commonly, but not always, laid in the direction of the roadway. Where the subgrade cross section has large areas and leveling is not practical, the fabric may be cut and laid transverse to the roadway. Large wrinkles should be avoided. In the case of wide roads, multiple widths of fabric are laid and overlapped. Lap length normally depends on subgrade strength. Table H-7, page H-12, provides general guidelines for lap lengths.

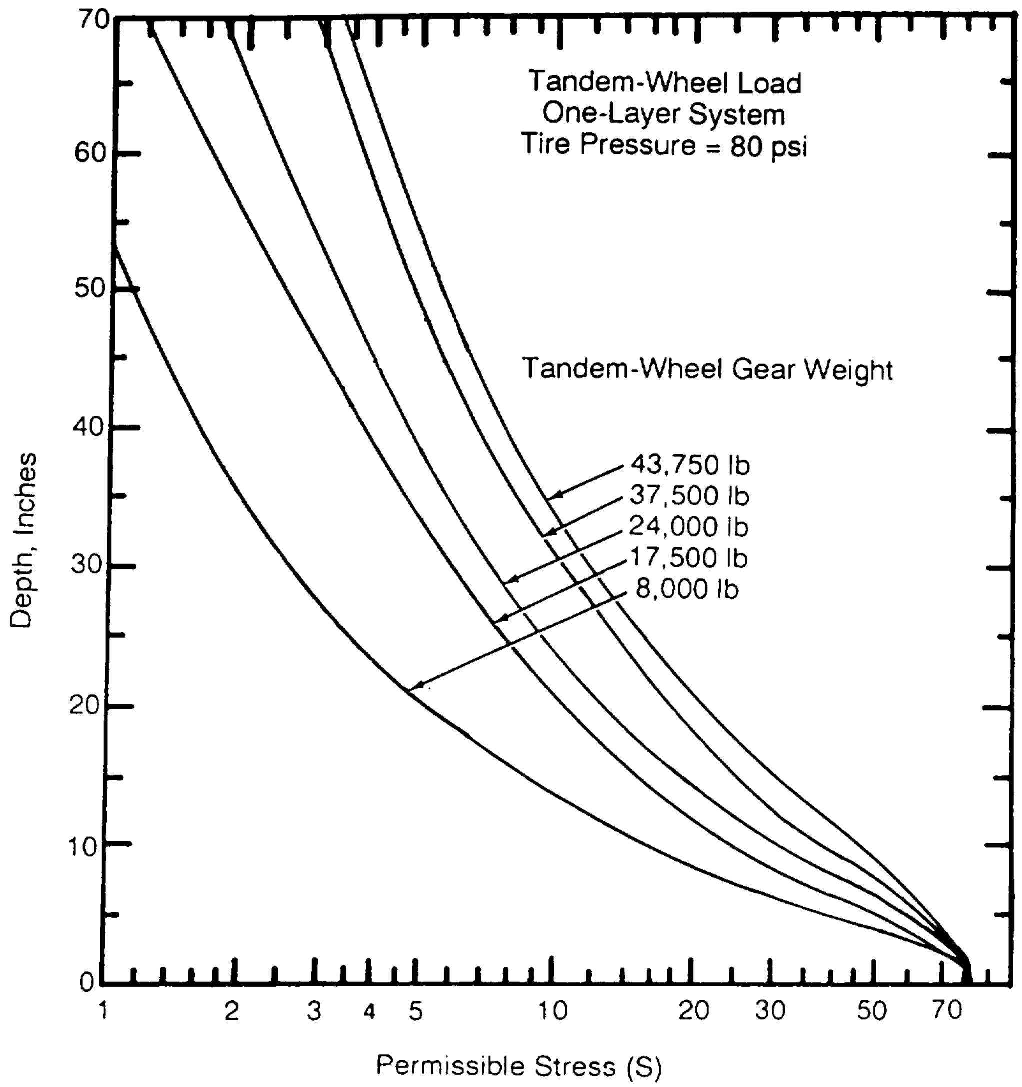


Figure H-4. Thickness design curve for tandem-wheel load on gravel-surfaced pavements

LAYING OF BASE

If angular rock is to form the base, it is common to first place a protective layer of 6 to 8 inches of finer material, Base material is then end-dumped directly onto the previously spread load, pushed out over the fabric, and spread from the center using a bulldozer. Vehicles must not be driven directly on the fabric because they might puncture it. Small, tracked bulldozers (with a maximum ground pressure of 2 psi) are commonly used for spreading. The blade is also kept high to avoid driving rock down into the fabric. After spreading, compaction and grading can be carried out with standard compaction equipment. If the roadway has side drains, they are constructed after the pavement.

Table H-4. Geotextile evaluation

Criteria and Parameter	Property	Application			
		F	D	S	R
Design Requirements					
Mechanical strength					
Tensile strength	Wide-width strength	–	–	–	x
Tensile modulus	Wide-width modulus	–	–	–	x
Seam strength	Wide width	–	–	–	x
Tension creep	Creep	–	–	–	x
Soil-fabric friction	Friction angle	–	–	–	x
Hydraulic					
Flow capacity	Permeability,	x	x	x	x
	Transmissivity	–	x	–	–
Piping resistance	Apparent opening size (AOS)	x	–	x	x
Clogging resistance	Pommetry	x	–	x	x
	Gradient ratio	x	–	–	–
Constructability Requirements					
Tensile strength	Grab strength	x	x	x	x
Seam strength	Grab strength	x	x	x	x
Bursting resistance	Mullen burst	x	x	x	x
Puncture resistance	Red puncture	x	x	x	x
Tear resistance	Trapesoidal tear	x	x	x	x

F – Filtration
D – Drainage
S – Separation
R – Reinforcement

Table H-5. Required degree of geotextile survivability as a function of cover material and construction equipment

Cover Material	6- to 12-inch Initial Lift Thickness		12- to 18-inch Initial Lift Thickness		18- to 24-inch Initial Lift Thickness	>24-inch Initial Lift Thickness
	Low-Ground-Pressure Equipment <4 psi	Medium-Ground-Pressure Equipment $\geq$4 psi, <8 psi	Medium-Ground-Pressure Equipment $\geq$4 psi, <8 psi	High-Ground-Pressure Equipment $\geq$8 psi	High-Ground-Pressure Equipment $\geq$8 psi	High-Ground-Pressure Equipment $\geq$8 psi
Fine sand to ±2-inch-diameter gravel, round to subangular	Low	Moderate	Low	Moderate	Low	Low
Coarse aggregate with diameter up to one-half proposed lift thickness, may be angular	Moderate	High	Moderate	High	Moderate	Low
Some to most aggregate with diameter greater than one-half proposed lift thickness, angular and sharp-edge few fines	High	Very High	High	Very High	High	Moderate

NOTES:

1. **For special construction techniques such as prerutting, increase geotextile survivability requirement one level.**
2. **Placement of an excessive initial cover-material thickness may cause bearing failure of soft subgrades.**

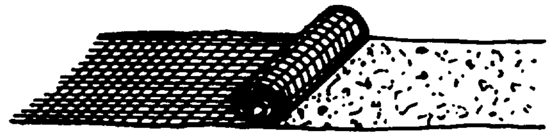
Table H-6. Minimum properties required for geotextile survivability

Required Degree of Geotextile Survivability	Grab Strength [1] lb	Puncture Strength [2] lb	Burst Strength [3] psi	Trap Tear [4] lb
Very high	270	110	430	75
High	180	75	290	50
Moderate	130	40	210	40
Low	90	30	145	30

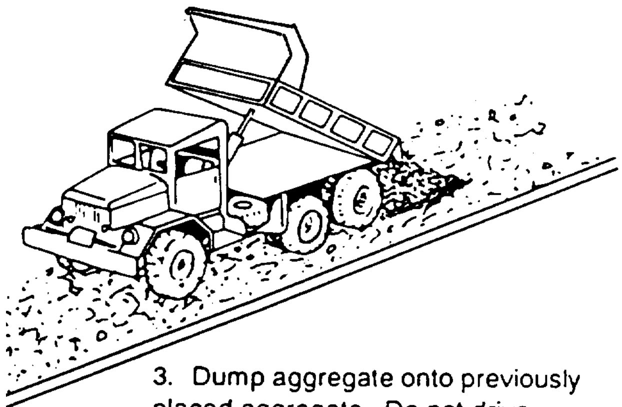
[1] ASTM D 4632
[2] ASTM D 4833
[3] ASTM D 3786
[4] ASTM D 4533, either principal direction

Note: All values represent minimum average roll values (for example, any roll in a lot should meet or exceed the minimum values in this table). These values are normally 20 percent lower than manufacturer-reported typical values.

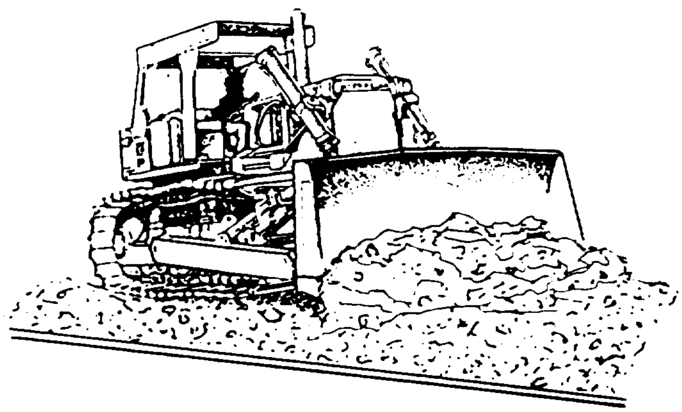
1. Prepare the ground by removing stumps, boulders, and so forth; fill in low spots.



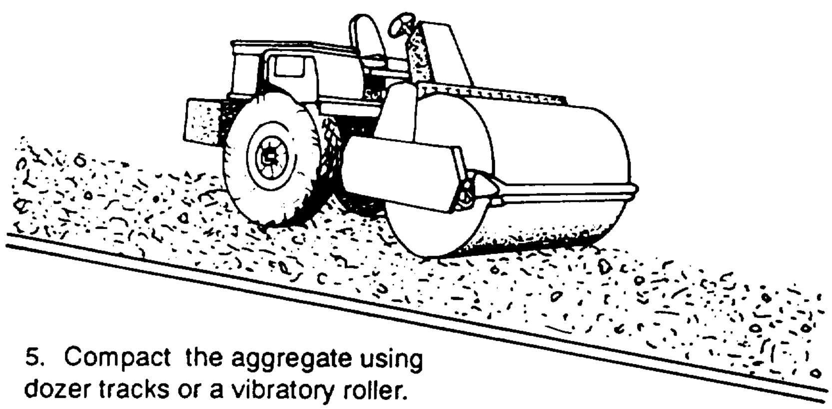
2. Unroll the geotextile directly over the ground to be stabilized. If more than one roll width is required, overlap the rolls. Inspect the geotextile.



3. Dump aggregate onto previously placed aggregate. Do not drive directly on the geotextile. Maintain at least 6 to 12 inches cover between the truck tires and the geotextile.



4. Spread the aggregate over the geotextile to the design thickness.



5. Compact the aggregate using dozer tracks or a vibratory roller.

Figure H-5. Construction sequence using geotextiles

Table H-7. Recommended minimum overlap requirements

CBR	Minimum Overlap
> 2	1 – 1.5 feet
1 – 2	2 – 3 feet
0.5 – 1	3 feet or sewn
< 0.5	Sewn
All roll ends	3 feet or sewn

GLOSSARY

AABNCP	advanced airborne control platform
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ABS	acrylonitrile-butadiene-styrene (plastic)
AC	asphalt cement
ACE	armored combat earthmover
adj	adjusted
ADR	air base damage repair
AFCS	Army Facilities Component System
AFM	Air Force manual
AFP	Air Force pamphlet
AFR	Air Force regulation
agg	aggregate
AHD	average haul distance
AML	airfield marking and lighting
ammo	ammunition
APC	armored personnel carrier
approx	approximately
Apr	April
AR	Army regulation
ASCE	American Society of Civil Engineers
ASTM	American Society of Testing and Materials
Atterberg Limits	Soil plasticity test used to measure soil cohesiveness: that is, cohesive or cohesionless,
ATTN	attention
Aug	August

av absolute volume

average daily traffic (ADT) The anticipated average number of vehicles per day that will use a completed facility.

banked cubic yardage (BCY) Soil measured in its natural state.

average running speed The speed expected to be maintained by most vehicles. It is equal to the total traveled distance divided by total time consumed.

base course or base Base course consists of well-graded, granular materials that have a liquid limit less than 25 percent and a plastic limit less than 5 percent, The base course is the most important element in a road structure. It functions as the primary load-bearing component of the road, ultimately providing the pavement (or surface) strength. Therefore, it is made of higher quality material than subbase material.

bearing capacity The ability of a soil to support a vehicle without undue sinkage of the vehicle.

benching Terracing on a slope.

berm A raised lip, usually of earth, placed at the top edge of a channel to prevent flow into the channel at places not protected against erosion.

bitumen or bituminous The most common type of asphalt surface placed in the theater of operations.

Bn battalion

borrow pit An excavated area where material has been dug for use as fill at another location.

BTU British thermal unit

BVM Bays Village of Maryland

C Celsius

C cut

CAD computer-aided design

CAMMS Condensed Army Mobility Modeling System

California Bearing Ratio (CBR) A measure of the shearing resistance of a soil under carefully controlled conditions of density and moisture.

CDR commander

CE 55 Laboratory compactive effort (CE) accomplished by the impact of 55 hammer blows per layer.

CES civil engineering squadron

CEV combat engineer vehicle

cf cubic feet

cfs cubic feet per second

CH clays, high compressibility (LL>50)

CI cone index

℄ centerline

CL clays, low compressibility (LL<50)

cm centimeter

cm/sec centimeters per second

CMD command

CMP corrugated metal pipe

co company

coarse-grained soil A free-draining soil of which more than 50 percent by weight of the grains will be retained on a No. 200 sieve. For traffic ability purposes, these are dry beach and desert soils usually containing less than 7 percent of material passing the No. 200 sieve. Gravels are not considered to pose a trafficability problem.

compacted cubic yards (CCY) A measurement of compacted soil.

compaction Process of mechanically densifying a soil, normally by the application of a moving (or dynamic) load.

compactive effort (CE) Method used to compact the soil.

cone index (CI) An index of the shearing resistance of soil. The CI is obtained with a cone penetrometer. The number represents resistance to penetration into the soil of the 30-degree cone with a 1/2-square-inch base area (actual load in pounds on cone base area in square inches), using a dial calibrated to produce an index of 300 when 150 lb of pressure are exerted on the handle. The CI reading is normally taken at the 0-inch (base of the cone) and at every 3-inch interval down to 18 inches or until the dial reaches the maximum of 300. A number of tests will be taken and each specified interval reading will be averaged. That average becomes the CI for the inch level.

CONUS continental United States

CPT captain

critical layer The soil layer that determines the rating cone index (for fine-grained soil) or cone index (for coarse-grained soil) of the area considered. Its depth varies with the soil profile and the weight and type of vehicle. Generally, the critical layer for fine-grained soils is 6 to 12 inches below the surface when subjected to passes of a vehicle. For coarse-grained soils, the critical layer is usually from the surface to a 6-inch depth for all vehicular passes.

crown The difference in elevation between the centerline and the surface edge. The crown expedites surface-water runoff on the road. The amount of crown depends on the surface used. Surfaces such as concrete or bituminous materials require little crown because of their impermeability, but permeable surfaces such as earth or gravel require a large crown.

crown The outside top of the culvert.

CSS cationic slow setting

cu cm cubic centimeter

cu ft cubic foot

CUCV commercial utility cargo vehicle

culvert An enclosed waterway used to pass water through a structure consisting of an embankment or fill.

cut or cutting That portion of through construction produced by the removal of the natural formation of earth or rock, whether sloped or level. The terms *sidehill cut* and *through-hill* cut describe the resulting cross sections commonly encountered.

cut slope The slope from the top of a cut to the ditch line (bottom of ditch). Sometimes it is called the back slope.

cy cubic yard

DA Department of the Army

DBH diameter at breast height

DD Department of Defense

Dec December

deg degree

dept department

design hourly volume (DHV) The number of vehicles that a road may typically be expected to accommodate in an hour. The DHV is 15 percent of the ADT.

design speed The speed for which a facility is designed. Pertinent geometric features, such as horizontal curves and grades, may be based on design speed.

design storm The storm of greatest intensity for a given period. For example, a "2-year design storm" is a storm expected to be equalled once in 2 years.

detention The storage of water in depressions in the earths surface.

dia diameter

dip A paved ford used for crossing dry, wide, shallow arroyos or washes in semi-arid regions subject to flash floods.

ditch slope The slope of the ditch extending from the outside edge of the shoulder to the bottom of the ditch. This slope should be relatively flat to avoid damage to vehicles driven into the ditch and to permit easy recovery.

diversion ditch A ditch used to transport water away from roadways or airfields.

DMZ demilitarized zone

drop A structure that absorbs the impact energy of water as it falls vertically to a lower level waterway.

DT ditch time

E east

elev elevation

EM engineer manual

EM enlisted member

Engr engineer

EOD explosive ordnance disposal

erosion The transportation of weathered materials by wind or water.

EW east-west

F fill

F Fahrenheit

Feb February

fill or filling Material used to fill a receptacle, cavity, passage, or low place, Using material to fill a cavity or low place.

fill slope The incline extending from the outside edge of the shoulder to the toe (bottom) of a fill.

fine-grained soil A silt or clay soil of which more than 50 percent by weight of the grains will pass a No. 200 sieve (smaller than 0.074 millimeter in diameter).

FM field manual

ford A shallow place in a waterway where the bottom permits the passage of personnel and vehicles.

fpm feet per minute
fps feet per second

frost action Processes which affect the ability of soil to support a structure when accumulated water in the form of ice lenses in the soil is subjected to natural freezing conditions.

frost-susceptible soil Soil in which significant ice segregation will occur when the necessary moisture and freezing conditions are present.

ft feet

FT Fort

ft/ft feet per foot

ft/in feet per inch

ft^2/yd^2 square feet per square yard

G gravel

gabion Large, steel wire-mesh baskets filled with stones, usually rectangular in shape and variable in size. They are designed to solve the problem of erosion.

gal gallon

gal/lb gallons per pound

gal/yd^2 gallons per square yard

GC clayey gravel

geometric design (geometry or geometric features) Refers to all visible features of the road such as lane width, shoulder width, and alignment.

GLE grade-line elevation

GM silty gravel

gm gram

GP poorly graded gravel

grade To level off to a smooth horizontal or sloping surface.

ground icing An icing whose source of water is from groundwater flow above permafrost.

groundwater table The upper limit of the saturated zone of free water.

gunite A mixture of cement, sand, and water sprayed from a high pressure nozzle onto a surface to protect it.

GW well-graded gravel

HMMWV high mobility, multipurpose wheeled vehicle

HP high point

HW high water

hydraulic gradient The slope in feet per foot of a drainage structure.

hydrologic cycle The continuous process in which water is transported from the oceans to the atmosphere to the land and back to the sea.

icing An irregular sheet or field of ice.

in inch

infiltration The absorption of rainwater by the ground on which it falls.

in/hr inches per hour

in situ Soil in its natural (undisturbed] state.

interception The holding of rainfall in the leaf canopy of trees and plants.

Jan January

Jul July

Jun June

kg kilogram

kip kilopound (1,000 pounds)

km kilometer

kph kilometers per hour

laminar flow The type of flow that occurs when viscosity forces predominate and the particles of the fluid move in smooth, parallel paths.

lat latitude

lb pound

LIP length in place

liq liquid

LL liquid limit

LOC lines of communication

LP low point

M silt

m meter

Mar March

mass diagram Earthwork volume plotted on graph paper, showing cut and fill operations.

max maximum

maximum towing force (T1) The maximum continuous towing force in pounds a vehicle can exert. It is expressed as a ratio or percentage of vehicle weight.

MD Maryland

MH silt, high compressibility (LL>50)

mi mile

min minimum

min minute

ML silt, low compressibility (LL<50)

mm millimeter

MO maximum offset

MO Missouri

mobility index (MI) A number that results from a consideration of certain vehicle characteristics.

MOPP mission-oriented protective posture

mph miles per hour

MS medium setting

N Slipperiness symbol meaning not slippery under any conditions.

N north

N/A not applicable

NATO	North Atlantic Treaty Organization
NBC	nuclear, biological, chemical
NCO	noncommissioned officer
NE	northeast
NFS	nonfrost susceptible
No.	number
Nov	November
NP	number of pipes
NRMM	NATO Reference Mobility Model
NRS	naval radio station
NS	north-south
NSN	national stock number
Ø	offset
Ott	October
OL	order length
P	Slipperiness symbol meaning slippery when wet.
PC	point of curvature
perm	permanent
permafrost	Constantly frozen ground.
PFS	possibly frost susceptible
PI	plasticity index
PI	point of intersection
POL	petroleum, oils, and lubricants
pending	The accumulation of water at the upstream end of a culvert.
pop	population
R	probability
Prime BEEF	prime base engineer emergency forces

psi pounds per square inch

PT point of tangency

PVC polyvinyl chloride

PVC point of vertical curvature

PVI point of vertical intersection

PVT point of vertical tangency

QSTAG Quadripartite Standardization Agreement

rating cone index (RCI) The measured cone index multiplied by the remolding index (RCI = CI x RI). The RCI expresses the soil-strength rating of a soil area subjected to sustained traffic.

RC rapid curing

RED HORSE rapid engineering deployable heavy operational repair squadrons, engineering

remoldable sand A poorly drained, coarse-grained soil, usually containing 7 percent or more material passing a No. 200 sieve. Poor internal drainage increases the water content greatly influencing the trafficability characteristics and permitting the remolding test to be performed. When wet, these soils react to traffic in a manner similar to fine-grained soils and are more sensitive to remolding.

remolding The changing *or* working of a soil by traffic or a remolding test. The beneficial, neutral, or detrimental effects of remolding may change soil strength.

remolding index (RI) The ratio of remolded soil strength to original strength. Soil conditions that permit the remolding test to be performed with ease will usually result in a loss of strength.

Reqd required

required towing force (T2) The force in pounds required to tow an operable, powered vehicle on level terrain.

RI remolding index

riprap Rocks or rubble placed in the bottom and on the sides of a ditch to prevent soil erosions.

river icing An icing formed along rivers or streams and adjacent areas having a source of water above or below the riverbed.

roadbed The entire width of surface on which a vehicle may stand or move. The roadbed consists of both the traveled way and the shoulders.

road classification system An organized list of four road types based on the number of vehicles each is designed to accommodate in a 24-hour period, Road characteristics are based on average daily traffic.

roadway The entire width within the limits of earthwork construction and is measured between the outside edges of cut or fill slopes. Roadway width does not include interceptor ditches if they fall outside the slopes, The roadway width varies from section to section depending on the height of cut or fill, depth of ditches, and slope ratios.

RR railroad

RRR rapid runway repair

RS rapid setting

RT road tar

RTCB road tar cutback

RTO radiotelephone operator

S Slipperiness symbol meaning slippery at all times,

S sand

S2 Intelligence Officer (US Army)

S3 Operations and Training Officer (US Army)

sand grid A honeycomb shaped geotextile measuring 20 feet by 8 feet by 8 inches deep when fully expanded. It is used to develop a beachhead for logistics-over-the-shore operations. It is also useful in expedient revetment construction.

SC supply catalog

SC slow curing

SCIP scarify and compact in place

SEATO Southeast Asia Treaty Organization

sec second

Sept September

SFC sergeant first class

shoulder That part of the top surface of an approach embankment, causeway, or cut immediately adjoining the roadway that accommodates stopped vehicles in emergencies and laterally supports base and surface courses.

shoulder slopes These may be the same as the traveled way, but usually they are greater because shoulders are more previous than the surface course.

sight distance restriction factor The percent of the total length of the road on which the sight distance is less than 1,500 feet.

slipperiness The low traction capacity of a thin soil surface owing to its lubrication by water or mud without the occurrence of significant vehicle sinkage.

slope The inclined surface of an excavated cut or an embankment.

slope ratio The relative steepness of the slope expressed as a ratio of horizontal distance to vertical distance. Thus, a 2:1 slope ratio signifies that for every 2 feet horizontally there is a rise or fall of 1 foot. The value of the slope ratio used in construction depends on the properties of the soil and the vertical height of the slope. Ditch slopes may also be governed by the amount of water to be drained and the possibility of erosion.

SM silty sands and poorly graded sand-silt mixture

SOP standing operating procedure

SP poorly graded sand

spring icing An icing whose source of water is from subpermanent levels.

sq square

sq ft square feet

sq in square inch

Sr senior

SS slow setting

SSG staff sergeant

sta station

STANAG Standardization Agreement

stickiness The ability of a soil to adhere to the vehicle undercarriage or running gear.

stilling basin A structure used to protect the culvert outlet against erosion.

subbase or subgrade Describes the in situ soil on which a road, airfield, or heliport is built. The subgrade includes soil to the depth that may affect the structural design of the project or the depth at which climate affects the soil.

subsurface water Water beneath the surface of the land.

superelevation The transverse downward slope from the outside to the inside of the traveled way on a curve. It is usually expressed in inches of drop per horizontal foot or foot-drop per horizontal foot.

surface course The surface course provides a smooth, hard surface on which the traffic moves. It may be constructed from asphalt or tar products, concrete, gravel, or compacted earth with certain types of binders. The surface course should be all-weather and should provide for the rapid runoff of water. The use of treated surfaces is limited to roads that have a long design life. A divisional road with a life expectancy of 6 months or less will receive only an earth or gravel surface.

SUSV small-unit support vehicle

SW southwest

SW well-graded sand

T1 maximum towing force

T2 required towing force

TBM temporary bench mark

TC training circular

temp temperature

time of concentration (TOC) The time it takes for an entire drainage basin to begin contributing runoff to a drainage structure.

TM technical manual

TN air transport

TO theater of operations

TOE table(s) of organization and equipment

TP transition point

traction capacity The ability of soil to resist the vehicle tread thrust required for steering and propulsion.

traffic lane The traffic lane consists of the road surface over which a single lane of traffic will pass,

transpiration The process by which water that has traveled from the ground through the plant's system is returned to the air through the leaf system.

traveled way The road surface upon which all vehicles move or travel. For a single-lane road, the traveled way is the same as one traffic lane. For a multilane road, the traveled way is the sum of the traffic lanes, If a surface course is provided, it normally extends only across the traveled way.

turbulent flow The type of flow that occurs when viscosity forces are relatively weak and the individual water particles move in random patterns within the aggregate forward-flow pattern.

US United States

USAES US Army Engineer School

Uses Unified Soil Classification System

UXO unexploded ordnance

VC vitrified clay

vehicle cone index (VCI) The index assigned to a given vehicle that indicates the minimum soil strength in terms of rating cone index (or cone index for coarse-grained soil) required for one pass (VCI_1) or other passes (VCI) of the vehicle. Usually one and fifty passes are used as extremes.

VMC visual meteorological conditions

Vol volume

W1 weight of a towing vehicle

W2 weight of a towed vehicle

w/ with

w/o without

WF waste factor

wp wetted perimeter

W.R.C. wire rope cable

wt weight

WT weight type

yd yard

yr year

< less than

≤ less than or equal to

> greater than

≥ greater than or equal to

ΔG change of grade

REFERENCES

SOURCES USED

These are the sources quoted or paraphrased in this publication.

Standardization Agreements (STANAGs and QSTAGs)

QSTAG 306, *Fortification for Parked Aircraft* 14 June 1978.
STANAG 2929. *Airfield Damage Repair.* 11 January 1989.
STANAG 3346 AML (Edition 4). *Marking and Lighting of Airfield Obstructions.* 17 October 1988.
STANAG 3601 TN (Edition 3). *Criteria for Selection and Marking of Landing Zones for Fixed Wing Transport Aircraft.* 2 July 1985.
STANAG 3619 AML (Edition 2, Amendment 2). *Helipad Marking.* 7 October 1980.
STANAG 3652 AML (Amendment 3). Helipad *fighting, Visual* Meteorological Conditions. 1 March 1979.
STANAG 3685 AML. *Airfield Portable Marking.* 26 April 1988.

Joint and Multiservice Publications

AR 415-30/AFR 93-10. *Troop Construction and Engineering Support of the Air Force Overseas.* 15 May 1979.
FM 5-430-00-2/AFPAM 32-8013, Vol 2. *Planning and Design of Roads, Airfields, and Heliports in the Theater of Operations:* Volume 2, Airfield and Heliport Design. To be published within six months.
FM 5-530. *Materials Testing.* (NAVFAC MO-330; AFM 89-3.) 17 August 1987.
TM 5-624. *Maintenance and Repair of Surface Areas.* (NAVFAC MO-102: AFR 85-8.) 31 March 1977.
TM 5-822-2. *General Provisions and Geometic Design for Roads, Streets, Walks, and Open Storage Areas.* (NAVFAC DM-5.5; AFM 88-7, Chap 5,) 14 July 1987.
TM 5-822-5, *Pavement Design for Roads, Streets, Walks, and Open* Storage *Areas.* (AFM 88-7, Chap 1.) 12 June 1992,
TM 5-825-2. *Flexible Pavement Designs for Airfields.* (NAVFAC DM-21.3; AFM 88-6.) 1 August 1978.
TM 5-852-7. *Surface Drainage Design for Airfields and Heliports in Arctic and Subarctic Regions* (AFM 88-19, Chap 7.) 15 April 1981,

Air Force Publications

AFR 93-2. *Contingency Response Planning. 19* January 1990.

Army Publications

FM 5-34. *Engineer Field Data.* 14 September 1987,
FM 5-36. *Route Reconnaissance and Classification.* 10 May 1985.
FM 5-233. *Construction Surverying.* 4 January 1985.
FM 5-250. *Explosives and Demolitions.* 15 June 1992.
FM 5-410. *Military Soils Engineering.* 23 December 1992.
FM 5-434. *Earthmoving Operations.* 30 September 1992.

FM 5-446. *Military Nonstandard Fixed Bridging.* 3 June 1991.
MIL-STD-621. *Test Methods for Pavement Subgrade, Subbase, and Base Course Material.* 22 December 1964.
SC-6635 -98-CL-E02-HR. *Hand Receipt Catalog Covering Content of Test Set, Soil.* 19 April 1982.
Special Report 83-27. *Revised Procedures for Pavement Design Under Seasonal Frost* Conditions, US Army Corps of Engineers. Office of the Chief of Engineers, Washington, DC 20314. September 1983.
TM 5-232. *Elements of Surveying.* 1 June 1971.
TM 5-302-Series. *Army Facilities Components System: Design. 28* September 1973.
TM 5-332. *Pits and Quarries.* 15 December 1967.
TM 5-337. *Paving and Surfacing Operations.* 21 February 1966.
TC 5-340. *Air Base Damage Repair (Pavement Repair]. 27* December 1988.

Nonmilitary Publications

ASTM D1557. *Soil and Soil Aggregate Mixture Using 10 Pounds 4.54KJ Rammer and 18-Inch 457* mm Drop, Moisture Density Relation of 27 Apr 78. 1989.
ASTM D1633. *Compression Strength of Molded Soils Cementing Cylinders.* 1990.
ASTM D3786. *Test Method for Hydraulic Bursting Strength of Knitted Goods and Nonwoven* Fabrics: Diaphragm Bursting Strength Tester Method. 1987.
ASTM D4533. *Test Method for Trapezoid Tearing Strength of Geotextiles.* 1991.
ASTM D4632. *Test Method for Breaking Load and Elongation of Geotextiles (Grab Method).* 1991.
ASTM D4833. Test *Method for Index Puncture Resistance of Geotextiles Geomembranes,* and Related Products. 1988.

DOCUMENTS NEEDED

These documents must be available to the intended users of this publication.

DA Form 1248. *Road Reconnaissance Report.* July 1960.
DA Form 1711-R. *Engineer Reconnaissance Report (LRA).* May 1985.
DA Form 2028. *Recommended Changes to Publications and Blank* Forms. February 1974.
DD Form 2641. *Trafficability Test Data.* August 1993

INDEX

A

B

C

D

E

F

G

H

I

L

M

N

O

P

R

S

T

U

V

W

By Order of the Secretary of the Army:

GORDON R. SULLIVAN
General, United States Army
Chief of Staff

Official:

Milton H. Hamilton
MILTON H. HAMILTON
Administrative Assistant to the Secretary of the Army
06955

By Order of the Secretary Air Force:

MERRILL A. McPEAK
General United States Air Force
Chief of Staff

Official:

JAMES E. McCARTHY, Maj General, USAF
The Civil Engineers

DISTRIBUTION:

Active Army, USAR, and ARNG: To be distributed in accordance with DA Form 12-11E, requirements for FM 5-430-00-1, Vol I, *Planning and Design of Roads, Airfields, and Heliports in the Theater of Operations - Road Design* (Qty rqr. block no. 0759).

☆ U.S. GOVERNMENT PRINTING OFFICE:1994-528-027/20039

PIN: 072867-000