TM 3-34.48-1

THEATER OF OPERATIONS: ROADS, AIRFIELDS, AND HELIPORTS—ROAD DESIGN

FEBRUARY 2016

DISTRIBUTION RESTRICTION. Approved for public release; distribution is unlimited.

This publication, along with TM 3-34.48.2, supersedes FM 5-430-00-1/AFJPAM 32-8013, 26 August 1994, and FM 5-430-00-2/AFJPAM 32-8013, 29 September 1994.

HEADQUARTERS, DEPARTMENT OF THE ARMY

This publication is available at Army Knowledge Online at https://armypubs.us.army.mil/doctrine/index.html. To receive publishing updates, please subscribe at http://www.apd.army.mil/AdminPubs/new_subscribe.asp. The United States Marine Corps Web site is <https:/www.doctrine.usmc.mil>. The United States Navy Web sites are <https://ndls.nwdc.navy.mil> and https://ndls.nwdc.navy.smil.mil. The United States Air Force Web site is <www.e-publishing.af.mil>.

Technical Manual
No. 3-34.48-1

Headquarters,
Department of the Army
Washington, D.C., 29 February 2016

Theater of Operations: Roads, Airfields, and Heliports—Road Design

Contents

		Page
	PREFACE	**xi**
	INTRODUCTION	**xii**
Chapter 1	**GENERAL INFORMATION**	**1-1**
	Basic Planning Considerations	1-1
	Airfield Construction	1-1
	Engineering Study	1-2
Chapter 2	**RECONNAISSANCE AND SITE SELECTION**	**2-1**
	Location Factors	2-1
	Reconnaissance Operations	2-5
	Route and Road Reconnaissance	2-10
	Fords, Dips, Causeways, and Bridges	2-12
	Engineer Reconnaissance	2-18
	Airfield Reconnaissance	2-18
Chapter 3	**SURVEYS AND EARTHWORK OPERATIONS**	**3-1**
	Construction Surveys	3-1
	Construction Stakes	3-2
	Earthwork Estimation	3-6
	Mass Diagram	3-20
Chapter 4	**CLEARING, GRUBBING, AND STRIPPING**	**4-1**
	Forest Types and Environmental Conditions	4-1
	Geologic and Permafrost Conditions	4-3
	Preparation	4-3
	Clearing Considerations	4-4

Distribution Restriction: Approved for public release; distribution is unlimited.

*This publication, along with TM 3-34.48.2, supersedes FM 5-430-00-1/AFJPAM 32-8013, 26 August 1994, and FM 5-430-00-2/AFJPAM 32-8013, 29 September 1994.

Performance Techniques 4-6

Chapter 5 SUBGRADES AND BASE COURSES 5-1
Design Considerations 5-1
Subgrades 5-3
Base Course 5-10
Soil Stabilization 5-19

Chapter 6 STORM DRAINAGE DESIGN 6-1
Design Considerations 6-1
Construction 6-1
Hydrograph 6-11
Design Data Requirements 6-13
Design Procedures 6-13
Estimate Runoff Using the Rational Method 6-22

Chapter 7 OPEN-CHANNEL DESIGN 7-1
Factors 7-1
Equations 7-5
Considerations 7-7
Techniques 7-9
Construction and Maintenance 7-17

Chapter 8 CULVERT DESIGN 8-1
Culvert Types and Designs 8-1
Subsurface Drainage 8-36

Chapter 9 SURFACE DRAINAGE DESIGN IN ARCTIC AND SUBARCTIC REGIONS ... 9-1
Arctic and Subarctic Regions 9-1
Design Criteria for Cold Climates 9-1

Chapter 10 SOILS TRAFFICABILITY 10-1
Basic Trafficability Factors 10-1
Instruments and Tests for Trafficability 10-2
Soil Measurements for Trafficability 10-5
Application of Trafficability Procedures in Fine-Grained Soils and Remoldable Sands 10-11
Operation in Coarse Grained Soils 10-28
Trafficability Data 10-30
Soil-Trafficability Classification 10-35

Chapter 11 HELIPORTS MAINTENANCE, REPAIR, AND REHABILITATION OF ROADS, AIRFIELDS, AND 11-1
Maintenance and Repair Considerations 11-1
Maintenance and Repair Operations 11-2
Road Maintenance 11-9
Airfield and Heliport Maintenance 11-17
Rehabilitation of Captured Airfields 11-22

Chapter 12 ROAD DESIGN AND AIRFIELD LAYOUT 12-1
Geometric Design 12-1
Vertical Alignment 12-25
Special Considerations for Roads in Mountainous Terrain 12-40

Construction Methods 12-53
Surface Treatments 12-55
Appendix A METRIC CONVERSION CHART A-1
Appendix B HYDROLOGIC AND HYDRAULIC TABLES AND CURVES B-1
GLOSSARY Glossary-1
REFERENCES References-1
INDEX Index-1

Figures

Figure 2-1. Safe-slope ratio 2-3
Figure 2-2. River action 2-11
Figure 2-3. Stream cross section 2-14
Figure 2-4. Cross section of dip section 2-15
Figure 2-5. Overflow (causeway) 2-16
Figure 2-6. Waterway cross section at bridge site 2-17
Figure 2-7. Sample airfield-siting template 2-20
Figure 2-8. Sample air reconnaissance report 2-22
Figure 2-9. Sample ground reconnaissance report–undeveloped airfield site format 2-24
Figure 2-10. Sample ground reconnaissance report–captured enemy airfield format 2-25
Figure 2-11. Sample air landing area report format 2-26
Figure 2-12. Sample sketch to accompany an airfield reconnaissance report format 2-27
Figure 3-1. Centerline stakes 3-3
Figure 3-2. Marking and placing slope stakes 3-4
Figure 3-3. Marking and placing offset stakes 3-5
Figure 3-4. Typical fill cross section 3-7
Figure 3-5. Cross-section cut with verticals drawn at critical points 3-8
Figure 3-6. Triangle base and height dimension locations 3-8
Figure 3-7. Trapezoid base and height dimension locations 3-9
Figure 3-8. Cross-section cut showing distances and elevations 3-9
Figure 3-9. Fill cross section arranged to show the stripper method 3-10
Figure 3-10. Cross section with the sum of vertical lines added on the stripper 3-11
Figure 3-11. Cross-section area by the double-meridian method 3-11
Figure 3-12. Computation grid system for a borrow pit 3-15
Figure 3-13. Excavation volume for one square 3-16
Figure 3-14. Sample grid-system work sheet 3-17
Figure 3-15. Sample earthwork volume calculation sheet 3-20
Figure 3-16. Plotting the mass diagram 3-21
Figure 3-17. Properties of a mass diagram 3-21
Figure 3-18. Mass diagram with a balance line 3-23

Figure 3-19. Mass diagram with two balance lines 3-23
Figure 3-20. Work in earthmoving operations 3-24
Figure 3-21. Balance lines for equipment efficiency 3-24
Figure 3-22. Determining average grade, Step 1 3-25
Figure 3-23. Determining average grade, Step 2 3-26
Figure 3-24. Minimizing work with two nodes 3-26
Figure 3-25. Minimizing work with an even number of nodes 3-27
Figure 3-26. Minimizing work with an odd number of nodes 3-27
Figure 3-27. Waste and borrow on a mass diagram 3-27
Figure 3-28. Mass diagram showing analysis results 3-28
Figure 3-29. Sample mass-diagram analysis sheet 3-28
Figure 4-1. Bulldozer removing medium-size trees 4-8
Figure 4-2. Four steps for removing large trees with a bulldozer 4-9
Figure 4-3. Counterclockwise method 4-10
Figure 4-4. Clockwise method 4-11
Figure 4-5. Steep-slope method 4-11
Figure 4-6. Level-terrain method 4-12
Figure 4-7. Stump-blasting methods for different root structures 4-14
Figure 5-1. Typical flexible-pavement section 5-2
Figure 5-2. Distribution of pressures under single-wheel loads 5-3
Figure 5-3. Distribution of pressures produced by multiple-wheel assemblies 5-3
Figure 5-4. Recommended compaction requirements for support areas 5-4
Figure 5-5. Distribution of stress in base courses and effects of subgrade strength on base course thickness 5-10
Figure 5-6. Gradation triangle for soil classification 5-24
Figure 6-1. Deep interceptor ditch 6-5
Figure 6-2. Typical storm hyetographs 6-7
Figure 6-3. World isohyetal map 6-9
Figure 6-4. Standard rainfall intensity-duration curves 6-10
Figure 6-5. Typical hydrograph 6-12
Figure 6-6. Typical airfield drainage features 6-14
Figure 6-7. Delineation of roads and airfields 6-15
Figure 6-8. Delineation of superelevated roads 6-16
Figure 6-9. Area delineation by cover or soil type 6-16
Figure 6-10. Delineation of a hill 6-17
Figure 6-11. Delineation of valleys and draws 6-18
Figure 6-12. Delineation of ridges, spurs, and saddles 6-18
Figure 6-13. Area measurement, counting-squares method 6-19
Figure 6-14. Area measurement, geometric-shapes method 6-20
Figure 6-15. Area measurement–stripper method 6-21
Figure 6-16. TOC, regular area 6-25

Figure 6-17. TOC, irregular area 6-26
Figure 6-18. Flow travel time 6-27
Figure 6-19. Delineating runoff area 6-33
Figure 6-20. Determining average slope 6-34
Figure 6-21. Schematic example for successive areas 6-35
Figure 6-22. Example of successive areas 6-39
Figure 7-1. Ditch cross sections 7-2
Figure 7-2. Definition sketch 7-3
Figure 7-3. Nomograph for Manning's equation 7-6
Figure 7-4. Hydraulic radius 7-8
Figure 7-5. Calculating hydraulic radius 7-8
Figure 7-6. Determination of the area of a triangular ditch 7-14
Figure 7-7. Determination of the wetted perimeter of a triangular ditch 7-15
Figure 7-8. Typical runway gutter 7-16
Figure 7-9. Typical berm emplacement 7-17
Figure 7-10. Terrace spacing and gradients 7-20
Figure 7-11. Erosion-control checks 7-21
Figure 7-12. Recommended riprap sizes for open channels 7-22
Figure 7-13. Recommended riprap sizes for culvert outlets 7-23
Figure 7-14. Recommended gradation of stones for riprap 7-24
Figure 7-15. Assembly of a gabion 7-25
Figure 7-16. Assembly and construction of gabions 7-26
Figure 7-17. Repairing a broken wire 7-27
Figure 7-18. Typical channel lining using gabions 7-27
Figure 7-19. Culvert inlet or outlet using gabion headwall and channel lining 7-28
Figure 7-20. Check-dam design 7-31
Figure 7-21. Check-dam maintenance 7-32
Figure 8-1. Oil-drum culvert 8-3
Figure 8-2. Landing-mat and sandbag culvert 8-4
Figure 8-3. Log culverts 8-4
Figure 8-4. Culvert alignment 8-5
Figure 8-5. Ditch relief culverts 8-6
Figure 8-6. Changing the normal grade of a culvert 8-7
Figure 8-7. Culvert specifications for CMP 8-7
Figure 8-8. Elevation of center section of culvert 8-13
Figure 8-9. Cradle footings 8-14
Figure 8-10. Footings for timber culverts 8-14
Figure 8-11. Concrete-culvert installation in rock 8-14
Figure 8-12. Headwalls and wing walls for culverts 8-16
Figure 8-13. Culvert outlets without headwalls 8-16
Figure 8-14. Types of scour at culvert outlets 8-17

Figure 8-15. Outlet erosion 8-17
Figure 8-16. Hydraulic gradient, S, and heads, H, for culverts 8-18
Figure 8-17. Nomograph for culverts flowing full with ponding inlets 8-20
Figure 8-18. Calculating cover 8-23
Figure 8-19. Unsubmerged inlet culvert problem 8-27
Figure 8-20. Calculating cover (sample problem) 8-27
Figure 8-21. Ponding areas 8-29
Figure 8-22. Ponding area enlarged 8-30
Figure 8-23. Cumulative runoff curve 8-32
Figure 8-24. Drop inlets 8-34
Figure 8-25. Grating heads 8-34
Figure 8-26. Typical base drainage installation 8-36
Figure 8-27. Typical subgrade drainage installation 8-38
Figure 8-28. Types of subsurface drainage 8-39
Figure 8-29. Typical intercepting drainage installations 8-40
Figure 8-30. Mechanical analysis curves for filter material 8-42
Figure 9-1. Intensity-duration curves, arctic and subarctic 9-2
Figure 9-2. Design storm index for Alaska 9-2
Figure 10-1. Soil-trafficability test set 10-3
Figure 10-2. Carrying a soil-trafficability test set 10-3
Figure 10-3. Cone penetrometer 10-4
Figure 10-4. Using a cone penetrometer in the upright position 10-4
Figure 10-5. Using a cone penetrometer in the prone position 10-5
Figure 10-6. Trafficability test data form 10-8
Figure 10-7. Relationship between MI and VCI 10-13
Figure 10-8. One-pass performance curves for self-propelled vehicles operating in fine-grained soils or remoldable sands 10-19
Figure 10-9. Fifty-pass performance curves for self-propelled vehicles operating in fine-grained soils or remoldable sands 10-21
Figure 10-10. One-pass performance curves for vehicles towed in level, fine-grained soils or remoldable sands 10-23
Figure 10-11. Fifty-pass performance curves for vehicles towed in level, fine-grained soils or remoldable sands 10-25
Figure 10-12. Photomap with trafficability data 10-31
Figure 10-13. Soil-trafficability classification in USCS terms 10-36
Figure 10-14. Trafficability classification of dry-to-moist, course-grained soils for wheeled vehicles 10-37
Figure 11-1. Check dam 11-3
Figure 11-2. Side-hill terracing 11-4
Figure 11-3. Culvert entrance 11-4
Figure 11-4. Improvised road drag 11-5
Figure 11-5. Road maintenance problems and proper corrective action 11-6

Figure 11-6. Crater backfill materials 11-9
Figure 11-7. Location of snow fences 11-13
Figure 11-8. Snow fence control of road cut drifts 11-13
Figure 11-9. Types of snow fences 11-14
Figure 12-1. Road nomenclature 12-1
Figure 12-2. Road cross section and nomenclature 12-2
Figure 12-3. Interpolation of design hourly volume for selection of road class 12-5
Figure 12-4. Types of horizontal curves 12-7
Figure 12-5. Elements of a simple, horizontal curve 12-8
Figure 12-6. Arc definition for degree of curvature 12-9
Figure 12-7. Chord definition for degree of curvature 12-9
Figure 12-8. Derivation of external distance 12-11
Figure 12-9. Horizontal curve without sharpness restriction 12-13
Figure 12-10. Horizontal curve with restriction on tangent 12-14
Figure 12-11. Horizontal curve with restriction on the external distance 12-16
Figure 12-12. Deflection angle 12-19
Figure 12-13. Deflection angles for 100-foot arcs 12-19
Figure 12-14. Subarc deflection angles 12-20
Figure 12-15. Calculation of chord lengths 12-20
Figure 12-16. Deflection-angle and chord-length determinations 12-21
Figure 12-17. Obstruction on a curve 12-23
Figure 12-18. Horizontal curve design 12-24
Figure 12-19. Elements of a vertical curve 12-27
Figure 12-20. Equal tangent grades 12-29
Figure 12-21. Same sign tangent grades 12-30
Figure 12-22. Unequal tangent grades 12-30
Figure 12-23. Vertical curve design procedures 12-34
Figure 12-24. Normal crown cross section for a Class A road 12-36
Figure 12-25. Superelevated cross section 12-38
Figure 12-26. Method for transition (not to scale) 12-39
Figure 12-27. Sliver bench cut 12-40
Figure 12-28. Full bench cut 12-41
Figure 12-29. Fill-side retaining wall 12-41
Figure 12-30. Earth embankment with riprap keyway 12-42
Figure 12-31. Placed riprap with riprap keyway 12-43
Figure 12-32. Keyed riprap with rock dowels 12-43
Figure 12-33. Grouted riprap with rock dowels 12-44
Figure 12-34. Rock dowel 12-44
Figure 12-35. Petroleum-asphalt flowchart 12-47
Figure 12-36. Tar simplified flow chart 12-48
Figure 12-37. Identification of unknown bituminous materials 12-50

Figure 12-38. Tack-coat sequence of operations 12-54
Figure 12-39. Single-surface treatment 12-57
Figure 12-40. Multiple-surface treatment 12-59
Figure 12-41. Typical hopper-type aggregate spreader 12-61
Figure 12-42. Spray bar coverage 12-62
Figure B-1. Rates of overland flow corresponding to standard supply curves, supply curve number 0.4 and 0.6; n=0.40, S=1 percent B-2
Figure B-2. Rates of overland flow corresponding to standard supply curves, supply curve numbers 0.8 and 1.0; n = 0.40, S = 1 percent B-3
Figure B-3. Rates of overland corresponding to standard supply curves, supply curve numbers 1.2 and 1.4; n = 0.40, S = 1 percent B-4
Figure B-4. Rates of overland flow corresponding to standard supply curves, supply curve numbers 1.6 and 1.8; n = 0.40, S = 1 percent B-5
Figure B-5. Rates of overland flow corresponding to standard supply curves, supply curve numbers 2.0 and 2.2; n = 2.0 and 2.2; n = 0.40; S = 1 percent B-6
Figure B-6. Rates of overland flow corresponding to standard supply curves, supply curve number 2.4 and 2.6; n = 0.40, S = 1 percent B-7
Figure B-7. Rates of overland flow corresponding to standard supply curves, supply curve numbers 2.8 and 3.0; n = 0.40, S = 1 percent B-8

Tables

Table 2-1. Typical airfield ground-reconnaissance party 2-9
Table 2-2. Suggested equipment list for airfield ground-reconnaissance party 2-10
Table 2-3. Requirements for military fords 2-12
Table 3-1. Earthwork average cut or fill 3-14
Table 3-2. Soil conversion factors 3-18
Table 4-1. Species of trees and their normal root systems 4-2
Table 4-2. Applications and limitations of engineer equipment in land clearing 4-7
Table 5-1. Recommended maximum permissible values of gradation and Atterberg limit requirements in subbase and select materials for roads and airfields 5-7
Table 5-2. Frost-design soil classification 5-8
Table 5-3. Generally suitable base course materials 5-11
Table 5-4. Desirable gradation for crushed rock or slag, and uncrushed sand and gravel aggregates for nonmacadam base courses 5-11
Table 5-5. Desirable gradation for crushed rock, gravel, or slag aggregates for macadam base courses 5-11
Table 5-6. Assigned CBR ratings for base-course materials 5-12
Table 5-7. Traffic type and gyratory compaction requirements 5-17
Table 5-8. Estimated time required for test procedures 5-23
Table 5-9. Soil stabilizers for different soil types 5-25
Table 5-10. Suggested stabilizer amounts for expedient stabilization 5-26
Table 6-1. Example critical duration calculation 6-11
Table 6-2. Runoff coefficients 6-23

Table 6-3. Examples of travel time estimates 6-28
Table 6-4. Estimating flow velocity 6-28
Table 6-5. Tabular solution to determine TOC 6-34
Table 6-6. Determining travel time 6-39
Table 7-1. Recommended requirements for slope ratios in cuts and fills—homogeneous soils 7-3
Table 7-2. Recommended requirements for slope ratios in cuts—rock with bedding or other planes of weakness 7-4
Table 7-3. Manning's *n* values and maximum permissible velocities of flow in open channels 7-7
Table 7-4. Sample ditch design work sheet example 7-14
Table 7-5. Suggested maximum velocities 7-18
Table 7-6. Recommended side slopes 7-19
Table 7-7. Requirements for gabion protection 7-28
Table 7-8. Discharge for weir notches in check dams 7-32
Table 8-1. Sizes of CMP 8-2
Table 8-2. Characteristics of United States aircraft 8-9
Table 8-3. Minimum cover requirements, in feet, for airfields (CMP) 8-10
Table 8-4. Minimum cover requirements, in feet, for airfields (reinforced concrete pipe) 8-11
Table 8-5. Maximum permissible cover for CMP 8-12
Table 8-6. Maximum permissible cover for corrugated aluminum-alloy pipe 8-13
Table 8-7. Capacity and critical slope of culverts 8-19
Table 8-8. Unsubmerged inlet sample data 8-28
Table 8-9. Cumulative runoff data tabulation, rational method 8-31
Table 8-10. Discharge capacity of square grate inlets 8-35
Table 8-11. Base drainage criteria for K 8-37
Table 8-12. Drainage characteristics of soil 8-37
Table 8-13. Tabulated data 8-45
Table 8-14. Filter fabric criteria 8-46
Table 10-1. Critical-layer depth variations 10-6
Table 10-2. Examples of soil strength profiles 10-9
Table 10-3. VCI for tracked vehicles (MI versus VCI for one and 50 passes) 10-14
Table 10-4. VCI for wheeled vehicles (MI versus VCI for one and 50 passes) 10-16
Table 10-5. Trafficability symbols 10-33
Table 11-1. Proprietary materials for patching partial-depth spalls in concrete 11-17
Table 11-2. Characteristics of grass 11-19
Table 12-1. Geometric design data for military roads 12-3
Table 12-2. Deflection angles and chord distances 12-22
Table 12-3. Staking intervals (in feet) 12-23
Table 12-4. Staking intervals (in meters) 12-24
Table 12-5. Compiling vertical curve design data 12-34

Table 12-6. Desirable and maximum slopes....12-36
Table 12-7. Superelevation lengths and transition lengths....12-37
Table 12-8. Pavement widening 12-39
Table 12-9. Maximum rock cut slopes 12-45
Table 12-10. Use of bituminous materials 12-46
Table A-1. Metric conversion chart A-1
Table B-1. Hydraulic radius and area of symmetrical triangular channels B-9
Table B-2. Hydraulic radius and area of nonsymmetrical triangular channels B-10
Table B-3. Hydraulic radius and area of symmetrical trapezoidal channels (2-foot bottom width) B-11
Table B-4. Hydraulic radius and area of symmetrical trapezoidal channels (4-foot bottom width) B-12
Table B-5. Hydraulic radius and area of symmetrical trapezoidal channels (6-foot bottom width) B-13
Table B-6. Hydraulic radius and area of symmetrical trapezoidal channels (8-foot bottom width) B-14
Table B-7. Hydraulic radius and area of symmetrical trapezoidal channels (10-foot bottom width) B-15
Table B-8. Hydraulic radius and area of nonsymmetrical trapezoidal channels (2-foot bottom width) B-16
Table B-9. Hydraulic radius and area of nonsymmetrical trapezoidal channels (4-foot bottom width) B-17

Preface

TM 3-34.48-1 provides general guidance for materials and soils testing.

The principal audience for TM 3-34.48-1 is the United States Army, United States Marine Corps, United States Navy, and the United States Air Force.

Commanders, staffs, and subordinates ensure their decisions and actions comply with applicable United States, international, and, in some cases, host-nation laws and regulations. Commanders at all levels ensure their Soldiers operate in accordance with the law of war and the rules of engagement (see FM 27-10).

TM 3-34.48-1 uses joint terms where applicable. Selected joint and Army terms and definitions appear in both the glossary and the text. Terms for which TM 3-34.48-1 is the proponent (the authority) are marked with an asterisk (*) in the glossary. Definitions for which TM 3-34.48-1 is the proponent publication are boldfaced in the text. For other definitions shown in the text, the term is italicized and the number of the proponent publication follows the definition.

TM 3-34.48-1 applies to the Active Army, Army National Guard/Army National Guard of the United States, and United States Army Reserve unless otherwise stated. This manual is adopted for use by the United States Marine Corps, United States Navy, and the United States Air Force personnel.

The proponent of TM 3-34.48-1 is the United States Army Engineer School. The preparing agency is the Maneuver Support Center of Excellence (MSCoE) Capabilities Development and Integration Directorate (CDID); Concepts, Organizations, and Doctrine Development Division (CODDD); Doctrine Branch. Send comments and recommendations on DA Form 2028 (Recommended Changes to Publications and Blank Forms) to Commander, MSCoE, ATTN: ATZT-CDC, 14000 MSCoE Loop, Suite 270, Fort Leonard Wood, MO 65473-8929; by e-mail to usarmy.leonardwood.mscoe.mbx.cdidcodddengdoc@mail.mil; or submit an electronic DA Form 2028.

Send other military branch comments and recommendations to—

- **Marine Corps**
  Deputy Commandant for Combat Development and Integration
  ATTN: MCCDC CDD MID DCB C116
  3300 Russell Road, Suite 204
  Quantico, VA 22134-5021
  Defense Switched Network (DSN) 278-6233; Commercial (703) 784-6233
  Web site: <https://www.doctrine.usmc.mil/>
- **United States Navy**
  Commander
  Navy Warfare Development Command
  ATTN: N5-N52
  1530 Piersey Street, Building 0-27
  Norfolk, VA 23511-2723
  DSN 341-4185; Commercial (757) 341-4185
  Web site: <https://atsapubs@nwdc.navy.mil/>
- **United States Air Force**
  Headquarters Air Force Civil Engineer Support Agency
  ATTN: CEXR
  139 Barnes Drive, Suite 1
  Tyndall Air Force Base, FL 32403-5319
  DSN 523-6173; Commercial (850) 283-6173
  Web site: <http://afpubs.hq.af.mil/>

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

Introduction

The United States Army Engineer School developed this publication with the joint participation of the approving Service commands. The test procedures and terminology used in this manual conform to the latest methods and specifications of the American Society for Testing and Materials (ASTM), the American Concrete Institute, and the Portland Cement Association. The ASTM source publications are not required for testing specified in this document as the procedure is fully explained for military personnel use.

This manual is adopted for use by United States Navy and United States Air Force personnel. Certain tests and procedures prescribed differ in principle or method and are more detailed than counterpart tests that are currently required by the United States Navy for new construction at Navy installations (including those in forward areas). The tests in this manual also apply to arctic construction. However, cold-weather effects present different problems and additional tests will be required for correct evaluation of the materials. These additional tests and considerations associated with arctic construction are in TM 5-349.

TM 3-34.48-1 and TM 3-34.48-2 should be used in conjunction with each other.

Appendix A contains a metric conversion chart for this manual. A complete listing of preferred metric units for general use is contained in Federal Standard 376B at <http://www.usaid.gov/policy/ads/300/fstd376b.pdf>.

Chapter 1

General Information

Military engineers are called on to plan, design, and construct airfields, heliports, and roads in joint areas of operations (JOA). In a high operational tempo environment, the majority of pavement engineering efforts focus on maintenance, repair, and rehabilitation of existing facilities. To ensure that these facilities meet proposed requirements, the responsible engineer officer must coordinate closely with appropriate ground and air commanders. The engineer relies on information from commanders and the operations plan on the weight and traffic frequency of using air or ground vehicles, facility life, geographic boundaries governing site selection, and the time available for construction. Detailed planning, reconnaissance, and site investigations are often limited by the lack of time and the tactical situation. However, when time and security permit, the engineer should conduct normal ground reconnaissance and on-site investigations. It is important for the engineer to obtain the best site information available before entering the design or site selection phases. This manual provides guidance on current military pavement engineering practices relative to the JOA. Reference publications are provided for additional information beyond the scope of this manual.

BASIC PLANNING CONSIDERATIONS

1-1. Military engineers in the JOA should follow the following guidelines:

- Keep designs simple. Simple designs require minimum skilled labor and specialized materials.
- Use local materials when possible. This eliminates construction delays associated with a long communications and logistics line.
- Use existing facilities when possible. This avoids unnecessary construction.
- Remember that safety factors in design are drastically reduced in the JOA because of time constraints and the inherent risks of war.
- Design for initial or temporary structures. Initial design life is for up to six months, and temporary design life is for up to two years.
- Phase construction to permit early use of the facility while construction and improvements continue.
- Avoid sites with dense brush, timberland, and rolling terrain that require heavy clearing or grading.
- Prevent destruction of natural drainage channels, culverts, and roads. Repairs generally require time and labor far exceeding those needed to prevent damage.

AIRFIELD CONSTRUCTION

1-2. The planning and construction of Air Force bases in the JOA is a joint responsibility of Army and Air Force personnel. A summary of Air Force and Army responsibilities follow.

AIR FORCE RESPONSIBILITIES

1-3. The Air Force provides the following support:

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

ARMY RESPONSIBILITIES

1-4. The Army provides the following troop construction support for the Air Force:

- Emergency repair assistance for war damaged air bases that exceed the Air Force organic capability.
- 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.
- 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.

1-5. 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

1-6. After the specific requirements for roads, airfields, and heliports have been determined, engineers should prepare the facilities for use as soon as possible. In most cases, the need is critical because mission accomplishment depends on using certain roads and airfields. 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.

Chapter 2

Reconnaissance and Site Selection

This chapter outlines the location, layout, and design of military roads and airfields. Before constructing a road or airfield, the best facility location is determined and the essential areas and construction features are formulated. Throughout the preconstruction phase, problems can be avoided by a well-planned site selection.

LOCATION FACTORS

2-1. Construction of a road or airfield initially consists of providing a prepared subgrade and base course according to the 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 use the following factors to locate and lay out construction projects:

- Mission.
- Existing facilities.
- Location and design.
- Soil characteristics.
- Drainage.
- Geology.
- Topography.
- Earthwork.
- Alignment.
- Obstacle crossings.
- Bridge approaches.
- Ground cover.
- Required areas.
- Material and utility accessibility.
- Marginal material.
- Flight-way obstructions.
- Sunlit slopes.
- Tactical considerations.
- Future expansion.

MISSION

2-2. Ensuring that a site will fulfill mission requirements is the most important factor in site selection. Lines of communication must be built to accomplish a specific mission in the most direct and efficient manner possible. Each location factor must be evaluated to support the mission.

EXISTING FACILITIES

2-3. Use existing facilities. Wartime missions of engineer troops are extensive, and new construction should be avoided. Extensive road nets of varying quality and capacity already exist in most areas of the world. Where possible, use these road nets to the fullest extent. In many cases, expansion and rehabilitation of existing facilities are adequate for mission accomplishment.

2-4. Except in highly developed areas, existing airfields are seldom adequate to handle modern, high-performance aircraft. However, these airfields can usually be made adequate with minimum rehabilitation. Rapid parking expansion can improve the maximum on-ground capability of existing airfields that are structurally capable of handling high-performance aircraft. It may serve as a 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

2-5. To the greatest extent possible, the location and design for a facility must provide the best response to requirements. Alternative road and airfield plans can be evaluated from the standpoint of total earthwork and drainage structure requirements to reduce construction effort.

2-6. Construct airfields in areas that 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 when locating roads. A road segment to be constructed or improved should be evaluated for its contribution to the overall network. Similarly, an airfield should be evaluated for its ability to enhance an airfield network.

SOIL CHARACTERISTICS

2-7. Locate roads and airfields on terrain with the best possible subgrade soil conditions. This will decrease construction effort and result in a better facility. The subgrade should be compacted under conditions that allow it to support the design loads. Conduct a basic soils investigation before construction to provide the data needed to ensure good construction decisions. Refer to TM 3-34.64/MCRP 3-17.7G for soils information and TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034 for soil survey procedures.

DRAINAGE

2-8. 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 construction, 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 chapters 6 through 9.

2-9. Avoid valley low points and 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. If possible, avoid construction on unprotected floodplains and alluvial fans due to flood hazard. Alluvial terraces are often ideal locations for airfields. They offer flat expanses that are above the river floodplain and are normally protected from flooding.

2-10. Avoid constructing facilities in high water table areas. 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

2-11. Before locating lines of communication, carefully analyze the geology of the area. Sizeable quantities of rock along a construction project may cause a large removal problem, slow construction, and increased construction effort. Engineer troop units require special equipment and training to excavate rock.

2-12. Rocky outcrops are more common in hilly terrain than in flat or rolling country. In areas where the preliminary design indicate that cutting is required to reach final grade, take enough borings or dig enough test pits to determine the location of the rock.

2-13. 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. It is best to align road cuts perpendicular to the strike in sedimentary rocks. If this is not possible, use safe-slope ratios. (See figure 2-1.)

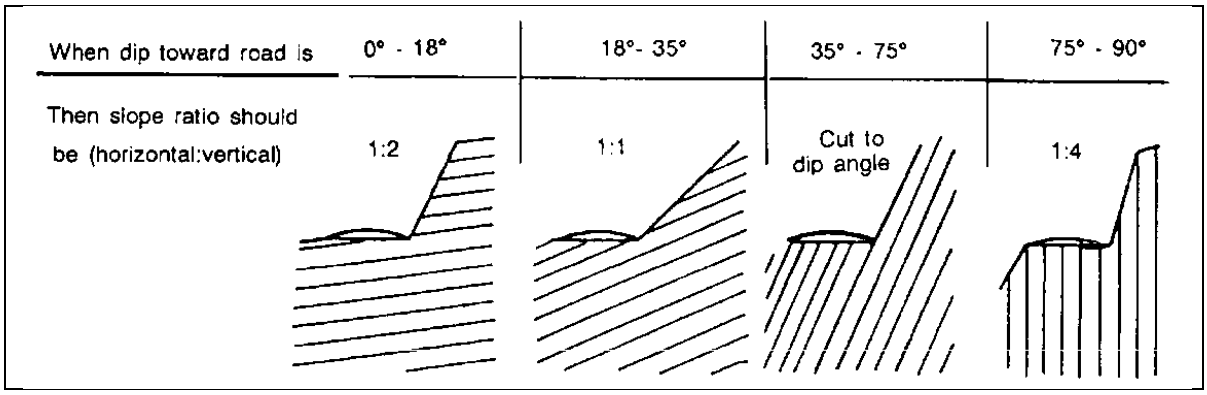


Figure 2-1. Safe-slope ratio

TOPOGRAPHY

2-14. Construct roads and airfields within maximum grade specifications. The specifications depend on the facility construction standard. 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.

EARTHWORK

2-15. The largest single work item during lines of communication construction is earthwork operations. Any step that simplifies earthwork operations will decrease the required work and increase job efficiency. When cutting and filling on a project, using the material excavated to construct required embankments will generally reduce earth handling. This balancing must be within the haul capabilities of available equipment. Even though earthwork should be balanced throughout a project, it may be more practical to open a nearby borrow pit or establish spoil areas if the haul distance is excessive. Balancing cannot be done where the excavated material is not acceptable for use in an embankment.

ALIGNMENT

2-16. Keep the number of curves and grades at a minimum for efficient traffic flow. Vehicles have difficulty in negotiating sharp curves; even gentle curves decrease traffic capacity. Lay 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 efficient location principles (minimizing earthwork, avoiding excessive grades, obtaining suitable fill material). Align runways in the direction of the prevailing wind to optimize aircraft operations.

OBSTACLE CROSSINGS

2-17. When a route crosses a major obstacle (river, ravine, canal), bridges or crossing structures are required. Construction is time consuming and requires materials that may be in short supply. Avoid these obstacles when possible by forgoing other location principles to decrease the number of obstacle crossings or using existing structures to decrease total work requirements. This may require the strengthening of an existing bridge or no bridging work. When possible, the road should not cross a particular obstacle, such as a winding river, more than once.

BRIDGE APPROACHES

2-18. When locating routes, carefully evaluate construction requirements for approaches to obstacles. Construction of approaches over marshes or floodplain areas can cause greater concerns than the obstacle crossing itself. Approach conditions may be the prime factor in obstacle crossing and may dictate route location. Consider the approach and the obstacle when establishing the optimum route.

GROUND COVER

2-19. Routes should avoid heavily wooded areas that require extensive clearing. If this is not possible, the route should pass through areas having the least vegetation. Strip unsuitable material before beginning earthwork.

REQUIRED AREAS

2-20. Airfields need large areas of relatively flat land to efficiently accomplish the 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. Consider the ability to construct this connecting network to appropriate specifications during planning.

2-21. 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.

MATERIAL AND UTILITY ACCESSIBILITY

2-22. The efficient operation of airfields requires the use of electricity, water, gas, and sewer systems. Locate new airfields near existing utility systems to avoid construction of new facilities or long transmission lines. A nearby railhead will help the construction effort.

2-23. Consider the quality and availability of construction materials when locating a facility. Obtain suitable base-course materials from existing pits and quarries when possible. 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

2-24. When planning the location of a project, consider using nearby marginal materials. Using marginal materials for subbase and base construction and as an aggregate in pavements is sometimes made possible with geotextiles, mechanical stabilization, or admixtures. Often, the use of marginal material is unavoidable. When possible, poor-quality material should be excavated and replaced with more suitable material or the project should be relocated. When marginal materials must be used, guidance is provided in UFC 3-220-04FA.

FLIGHT-WAY OBSTRUCTIONS

2-25. The safe operation of fixed- or rotary-wing aircraft requires the removal of obstacles above the lines specified by design criteria. These criteria depend on 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 support 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. Avoid locations that require extensive work to achieve the necessary glide angle. More information regard variables and requirements is provided in UFC 3-260-01, UFC 3-260-02, and UFC 3-260-03.

2-26. A similar clearance is required on runway sides. The specified width must be cleared of obstacles and graded according to specifications.

SUNLIT SLOPES

2-27. If 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 and airfields. Prevailing winds will carry snow, rain, and sand onto the roadway if the orientation of the road is undesirable. Protective snow or sand fences should be oriented to account for prevailing winds.

TACTICAL CONSIDERATIONS

2-28. 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:

- Locate roads in a defilade position on the reverse side of a hill or ravine to avoid enemy observation and provide cover from direct artillery or mortar fire.
- Take advantage of natural camouflage and concealment when constructing a road or airfield in an exposed area.
- Disperse facilities to minimize bombing effects or strafing attacks when designing an airfield. Airfields in forward areas are prime targets for enemy air and ground attacks. It may be necessary to use ground troops in defensive positions against enemy ground action.

FUTURE EXPANSION

2-29. 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 current maneuvers may be inadequate for future operations. Airfields built for small aircraft with a limited evacuation mission may require modification to meet stringent design criteria for high-performance aircraft. Improvement and expansion are continuing jobs for military construction.

2-30. Avoid constructing 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 OPERATIONS

2-31. Reconnaissance operations vary with the operational environment; assigned mission; and size, type, and composition of the reconnaissance element. An aerial map or ground reconnaissance is necessary to determine the best existing or possible location for a future road or airfield.

MISSION

2-32. The primary mission of a reconnaissance party is to find a site that meets most of the 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. Final construction plans and schedules are made with regard to the tactical and logistical situation and the construction time available.

2-33. When the reconnaissance mission is complete, a reconnaissance report serves as the basis for tactical plans and construction schedules. The reconnaissance report is submitted by the personnel that conducted the investigation. It must be complete, comprehensive, and sufficiently detailed to permit careful analysis.

RECONNAISSANCE-PARTY CAPABILITIES

2-34. 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 other engineer reconnaissance. Even a qualified civil engineer with civilian or military experience requires special training for this activity. It would be unusual for one person to be proficient in every task that a thorough reconnaissance must include;

therefore, the assignment of personnel must provide overall efficiency as a unit. The party must be selected with regard to the conditions the unit may encounter.

2-35. Conditions 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 equipment assigned to the party. The equipment should include the 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.

RECONNAISSANCE STEPS

2-36. Reconnaissance steps include planning, briefing, a preliminary study, air reconnaissance, and ground reconnaissance.

Planning

2-37. Planning forms the reconnaissance mission. It involves the coordination of reconnaissance efforts by appropriate headquarters, the estimation of needs, and the assignment of a reconnaissance mission. 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 headquarters to achieve proper coordination. Improper coordination results in duplication of effort in some areas and inadequate reconnaissance in other areas.

Briefing

2-38. 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 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 photographs should be used in the briefing.

2-39. 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 required or the exact location of the site or facility to be investigated for a specific reconnaissance.
- The nature of the proposed facility; the types of vehicles or aircraft expected to use it; the length of time use is anticipated; and the minimum requirements for dimensions, grades, and clearances. These items are covered by the applicable standard layout and specifications published by the joint force commander in the theater. The reconnaissance officer may be familiar with the items, but they should be briefed for reference.
- The anticipated vehicle traffic, 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.

2-40. When the reconnaissance party will be away from the parent unit for a lengthy, continued reconnaissance, additional instructions must be covered in the briefing. The reconnaissance party is briefed on locations for drawing rations, clothing, and equipment replacements; drawing petroleum, oil, and lubricant supplies; and service facilities for vehicle maintenance. In addition, they are briefed on the type of communication equipment (radio, messenger, television satellite) to arrange.

2-41. When ground reconnaissance is ordered ahead of forward ground-force elements, the following additional information is necessary:

- The friendly-force situation.
- The known enemy-force situation.
- The location of adjacent friendly units.

2-42. The following information is applicable for parties engaged in air reconnaissance:

- The instructions for alternative and emergency-landing.
- The location of available aviation petroleum supplies.
- The location of the forward flying line.

Preliminary Study

2-43. A preliminary study consists of studying information obtained during the briefing, conducting a map reconnaissance of the area, studying aerial photographs, delineating soil boundaries, assembling other available preliminary information, and planning and preparing for the actual reconnaissance.

2-44. Information sources that may be useful in preliminary planning and the preliminary study of a specific mission must be verified by ground reconnaissance. These information sources include—

- 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.
- The Office of the Chief of Engineers and subordinate agencies prepare strategic and technical reports, studies, and summaries on areas of actual or potential military importance. These reports provide the best available data at the time of publication. Topographic, geologic, and soil maps and data on climate and groundwater tables are usually included. The reports may contain information on water supply, construction materials, vegetation, and special physical phenomena.
- Army and Air Force tactical intelligence reports are important, reliable sources of information. Intelligence reports are usually prepared in the interior zone but tactical intelligence reports are field-prepared reports of all-around force elements. They may include facts learned by obtained from detainee interrogations, tactical data, reports, records, and interaction with the local population. Intelligence reports are used to prepare strategic and technical reports.
- Road, topographic, soil, vegetation, and geologic maps published by friendly or enemy governments 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.
- Topographic maps should be prepared from aerial photographs if time and facilities are available.
- 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 that 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

2-45. Air reconnaissance involves a general study of the topography, drainage, and vegetation of an area. Construction problems, camouflage possibilities, and access routes should be visualized. Usually the specific ground-reconnaissance procedure is planned by selecting areas that need investigated from the air and determining what questions need answered. Air reconnaissance provides valuable negative information by eliminating unsuitable sites, but it cannot be solely relied on for positive information.

Ground Reconnaissance

2-46. While air reconnaissance can effectively reduce the amount of ground reconnaissance, it cannot replace ground reconnaissance. Questions that are tentatively answered from the air are verified on the ground. Often, ground and air reconnaissance are not separate missions. A continuing air reconnaissance may be interspersed with specific ground reconnaissance.

REPORTING

2-47. The reconnaissance party must always submit the report on time. Reports are submitted for sites investigated, even if the reconnaissance party considers the site unsuitable.

2-48. 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 secure telecommunications. 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 that have been reconnoitered by different parties. This simplifies the reporting process.

2-49. Military roads and road networks are classified according to their location and use. They are classified according to width, surface, and obstructions.

Notes.

1. Terms and formulas approved by the member nations of the North Atlantic Treaty Organization, the Southeast Asia Treaty Organization, the United States, the United Kingdom, the Canadian and Australian Armies Nonmaterial Standardization Program, and other treaty nations are covered in FM 3-34.170/MCWP 3-17.4.

2. Abbreviations, symbols, and notations used in route reconnaissance may also be used in airfield reconnaissance. (See FM 3-34.170/MCWP 3-17.4.) Information given in road reconnaissance reports is useful in reporting on access roads to airfield and heliport sites.

Air Reconnaissance

2-50. An air-reconnaissance team generally consists of two members—the pilot and the engineer observer. Having the officer in charge of the ground-reconnaissance party serve as the engineer observer for the air-reconnaissance party is advantageous and should be arranged when possible. Time is saved and errors of omission are minimized because a report from the engineer observer to the officer in charge of the ground-reconnaissance party is unnecessary except as a matter of record. The pilot can also assess the site and make the appropriate recommendations.

2-51. 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 information on—

- Terrain features.
- Obstacle descriptions.
- Lines of communication evaluation.
- Area suitability for various construction types.
- Water sources.
- Construction materials in the area of operations.
- Cover and concealment.

Ground Reconnaissance

2-52. The composition of the ground-reconnaissance party depends on the scope and extent of the mission, nature of the terrain it must traverse, probability of enemy contact, attitude of the civilian population, and prevalence of mines in the area to be reconnoitered. See table 2-1 for a list of personnel suitable for an airfield reconnaissance.

Note. The list in table 2-1 can be modified to meet the particular needs of the situation.

Table 2-1. Typical airfield ground-reconnaissance party

	Grade	***Primary Duty***	***Secondary Duty***
1	Officer	Command party	General reconnaissance
2	Senior NCO	Second in command	Not applicable
3	EM	Technical engineer	Machine gunner
4	EM	Plane-table man	Machine gunner
5	EM	Terrain intelligence analyst	Rodman
6	EM	Geospatial Engineer	Geospatial Intelligence Analyst
7	EM	Driver/RTO	Assistant machine gunner
8	EM	Driver/RTO	Wheel-vehicle mechanic
Legend:			
EM		enlisted member	
NCO		noncommissioned officer	
RTO		radiotelephone operator	

2-53. Personnel involved should be trained in ground reconnaissance. It is important that the person in charge and the assistant be well versed in the aspects of reconnaissance.

2-54. 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.

2-55. Map and air studies are not substitutes for ground reconnaissance; they reduce the amount of ground effort required. Ground reconnaissance should determine the—

- Estimated grades to be encountered.
- Estimated amount of clearing involved, to include trees, tree stumps, and boulders. Sometimes objects, such as buildings and concrete foundations, are included.
- Estimated amount of debris that will be generated during clearing operations. In some cases, the trees that are removed may be used in the construction operation. (Clearing operations are discussed in chapter 4 of this manual.)

- Nature of the soil encountered, field determination of gradation, percent of fine-gradient materials, and plasticity characteristics.
- Stream conditions at the crossing sites (width, depth, and velocity of the stream; bank conditions; and high water level indications).
- Presence or absence of local construction materials. This includes possible sources of sand, gravel, cement, tar, asphalt culvert pipe, and lumber; construction capabilities; and labor conditions.
- Estimated amount of earthwork necessary, approximate balance between cut and fill, and necessity for long hauls of earth material.
- Errors or discrepancies on maps of the site and the effects of these errors on the selection.
- Local rainfall data and other pertinent information about the seasons and the weather obtained through local inhabitants or other sources.
- Information or observations that affect the final facility location.
- Relationship with the local population.

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	TM 3-34.85/MCRP 3-17A	1
MOPP gear	As required	Reconnaissance report forms and formats	As required
Radio set, mounted in truck	1	Adequate map and aerial photograph coverage	As required
Flashlight	4	Tracing tape (tape, textile)	As required
Camera	1	Lensatic compass	2
*Desirable when operating in support of armored forces or in northern areas.			
Legend:			
mm	millimeter		
MOPP	mission-oriented protective posture		

ROUTE AND ROAD RECONNAISSANCE

2-56. 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 lines of communication 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

2-57. 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.

2-58. 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.

2-59. 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 include a reconnaissance report that provides a permanent record and ensures that enough detail is recorded. The use of these forms is explained in FM 3-34.170/MCWP 3-17.4.

ROAD RECONNAISSANCE

2-60. Road reconnaissance is conducted to determine the traffic capabilities of existing roads and to provide more detailed information than 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 (Road Reconnaissance Report) is used to record this information. (See figure 2-2.) Maps, overlays, and sketches are used as needed.

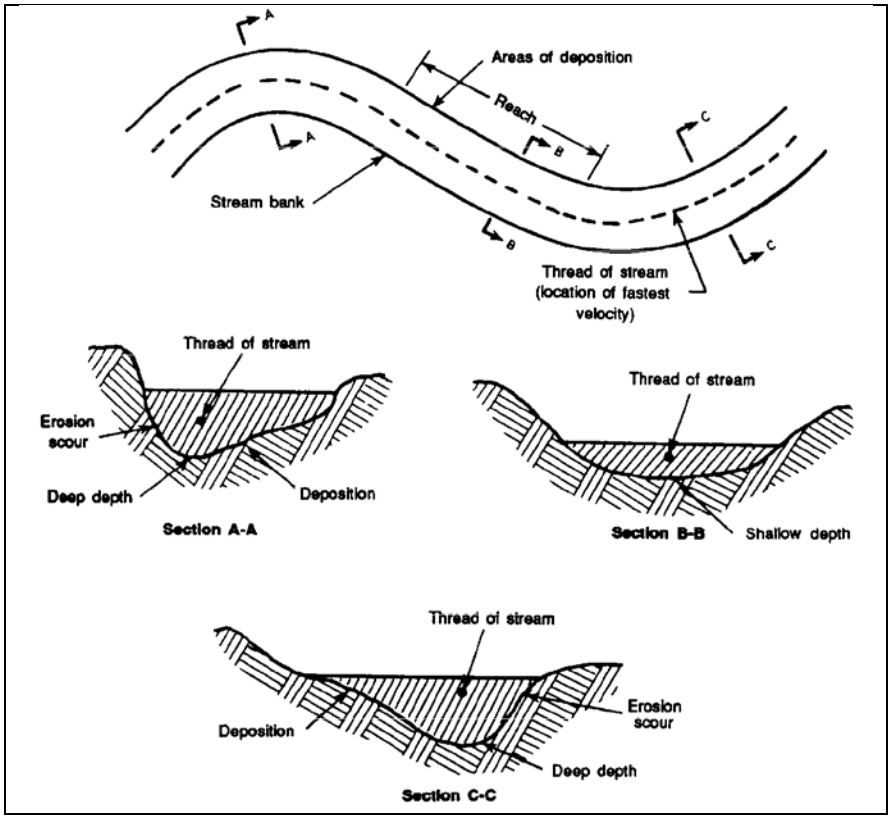


Figure 2-2. River action

2-61. The most important factor in planning military roads is making maximum use of the existing road net. Subject to the requirements of the tactical plan, the existing road net 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. Improving an existing road rather than constructing a new one saves time.

2-62. 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 are based on periodic reconnaissance and must be coordinated with the agencies using the roads to ensure proper standards of maintenance and to avoid work on roads that are no longer needed.

LOCATION RECONNAISSANCE

2-63. When a new road is necessary, the first step is location reconnaissance. This requires reconnaissance of 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.

FORDS, DIPS, CAUSEWAYS, AND BRIDGES

2-64. Gap crossings are vital to road networks and can absorb a significant proportion of the cost of both construction and maintenance. It is important that these structures are appropriate to the category and volume of the traffic carried. The success or life of these structures depends very much on the hydraulic design. Fords and bed-level causeways, like conventional bridges, may be constructed so that they cause little interference with the design flood.

FORDS

2-65. A ford is a shallow place in a waterway where the bottom, 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. An increase in depth can be sudden and endanger personnel or vehicles in the ford. Stream bottoms can be composed of material that requires a lot of effort to make fords usable.

Reconnaissance

2-66. 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 3-34.170/MCWP 3-17.4.

Requirements

2-67. 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. See table 2-3 for width, depth, and bank slope requirements for fords.

Table 2-3. Requirements for military fords

	Maximum depth (feet)	***Maximum width (one-way traffic) (feet)***	***Type of bottom***	***Maximum allowable slope on approaches****
Infantry	3 1/2	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 the hard, dry surface. If wet and slippery, the slope must be less.				

2-68. 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 a slight channel in the center. The influence of river action on possible fording locations is shown in figure 2-2, page 2-11. 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 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.

2-69. For a natural ford, the bottom material should be hard, durable, and interlocking. These materials resist erosion and cutting by wheeled traffic. The general terrain determines bottom conditions as follows:

- **Mountainous terrain.** 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 where the widening of the stream reduces the velocity. It may be necessary to remove this material before traffic can cross the ford.
- **Moderate or gentle slope terrain.** In terrain of moderate or gentle slopes, the velocity will prevent the deposition of fine material. This tendency and the scouring action of water will leave a good, firm bottom. The bottom material may be disturbed by traffic and will require protection.
- **Slightly sloping or flat terrain.** In slightly sloping or flat terrain, streams meander and have a low velocity. Sometimes there is a high water table. In such cases, the bottom material may be very soft. This requires a completely improved ford with special emphasis on bottom requirements. Timber, lumber, matting, gravel, or gabions can be used to improve trafficability.

2-70. A maximum water depth of 2 feet is allowable for truck traffic. (See table 2-3.) To ensure maximum use of the ford, determine the depth that the stream will flow at frequently recurring times. Estimates of various depths of flow 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 on the quantity of the ground water.
- **High water flow.** During the year, many rainfalls normally occur that cause flows above the base level. The velocity generates slight erosion cut. (See figure 2-3, 2-14) The occurrence of such flows prevents vegetation growth. 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 that the depth is greater in places than the fording depth of trucks, it may be necessary to fill these gaps with rock or gravel. (See table 2-3.) 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. (See figure 2-3.)

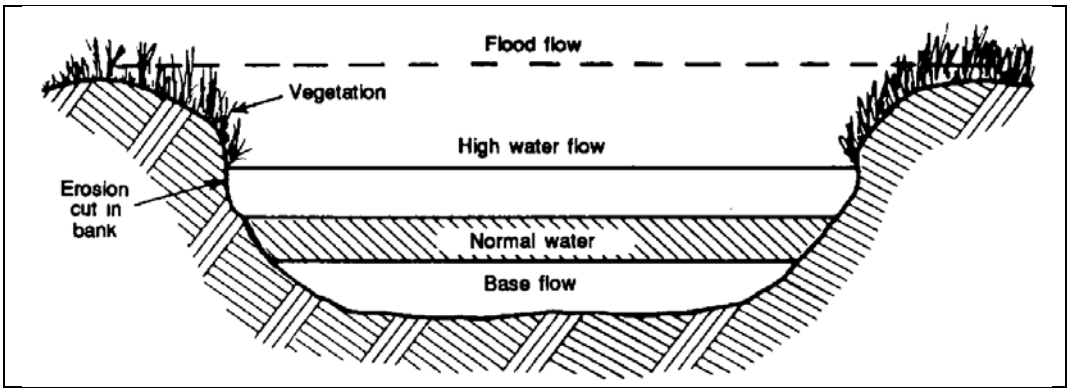


Figure 2-3. Stream cross section

2-71. Carefully note the height of the bank and the type of soil. This information helps to determine construction requirements. Normal stream velocity at the fording site should not exceed 3 feet per second. This velocity would, in most cases, occur at low or moderate levels.

2-72. Make a cross section of a ford location similar to those illustrated in figure 2-2, page 2-11, and figure 2-3. Include full details on the 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. Record the character of the streambed, to include vegetation density and type, whether or not the channel is scoured, and the type of soil. This information determines the value of Manning's roughness coefficient.

Construction

2-73. The phases of ford construction are—

- **Approach Development.** The maximum slopes for ford approaches should be as recommended in table 2-3, page 2-12. 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. Provisions should be made for protecting the surface.
- **Bottom Preparation.** Ford-bottom preparation will depend on site conditions. Fill short, deep gaps with rock or gravel, preferably retained by wire mesh. Soft, mud bottoms can be improved by first covering the bottom with willow, brush mattresses, or timbers. Second, cover the bottom with metal planking, rock, or coarse gravel. Even a hard and tenacious bottom deteriorates under traffic conditions and requires protective maintenance. Consider the following factors when raising the bottom of the ford:
  - The depth upstream from the ford increases in proportion to the amount of rise of the ford bottom.
  - The flow velocity over the ford increases at an increased fording depth. Vehicles may be difficult to operate and control.

Marking

2-74. Place marking posts at each end of the ford and at as many intermediate points as 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

2-75. 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

2-76. Dips are paved fords used for the crossing of dry, wide, and shallow arroyos in semiarid regions that are subject to flash floods.

Reconnaissance

2-77. 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 ponding will occur and to what level.

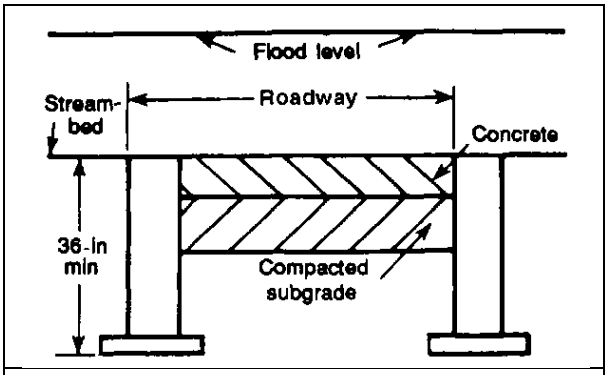
2-78. Determine the type of soil in the banks and bottom for construction requirements. In addition, note the size and type of bottom rock. This type of information gives an indication of the volume and velocity of flash floods that move or carry large material.

2-79. 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

2-80. 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 2-4. 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.
- 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.



Legend:

in	inch
min	minimum

Figure 2-4. Cross section of dip section

Marking

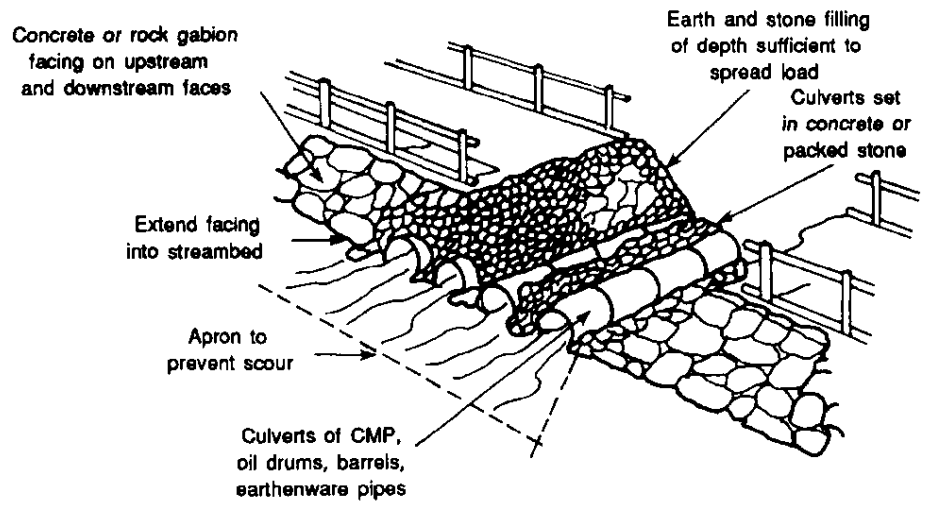
2-81. The marking for dips is the same as for fords. Place marking posts at each end and at intermediate points, as needed. Mark a post at each end with an index to indicate depth. Warning notices should be clearly and prominently placed to alert drivers that flooding can occur suddenly and without warning.

Maintenance

2-82. Dips, like fords, must be examined after each flooding. Repair scour damage upstream and downstream with riprap, and remove boulders and other debris. When macadam is used, holes may be scoured in the roadway. Consider stockpiling rock adjacent to the area for immediate maintenance repair.

CAUSEWAYS

2-83. 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 2-5.



Legend:
CMP corrugated metal pipe

Figure 2-5. Overflow (causeway)

Construction

2-84. 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 enough to protect the waterway against traffic loads.

2-85. 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 so that there is minimum obstruction to water flow.

2-86. Provide guardrails to guide and direct traffic. Because the structure can be overtopped, prepare for ready replacement of the guardrails.

Maintenance

2-87. Inspect the structure for scour or erosion after each flow that causes partial submergence or overtopping. Repair any damage immediately, otherwise, at the next heavy flow, the structure could be destroyed. Stockpiling heavy rock and gabions at each end of a structure may be required.

BRIDGES

2-88. 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 cross-stream structures (fords, dips, causeways).

Location

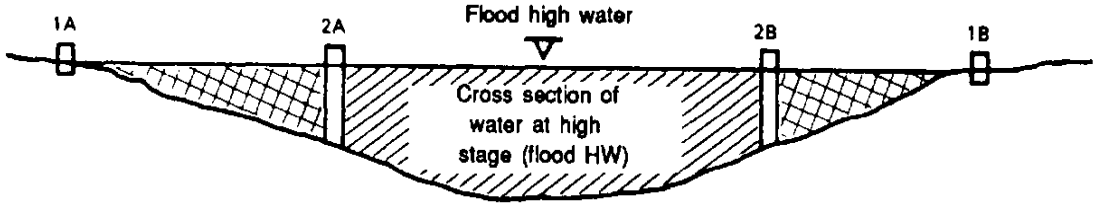
2-89. The location of a bridge should be away from bends in the straight section or reach of the stream (figure 2-2, page 2-13). In this location, there are moderate, even depths from bank to bank. The deep channel tends to be in the center. Abutments will be placed on the edge of the stream. Because of the even distribution of flow, scour and erosion at the stream pier and bank abutments are not expected to be so excessive as to cause maintenance problems.

Construction

2-90. The height of a bridge depends on 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 the bank level and may require a considerable length and height of approach.
- When the flood-flow high watermark is below the 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.

2-91. The location of bridge abutments depends on an analysis of the flood level and the cross section of the stream. (See figure 2-6.) Abutments placed at Point 1A and Point 1B present the most direct solution because they are located on high, dry ground. 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 floodwater.



Legend:
HW high watermark

Figure 2-6. Waterway cross section at bridge site

2-92. Abutments placed at Point 2A and Point 2B would reduce construction time because fewer intermediate piers and spans would be required. In addition, the fill in Area 1A-2A and Area 1B-2B could be accomplished as the bridge is constructed. As can be seen from figure 2-6, page 2-17, 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 Area 1A-2A and Area 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 Area 1A-2A and Area 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 2-3, page 2-12, 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

2-93. 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.

ENGINEER RECONNAISSANCE

2-94. 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. The types of engineer reconnaissance are general and special. 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 (Engineer Reconnaissance Report) is a required enclosure to the route reconnaissance. (See FM 3-34.170/MCWP 3-17.4 for additional information.)

AIRFIELD RECONNAISSANCE

2-95. Airfield reconnaissance differs from road-location reconnaissance in the scope of information. (See FM 3-34.170/MCWP 3-17.4 for additional 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.

2-96. Recent advances in geographic information system technologies allow reconnaissance efforts to be conducted in a digital environment. Most data sets used in theaters of interest are archived digitally and can be readily loaded into commercial software with relevant features highlighted for analysis. Key design parameters can be overlain from data obtained from several map types to create a composite view of the area of interest to help with selecting airfield locations or expansions.

PLANNING

2-97. 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.

2-98. Most known airfield locations, the geometry, and the civilian population attitude are contained within digital archives that can be readily queried for an area of interest. It is recommended that the planning team access the required geographic information systems for an area of interest to direct future air and ground reconnaissance.

2-99. For an undeveloped potential site, the object of the reconnaissance is to verify or amend tentative selections and layouts and estimate the material, equipment, and troop requirements for the construction planned. If it is a captured enemy airfield, a decision to use the captured field or develop a completely new site is needed. Estimates for the engineering effort necessary to restore the airfield may also be required.

2-100. New airfields added to an area where friendly aircraft are operating can be developed as follows:

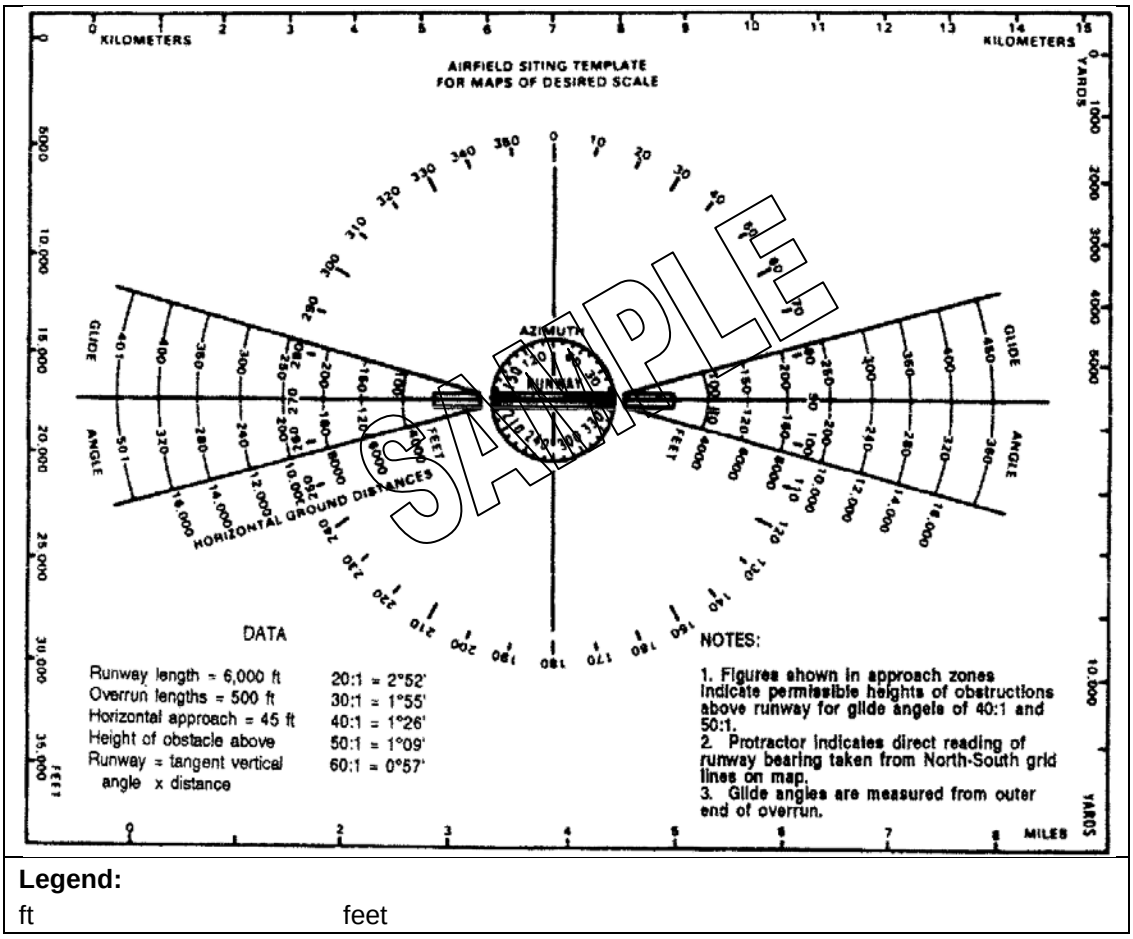
- Select the best available map of the area where a new airfield will be located. Draw a 5 mile diameter circle around existing airfields, and shade them. Note high-tension, electric transmission lines and shade a 2 mile wide strip centered on these lines. Locate and shade similar obstructions on the map. Assault or hasty airfield selection is discussed in TM 3-34.48-2. This exercise can now be performed in a digital environment expeditiously. Most archived aerial maps contain layers of information grouped according to features of interest (power lines, roadways, streams, railways) that can be turned on or off to allow rapid viewing of potential hazards.
- Confine the study for a potential airfield 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 A RUNWAY LOCATION

2-101. An airfield siting template is a convenient way to select a runway location that meets glide-angle requirements at a site. (See figure 2-7, page 2-20.) This template can be drawn on acetate or heavy cellophane to place on a map. The template indicates specifications for flight way, horizontal approach, and glide angle. When placed on the map, the template shows landforms and natural or manufactured obstacles that are in or above the plane of the glide angle.

2-102. Digital geographic information system environments allow similar airfield sighting from a computer. For elevation information, the ready availability of digital satellite terrain elevation data imagery can provide topography at a coarse scale to allow rapid determination of potential airfield sites.

2-103. For example, a hill within the approach zone at a distance of 8,000 feet from the end of the overrun with an elevation of more than 160 feet above the end of a proposed runway is in a 50:1 glide angle. (See figure 2-7.) This runway location would be unsuitable according to the specifications. The template is a useful tool for the reconnaissance officer and the preliminary planning group. Prepared templates can measure distances in feet, yards, miles, and kilometers by placing gradations along the edges.



Legend:

ft feet

Figure 2-7. Sample airfield-siting template

PROCEDURES FOR AIRFIELD AIR RECONNAISSANCE

2-104. While en route to a site or general area, the engineer notes open borrow pits, large construction material stockpiles, rail and road access to the site, and errors on the 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 familiar with operational requirements and the performance characteristics of tactical aircraft is valuable.

2-105. 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 flying related characteristics of the sites investigated and modifies estimates according to these recommendations.

2-106. To be effective as an engineer observer, the officer should possess the following qualifications:

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

2-107. Similar second and third passes are flown. Obstructions, main slopes, and general features are noted during these circuits. The pilot begins to formulate an estimate of 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.

2-108. 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 that the runway length can be estimated by multiplying the air speed by the average flight time.

Note. The length is usually 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 that the wind is constant).

2-109. 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 fields flying suitability.

2-110. In departing, the observer reviews dispersal areas and checks access roads again. Additional passes over the site are made if questions arise as a result of this last check. An air reconnaissance report format similar to figure 2-8, page 2-22, may be used.

2-111. An area reconnaissance proceeds with an inspection of other possible sites. Complete notes must be kept to avoid reviewing sites that have been already checked. However, reinvestigation of the final site selected and selected alternative sites may sometimes be necessary.

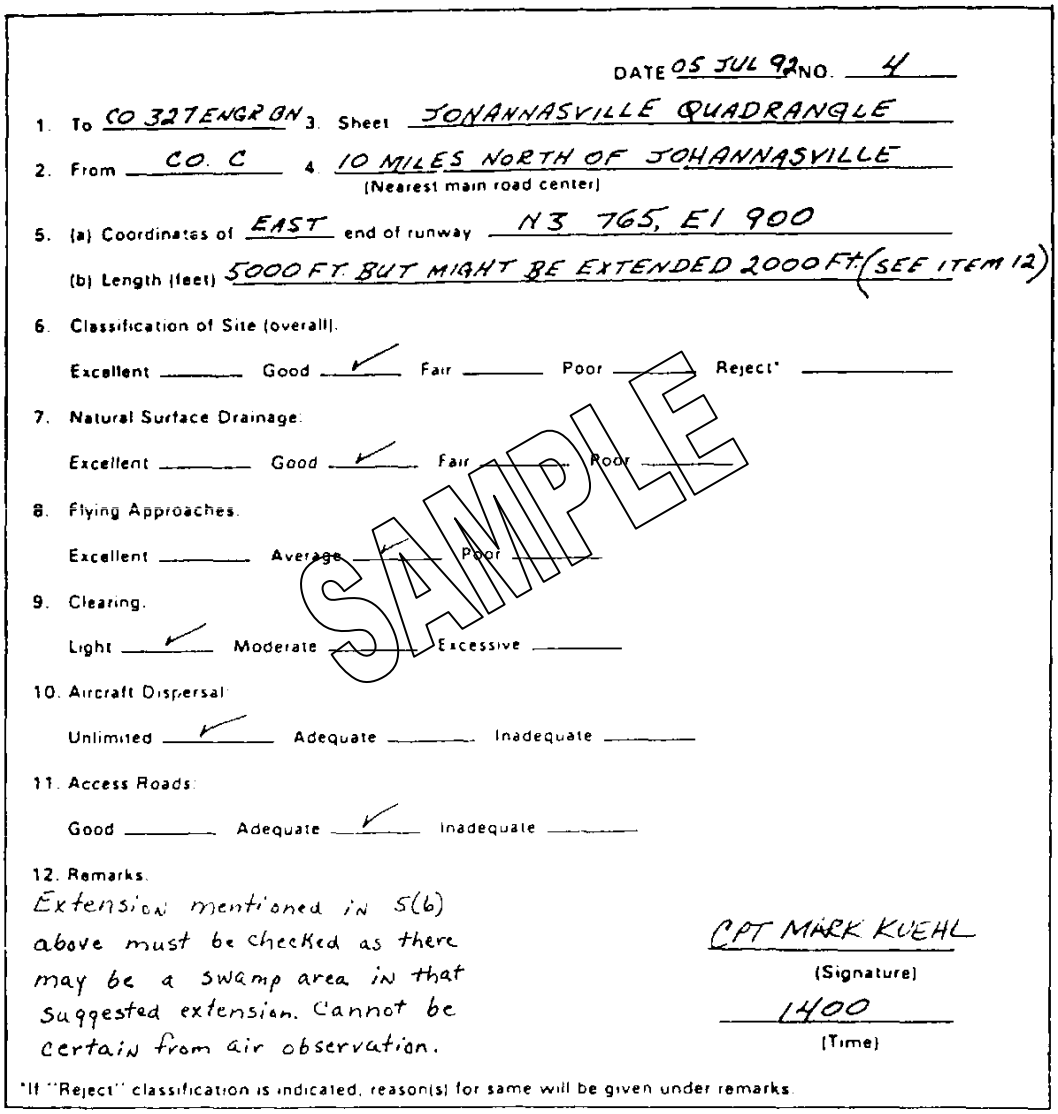
PROCEDURES FOR AIRFIELD GROUND RECONNAISSANCE

2-112. The ground-reconnaissance phase is preceded by map and air reconnaissance to discover what specific sites and questions warrant ground investigation.

2-113. 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 potential water points to be investigated on the ground. 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, features that might hinder the movement of construction equipment to the site, and 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.

2-114. 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 and ideally conducted with global positioning system referenced positions. Lengths are paced, critical slopes are measured with a clinometer, and directions are determined with a magnetic compass.

2-115. 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.



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(6) 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.

SAMPLE

Legend:

BN	battalion
CO	company
CPT	captain
ENGR	engineer
ft	feet
JUL	July
No.	number

Figure 2-8. Sample air reconnaissance report format

2-116. Soil type plays a critical role in the design and construction of a runway, taxiway, or apron. It determines what the required level of effort will be to improve the natural ground. It also determines the suitability of the surrounding soils to provide sufficient bearing strength to withstand the structural loads of the intended aircraft.

2-117. Expedient soil evaluations can be performed, and the rapid soils analysis kit that is tailored to evaluate soils in contingency aircraft operations can be used. These techniques can be conducted within 1 to 2 hours. (See TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034) and United States Army

Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC/GSL) TR-08-3 for additional information.)

2-118. If the area 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 the obstacles (gullies, rocky outcrops, swampy areas) that cannot readily be eliminated on a large-scale map or sketch. Examination of the results discloses possible runway locations.

2-119. The best runway location is selected by considering these centerline investigations with prevailing wind direction; air approaches; glide angles; groundwater conditions; and discharge areas for collected runoff, clearing, grubbing, and earthwork. If a suitable runway does not exist, a negative site report on the site is submitted.

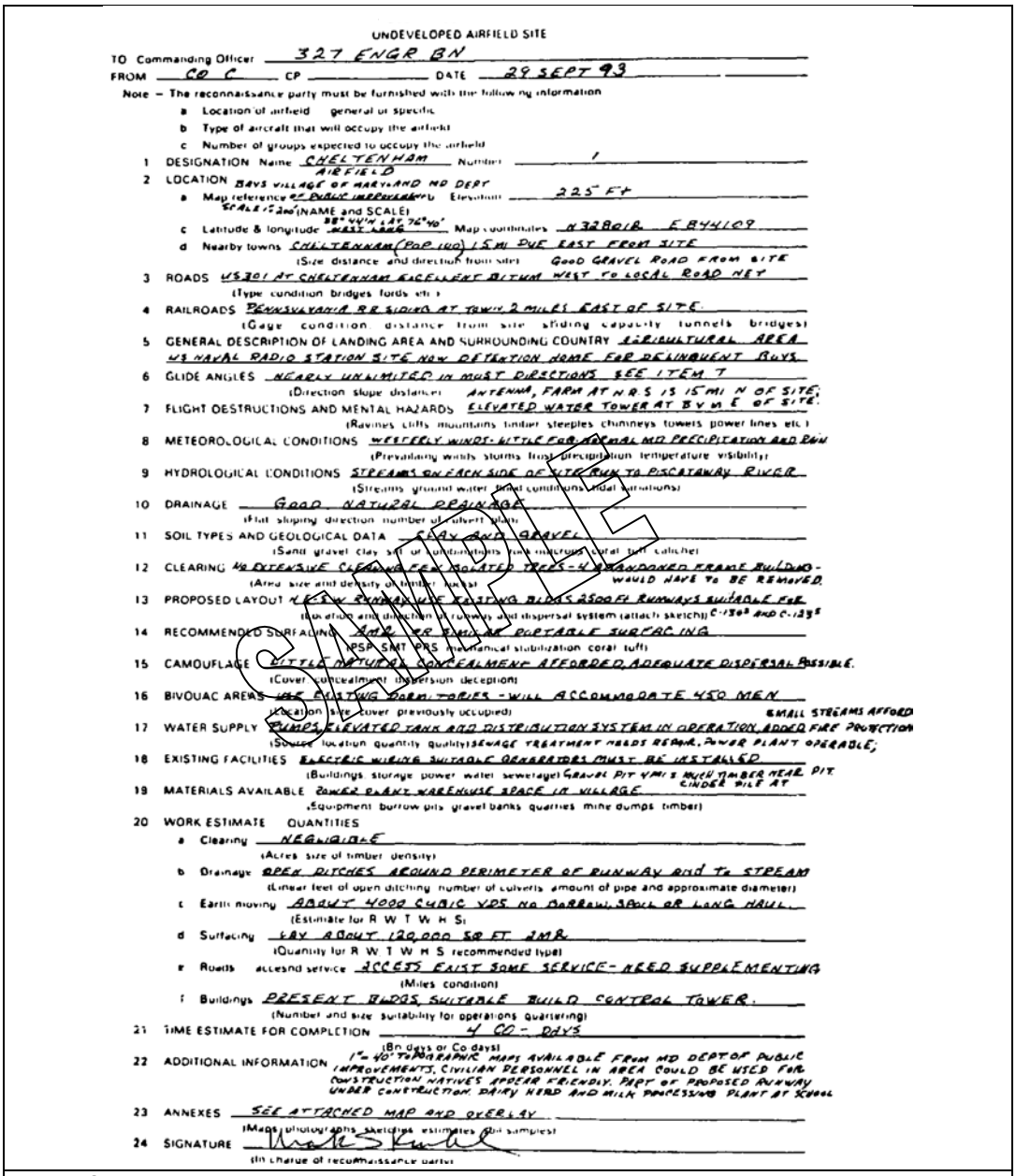
2-120. Once the selection of a potential runway is made, a careful and detailed walk of the runway centerline is made to recheck its suitability. Stakes are driven at each end of the runway, and prominent features are properly referenced to expedite the runway location by construction unit surveyors.

2-121. The reconnaissance party survey sergeant stakes out the centerline of the runway and runs a ground profile at the centerline and each shoulder line. Levels are taken at 500-foot intervals and at intermediate breaks or slope changes. In a flat area, 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 10 of this manual for more information about soil conditions.

2-122. Previously acquired information is checked for accuracy at the site. Errors, including discrepancies on the maps and mistakes, are included in the report. (See figure 2-9, page 2-24, for a sample report of aerial photograph interpretations format.)

2-123. 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. The information must be weighed carefully with regard to the credibility of each person questioned.

2-124. The reconnaissance of a designated site should be accomplished in one day unless hostile forces delay the work. A specific reconnaissance of a captured enemy airfield is somewhat different. Detailed information about existing facilities and their condition is desired. (See figure 2-10, page 2-25, for a sample reconnaissance report of a captured enemy airfield format.) A structural strength of the supporting layers will add tremendous value to the survey through the use of a dynamic cone penetrometer. (See TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034 for additional information.)



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 b Elevation 225' Ft

SCALE 1:20,000 (NAME and SCALE)

c Latitude & longitude WEST LONG 38° 49'N LAT 76° 40' Map coordinates N 32808 E 844109

d Nearby towns CHELTENHAM (POP 100) 1.5 MI DUE EAST FROM SITE

(Size distance and direction from site) GOOD GRAVEL ROAD FROM SITE

3 ROADS US 301 AT CHELTENHAM EXCELLENT BITUM 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 sliding 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) ANTENNA, FARM AT N.R.S. IS 15 MI N OF SITE;

7 FLIGHT OBSTRUCTIONS AND MENTAL HAZARDS ELEVATED WATER TOWER AT B Y M E OF SITE.

(Ravines cliffs mountains timber steeples chimneys towers power lines etc)

8 METEOROLOGICAL CONDITIONS WESTERLY WINDS- LITTLE FOG FOG AND 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 tidal conditions tidal variations)

10 DRAINAGE GOOD NATURAL DRAINAGE

(Flat sloping direction number of culvert pipes)

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 BUILDING - WOULD HAVE TO BE REMOVED

(Area size and density of timber type)

13 PROPOSED LAYOUT LESS THAN MILE EXISTING BLDGS 2500 FT RUNWAYS SUITABLE FOR

(Location and direction of runways and dispersal system (attach sketch)) C-130 AND C-135

14 RECOMMENDED SURFACING SAME OR EQUAL PORTABLE SURFACING

(PSP SMT APS mechanical stabilization coral tuff)

15 CAMOUFLAGE LITTLE NATURAL CONCEALMENT AFFORDED, ADEQUATE DISPERSAL POSSIBLE.

(Cover concealment dispersion deception)

16 BIVOUAC AREAS USE EXISTING BARRACKS - WILL ACCOMMODATE 450 MEN

(Location size cover previously occupied)

17 WATER SUPPLY PUMPS ELEVATED TANK AND DISTRIBUTION SYSTEM IN OPERATION, ADDED FIRE PROTECTION SMALL STREAMS AFFORD

(Source location quantity quality) SEWAGE TREATMENT NEEDS REPAIR, POWER PLANT OPERABLE;

18 EXISTING FACILITIES ELECTRIC WIRING SUITABLE GENERATORS MUST BE INSTALLED

(Buildings storage power water sewerage) GRAVEL PIT 4 MI S MILL TIMBER NEAR PIT CINDER PILE AT

19 MATERIALS AVAILABLE 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, HAUL OR LONG HAUL.

(Estimate for R W T W H S)

d Surfacing SAY ABOUT 120,000 SQ FT 2MR

(Quantity for R W T W H S recommended type)

e Roads accesnd service ACCESS EAST 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]

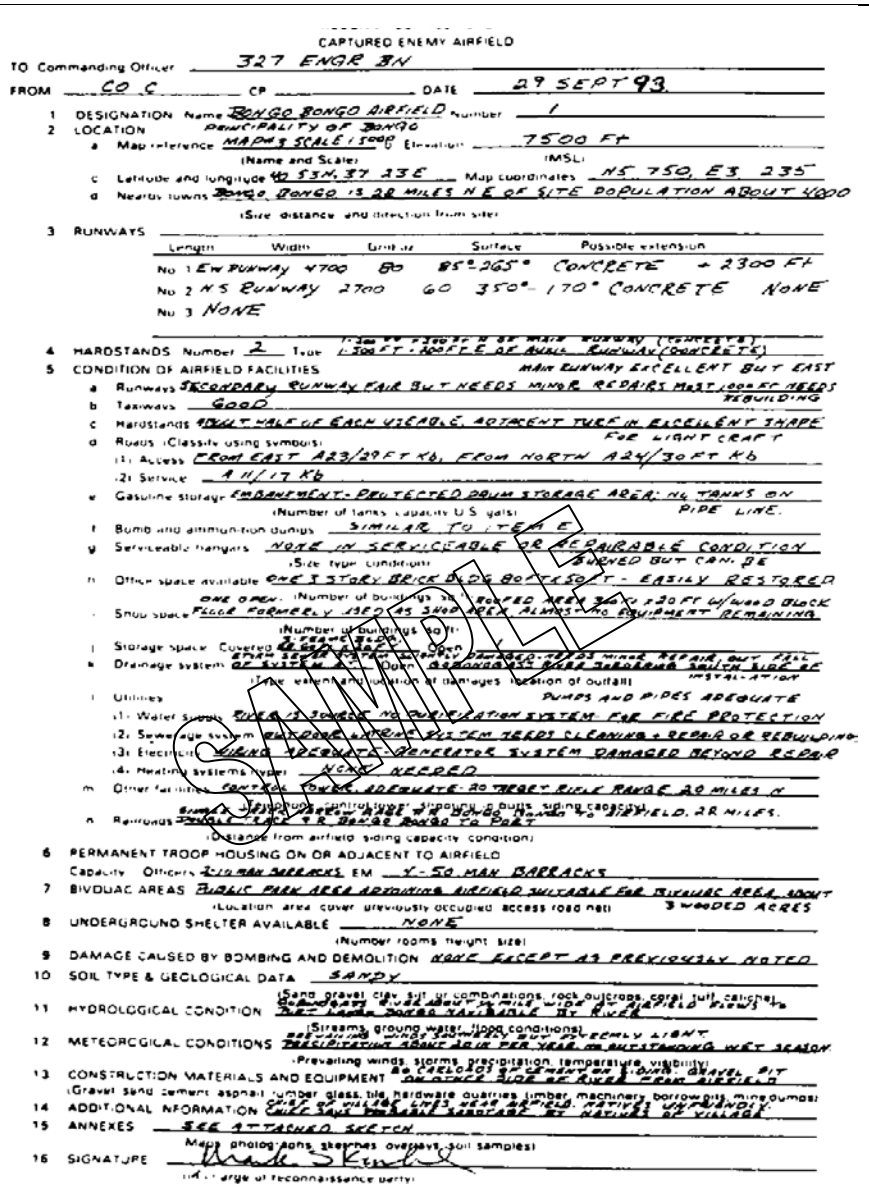
(In charge of reconnaissance party)

SAMPLE

Legend:

bldg	building
BN	battalion
CO	company
CP	command post
E	East
ENGR	engineer
ft	feet
mi	mile
N	North
pop	population
SEPT	September
SQ	square
yds	yards

Figure 2-9. Sample ground reconnaissance report–undeveloped airfield site format



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 Municipality of BONGO

a Map reference MAPS SCALE 1:50000 Elevation 7500 Ft (Name and Scale) (MSL)

c Latitude and longitude 42 53N, 37 23E Map coordinates N5 750, E3 235

d Nearest towns BONGO, BONGO IS 28 MILES NE OF SITE POPULATION ABOUT 4000 (Size distance and direction from site)

3 RUNWAYS

	Length	Width	Grade	Surface	Possible extension
No 1	EW RUNWAY 4700	80	85°-265°	CONCRETE	+ 2300 FT
No 2	NS RUNWAY 2700	60	350°-170°	CONCRETE	NONE
No 3	NONE				

4 HARDSTANDS Number 2 Type (SOFT - HOOFS OF ANEL RUNWAY (CONCRETE))

5 CONDITION OF AIRFIELD FACILITIES MAIN RUNWAY EXCELLENT BUT EAST

a Runways SECONDARY RUNWAY FAIR BUT NEEDS MINOR REPAIRS MUST 1000 FT NEEDS REBUILDING

b Taxiways GOOD

c Hardstands ABOUT HALF OF EACH VIEADUCT ADJACENT TURF IN EXCELLENT SHAPE

d Roads (Classify using symbols) FOR LIGHT CRAFT

(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: NIL TANKS ON (Number of tanks capacity U S gals) PIPE LINE.

f Bomb and ammunition dumps SIMILAR TO ITEM E

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

h Office space available ONE 3 STORY BRICK BLDG 80 FT SQ FT - EASILY RESTORED

ONE OPEN (Number of buildings sq ft) STORED AREA 30 FT X 20 FT W/WOOD BLOCK

i Shop space FLOOR FORMERLY USED AS SHOP AREA ALMOST NO EQUIPMENT REMAINING (Number of buildings sq ft)

j Storage space Covered 2 FRAME BLDGS Open

k Drainage system OF SYSTEM 2 Open DRAINAGE DITCHES DAMAGED - NEED REPAIR, BUT FEEL EASILY RESTORED WITH SIDE OF INSTALLATION (Type extent and location of damages location of outfall)

l Utilities PUMPS AND PIPES ADEQUATE

(1) Water supply RIVER IS SOURCE NO PURIFICATION SYSTEM FOR FIRE PROTECTION

(2) Sewerage system OUTDOOR & INDOOR SYSTEM NEEDS CLEANING & REPAIR OR REBUILDING

(3) Electricity MAIN 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, shops) 2 butts, siding capacity)

n Railroads SINGLE TRACK RR BONGO BONGO TO PORT NO SIDING AT 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 (silt) caliche)

11 HYDROLOGICAL CONDITION GROUNDWATER DRAINAGE ABOUT 10 MILE WIDE OF AIRFIELD RIVERS TO NEAR LARGE SOURCE NAVIGABLE BY RIVER (Streams ground water flood conditions)

12 METEOROLOGICAL CONDITIONS SEE WINDS SUMMARY, BUT GENERALLY LIGHT PRECIPITATION ABOUT 60 IN PER YEAR IN OUTSTANDING WET SEASON (Prevailing winds storms precipitation temperature visibility)

13 CONSTRUCTION MATERIALS AND EQUIPMENT 80 CARLOADS OF CEMENT AT LUANDA GRAVEL PIT ON COAST SIDE OF RIVER FROM AIRFIELD (Gravel sand cement asphalt lumber glass tile hardware quarries timber machinery borrow pits mine dumps)

14 ADDITIONAL INFORMATION CHIEF SOURCE OF WATER SUPPLY NEAR AIRFIELD. NATIVES UNFRIENDLY. CHIEF SUPPLY VILLAGE 5 MILES NORTH OF AIRFIELD

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

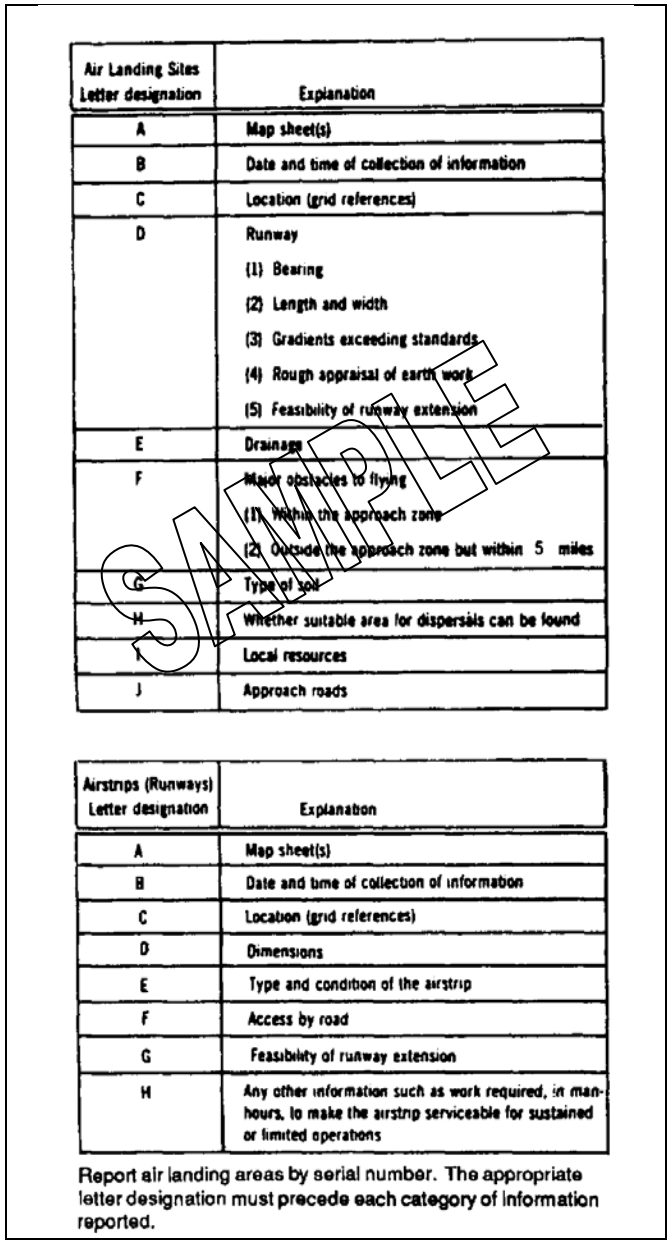
16 SIGNATURE (Officer in charge of reconnaissance party)

SAMPLE

Legend:	
BN	battalion
CO	company
E	East
ENGR	engineer
ft	feet
kb	kilobytes
N	North
no.	number
SEPT	September
sq	square

Figure 2-10. Sample ground reconnaissance report–captured enemy airfield format

2-125. When reconnaissance parties are operating at a considerable distance from the headquarters directing the reconnaissance, it is imperative that an initial report reaches headquarters without delay. Submit completed reports with organic radio equipment or satellite uplinks for internet transmittal. (See figure 2-11 for a sample message format.) The tactical situation may dictate the amount of information transmitted. Unit standing operating procedures should indicate what information is critical for radio reports. A complete, written report should follow the radio 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

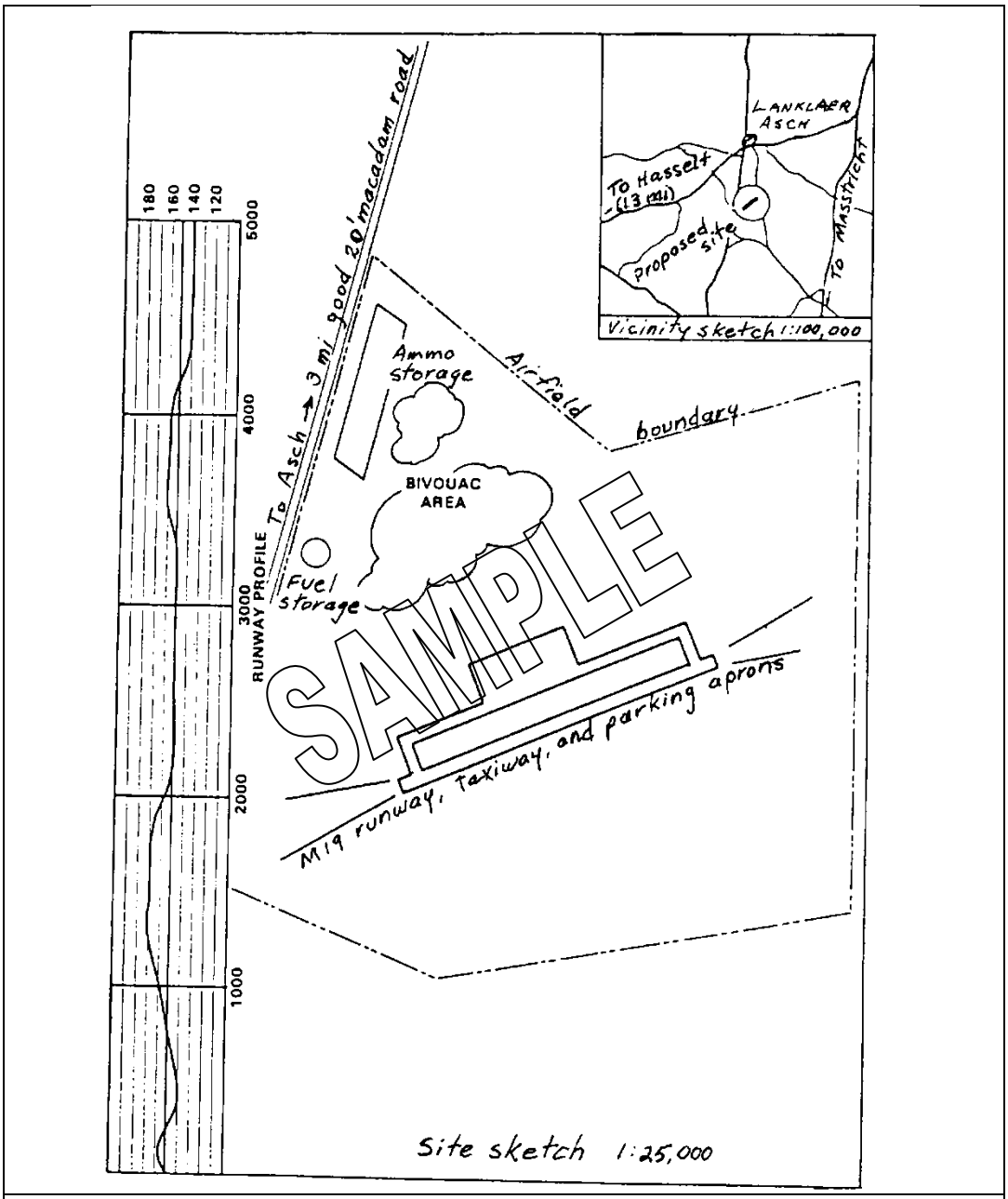
Airstrips (Runways) Letter designation	Explanation
A	Map sheet(s)
B	Date and time of collection of information
C	Location (grid references)
D	Dimensions
E	Type and condition of the airstrip
F	Access by road
G	Feasibility of runway extension
H	Any other information such as work required, in man-hours,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

SAMPLE

Figure 2-11. Sample air landing area report format

2-126. Written reports should include the information shown in the previous samples. (See figure 2-9, page 2-24, and figure 2-10, page 2-25.) Suitable sketches should be attached to written reconnaissance reports. (See figure 2-12 for a sample sketch.)



Legend:

ammo	ammunition
ASCM	air space control measures
mi	mile

Figure 2-12. Sample sketch to accompany an airfield reconnaissance report format

This page intentionally left blank.

Chapter 3

Surveys and Earthwork Operations

Construction surveys are initiated when new construction is necessary. These surveys provide data for earthwork estimation (including the estimation method to use) and information for the mass diagram. The finished survey books should be filed with the construction project records of the operations and training officer (United States Army). The earthwork operation is one of the most important construction aspects in road and airfield construction. Earthwork requires the greatest amount of engineering effort from the standpoint of personnel and equipment. Therefore, planning, scheduling, and supervision of earthwork operations are important in obtaining an efficiently operated construction project.

CONSTRUCTION SURVEYS

3-1. 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.

3-2. The purpose of construction surveys is to control construction activities. The number and extent of surveys conducted is governed by the time available, the construction standard desired, and the availability of personnel and materials. In the combat zone, roads and airfields are constructed with minimum preplanning and construction control activities. However, extensive surveys may be conducted 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 TM 3-34.55. The equipment used for the surveys can be manual, automated, and global positioning system surveying equipment. There are off the shelf software packages that provide the ability to support the raw survey data and develop a three-dimensional geometric design model and a set of construction drawings.

3-3. After completing a thorough construction survey, transfer the design information from the computer software to automated survey equipment and then to the field on construction stakes. These stakes are the guides and reference markers for earthwork operations. Mark the stakes so that the construction conforms to the planned line and construction crews properly interpret the grade of the road or airfield and the information on the stakes.

RECONNAISSANCE

3-4. 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 the reconnaissance survey, it must be determined by the preliminary survey.

PRELIMINARY

3-5. 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. 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.

3-6. An airfield survey establishes controls, notes terrain features, measures glide-angle clearance, makes soil profiles, and investigates drainage patterns and approaches. Accurately establish the final centerline during this survey.

FINAL LOCATION

3-7. As time permits, conduct a final location survey. Establish permanent benchmarks for vertical control and well-marked points for horizontal control. These points are called hubs because of the short, square stake used; hub stakes are 2 inches by 2 inches. On most surveys, the hub is driven flush with the ground, and a tack in the top of the stake marks the exact point for angular and linear measurements. A flat guard stake—extended above the ground and driven in at a slope so that the top is over the hub—indicates the hub location. The guard stake is about 3/4 inch by 3 inches.

Horizontal Control

3-8. The purpose of horizontal control is to accurately determine horizontal locations 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.

3-9. As 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 that the surveyor can make frequent checks on alignment while the line is being staked out. Target boards may be set up on a line that requires precision alignment. Reference control stakes, and replace them if they are disturbed or lost. Locate the target board just beyond the outermost control point stake.

Vertical Control

3-10. Vertical control methods determine the difference in elevation between points. If available, establish a level reference surface or datum from a known benchmark. Differences in elevation, with corrections, are subtracted from or added to this assigned value, resulting in elevation of the points. Take the benchmark system datum from a known elevation or barometer reading or make an arbitrary assumption for establishing relative elevation differences.

CONSTRUCTION LAYOUT

3-11. 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 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

3-12. Use construction stakes for centerline, slope, offset, shoulder, grade, reference, ditch, culvert, intermediate, and temporary benchmarks. The stakes should be approximately 1 inch by 3 inches by 2 feet. Use finished lumber when possible. If it is not possible to use finished lumber, use small trees or branches blazed on both sides and cut to length. Finished grade stakes and temporary benchmarks are 2 inches by 2 inches by 12 inches. Place stakes using a three- to five-person crew equipped with manual or automated survey equipment or a real-time kinematic global positioning system, an ax, sledgehammer, and machete.

3-13. The primary functions of construction stakes are to indicate facility alignment control elevations, guide equipment operators, and eliminate unnecessary work. They also determine the clearing width required by indicating the cut and fill limits at right angles to the centerline of a road.

3-14. 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 that the construction crew can properly interpret the information on the stakes. Construction stakes indicate the—

- Stationing or location of a 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, point of intersection, or point of tangency of a curve), note this on the stake.
- Cut or fill height from the existing ground surface to the top of the subgrade for centerline stakes or to the shoulder grade for shoulder or slope stakes.
- Horizontal distance from the centerline to the stake location.
- Side-slope ratio used on slope stakes.

3-15. 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 it 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

3-16. The centerline or alignment (hub) stakes are placed on the centerline of a road or airfield and indicate its alignment, location, and direction. (See figure 3-1.) They are the first stakes placed, and they must be located accurately. These stakes are used as reference points in locating the remaining stakes. Centerline stakes are placed at 100-foot intervals. On rough ground or sharp horizontal and vertical curves, place the stakes closer together. Stake the point of curvature, point of intersection, and point of tangency on horizontal curves. On vertical curves, stake the point of vertical curvature, the point of vertical intersection, the point of vertical tangency, and the low point or high point of the curve.

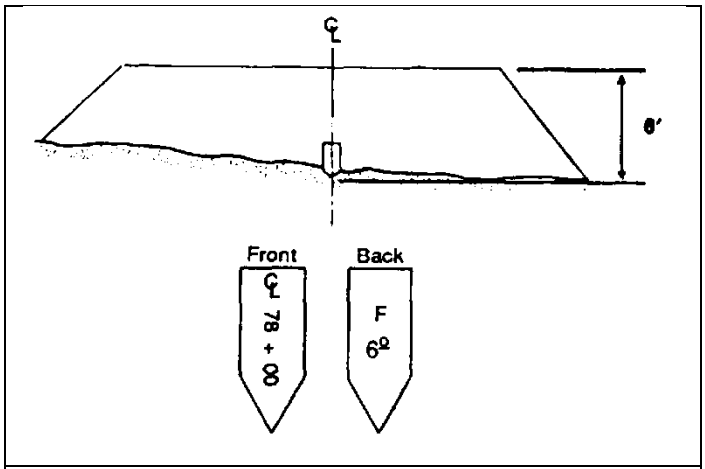


Figure 3-1. Centerline stakes

3-17. 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 symbol for centerline and point of curvature, point of intersection, or point of tangency if applicable. On the front, mark the distance from zero or the starting point in 100-foot stations and the fractional part of a station if used. For example, *6 + 5422* marked on a stake indicates that 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.

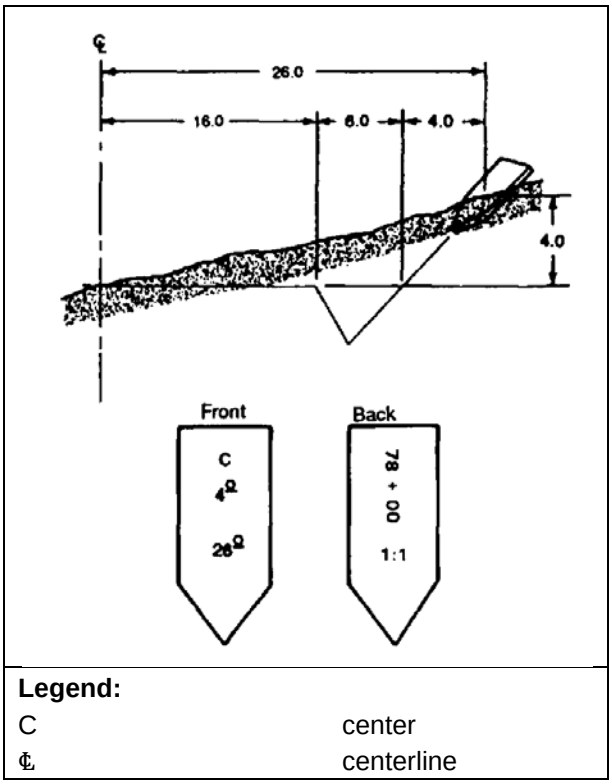
3-18. Place the amount of cut or fill required at the station on the reverse side of the stake. A cut is marked with a *C*, and a fill is marked with an *F*. A centerline stake placed at Station *78 + 00* and requiring a fill of 6 feet to bring this station up to the final grade line would be placed as shown in figure 3-1, page 3-3.

3-19. The cut or fill amount indicates the difference between the final grade line and the ground line where the stake is placed. A point on the stake is seldom used as the line of reference to the final grade.

3-20. To prevent misinterpretation of the cut or fill amount, decimal parts of a foot are written smaller, raised, and underlined. (See figure 3-1.) 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

3-21. Slope stakes define the limits of grading work. (See figure 3-2.) When used in read work, they can be used as guides to determine the clearing width 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.



Legend:	
C	center
℄	centerline

Figure 3-2. Marking and placing slope stakes

3-22. Sloping stakes outward allows equipment to work up to the stake without removing it. The slope indicates the direction of the centerline of the road, and equipment operators can read the stakes easier. Place slope stakes at 100-foot intervals on tangents and at 50-foot intervals on horizontal or vertical curves. When a sharp break in the original ground profile occurs, it should be staked.

3-23. 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 the 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

3-24. 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 when 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.

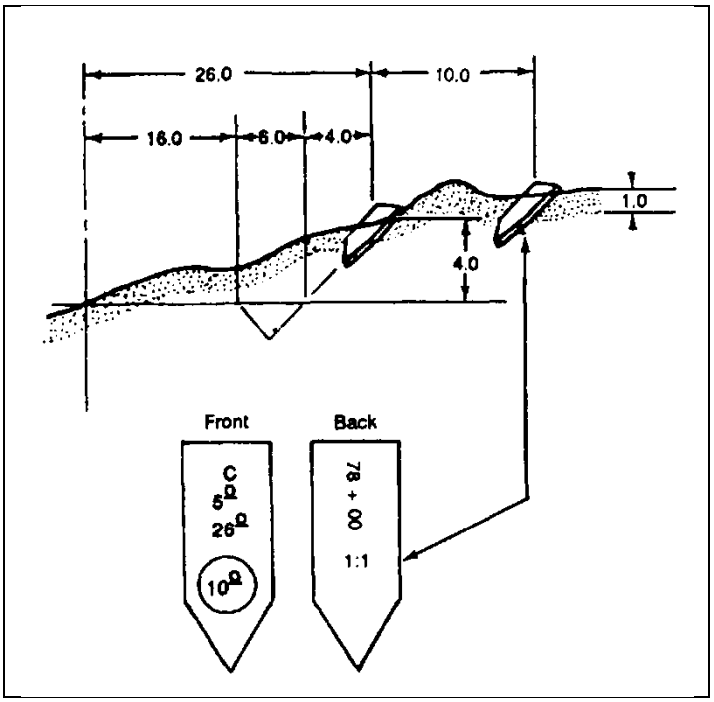


Figure 3-3. Marking and placing offset stakes

3-25. 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 place the new centerline stake at that location.

3-26. An offset stake contains 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 that 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 at 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. The surveyor and grade supervisor coordinate on the meaning of the markings, regardless of the type of marking used.

FINISH-GRADE

3-27. Use wooden stakes (2 inches by 2 inches) with the tops colored red or blue for finish-grade stakes. Finish-grade stakes, also called blue tops or red tops, indicate the final grade elevation of the completed facility. They are used when the grade is within inches 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 guard stakes with each finish-grade stake.

3-28. 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. This is decided by the surveyor and construction foreman.

3-29. With the final-grade stakes set and marked at a predetermined distance above the finished grade, stretch a string between two stakes across the work area, 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 roadwork, place the stakes along the centerline and the edge of the shoulder; they may or may not be placed on the slopes.

REFERENCE

3-30. 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 complete. As an aid in relocating a point that may become hidden by vegetation, or as a means of replacing points that may have been destroyed, measurements are made to nearby permanent or semipermanent objects. This process is known as referencing or witnessing a point. On many surveys, permanent objects may not be available as reference points. In such cases, additional stakes may be driven. These stakes are approximately 2 inches by 2 inches by 18 inches.

3-31. 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 as an advantage in the second case. The method of using two known distances can be used 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.

3-32. To be of most value in replacing a missing station or point, the reference stakes will be less than 100 feet from the point and, if possible, the arcs should intersect at approximate right angles. Place the stakes 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

3-33. Culvert stakes are located on a line parallel to, and offset by 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.

BENCHMARKS

3-34. 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 benchmarks, established by geodetic surveys. From these benchmarks, run a line of levels and set temporary benchmarks. On small projects, running the levels from a point of assumed elevation frequently sets the temporary benchmarks. This is especially true of construction in combat areas.

3-35. Usually, temporary benchmarks are placed at 500- to 1,000-foot intervals and are placed off the limits of construction. Stakes 2 inches by 2 inches, solidly placed 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 temporary benchmarks. Temporary benchmarks are set before centerline stakes are set because vertical control must be established before construction begins.

EARTHWORK ESTIMATION

3-36. Earthwork computations involve calculating earthwork volumes, determining final grades, balancing cuts and fills, and planning the most economical haul route for material. How exact earthwork computations are depends on the extent and accuracy of field measurements, which are 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 that the most efficient type and quantity of earthmoving equipment can be chosen and the appropriate time allotted.

3-37. When time is critical, earthwork quantities are estimated very roughly or not at all. When time is not critical, higher construction standards are possible, and earthwork quantities are estimated and controlled by more precise methods.

FUNDAMENTAL VOLUME DETERMINATION

3-38. The volume of a rectangular object may be determined by multiplying the area at one end by the length of the object. This relationship can be applied to the earthwork determination 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. If a three-dimensional computer software package is used in the road or airfield design, volume determinations are calculated using the survey and design data.

END-AREA DETERMINATION METHODS

3-39. When the construction area centerline has been located, measurements are taken in the field to compute the required quantities of cut or fill. 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. (See figure 3-4 for a typical cross section.)

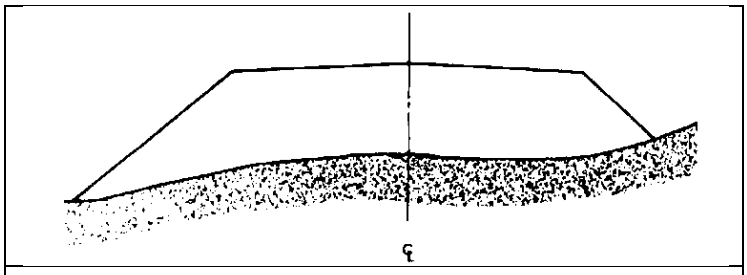
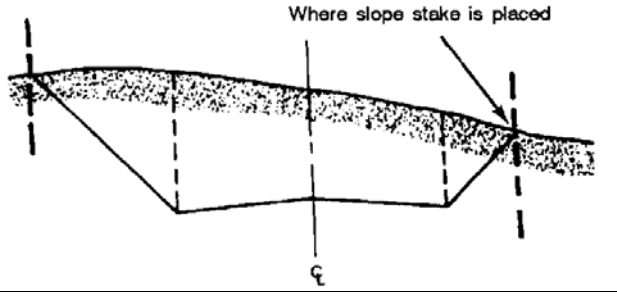


Figure 3-4. Typical fill cross section

3-40. 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 cut and fill, draw the appropriate lines of each template. When the sections are complete, begin the end-area measurements and 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 on the time available, the accuracy desired, the available equipment, and the engineer's preference.

Trapezoidal

3-41. 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 a 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 to divide the cross section into two triangles and two trapezoids. (See figure 3-5, page 3-8.) 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.



Legend:

℄	centerline

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

Basic Formulas

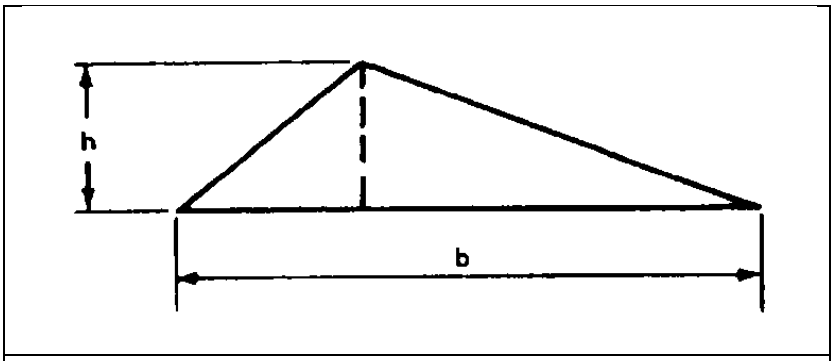
3-42. Before the area of the cross section can be computed, the basic formulas for the computation of the areas of triangles and trapezoids must be understood. If a line is drawn from one of the vertices of a triangle perpendicular to the side or base opposite this vertex, the line formed represents the altitude of the triangle. (See figure 3-6.) The area of a triangle can be expressed as the product of one-half the base multiplied by the altitude. This relationship is expressed by the following formula:

$$A = 0.5bh$$

where—

b = *base*

h = *altitude*



Legend:

b	base
h	altitude

Figure 3-6. Triangle base and height dimension locations

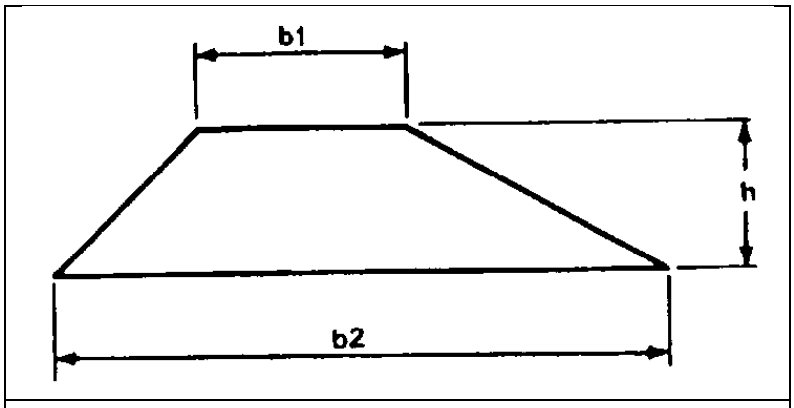
3-43. A trapezoid is a four-sided figure having two sides parallel but not equal in length. (See figure 3-7.) If a perpendicular line crosses the two parallel sides of the bases, the distance between the two bases along this perpendicular line is the altitude of the trapezoid. The area of a trapezoid can be expressed as the average length of the bases multiplied by the altitude. This relationship can be expressed by the following formula:

$$A = 0.5(b1 + b2)h$$

where—

b = *base*

h = *altitude*



Legend:	
b	base
h	altitude

Figure 3-7. Trapezoid base and height dimension locations

Area Computation

3-44. The first step in computing areas by the trapezoidal method is to break the cross-sectional area into triangles and trapezoids by drawing verticals. (See figure 3-5.) Determine the area of these small figures using the appropriate formula.

3-45. 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 number below the line indicates the horizontal distance from the centerline to that point on the ground. The number 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. (See figure 3-8.) Thus, the note $\frac{32.0}{21}$ indicates that the point 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 slopes to solve for the area.

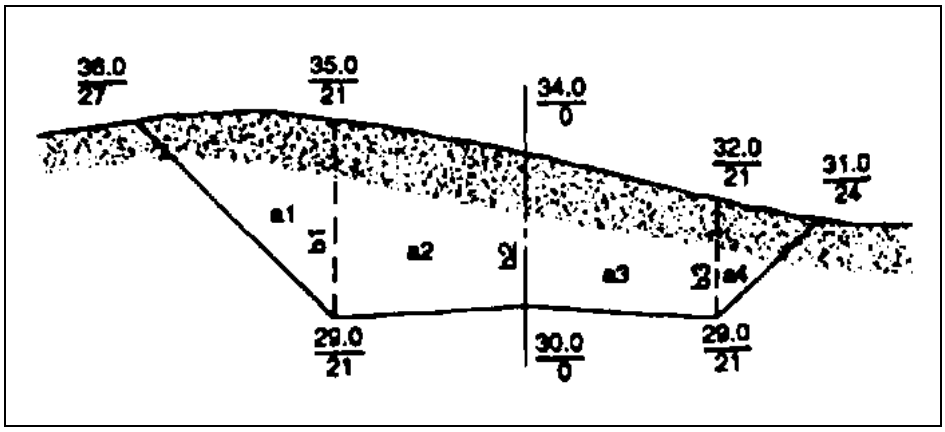


Figure 3-8. Cross-section cut showing distances and elevations

3-46. To solve the triangles and trapezoids formed, consider the bases of these figures to be vertical and the altitudes to be horizontal. Vertical bases are found by subtracting elevations, and horizontal altitudes are found by subtracting horizontal distances from the closest vertical in the direction of the centerline.

Example

Calculate area a_1 from figure 3-8, page 3-9. Substitute in the formula for the area of a triangle.

$$a_1 = 0.5bh = (35 - 29)(27 - 21)$$
$$= 0.5(6)(6) = 18\ square\ feet$$

Calculate area a_2. Substitute in the formula for the area of a trapezoid.

$$a_2 = 0.5(b_1 + b_2)h = \frac{(35 - 29) + (34 - 30)}{2}(21) = 0.5(6 + 4)(21) = 105\ square\ feet$$

where—

b = *base*
h = *altitude*

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

Stripper Method

3-47. The stripper method is a variation of the trapezoidal method. (See figure 3-9.) To use this method, measure (graphically) the lengths, and multiply the sum by the constant width. The distance between vertical lines may be a 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.

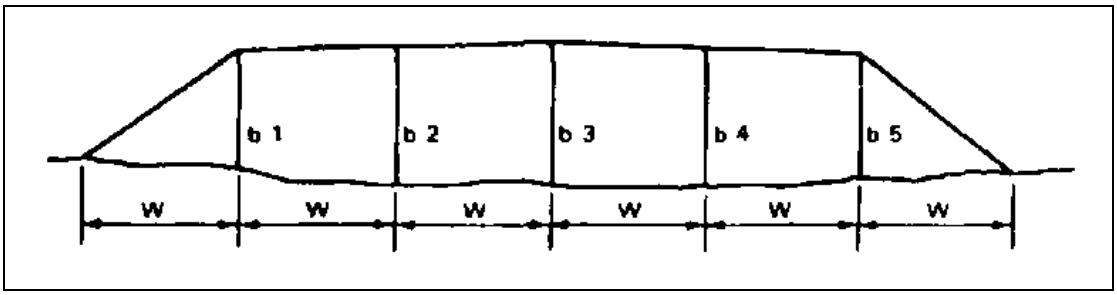


Figure 3-9. Fill cross section arranged to show the stripper method

3-48. One of the easiest and most convenient ways to measure the vertical lines is with a strip of paper or plastic. Lay the strip along the vertical lines, adding the lines to get a total. The strip will show the sum of the vertical lines in the same scale that the cross section is plotted. This figure, multiplied by the width, will give the area of the cross section.

3-49. Inaccuracies result when a triangle or trapezoid falls within the limits of the width 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 the vertical lines and the width. The stripper indicates that the sum of the vertical lines is 21.7 feet (the width is 10 feet). Applying these figures to the formula, then—

$$A = w(\Sigma b)$$
$$= 21.7 \times 10 = 217\ square\ feet$$

where—

A = *area*
b = *length*
w = *width*
Σ = *sum*

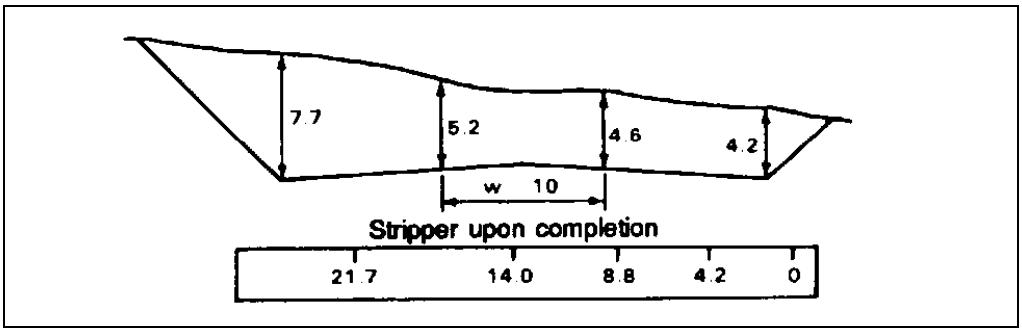


Figure 3-10. Cross section with the sum of vertical lines added on the stripper

Example

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

$$A = 0.5b_1w + 0.5(b_1 + b_2)\,w + 0.5(b_2 + b_3)\,w + 0.5(b_3 + b_4)\,w + 0.5(b_4 + b_5)\,w + 0.5b_5w$$

Then, factor in and combine terms—

$$A = 0.5\,w\,(2b_1 + 2b_2 + 2b_3 + 2b_4 + 2b_5)$$

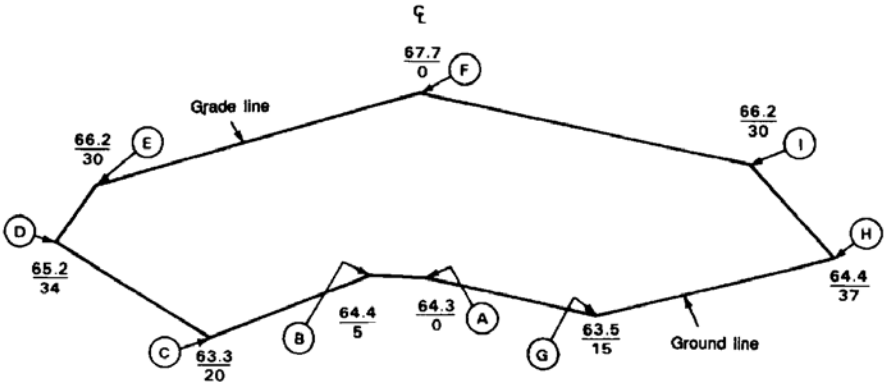
$$A = w(\Sigma b)$$

where—

A	=	*area*
b	=	*length*
w	=	*width*
Σ	=	*sum*

Double-Meridian Triangle

3-50. The double-meridian triangle method gives a more precise value for a cross-section area than the stripper method. However, it involves more time. With this method, the area is subdivided into two series of trapezoids using the elevations of adjacent points and their projections on the centerline (the distances). (See figure 3-11.)



Legend:

℄	centerline

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

3-51. 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 in elevation 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 following steps for the computation are simple arithmetic—

- **Step 1.** Subtract the adjoining elevations.
- **Step 2.** Multiply by the distance from the centerline.
- **Step 3.** Add the multiplied results and list plus and minus quantities.
- **Step 4.** Add these quantities.
- **Step 5.** Divide by 2.

3-52. The steps for completing the procedure for the double-meridian triangle method follow:

- **Step 1.** Start at the centerline ground or grade elevation, whichever is lower. (This is Point A. See figure 3-11, page 3-11.) Work from the centerline in a clockwise direction to the left (Point A, B, C, D, E, and F) and counterclockwise to the right (Point A, G, H, I, and F) to the centerline ground or grade elevation, whichever is higher (Point F).
- **Step 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 column.
- **Step 3.** Divide the algebraic sum of the plus and minus quantities by 2 to obtain the cross-section area in square feet. In sections with cut and fill, treat each part as a separate section.

Example

The cross-section area is computed for plus quantities as follows:

$$Point\ A\ to\ point\ B\text{: } (64.4 - 64.3) \times (5 + 0) = 0.1 \times 5 = 0.5$$
$$Point\ C\ to\ point\ D\text{: } (65.2 - 63.3) \times (34 + 20) = 1.9 \times 54 = 102.6$$
$$Point\ D\ to\ point\ E\text{: } (66.2 - 65.2) \times 30 + 34) = 1.0 \times 64 = 64.0$$
$$Point\ E\ to\ point\ F\text{: } (67.7 - 66.2) \times (0 + 30) = 1.5 \times 30 = 45.0$$
$$Point\ G\ to\ point\ H\text{: } (64.4 - 63.5) \times (37 + 15) = 0.9 \times 52 = 46.8$$
$$Point\ H\ to\ point\ I\text{: } (66.2 - 64.4) \times (0 + 30) = 1.5 \times 30 = 45.0$$
$$Total\ of\ plus\ quantities = 424.5$$

The cross-section area is computed for minus quantities as follows:

$$Point\ B\ to\ point\ C\text{: } (63.3 - 64.4) \times (20 + 5) = 1.1\ \times 25 = -27.5$$
$$Point\ A\ to\ point\ G\text{: } (63.5 - 64.3) \times (15 + 0) = 0.8 \times 15 = -12.0$$
$$Total\ of\ minus\ quantities = -39.5$$
$$Algebraic\ sum = 424.5 - 39.5 = 385.0$$
$$Area\ of\ section = 385.0\ divided\ by\ 2 = 192.5\ square\ feet$$

Computer-Aided Design

3-53. Very accurate measurements can be made if cross sections are digitized using computer-aided design. Cross sections are placed on a digitizing pad; points are plotted into the computer; and with one command, the area is calculated.

VOLUME DETERMINATION METHODS

3-54. An engineer can accomplish the necessary earthwork computations by using one of the following methods of volume determination:

- Average end-area method.
- Prismoidal formula method.
- Average-depth-of-cut-or-fill method.
- Grid method.

Average End Area

3-55. The average-end-area method is most commonly used to determine the volume bounding two cross sections or end areas of a prismoid. The volume is measured in cubic yards (1 cubic yard equals 27 cubic feet). Use the following formula:

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

where—

A = *area*
L = *distance between end sections A_1 and A_2 in feet*
V = *volume in cubic yards*

3-56. If cross sections are taken at full 100-foot stations, the volume in cubic yards between successive cross sections in square feet may be found using the following formula:

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

3-57. In either form, the formula is only accurate when A_1 and A_2 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

3-58. The prismoidal method is used where 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 what is required for most road and airfield construction. The prismoidal formula is—

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

where—

A_m = *area of the section midway between A_1 and A_2*
L = *distance between end sections A_1 and A_2 in feet*
V = *volume in cubic yards*

3-59. Determine A_m by averaging the corresponding linear dimensions of A_1 and A_2, and determine its area rather than averaging the areas of A_1 and A_2.

Average Depth of Cut or Fill

3-60. With the centerline profile and final grade established, earthwork could 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 in establishing the centerline profile, the instruments used, and the accuracy of field reconnaissance. However, the volumes obtained by this method are generally adequate for most military construction.

Table 3-1. Earthwork average cut or fill

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

1. The 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. The average amount of cut or fill.
3. The slope ratio. Column 2 gives the correct amount of earthwork when the side slopes have a 1:1 ratio. When the slope has a different ratio, an adjustment must be made. (See column 4.)

Column 1 ***Average amount of cut or fill in feet***	***Column 2*** ***Width of the base of the cut (or top of the fill) in feet***										***Column 3*** ***For each additional foot of width, add:***	***Column 4***	
	26	28	30	32	34	36	38	40	42	44		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

Note. The final answer obtained is for a 100-foot section. If the actual length of the cut or fill is not 100 feet, an adjustment must be made. For example, for an 85-foot section, multiply by 0.85; for a 50-foot section, multiply by 0.50.

3-61. The centerline profile of a road is typical of the entire transverse section because of narrow widths. Because of the greater width required on an airfield runway, the centerline profile may be misleading due to the typical conditions across the entire transverse width at that point. Therefore, earthwork quantities for airfields should be estimated mainly from cross sections. In the absence of sufficient time, the average-cut-or-fill method is better than none. Before using table 3-1, determine the—

- Average cut or fill amount.
- 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.
- Quantity to be added if the slope ratio on the sides is 1.5:1 or 2:1.

3-62. To use table 3-1 use the following steps:

- **Step 1.** Find the row in column 1 with the correct average amount of cut or fill.
- **Step 2.** Move horizontally to the right, and find the correct figure under the appropriate width of the base of the cut (or top of the fill) in column 2.
- **Step 3.** Make corrections to this figure using column 3 and column 4 if needed.
- **Step 4.** If the length is not 100 feet between the points considered, adjust the answer proportionately.

Grid

3-63. 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. If this is not possible, locate borrow pits at another area. The grid method is a convenient method of computing the borrow material available in a borrow pit.

3-64. In this method, stake out the area with a system of squares, referencing points outside the limits of work. The dimensions of the 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 sized so that no significant breaks in the original ground surface or in the pit floor exist between the corners of the square or the edges of the excavation and the nearest interior corner.

3-65. 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 3-12 shows a borrow pit with 25 squares staked. To identify the various intersecting points, label lines in one direction with numbers and in the other direction with letters. Thus, the intersection of lines C and 3 would be labeled C3.

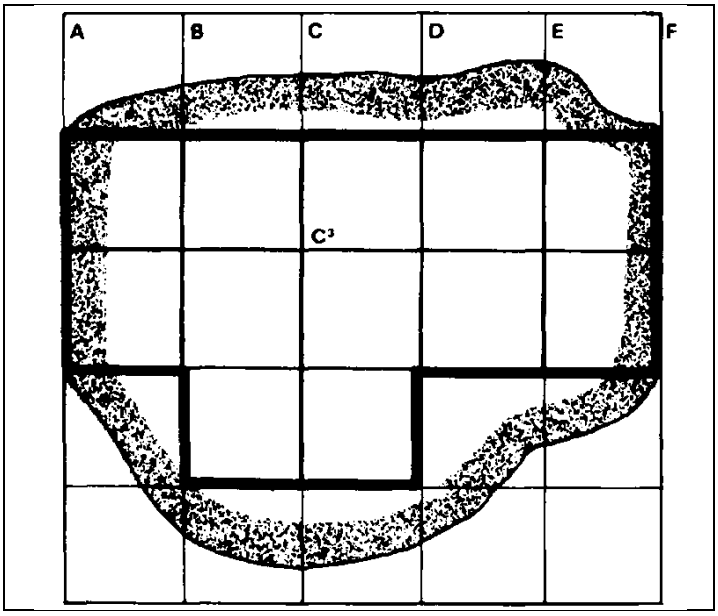


Figure 3-12. Computation grid system for a borrow pit

3-66. Outline squares falling completely within the excavation with a heavy line. Within that line, determine the volume of excavation for each square. Label the points on one square. (See figure 3-13, page 3-16.)

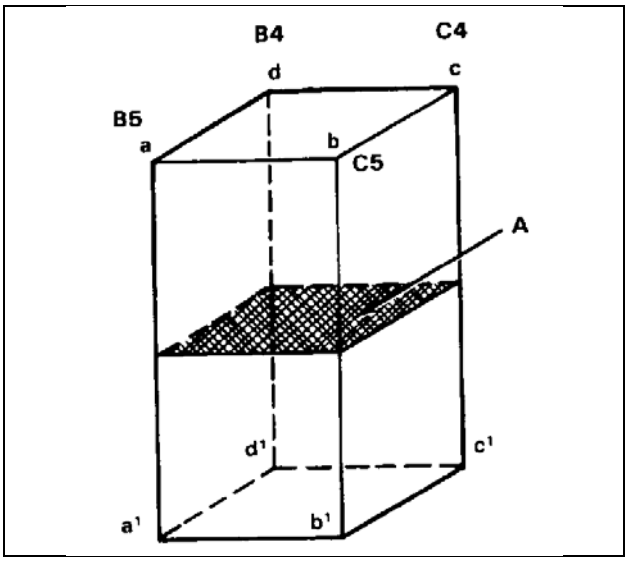


Figure 3-13. Excavation volume for one square

3-67. Points a, b, c, and d are on the original ground line, and points a^1, b^1, c^1, and d^1 are on the final ground line. The volume of the 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. Use the following formula:

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

where—

A	=	*cross-sectional area*
V	=	*volume*

3-68. 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 that gives the total volume, represented by the complete squares:

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

where—

A	=	*cross-sectional area*
h	=	*height*
V	=	*volume*
Σ	=	*sum*

Note. This could be approximated by adding the corner cuts and multiplying by A, or—

$$V = \frac{A}{27}\left(\Sigma h\right)$$

where—

A	=	*cross-sectional area*
h	=	*height*
V	=	*volume*
Σ	=	*sum*

3-69. 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 h_4 (refer to figure 3-12, page 3-15, and figure 3-13). 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 following formula:

$$V = \frac{A}{27}(\Sigma h)$$

where—

A	=	*cross-sectional area*
h	=	*height*
V	=	*volume*
Σ	=	*sum*

3-70. A grid is illustrated in figure 3-14. The length of the sides of each square is 50 feet. Use the following formula:

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

$$V = \frac{50 \times 50 \times 4.92}{4 \times 27} = 11{,}388\ cubic\ yards$$

where—

A	=	*cross-sectional area*
h	=	*height*
V	=	*volume*
Σ	=	*sum*

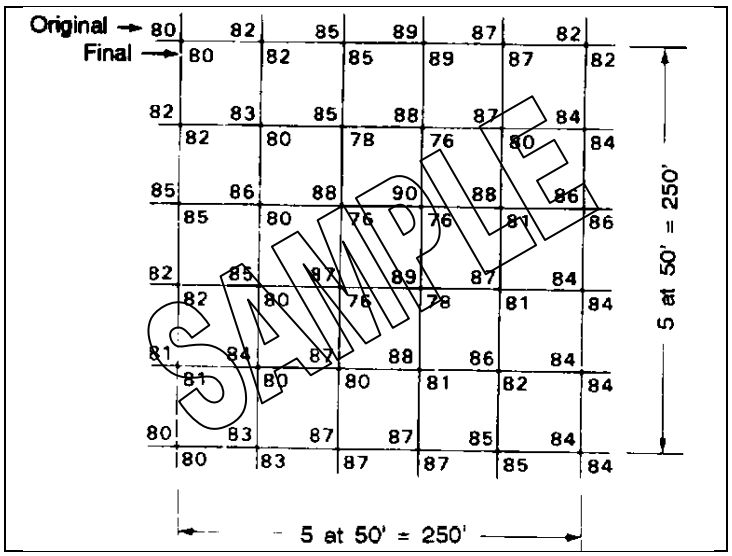


Figure 3-14. Sample grid-system work sheet

3-71. An alternative method is to compute the total of the cuts at each corner (123 feet), compute the average cut across the squares (123/25 = 4.92), and multiply by the length of the sides of the figure. Use the following formula:

$$V = \frac{4.92 \times 250 \times 250}{4 \times 27} = 11{,}388\ cubic\ yards$$

where—

V	=	*volume*

FACTORS INFLUENCING EARTHWORK CALCULATIONS

3-72. On many projects, one objective of the paper location study is to design the grade line so that the total cut within the limits of the work equals the total fill. The uncertain change in the material volume makes this difficult. It is usually more economical to haul excavated material to the embankment sections, eliminating borrow and waste.

Shrinkage

3-73. 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. The 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 to make the embankment.

3-74. 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) are called stripping.

3-75. Loose and swell refer to a condition that 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.

3-76. Shrinkage varies with changes in the soil constituents, 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.

3-77. 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

3-78. Compute the volume of cut and fill and the net volume between two points on the construction project. The net volume is the difference between the volume of cut and the volume of fill between two specified stations. The net volume may apply to the entire project or to a few stations. Net volume may be described as in-place, compacted, or loose. Table 3-2 provides conversion factors used to find the net volume.

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
	Compacted	1.11	1.39	
Clay	In place		1.43	0.90
	Loose	0.70		0.63
	Compacted	1.11	1.59	

Earthwork Volume Sheet

3-79. The earthwork volume sheets allow for systematically recording this information and making the necessary 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 (see figure 3-15, page 3-20) is divided into columns for recording and calculating information as follows:

- **Stations (column 1).** List station cross-sectional areas that have been plotted. Normally, these areas are taken at full stations and at intermediate stations that are required to fully represent the actual ground conditions and earthwork involved.
- **End area cut (column 2).** Record 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.
- **End area 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).

Note. This volume represents 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 compacted yardage rather than in-place yardage.
- **Stripping fill cut (column 6).** Before actual earthmoving operations begin, remove the layer of humus or objectionable material that lies on the ground surface.

Note. 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 the volume between stations of this humus material over sections of cut in column 6.

- **Stripping 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 the volume of this material between stations over sections of fill.
- **Net cut (column 8).** Indicate the volume of cut material between stations that is available for the 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 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 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 that the comparisons can be made. Changes in volume of earthwork are discussed in this chapter. (See table 3-2 for conversion factors.) Column 9 is column 8 multiplied by the appropriate conversion factor (0.9 in this case) to convert it from in-place yardage to compacted yardage.
- **Total fill (column 10).** Indicate 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 required to achieve the original ground line.
- **Algebraic sum (column 11).** Column 11 is the difference between column 10 and column 9. This indicates the volume of 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.

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

Legend:

BCY	banked cubic yardage
ft	feet
CCY	compacted cubic yardage
cy	cubic yards
sq	square

SAMPLE

Figure 3-15. Sample earthwork volume calculation sheet

MASS DIAGRAM

3-80. The first step in planning earthmoving operations is to estimate the earthwork quantities involved in the project. With these estimates, the engineer can prepare detailed plans for economical and efficient completion of the earthmoving mission.

3-81. 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 distances, and the average slope for each operation when combined with a ground profile. 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 types of projects. However, it is one of the most effective engineer tools and is easily and rapidly prepared.

CONSTRUCTION

3-82. Using column 1 (station) and column 12 (mass ordinate, cumulative total) of a completed earthwork volume sheet, a mass diagram can be plotted. (See 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 the plotted points to complete the mass diagram. Positive numbers are plotted above the zero datum line, and negative numbers are plotted below.

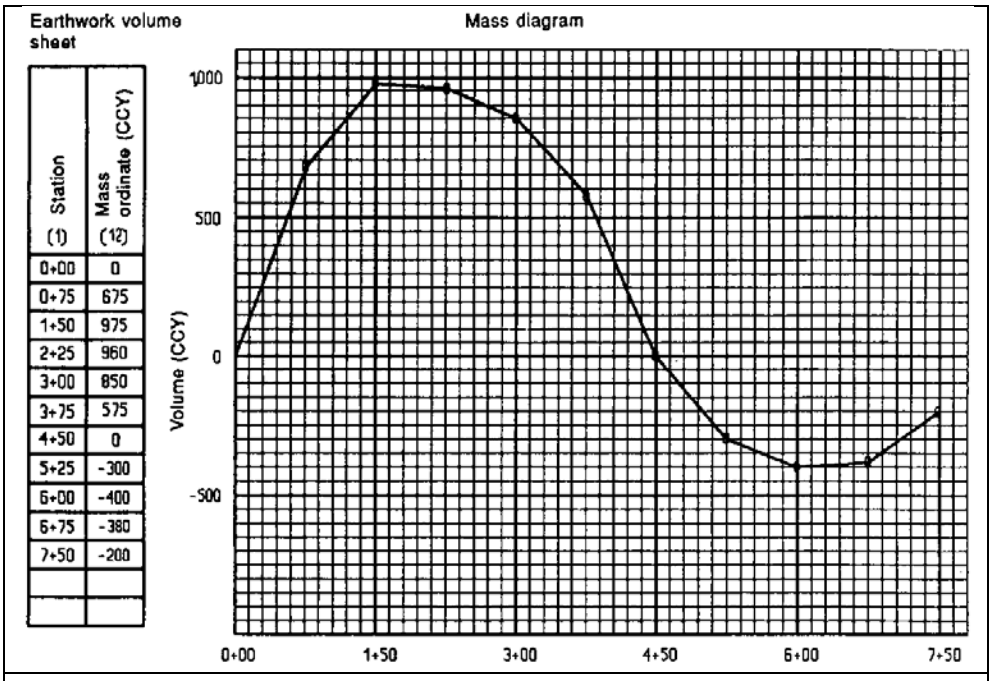


Figure 3-16. Plotting the mass diagram

Properties

3-83. Figure 3-17 shows a typical mass diagram with the actual ground profile and final-grade line of the project plotted. They use the same horizontal axis (stations). The ground profile is placed above the mass diagram to facilitate the calculation of the average grade the equipment will work. The horizontal axis is the only thing these graphs have in common.

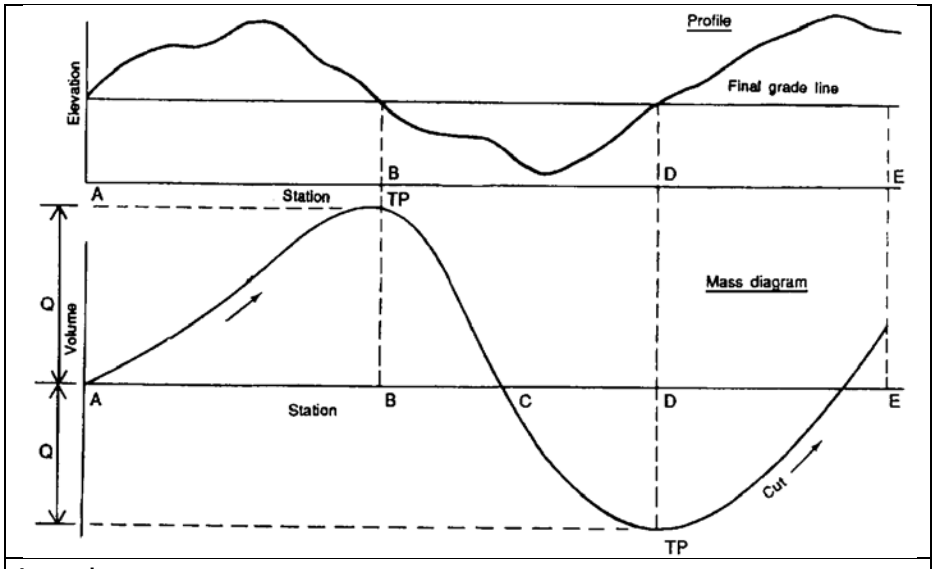


Figure 3-17. Properties of a mass diagram

3-84. The mass diagram is a running total of the quantity of earth that is in surplus or deficient along the construction profile. If more material is being cut than filled at one station, it is 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. In figure 3-17, page 3-21, Stations A to B and Stations D to E are cutting operations. The total volume for the cut at Stations 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.

3-85. If more material is being filled than cut at one station, it is a fill operation at that station. The deficient material quantity or volume will be increasing as filling operations continue through the station, producing a descending mass-diagram curve line. In figure 3-17, Stations B to D are filling operations. 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.

3-86. The maximum or minimum point on the mass diagram where the curve changes from rising to falling indicates a change from cut to fill (or vice versa). This is referred to as a transition point. On the ground profile, the grade line crosses the ground line at the transition point, as illustrated at Stations B and D.

3-87. 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.

3-88. The final position of the mass diagram line (above or below the datum line) indicates if the project was predominately a cut or fill operation. In figure 3-17, where the mass diagram ends at Station E, the operation was a cutting operation; 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.

Project Analysis

3-89. Once the basic properties of the mass diagram are understood, the engineer can conduct a detailed analysis to 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 operating equipment beyond this point would not be efficient. The maximum haul distances (balance-line lengths) are up to 300 feet for a dozer; 301 to 5,000 feet for a scraper; and 5,001 feet to several miles for a dump truck.

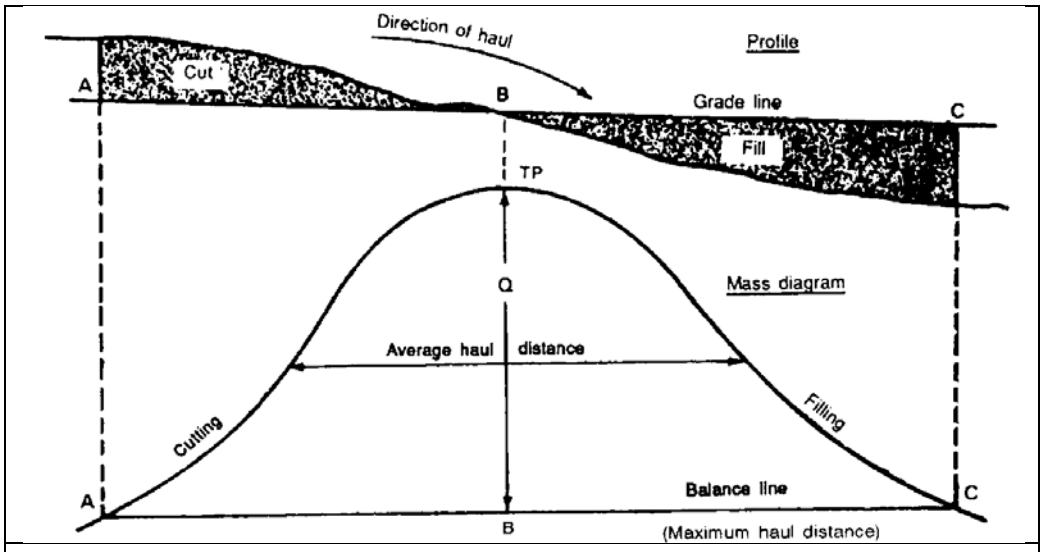
3-90. These lengths are measured using the horizontal scale (stations measured in hundreds of feet). Always use the maximum haul distance (length) of each piece of equipment, provided the project or node is at least that length. Start excavating each node with the dozer, followed by the scraper and dump truck.

3-91. Figure 3-18 shows a balance line drawn on a portion of a mass diagram. If this were a dozer balance line, the distance between Stations A and C would be 300 feet.

3-92. The cut equals the fill between the ends of the 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, indicating an exact balance of earthwork.

3-93. In figure 3-18, the vertical line labeled *Q* measures the quantity of material made available by cutting between Stations A and B. This is also the amount of embankment material required between Stations 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*.

3-94. In accomplishing balanced earthwork operations 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 is the length of the horizontal line placed midway between the balance line and the top or bottom point (transition point) of the curve and is found by dividing the vertical distance of the quantity in half.



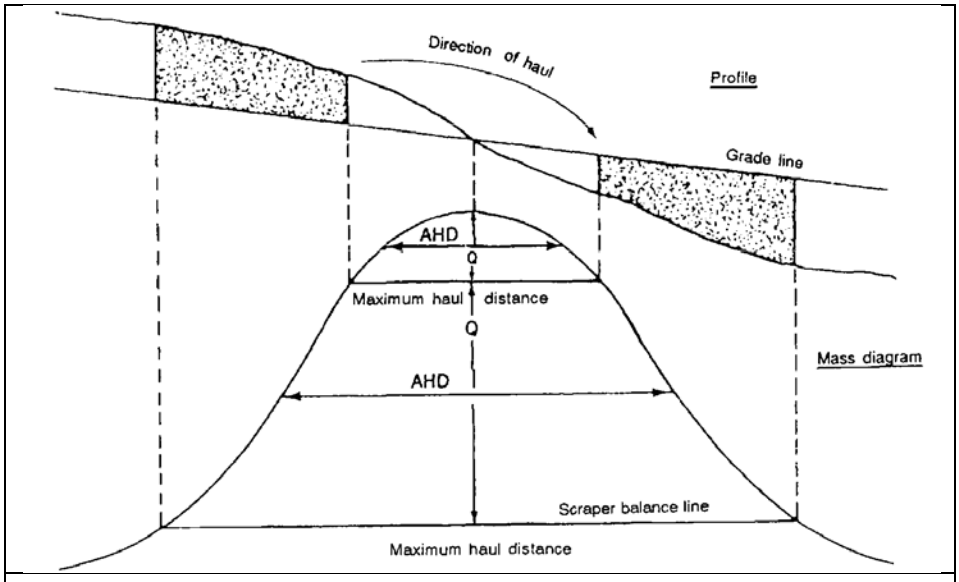
Legend:

Q	quantity of material
TP	transition point

Figure 3-18. Mass diagram with a balance line

3-95. 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.

3-96. Figure 3-19 shows a part of a mass diagram that two balance lines have been drawn. The same principles apply for the area between the lines with one balance line. The quantity balanced is the vertical distance between the balance lines, 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 on the position of the curve with respect to the balance lines.



Legend:

AHD	average haul distance
Q	quantity of material

Figure 3-19. Mass diagram with two balance lines

3-97. 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 multiplied by the 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. 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. (See figure 3-20.)

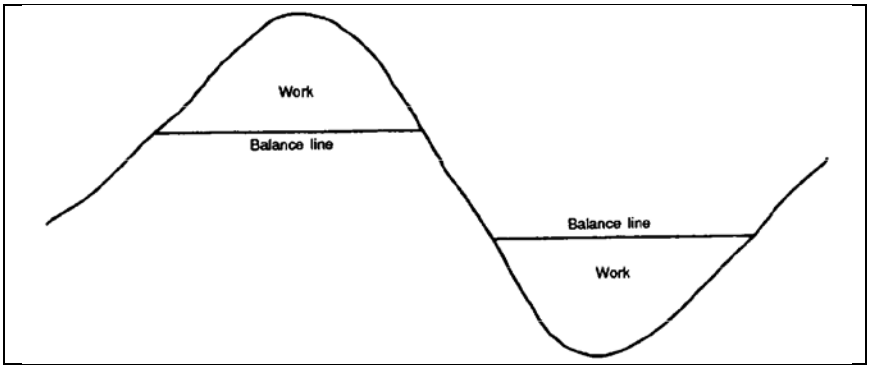


Figure 3-20. Work in earthmoving operations

3-98. Another item calculated from the mass diagram is average grade. This value is used when computing equipment scheduling and use. Figure 3-21 illustrates a portion of the mass diagram where the average grade has been determined.

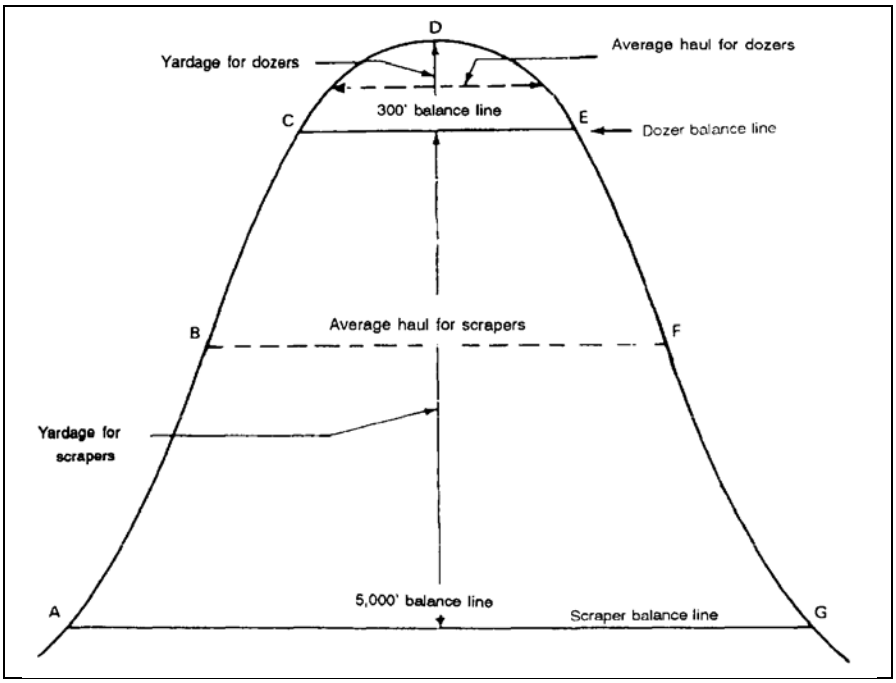


Figure 3-21. Balance lines for equipment efficiency

APPLICATION

3-99. The mass diagram is used to find the cost of a project in terms of haul distance and yardage, to locate the areas of operation for various types of equipment, to establish the requirements for borrow pits and waste areas, and to provide 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 on the proper location of balance lines.

3-100. 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.

3-101. Figure 3-21 illustrates a portion of a mass diagram where two balance lines have been drawn: 300 feet for dozer capabilities and 5,000 feet for scraper capabilities. The following job analysis can be made from the diagram:

- 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. The average haul distance is the horizontal line midway between the balance lines. The vertical line indicates the amount of earthwork.

3-102. To determine the average grade for the scraper or dozer work area, use the following steps:

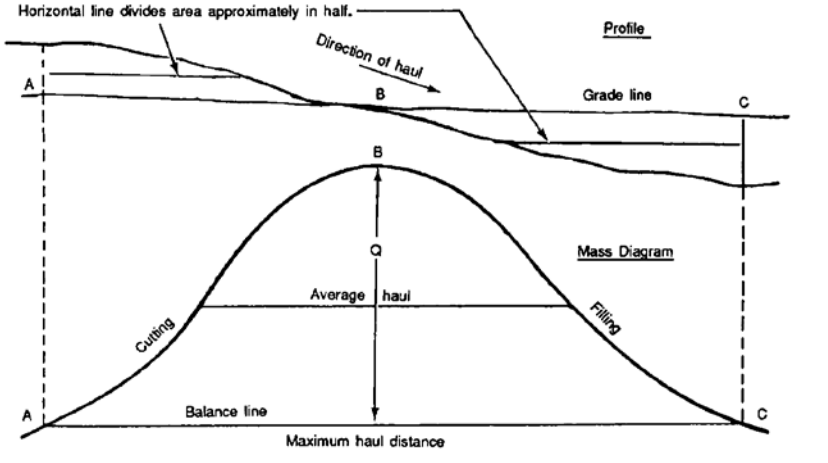
- **Step 1.** Draw a horizontal line through the work area that roughly divides the area in half on the profile. (See figure 3-22.) This is a rough estimate.
- **Step 2.** Extend a vertical line from the end points of the average haul line up through the project profile. (See figure 3-23, page 3-26.) These lines are referred to as the average haul vertical.
- **Step 3.** Draw a final line connecting the intersecting points of the lines drawn in Steps 1 and 2. This line represents the average grade.
- **Step 4.** Determine the average change in elevation (vertical distance between the cut and fill).
- **Step 5.** Calculate the average grade using the following equation:

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

Using figure 3-23, the average grade for the dozer would be—

$$Average\ grade\ \% = \frac{18\ feet}{103\ feet} \times 100 = -8.87\%$$

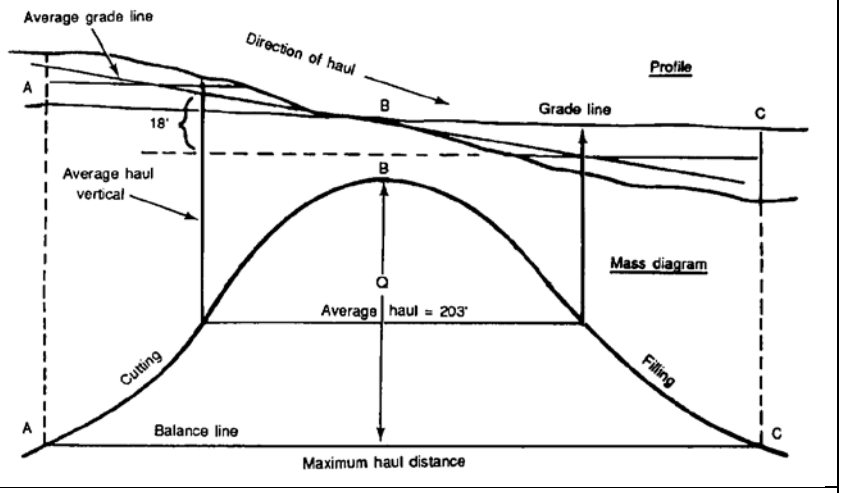
Note. Since this is an operation that moves earth downhill, the grade would be negative or -8.87 percent. An uphill cut would have a positive grade.



Legend:

Q	quantity of material

Figure 3-22. Determining average grade, Step 1



Legend:

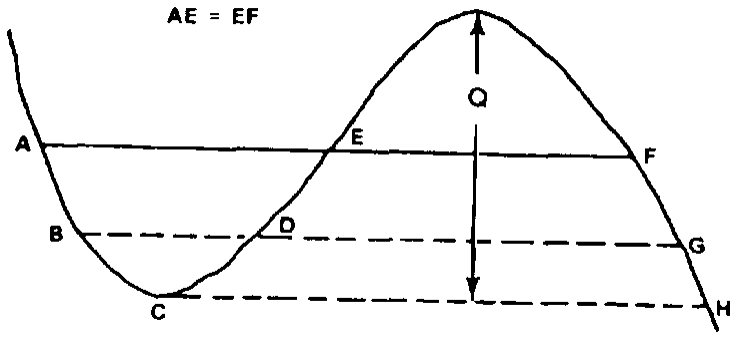
Q	quantity of material

Figure 3-23. Determining average grade, Step 2

Placement of Balance Lines to Minimize Work

3-103. 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 if possible. However, the method used to minimize the area depends on the shape of the mass diagram and the number of adjacent nodes that can be used.

3-104. 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 distance for equipment. The best placement of balance lines on the portion of a mass diagram would be equal lines. (See figure 3-24.) Only one balance line (CH) may be needed if it is within efficient haul distance specifications. The quantity of materials (in yards) involved is indicated by line Q, and the work involved would be the area above line CH.



Note. The best placement of balance lines would be lines AE and EF.

Legend:

Q	quantity of material

Figure 3-24. Minimizing work with two nodes

3-105. If two balance lines—BD and DG (and BD is less than DG) replaced this one balance line—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 the point where AE equals EF.

3-106. If there is an even number of adjacent nodes, work is reduced when the balance lines are one continuous line (AB + CD + EF = BC + DE + FG). (See figure 3-25.) The length of each balance line must be within equipment maximum haul capabilities as defined earlier.

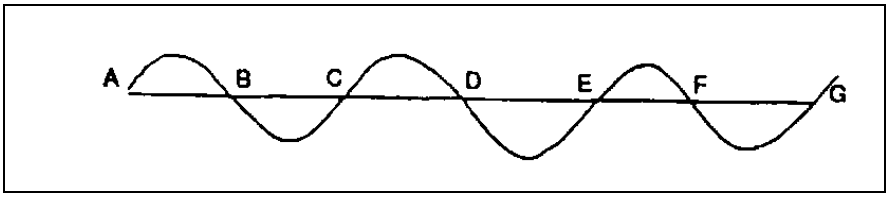


Figure 3-25. Minimizing work with an even number of nodes

3-107. If there is an odd number of adjacent nodes, 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. (See figure 3-26.) Balance lines must be within equipment limits.

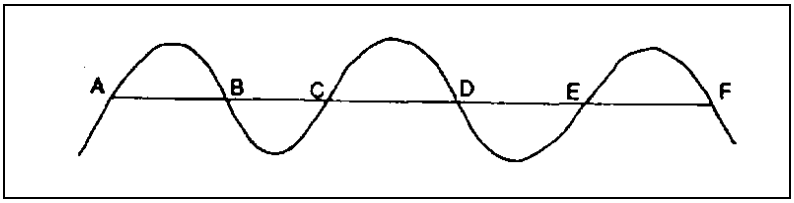


Figure 3-26. Minimizing work with an odd number of nodes

Calculation of Earthwork Not Within Balance Lines

3-108. 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. (See figure 3-27.) Concentrate necessary borrow and waste operations in one general area.

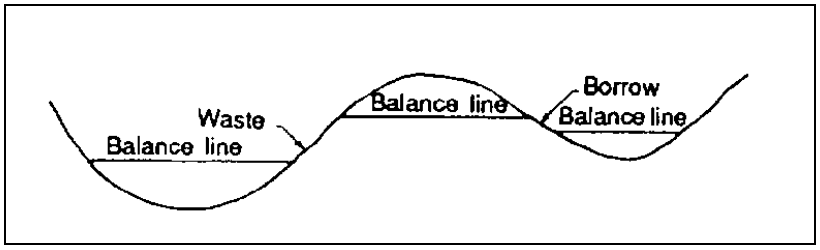
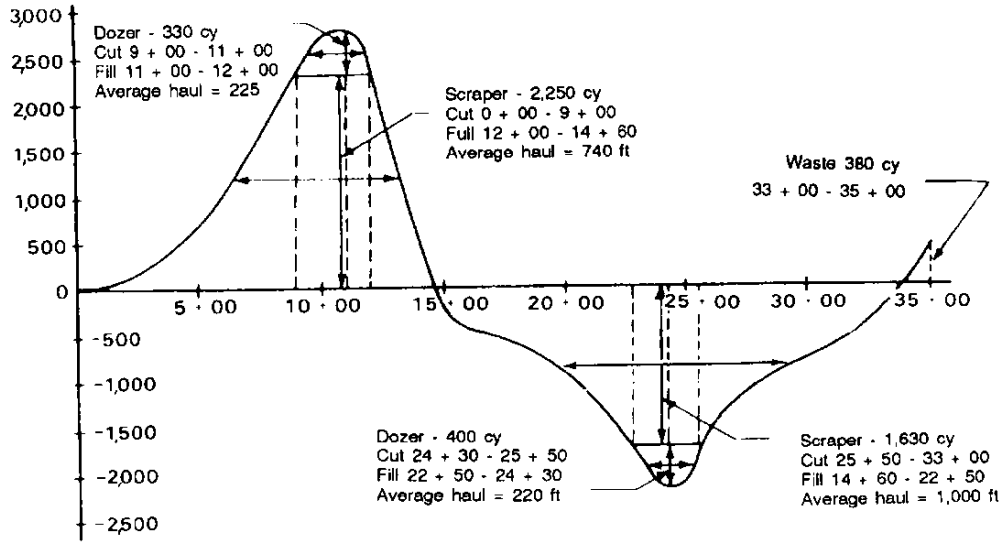


Figure 3-27. Waste and borrow on a mass diagram

Analysis Format

3-109. The simplest and most practical method of tabulating the results of a mass diagram is to write the quantities and distances on the diagram. (See figure 3-28, page 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. (See figure 3-29, page 3-28.)



Legend:

cy	cubic yards
ft	feet

Figure 3-28. Mass diagram showing analysis results

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	200 cy/hr	4	2.8 hr
12 + 00 – 14 + 60	Fill						
9 + 00 – 11 + 00	Cut	300	225 -2%	Dozer	370 cy/hr	1	0.8 hr
11 + 00 – 12 + 00	Fill						
14 + 60 – 22 + 50	Cut	1,630	1,000 -1%	Scraper	220 cy/hr	4	1.9 hr
25 + 50 – 33 + 00	Fill						
22 + 50 – 24 + 30	Cut	400	220 -1%	Dozer	420 cy/hr	1	1.0 hr
24 + 30 – 25 + 50	Fill						
33 + 00 – 35 + 00	Waste	380					

Legend:

cy	cubic yards
hr	hour

SAMPLE

Figure 3-29. Sample mass-diagram analysis sheet

MASS DIAGRAM LIMITATIONS

3-110. The mass diagram has many limitations that preclude its use in earthmoving operations. At best, it is merely a guide indicating the general manner in which the operation should be controlled. An attempt to get exact quantities and distances from it may be misleading, but it is a good starting point.

3-111. The mass diagram is most effective when used to plan operations along an elongated project (road, airfield runway, 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.

3-112. The mass diagram is used to analyze the balancing potential within one 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. It may be better to balance yardage with an adjacent taxiway where the haul distance will be 1,200 feet. The mass diagram can deal with the runway or the taxiway, not both simultaneously.

3-113. The mass diagram assumes that material excavated in the cut sections is acceptable for use in the embankment sections. This is not necessarily true. However, unacceptable quantities can be eliminated from the earthwork table.

3-114. 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.

This page intentionally left blank.

Chapter 4

Clearing, Grubbing, and Stripping

Land clearing is the removal and disposal of vegetation, rubbish, and surface boulders embedded in the ground. In the JOA, land clearing 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 the method used are the acreage to be cleared, the type and density of vegetation, weather conditions, the effect of the terrain on equipment operation, 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 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

4-1. 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.

4-2. 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.

TEMPERATE FORESTS

4-3. Temperate forests contain softwood and hardwood trees. Hardwoods are dominant where soil is old, deep, and fertile. Softwoods are dominant where soil is 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 clearing operations. Root systems vary according to geologic conditions and tree species. The types of root systems typical of various tree species are listed in table 4-1, page 4-2.

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

RAIN FORESTS

4-4. 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 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

4-5. Monsoon forests occur in climates of heavy seasonal rains with strong, warm winds. The forests are dense, with varied species of hardwoods that are moderate-sized, broad-leaved, and have shallow root systems. The undergrowth is very dense with shrubs, vines, and plants.

DRY FORESTS

4-6. Dry forests occur in arid and tropical regions where there is little precipitation. The forests are 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

4-7. An investigation of the geologic conditions of a forest can help when estimating the density and depth of tree root systems. The investigation should be concerned with hardpan, marshy, and permafrost conditions.

HARDPAN OR ROCK

4-8. Where a forest is closely underlain 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 form large, deep taproots that make uprooting difficult.

INUNDATED, MARSHY, AND BOGGY AREAS

4-9. In these areas, trees have thick, wide-spreading, and shallow root systems near the surface of the ground.

PERMAFROST

4-10. 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. Taproots develop where permafrost is deeper.

PREPARATION

4-11. Taking time to understand the site conditions before committing equipment and personnel to a clearing, grubbing, and stripping task will allow mission planners to provide the best available resources to the given tasks. These preparations can be very critical to planning and executing mission timelines for what is typically the first in a series of construction activities.

RECONNAISSANCE AND PLANNING

4-12. 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, the surface, and aerial and ground reconnaissance (refer to chapter 2 of this manual for more detailed information).

4-13. After the information has been verified, estimate the quantity of work, select the available equipment, determine the number of personnel needed, and plan a sequence of operations to complete the clearing rapidly and efficiently. In 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

4-14. Timber cruising is performed to estimate the diameter, height, and number of each tree species in an area. It is used to determine the quantity of usable timber or to estimate the amount of work required in clearing. A sample forested area (usually 10 percent of the total forested area) is cruised, and the result is applied to the entire area. The sample size may be increased or decreased. In small areas, a 100-percent cruise is usually made.

4-15. In timber cruising for land clearing, record the tree diameters at breast height, 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 tree sizes and species.

CLEARING CONSIDERATIONS

4-16. Properly executed site clearing operations require careful consideration of the effects of this activity on the site and surrounding mission area. The following paragraphs discuss the most prominent of these potential considerations.

PERMAFROST

4-17. Clearing of ground cover over permafrost that is near the freezing point may result in thawing of material, causing considerable ground-surface subsidence.

SAFETY

4-18. 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.

4-19. 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. Equipment used in clearing should, if practical, be equipped with heavy steel plating for protection of the undercarriages. This will prevent stumps, logs, and boulders from damaging vulnerable equipment parts.

CAMOUFLAGE

4-20. 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 to avoid breaking surrounding trees.

TIMBER SALVAGE

4-21. Trim 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

4-22. 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 original drainage

features as much as possible without disturbing natural grades. Slope drainage ditches downhill to allow water runoff.

4-23. 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 ponding on the surface. Backfill existing ditches at the latest possible time to permit the best use of the existing drainage.

DISPOSAL

4-24. Use waste areas or burning to dispose of cleared materials. The choice of method depends on the type of construction, environmental concerns, location, threat, and 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 clearing and grubbing. The disposal method should be consistent with the methods of camouflage, salvage, and drainage used for clearing. Cleared material can be disposed of by using it as fill material in revetments around hardstands when protective measures are needed.

Waste Areas

4-25. In airfield construction, consideration must be given to the areas used for disposal of construction waste. Waste material must not become an obstruction or hazard to air traffic. Approach and takeoff slopes must be considered when forming disposal areas.

Dumps Adjacent to Work Areas

4-26. 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 construction plans to determine where the least interference will be located.

Off-Site Areas

4-27. If construction debris cannot be piled up without causing drainage or other potential work impacts to the main project, it may be necessary to clear some adjacent land to dispose of the debris. 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 are used in clearing work areas.

Burning

4-28. 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 from headquarters.

4-29. 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

4-30. Before starting a fire, create a firebreak by stripping the area around the debris to be burned. If burning large areas, establish firebreaks on each side 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 firefighting equipment should be available to extinguish fires caused by flying sparks. Burning greatly reduces disposal volume, but large debris piles produce large fires that can be difficult to control once ignited. Be aware of the weather and wind conditions before burning debris. Gusty winds can spread fire into neighboring forests, resulting in uncontrollable fires.

Burn Pits

4-31. 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 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 into the pit because it smothers the fire and fills 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

4-32. If burn pits are not desirable, burn 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 maintain a good blaze.

4-33. To burn green, wet logs, it may be necessary to use fuel oil or diesel to furnish enough heat to dry out the logs and start the burning process. Pile the logs parallel, one on top of the other. A pipe with drilled or cut holes carries the fuel oil to the center of the pile. Once the pile is burning well, the fuel is stopped and the pipe is removed. Care must be taken to avoid ground contamination.

4-34. Fuel oil is also a quick and convenient means of starting brush fires, particularly if the brush is green and wet. If pushing material into the pile while the pipe is being used, bury the outside end of the pipe. This protects it from damage from tractor grousers and bulldozer blades.

Clearing and Piling Stumps

4-35. 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 the sun and wind so that the dirt will 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 burning.

AIRFIELD APPROACH ZONES

4-36. Obstructions that extend above the glide angle must be removed. Although glide-angle requirements may be met by topping trees, it is best to fell or uproot trees that extend above the glide angle. Demolished material may be left in place if conditions allow. Airfield glide angles and approach zones are further discussed in TM 3-34.48-2.

PERFORMANCE TECHNIQUES

4-37. The following paragraphs describe recommended techniques for clearing, grubbing, and stripping activities. Many times, equipment and personnel resources dictate which technique is used, but using the following guidance can significantly increase productivity.

CLEARING WITH EQUIPMENT

4-38. The use of engineer equipment is the most rapid and efficient clearing method. Unusually large trees, stumps, and terrain that decrease the maneuverability of equipment and increase maintenance requirements limit the use of equipment. This type of equipment includes bulldozers, tree-dozers, 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 TM 3-34.62/MCRP 3-17.7I.

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

Equipment	*Applications*	*Limitations*
Bulldozer	• Use for primary land clearing equipment. • Remove brush, trees, and stumps up to 6 inches in diameter with excellent results. • Push, pull, or skid cleared material for disposal.	• Trees over 6 inches in diameter require special and slower methods of removal by a bulldozer. • Maneuverability is limited in muddy or swampy terrain and in dense, heavy growth.
Tree-dozer, tractor-mounted unit	• Use for medium clearing of brush and trees at ground level rather than uprooting.	• Skilled personnel are required for cutting of trees; other units are required for completion of clearing when burning is not permitted.
Tractor-mounted clearing unit	• Use for extensive clearing operations requiring heavy pulling. • Uproot trees and stumps of almost unlimited diameters. • Skid cleared material for disposal. • Extricate mired equipment. • Operate in jungles, swamps, and bottom lands with heavy growth with excellent results.	• Skilled personnel are required for rigging. • Clearing an area is slow; other units are required for speedy completion. • The unit is not on the table of organization and equipment.
Winches (towing): Tractor-mounted	• Use for general light and medium pulling. • Uproot trees and stumps up to 24 inches in diameter. • Skid cleared material for disposal. • Extricate mired equipment.	• The pulling capacity is limited by the size of the tractor. • Terrain affects the maneuverability of the tractor.
Truck-mounted	• Use for expedient light pulling of trees up to 6 inches in diameter. • Skid small trees and brush. • Extricate mired equipment.	• Rigging personnel are required. • Terrain must be suitable for truck use. • Pulling capacity is too limited for most operations.
Felling equipment: Chain saw	• Control felling of trees of almost unlimited diameters. • Saw timber for salvage. • Use for rapid felling.	• Other units are required for uprooting stumps and disposing of felled timber. • Pneumatic saws are very dangerous to use on steep, rugged ground. Air hoses are frequently 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. They can be used with a reasonable degree of safety if handled by skilled operators.

Bulldozer

4-39. When clearing an area in dry or temperate forests, the bulldozer is most efficient 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 the primary unit of equipment in clearing operations.

4-40. 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 area to clear, select spoil areas for disposal of cleared material based on the shortest haul, a downgrade slope, effective camouflage, and general accessibility.

4-41. Start clearing at the disposal area and move in each direction away from it. Use one or two bulldozers to clear small trees and brush, and use another pair of dozers to remove larger trees and stumps bypassed by the other units. If necessary, add more bulldozers for a third cycle of operation to take care of the heaviest removals.

4-42. 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.

4-43. Multiple operations are possible when other types of equipment are available, using each type where it is most effective. For example, use power saws 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 TM 3-34.62/MCRP 3-17.7I.

Small Trees (Six Inches or Less in Diameter) and Brush

4-44. 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 be pushed into windrows for later removal or pushed off to one side of the area to be cleared.

Medium Trees (Six to Twelve Inches in Diameter)

4-45. To push over medium trees, set the blade of the bulldozer as high as possible to gain added leverage. (See figure 4-1.) As the tree falls, the bulldozer is backed up quickly to clear the roots. With the blade lowered, the bulldozer travels forward again and digs the roots free by lifting the blade. The felled tree is ready for removal to the spoil areas.

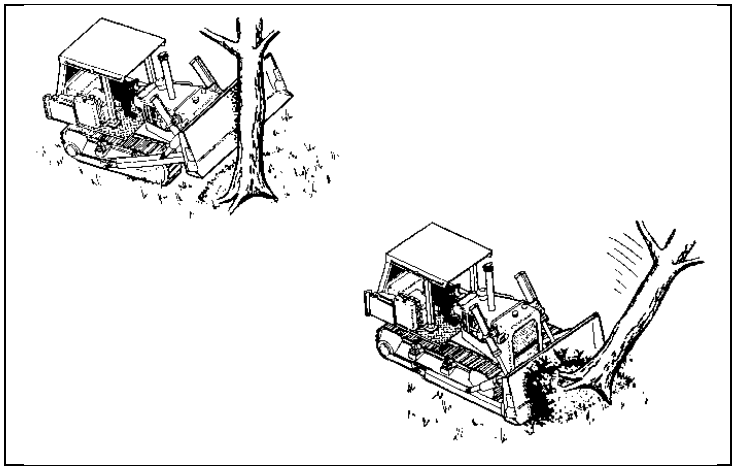


Figure 4-1. Bulldozer removing medium-size trees

Large Trees (Twelve or More Inches in Diameter)

4-46. Removing large trees is much slower and more difficult than clearing brush and small trees. (See figure 4-2.) First, gently and cautiously probe the tree for dead limbs that could fall. Then, position the blade high, and center the blade for maximum leverage. Determine the direction of fall before pushing the tree over; the direction of lean is usually the direction of fall. If possible, push the tree the same as a medium tree. If the tree has a large, deeply embedded root system, use the following steps:

- **Step 1.** Make a cut deep enough to cut some of the large roots on the side opposite the direction of fall. 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.** Build an earth ramp on the same side as the original cut to obtain greater pushing leverage. Then, push the tree over. As the tree starts to fall, reverse the tractor quickly to get away from the rising root 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.

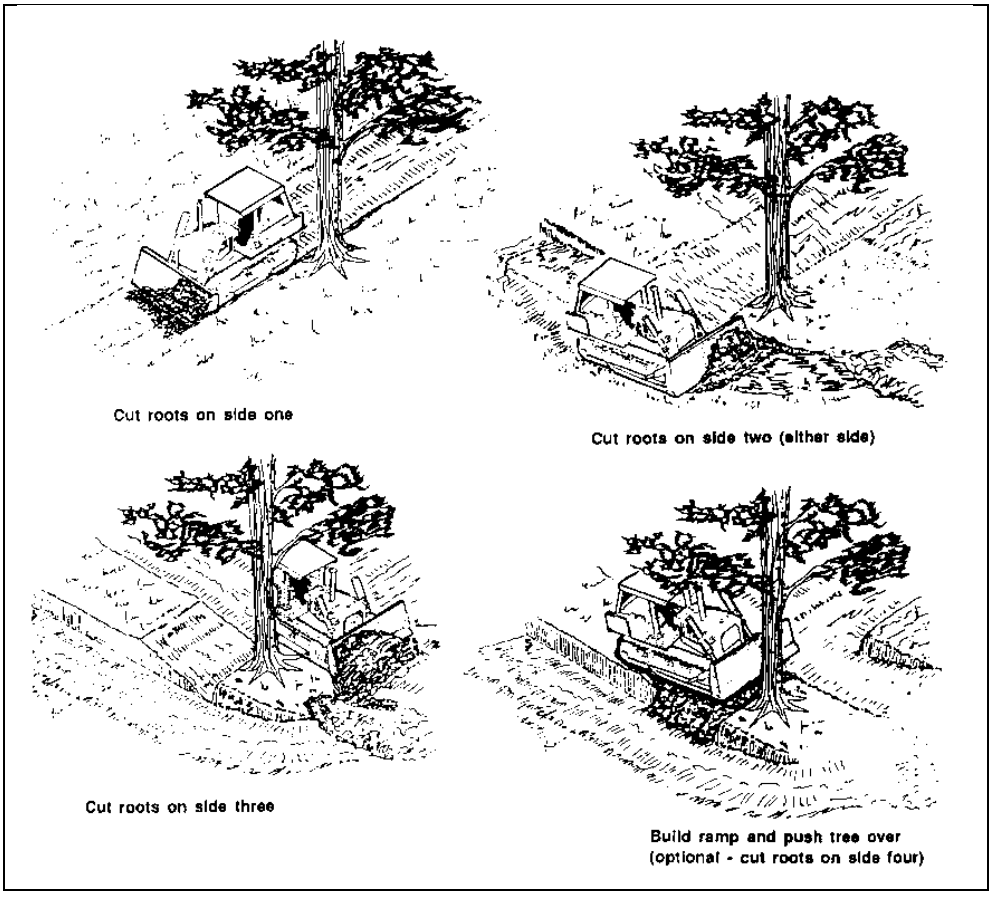


Figure 4-2. Four steps for removing large trees with a bulldozer

Tree-Dozer, Tractor-Mounted Unit

4-47. The tree-dozer is a tractor with a blade that splits 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. A steel canopy and a guide bar control the direction of falling trees and protect the operator.

4-48. The tree-dozer is a simple, efficient piece of equipment used for military land-clearing operations. It does not disturb the soil to a noticeable degree. 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 enemy capability of ambush.

4-49. Before committing a tractor equipped with a tree-dozer mount, investigate soil conditions in the area to determine if the soil can support the equipment. Use the tree-dozer mounting to make cuts through forestland but not 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: counterclockwise, clockwise, steep-slope, or level terrain.

Counterclockwise Method

4-50. When the tractor can move forward almost continuously, it shears anything in its path to ground level. 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. Sweeping with the tree-dozer mounting forms the final debris piles. Sweep one blade width at a time. Work from the center of each area, at a right angle to the border. (See figure 4-3.)

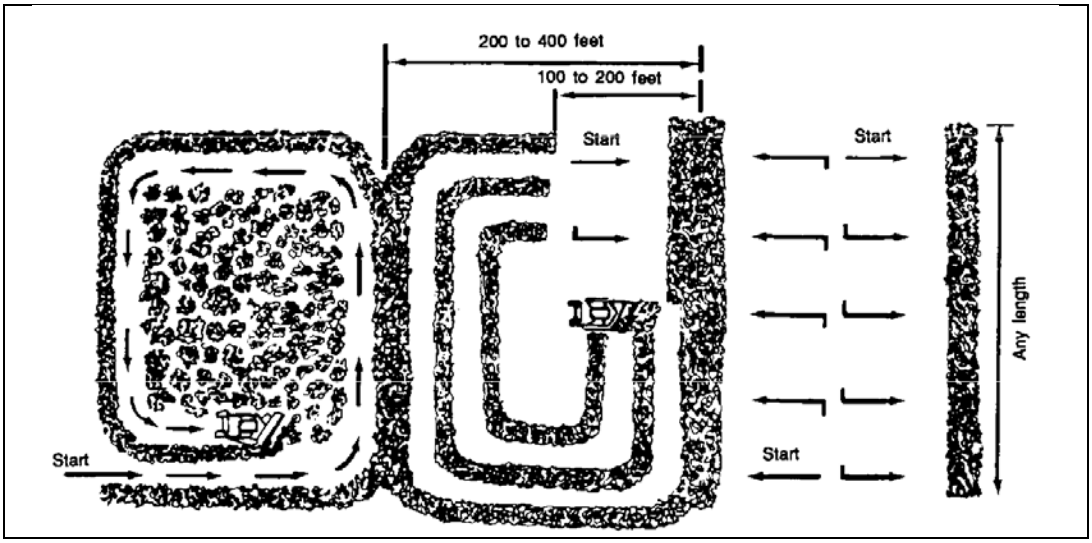


Figure 4-3. Counterclockwise method

Clockwise Method

4-51. 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. (See figure 4-4.) When windrowing, the operator keeps the cutting edge on the ground while pushing into the windrow and raises it when backing away. This allows accumulated soil to sift away and lessens soil deposits in the windrow.

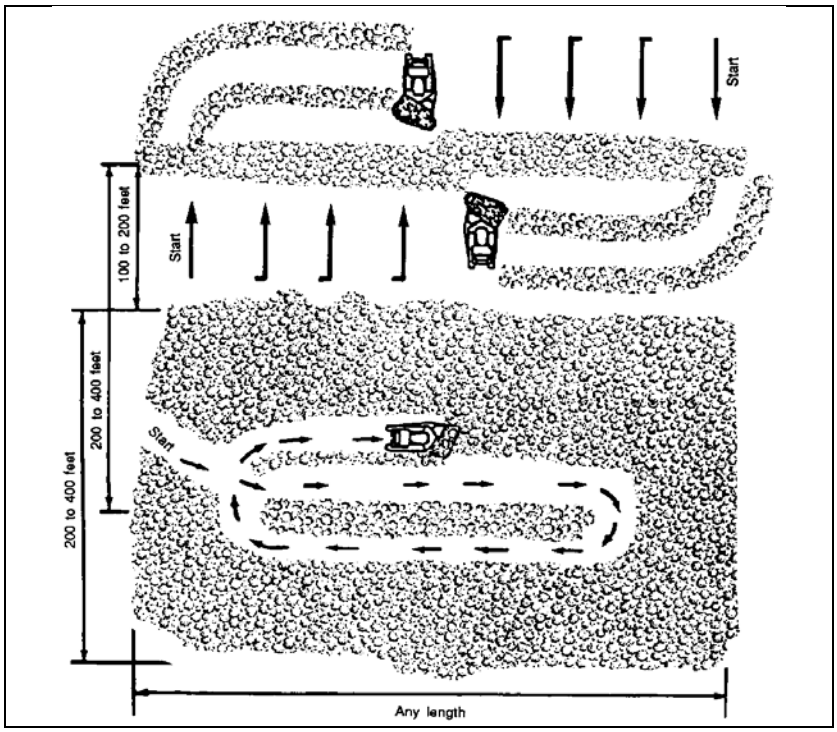


Figure 4-4. Clockwise method

Steep-Slope Method

4-52. On extreme slopes, rapid production is obtained by working in a semicircular pattern, from left to right, at approximate right angles to the windrow. (See 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.

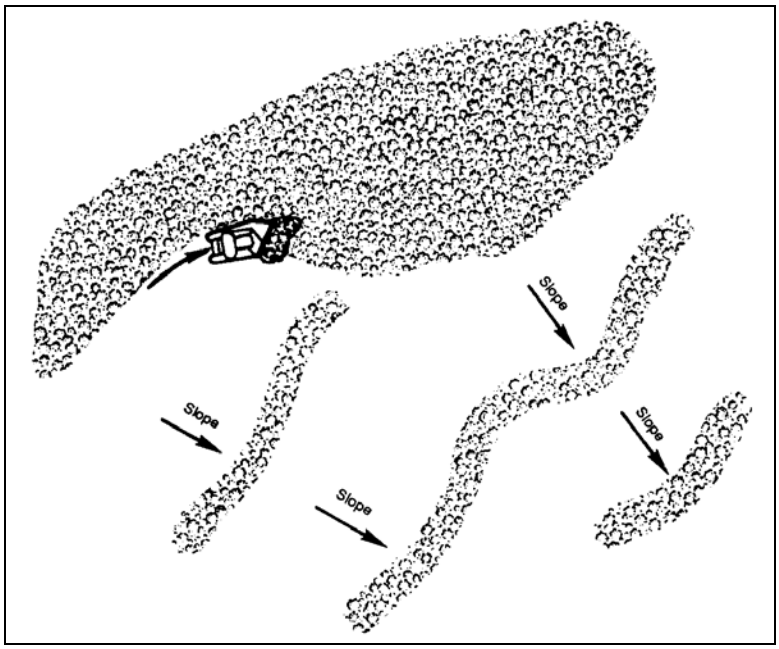


Figure 4-5. Steep-slope method

Level-Terrain Method

4-53. Where the vegetation is dense and small, cutting and windrowing simultaneously can obtain the highest production. Work from left to right at a 90° 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 moldboard.

4-54. 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, reducing the time lost in backing up. (See figure 4-6.) When the tractor reaches the previously cut material, the operator should deposit cut material and form another windrow.

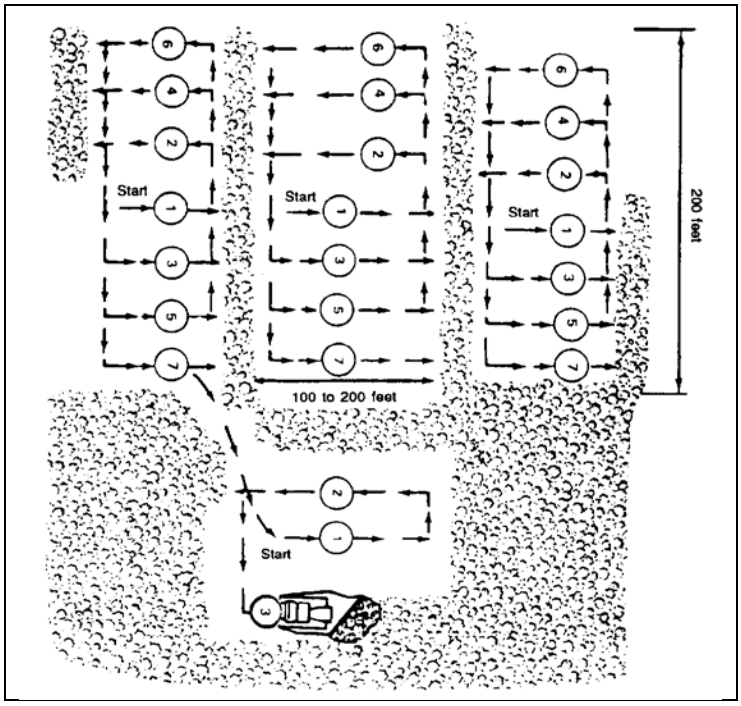


Figure 4-6. Level-terrain method

4-55. The operator works in patches, from inside to outside, in a counterclockwise direction and at right angles to the windrows. (See figure 4-6.) Sweeping and piling the resulting debris can be accomplished faster when tractors are used in teams traveling abreast.

Winches

4-56. Towing winches mounted on tractor-dozers or trucks are limited in use for clearing because of their small capacities.

Tractor-Mounted Winch

4-57. Use tractor-mounted winches for uprooting trees and stumps up to 24 inches in diameter, 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 about one-half the pull of a standard tree- and stump-pulling unit.

Truck-Mounted Winch

4-58. 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

4-59. 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. Hand clearing is usually slow and difficult for military requirements unless explosives or mechanical methods are used. This clearing method can be used with good results when labor is plentiful, forests are dense, and the terrain is rough. Powered equipment, such as chain saws and circular saws, are primary felling equipment.

Ripper

4-60. In land clearing, the ripper helps bulldozer removal operations 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 the rippers use to shallow root systems. Before stripping operations, the ripper is used to loosen and break up frozen soil or organic material for easier removal by graders or scrapers.

Grader

4-61. The grader is used to 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. Graders, used with rippers and bulldozers, can windrow the cleared material for removal by other equipment. The grader is extremely limited in most clearing operations.

CLEARING WITH EXPLOSIVES

4-62. Explosives may be used to fell standing trees, uproot entire trees and stumps, and remove and dispose of large boulders. Explosives, however, have several disadvantages. The sound of explosives can travel farther than the sound of construction equipment. In loose soil, the initial charge may be entirely expended in compacting the soil under a tree or stump; a second charge may be required to remove it. Often, deep taproots are only broken by explosives and have to be removed by hand. Also, explosives take time to place and create large craters that 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 3-34.214//MCRP 3-17.7L for the correct application of explosives and demolitions.

Trees and Stumps

4-63. Methods of tree and stump blasting vary with the size and condition of the tree, root structure, and ground conditions. Figure 4-7, page 4-14, shows the methods of placing charges to blast stumps with different root structures. Refer to table 4-1, page 4-2, for the root system type for several tree species in temperate forests.

4-64. 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.

4-65. Various tool types (pinch bars, earth augers, spoons) can be used when drilling holes for the charge. Wood augers can be used for taproots. For loading and tamping, a smooth, wooden pole about 5 feet long and 1 1/2 inches in diameter can be used (for example, the handle from a long-handled shovel; the crook of the blade end provides a good grip). Holes must be tamped firmly with earth to retain the full force of the explosion.

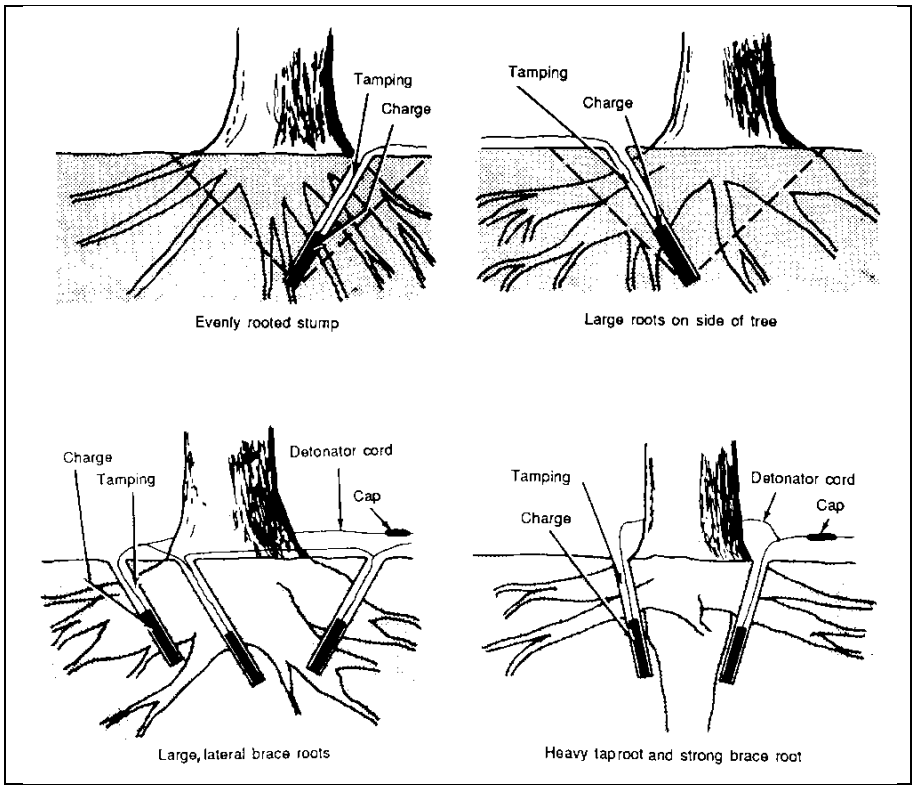


Figure 4-7. Stump-blasting methods for different root structures

Boulders

4-66. 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. Mud-capping, blockholing, or snakeholing may be used (refer to FM 3-34.214//MCRP 3-17.7L or TM 3-34.85/MCRP 3-17A for additional information). Refer to TM 3-34.65/NTRP 4-04.2.12/AFMAN 10-903 for quantities and types of explosives used and details regarding blasting rock.

REMOVAL OF SURFACE ROCKS

4-67. Surface rocks must be removed in certain types of construction. The following are the three methods used in this operation: hand, bulldozer, and crane or scoop loader. The choice of method depends on the situation.

Hand

4-68. When there is sufficient time and personnel, rocks are picked up and loaded into hauling units by hand. This method is slow and used in military construction as an expedient.

Bulldozer

4-69. The bulldozer is the most commonly used engineer equipment for moving rocks to a fill or disposal area. The rocks may be windrowed by bulldozers for later removal by scrapers or shovels and trucks. If the distance is short, bulldozers may push them to the designated disposal area.

Crane or Scoop Loader

4-70. 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 loads trucks quickly and efficiently. The rocks can then be hauled long distances for disposal.

STRIPPING

4-71. Stripping consists of removing and disposing of the topsoil and sod that cannot be used as 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 where 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 (dust control, erosion control, camouflage).

REMOVAL OF STRUCTURES

4-72. An airfield construction site may be acceptable except for obstructions (houses, railroads, power lines) on the proposed site or near the operation of aircraft. The primary selection of a site 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

4-73. Power lines obstructing forward area construction or glide angles should be removed. Sometimes the lines may be needed intact and a different approach to the airfield must be used. If the lines cross the runway, relocate the lines 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 support area, power lines that are not in an approach zone or high enough to extend into the glide angle when located in an approach zone may be marked with suitable warning lights.

Roads and Railroads

4-74. In general, roads and railroads near airfields are not obstructions if traffic does not interfere with aircraft approach or takeoff. Do not destroy paved main highways or 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 that supplies can be transported to the site.

Buildings

4-75. Buildings may be completely demolished with explosives or heavy equipment, leaving no salvage. Buildings may also be torn down in a way that conserves usable material or allows them to be relocated.

REMOVAL OF BURIED EXPLOSIVES

4-76. Mines, booby traps, unexploded ordnance, and other buried explosives must be located and removed or neutralized before construction operations begin. Check reports and data on the area to locate and identify explosives.

4-77. Establish the boundaries of the construction area, and 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.

4-78. The safety of personnel and equipment is primary during the removal of mines. The speed of clearance is secondary. When possible, use trained explosive ordnance disposal detachments especially when the size of the bomb precludes detonation in place.

4-79. The method of mine and unexploded ordnance removal is a command decision. For a minefield with booby traps or other antipersonnel devices, it is best to destroy the mines in place by explosives or mechanical means. Unexploded ordnance 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.

4-80. Surface mines or bomblets may be pushed from an area using a drag chain or bulldozer. If devices are destroyed in place, resulting holes and craters must be filled and compacted with acceptable material using bulldozers or graders. (See TM 3-34.62/MCRP 3-17.7I for additional information.)

Chapter 5

Subgrades and Base Courses

This chapter discusses the functions of the subgrade, subbase, and base courses. It also covers material selection and construction procedures. Refer to chapter 12 of this manual for the determination of the base- and surface-course thickness for roads and expedient construction methods in the JOA. Refer to TM 3-34.48-2 for similar information on classes of airfields and heliports and commonly used types of surface materials. For additional information on soil, compaction, and the California bearing ratio (CBR) requirement, refer to TM 3-34.64/MCRP 3-17.7G and TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034.

DESIGN CONSIDERATIONS

5-1. Pavement structures may be rigid or flexible. In rigid pavements, the wearing surface is made of cement. A rigid pavement made of cement will have great flexural strength, act as a beam, and bridge minor irregularities that may occur in the base of the subgrade it rests upon.

5-2. Other types of pavements are classified as flexible. In a flexible pavement, a distortion or displacement occurring in the subgrade is reflected in the base course and continues upward into the surface course. Flexibility denotes the tendency of courses in this type of structure to conform to the same shape under traffic.

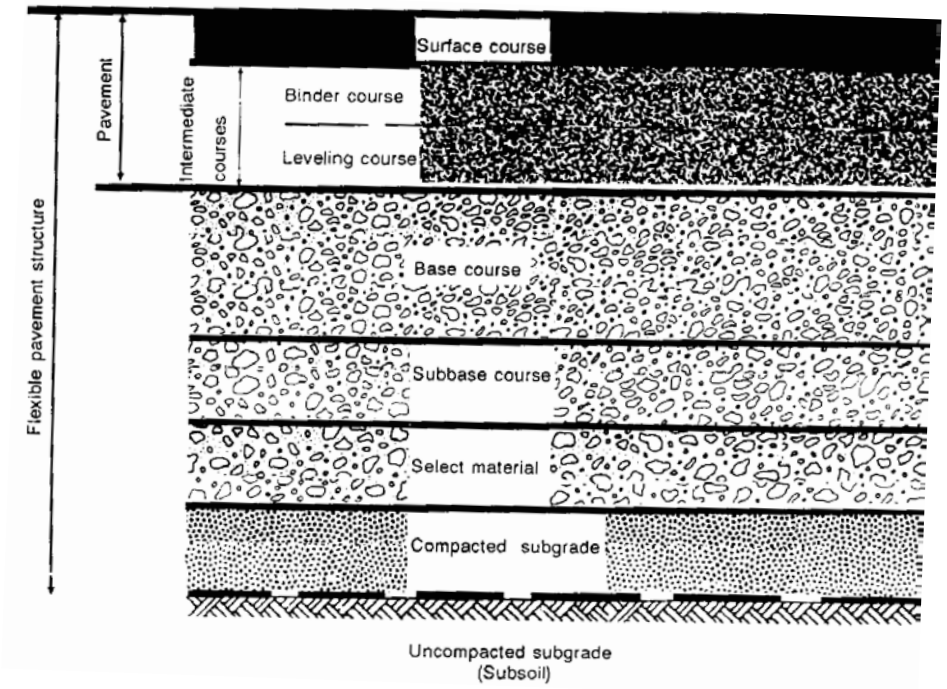
5-3. Flexible pavements are used almost exclusively in the JOA for road and support area airfield construction. They are adaptable to most situations, and they fall within the construction capabilities of the combat heavy engineer battalion and its support units. In most cases, the hot-mixed asphalt job mix formula is designed with the Marshall method, using the Marshall impact hammer. For road and street projects, the Superpave gyratory compactor may be used if available. Rigid pavements are generally not suited to construction requirements (unless the materials are readily available) and are not discussed in detail in this manual. (See TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034 for additional information.)

FLEXIBLE-PAVEMENT STRUCTURE

5-4. 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. The layers shown in figure 5-1 are not present in every flexible pavement. For example, a flexible-pavement structure may consist 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

5-5. Military engineers should classify soils according to TM 3-34.64/MCRP 3-17.7G, and then select representative samples for detailed tests. Detailed tests determine compaction characteristics, CBR values, and other properties 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 are 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).



Note. 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

LOAD DISTRIBUTION

5-6. 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 sections of flexible pavement, one with a thick flexible-pavement structure and one with a thin flexible-pavement structure. In both cases, the subgrade is the foundation that eventually carries the load applied at the surface. Figure 5-2 demonstrates that the stress magnitude on the subgrade decreases as the flexible-pavement structure is thickened. On the left diagram, the flexible-pavement structure is thick, the load at the subgrade level is spread over a wide area, and the stress on the subgrade is low. On the right diagram, the structure is thin, the load at the subgrade level is confined to a much smaller area, and the stress on the subgrade level is significantly higher. The pattern of decreasing stress with increasing depth is the basis of the conventional flexible-pavement design in which subgrade materials of low-bearing capacity are covered with thick flexible-pavement structures.

5-7. The distribution of pressures under a multiple-wheel assembly is shown in figure 5-3. 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.

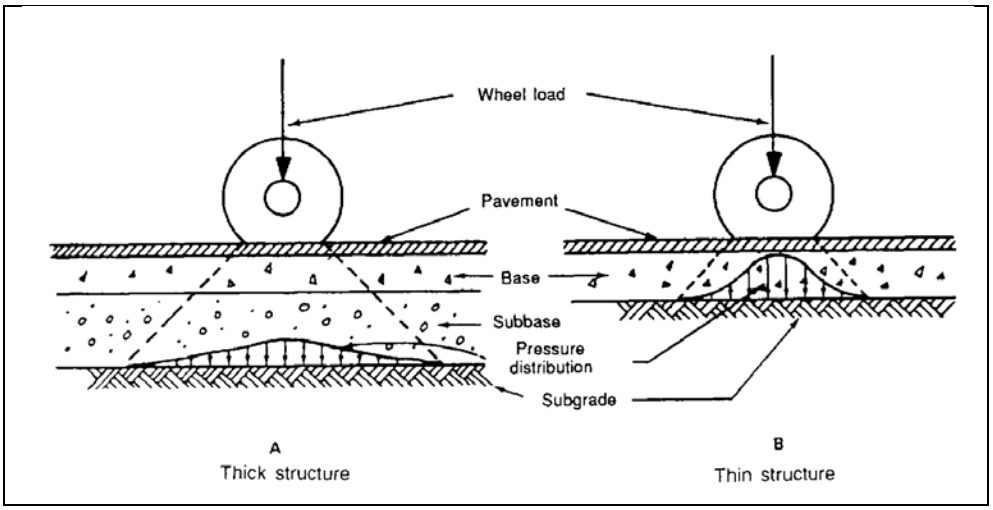
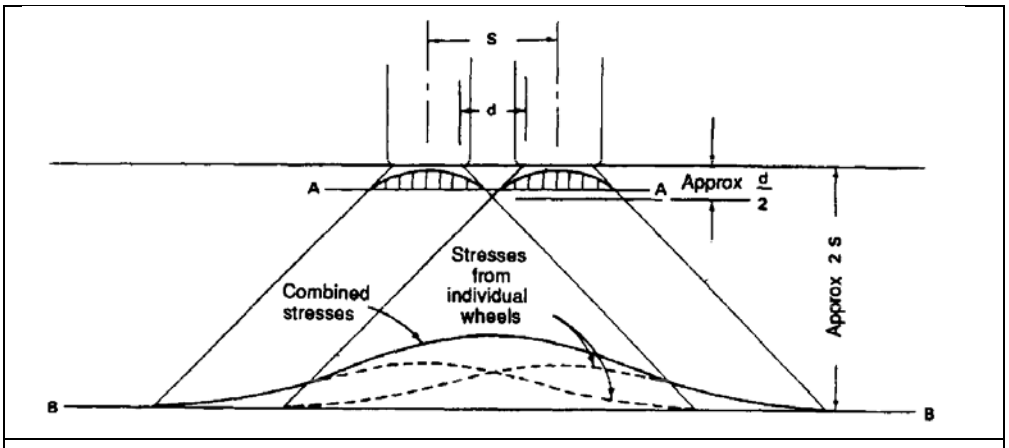


Figure 5-2. Distribution of pressures under single-wheel loads



Legend:

approx approximate

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

5-8. The tire-contact area and tire pressure affects the intensity of stress at a given point in a flexible pavement. 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 are seriously affected by high tire pressures.

SUBGRADES

5-9. Using information from a deliberate soil survey as outlined in TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034, consider the following factors when determining the suitability of a subgrade:

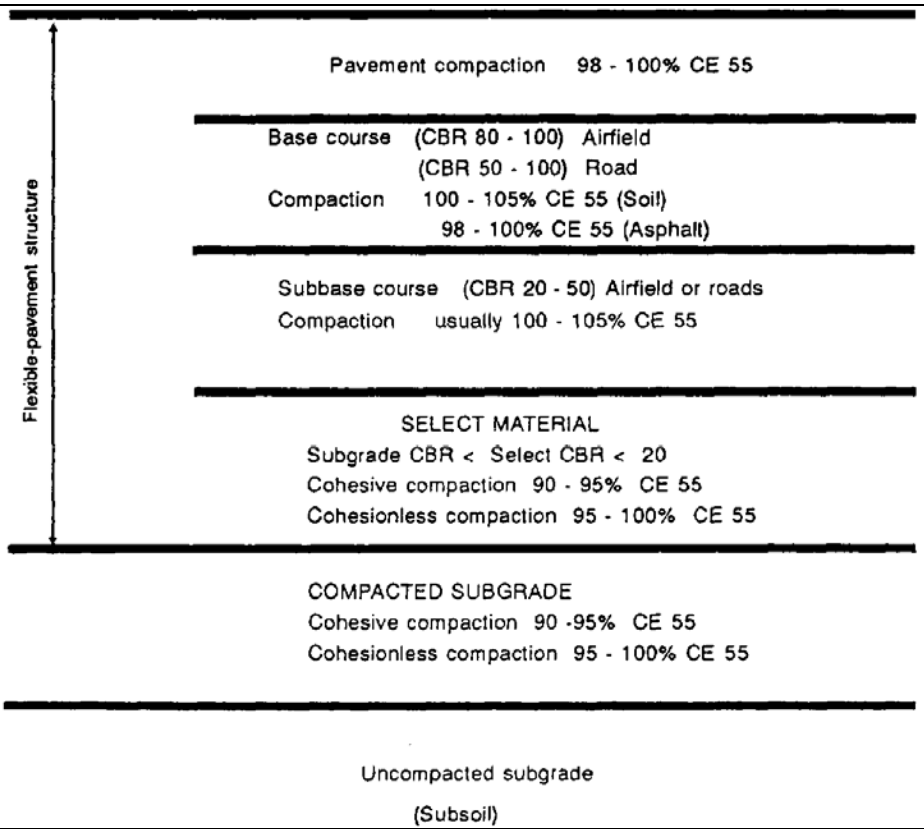
- General characteristics of the subgrade soils.
- Depth to the bedrock.
- Depth to the water table.
- Compaction that can be attained in the subgrade.
- CBR values of uncompacted and compacted subgrades.
- Weak or soft layers or organics in the subsoil.
- Detrimental frost action or excessive swell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

GRADE LINE

5-10. Classify the subgrade soil according to the Unified Soil Classification System (USCS) as described in TM 3-34.64/MCRP 3-17.7G, 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. Simplicity of construction must also be considered.

COMPACTION

5-11. Compaction normally increases the strength of subgrade soils. The typical procedure is to specify compaction according to recommended requirements. (See 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 less than 5.
2. A cohesionless soil is one with a PI greater than or equal to 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.

Legend:

CBR	California bearing ratio
CE	compactive effort
PI	plasticity index

Figure 5-4. Recommended compaction requirements for support areas

5-12. Compaction is relatively simple in fill sections because the layers are subjected to construction processes and can be compacted during construction. Compaction is more difficult in cut sections. During

construction, compaction must be obtained to a depth where the natural density of material will resist further consolidation under traffic.

5-13. Specific requirements for minimum depth of subgrade compaction for cohesive (clayey) and cohesionless (sandy) soils for road designs are described in chapter 12 of this manual. This same information for airfield designs is described in TM 3-34.48-2.

Typical Cases

5-14. 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. Initially, replace material in 6-inch lifts and 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 rollers.

5-15. Compare the native subsoil compaction with compaction requirements for the subgrade as described in chapter 12 of this manual and TM 3-34.48-2 to determine if consolidation of the subsoil is likely to occur under the design traffic load. If consolidation is likely to occur, compact the subsoil or design a thicker flexible-pavement structure to prevent subsoil consolidation.

5-16. Cohesive materials, including those of relatively low plasticity that show little swell tendency, should be compacted at the optimum moisture content determined from the density-moisture curves developed for that soil using the modified proctor test per UFGS-32 11 16.16. Cohesionless, free-draining materials should be compacted at moisture contents approaching saturation.

Special Cases

5-17. Although compaction normally increases soil strength, 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

5-18. The soil types 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 high degree by alternate cycles of wetting and drying or by geologic overburden loads. They have developed a definite internal structure with a high strength when left undisturbed. Scarifying, reworking, and rolling these soils in cut areas will reduce the soils load-bearing capacity.

5-19. When these soils are encountered, obtain CBR values for the soil in undisturbed and disturbed conditions. Compact the removed soil 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 required depth of compaction may govern the total thickness design above the subgrade instead of the CBR method. (See UFC 3-260-02 for additional information.)

Silts

5-20. The bearing capacity for silts, very fine sands, and rock flour (predominantly USCS ML and SC groups) is good if properly compacted within the specified moisture range. Compaction of silt, very fine sand, and rock flour in a high water table area can pump water to the surface. The material becomes very soft and loses a considerable amount of 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 air voids so that the available water fills the void space. Therefore, it is difficult to obtain the desired density in these silts and very fine sands at moisture contents greater than optimum.

5-21. Water from a wet, spongy silt subgrade often enters the subbase and base during compaction through capillary action. This additional moisture will 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 water is often difficult and, in some cases, impossible in the allotted construction period.

5-22. 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.

5-23. Do not disturb the subgrade where drainage is not feasible or a high water table cannot be lowered. Also, do not disturb the subgrade where soils become saturated from sources other than high water tables and cannot be dried, such as construction during wet seasons. Compaction of lifts during wet periods can cause fines from the subgrade to contaminate the upper layers of the flexible-pavement structure.

5-24. The pumping and detrimental actions previously described should be anticipated when 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

5-25. Soils are characterized as swelling if they display a significant increase in volume with the addition of moisture. These soils can cause trouble in a 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.

5-26. Preswelling is a common method for treating subgrade soils with expansive characteristics. The soil should be compacted at a moisture content where a 3-percent or greater swelling has 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 CALIFORNIA BEARING RATIO VALUES

5-27. The CBR test described in TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034 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 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. (See table 5-1.)

5-28. Subgrades can be stabilized mechanically (by adding granular materials), chemically (by adding chemical admixtures), or with a stabilization expedient (sand-grid, matting, or geosynthetics). Stabilization with chemical admixtures (lime, Portland cement, fly ash) 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 TM 3-34.64/MCRP 3-17.7G.

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

Maximum Permissible Value						
	Maximum Design CBR	*Size in Inches*	*Gradation Requirements Percent Passing*		*Atterberg Limits*	
			No. 10	*No. 200*	*LL**	*PI**
Subbase	50	3	50	15	25	5
Subbase	40	3	80	15	25	5
Subbase	30	3	100	15	25	5
Select Material	20	3	—	25	35	12
*Determination of these values will be made according to the ASTM for standard test methods for LL limit, plastic limit, and plasticity index of soils						
Legend:						
ASTM	American Society for Testing and Materials					
CBR	California bearing ratio					
LL	liquid limit					
No.	number					
PI	plasticity index					

5-29. 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 not be considered a select material.

5-30. 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 manmade stabilizers are sand grid, matting, and various types of geosynthetics, especially geotextiles. Matting and sand grid are expedient methods of stabilizing loose sands 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

5-31. In areas subjected to seasonal freezing and thawing, subgrade materials may exhibit frost heave and thaw weakening. Table 5-2, page 5-8, lists the frost-susceptibility ratings of soils. Materials with 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.

5-32. Select and subbase materials that 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 that have more than 20 percent fines (particles passing the number 200 sieve). Materials categorized as F1, F2, F3, and F4 in table 5-2, are unsuitable as base or subbase materials in seasonal frost and permafrost areas. Materials categorized as F1 and F2 may be suitable as a select material in seasonal frost and permafrost areas, provided the plasticity meets the requirements of table 5-1.

SELECT MATERIALS AND SUBBASE COURSES

5-33. When designing flexible pavements, locally available 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.

Table 5-2. Frost-design soil classification

Frost Group	*Type of Soil*	*Percent By Weight < 0.02 mm*	*Typical Soil Types Under the USCS*
NFS	Gravels (e ≥ 0.25)	0–3	GW, GP
	Crushed stone	0–3	GW, GP
	Crushed rock	0–3	GW, GP
	Sands (e ≤ 0.30)	0–3	SW, SP
	Sands (e > 0.30)	3–10	SP
S1	Gravels (e < 0.25)	0–3	GW, GP
	Crushed stone	0–3	GW, GP
	Crushed rock	0–3	GW, GP
	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	Gravelly soils	10–20	GM, GC, GM-GC
	Sands	6–15	SM, SC, SW-SM, SP-SM, SW-SC, SP-SC, SM-SC
F3	Gravelly soils	> 20	GM, GC, GM-GC
	Sands, except very fine silty sands	> 15	SM, SC, SM-SC
	Clays (PI > 12)	—	CL, CH, ML-CL
F4	Silts	—	ML, MH, ML-CL
	Very fine sands	> 15	SM, SC, SM-SC
	Clays (PI < 12)	—	CL, ML-CL
	Varved clays and other fine-grained, banded sediments	—	CL or CH layered with ML, MH, SM, SC, SM-SC, or ML-CL

Legend:

CH	clay high plasticity
CL	clay low plasticity
e	void ratio
GC	clayey gravels, gravel-sand-clay mixtures
GM	silty gravels, gravel-sand-silt mixtures
GP	gravel poorly graded, gravel-sand mixtures, little or no fines
GW	gravel well-graded, gravel-sand mixtures, little or no fines
MH	silt high plasticity
ML	silt low plasticity
mm	millimeters
PI	plasticity Index
NFS	nonfrost susceptible
SC	clayey sand, sand-clay mixtures
SM	silty Sands, sand-silt mixtures
SP	poorly graded sands, gravelly sands, little or no fines
SW	well-graded sand, gravelly sands, little or no fines
USCS	Unified Soil Classification System

5-34. Where the CBR value of the subgrade, without processing, is in the range of 20 to 50, 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.

5-35. 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.

Select Materials

5-36. Select materials will typically be locally available as coarse-grained soils (gravels and sands) although fine-grained soils in the ML and CL groups may be used in certain cases. Consider limestone rock, sandstone rock, coral, oyster shells, 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, page 5-7. A maximum aggregate size of 3 inches will aid in meeting aggregate gradations.

Subbase Materials

5-37. Subbase materials may consist of native coarse-grained soils or blended and processed soils. Limestone rock, sandstone rock, coral, oyster shells, ashes, cinders, caliche, and disintegrated granite may be used as subbases when they meet the requirements in table 5-1. The existing subgrade may meet the requirements for a subbase course, or it may be possible to modify the existing subgrade to produce a subbase. Do not mix native or processed materials unless the unmixed subgrade meets the LL and PI requirements for subbases.

5-38. A suitable subbase may be formed using material that is stabilized with commercial admixtures. Portland cement, hydrated lime, fly ash, and bituminous materials are commonly used. The plasticity of some materials can be decreased by adding lime or Portland cement, enabling them to be used as subbases.

Compaction

5-39. 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 5-4.

SELECTION OF DESIGN CALIFORNIA BEARING RATIO

5-40. 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 apply to select and subbase materials. To be used as a select or subbase, the material must comply with the requirements indicated in table 5-1, page 5-7, 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 CBR value does not meet the maximum design CBR for that category, assign the material a CBR design value equal to the measured CBR results. For example, a material with a measured CBR value of 37 that 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 not a CBR 37 subbase.

5-41. Some native 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, if supported by adequate in-place CBR tests on construction projects using the materials that have been in service for several years.

5-42. The CBR test is not applicable for 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 TM 3-34.64/MCRP 3-17.7G.) Ratings as high as 100 can be assigned to these materials when proper construction procedures are followed.

BASE COURSE

5-43. The purpose of a base course is to distribute the induced stresses from the wheel load so that the load will not exceed the available strength of the underlying soil layers. When the subgrade strength is low, the stress must be reduced to a lower value or a thicker base is needed. When the subgrade strength is high, a thinner base course will provide adequate stress distribution. Because the stresses in the base course are always higher than in the subgrade, the base course must have higher strength. (See figure 5-5).

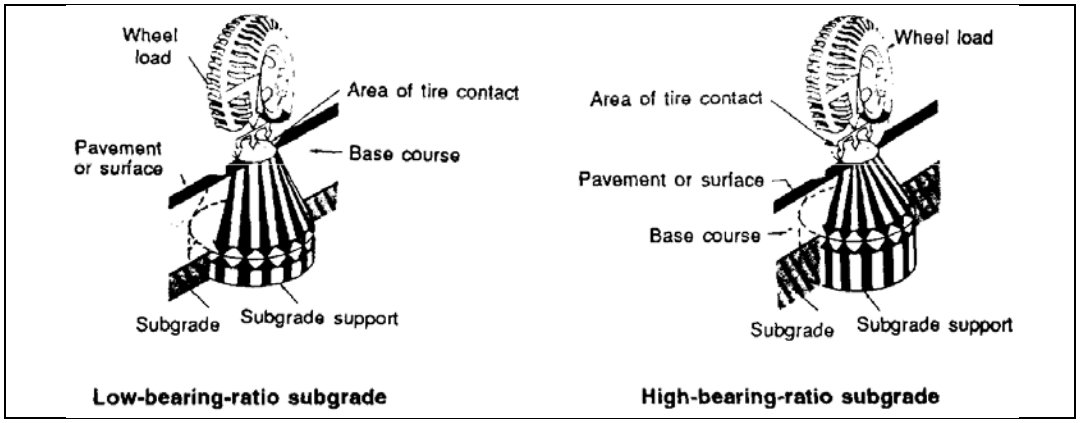


Figure 5-5. Distribution of stress in base courses and effects of subgrade strength on base course thickness

5-44. 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

5-45. 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

5-46. Normally, a material used as a base course must meet the gradation requirements outlined in table 5-3, table 5-4, and table 5-5 (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 that have more than 15 percent fines (particles passing the number 200 sieve).

Plasticity Index and Liquid Limit

5-47. In addition to the gradation requirement, a base-course material must meet the same PI and LL requirements for a subbase material. (See table 5-1, page 5-7.) Material with a LL greater than 25 or a PI greater than 5 should not be used as a base course.

Table 5-3. Generally suitable base course materials

Rock Type		*Use as an Aggregate*		*Use as a Base Course or Subbase*
		Concrete	*Asphalt*	
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 to 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
Legend: No. number				

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	—	—
Number 4	30-50	20-40	25-45
Number 10	15-40	—	—
Number 40	5-25	—	—
Number 200	<10	<10	<10

Compaction

5-48. Base-course material must be capable of being compacted to meet the requirements given in figure 5-4, page 5-4. When constructing a base course, lift thickness must be designed on its ability to attain the required density. Lift thickness is dependent on the type of material, the compaction equipment used, and the method of construction.

5-49. 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 shows nine material types commonly used as base courses for roads and airfields. A typical design CBR is given for each type of material. Laboratory CBR tests to determine design CBR values are not necessary.

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.	
Legend:	
CBR	California bearing ratio
PSI	pounds per square inch

MATERIALS

5-50. Natural, processed, and other materials are used for base courses. Descriptions of these materials follow.

Natural

5-51. A wide variety of gravels, sands (gravelly and sandy soils), and other natural materials (lime rock, coral, shells, some caliches) can be used alone or blended as a suitable base course. Sometimes natural materials require crushing or removal of oversized 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

5-52. 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. Blending materials from two or more deposits can often produce satisfactory base materials. Uncrushed, clean, washed gravel is normally not suitable for a base course because there are not enough fine sands present. Fines act as a binder and fill the voids between coarser particles.

Sand and Clay

5-53. Natural mixtures of sand and clay of varying thicknesses are often located in riverbed deposits. 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.

5-54. 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

5-55. 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, gravel, talus deposits, coarse mine tailings, and similar hard and durable rock fragments are typical sources of processed materials. Table 5-3, page 5-11, shows the common rock types that are suitable for base-course material. Generally, rock that is hard enough to require blasting during excavation makes suitable base-course material.

5-56. Base courses made from processed materials can be divided into the following general types:

- **Stabilized.** In a stabilized base course, material ranging from coarse to fine is mixed to meet the gradation requirement given in table 5-4, page 5-11. 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 clay-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 water-bound macadam. The crushed rock used for macadam base courses should consist of clean, angular, durable particles that are free of clay, organic matter, and other unwanted material or coating. A 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 5-11.

Other

5-57. In some JOA 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 similar materials. Some of these materials are weak rock that crush or degrade under construction traffic to produce composite base materials similar to those described in the preceding paragraphs. Others develop a cementing action that results in a satisfactory base.

5-58. These materials cannot be judged on the basis of the gradation limits used for other materials. Rather, they are judged on the basis of service behavior. Strength tests on laboratory samples are 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 ($\leq$5) is a necessity. However, observation of these types of base materials in existing roads and pavements is the most reliable indicator of whether they will be satisfactory.

Coral

5-59. Coral is commonly found along tropical ocean coastlines. Coral is normally very angular and, as such, its greatest asset as a construction material is 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 are 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 a seawater or sodium chloride (salt) solution promotes bonding when rollers are used. Coral should cure for a minimum of 72 hours after compaction is complete.

Caliche

5-60. 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 time to cure. Caliche varies in content (limestone, silt, and clay) and degree of cementation. It is important that caliche of good, uniform quality is obtained from deposits and that it be compacted within a specified moisture range.

5-61. 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 a 2-inch sieve: 100.
- Percent passing a number (No.) 40 sieve: 15 to 35.
- Percent passing a No. 200 sieve: 0 to 20.

5-62. Stripping should be used to remove undesirable material from surface deposits of caliche.

Tuff

5-63. Tuff and other cement-like materials of volcanic origin may be used for base courses. Tuff bases are constructed the same 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 graded, and final compaction and finishing are accomplished.

Rubble

5-64. The debris or rubble of destroyed concrete/masonry 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.

DANGER

When using rubble in a tactical environment, use caution. Rubble may contain mines or booby traps.

Bituminous Base

5-65. Bituminous mixtures are frequently used as base courses beneath high-use bituminous pavements, particularly for support area airfields carrying heavy traffic. Bituminous bases may be advantageous when locally available aggregates are relatively weak and have poor quality, when mixing plant and bituminous materials are readily available, or when a relatively thick structure is required for traffic.

5-66. 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

5-67. 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 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 preferred to bituminous or cement-stabilized types. This is true even where suitable admixture materials for such construction are readily available, but is not true in many areas of the world.

SPECIAL CONSIDERATIONS FOR SEASONAL FROST AND PERMAFROST CONDITIONS

5-68. Since base-course materials are near the surface of the road or airfield, the amount of strength lost during thawing periods will have a strong influence on the life of the facility. If possible, the materials listed in table 5-2, page 5-8, as nonfrost susceptible or frost susceptible (frost group S1 or S2) should be used as base courses in seasonal frost and permafrost areas.

CONSTRUCTION OPERATIONS

5-69. Construction operations for roads and airfields include fine grading; hauling, placing, and spreading; blending and mixing; watering base materials; compacting; and finishing. These tasks are organized according to the construction schedule and quality control plan for the project.

Fine Grading

5-70. 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. Ruts and other soft spots should be corrected.

Hauling, Placing, and Spreading

5-71. 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 nearest point to the source is that haul vehicles can be routed over the spread material, which will compact the base and avoid damage to the subgrade. An advantage of working from the point farthest from the source is that hauling equipment will further compact the exposed subgrade. Also, this practice will not overwork the base course, which can cause unwanted segregation. This method also reveals weak spots in the subgrade so that they can be corrected before placement of the base courses and interferes less with spreading and compaction equipment.

5-72. The self-propelled aggregate spreader is preferred 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.

5-73. 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

5-74. 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.

5-75. 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

5-76. 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 nuclear density meter, a speedy moisture tester, or by expedient methods. Given the on-site moisture content, the engineer in charge can calculate exactly how much water to add or if the base needs to be aerated to achieve the specified moisture content range.

5-77. A truck-mounted water distributor can control watering. Asphalt distributors should not be used because the pump lubrication system is not designed for water. A container capable of movement and gravity discharge of water may be used as an expedient water distributor.

Compacting

5-78. Base-course compaction 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 layer 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 quality of the finished flexible pavement is a direct result of the care and judgment used when constructing the base course. 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

5-79. Finishing operations must closely follow compaction, resulting in a crowned, tight, and water-shedding surface free of ruts and depressions that would inhibit runoff. Use the grader or 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 that 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 delamination may result. Final rolling is done with rubber-tired and steel-wheeled rollers.

ASPHALT CONCRETE COMPACTION

5-80. Asphalt compaction for the base course and the asphalt concrete layer requires that the asphalt in place be compacted to some percentage of the laboratory density. Typically, density specifications require that field densities should be 98 to 100 percent of laboratory densities. Laboratory density should be determined according to the ASTM on standard test method for bulk specific gravity and density of no-absorptive compacted bituminous mixtures.

5-81. The Army and Air Force allow two methods of compacting asphalt paving mixtures in the laboratory: the Marshall method and the Gyratory Testing Machine (GTM) method. The procedures for conducting the Marshall mix design tests are described in CRD-C 649-95. The procedure for gyratory compaction is given in CRD-C 651-95 or the standard testing method for the ASTM Standard Test Method for Compaction and Shear Properties of Bituminous Mixtures by Means of the United States Corps of Engineers GTM. The exceptions are as follows:

- Use 4-inch diameter molds, not 6-inch molds, when Marshall Stabilities and flows are to be determined.

- Use the mixing and compaction temperature requirement as given in United States Army Corps of Engineers (USACE) CRD-C 649-95.

5-82. The GTM setting and equivalent compaction requirements are shown in table 5-7.

Table 5-7. Traffic type and gyratory compaction requirements

Type of Traffic	*Gyratory Compaction Requirement*
Tire pressures less than 100 psi	Gyratory compaction at 100 psi, 1-degree, 30 revolutions (50 blows Marshall equivalent)
Tire pressures 100 psi and greater, but less than 230 psi	Gyratory compaction at 200 psi, 1-degree, 30 revolutions (75 blows Marshall equivalent)
Tire pressures 230 psi and greater	Gyratory compaction at 240 psi, 1-degree, 60 revolutions (preferred but not mandatory for Army and Air Force airfields)
Legend: psi pounds per square inch	

5-83. In the event that an airfield pavement is to be subjected to aircraft with tire pressures of 230 pounds per square inch or more, the GTM method is mandatory for Army airfield pavements. The GTM method is preferred, but not mandatory for Air Force airfield pavements. If the GTM method is used for design, it should also be used for control testing. If the GTM method cannot be used for control testing, the Marshall apparatus can be used by developing a correlation between the GTM and Marshall specimens for the job mix. Care should be taken to ensure that excess breakage of the aggregate particles does not occur during Marshall compaction. Additional information is presented in UFC 3-250-03.

SPECIAL PROCEDURE FOR MACADAM BASES

5-84. Macadam base-course construction procedures are discussed in the following paragraphs.

Preparing Subgrade

5-85. If a macadam base course is constructed on a material with high plasticity, there may be base infiltration of fine materials. 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 instead of the blanket course.

Spreading

5-86. Macadam aggregate must be placed and spread carefully to ensure that hauling vehicles do not add objectionable material to the aggregate. Care is necessary when placing the aggregate at the point nearest the source and when 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 each layer.

Compacting

5-87. After spreading, immediately 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 there is no creep or wave movement of the aggregate in front of the roller; this indicates that the aggregate is stable. Do not attempt rolling when the subgrade is softened by rain.

Applying Screenings

5-88. 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 that the jarring effect of the roller will cause the screenings to settle into surface voids. Spread screenings in thin layers by using hand shovels, mechanical spreaders, or moving trucks. Do not dump the screenings in piles on the coarse aggregate. If necessary, use hand or drag brooms to distribute screenings during rolling.

5-89. Do not apply screenings too 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 screenings cannot be forced into the voids. Start sprinkling the surface with water after the screenings have been spread. Sprinkling causes the screenings to be flushed down into the voids of the aggregate. The surface is then rolled. Do not saturate; it will soften the subgrade.

5-90. Continue sprinkling and rolling until a mixture of screenings and water forms, fills the voids, and gathers in a small wave before each roller. When a section of a strip has been rolled thoroughly, allow it to dry completely before performing additional work.

FINISHED SURFACES

5-91. The base-course surface determines the smoothness of the finished pavement. If the finished base does 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.

5-92. 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 deviation in excess of 3/4 inch for roads and airfields (for propeller-type aircraft) or 1/8 inch for jet aircraft. Correct a deviation in excess of these figures, remove material to the total depth of the lift, and replace with new material and compact as specified above.

SLUSH ROLLING

5-93. 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 only be permitted on a free-draining, cured base course. Slushing requires a considerable amount of water on the surface. The quantity varies with the type of material, the temperature, and the humidity. If the surface is mostly satisfactory but has some large areas that require 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

5-94. Engineers must calculate a water application rate in terms of gallons per square yard 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

5-95. Use pneumatic-tire, vibratory, or steel-wheeled rollers to obtain a smooth finish on the base course. Continue rolling until compaction has been obtained.

Finishing

5-96. 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

5-97. 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 the job may require a combination of the two in some cases.

Applying Water

5-98. 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 percent above the minimum moisture content. The percent of moisture will vary with the type of material and is a matter of judgment by the project quality control manager.

Finishing

5-99. 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-tire rollers must be continued until a smooth, dense surface is obtained. This method can also be used to correct minor surface irregularities in the base course.

QUALITY CONTROL

5-100. Quality control is essential to the success of a project. Although visual inspection is important, it is not, by itself, sufficient to control the construction of the courses described in this chapter. 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 TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034. Before starting construction, a detailed quality control plan should be developed that addresses testing procedures, frequency, location, and, most importantly, remedial actions.

SOIL STABILIZATION

5-101. This chapter presents an introduction and discussion of soil stabilization for improving the structural quality and workability of soils used for base courses, subbase courses, select materials, and subgrades for pavements. It is applicable to the elements responsible for Army, Air Force, or Navy pavement construction. Stabilized soils can often be adequate for airfields, traffic pavements, and parking and storage areas where an all-weather surface is required, yet traffic does not justify a higher-strength pavement. Surface treatments are also used to provide dust control. The most widely recognized form of stabilization is compaction, which improves the mechanical stability of most soils. However, compaction alone is often not sufficient to meet project requirements.

5-102. Stabilization is the process of blending and mixing materials with a soil to improve the soil strength and durability. The process may include blending soils to achieve a desired gradation or mixing commercially available additives that may alter the gradation, change the strength and durability, or act as a binder to cement the soil. For details refer to UFC 3-250-11 and TM 3-34.64/MCRP 3-17.7G.

USES

5-103. Pavement design is based on the premise that specified levels of quality will be achieved for each soil layer in the pavement system. Each layer must—

- Resist shearing within the layer.
- Avoid excessive elastic deflections that would result in fatigue cracking within the layer or in overlying layers.
- Prevent excessive permanent deformation through densification.
- Control dust.

5-104. As the quality of a soil layer is increased, the ability of that layer to distribute the load over a greater area is generally increased enough to permit a reduction in the required thickness of the soil and surface layers.

Improve Soil Quality

5-105. Stabilization is commonly used for better soil gradation, reduction of the PI or swelling potential, and increased durability and strength. Stabilized pavement surfaces can be used for many common applications (roads, airfield facilities, construction platforms). Stabilized pavements generally do not provide the durability of asphalt or concrete surfacing.

Reduce Surface Pavement Thickness

5-106. A soil layer's tensile strength and stiffness can be improved by using additives and can reduce the thickness of the stabilized layer and overlying layers within the pavement system. Procedures for designing pavements that use stabilized soils are presented in UFC 3-250-01FA. Before a stabilized layer can be used to reduce the required thickness in the design of a pavement system, the stabilized material must meet the durability requirements of various types of additive stabilization and the minimum strength requirements. Generally, as the percent of fines and the PI increase, pulverization becomes more difficult and it is harder to obtain uniform distribution of the stabilizing additive. For these types of soils, preprocessing or pretreatment with other additives may be necessary. For example, fine-grained soils may be pretreated with lime to aid in their pulverization, making the mixing of a bitumen or cement additive more successful.

METHODS

5-107. The two general stabilization methods are mechanical and additive. The effectiveness of stabilization depends on the ability to obtain uniformity in blending the various materials. Mixing in a stationary or traveling plant is preferred. However, other means of mixing (scarifiers, plows, disks, graders, rotary mixers, and reclaimer/stabilizers) have been satisfactory.

5-108. The soil stabilization method is determined by the amount of stabilizing required and the conditions encountered on the project. An accurate soil description and classification are essential for selecting the correct materials and procedure. TM 5-822-14/AFJMAN 32-1019 lists the most suitable treatments for various soil types to stabilize the soil for different objectives and contains detailed descriptions of proper construction techniques

Mechanical

5-109. Mechanical stabilization is accomplished by mixing or blending two or more gradations of material to obtain a mixture that meets the required specifications. These materials may be blended at the construction site, a central plant, or a borrow area. The blended material is spread and compacted to the required density by conventional means. If the mixture does not meet the specifications after blending, an additive may be necessary for stabilization. Compaction is also a form of mechanical stabilization.

Additive

5-110. Additive refers to a manufactured commercial product that, when added to the soil in the proper quantities, will improve the quality of the soil layer. Additives may stabilize the soil by chemical, physical,

or a combination of the two (physicochemical). Chemical stabilizers include lime, acids, and enzymes that cause a chemical change in the soil, such as ion-exchange or covalent reactions. Physical stabilizers include bitumens that bind the soil by adhesion and fibers that bind by mechanical friction and interpenetration. Physicochemical stabilizers, such as lignosulfonates and lime-cement-fly ash (LCF), may bind soil particles together and induce chemical changes in the soil (pozzolonic reactions). Selecting and determining the percentage of additives depend on the soil classification and the degree of improvement in the soil quality desired. Smaller amounts of additives are usually required to alter soil properties (gradation, workability, plasticity) than to improve strength and durability sufficient to permit a thickness-reduction design. After the additive has been mixed with the soil, ding and compacting are achieved by conventional means.

MODIFICATION

5-111. Soil modification refers to the stabilization process that results in the improvement in some property of the soil but does not, by design, result in a significant increase in the soil's strength and durability. Soil modification usually results in something less than a thoroughly cemented, hardened, or semihardened material. This type of stabilization may be accomplished by—

- Compacting.
- Blending mechanically.
- Adding cementing material in small amounts.
- Adding chemical modifiers.

5-112. Cement and lime modifiers (cement-modified soils and lime-modified soils) are used in quantities too small to provide high strength cementing action. They reduce the plasticity of clay soils. Calcium chloride or sodium chloride are added to the soil to retain moisture and control dust, to hold fine material for better compaction, and to reduce frost heave by lowering the freezing point of water in the soil.

STABILIZER TYPES

5-113. This section discusses the different types of stabilizers available and provides some limited information on the soil types appropriate for the stabilizer type. Stabilizers are broadly classified as traditional and nontraditional, referring mostly to historical use. It must be noted that there is no one-size-fits-all solution for stabilization. Soils are notoriously variable even within classification types and may have different chemistry that would negate the use of some stabilizers. It must be emphasized that each soil/stabilizer combination should be treated on a case-by-case basis.

Traditional Stabilizers

5-114. Traditional stabilizers generally rely on pozzolanic reactions and cation exchange to modify and/or stabilize. Although pozzolanic processes are slow, some amount of pozzolanic strength gain may occur during the primary reactions, caution exchange, and flocculation/agglomeration. Extent of this strength gain may vary with soils depending on the differences in their mineralogical composition.

Cement

5-115. Cement can be used to modify and improve the quality of the soil or to transform the soil into a cemented mass with increased strength and durability. Cement can be used effectively as a stabilizer for a wide range of materials; however, the soil should have a PI of less than 30. For coarse-grained soils, the amount passing the No. 4 sieve should be greater than 45 percent. The amount of cement used depends on if the soil is to be modified or stabilized.

Lime

5-116. Experience shows that lime will react with many medium-grained soils, moderately fine-grained soils, and fine-grained soils to produce decreased plasticity, increased workability, reduced swell, and increased strength. Soils classified according to the USCS as CH, CL, MH, ML, OH, OL, SC, SM, GC, GM, SW-SC, SP-SC, SM-SC, GWGC, GP-GC, ML-CL, and GM-GC should be considered as potentially capable

of being stabilized with lime. Lime should be considered with soils having a PI of greater than 10 and more than 25 percent of the soil passing the No. 200 sieve.

Fly Ash

5-117. Fly ash, when mixed with lime, can be used effectively to stabilize most coarse-grained soils and medium-grained soils; however, the PI should not be greater than 25. Soils classified by the USCS as SW, SP, SP-SC, SW-SC, SW-SM, GW, GP, GP-GC, GW-GC, GP-GM, GW-GM, GC-GM, and SC-SM can be stabilized with fly ash.

Bituminous

5-118. Most bituminous soil stabilization has been performed with asphalt cement, cutback asphalt, and asphalt emulsions. Soils that can be stabilized effectively with bituminous materials usually contain less than 30 percent passing the No. 200 sieve and have a PI of less than 10. Soils classified by the USCS as SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, SC, SM-SC, GW, GP, SW-GM, SP-GM, SW-GC, GP-GC, GM, GC, and GM-GC can be effectively stabilized with bituminous materials, provided the above-mentioned gradation and plasticity requirements are met.

Combination

5-119. Combination stabilization is specifically defined as lime-cement, lime-asphalt, and LCF stabilization. Combinations of lime and cement are often acceptable expedient stabilizers. Lime can be added to the soil to increase the soils workability and mixing characteristics and reduce its plasticity. Then, cement can be mixed into the soil to provide rapid strength gain. Combinations of lime and asphalt are often acceptable stabilizers. The lime addition may prevent stripping at the asphalt-aggregate interface and increase the mixture stability.

Nontraditional Stabilizers

5-120. Nonstraditional stabilizers for military use generally consist of either polymers or fibers. Polymers stabilize similarly to a solid surface product whereas fibers are added to base material and intertwine with it to effectively bond materials more rigidly.

Polymers

5-121. Polymer emulsions (sometimes referred to as latex) used for soil stabilization are generally vinyl acetate or acrylic-based polymers suspended in water by surfactants. They typically consist of 45 to 50 percent polymer content by weight. Analogous to latex paint, the emulsion cures primarily by water loss. The polymers used for soil stabilization typically have excellent tensile and flexural strength, adhesion to soil particles, and resistance to water. These materials have a shelf life which is up to two years, in proper storage. Polymer emulsions should not be mixed with gray or salt water for dilution.

Fibers

5-122. There are numerous fiber types available. For soil stabilization, a combination of availability, fiber properties, and cost per performance pares the fiber type to polypropylene. These fibers are low-cost, available in different lengths and diameters, and have been extensively tested. Monofilament, fibrillated, and tape fibers are available. Testing has shown that monofilament and tape fibers are more efficient for rapid stabilization. Fibrillated fibers may require extensive mixing to effectively defibrillate the fiber for maximum efficiency.

5-123. The most important fiber property for soil stabilization is fiber length. Longer fibers generally result in improved performance but hamper construction. The longer fibers are more difficult to mix and considerable balling may occur, diminishing efficiency. Testing of different fiber lengths has shown that fiber lengths from 3/4 inch to 1.5 inch results in good soil improvements with minimal balling.

5-124. Soil type is of a secondary role in fiber selection. Polypropylene fibers mixed into soil without adhesives (cement) rely exclusively on mechanical interlock, friction, and fiber entanglement between soil

particles to anchor the fiber in place. Thus, a high-surface area fiber, such as tape fiber, may be more efficient in a fine-grained clay soil than in a coarse-grained soil, such as gravel, where fewer soil grains are in contact with the fiber.

Other

5-125. There are a variety of other nontraditional soil stabilization and modification additives available from the commercial sector (acids, lignin derivatives, enzymes, tree resin emulsions, silicates). These additives vary widely in their chemical composition and mechanism of action, may be in liquid or solid form, and are often touted to be applicable for most soils. Research studies in this area have demonstrated that many nontraditional soil additives have little to no benefit for granular soil types. Sandy and gravelly soils are often problematic for stabilization, requiring a binder (such as a cement or asphalt emulsion) to provide cohesion between soil grains. Medium- and fine-grained soils are likely to benefit most from other nontraditional stabilizers. However, these must be treated on a case-by-case basis.

Nontraditional Combinations

5-126. Blends of soil with polymer emulsions and hydraulic cements or with fibers and hydraulic cements have shown promise in laboratory testing and field demonstrations. A synergism exists between emulsions and hydraulic cement as the water in the emulsion hydrates the cement and the polymer adds flexibility, durability, and strain resistance to the soil cement, increasing the cracking resistance. The use of fibers is also very attractive due to their ability to improve the resistance of soil to deformation and resist the tensile stresses that lead to cracking. Fibers have been found to be more efficient than polymer emulsion on a weight-to-weight basis of comparison. However, fibers may be difficult to use because they tend to ball when graded and may be difficult to place in windy situations.

TIME REQUIREMENTS FOR TESTING

5-127. The more thorough a testing program, the more assurance there is for the long-term success of the project. Time is often of primary concern to a military engineer—particularly in a tactical situation—and the rapid completion of a project may override the requirement for a complete series of laboratory tests. (See table 5-1, page 5-7.) Because of this, the method presented allows for a rapid or expedient approximation with a more precise laboratory determination of the type and quantity of stabilizer. An estimate for testing time is presented in table 5-8.

Table 5-8. Estimated time required for test procedures

Construction Type	*Stabilizing Agent*	*Time Required**
Expedient	Any	None
Nonexpedient	Lime	30 days
	LCF	30 days
	Cement	6 to 9 days
	Bitumen	1 day
	Nontraditional	1 to 9 days
*These criteria do not include time required for gradation.		
Legend: LCF lime-cement-fly ash		

STABILIZER SELECTION AND CONCENTRATIONS

5-128. When selecting a stabilizer additive, many factors must be considered. These factors, design criteria, and the selection and mixing of stabilizers can be found in TM 5-822-14/AFJMAN 32-1019 and TM 3-34.48-2. If lime is used as a preliminary additive to reduce the PI or to alter the soil gradation before adding a primary stabilizing agent, such as bitumen or cement, the designs lime content is the minimum treatment

level that will achieve the desired results. For nonplastic and low PI materials where lime alone is generally not satisfactory for stabilization, adding fly ash may produce the necessary reaction.

5-129. Several factors need to be considered in the selection of a stabilizer, including environmental conditions, availability and cost of stabilizers, logistical considerations, equipment available, soil type, and the strength and durability needed for the desired application. For in-depth details about the data needed for stabilizer selection, see TM 5-822-14/AFJMAN 32-1019. In general, the higher amount of fines in the soil, the harder it is to stabilize. Thus, clays can be difficult to stabilize. Lime may be needed to reduce the plasticity and increase friability of high-plasticity clay before cement is added. For sandy and gravelly soils, some type of binding agent (cement or emulsion) is necessary to bind the particles together.

5-130. To select the proper stabilizer type for a particular soil, perform a sieve-analysis test to determine gradation and fines content and an Atterberg-limits test according to the procedures given in this manual. The selection of a stabilizer begins with a classification of the soil type. (See figure 5-6.) Use figure 5-6 to classify the soil. Table 5-9 is then used to make an initial selection of appropriate stabilizers for the soil type, and table 5-10, page 5-26, is used to choose an appropriate stabilizer dosage level.

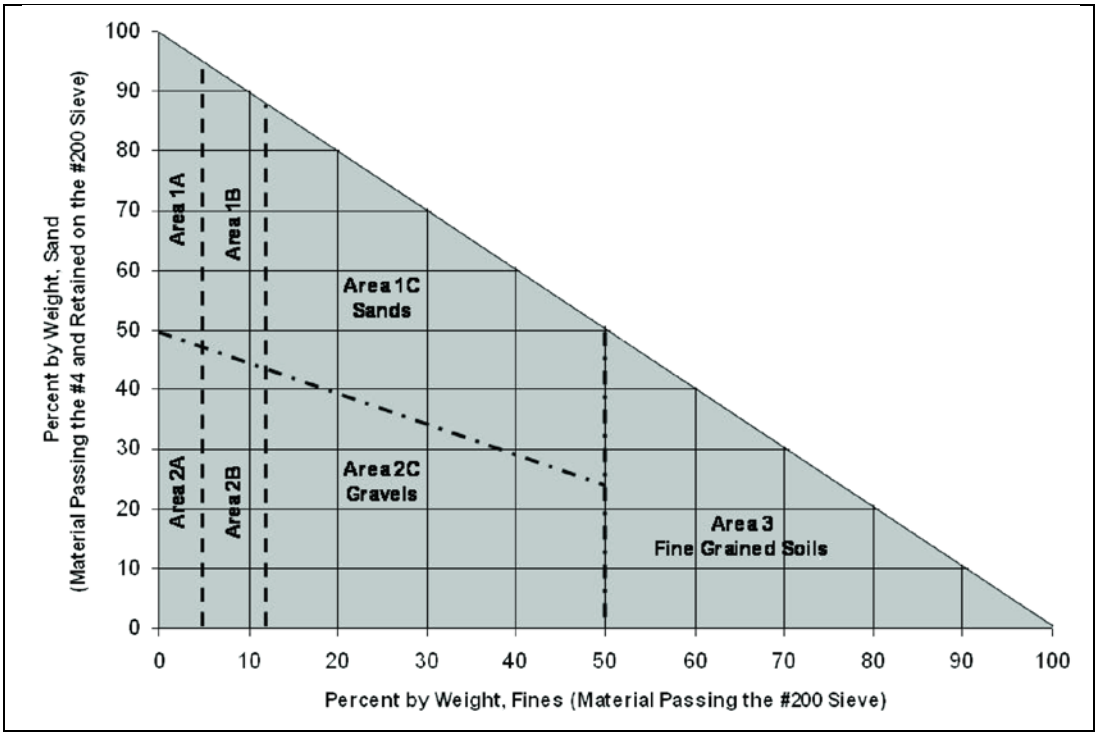


Figure 5-6. Gradation triangle for soil classification

5-131. The environmental conditions present at the stabilization site may need special consideration. In particular, temperature will be the most critical factor that influences the curing time. When temperatures are below 45 degrees Fahrenheit, curing times will be significantly increased. In general, for every 20 degrees Fahrenheit temperature decrease, curing time doubles. For example, for 3 percent type III cement, one day of cure time is needed at 75 degrees Fahrenheit, and two days is needed at 55 degrees Fahrenheit. The curing time may be offset by increasing the stabilizer concentration from 3 to 6 percent for a one day curing time at 55 degrees Fahrenheit.

5-132. The amount of stabilizers used varies with the particular soil type and the expected durability needed for the mission. In TM 5-822-14/AFJMAN 32-1019, stabilizer amounts are determined by specific tests and criteria. Given the constraints of a rapid deployment scenario, there is not enough time to perform mix design tests with various stabilizers and amounts to determine if those specific criteria are met. If time allows, it is always best to perform a strict mix design according to TM 5-822-14/AFJMAN 32-1019. However, the suggested stabilizer amounts given in table 5-9 and table 5-10, pages 5-25 and 5-26, should provide acceptable performance.

Table 5-9. Soil stabilizers for different soil types

Area	*Soil Class*	*Type of Stabilizer*[1]	*Restriction (LL and PI)*	*Restriction (% Pass No. 200)*	*Remarks*
1A	SW or SP	1. Polymer/bituminous emulsion and cement 2. Portland cement and fiber 3. Lime-cement-fly ash and fiber	PI not to exceed 25	—	Soils near or above their optimum moisture content may require drying before emulsion stabilization.
1B	SW-SM, SP-SW, SW-SC, or SP-SC	1. Polymer/bituminous emulsion and cement 2. Portland cement and fiber 3. Lime and fiber 4. Lime-cement-fly ash and fiber	1. PI not to exceed 10 2. PI not to exceed 30 3. PI not less than 12 4. PI not to exceed 25	—	Soils near or above their optimum moisture content may require drying before emulsion stabilization.
1C	SM, SC, or SM-SC	1. Polymer/bituminous emulsion and cement 2. Portland cement and fiber 3. Lime and fiber 4. Lime-cement-fly ash and fiber	1. PI not to exceed 10 2. PI 3. PI not less than 12 4. PI not to exceed 25	Not to exceed 30% by weight	Soils near or above their optimum moisture content may require drying before emulsion stabilization.
2A	GW or GP	1. Polymer/bituminous emulsion and cement 2. Portland cement and fiber 3. Lime-cement-fly ash and fiber	PI not to exceed 25	—	Well-graded material only. Material should contain at least 45% by weight of material passing No. 4 sieve. Soils near or above their optimum moisture content may require drying before emulsion stabilization.
2B	GW-GM, GP-GM, GW-GC, or GP-GC	1. Polymer/bituminous emulsion and cement 2. Portland cement and fiber 3. Lime and fiber 4. Lime-cement-fly ash and fiber	1. PI not to exceed 10 2. PI not to exceed 30 3. PI not less than 12 4. PI not to exceed 25	—	Well-graded material only. Material should contain at least 45% by weight of material passing No. 4 sieve. Soils near or above their optimum moisture content may require drying before emulsion stabilization.
2C	GM, GC, or GM-GC	1. Polymer/bituminous emulsion and cement 2. Portland cement and fiber 3. Lime and fiber 4. Lime-cement-fly ash and fiber	1. PI not to exceed 10 2. PI[2] 3. PI not less than 12 4. PI not to exceed 25	Not to exceed 30% by weight	Well-graded material. Material should contain at least 45% by weight of material passing No. 4 sieve.
3	CH, CL, MH, ML, OH, OL, or ML-CL	1. Portland cement and fiber 2. Lime and fiber	1. LL less than 40 and PI less than 20 2. PI not less than 12	—	Organic and strongly acid soils falling within this area are not susceptible to stabilization by conventional means.

[1]Monofilament polypropylene fiber—length and denier will vary depending on soil type.

[2]PI ≤ 20 + 50- percent passing No. 200 sieve.

Legend:

class	classification
LL	liquid limit
No.	number
PI	plasticity index

Table 5-10. Suggested stabilizer amounts for expedient stabilization

Soil Type	*Fine Content*	*Plasticity Index*	*Stabilizer Type (a)*	*Percent Additives*
Sand 1A and 1B	Low	All	Portland Cement Emulsion Polymer Fiber (b)	3–5 3–5 0.2–0.5
Sand 1C	High	High Low Intermediate	Lime and Portland Cement Portland Cement Fiber (b)	3–5 and 3–5 5 As above 0.2–0.5
Gravel 2A and 2B	Low	All	Portland Cement Emulsion Polymer Fiber (b)	3–5 3–5 0.2-0.5
Gravel 2C	High	High Low Intermediate	Lime and Portland Cement Portland Cement Fiber (b)	3–5 and 3–5 5 As above 0.2–0.5
Clays, Silts 3	High High High	High (>20) Low (<12) Intermediate (12–20)	Lime and Portland Cement Portland Cement Lime or Portland Cement Fiber (b)	5–7 and 5–7 (c) 7 (c) As above 0.2–0.5

(a) All of these stabilizers require a polymer emulsion surface coating of 0.25–1 gallon/square yard (1:2 dilution with water) for weatherproofing. Reapplication of surface coating may be necessary depending on traffic and conditions.

(b) Monofilament polypropylene fiber used in combination with stated stabilizers—length and denier will vary depending on soil type. In general, the finer the soil, the more fiber surface area (length and shape) is needed to interact with the soil-stabilizer matrix.

(c) In some cases, intermediate and high plasticity clays may require stabilizer amounts significantly higher than suggested.

EXPEDIENT STABILIZATION

5-133. Usually, the military engineer needs soil improvement quickly in the JOA with a minimum of logistics and stabilization material. In this case, the best stabilizer of choice is cement. It is found around the world, will stabilize a wide variety of soils, and can be used for surface pavements. However, common Portland (Type I) cement is not a rapid-cure material and may take up to seven days to provide enough strength to support large loads, such as cargo aircraft. It provides limited durability and can fail by cracking and surface raveling, which may endanger aircraft with foreign object debris. Cement is extremely heavy due to its high specific gravity, placing a further logistical burden on the military planner. Using less cement reduces the logistical burden at the expense of durability. For expedient and temporary structures, suggested amounts of cement for common soils are given in table 5-10. Many of the concerns about cement can be reduced by incorporating monofilament polypropylene fibers into the soil cement mix and waterproofing the soil cement surface. Using monofilament fibers between 3/4 and 2 inches can—

- Increase the load bearing capacity so that large loads can be handled in three days (sometimes less depending on the temperature).
- Reduce the foreign object debris potential by bonding to the soil cement and creating a mat-like composite.
- Be used with high-early strength cement (Type III) so that large loads, such as C-130 and C-17 aircraft, can be placed on the structure within 24 hours of construction.

5-134. Construction with fibers is conducted similar to cement stabilization; calculate the correct spacing of the fiber bags to achieve the desired concentration of fiber the same as with cement. Use of a reclaimer or stabilizer for soil mixing is mandatory. Fiber concentrations between 0.1 and 0.5 percent, based on the weight of soil, have shown to be successful. For most soils, fiber concentrations above 0.5 percent have demonstrated decreased density compared to lower fiber contents, which may decrease durability. Stabilization depths of 6 inches are recommended and can be used for soils above CBR 8 to 10 percent. Stabilization of more than one lift is typically not practical in expedient situations. A balance between fiber length and constructability must be maintained. Longer fibers improve soil engineering characteristics to a larger degree but are harder to use because balling and tearing out of the soil occur during grading. Grading soil fiber cement is extremely difficult and should only be attempted, if absolutely necessary, within an hour or two of compaction. If an area needs smoothing during compaction, stop compaction, grade, and recompact. Typically, the fiber will ball up in front of the grader blade, rip out of the soil, and leave large surface scars and tears. In practice, this is best accomplished by preparing, compacting, and grading the surface to final elevation immediately before stabilization.

5-135. Measurement of density for fiber-stabilized materials can be accomplished in the laboratory but is not possible in the field. Nuclear density devices give false readings because the fiber and sand-cone measurements give large errors due to problems excavating the fiber-stabilized soil. If laboratory measurements of density can be determined, the highest fiber content that produces equal density to a nonfiber stabilized specimen should be chosen.

5-136. If laboratory data is unavailable for optimum water content, use the following guidelines for cement stabilization:

- For cement amounts less than 5 percent, use a water content of 1 percent above the optimum for unstabilized soil.
- For cement contents above 5 percent, use a water content of 2 percent above the optimum for unstabilized soil.

5-137. Generally, it is best to compact cement-stabilized soil slightly on the wet side of optimum to provide extra water for hydration of the cement.

5-138. For the low stabilizer amounts suggested in table 5-10 it is necessary to cap the surface of the soil with a waterproofing agent, such as a polymer emulsion designed for soil stabilization. This prevents water from entering the stabilized soil matrix and reducing its strength. It will also prevent fiber pullout from the surface and provide dust control. The minimum rate of application is suggested at 0.25 gallons per square yard and must be reapplied as necessary depending on traffic. For airfield structures, application rates as high as 1 gallon per square yard may be used if the soil will accept that application rate. Bituminous emulsions can also be used, but polymer emulsions are more durable under traffic. Note that synthetic fluids used for dust control should not be applied to cement-stabilized surfaces because they reduce the strength and durability to traffic.

5-139. If logistics allow, stabilization according to TM 5-822-14/AFJMAN 32-1019 is recommended. However, if testing facilities are not available, the high dosage rates suggested for stabilizers can be used if logistics allow and the structure may become a long-term use facility in the future. Fibers can be used with a stabilizer. For a stabilized soil to be used as a pavement surface, it should be treated with a polymer or bituminous emulsion to extend its durability.

WEATHER

5-140. Hot, dry weather is preferred for bituminous stabilization. When asphalt cements are used for stabilization, proper compaction must be obtained. If thin lifts of asphalt-stabilized material are being placed, the air temperature should be 40 degrees Fahrenheit and rising, and the compaction equipment should be used immediately after lay-down operations. Adequate compaction can be obtained at freezing temperatures if thick lifts are used. When cutbacks and emulsions are used, the air and soil temperatures should be above freezing. Heavy rains on mixed, uncompacted material may be detrimental.

FROST AREAS

5-141. While bituminous, Portland cement, lime, and combinations of LCF stabilization are the most common additives, other stabilizers may be used for pavement construction in areas of frost design with approval. Approving agencies for are as follows:

- Headquarters, Department of the Army (DAEN-MPE-D), Washington, DC 20314 for Army projects.
- Headquarters, Air Force Engineering and Services Center (AFESC/DEM), Tyndall Air Force Base, FL 32401 for Air Force projects.
- Headquarters, Naval Facilities Engineering Command, Alexandria, VA 22330 for Navy or Marine Corps projects.

Limitations

5-142. In frost areas, stabilized soil should be used in a single layer or layers comprising one of the upper elements of a pavement system and directly beneath the pavement surfacing layer. The structural advantage in reducing the required thickness of the pavement system compensates for the added cost of stabilization. Treatment with a lower degree of chemical stabilization should be used in frost areas with caution and after intensive tests because weakly cemented material has less capacity to endure repeated freezing and thawing than firmly cemented material. A possible exception is using a low level of stabilization to improve a soil that will be encapsulated within an impervious envelope as part of a membrane-encapsulated soil-layer pavement system. A soil that is unsuitable for encapsulation due to excessive moisture migration and thaw weakening may be made suitable by using a moderate amount of stabilizing additive. Materials that are modified by a small amount of chemical additive to improve certain properties of the soil without significant cementation should also be tested to determine if the desired improvement is durable through repeated freeze-thaw cycles. The improvement should not be achieved at the expense of making the soil more susceptible to ice segregation.

Construction Cutoff

5-143. For materials stabilized with cement, lime, or LCF (strength increases with curing time), it is essential that the stabilized layer be constructed early in the season to allow strength to develop before the first freezing cycle begins. The rate of strength gain is substantially lower at 50 degrees Fahrenheit than at 70 degrees Fahrenheit or 80 degrees Fahrenheit. Chemical reactions will not occur rapidly for lime-stabilized soils when the soil temperature is less than 60 degrees Fahrenheit and is not expected to increase for one month. Chemical reactions will also not occur rapidly for cement-stabilized soils when the soil temperature is less than 40 degrees Fahrenheit and is not expected to increase for one month. In frost areas, it is not always sufficient to protect the mixture from freezing during a 7-day curing period, as required by the applicable guide specifications. A construction cutoff date before the onset of freezing conditions may be essential.

DRYING OF WET SOILS

5-144. Executing earthwork under wet conditions is one of the perpetual challenges of engineering. Proper compaction and engineering properties of soils are highly dependent on moisture content, yet nature seldom provides natural materials at the ideal moisture content. Experience teaches the field military engineer that attempting to subvert the fundamental laws that describe the effect of moisture content on construction and engineering properties of soils and aggregates leads to frustration and failure. These wet soils may be due to temporary weather conditions, such as thawing of frozen soils or rainfall, or to permanent situations, such as a high water table. Drying soils by conventional methods, such as spreading or disking, is likely to be slower and more weather dependent than military operations can tolerate.

5-145. When it is necessary to work with soils that are wetter than desired, chemical drying may be used to deal with isolated wet spots or, if necessary, more extensive areas of wet soils.

5-146. The chemical of choice for drying is quicklime (oxides of calcium and magnesium). Depending on the source, it is further defined as high-calcium quicklime, magnesian quicklime, or dolomitic quicklime; each is adequate for drying. The oxides hydrate with water and chemically bind water in the soil into a new

form of hydrated lime. This reaction releases considerable heat (is exothermic), which evaporates more water and reduces the moisture content of the soils. Because this reaction is very avid and exothermic, it is effective for drying soils down to freezing temperatures.

5-147. Lime is a generic term used to cover a variety of materials. Only quicklime is effective for drying soils. Hydrated lime has already combined with water and will not draw moisture into new chemical forms. It will have certain chemical reactions (cation exchange) with some clay minerals that will make them less plastic and easier to dry by conventional methods, but this is much different from the avid drying of quicklime that is not related to soil type. Calcium carbonate is sometimes marketed as agricultural lime and has no value for engineering stabilization or drying uses.

5-148. A product that contains free quicklime can be used as a drying agent. The more quicklime it contains, the more effective it will be as a drying agent. Portland cement, fly ash (particularly high calcium Class C ashes), and kiln dust are examples of common industrial materials with significant quicklime content that can be used for soil drying. Their effectiveness for drying is directly dependent on the free quicklime content.

This page intentionally left blank.

Chapter 6
Storm Drainage Design

Inadequate water 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 construction and operation.

DESIGN CONSIDERATIONS

6-1. The objective of storm drainage design is to provide safe passage for vehicles or the operation of facilities during a design storm event. The drainage system is designed to collect storm water runoff from the pavement surface and adjacent areas, convey it along and through the adjacent areas, and discharge it to an adequate receiving body without causing adverse on-site or off-site impacts.

6-2. Storm water collection systems must be designed to provide adequate surface drainage. Surface drainage is important for traffic safety. Rapid removal of storm water from the pavement minimizes hazardous conditions that can result in hydroplaning. Surface drainage is a function of transverse and longitudinal pavement slope, pavement roughness, inlet spacing, and inlet capacity.

6-3. Storm water conveyance systems (storm drain piping, ditches, channels, pumps) provide an efficient mechanism for conveying design flows from inlet locations to the discharge point without surcharging inlets or otherwise causing surface flooding. Erosion potential must also be considered in the design of open channels or ditches used for storm water conveyance.

6-4. The design of appropriate discharge facilities for storm water collection and conveyance systems includes consideration of storm water quantity and quality. Local, state, and federal regulations often control the allowable quantity and quality of storm water discharges. To meet these regulatory requirements, storm drainage systems usually require detention or retention basins and other best management practices for the control of discharge quantity and quality.

6-5. Comprehensive criteria for surface drainage design of military transportation facilities and civilian airport pavements are contained in UFC 3-230-01 (using the document is optional, but familiarization is encouraged).

CONSTRUCTION

6-6. Commanders and construction supervisors must ensure continuous maintenance of the drainage system during military road or airfield construction. 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.

6-7. Construction drainage must be completed before it is 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

6-8. An on-site investigation of the system and tributary area is a prerequisite for a study of drainage requirements. Information regarding the capacity, elevations, and condition of existing drains will be obtained. Topography; size and shape of the drainage area; extent and type of development; profiles, cross-sections, and roughness data on pertinent existing streams and watercourses; and possible ponding area locations will be determined.

6-9. Thorough knowledge of climatic conditions and precipitation characteristics is essential. Adequate information on soil conditions (types, permeability, vegetative cover, depth and movement of subsurface water, depth of frost) will be secured. Outfall and downstream flow conditions (including high water occurrences and frequencies) must be determined. The effect of base drainage construction on local facilities and local requirements will be evaluated.

6-10. Where diversion of runoff is proposed, downstream conditions leading to unfavorable public relations, costly litigations, or damage claims will be avoided. Agreements needed to obtain drainage easements or avoid interference with water rights will be determined during design and accomplished before construction. Possible adverse effects on water quality due to drainage disposal in waterways involved in water supply systems will be evaluated.

Environmental Considerations

6-11. Environmental considerations include the National Environmental Policy, the United States Army Environmental Quality Program, the United States Air Force Environmental Quality Program, the United States Navy Environmental Quality Program, the environmental impact analysis, and environmental effects of surface drainage systems.

National Environmental Policy

6-12. The National Environmental Policy Act of 1969 (Public Law 91-190), approved 1 January 1970, sets forth the policy of the Federal government, in cooperation with state and local governments and other concerned public and private organizations, to protect and restore environmental quality. The National Environmental Policy Act states, in part, that Federal agencies have a continuing responsibility to use practical means, consistent with other essential considerations of national policy, to create and maintain conditions under which man and nature can exist in productive harmony. Federal plans, functions, and programs are to be improved and coordinated to—

- Preserve the environment for future generations.
- Assure safe, healthful, productive, and aesthetically pleasing surroundings.
- Attain the widest beneficial uses of the environment without degradation, risk to health or safety, or other undesirable consequences.
- Enhance the quality of renewable resources and approach the maximum attainable recycling of depletable resources.

6-13. Federal agencies, in response to the National Environmental Policy Act, must be concerned with the impact of their activities on technical and economic considerations and the environment.

United States Army Environmental Quality Program

6-14. AR 200-1 outlines the Army fundamental environmental policies, management of its programs, and its various types of activities. One of which, water resources management, includes minimizing soil erosion and attendant pollution caused by rapid runoff into streams and rivers. The overall goal is to *plan, initiate, and carry out actions and programs in a manner that minimizes adverse effects* on the quality of the human environment without impairment of the Army mission. A primary objective is to eliminate the discharge of pollutants produced by Army activities. Provision of suitable surface drainage facilities is necessary in meeting this objective.

United States Air Force Environmental Quality Program

6-15. AFPD 32-70 outlines Air Force policy in compliance with the National Environmental Policy Act executive orders and Department of Defense directives. Procedures outlined in AFPD 32-70 are similar to those described for Army installations. AFI 32-7061 establishes 32 CFR 989 as the controlling document on environmental assessments and statements for Air Force facilities.

United States Navy Environmental Quality Program

6-16. The Navy environmental quality initiative is a comprehensive initiative focused on maximizing the use of pollution prevention to achieve and maintain compliance with environmental regulations. The environmental quality initiative is a fundamental part of the Navy environmental strategy called assess, implement, manage and measure to achieve sustained compliance and operational readiness through environmental excellence.

Environmental Impact Analysis

6-17. ATP 3-34.5/MCRP 4-11B presents recommended procedures for use by Army personnel in preparing and processing environmental impact assessments and environmental impact documentation. The procedures list the step-by-step actions considered necessary to comply with the requirements of the National Environmental Policy Act and subsequent guidelines.

Environmental Effects of Surface Drainage Systems

6-18. Arctic or subarctic facilities could have beneficial or adverse environmental impacts affecting water, land, ecology, and socioeconomic (human and economic) considerations. Despite low-population density and minimal development, the fragile nature of the ecology in the arctic and subarctic has attracted the attention of environmental groups interested in protecting these unique assets.

6-19. Effects on surrounding land and vegetation may cause changes in various conditions in the existing environment (surface water quantity and quality, groundwater levels and quality, drainage areas, animal and aquatic life, land use). Proposed systems may also have social impacts on the community, requiring relocation of military and public activities, open space, recreational activities, community activities, and quality of life.

6-20. Environmental attributes related to water could include erosion; aquifer yield; flood potential; flow or temperature variations (the latter affecting permafrost levels and ice jams); biochemical oxygen demand; and content of dissolved oxygen, dissolved solids, nutrients, and coliform organisms. These are among many possible attributes to be considered in evaluating environmental impacts, beneficial and adverse, including affects on surface water and groundwater.

Reconnaissance

6-21. Before construction starts, preliminary area reconnaissance should disclose features that require advance drainage planning and operations. These features include—

- Springs and seepage on hillsides that 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 these areas could change the movement of the flow.
- Vegetation or cover that could increase surface runoff if removed during the clearing and grubbing phase of construction.
- Level areas with 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

6-22. Controlling runoff during construction can be costly. The following measures help maintain satisfactory drainage during construction:

- Make maximum use of existing ditches and drainage features. Where possible, grade downhill to allow economical grading and take advantage of natural drainage.
- Use temporary ditches to help construction drainage. Ensure that efforts are made to drain pavement subgrade excavations and base courses to prevent detrimental saturation. Carefully consider the drainage of 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.
- 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

6-23. Considerations for clearing for the purpose of adequate drainage is discussed below. Natural drainage patterns is always preferred.

Existing Drainage

6-24. Clear excess vegetation from streams. This increases the velocity and quantity of flow. Widening the stream can 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. Changes can adversely affect other parts of the stream.

Vegetation Removal

6-25. Military projects may require the removal of 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 ponding and may otherwise effect adjacent areas.
- Be aware that removing the vegetation from an area can lead to excessive surface runoff and erosion. This could lead 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.
- Control runoff sediments in large, cleared areas to prevent failure of other structures and possible adverse environmental effects. This is more of a concern in permanent construction than in wartime JOA construction.

Clearing, Grubbing, and Stripping

6-26. Clearing, grubbing, and stripping 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 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

6-27. When placing fill, exercise firm control over the project to prevent adverse affects from improper drainage procedures. Consider the following factors:

- The fill section must be rolled smooth at the end of each working day to seal the surface. No area should be left that can hold standing water.
- The fill surface must be free of ruts caused by trucks and other equipment. These depressions collect rain and saturate the subgrade. 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

6-28. Use interception ditches during construction to collect and divert surface runoff before building the designed system. Before 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.

6-29. Locate interceptor ditches on hillsides and at the foot of the slope to intercept and divert runoff from the construction site. Make these ditches part of the final drainage system if possible. Roadside ditches are required during each construction stage and should be placed at design locations.

6-30. During construction, use deep ditches for subsurface drainage. They intercept groundwater flow. (See figure 6-1.) If groundwater flow must be intercepted but ditching is not possible, modify the ditch into a subsurface drainpipe system.

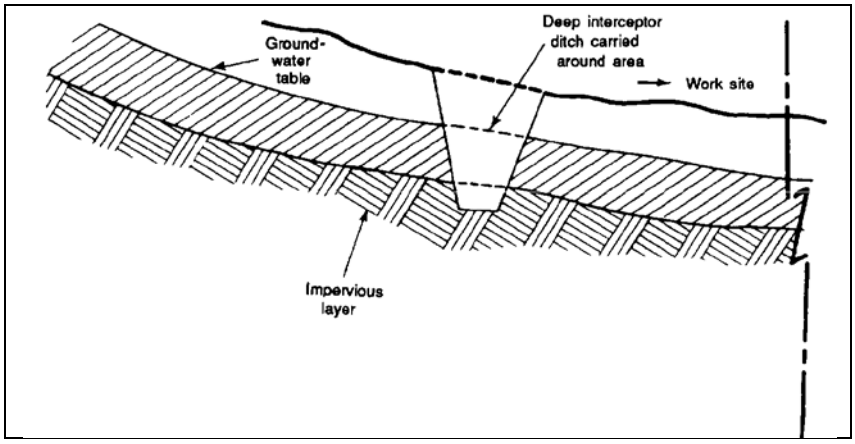


Figure 6-1. Deep interceptor ditch

6-31. Ditching may be required in swamp areas to continue drainage ditches to an outlet point or drain the area. Engineers may use explosives in some cases since the soil may not be capable of supporting construction equipment. Draglines should also be considered. Timber or steel mats can be used to provide a firm foundation and support equipment during the operation.

Culverts

6-32. 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 dry. If the 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

6-33. The design capacity of surface drainage systems should economically drain the facilities with consideration to the mission and importance of the particular facility and environmental impacts. The degree of protection provided by the drainage system is determined according to the type of traffic, the volume of traffic, and the necessity for uninterrupted service. Although the degree of protection should increase with the importance of the facility, minimum requirements must be adequate to avoid operation hazards. The possibility of a severe accident due to inadequate drainage offsets the cost difference between adequate and inadequate drainage facilities.

6-34. To protect important facilities, the use of design storm frequencies above minimum criteria can be justified. In some designs, portions of the drainage system are based on as high as a 50-year design frequency to prevent loss of life and to reduce the likelihood of flooding a facility essential to operations.

PRECIPITATION

6-35. 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/AFM 88-19. The amount of rainfall that evaporates depends on the surface temperature of ground features, air temperature, wind speed, and relative humidity. Evaporation occurs while rain is falling to the ground and after it lands on vegetation and other ground cover.

6-36. Rainfall may be intercepted by vegetation. 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.

6-37. A significant portion of water that strikes the soil soaks into the ground. This process is called infiltration. The rate of absorption and the quantity of water absorbed depends on the soil type, vegetation, terrain slope, and the soil moistness before the rain. Storm water runoff begins to accumulate when the rate of rainfall exceeds the rate of infiltration.

6-38. Before overland water flow can begin its downhill motion, it must be deep enough to overcome obstacles to its movement. Detention is the amount of water required to fill depressions on the surface of the earth. Except by infiltration or evaporation, no water can leave a depression until the holding capacity of the depression has been exceeded.

6-39. Evaporation, interception, infiltration, and detention are moisture losses. Runoff is precipitation minus these moisture losses.

STORMS

6-40. 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 also describes procedures for determining maximum storms. The five main types of storms are—

- **Thunderstorms.** Thunderstorms are local atmospheric disturbances of short duration and a 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 cover large areas for several hours with moderate intensity. These storms 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 often have several peaks of high rainfall. They may last several days and develop very large amounts of precipitation at relatively low average rates of rainfall. With a low average rate of rainfall, these storms have little or no impact to small- or medium-sized drainage basins, but they normally control the design of drainage structures for large basins.

- **Monsoons.** Monsoons are seasonal winds of the Indian Ocean and southern Asia. These winds blow from the south from 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.

Duration

6-41. Duration is the length of time that 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 hydrographs 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.

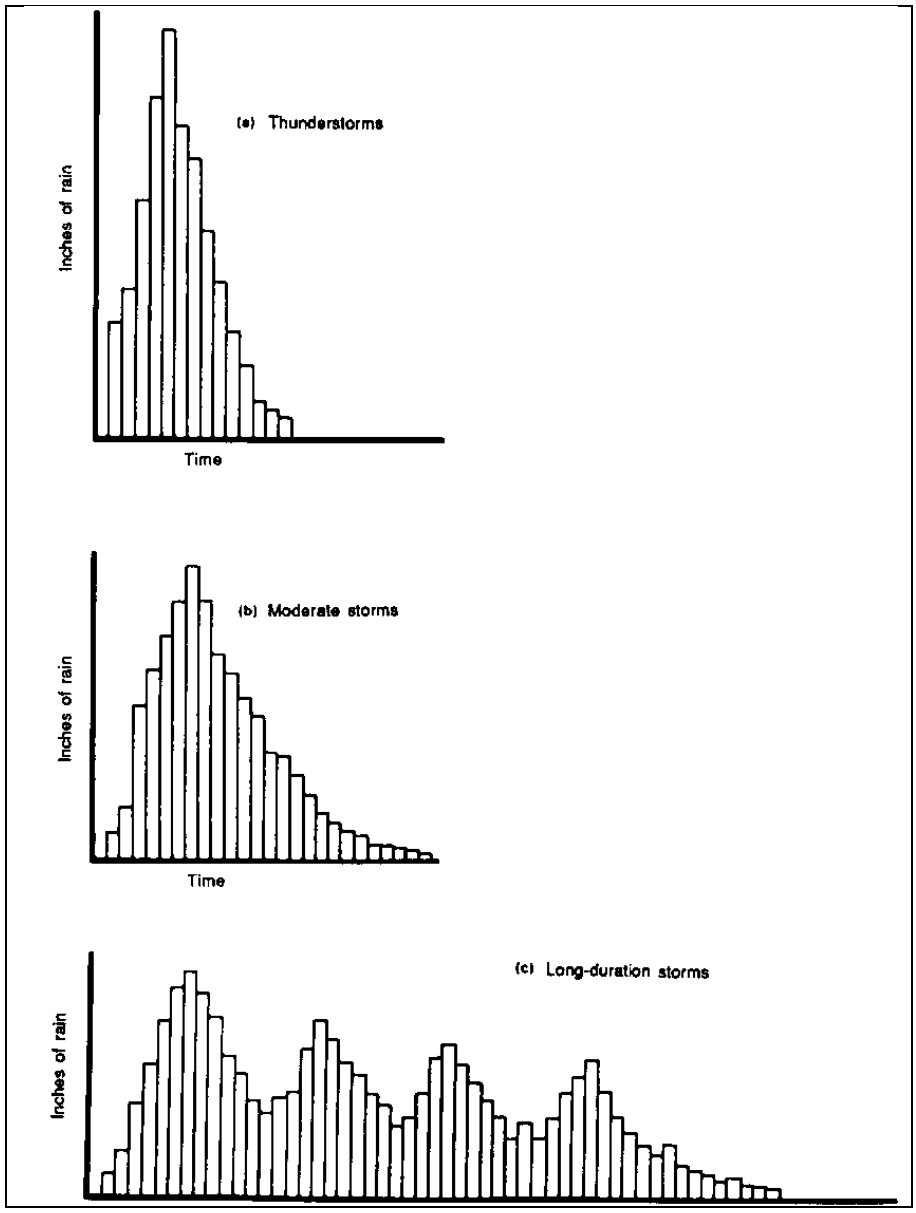


Figure 6-2. Typical storm hyetographs

Design Life Versus Actual Structure Life

6-42. The design life of a structure is the period of time during which the item is expected by its designers to work within it specified parameters; in other words, the life expectancy of the item. It is the length of time between placement into service of a single item and that items on-set of wear-out.

Design Storm Frequency

6-43. For airfields and heliports, a minimum two-year storm event is required unless a waiver is obtained. This event will have no encroachment of runoff on taxiway and runway pavements (including paved shoulders). After this design storm frequency is specified, computations must be made to determine the critical duration of rainfall required to produce the maximum runoff rate for each area. This depends on the slope and length of the overland flow. Another important aspect of the design is minimizing ponding during rain events. Ponding is the accumulation of water around an inlet structure during a rain event. Typically, ponding will be limited around the apron inlets so that it does not exceed 4 inches.

6-44. For areas other than airfields, such as developed portions of military installations (roadways, administrative, industrial, housing areas), the design storm is based on rainfall of a 10-year frequency. Potential damage or operational requirements may warrant a more severe criterion; in certain storage and recreational areas, a lesser criterion may be appropriate. With concurrence of the using Service, a lesser criterion may be employed in regions where storms of appreciable magnitude are infrequent, and large expenditures for drainage are not justified based on damage or operational capabilities. An additional design consideration is the spread of water around inlets. Spread is the width of the water on the paved surface measured perpendicular to the curb face.

Surface Runoff from Storms Exceeding Design Storm

6-45. The design storm frequency alone is not a reliable criterion of the adequacy of storm drain facilities. It is advisable to investigate the probable consequences of storms more severe and less frequent than the design storm before making final decisions regarding the adequacy of proposed drain-inlet capacities. Surface runoff from storms greater than the design storm will be disposed of with the minimum damage to the airfield or heliport. The center 50 percent of runways, the center 50 percent of taxiways serving these runways, and helipad surfaces along the centerline should be free from ponding resulting from storms of a 10-year frequency and intensity determined by the geographic location. For areas other than airfields and heliports, check with the appropriate local regulatory agency for guidance on design storm requirements.

6-46. For the two-year frequency storm, the probability of having a storm equaling or exceeding the value in any one year is 0.5 (two out of four times). The design storm frequency for JOA 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.

6-47. 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 every two years on average, but 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 within a two year time period is 75 percent (three out of four times). Details of the statistics involved can be found in hydrology textbooks.

WEATHER DATA

6-48. If there is extensive rainfall and rain-rate records for the location of interest and if hydrologists have examined those records statistically to formulate intensity-duration tables, those tables can be obtained through the Air Force staff weather officer. The staff weather officer is normally located at the division level.

6-49. Within the United States, the data generally comes from the National Weather Service, 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 JOA 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 that shows the isointensity lines for a 60-minute, 2-year storm.

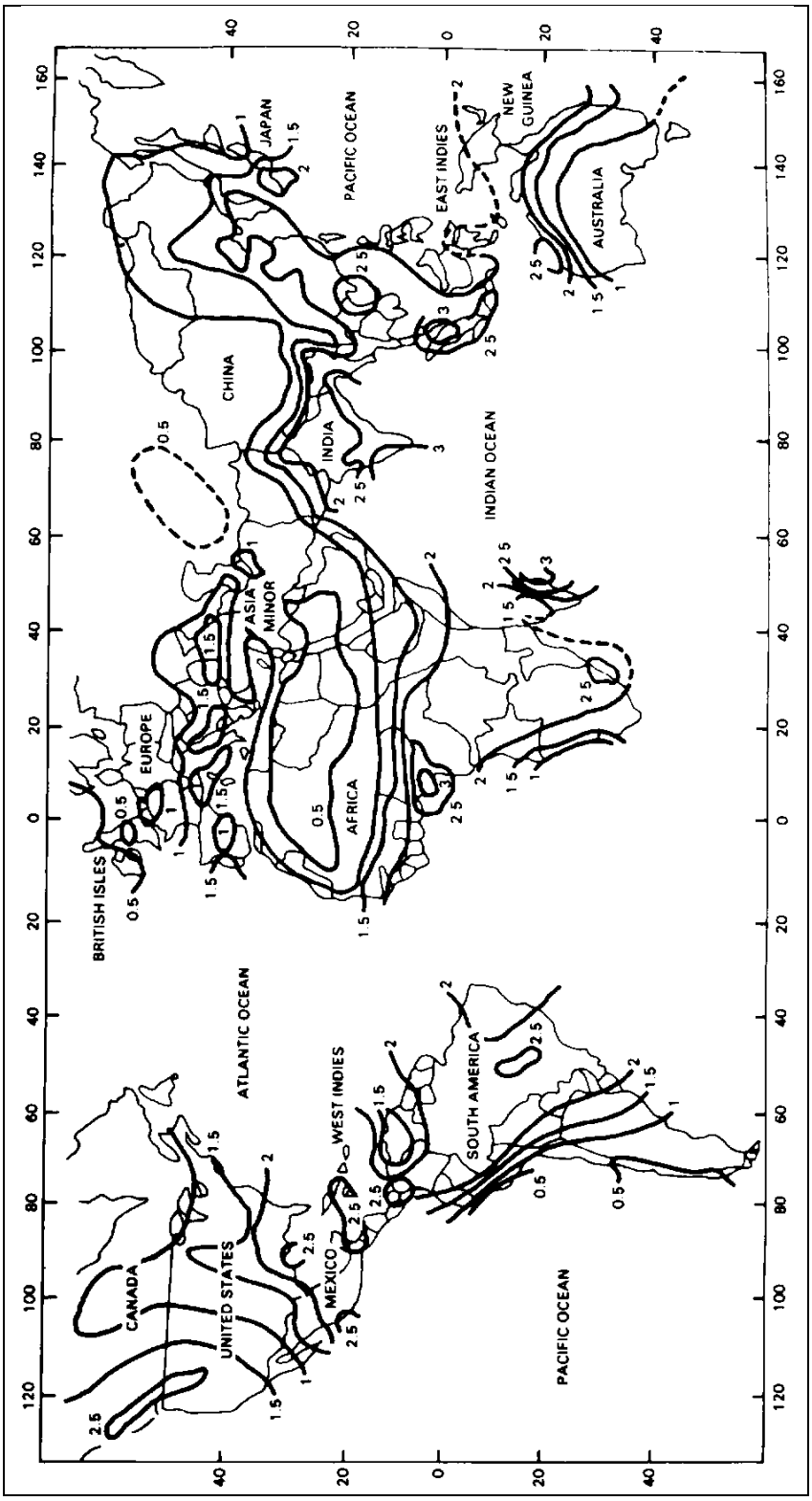


Figure 6-3. World isohyetal map

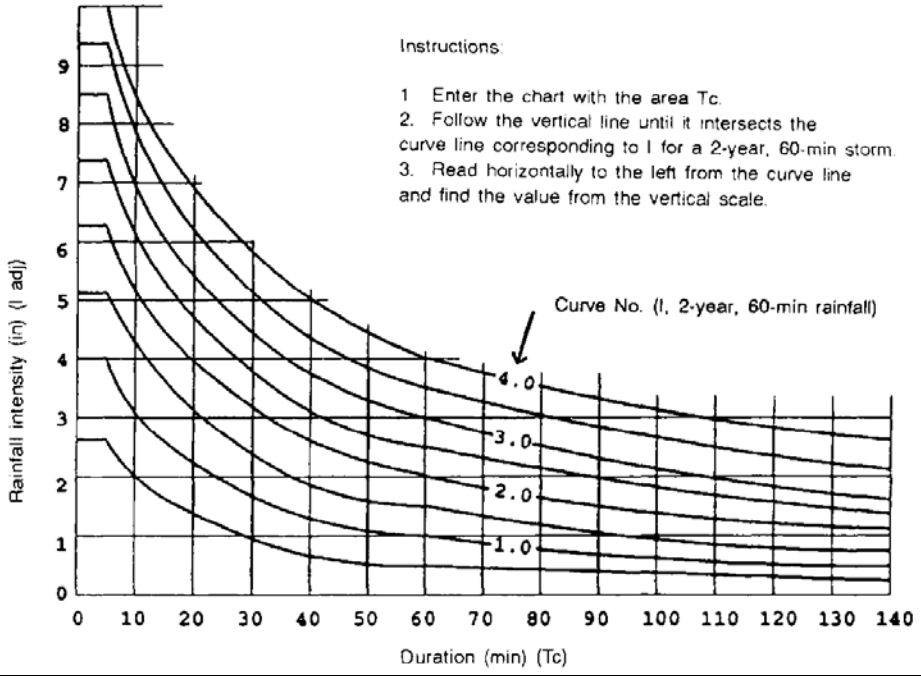
6-50. To properly read a value on the isohyetal map, find the project location, and read the value of the appropriate isohyets. Do not interpolate. If a project location falls—

- 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.

6-51. For example, the isohyet values in inches per hour for—

- Southern Australia is 1.0.
- North Dakota is 1.5.
- Florida is 2.5.
- Washington, DC, is 1.5.
- Vietnam is 2.5.
- Cuba is 2.5.
- New Orleans, LA, is 2.5.

6-52. These intensities 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 shown in figure 6-4.



Legend:

adj	adjustment
I	intensity
in	inch
min	minute
No.	number
Tc	duration in minutes

Figure 6-4. Standard rainfall intensity-duration curves

6-53. In figure 6-4, the standard curves are numbered 1.0, 2.0, 3.0, and 4.0, with intermediate values readily interpolated. Curve number 1 passes through 1 inch per hour at 60 minutes, and curve number 2 passes through 2 inches per hour at 60 minutes.

6-54. Where intensity is known for any nonarctic location (taken from the isohyetal map) and critical duration is calculated, the intensity can easily be determined. The standard intensity-duration curves are applicable to any frequency, not just a two-year frequency.

6-55. To use figure 6-4, enter the graph using the duration in minutes. 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 not from an intensity-duration curve). (See table 6-1.)

Table 6-1. Example critical duration calculation

I60, two-year Intensity (inches per hour)	Critical Duration (minutes)	Intensity Adjustment (inches per hour)
1.0	50	1.2
1.5	30	2.4
2.0	10	5.2
Legend: I intensity		

RUNOFF

6-56. 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 (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. Runoff begins sometime after the beginning of precipitation and may continue long after precipitation ends.

6-57. The total quantity of runoff from a given area, after it is collected in channels and streams, is the flow estimate used to design area drainage structures.

6-58. 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.

HYDROGRAPH

6-59. 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 hydrograph depicts the fluctuations of flow with regard to time.

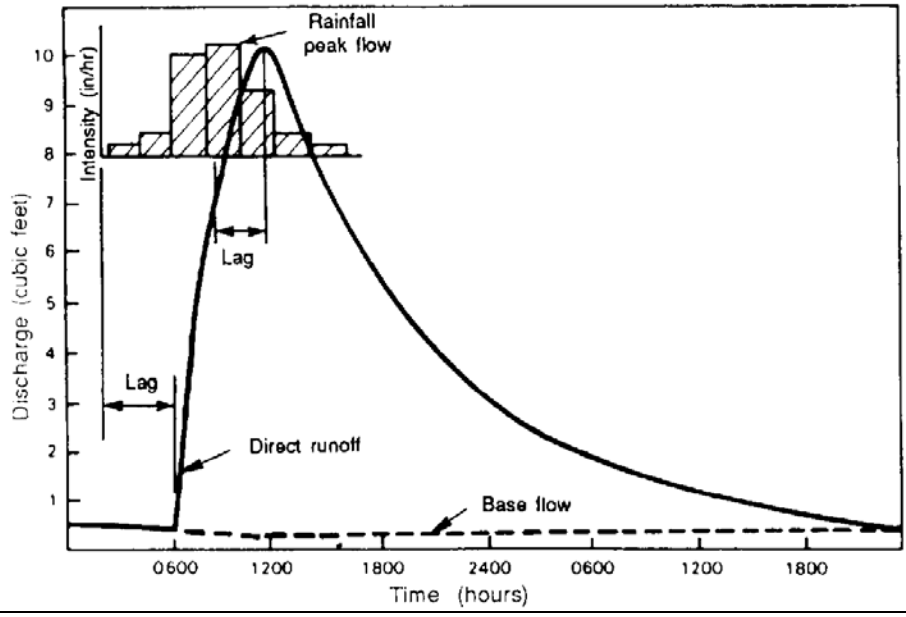
6-60. The elements of a hydrograph are base flow, lag time, peak flow time of concentration (TOC), and flow volume. Each stream will have its own characteristic hydrograph with widely varying values for the elements. A typical stream-flow hydrograph is shown in figure 6-5, page 6-12.

BASE FLOW

6-61. The base flow of a stream depends on the amount of groundwater that seeps into the stream and its tributaries from the 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 periods of time.

LAG TIMES

6-62. When precipitation begins over an area, there is an initial period where the loss factors induced by interception, infiltration, and detention take effect before 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.



Legend:	
hr	hour
In	inch

Figure 6-5. Typical hydrograph

6-63. The length of this lag time is influenced by 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 that the storm reaches its peak precipitation rate and the time that 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 moderate-sized drainage areas, there will be slight differences between storm peak and stream peak.

PEAK FLOW

6-64. The peak of the hydrograph 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.

CONCENTRATION TIME

6-65. The TOC is the time it takes a drainage basin to begin contributing runoff to the stream. Assuming uniform rainfall, the hydrograph 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—the storms critical duration.

FLOW VOLUME

6-66. The area under the curve of the hydrograph indicates the total flow (in cubic feet) that results from a particular storm. It is used to determine ponding times when it is practical to use culverts with submerged inlets.

HYDROGRAPHY CONSTRUCTION

6-67. A hydrograph is constructed by measuring a streams rise and fall and the times related to these changes in flow. When constructing a hydrograph—

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

DESIGN DATA REQUIREMENTS

6-68. 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 topography, meteorological records, soil characteristics, and available construction resources.

6-69. Give special attention to the vicinity of the proposed facility and the presence of any topographical features that may contribute runoff to the project area. The completed facility will often interfere with the natural drainage of the site. Therefore, when analyzing the effects of the surrounding terrain—

- Identify areas that contribute runoff to the site.
- Determine the general size and shape of the 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.

6-70. 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.

6-71. 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 that the rainfall infiltrates the ground, which greatly influences the presence and movement of subsurface water.

6-72. 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

6-73. 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.

6-74. The drainage system must be planned and designed for the predetermined location of the facility. There are three basic procedures in drainage structure design:

- Determine the area (usually in acres) contributing runoff to the facility.
- Estimate the quantity of runoff for which the facility will be protected.
- Design the drainage system to carry the estimated runoff.

DETERMINE THE AREA CONTRIBUTING RUNOFF

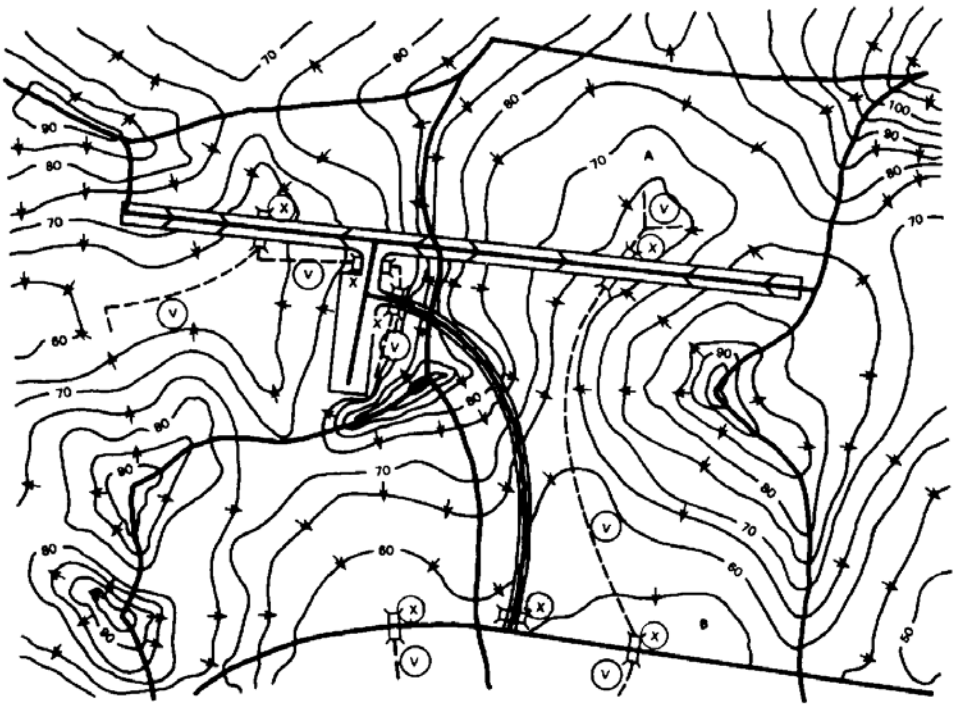
6-75. When developing a tentative layout for a drainage system, identify locations within the site that requires drainage structures because of topographical or manufactured features. This is best done from a topographic map of the area or a sketch of the project site.

6-76. Next, determine the acreage of the areas that contribute runoff to these required drainage structures. An analysis of existing channels is helpful to establish locations for required structures. Upon completion, this tentative plan should be field checked at the project site.

Establish Drainage-Structure Locations

6-77. The initial step in developing a drainage-structure layout is to establish the location of the required drainage structures. Generally, placement will be controlled by the topography. For example, a fill section that 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.

6-78. Figure 6-6 shows an airfield with required culverts and open channels or ditches. 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 should be as straight and smooth as possible. Sharp bends in ditches or near culverts will cause erosion. Not shown on this sketch are the standard ditches constructed along the sides of military roads and airfields.



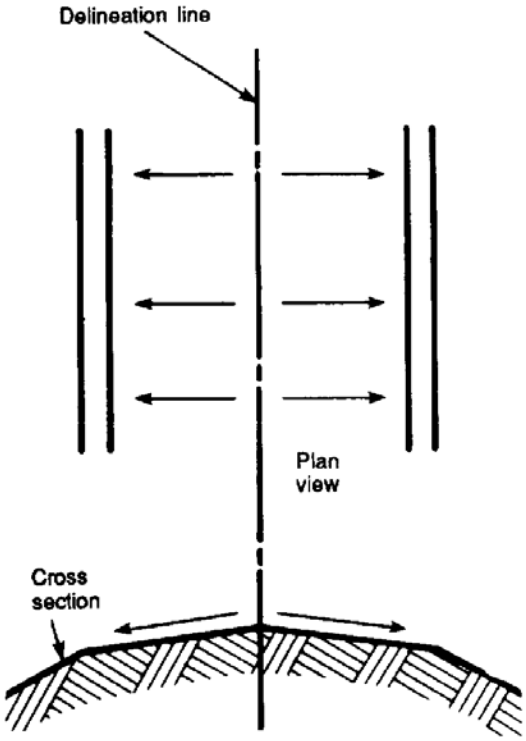
Legend:	
V	open channel or ditch
X	culvert

Figure 6-6. Typical airfield drainage features

Delineate Watersheds

6-79. 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 the following six simple steps:

- **Step 1.** Locate existing or proposed drainage structures on the topographic map or sketch.
- **Step 2.** Identify and mark 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 on the culvert or the end of the ditch. 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 are located so that flow arrows cannot cross them. Flow arrows cross delineation lines at culverts or ditches. (See figure 6-7 and figure 6-8, page 6-16.)
- **Step 6.** Since each cover or soil type will have an effect on the basin, if there are 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 6-9, page 6-16.)



Note. Delineation lines are drawn between opposing arrows.

Figure 6-7. Delineation of roads and airfields

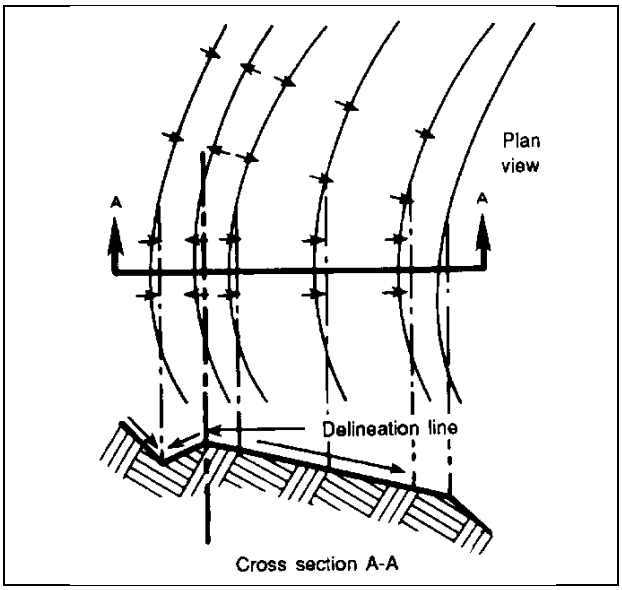


Figure 6-8. Delineation of superelevated roads

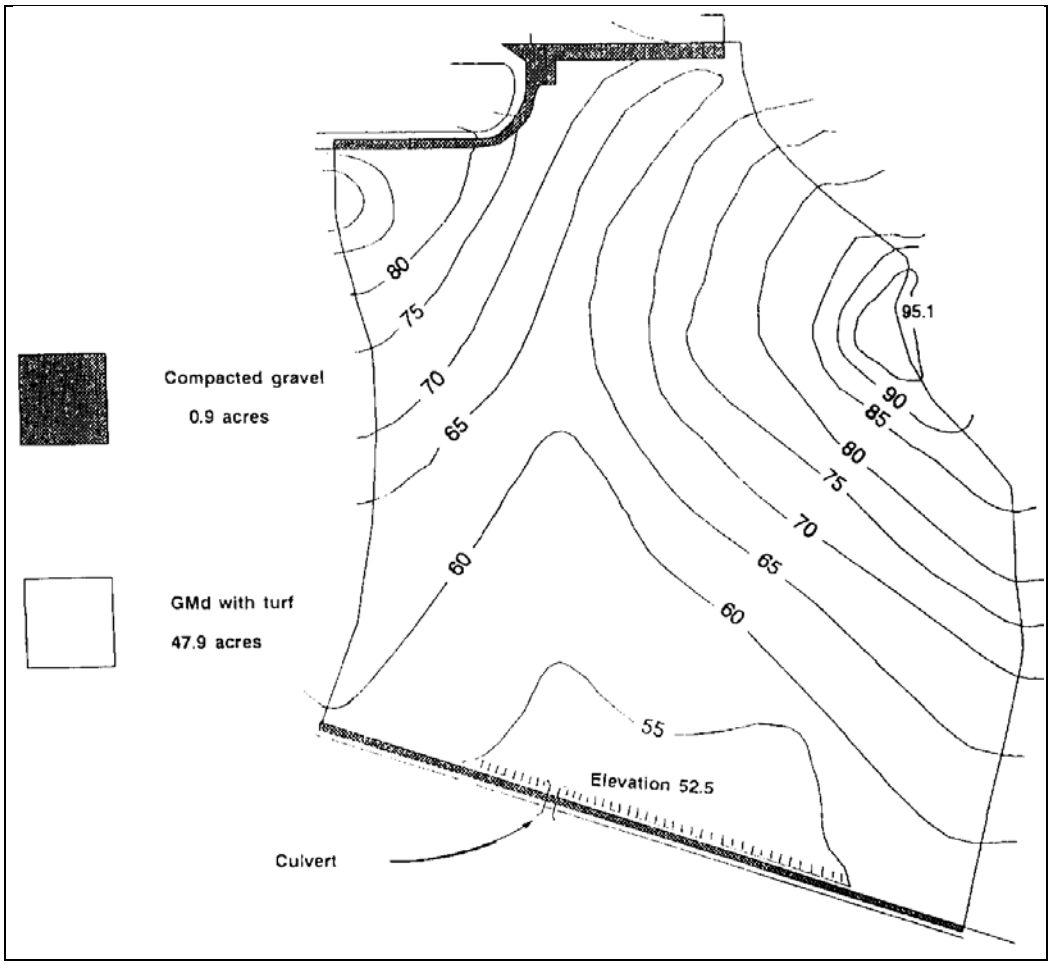


Figure 6-9. Area delineation by cover or soil type

6-80. Figure 6-7, page 6-15, and figure 6-8 depict the use of flow arrows and delineation lines for special manufactured structures (roads, airfields, superelevated roads). When airfields or straight roadways are properly constructed, they are 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 where the accumulated storm water would separate and flow in opposing directions. Figure 6-8 shows how superelevated roadways (roadways that are banked to ease the flow of traffic through a curve) are delineated. In a properly constructed, superelevated road, storm water will always separate at the outside edge of the curve. See figure 6-10; figure 6-11, page 6-18; and figure 6-12, page 6-18, for illustrations of special terrain features (hills, ridges, valleys, saddles).

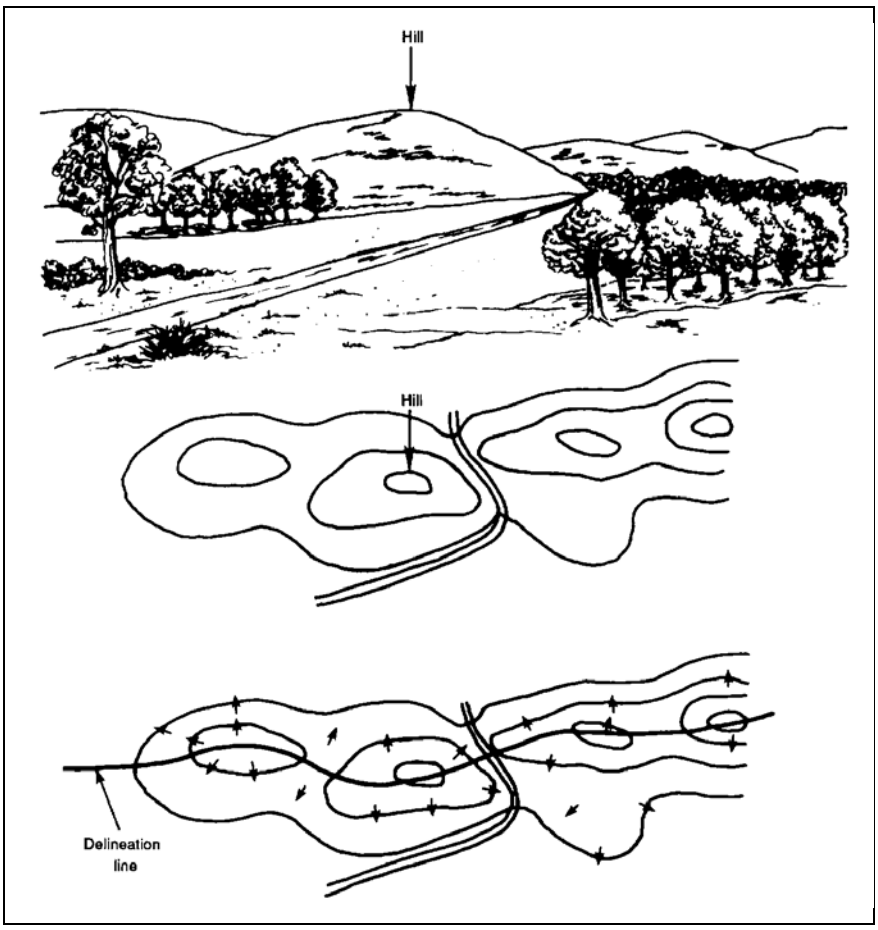


Figure 6-10. Delineation of a hill

Determine Drainage System Size

6-81. 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 an accurate method of measurement. A planimeter (measures the area of a plane figure while a mechanically coupled pointer traverses the figure perimeter) is quite accurate and should be used if available. However, several other methods are suitable for field estimation.

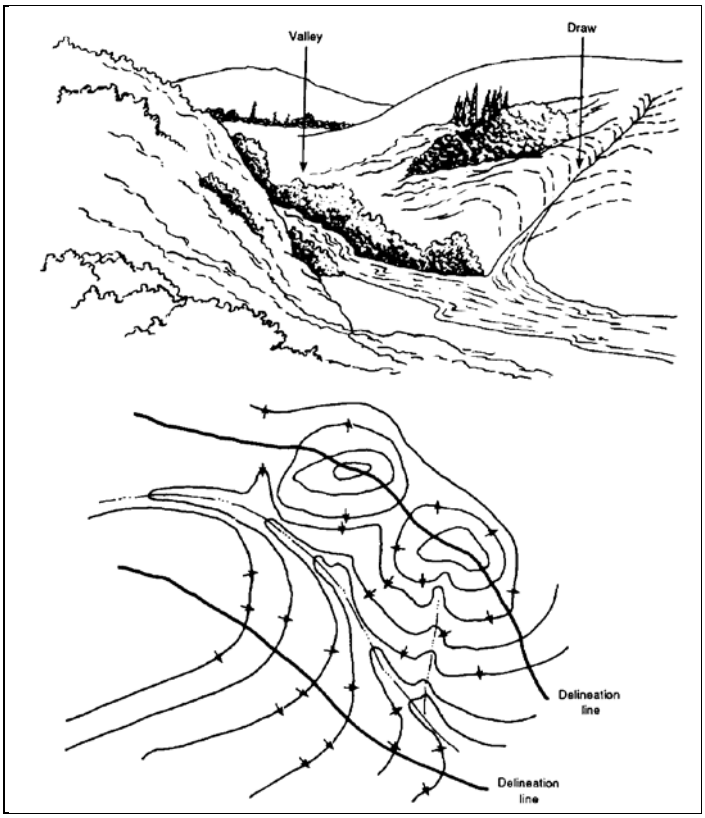


Figure 6-11. Delineation of valleys and draws

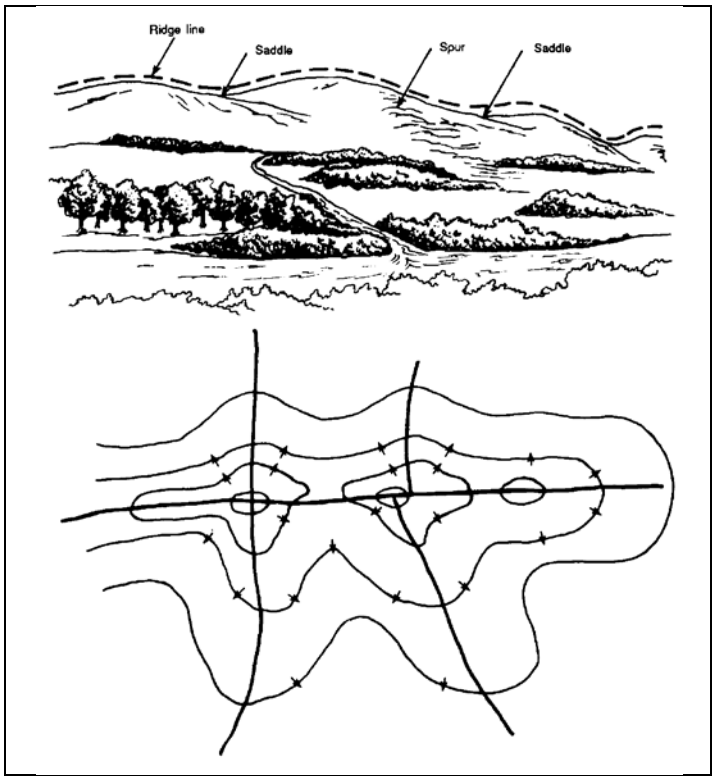
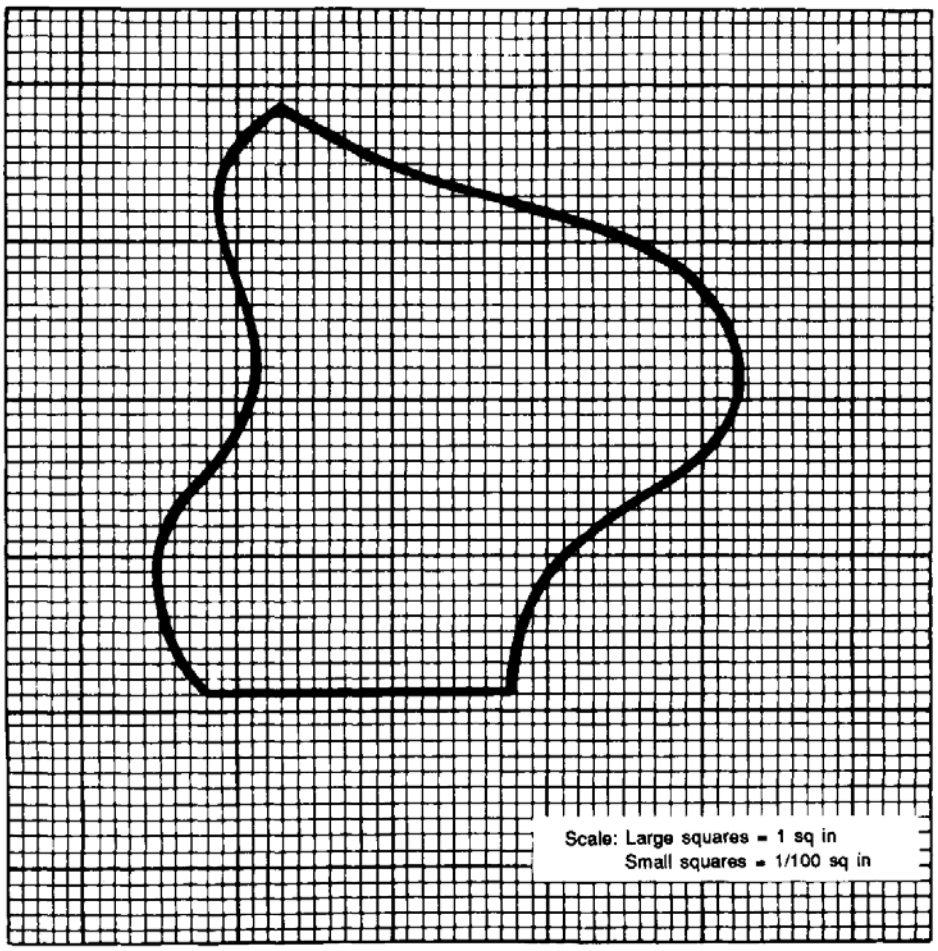


Figure 6-12. Delineation of ridges, spurs, and saddles

Counting-Squares Method

6-82. To make a hasty approximation of an area, transpose the outline of the watershed to graph paper or another 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 this measurement into acres (1 acre equals 43,560 square feet). (See figure 6-13.)



Legend:

ft	feet
in	inch
sq	square

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

Geometric-Shapes Method

6-83. This method involves estimating the watershed shape in terms of rectangles, triangles, or trapezoids. Determine the area of each shape, and total the shapes to estimate the area of the watershed. (See figure 6-14.) Determine the area of each shape using the following formulas:

- **Rectangle.** $Area = base \times height \ or \ A = bh$

  where—

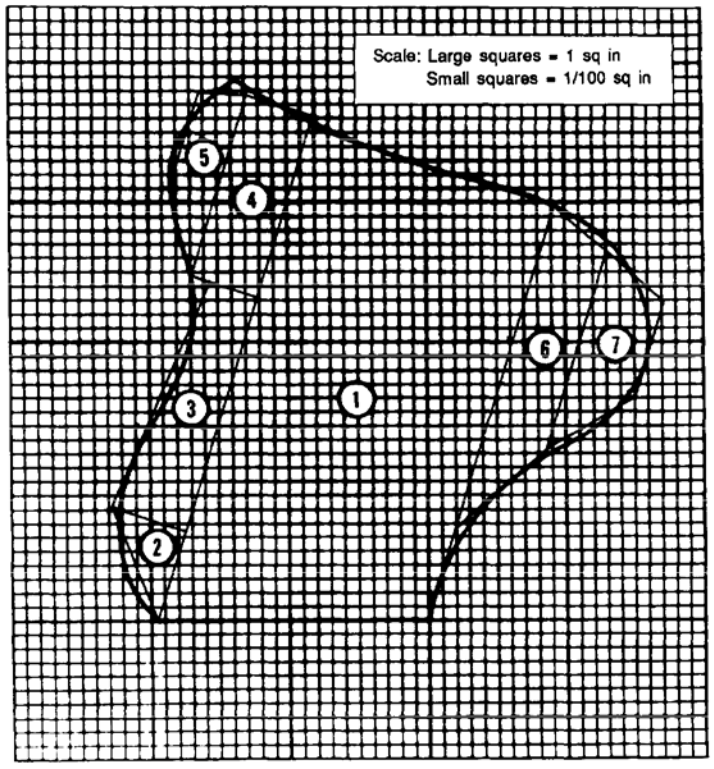
  A = *area*

  b = *base*

  h = *height*

- **Triangle.** $Area = \frac{base \times height}{2} \ or \ A = \frac{bh}{2}$

- **Trapezoid.** $Area = sum \ of \ bases \times height \ or \ A = (b1 + b2)(h)$



Calculations:

Area #1 = trapezoid = 1/2 (3.90 + 3.10) 1.94 = 6.79 square inch
Area #2 = triangle = 1/2 (0.5)(0.93) = 0.23 square inch
Area #3 = trapezoid = 1/2 (0.5 + 0.34) 1.35 = 0.57 square inch
Area #4 = rectangle = 0.30 x 1.56 = 0.47 square inch
Area #5 = trapezoid = 1/2 (1.43 + 0.75) 0.85 = 0.93 square inch
Area #6 = trapezoid = 1/2 (2.37 + 1.65) 0.41 = 0.82 square inch
Area #7 = trapezoid = 1/2 (1.65 + 0.94) 0.50 = 0.69 square inch

Example:

If one square represents 10,000 actual square feet on the ground, then the designated area represents:

$$\frac{10.5 \times 10{,}000}{43.580} = 2.41 \ acres$$

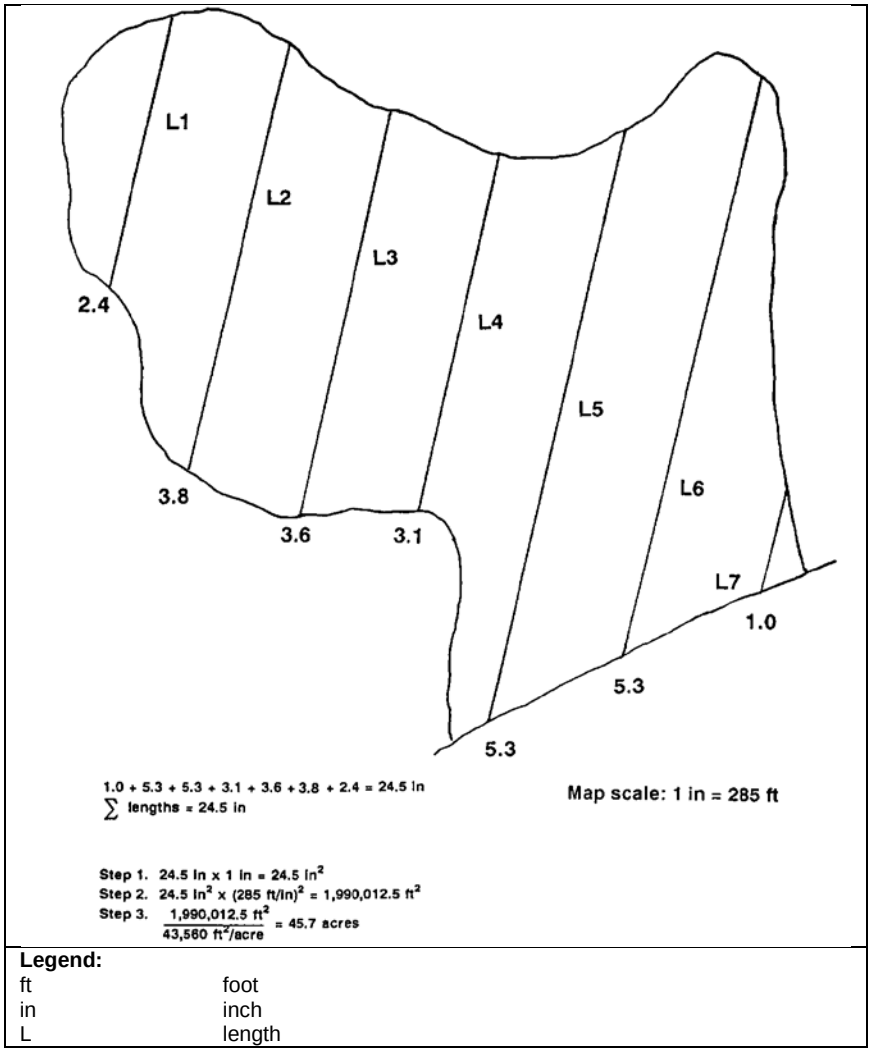
Legend:

in	inch
sq	square

Figure 6-14. Area measurement, geometric-shapes method

Stripper Method

6-84. The stripper method is a variation of the geometric-shapes method. (See figure 6-15.) Approximate the area by drawing a series of lines that are equal distance (a stripper width of 1 inch) across the delineated area, measure the lines, and total. This method is more applicable for field estimations.



Legend:

ft	foot
in	inch
L	length

Figure 6-15. Area measurement–stripper method

6-85. The total of the lengths is 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 the total of the length (in inches) by the stripper width (in inches) by the map scale (in feet per inch) by 2 and dividing the product by 43,560 feet per square acre.

Example

Use the stripper method to estimate the area. Given—

L = 12.5

Map scale = 175 feet per inch

Stripper width = 1 inch

$$12.5 \times 1\ inch = 12.5\ inch^2$$

$$12.5\ inch^2 \times (175\ feet/inch)^2 = 382{,}812.2\ feet^2$$

$$\frac{382{,}812.5\ feet^2}{43{,}560\ feet^2 \quad / acre} = 8.78\ acres$$

where—

L = length

ESTIMATE THE QUANTITY OF RUNOFF

6-86. Drainage systems must be designed to accommodate the peak flow generated by runoff from contributing watersheds during the design storm. Many techniques are available for determining the peak flow, but most are too complex for general field use. This manual discusses the most common method for estimating runoff—the rational method.

DESIGN DRAINAGE STRUCTURES FOR MAXIMUM RUNOFF

6-87. To accommodate peak flow of the design storm, design structures must provide a sufficient cross-sectional area and longitudinal slope for passing storm runoff. If ponding or flooding of adjacent areas must be prevented, the design must be for peak flow. However, the water velocities generated at peak flow must not be so great that they cause damage to the drainage structure or excessive erosion and scouring to the protected facility.

6-88. Determine the capacity of drainage structures by calculating the runoff from contributing drainage areas. Specific procedures for designing open channels and culverts are discussed later in this chapter.

ESTIMATE RUNOFF USING THE RATIONAL METHOD

6-89. 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

6-90. 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 (rainfall is uniform over time and space).
- No active streams drain the area. If an active stream drains the basin, use the hasty method. (See TM 3-34.85/MCRP 3-17A for additional information.)

FORMULA

6-91. The rational method uses the following formula:

$$Q = CIA$$

where—

Q = *quantity of runoff (cubic feet per second)*

C = *runoff coefficient*

I = *rainfall intensity (inches per hour)*
A = *drainage area (acres)*

6-92. The following conversion factor is applied to the formula:

$$\frac{1\ acre \times 1\ inch}{1\ hour} = 1.0083\ cubic\ feet\ per\ second$$

6-93. This is so close to unity that no correction factor is added. Hence, the name rational is used because of a rational conversion of units. The rational formula has three variables (runoff coefficient, intensity, and drainage area).

Runoff Coefficient Variable

6-94. The runoff coefficient accounts for losses from precipitation. The runoff coefficient 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—

$$C = \frac{runoff}{rainfall}$$

where—
C = *runoff coefficient*

6-95. Table 6-2 gives conservative values of the runoff coefficient variable. Knowledge of an area USCS classification or an estimate of the soil permeability allows selection of a runoff coefficient variable. The runoff coefficient variable also appears in the table for manufactured surfaces and wooded areas. An area of SP soil (a pervious, sandy soil with a slope less than or equal to 2 percent) with turf has a runoff coefficient variable factor of 0.10; that is, 10 percent of the rain falling on this soil will run off. The remaining 90 percent becomes lost to runoff through infiltration and other factors. At the other extreme, an asphalt pavement has a runoff coefficient variable of 0.95. Five percent of the rain falling on the asphalt will be lost. The remaining 95 percent is expected to become runoff.

Table 6-2. Runoff coefficients

Soil or Cover Classification	***C Variables***					
	Slope ≤2%		***Slope >2% and <7%***		***Slope ≥7%***	
	Turf	***No Turf***	***Turf***	***No Turf***	***Turf***	***No Turf***
GW, GP,SP, SW	.10	.20	.15	.25	.20	.30
GMd, MH, ML, Pt, SMd	.30	.40	.35	.45	.40	.50
CH, CL, GC, GMu, OH, OL, SC, SMu	.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

Legend:
C runoff coefficient
LL liquid limit
PI plasticity index

Note. The runoff coefficient variables that are given in table 6-2 are maximum 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 these maximums, refer to a reliable civil engineering text dealing with hydrology. The table is arranged with three columns for varying slope conditions.

Runoff Coefficient Variable Versus Slope

6-96. As terrain becomes steeper, water flows sooner and more rapidly. This allows less time for infiltration to occur and results in a larger runoff coefficient variable for natural cover or soil categories. For this reason, when the average slope of an area exceeds 2 percent, an adjustment must be made.

6-97. Table 6-2, page 6-23, has three columns representing different slope conditions and their corresponding runoff coefficients. Use the column that corresponds with the average slope percentage.

6-98. The runoff coefficient variable for turfed soil is different from the runoff coefficient variable for bare soil. The turf (grass or other ground cover) exerts a drag on water that causes a slower flow and provides more time for infiltration to occur. Therefore, a lower runoff coefficient results. Denuded soil (the turf or cover has been removed from the soil) requires an increased runoff coefficient variable because a swifter flow results in less time available for losses to occur. If one cover type has more than one flow path, average the slopes and use the appropriate column.

Example

Average the slope for flow paths 1A and 1B. Given—
Flow path slope 1A = 2.3%
Flow path slope 1B = 1.9%.

$$\frac{2.3\% + 1.9\%}{2} = 2.1\%\ slope$$

Nonhomogeneous Areas

6-99. An assumption made when using 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.

6-100. If one soil and cover type is predominate in at least 80 percent of the area, the area is considered a simple area, and the runoff coefficient variable for the predominant soil and cover type is used. If no one soil and cover type is predominate in 80 percent of the area, the area is considered a complex area. The runoff coefficient variable must be weighted in a complex area. The runoff coefficient variable is adjusted to account for the runoff coefficient variable proportions contributed by each subarea. For example, a complex area has two subareas: one with average turf and the other with bare soil. The bare soil slope does not affect how fast (or slow) water runs off the turfed area or 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 6-2 shows the runoff coefficient variables for slopes with and without turf.

6-101. Multiply the runoff coefficient variables by the area (in acres) that the runoff coefficient variables affect, total the products, and divide by the total acreage. Expressed mathematically, the formula is—

$$C = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \cdots C_nA_n}{A_1 + A_2 + A_3 + \cdots A_n}$$

where—
C_1A_1 = runoff coefficient variable and area for first subarea
C_2A_2 = runoff coefficient variable and area for second subarea

Intensity Variable

6-102. 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 are available. The task of calculating the critical duration for any given drainage area is detailed here.

Drainage Area Variable

6-103. The drainage area is the area that contributes storm water runoff to the culvert or ditch being designed. It must be calculated in acres.

TIME OF CONCENTRATION

6-104. 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 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 segments of the area at the same time. This time is called the area TOC. A storm with a 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 would only be filled with the runoff from nearby segments. Therefore, runoff would not be maximized.

6-105. In figure 6-16, 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 drain in 10 to 20 minutes. Similarly, the runoff from the area between the 20- and 30-minute lines will drain in 20 to 30 minutes. After 30 minutes, the entire area is draining. Therefore, the TOC at the outlet for this area is 30 minutes.

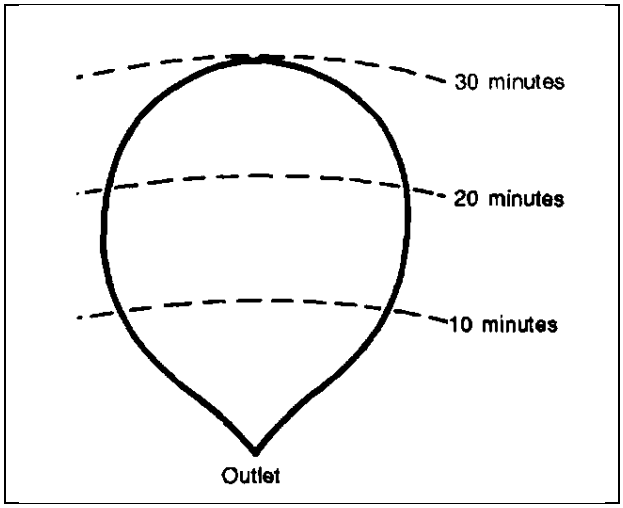


Figure 6-16. TOC, regular area

6-106. 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. 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 storm sweeps over the same area in a uniform fashion, the entire area contributes runoff to the outlet in the 30-minute time frame.

6-107. If a storm with a longer duration than the TOC occurs, the drainage designer can ascertain (using the standard intensity-duration curve) that the intensity will be less than the intensity of the 30-minute storm. Examination of the rational-method equation reveals that since the runoff coefficient and area would not change as intensity decreases, the quantity must decrease as well. The critical-storm duration must then be equal to the contributing area TOC.

Determine Area Time of Concentration

6-108. 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 representative because drainage areas are not as regular in shape as the area in figure 6-16. The path selected must be representative of the time most of the area will be contributing water to the outlet point. Establishing representative flow paths is based on experience and judgment (trial and error).

6-109. Unlike the area depicted in figure 6-16, the area in figure 6-17, 6-26, is irregular as most natural areas will be. In irregular areas, it is critical that the flow paths chosen represent the time required for most of the area to drain. 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 10 to 20 minutes (and so forth up to the 40-minute line). Flow lines a, b, and c are determined, and 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 30 to 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. Flow line b is not representative.

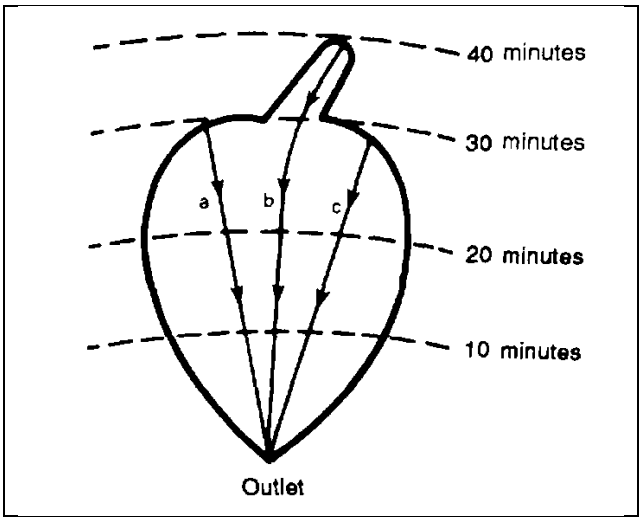


Figure 6-17. TOC, irregular area

6-110. Let the runoff coefficient equal 1.0 and assume that the 1-hour, two-year intensity is 2.0 inches per hour. If a 40-minute duration storm occurs, the entire area will be wet and contributing water to the outlet point in 40 minutes. The standard intensity-duration curve in figure 6-4, page 6-10, shows that the intensity for a 40-minute storm is 2.7 inches per hour; therefore, the estimated runoff is—

$$Q = CIA$$

$$Q = (1.0)(2.7\ inches\ per\ hour)(100\ acres) = 270\ cubic\ feet\ per\ second$$

where—

Q = *quantity of runoff (cubic feet per second)*
C = *runoff coefficient*
I = *rainfall intensity (inches per hour)*
A = *drainage area (acres)*

6-111. 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. The volume will be—

$$Q = (1.0)(3.2\ inches\ per\ hour)(90\ acres) = 288\ cubic\ feet\ per\ second$$

—which is larger than the 270 cubic feet per second estimated for the entire area.

6-112. The chosen flow paths represent the time required for most of the area to drain. A shorter storm of higher intensity may cause a larger flow. After 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—

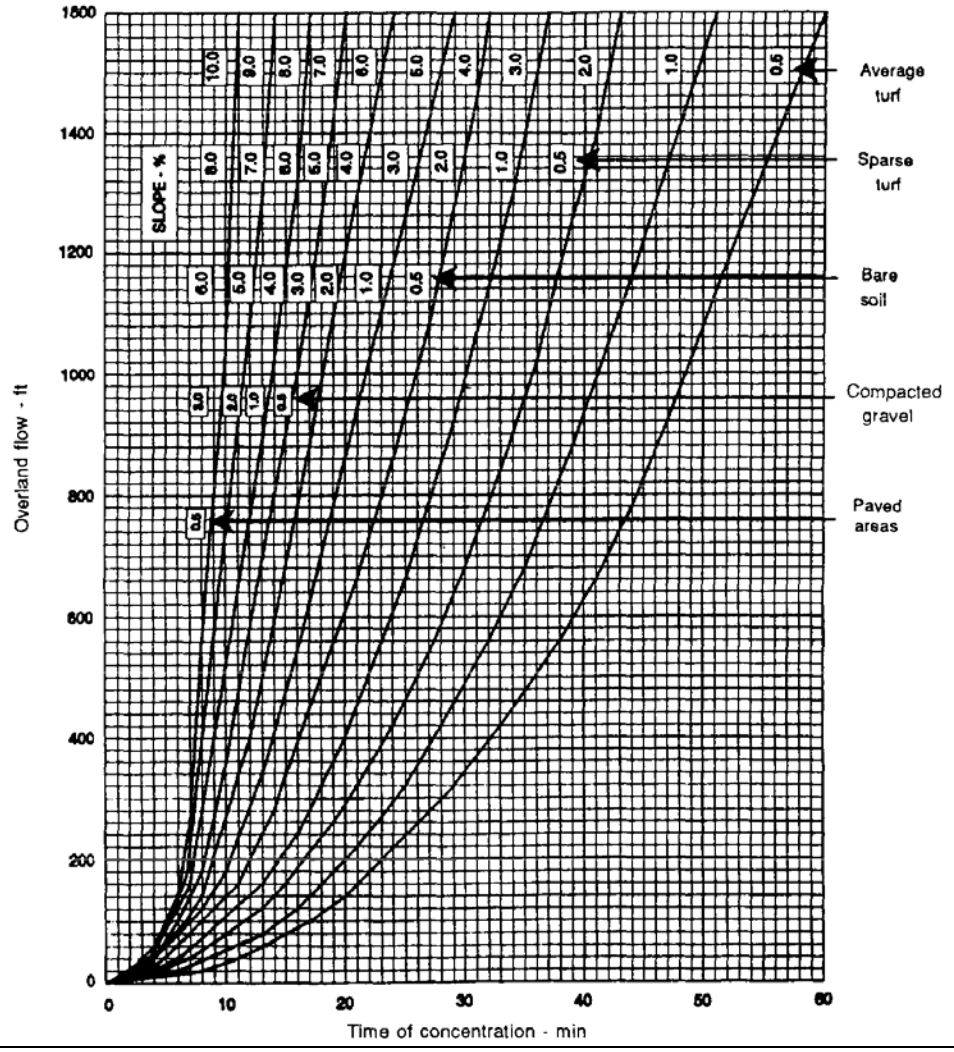
$$Q = (1.0)(3.2\ inches\ per\ hour)(100\ acres) = 320\ cubic\ feet\ per\ second$$

6-113. After the 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—the steeper the slope, the faster the water runs. Water travels faster over pavement than across a grassy area of the same slope because grass slows the water flow. Water travels slower over bare soil than over pavement, but water

travels faster over bare soil than over grass. Water flow in a ditch is more rapid than overland flow over turfed, bare, or compacted gravel surfaces.

Estimate Flow Time for Single Covers

6-114. After establishing the location, the cover, and the slope of a flow path, figure 6-18 is used to estimate the travel time along the flow path. It is important to understand what the illustration is depicting and how to use it properly. 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 increases until the flow becomes constant. This represents the fact that water initially moves slowly and begins to pick up speed as its accumulated depth increases. Initially, the water flows slowly 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 a steady-state velocity. It is apparent 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.



Note. It is valid to interpolate between percent slope curves.

Legend:

ft	feet
min	minute

Figure 6-18. Flow travel time

6-115. To estimate travel time in sheet-flow conditions using figure 6-18, page 6-27, select the appropriate cover type of the flow path (right side). Proceed horizontally to the left until reaching the curve labeled with the slope of the path. Follow the curve up or down to the intersection of the horizontal line that equals the flow-path length (left side). Read the travel time, which is found by drawing a line vertically from the intersection to the lowermost axis. (See table 6-3 for examples.)

Table 6-3. Examples of travel time estimates

Path	*Cover*	*Length (feet)*	*Slope (%)*	*Travel Time (minutes)*
1	Sparse turf	800	3.0	19
2	Paved area	500	1.5	8
3	Average turf	600	2.5	26

6-116. 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, use table 6-4. Using the slope of the flow path, locate the velocity in feet per minute in the column to the right. To calculate velocity, divide the length of the path by the velocity obtained from the chart.

Table 6-4. 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.
Legend:
fpm feet per minute
fps feet per second

Example

Estimate the flow velocity of path 1B (ditch cover). Given—
Length = 1,015 feet
Slope = 0.9%

$$\frac{\mathbf{1.015}}{\mathbf{135}\ \boldsymbol{feet\ per\ minute}} = \mathbf{7.5}\ \boldsymbol{minutes}$$

Estimate Flow Time for Multiple Covers

6-117. In many cases, a flow path traverses more than one cover type. Estimating travel time becomes more complicated because it is not appropriate to add times obtained from figure 6-18. Laminar flow occurs 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 6-4. For example, assume that path 1 and path 2 (see table

6-3) are the upper and lower lengths of one combined flow path. To estimate the combined travel time, perform the following:

- Since path 1 is uphill from path 2, nothing has changed. The travel time remains 19 minutes.
- Remember that the flow entering at the upstream end of path 2 is already moving. To estimate travel time, divide the length of the path by the estimated velocity listed in table 6-4.

  Given—

  Length = *500 feet*

  Slope = *1.5 percent*

$$Travel\ time = \frac{500\ feet}{165\ feet\ per\ minute} = 3\ minutes$$

Note. Each path is estimated separately in the order that the water would flow through them. Path 2 had an 8-minute travel time when considered alone.

- Add the partial travel times together for the total travel time.

$$Travel\ time = 19\ minutes\ +\ 3.0\ minutes = 22\ minutes$$

6-118. Sometimes it is helpful to estimate the travel time through a culvert. A reasonable assumption of culvert velocity is 5 feet per second (300 feet per minute) although more precise determinations are discussed later in the chapter.

Select Design Time of Concentration

6-119. 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 it takes for water to travel along each of the flow paths chosen. If the times are within a few minutes of each other, select the longest time as the area TOC. If the times are not within a few minutes of each other, analyze the area completely. 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.

RATIONAL METHOD APPLICATION

6-120. Application of the rational method of estimating drainage varies according to the type of drainage area. One type is a single, independent area that 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

6-121. The rational method of estimating single areas is simple and straightforward if it is done methodically. If followed step-by-step, the estimate will be as valid as the judgments that are made. The steps, listed in the proper order, are as follows:

- **Step 1.** Delineate the area to determine the area contributing runoff to the project location. (Refer to Delineating Watersheds.)
- **Step 2.** Delineate subareas by soil or cover type. (Refer to Step 6, Delineating Watersheds.)
- **Step 3.** Determine the acreage for each basin and subarea. (Refer to Determining Drainage System Size.)
- **Step 4.** Classify the drainage basin as simple or complex. A simple watershed has one cover or soil type over 80 percent of the total area. A complex watershed has no single cover or soil type covering at least 80 percent of the total area.
- **Step 5.** Determine representative flow paths. (Refer to Determining Time of Concentration.)
- **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, depending on factors such as soil

type, vegetation, and slope. It is best to visually investigate on-site for evidence of channeling and take measurements as needed. However, since in this example 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. Some clues may be the beginning of uphill swales or flow paths that converge into 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. (Refer to Determining Time of Concentration.)
- **Step 8.** Determine the average slope of the basin or each subarea based on flow paths. Define the slope as being less than or equal to 2 percent, greater than 2 percent and less than 7 percent, or equal to or greater than 7 percent.
- **Step 9.** Find the runoff coefficient variable of the basin or each subarea based on soil or cover type and slope from table 6-2, page 6-23. Select the runoff coefficient from the appropriate column in table 6-4, page 6-28, ensuring that the correct runoff coefficient variable column is selected. The columns are arranged with respect to slope and cover type. Calculate C_{wtd} if the area is complex. Use the formula—

$$C_{wtd} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + \cdots C_nA_n}{A_1 + A_2 + A_3 + \cdots A_n}$$

- **Step 10.** Determine the travel time for each flow path, and select the longest flow path as the basin TOC. (Refer to sheet flow and ditch flow in table 6-4.)
- **Step 11.** Find the intensity for a two-year, 60-minute storm on the world isohyetal map. (See figure 6-3, page 6-9.)
- **Step 12.** Adjust the intensity based on the TOC (Step 10), using figure 6-4, page 6-10.
- **Step 13.** Calculate the quantity using the runoff coefficient from Step 12 and the area from Step 3. Use the formula—

$$\mathrm{Q} = \mathrm{CIA}$$

where—

Q = *quantity of runoff (cubic feet per second)*
C = *runoff coefficient*
I = *rainfall intensity (inches per hour)*
A = *drainage area (acres)*

Example

Estimate the amount of runoff expected to arrive at the culvert shown in figure 6-19, page 6-33. The location is Giessen, Germany, and the design life is two years. A number of steps, quantities, and calculations will be given to illustrate the process. Given—

C = 0.35
I = 4.2 inches per hour
A = 48.8 acres

- **Step 1.** Delineate the area. (Given for this example.)
- **Step 2.** Delineate subareas by soil or cover type. (Given for this example.)
- **Step 3.** Determine the acreage for each basin or subarea. Given—

Average turf GM_d = *47.9 acres*
Compacted gravel = *.9 acres*
Total acreage = *48.8 acres*

- **Step 4.** Classify the drainage basin as simple or complex. Divide the largest soil or cover group with the turf subarea (47.9 acres) by the total area (48.8 acres) as follows:

$$\frac{47.9}{48.8} = 0.98 \; or \; 98 \; percent$$

Example (continued)

Since the percentage is greater than 80, this area will be treated as a simple area consisting of 48.8 acres of turfed GM_d soil.

- **Step 5.** Determine the representative flow paths. Determine the average slope of the entire simple area shown in figure 6-19, page 6-33. To do this, use representative flow paths. If the North-South running swale is 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 the number taken from the East. Figure 6-20, page 6-34, 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.

- **Step 6.** Divide the flow paths into two sections: sheet and ditch flow. Generally, overland flow concentrates into natural ditches after 500 feet. Path 1 and path 2 change to ditch flow in the valley where it slopes down. The contour lines near path 3 are relatively flat. The water flow occurs across a wide area, and no clear point can be seen where the flow changes. A good estimate would be 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, work would be performed on one consolidated map. For instance, to determine the slope for path 1A use the following formula:

$$Slope = \frac{96\ feet - 62\ feet}{575\ feet} = 0.059\ or\ 5.9\%$$

  This procedure is repeated for every flow path illustrated in figure 6-20. A tabular solution, shown in table 6-5, page 6-34, is recommended to determine the TOC. The original slope of path 3 is unchanged and remains at 3.6 percent. Path 2A and path 2B are now one path (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.) Revise the overall slope (as was done earlier). The earlier path 1B has been deleted, leaving path 1A, with a slope of 5.9 percent, as the new path 1. Path 1B is deleted because it provides the same information provided by the new path 2. Path 2B could have been deleted instead 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 runoff coefficient variable of the basin or subarea based on soil or cover type and the slope from table 6-2, page 6-23. Since we know the average slope is 3.93 percent, use table 6-2. Using the slope column, a runoff coefficient variable of 0.35 can be determined.
- **Step 10.** Determine the travel time of each flow path and select the longest flow path travel time as the basin TOC. Times for path 1B and path 2B were obtained by dividing their flow lengths by approximate velocities obtained from table 6-4, page 6-28. The travel times for each of the complete flow paths (22.0, 26.4, and 19.02 minutes) are obtained from figure 6-18, page 6-27. The variation between the smallest and largest time, although not small, is not excessively large. Perhaps path 1 is not representative and some ditch flow occurs along

Example (continued)

path 3 that could not be determined from the topographic information available. These possibilities are likely true, however; an actual field investigation is needed to justify revising the paths. Accept the travel times determined, and select the largest as the basin TOC. Thus, the TOC equals 26.4 minutes.

- **Step 11.** Determine the intensity value for a two-year, 60-minute storm. To determine the intensity factor, a source of rainfall data is necessary. The choice is between using pinpoint data (the most accurate means of determining the intensity) and 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 6-9. Knowing that the airfield is located near the demilitarized zone in Korea, rainfall intensity of the 1-hour, two-year storm is determined to be 2.5 inches per hour or—

$$I60, 2\ year = 2.5\ inches\ per\ hour$$

- **Step 12.** Adjust the intensity value. To determine the intensity, I60, two year must be adjusted so that its duration is equal to the basin TOC of 26.4 minutes. Use the set of standard intensity-duration curves in figure 6-4, page 6-10, 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 intersected, the intensity (adjusted to 26.4 minutes) is found to be 4.2 inches per hour, thus—

$$Iadj = I26.4\ minutes, 2\ year = 4.2\ inches\ per\ hour$$

- **Step 13.** Calculate quantity using the runoff coefficient from Step 9, Iadj from Step 12, and A from Step 3. The variables have been determined to solve the equation—

$$Q = CIA$$

$$Q = 0.35 \times 4.2\ inches\ per\ hour \times 48.8\ acres$$
$$Q = 71.74\ cubic\ feet\ per\ second\ or\ 71.7\ cubic\ feet\ per\ second$$

where—

Q	=	*quantity of runoff (cubic feet per second)*
C	=	*runoff coefficient*
I	=	*rainfall intensity (inches per hour)*
A	=	*drainage area (acres)*

The determination of quantity is the final solution to the example. If area 5 had been a complex area, Steps 1, 2, 3, and 5 would be unchanged. Step 4 would change as follows:

- Determine S_{avg} for each soil or cover area (except for manufactured covers).
- Determine a runoff coefficient variable for each soil or cover based on the average slope for each cover area.
- Determine an area-weighted runoff coefficient or C_{wtd} once the runoff coefficient or C_{wtd} corrections are made.
- Solve C_{wtd} using Q = CIA.

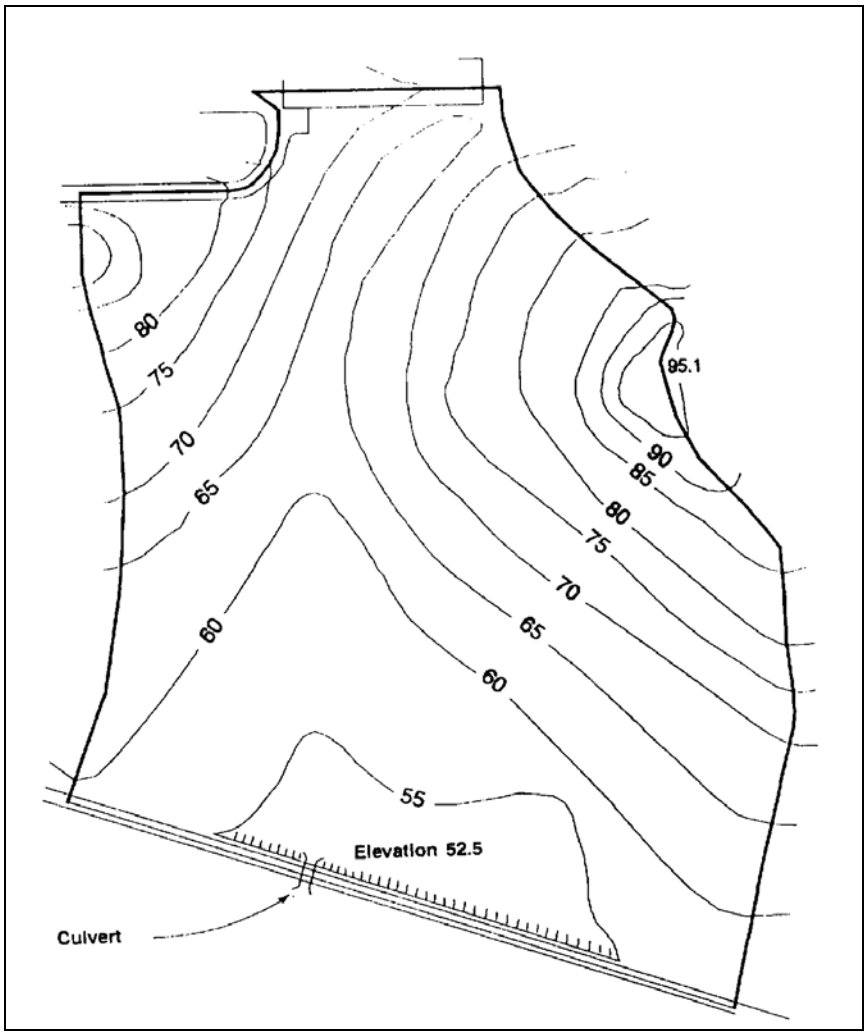
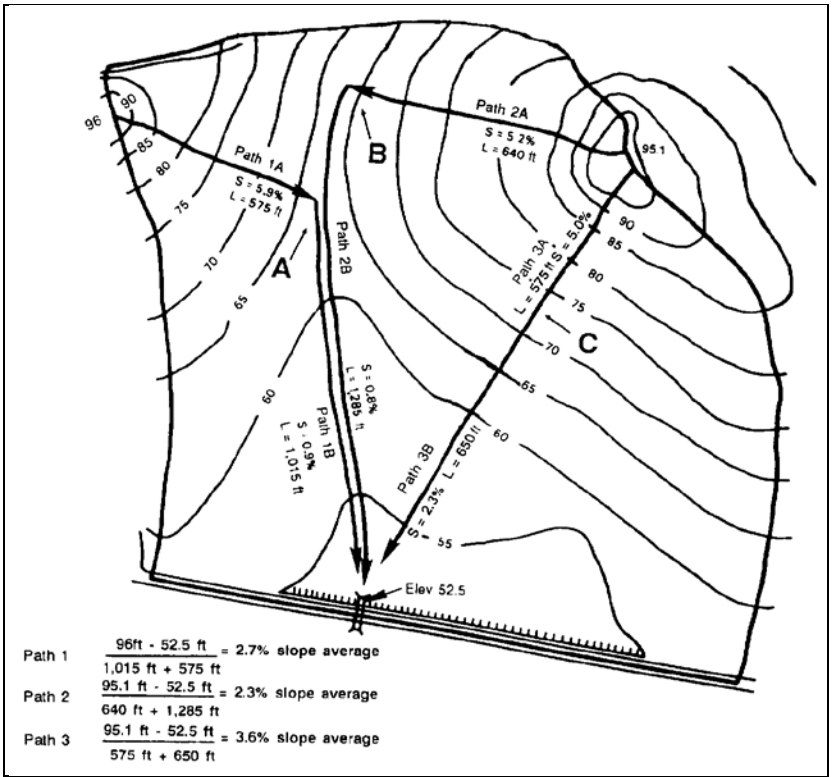


Figure 6-19. Delineating runoff area



Legend:

elev	elevation
ft	feet
L	length
S	slope

Figure 6-20. Determining average slope

Table 6-5. Tabular solution to determine TOC

Path	*Cover*	*Length (feet)*	*Slope (percent)*	*Time (minutes)*
1A	Average turf	575	5.9	14.5
1B	Ditch section	1,015	0.9 (135 fpm)	7.5
Total:				22.0
2A	Average turf	640	5.2	16.5
2B	Ditch section	1,285	0.8 (130 fpm)	9.9
Total:				26.4
3A	Average turf	575	5.0	16.0
3B	Ditch section	650	2.3 (215 fpm)	3.02
Total:				19.02

Legend:
fpm feet per minute

Successive Areas

6-122. Up to this point, the drainage areas discussed have been single, independent areas (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.

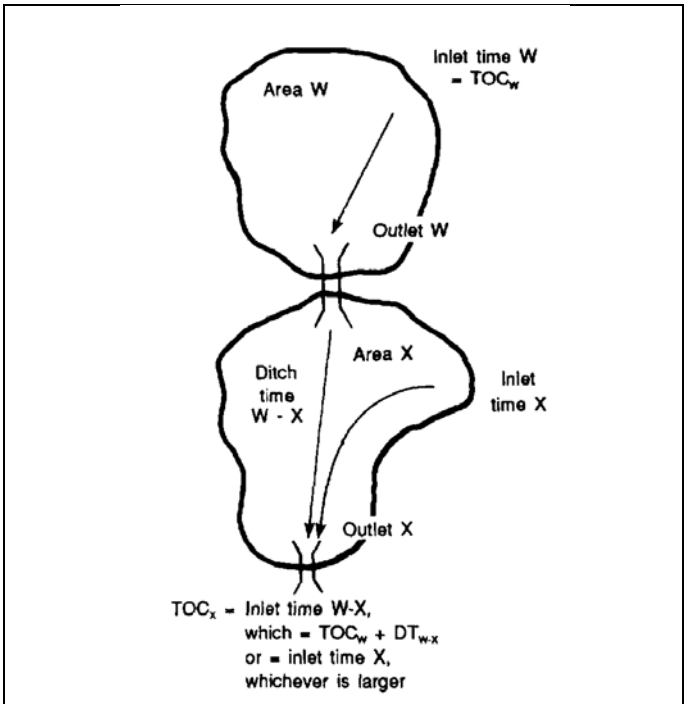
6-123. Sometimes, two or more areas discharge runoff into the same dependent downstream area. These contributing areas are called parallel areas.

6-124. Unfortunately, the increase in runoff is not the simple summation of the peak runoff for each individual area. Individual peak flows are acted upon by various factors, including storage and peak-flow 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.

Time of Concentration Calculation

6-125. Because peak flow accumulates in successive areas, calculating the TOC for those areas is 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 decreases.

6-126. The term TOC must be modified to reflect calculation differences when considering two areas (such as an upstream area and a downstream area). (See figure 6-21.) The maximum travel time from the most hydraulically remote representative points to the outlet area is defined as TOC. The maximum representative flow time for runoff originating from an area to arrive at the outlet is defined as inlet time.



Legend:	
DT	ditch time
TOC	time of concentration

Figure 6-21. Schematic example for successive areas

6-127. The TOC at the outlet upstream in area W is given as TOC_W, which equals inlet time W for this independent area. The ditch time 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 $time_{W-X}$. To a designer engaged in sizing the culvert that serves as outlet W, the TOC would simply be—

$$TOC_W = inlet\ time_W$$

6-128. 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 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.

6-129. The travel time for water originating in area W and arriving at outlet X is equal to the inlet time at area W (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 representative time for water to arrive at outlet X, is the larger time value identified in the comparison.

Successive Area Runoff Estimation

6-130. The modified definition of TOC is applied in estimating runoff for successive areas. This calculation requires collection of certain essential data and application of the principles used for single areas, appropriately modified, to determine the desired quantity. The required steps are as follows:

- **Step 1.** Delineate every subarea in the series of areas.
- **Step 2.** Determine the intensity of the two-year, 60-minute rainfall.
- **Step 3.** Determine the area, runoff coefficient, and inlet time. For each subarea, determine the area, runoff coefficient variable (corrected for slope and weighted as necessary), and inlet time.
- **Step 4.** Determine the ditch time (or transit time).

6-131. For each dependent subarea, the ditch time is determined from the upper outlet to the lower outlet using culvert flow (assume 300 feet per minute in the absence of more precise data).

6-132. To determine the specific quantity, start at the uppermost subarea and proceed downstream, working systematically. Subarea W is upstream from subarea X (refer to figure 6-21, page 6-35). Use the following steps:

- **Step 1.** Determine the subarea TOC. Use the following equations for—
  - An independent subarea—
    $$TOC_W = inlet\ time_W$$
  - A dependent area—
    $$TOC_W = ditch\ time_{W-X}$$
    Compare inlet $time_X$ and inlet $time_{W-X}$, and select the larger value of TOC_X.
- **Step 2.** Adjust the intensity 60-minute, two-year to intensity TOC, two year.
- **Step 3.** Calculate the quantity for subarea W. Use the following equation:
  $$Q_W = C_W I TOC_X A_W$$
- **Step 4.** Proceed downstream to subarea X and repeat Steps 1 through 3. Use the following equation:
  $$Q_W = C_X I TOC_X A_W$$
- **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 corresponding rainfall intensity for the intensity value. Use the total area for basins and the subareas for the area value. Use the rational-method formula—
  $$Q = CIA$$

where—

Q	=	*quantity of runoff (cubic feet per second)*
C	=	*runoff coefficient*
I	=	*rainfall intensity (inches per hour)*
A	=	*drainage area (acres)*

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, determine the runoff coefficient (in cubic feet per second) to expect at culvert 3 and culvert 4 at the Span II Army Airfield at Giessen, Germany. Use figure 6-22, page 6-39, and the information in table 6-6, page 3-39. The soil type is GMd. Use the following steps:

- **Step 1.** Delineate the area. Given that subarea A is a simple area with compacted gravel = 6.2 acres and average turf = 0.2 acres; subarea B is a simple area with compacted gravel = 0.9 acres and average turf = 12.5 acres.
- **Step 2.** Delineate the subareas by soil or cover type. (See Step 1.)
- **Step 3.** Determine the acreage for each basin or subarea. (See Step 1.)
- **Step 4.** Classify the basin as simple or complex. Use the following equations:

$$Subarea\ A = 6.2\ acres/6.4\ acres = 0.97\ or\ 97\ percent$$
$$Subarea\ B = 12.5\ acres/13.4\ acres = 0.93\ or\ 93\ percent$$

- **Step 5.** Determine representative flow paths (refer to table 6-6).
- **Step 6.** Divide the flow paths into two sections: sheet flow and channel flow. (See table 6-6.)
- **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 figure 6-22.) Using this information, we can now get an average slope for paths 2, 3, and 4—

$$Average\ slope = \frac{2.4\% + 2.3\% + 2.6\%}{3} = 2.4\%$$
$$Average\ slope\ for\ subarea\ 1 =< 2\%$$
$$Average\ slope\ for\ subarea\ 2 = 2.4\%$$

- **Step 9.** Find the runoff coefficient values for each subarea. Subarea A has compacted gravel with an average slope of less than 2 percent. Subarea B has average turf with a slope of 2.4 percent. According to table 6-2, page 6-23, the runoff coefficient for subarea A is 0.70, and the runoff coefficient for subarea B is 0.35.

Example (continued)

- **Step 10.** Determine 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 6-27, and ditch-flow travel time from table 6-4, page 6-28. Determine travel times and ditch time from table 6-6. Use the following equations:

$$Ditch\ time_{3-4} = \frac{370\ feet}{188\ feet\ per\ minute} + \frac{350\ feet}{194\ feet\ per\ minute} = 3.8\ minutes$$

$$Ditch\ velocity = 188\ feet\ per\ minute\ at\ 1.8\%\ and\ 194\ feet\ per\ minute\ at\ 1.9\%$$
$$Inlet\ time\ A = 16.3\ minutes$$
$$Inlet\ time\ B = 25.3\ minutes$$

- **Step 11.** Determine intensity and intensity adjustment. The intensity value for Giessen, Germany, is 1.7 inches per hour from local rainfall records.

  Subarea A—

$$TOC_A = 16.3\ minutes$$
$$I16.3 = 2.6\ inches\ per\ hour$$

  Subarea B. Compare inlet time B with—

$$TOC_A + ditch\ time = 16.3 + 3.8 = 20.1\ minutes$$

  Select the larger value of—

$$TOC_B = 25.3\ minutes\ and\ I25.3 = 1.8\ inches\ per\ hour$$

- **Step 12.** Adjust the intensity based on TOC using figure 6-4, page 6-10 (this step was included in Step 11).
- **Step 13.** Determine the runoff coefficient.
  To calculate runoff in subarea A, use—

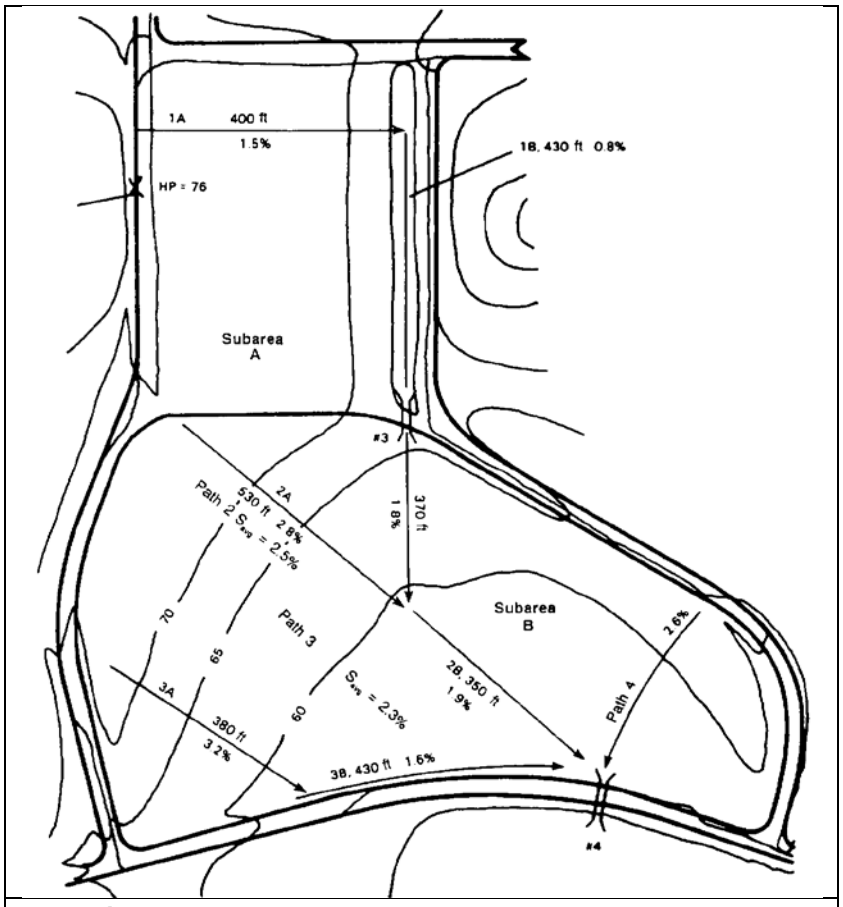
$$\begin{aligned} Q_A &= C_A I_A A_A \\ &= 0.70\ (2.6\ inches\ per\ hour)(6.4\ acres) \\ &= 11.6\ cubic\ feet\ per\ second \end{aligned}$$

  To calculate runoff in subarea B, use—

$$\begin{aligned} Q_B &= C_B I_B A_A \\ &= 0.35\ (1.8\ inches\ per\ hour)(13.4\ acres) \\ &= 8.4\ cubic\ feet\ per\ second \end{aligned}$$
$$Q_{inlet\ 3} = 11.6\ cubic\ feet\ per\ second$$
$$Q_{inlet\ 4} = 11.6 + 8.4 = 20\ cubic\ feet\ per\ second$$

where—

Q	=	*quantity of runoff (cubic feet per second)*
C	=	*runoff coefficient*
I	=	*rainfall intensity (inches per hour)*
A	=	*drainage area (acres)*



Legend:

avg	average
ft	feet
HP	high point
S	slope

Figure 6-22. Example of successive areas

Table 6-6. 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

Legend:

ft	feet
fpm	feet per minute
min	minutes

DESIGN COMPUTER PROGRAMS

6-133. Many private and public domain software products are available for the analysis and design of various components of storm drain systems. These products range from simple computational tools for specific components of the storm drain system to complex programs that can analyze complete storm drain systems using interactive graphical interfaces. The computer hardware and software industry is rapidly changing. New and more advanced applications software is developed each year. For public sector software, user support is minimal or nonexistent if the software is obtained directly from the government. Private vendors sell many of these packages and may offer user support. The following three programs may be useful in the analysis and design of drainage systems for urban areas. While these programs represent a start for most projects, many programs are available that may better suit a specific project.

Technical Release-55 Program

6-134. Technical Release (TR)-55 is a hydrology program that implements Natural Resources Conservation Service methods for calculating TOC, peak flows, hydrographs, and detention basin storage volumes. It is useful for urban drainage situations where detailed hydrograph routing procedures are not warranted. The program, now compatible with Windows™ operating systems, incorporates the procedures outlined in TR-55. TR-55 contains simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for storm water reservoirs. The procedures are applicable in small urbanizing watersheds in the United States.

6-135. TR-55 is extremely easy to use with interactive menus that prompt the user for specific inputs. Several screens of input are normally required before an analysis can proceed. Help screens assist the user in successfully performing an analysis.

Technical Release-20 Program

6-136. TR-20 is a comprehensive hydrology program that implements Natural Resources Conservation Service methods for generating and routing runoff hydrographs in a multibasin watershed. The program provides hydrographic analyses of a watershed under present conditions and various combinations of land cover or use and structural or channel modifications using single rainfall events. Output consists of runoff peaks, flood hydrographs, time of occurrence, and water surface elevations at a cross section or structure. Subarea surface runoff hydrographs are developed from storm rainfall using a dimensionless unit hydrograph, drainage areas, times of concentration, and runoff curve numbers. Hydrographs can be developed, routed, added, stored, diverted, or divided to convey floodwater from the headwaters to the watershed outlet. TR-20 is useful for large watersheds where detailed hydrograph routing is warranted.

Hydrologic Modeling System

6-137. The Hydrologic Engineering Center Hydrologic Modeling System is a flood hydrograph package developed by USACE. The Hydrologic Modeling System model, like TR-20, is designed to simulate the surface runoff response of a river basin to precipitation by representing the basin as an interconnected system of hydrologic and hydraulic components. Each component models an aspect of the precipitation-runoff process within a portion of the basin. A component may represent a surface runoff entity, a stream channel, or a reservoir. Representation of a component requires a set of parameters that specify the particular characteristics of the component and mathematical relations that describe the physical processes. The result of the modeling process is the computation of stream flow hydrographs at desired locations in the river basin. It is useful for large watersheds where detailed hydrograph routing is warranted.

6-138. Hydrologic Modeling System simulates a river basin as a group of subareas interconnected through channel routing reaches and confluences, and the program performs hydrologic calculations on a user-specified time step for a single storm (soil moisture recovery during dry spells is not included). Hydrologic Modeling System is used to generate discharge, not water surface elevations (although it does calculate normal depth). The Hydrologic Engineering Center River Analysis System model is typically used in conjunction with Hydrologic Modeling System to determine water surface profiles through detailed hydraulic computations.

SOFTWARE AVAILABILITY

6-139. Contact information for obtaining the software summarized in this chapter is as follows:

- **TR-55 and TR-20.** Natural Resources Conservation Service, National Water and Climate Center, 1201 Lloyd Boulevard, Suite 802, Portland OR 97232-1274, telephone (503) 414-3031.
- **Hydrologic Modeling System.** USACE Hydrologic Engineering Center, 609 Second Street, Davis, CA 95616, telephone (530) 756-1104.

This page intentionally left blank.

Chapter 7

Open-Channel Design

An open channel is a conduit with a free-water surface used to transport water. The ditch, an open channel cut into the soil, is the most common. The ditch can be lined along the bottom from bank to bank.

FACTORS

7-1. The size, shape, method of construction, and location of a ditch are determined by its purpose. These factors, once determined, will influence the design capacity and maintenance requirements.

LOCATION

7-2. The main types of ditches used in road and airfield construction are—

- **Interceptor.** 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 side hill cuts to prevent erosion of the cut.
- **Side ditch.** 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.
- **Diversion.** 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

7-3. 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 7-1, page 7-2.

7-4. In the JOA, the shape of a ditch is dictated, to a great extent, by the choice of engineer equipment available for construction. Two items of equipment are uniquely suited for speedy ditch excavation: the motor grader and the wheeled tractor-scraper. Other equipment that can be used to excavate a ditch section include the backhoe; bulldozer; front-end loader; trenching machine; and a crane equipped with a dragline, clamshell, or shovel front. However, production rates for these items are low compared to the grader or scraper so they are more likely to be used.

7-5. 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.

7-6. For flows larger than 60 cubic feet per second, the trapezoidal ditch is commonly specified. A wheeled scraper can excavate the flat bottom and midsection of this ditch rapidly, and the side slopes can be dressed back by subsequent passes of a road grader. Smaller bottom widths can be provided using the previously mentioned equipment, including the road grader with its blade turned to a high angle. Production rates will be much lower for the grader than those of the wheeled scraper.

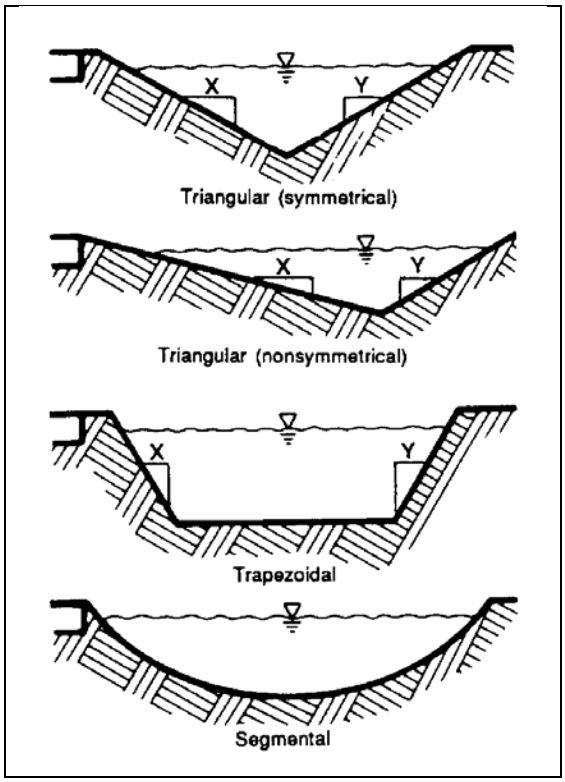


Figure 7-1. Ditch cross sections

7-7. The 60 cubic feet per second guideline is flexible. If a scraper is not available to excavate a ditch carrying 100 cubic feet per second, a grader is used to construct an oversized V ditch rather than using low-production-rate equipment to construct a trapezoidal ditch.

7-8. The segmental-ditch shape results when explosives are used to create the ditch. This technique is often used when the terrain is too soft to support excavating machinery. Ditches cut by hand will often bear this shape as well.

SIDE-SLOPE RATIOS

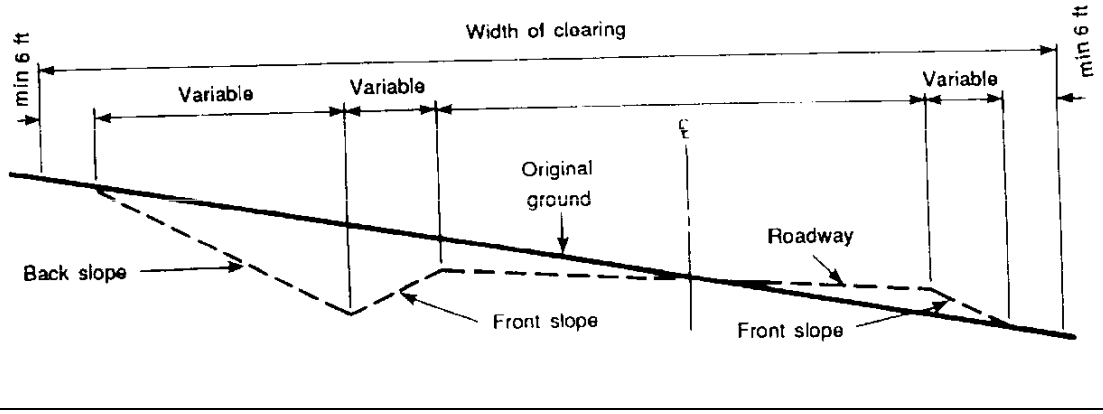
7-9. Ditches have two sides and two associated side-slope ratios. Side slope is the slope of the channel bank, normally expressed as a ratio of feet horizontal to feet vertical. For example, a side-slope ratio of 3:1 is 3 feet horizontal to 1 foot vertical. When the sidewalls on opposite sides are inclined equally, the ditch is called symmetrical. Nonsymmetrical ditches have two different side slopes.

7-10. 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. If the side slope is too severe, a serious accident could occur if a vehicle runs into the ditch and becomes entrapped or overturns. 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. (See figure 7-2 for an illustration.)

7-11. 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 is applicable only in shallow ditches in relatively flat terrain:

- Roadside ditches may be cut nonsymmetrical at 3:1 (front slope) and 1:1 (back slope).
- Diversion ditches may be cut symmetrically at 1:1.
- Ditches subject to cross traffic may be cut symmetrically at 3:1 or more gently.

Note. For calculation purposes, the horizontal component of the roadside ditch will be referred to as X, and the back slope will be referred to as Y. (See figure 7-1.)



Legend:

℄	centerline
ft	feet
min	minimum

Figure 7-2. Definition sketch

7-12. In most cases, interceptor and diversion ditches are located away from roadways and runways and do not present a hazard to passing vehicles or aircraft. Since there is little danger from either side, symmetrical side-slope ratios are specified for these types of ditches. Table 7-1 and table 7-2, page 7-4, are useful in selecting front slopes for fill sections.

Table 7-1. 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	1 1/2:1	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.

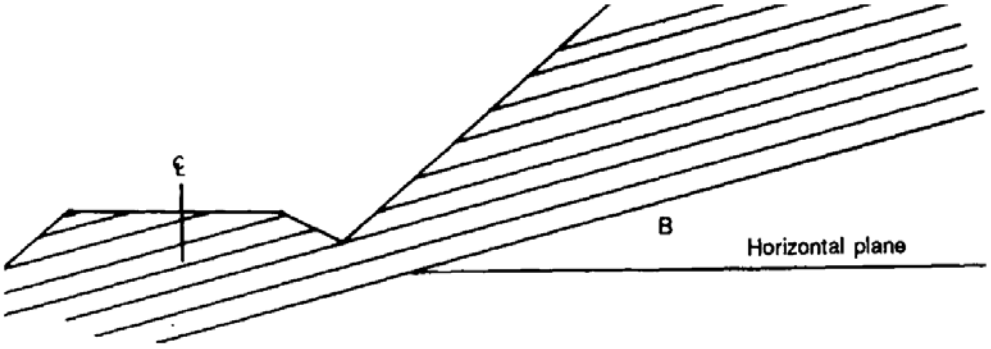
1. Recommended slopes are valid only in homogeneous soils that have an in-place or compacted density equaling or exceeding 95% CE 55 maximum dry density. For homogeneous soils or soils at lower densities, a deliberate slope stability analysis is required.
2. Back slopes cut into loose soil will seek to maintain a near-vertical cleavage. Do not apply loading above this cut face. Expect sloughing to occur.

Legend:

CE	compactive effort
USCS	Unified Soil Classification System

Table 7-2. 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:

$$\frac{angle\ between\ centerline\ and\ strike}{90°} \times true\ dip$$

Legend:

℄	centerline
H	horizontal distance
V	vertical rise

FLOW TYPES

7-13. Several flow types are associated with open channels. Some of these types occur simultaneously in the same channel. An understanding of these flow types and their interrelationship is essential to the effective design of drainage systems.

7-14. 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.

7-15. A *steady flow* is assumed in an open channel with a uniform depth during the design period. Changes in flow are generally slow, and errors that may be introduced by this assumption are not significant.

7-16. A *continuous flow* is assumed according to the principle of the conservation of mass.

7-17. 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.

7-18. The strength of viscosity forces and the thickness of moving fluid determine if channel flow is *turbulent* or *laminar*.

7-19. *Turbulent flow* is assumed for purposes of open-channel design. This flow type occurs when viscosity forces are relatively weak and the individual water particles move in random patterns within the aggregate forward-flow pattern.

7-20. *Laminar flow* occurs when viscosity forces are predominate and fluid particles 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 in natural and designed channels will be steady, continuous, uniform, and turbulent.

EQUATIONS

7-21. This section discusses 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

7-22. The equation of continuity is expressed as follows:

$$Q = AV$$

where—

A = *cross-sectional area in square feet*
Q = *rate of flow in cubic feet per second*
V = *velocity in feet per second*

MANNING'S VELOCITY OF FLOW EQUATION

7-23. Many empirical equations have been proposed for determining turbulent flow. The most widely used is the equation presented by Manning in 1889. It is as follows—

$$V = \frac{1.486\, R^{2/3}\, S^{1/2}}{n}$$

or, after transposing—

$$R = \left[\frac{V\, n}{1.486\, \sqrt{S}}\right]^{3/2}$$

or—

$$R = \frac{cross\ section\ area\ of\ water}{wetted\ perimeter}$$

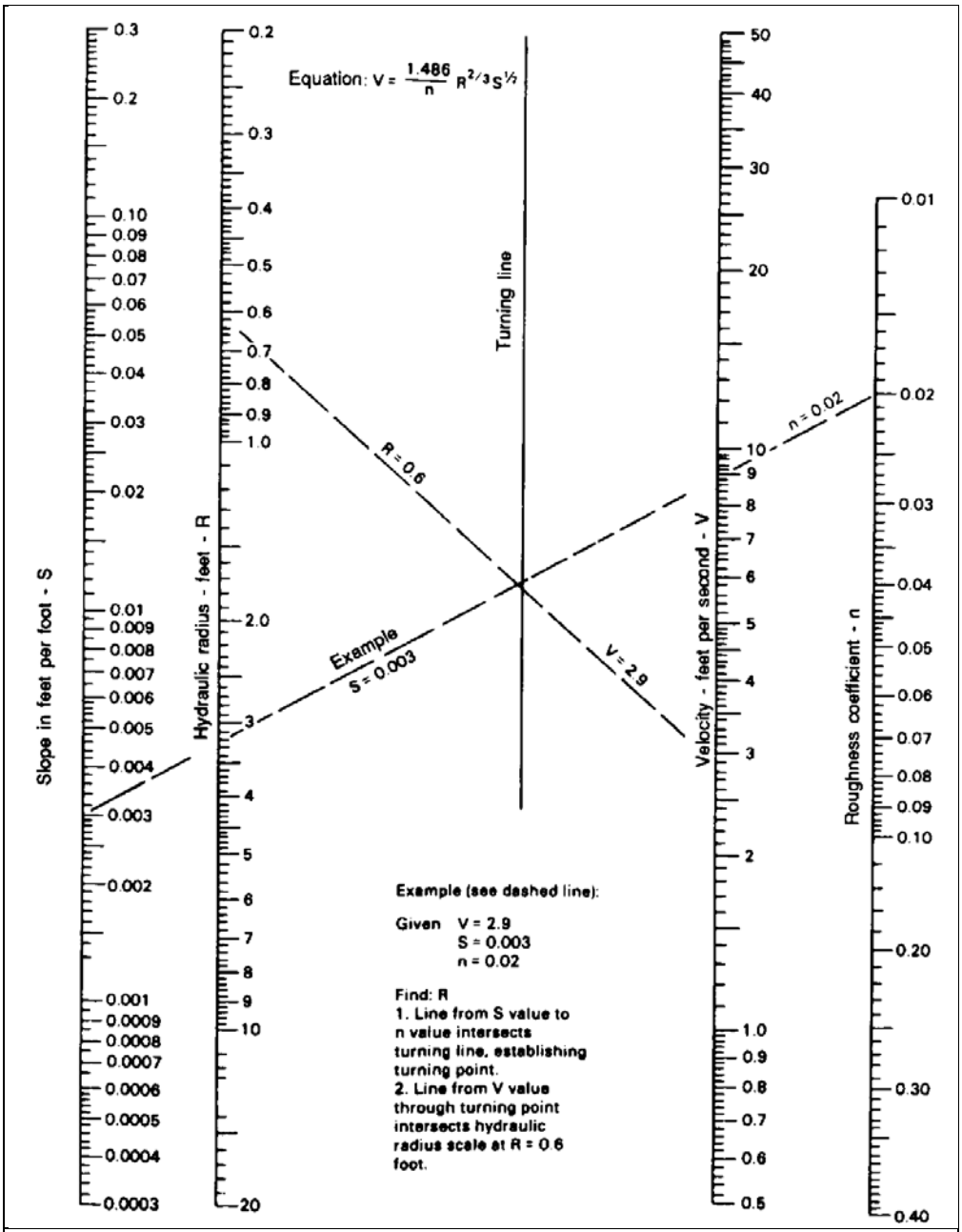
When using metric units—

$$V = \frac{R^{2/3}\, S^{1/2}}{n}$$

where—

n = *roughness coefficient or friction factor (depends on the channel lining material)*
R = *hydraulic radius*
S = *sloped or grade of the channel in feet per foot*
V = *velocity of flow in feet per second*

7-24. 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 7-3, page 7-6.)



Legend:

n	roughness coefficient
R	hydraulic radius
S	slope
V	velocity of flow

Figure 7-3. Nomograph for Manning's equation

VELOCITY OF FLOW

7-25. 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, the velocity of flow will be different. Since excessive velocity in a ditch will cause erosion and possible damage to adjacent structures, it must be contained within limits. Table 7-3 lists the maximum permissible velocities, depending on soil and other factors.

Table 7-3. Manning's *n* values 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 with 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, with 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.		
Legend:		
fps	feet per second	
N	roughness coefficient	
V	vertical rise	

7-26. The following applies to velocity relationships:

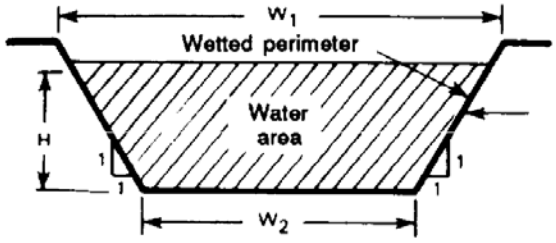
- As a slope increases, velocity increases.
- If the quantity of runoff increases and the area remains constant, velocity increases.
- If Manning's roughness coefficient increases, velocity decreases.
- If velocity increases, erosion increases.

HYDRAULIC RADIUS

7-27. The hydraulic radius is the area of the ditch water cross section divided by its wetted perimeter. (See figure 7-4, page 7-8.) 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. (See figure 7-5, page 7-8.)

CONSIDERATIONS

7-28. There are certain factors known for each ditch being designed. These factors affect the design details. The location, peak flow, effect of the terrain on the slope, and soil type or materials used to line the ditch have a bearing on the channel design.



Example

Assume H = 2 ft, W_1 = 14 ft, W_2 = 10 ft
Find hydraulic radius (R)
A = ½(H) (W_1 + W_2) = ½(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
$\frac{\text{Hydraulic radius area}}{\text{Wetted perimeter}} = \frac{24}{15\ 7} = 1\ 53$ ft

Legend:

A	area
ft	feet
H	height
R	hydraulic radius
sq	square
W	width

Figure 7-4. Hydraulic radius

1. If the calculator has the function $\sqrt{x}$ and y^2, determine R more quickly and accurately than with the nomograph provided in the workbook. The following shows Manning's equation transposed:

$$V = \frac{1.486\ R^{2/3}\ S^{1/2}}{n} \rightarrow R = \left(\frac{V \times n}{1.486\sqrt{S}}\right)^{1.5}$$

Since n and S are constant for various trial iterations, 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—

$$R = [V \times K]^{1.5}\ where\ K = \frac{n}{1.486\ \sqrt{S}}$$

2. To calculate R, use the above equation for numerous iterations until the *bracketed* flow rate, Q, is within acceptable limits. The keystrokes below will work on most calculators.

a. Input and store values for K.

.xxx	Enter the 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. 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 an acceptable R and Q is reached.

Legend:

K	constant	Q	quantity of runoff
M+	another way to store a number on a calculator	R	hydraulic radius
M	memory	RCL	recall button on a calculator
MRC	memory recall on a calculator	S	slope
n	roughness coefficient	STO	the store button on a calculator
		V	vertical rise

Figure 7-5. Calculating hydraulic radius

LOCATION

7-29. The ditch location determines the general shape of the ditch and the side slopes to be used in its design.

QUANTITY OF RUNOFF

7-30. Designers must know the quantity of runoff that the channel will have to carry. Usually, this is estimated using the rational method for 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 quantity of runoff also determines the type of ditch section (triangular or trapezoidal) to be used. This depends on if the quantity of runoff is greater than or less than 60 cubic feet per second.

SLOPE

7-31. The terrain will determine the slope. In general, the slope used will be the natural ground slope. Small modifications of the slope can be made for short ditch sections.

DITCH LINING

7-32. The ditch lining determines the velocity and roughness coefficient or resistance factor. Table 7-3, page 7-7, gives maximum erosion velocities for each type of soil and lining. The lower velocity on the chart indicates that the velocity erosion will start to occur in some portion of the ditch. At the high velocity value, the entire length of the ditch will probably be eroding.

7-33. Table 7-3 also provides Manning's roughness coefficient, which represents the friction resistance of the ditch, channel, or stream for various soil types and linings. Use the average roughness coefficient value for design purposes.

TECHNIQUES

7-34. Once design considerations have been examined, the interactive design procedure can begin.

STEPS

7-35. The steps used in design are—

- **Step 1.** Determine the peak volume of storm water runoff. Using the rational method, calculate the area contributing flow to the ditch. Find the quantity of runoff using the appropriate formula.
- **Step 2.** Determine the slope in feet per foot. If the slope is already known as a percentage, it may be converted to units of feet per foot by dividing by 100.
- **Step 3.** Select trial values for the roughness coefficient and velocity. From table 7-3, select the roughness coefficient and velocity values for the soil type of the ditch. The initial trial velocity should be held to 1 foot 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, the water will remove this deposited material during peak flow without causing extensive damage to the channel.
- **Step 4.** Determine the hydraulic radius. Use the nomograph or equation to find the hydraulic radius of the ditch using the slope, Manning's roughness coefficient, and velocity. Call this R_M to distinguish it from the radius value, R_t, used in appendix B.
- **Step 5.** Determine the ditch cross section type. For quantity of runoff greater than 60 cubic feet per second, use a trapezoidal ditch. For quantity of runoff equal to or less than 60 cubic feet per second, use a triangular ditch. The location determines if the ditch is symmetrical or nonsymmetrical.
- **Step 6.** Select the appropriate hydraulic radius for the desired ditch cross section. (See appendix B, tables B-1 through B-9, pages B-9 through B-17.) Select table by identifying the table title with the appropriate tentative side-slope ratios. Locate the value of R_t (hydraulic radius found using the tables in appendix B) that corresponds with R_m (hydraulic radius found using Manning's equation), and find the cross-sectional area and ditch depth corresponding to R_m and R_t. If the exact value of R_m is not available in the table, use the next smaller R_t value listed in that column.

- **Step 7.** Calculate the quantity of runoff. Use the following equation:

$$Q = A_W V$$

where—

A_W = *area*
Q = *quantity of runoff*
V = *velocity*

The area and velocity are determined in Steps 6 and 3. If the calculated quantity is—

- No more than 5 percent greater than the design quantity, the ditch selected can be used.
- More than 5 percent greater than the design quantity, reduce the velocity and repeat Steps 4 through 7.
- Smaller than the design quantity by more than 5 percent, increase the velocity. However, do not make it any larger than the maximum velocity permissible for the soil or lining. (See table 7-3, page 7-7.)
- Less than the 95-percent limit of the design quantity, 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 (the additional ditch depth over what is required to carry the design flow). Add 0.5 foot to the water depth for freeboard. This added depth enables 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 set ditch grade depth.

Example

Design a ditch to carry a peak volume of storm water runoff equaling 47.3 cubic feet per second 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 bare (unturfed) GM_d soil, as excavated. (See table 6-6, page 6-39, and table 7-3, page 7-7). Use the equation—

$$Q = 47.3\ cubic\ feet\ per\ second$$

$$Slope = \frac{invert\ at\ culvert\ or\ 7.0\ feet - 5.5\ feet}{distance\ from\ culvert\ to\ stream} = \frac{1.5\ feet}{289\ feet} = 0.0052\ feet\ per\ foot$$

$$Soil\ Velocity = 3\ to\ 5\ feet\ per\ second = 0.024$$

where—
Q = quantity of runoff

(Nomograph and table method) The solution of a ditch problem is a trial technique in which several values of velocity are used. Use a table to tabulate the results. (See table 7-4, page 7-14.) Follow these steps:

- **Step 1.** Select a ditch. Since design flow is less than 60 cubic feet per second, the ditch should be triangular. The channel is not a roadside ditch; 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 ditch selection column 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 erode. It is preferable not to exceed the velocity maximum of 5 feet per second, and the best choice is usually 1 feet per second lower than the velocity maximum, which is 4 feet per second. Enter this figure on the work sheet.

Example (continued)

- **Step 3.** Determine the hydraulic radius from the nomograph. Locate the turning line using the nomograph. (See figure 7-3, page 7-6.) The turning line is in the center on the nomograph. Find the turning point by locating the slope in the left column of the chart and the roughness coefficient 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 the same as long as the slope and roughness coefficient do not change. The hydraulic radius 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 hydraulic radius scale. Read the hydraulic radius in the second column from the left. This gives the required hydraulic radius for any given velocity in Manning's equation. Given—

  R = 0.86

  where—
  R = hydraulic radius

  Enter this value on the work sheet. The hydraulic radius value may also be computed by using the calculator method shown in Step 4 of the alternate solution on page 7-13.

- **Step 4.** Find the hydraulic radius in the ditch table using the appropriate table in appendix B. There are four types of tables: V-Triangular (Symmetrical), V-Triangular (Nonsymmetrical), Trapezoidal (Symmetrical), and Trapezoidal (nonsymmetrical). For this example, use table B-1, page B-9, for a symmetrical V ditch. Locate the 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, $R_m = 0.86$ cannot be found, but the values for 0.85 and 0.90 are given. Enter 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 B-1 is 9.72 square feet, and the depth 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.
- **Step 6.** Check for Q. Check to see if this particular ditch will meet the requirements of the design by performing these calculations—

$$\begin{aligned} Q &= A_W V \\ &= (9.72\ square\ feet)(4.0\ feet\ per\ second) \\ &= 38.9\ cubic\ feet\ per\ second \end{aligned}$$

  This calculated quantity of flow must fall within 5 percent of the design flow of 47.3 cubic feet 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 quantity of air flow 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. Perform these calculations, given—

  V = 4.5
  R_m = 1.0
  R_t = 0.95

$$A_W = 12.0\ square\ feet;\ d = 2.0\ feet$$
$$Q = (4.5\ feet\ per\ second)(12.0\ square\ feet)$$
$$Q = 54.0\ cubic\ feet\ per\ second$$

Example (continued)

—which exceeds 49.7 cubic feet per second so the next trial velocity must be less than 4.5 feet per second. Try 4.2 feet per second. Given—

V = 4.2
R_M = 0.92
R_T = 0.90
A_W = 10.83 square feet
d = 1.9 feet

$$Q = (4.2\ feet\ per\ second)(10.83\ square\ feet)$$
$$Q = 45.5\ cubic\ feet\ per\ second$$

where—

A_W = area
d = depth
Q = quantity of runoff
R_M = hydraulic radius found using Manning's equation
R_T = hydraulic radius found using tables B-1 through B-9, pages B-9 through B-17
V = velocity

—which is greater than the lower limit of 44.9 and less than 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 on the ditch design work sheet.

- **Step 7.** Determine if the ditch is appropriate. This process describes a symmetrical, triangular ditch in GM_d 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 feet per second, the ditch can be considered self-cleaning and requires little maintenance. Peak runoff will remove any silt buildup from the channel bottom. With velocity = 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. Use the following equation to calculate cutting depth:

$$Cutting\ depth = depth + freeboard$$

Using the rutting depth just calculated, cutting depth = 1.9 + 0.5 = 2.4 feet. 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.

Alternative Solution. (Calculator Method) If it is more convenient to use the calculator method than the nomograph, 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 foot 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. Given—

V = 4 feet per second
N = 0.024
S = 0.0052 feet per foot

$$R_M = \left[\frac{Vn}{1.486\sqrt{S}}\right]^{3/2}$$

Example (continued)

- **Step 4.** Determine the area of water (A_W) and the depth of water (d). As explained in figure 7-6, page 7-14—

$$A_W = 1/2(xd + yd)d \quad \text{for a triangular ditch}$$

For this problem, x = 3, y = 3

$$\begin{aligned} A_W &= 1/2(3d + 3d)d \\ &= 1/2(6d)d \\ &= 3d2 \end{aligned}$$

The wetted perimeter (wp), as explained in figure 7-7, page 7-15, is—

$$wp = Cl + C2 = (xd)2 + d + (yd)2 + d$$

Substituting values of x and y—

$$\begin{aligned} wp &= \sqrt{(3d)^2 + d^2} + \sqrt{(3d)^2 + d^2} \\ &= 2\sqrt{(9d^2 + d^2)} \\ &= 2\sqrt{10d^2} \\ &= 2d\sqrt{10} \end{aligned}$$

Therefore, the hydraulic radius (R_m) is—

$$\begin{aligned} R_m &= \frac{A_W}{WP} \\ &= \frac{3d^2}{2d\sqrt{10}} = \frac{3d^2}{2(3.16)d} = \frac{3d^2}{6.32d} = 0.474\ d \end{aligned}$$

For depth, the depth of water for the conditions assumed in Step 3 is calculated. Since R_m = 0.848 from Step 3 and R_m is equal to 0.474d, the depth of water can be computed.

$$d = \frac{0.848}{0.474} = 1.79\ feet$$

$$A_W = 3d2 = 3(1.79\ ft)2 = 9.61\ square\ feet$$

- **Step 5.** Check for Q.

$$Q = A_W V\ =\ 9.6(4)\ =\ 38.4\ cubic\ feet\ per\ second$$

The calculated quantity for the ditch must be within ± 5 percent of the design quantity of 47.3 cubic feet per second, or between 44.9 and 49.7. The calculated ditch quantity is 38.4, which is less than the lower limit and is unacceptable. Since the calculated quantity 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.

$$\begin{aligned} R_m &= \left[\frac{Vn}{1.486\sqrt{S}}\right]^{3/2} = \left[\frac{4.5\ (0.24)}{1.486\ (0.072)}\right]^{3/2} \\ &= \left[\frac{0.108}{0.107}\right]^{3/2} \end{aligned}$$

$$R_M = 1.015$$

$$d = \frac{R_M}{0.474} = \frac{1.01}{0.474} = 2.13\ feet$$

$$A_W = 3d^2 = 3\ (2.13)^2 = 13.6\ square\ feet$$

$$Q = A_W V = (13.6)\ 4.5 = 61.2\ cubic\ feet\ per\ second$$

Q is too high; try V = 4.2 feet per second.

Example (continued)

- **Step 7.** Make a third trial calculation.

$$d = \frac{0.912}{0.474} = 1.92\ feet$$
$$A_W = 3d^2 = 3(1.92)^2 = 11.06\ square\ feet$$
$$Q = A_W V = (11.06)4.2 = 46.5\ feet\ per\ second$$

The quantity of runoff is acceptable because it is between the limits.

- **Step 8.** Determine the ditch depth. With the quantity 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 on the conditions external to the ditch but cannot be less than 0.5 foot. Therefore—

$$Cutting\ depth = water\ depth(d) + 0.5 = 1.92 + 0.5 = 2.42\ feet\ for\ 2.4\ feet$$

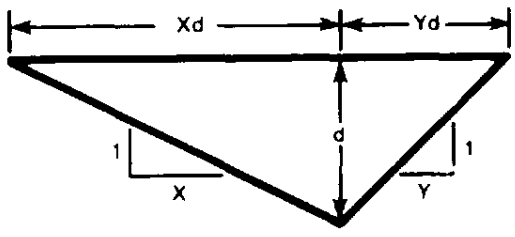
The velocity of 4.2 feet per second is acceptable. Some erosion may be anticipated, but it will not be serious. In addition, because of the high velocity, the flow will clean out the sediment from previous low flow.

Table 7-4. Sample ditch design work sheet example

Ditch Selection		*Assumed*	*Nomograph*	*Tables B-1 through B-9, pages B-9 through B-17*			*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

Legend:

cfs	cubic feet per second
D	depth
ft	feet
Q	quantity of runoff
sq	square
R_M	hydraulic radius found using Manning's equation
R_t	hydraulic radius found using the tables in appendix B
V	velocity

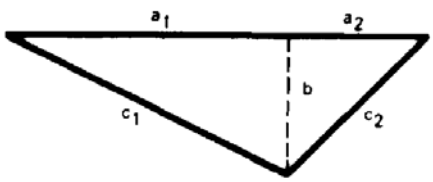


Area of any triangle = ½(base)(height)
Base of triangle = Xd + Yd
Height of triangle = d
Therefore, area = ½(Xd + Yd)d

Legend:

d	depth of water

Figure 7-6. Determination of the area of a triangular ditch



If the large triangle is divided into two right triangles, the Pythagorean theorem is applicable.

$(c_1)^2 = (a_1)^2 + b^2$, and $(c_2)^2 = (a_2)^2 + b^2$

Therefore, wetted perimeter (WP) = $c_1 + c_2$

$$= \sqrt{(a_1)^2 + b^2} + \sqrt{(a_2)^2 + b^2}$$

$$= \sqrt{(Xd)^2 + d^2} + \sqrt{(Yd)^2 + d^2}$$

Legend:

WP	wetted perimeter

Figure 7-7. Determination of the wetted perimeter of a triangular ditch

SPECIAL CHANNELS

7-36. Some facilities will have special channels where surface runoff will be intercepted and removed. These channels are similar to open channels except that they tend to be very wide and shallow. To determine runoff capacity (in cubic feet per second) in this type of channel, open channel hydraulics must be modified.

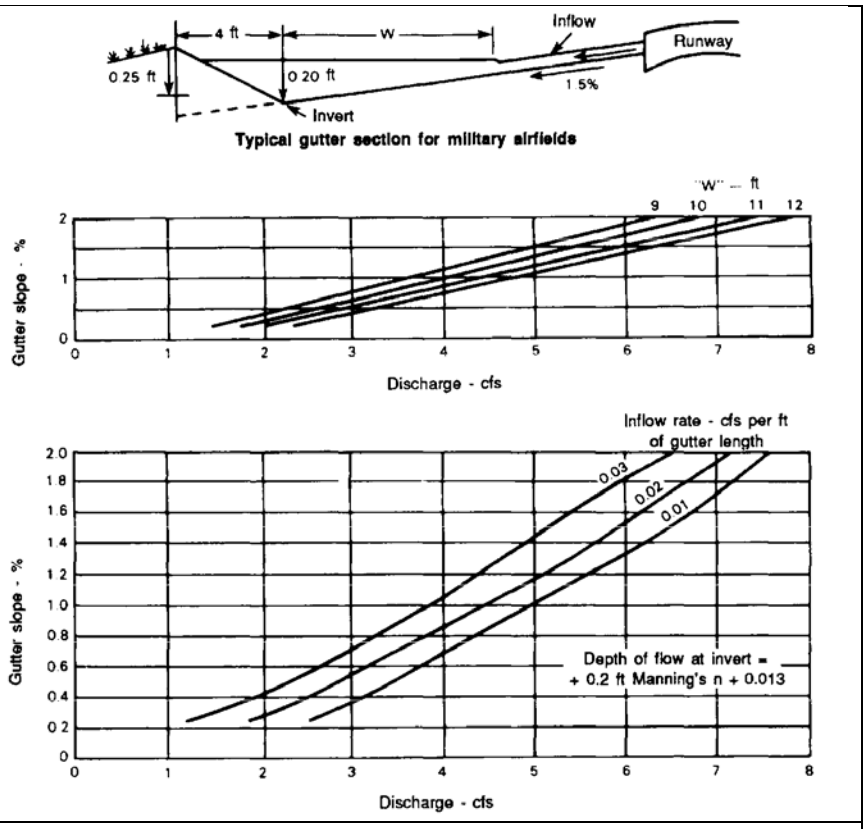
Gutters

7-37. Gutters are shallow, paved drainage channels used in more permanent construction adjacent paved or hard-surfaced areas. They provide positive removal of runoff, protection for easily eroded soils adjacent to the pavement, and prevent softening of turf shoulder areas commonly caused by a large volume of runoff from adjoining pavements.

7-38. A cross section of a typical runway gutter is shown in figure 7-8, page 7-16. This gutter conforms to United States 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

7-39. Channels will be placed down the center of medians, dual roads, runways, taxiways, and similar structures. These channels will be symmetrical but 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, roughness coefficient, flow, and the general width are known, the depth can be determined by trial using the flow equation or nomograph for flow in open channels.



Legend:

cfs	cubic feet per second
ft	feet
n	roughness coefficient

Figure 7-8. Typical runway gutter

Berms

7-40. 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 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. Constructing a berm (a raised lip that is usually made of earth) prevents this problem. (See figure 7-9.) Place the berm at the top edge of the channel. The berm prevents inflow into the channel except at designated points where an inlet—properly protected against erosion—is provided.

7-41. Where excavated material is wasted, such as a levee or dike parallel to the channel, frequent openings through the levee to permit inflow to the channel are needed. 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.

7-42. Runoff from the area is determined by the rational method. This runoff is collected and conveyed by the channel formed by the berm. The factors of quantity, roughness coefficient, slope, and width are used to solve for depth by the trial method.

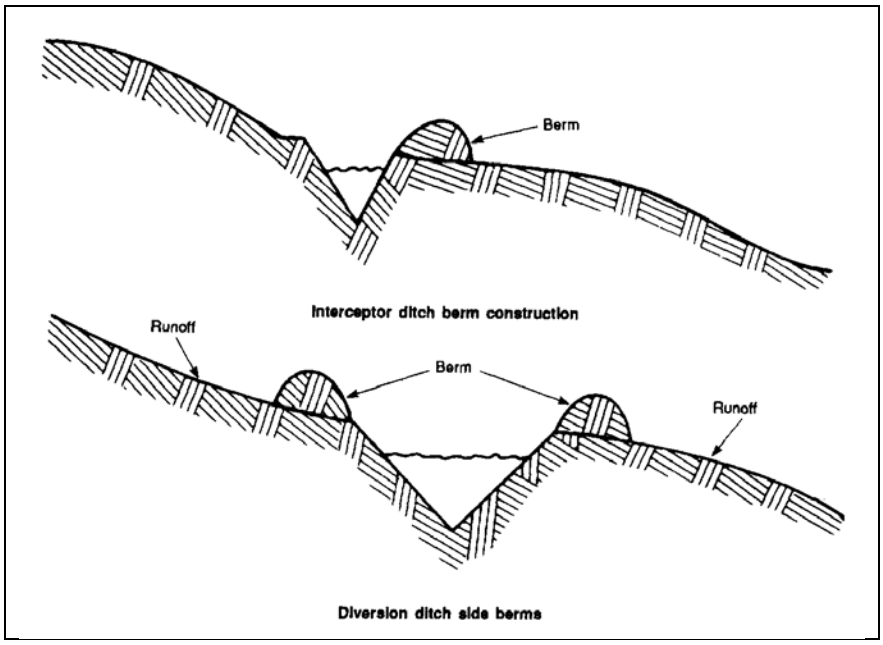


Figure 7-9. Typical berm emplacement

CONSTRUCTION AND MAINTENANCE

7-43. Ditch construction normally requires a grader or scraper. Sometimes they are used together. Depending on the location, other factors must be considered in designing and building ditches. Interceptor ditches should be set on grades, allowing 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 to prevent overflow and excessive erosion of the downhill slopes. (See figure 7-9.)

7-44. Abrupt changes in the normal flow pattern of a ditch will induce turbulence and cause excessive erosion. These conditions usually develop at channel transitions, junctions, and storm drain outlets. Accordingly, special attention must be given to these locations during design.

7-45. Channel maintenance problems exist in 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. When used together, channel protection and ditch shape control maintenance problems effectively.

SEDIMENT CONTROL

7-46. Water flowing over land tends to carry sediment into open channels. When channel velocity is 3 feet per second or less, 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 if the channel is not cleaned out; therefore, the channel should be kept clean through maintenance.

7-47. When designing channels, keep peak velocity flow above 3 feet per second. The channel is self-cleaning at this velocity. A higher velocity is preferred, but the velocity must not exceed the maximum velocity for the soil. (See table 7-5, page 7-18.) When the soil type requires a maximum allowable velocity less than 3 feet per second, sedimentation will be a maintenance problem. One solution is to line the ditch with asphalt or concrete. This reduces the coefficient of friction, increasing the velocity. When sedimentation is expected and linings cannot be used, increase the allowable freeboard to eliminate overflow.

Table 7-5. 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			
Legend: ASCE American Society of Civil Engineers fps feet per second			

EROSION CONTROL

7-48. Water flowing through open channels is turbulent. This turbulence increases as the velocity increases. Velocity and turbulence erode and carry away channel soil, which endangers nearby structures (roads, bridges, culverts) and the channel. In addition, the eroded material may be deposited in areas within the channel that will cause damage. In these cases, channel maintenance and repair will be a constant concern.

7-49. Erosion occurs when the velocity of flow exceeds the velocity at which channel soil erodes. (See table 7-5 for maximum velocities.) Lowering the velocity below the soil-erosion velocity can prevent erosion. This can be accomplished by lining the natural channel material with a more erosion-resistant material or by reducing the side slopes. Table 7-6 shows recommended side slopes. Erosion should be considered and accounted for in the design of channels.

Table 7-6. Recommended side slopes

Type of Channel	*(Horizontal:Vertical)*
Firm rock	Vertical to 1/4:1
Concrete-lined stiff clay	1/2:1
Fissured rock	1/2:1
Firm earth with stone lining	1:1
Firm earth, large channels	1:1
Firm earth, small channels	1 1/2:1
Loose, sandy earth	2:1
Sandy, porous loam	3:1

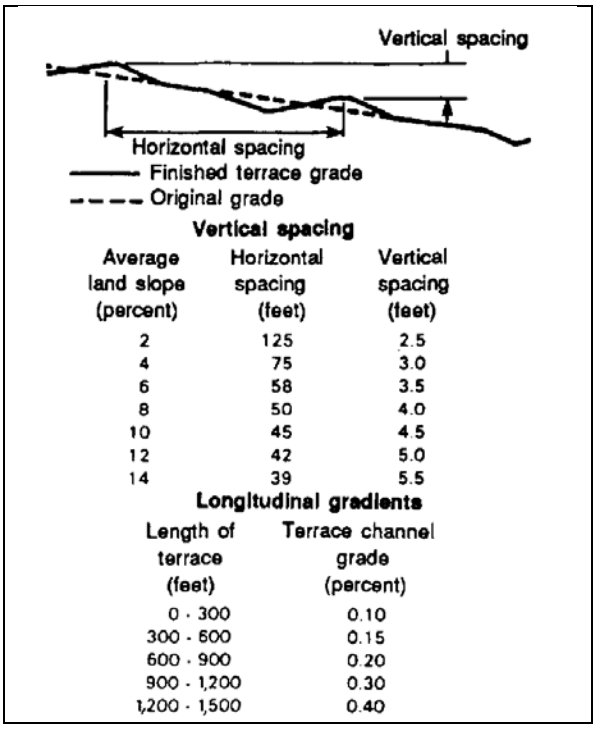
7-50. Reducing the hydraulic radius will decrease the velocity. This decrease in hydraulic radius can be accomplished by increasing the wetted perimeter in relation to the area. This can be accomplished by widening the ditch, flattening the side slopes, or widening the bottom. This increases the wetted perimeter without materially increasing the area. The required changes in ditch design are determined by the trial approach since the amount of runoff must be retained while reducing the velocity.

7-51. Erosion must be controlled to maintain an effective and clear drainage system with a minimum of maintenance and to reduce hazardous dust conditions. 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 the water, providing an erosion-resistant surface, or some combination of these techniques. This section ways to reduce or eliminate the erosive force of water.

7-52. 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 (ditches, pipe outfalls). In cases where riprap will be eroded, the use of gabions is a speedy and relatively inexpensive means of dissipating energy.

Terracing

7-53. A terrace is a low, broad-based earth levee constructed approximately parallel to the contours of the topography. A terrace 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 7-10, page 7-20.



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 7-10. Terrace spacing and gradients

Lining the Channel

7-54. Lining the bottom and sides of the channel can control erosion. Channel lining materials include grass or turf, pavement, and riprap.

Turfing

7-55. Natural linings (grass, turf) take considerable time to grow or effort to place. Because of this, they are seldom used in the JOA. Ditches are protected by placing strips of sod (held in place by wooden boards or stakes) perpendicular to the flow path at intervals along the ditch. (See figure 7-11.)

Paving and Gunite Lining

7-56. When design and construction are done properly, paving the ditch with asphalt or concrete will prevent erosion. Because of the low roughness coefficient 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 to slow the velocity before allowing the flow to continue into the natural soil channel. Because of the low roughness coefficient values of pavement, it can be used effectively to increase flow velocities if they are too low, thus preventing deposition.

7-57. 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 downslope at least to the point in the ditch that the erosive energy of the water is controlled or absorbed without erosion damage. Paving with asphalt or concrete provides superior erosion-resistant linings in gutters, ditches, and outfall structures.

7-58. Gunite lining of ditches controls erosion effectively. Gunite is a mixture of 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, and material and equipment expenses usually limit the use of paving or gunite linings to the most demanding conditions in JOA airfield construction.

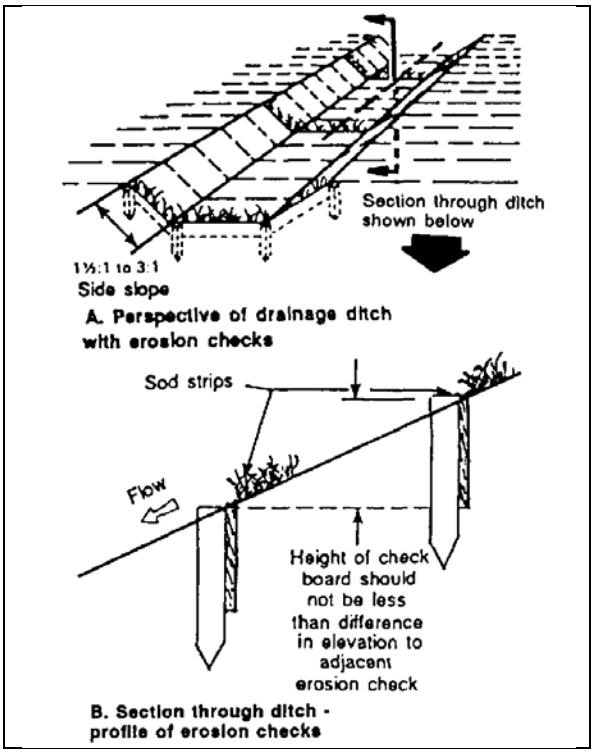


Figure 7-11. Erosion-control checks

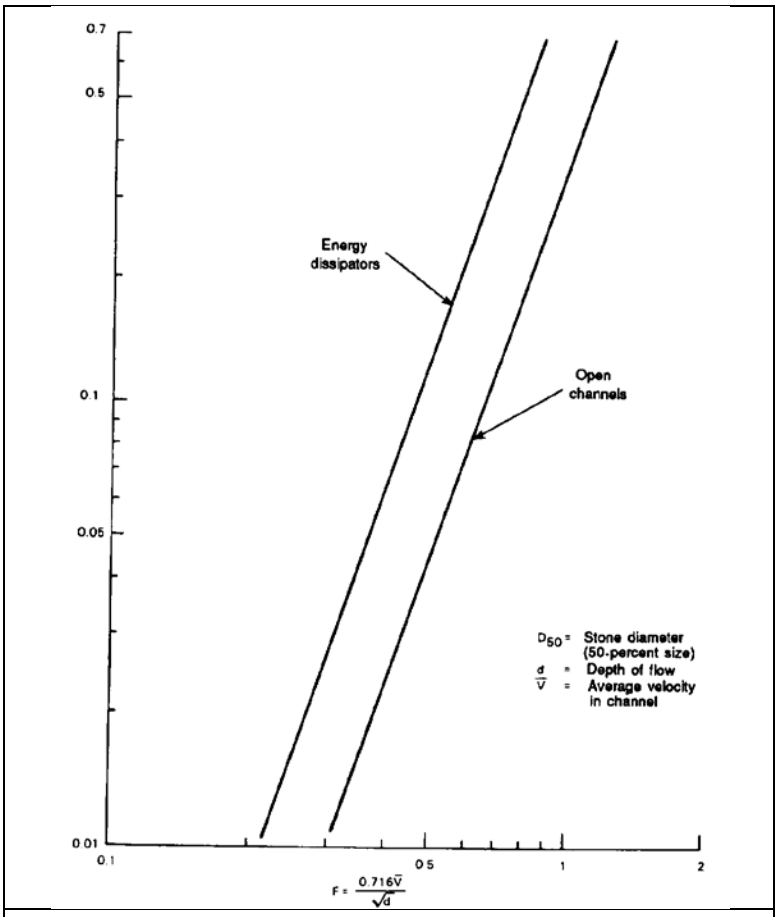
Placing Riprap

7-59. Riprap lining involves placing rocks or rubble in the bottom and on the sides of the ditch to prevent soil erosion and undesirable meandering. Rocks should be hand-placed in at least two layers and compacted individually. Riprap prevents erosion and decreases velocity in the channel because of its high roughness coefficient value. Riprap also helps prevent erosion when making transitions from paved to soil ditches or from other high-velocity ditches to those where lower velocity is required.

7-60. Riprap protection should be provided adjacent to hydraulic structures (drop structures, energy dissipaters). When placed on erodible surfaces, it prevents scour at 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. 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.

7-61. Consideration must be given to the selection of an adequate size of stone, use of adequately graded riprap, provision of a filter blanket, and proper treatment of the end of the riprap blanket. The stone size selection required for protection with Froude numbers (the ratio of inertial forces to gravity forces) and depths of flow in the channel is shown in figure 7-12, page 7-22. Two curves are given. One is for riprap subjected to direct flow or adjacent to hydraulic structures (side inlets, confluences, energy dissipaters) 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.



Legend:

d	depth of flow
D	diameter
F	flow
V	velocity

Figure 7-12. Recommended riprap sizes for open channels

7-62. With the depth of flow and average velocity in the channel known, the Froude number can be computed from the following equation:

$$F = \frac{0.716\text{V}}{\sqrt{d}}$$

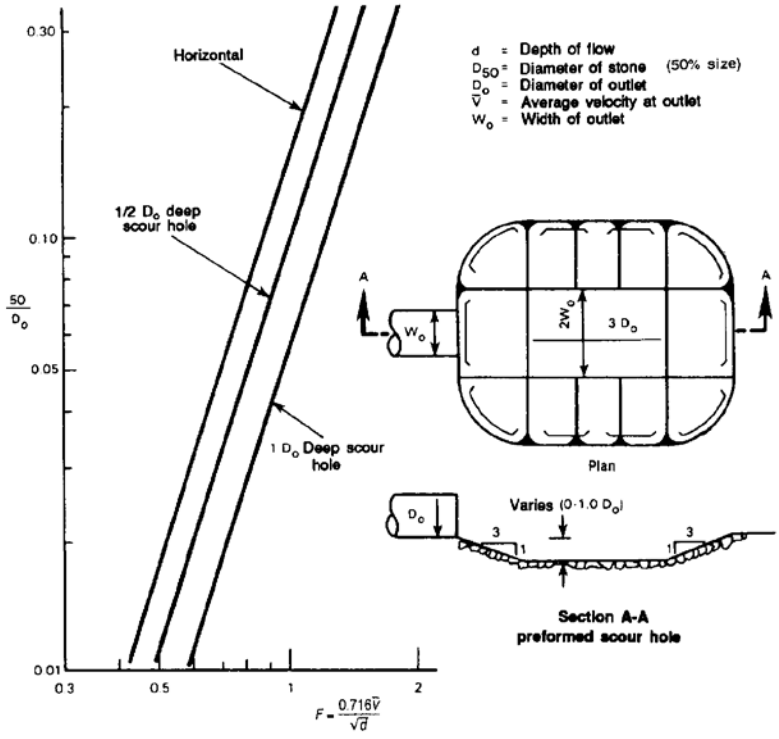
where—

D = *depth of flow*
F = *flow*
V = *velocity*

Note. The D_{50} value can be determined from the appropriate curve.

7-63. Curves for determining the riprap size required to prevent scour downstream from culvert outlets with scour holes of various depths are shown in figure 7-13. 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.

7-64. When the use of large rock is desirable but impractical, substituting a grouted reach of smaller rock in high velocity areas or turbulence may be appropriate. An ungrouted reach should follow grouted riprap.

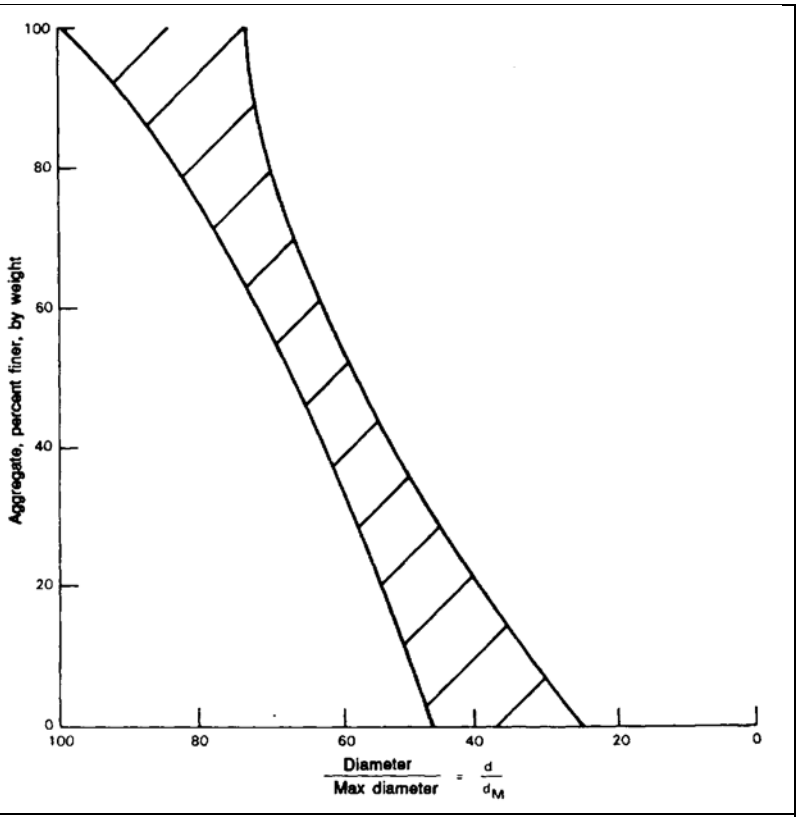


Legend:	
d	depth of flow
D	diameter
F	flow
V	velocity
W	width

Figure 7-13. Recommended riprap sizes for culvert outlets

7-65. A well-graded mixture of stone sizes is preferred to a relatively uniform size of riprap. A recommended gradation is shown in figure 7-14, page 7-24. In certain locations, the available material may dictate the gradation of riprap to be used. The 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, although a greater thickness is sometimes necessary.

7-66. 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 the bottom scour.



Legend:	
D	diameter
d_M	maximum diameter
Max	maximum

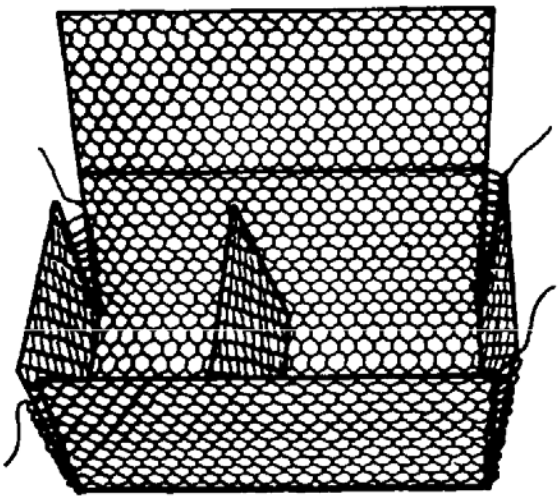
Figure 7-14. Recommended gradation of stones for riprap

Gabions

7-67. 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

7-68. 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. (See figure 7-15.) Securely lace the vertical edge and diaphragms with binding wire.



Note. Gabion ends and diaphragms will be lifted into a vertical position and laced to form a box.

Figure 7-15. Assembly of a gabion

Installation

7-69. 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 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. (See figure 7-16, page 7-26.)

7-70. 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-a-long* ratchet 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 the corners to ensure that the lacing is secure and the corners are closed. Keep gabions taut while they are being filled with stone.

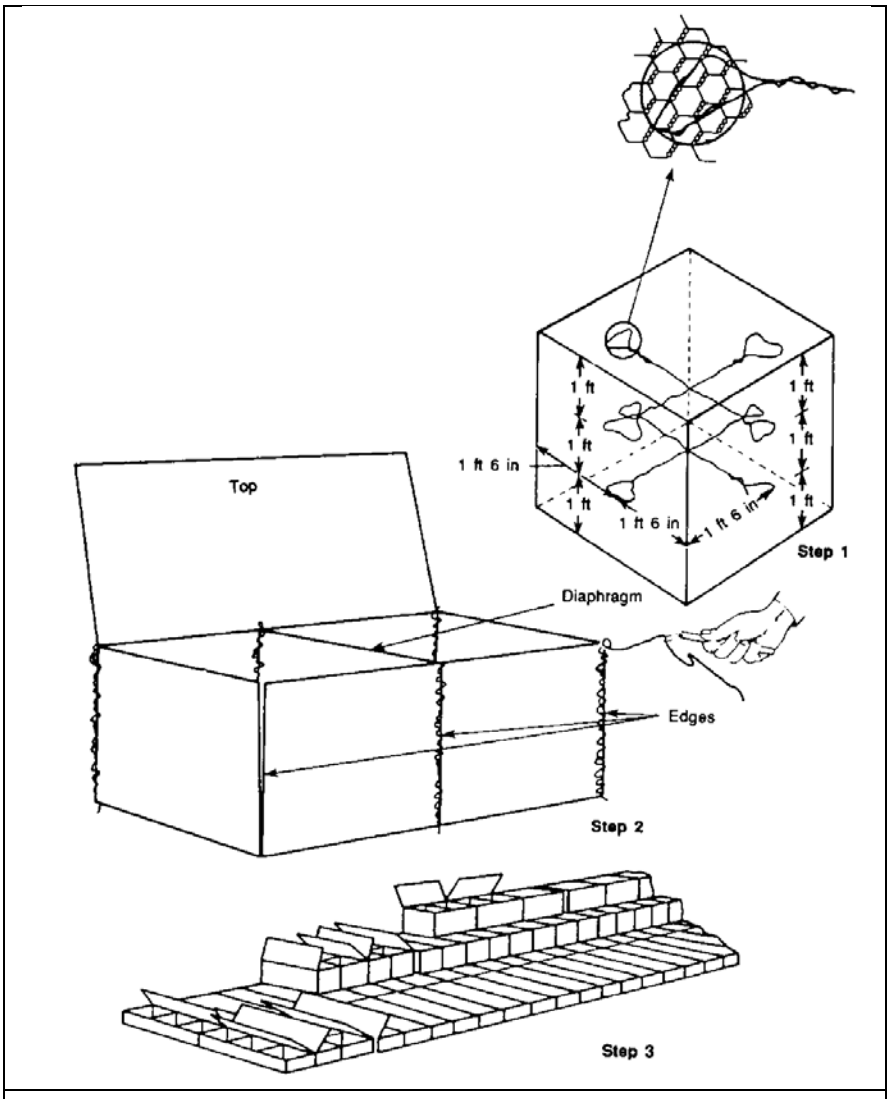
7-71. Where water, soil, and atmospheric conditions allow, galvanized wire mesh can have a life of 40 years or more. For soils and water showing a hydrogen (pH) factor of less than 7 or more than 12, plastic-coated wire must be used to form gabions.

Filling Procedures

7-72. 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. Use the following steps when performing the filling operation:

- **Step 1.** Fill the gabionsto a depth of 1 foot.
- **Step 2.** Place one connecting wire in each direction and loop around two meshes of the gabion wall. (See figure 7-16.)
- **Step 3.** Repeat this operation twice or until the gabion is filled; then fold the top shut and wire it to the ends, sides, and diaphragms.
- **Step 4.** Pack the stone inside the compartment as tight as practical.
- **Step 5.** Protect the vertical diaphragm during the filling operation, temporarily place rebars and lace them along the upper edges.

7-73.



Legend:	
ft	feet
in	inches

Figure 7-16. Assembly and construction of gabions

7-74. 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 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.

7-75. The strong interconnection of the 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, are wired to the filled gabions at the front and back.

7-76. Gabions may be filled by almost any type of earth-handling equipment (payloader, crane, conveyor, 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.

Maintenance and Repair

7-77. 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. (See figure 7-17.)

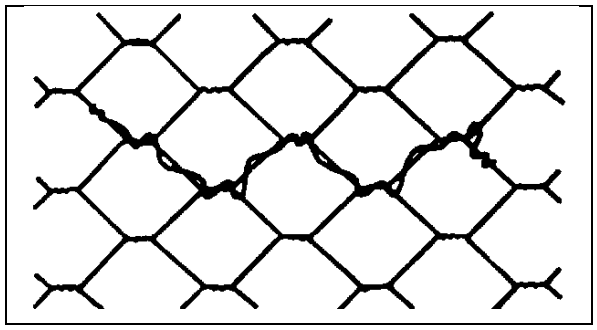
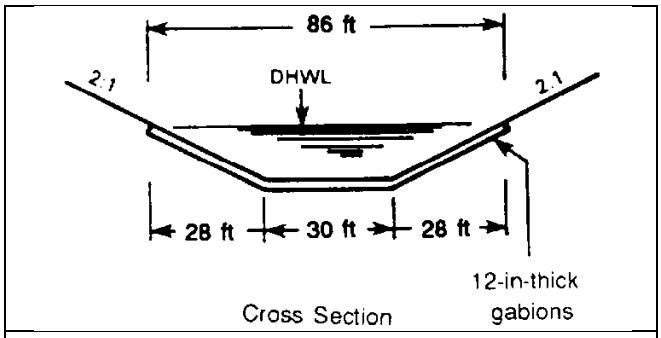


Figure 7-17. Repairing a broken wire

Thickness

7-78. 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°, and a flow velocity of about 10 feet per second. (See figure 7-18.) Use an 18-inch gabion lining for curved channel sections with a side slope of 45°. 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.



Legend:	
DHWL	design high water level
ft	feet
in	inch

Figure 7-18. Typical channel lining using gabions

7-79. 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 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. Gabion-lined channels may be designed using Manning's equation and the procedures for open-channel design.

Uses

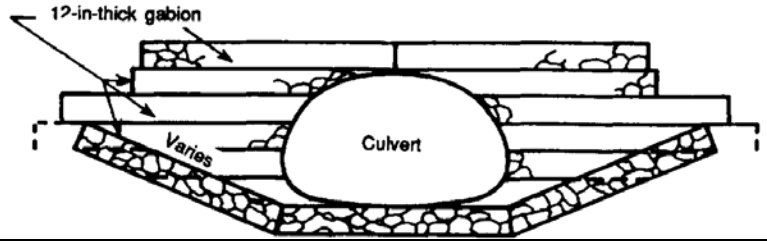
7-80. Gabions can be used in the following ways:

- Protective and antierosion structures on rivers (revetments, groins, spurs).
- Other antierosion structures (weirs, drop structures, check dams).
- Channel linings.
- Seashore protection.
- Low-water bridges or fords.
- Culvert headwall and outlet structures.
- Bridge abutments and wing walls.

7-81. Culvert inlets and outlets are often modified by using transition structures to reduce entrance losses and inhibit erosion. Therefore, gabions are most commonly used for headwalls and outlet aprons.

7-82. 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 7-18, page 7-27. Straight headwalls are generally used on small, roadside culverts under driveways and in small channels having a low approach velocity. They are also recommended where there is a tendency for lateral erosion to develop at the outlet.

7-83. 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 7-19.) Table 7-7 shows the gabion protection required for various outlet velocity ranges.



Legend:
in inch

Figure 7-19. Culvert inlet or outlet using gabion headwall and channel lining

Table 7-7. 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 dissipater or stilling basin required	—

Legend:
fps feet per second
ft feet
in inches

Culvert Outlets

7-84. Most culverts operate under free outfall conditions (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 feet per second in uniform-graded sand and cohesionless silts, 2.5 feet per second in well-graded sand, 3.0 feet per second in silty sand, 4.0 feet per second in clay, and 6.0 feet per second in gravel. These velocities should be used only as a general guide.

7-85. If possible, study local material to determine its erosion tendencies before a decision on the degree of protection is required. The study should consider the following types of outfalls offering three degrees of protection:

- 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 before releasing the flow to the outlet channel.
- Stilling basins result in dissipation of energy on the protective works.

Plain Outlets

7-86. 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.

7-87. Cantilevered 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

7-88. 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 attacks at the ends of the walls. A flared transition is effective if it is proportioned so that eddies induced by the effluent jet does not continue beyond the end of the wall or overtop a sloped wall.

7-89. As a guideline, it is suggested that the product of velocity and flare angle not exceed 150°. For example, if effluent velocity is 5 feet per second, each wing wall may flare 30°. If velocity is 15 feet per second, the flare should not exceed 10°. 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.

7-90. 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 that 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 superelevation of the water surface. A paved apron is required with warped end walls. Riprap is usually required at the end of a transition-type outlet.

Stilling Basins

7-91. At culvert outlets where a high concentration of energy or easily eroded materials makes excessive erosion likely, a stilling basin or other energy-dissipating device is required. For JOA construction, riprap or simple, concrete stilling basins are usually required. There are many types of energy-dissipating devices (hydraulic-jump basins, roller buckets, flip buckets, impact-energy-dissipating devices, stilling wells). In unusual cases involving major structures, the use of a special type of device should be considered. Three types of dissipaters that may offer a solution are the hydraulic-jump stilling basin with details developed at

the Saint Anthony Falls Hydraulic Laboratory, the impact-energy dissipater with details developed at the hydraulic laboratory of the Bureau of Reclamation, and stilling wells. These dissipaters are beyond the scope of JOA construction. Design procedures are not included in this manual.

Installing Check Dams

7-92. Decreasing the slope can also reduce the water velocity in a channel. 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. (See figure 7-20.)

7-93. Check dams should be considered when the slope ranges between two and eight percent. Channels with slopes of two percent or less generally do not require extensive erosion controls. With slopes in excess of eight percent, it is usually more economical to pave the ditch with asphalt or concrete than to build check dams.

Design

7-94. 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.)*

A = *slope of the original ditch in percent*

B = *desired slope in percent (This value should be set at two percent. This is the maximum slope that will not require additional erosion control.)*

Example

Find S. Given—

Original slope = *5 percent*

Desired slope = *2 percent*

H = *3 feet*

$$S = \frac{100(H)}{A - B} = \frac{100(3)}{5 - 2} = \frac{300}{3} = 100$$

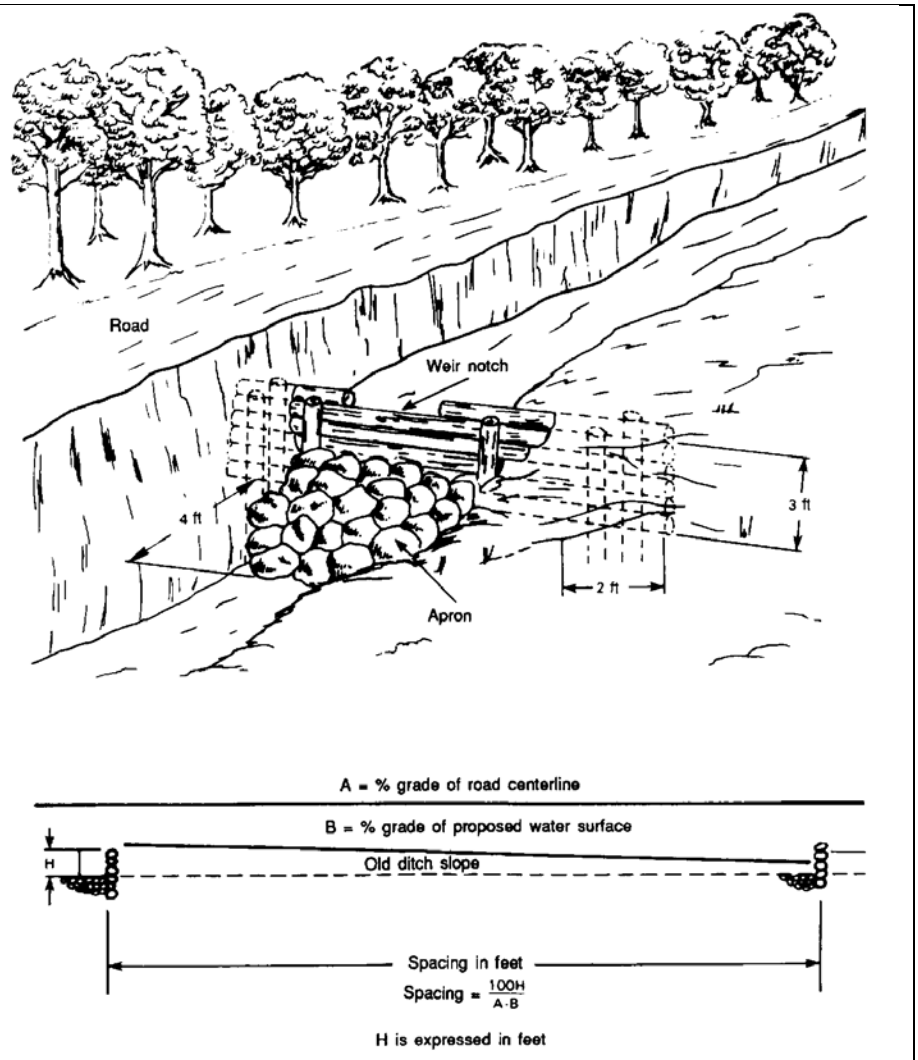
where—

S = *spacing*

H = *height*

A = *slope of the original ditch in percent*

B = *desired slope in percent*



Legend:

H	height

Figure 7-20. Check-dam design

7-95. The spacing between check dams is 100 feet. To prevent erosion of the channel sides 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 7-8, page 7-32.

Maintenance

7-96. Erosion is a common problem with check dams. It usually occurs when the weir 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. (See figure 7-21, page 7-32.) If this occurs, there will be more damage caused to adjacent structures than if the dam had never been installed.

7-97. Some protection usually takes the form of an extended horizontal or sloping apron. Anchoring the sides and bottom into at least 2 feet of compacted material can also prevent scour. Then place riprap along at least 4 feet of the downstream channel. These construction techniques will prevent scour and undercutting

Table 7-8. Discharge for weir notches in check dams

Compute by Formula $Q = 3.39L(H^{3/2})$
Where—
Q = discharge in cubic feet per second
L = length of weir notch in feet
H = depth of weir notch in feet

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

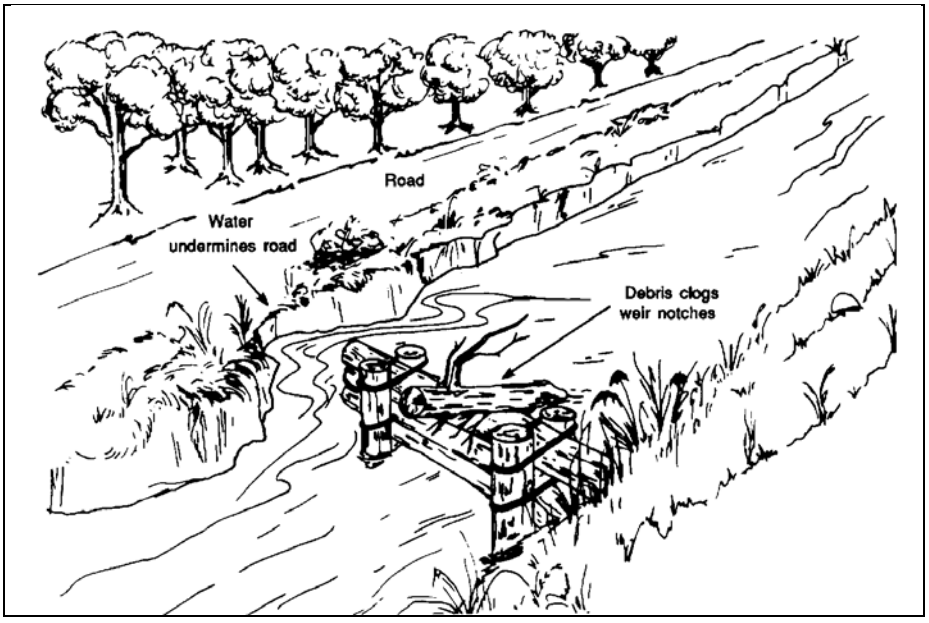


Figure 7-21. Check-dam maintenance

Chapter 8
Culvert Design

A culvert is an enclosed waterway used to pass water through an embankment or fill. The flow in a culvert depends on several factors (lining, slope, headwalls, wing walls, downstream conditions). Accordingly, culvert types and construction procedures are discussed before considering culvert hydraulics and design features.

CULVERT TYPES AND DESIGNS

8-1. There are several factors to consider when selecting and installing culverts for subsurface drainage. In this chapter, several common culvert types are presented along with considerations for when to use each type in design. Also discussed in this chapter are the parameters for proper design and installation of culverts.

PIPE SELECTION, BEDDING, AND BACKFILL

8-2. A drainage pipe is defined as a structure (other than a bridge) to convey water through a trench or under a fill or other obstruction. The selection of a suitable construction conduit will be governed by the availability and suitability of pipe materials for local conditions with due consideration of economic factors. When alternate designs are advantageous, each system will be economically designed for full capacity, best slope, least depth, and proper strength; installation provisions will be made for each material involved. When field conditions dictate the preference of one pipe material to another, the reasons will be clearly presented in the design analysis.

8-3. Consider life cycle cost factors in selecting the type of pipe to be used in construction. The factors include strength under maximum or minimum cover being provided, pipe bedding, backfill conditions, anticipated loadings, length of pipe sections, ease of installation, resistance to corrosive action by liquids carried or surrounding soil materials, suitability of jointing methods, provisions for expected deflection without adverse effect on the pipe structure or on the joints or overlying materials, and cost of maintenance. Although it is possible to obtain an acceptable pipe installation to meet design requirements by establishing special provisions for several materials, usually just one or two materials will economically meet individual requirements for a proposed drainage system.

8-4. Department of Defense has approved the use of plastic pipe for low volume roadway applications. However, plastic pipe is not approved for use under airfield pavement except for subsurface water collection and disposal.

8-5. Corrugated metal pipe (CMP) with any percentage of bituminous coating will not be installed where solvents will enter the pipe. If solvents are expected, polymeric coated corrugated steel pipe is recommended.

8-6. The selection of culvert materials to withstand deterioration from corrosion or abrasion will be based on these specific considerations. Rigid or plastic pipes are preferable where industrial wastes, spilled petroleum products, or other substances harmful to bituminous paving and coating in CMP are apt to be present. Typically, concrete pipe should not be used where soil is more acidic than pH 5.5 or where the fluid carried has a pH less than 5.5 or higher than 9.0. High density polyethylene pipe is unaffected by acidic or alkaline soil conditions. Concrete pipe can be engineered to perform very satisfactorily in the more severe acidic or alkaline environments. Type II or Type V cements should be used where soils and water have a moderate or high sulfate concentration. High-density concrete pipe is recommended when the culvert will be subject to tidal drainage and saltwater spray. Where highly corrosive substances are to be carried, the resistive qualities of vitrified clay pipe or plastic-lined concrete pipe should be considered.

8-7. Corrugated steel pipe will be galvanized and will generally be bituminous coated for permanent installations. Bituminous coating or polymeric coating is recommended for corrugated steel pipe subjected

to stagnant water; where dense decaying vegetation is present to form organic acids; where there is continuous wetness or continuous flow; and in well-drained, normally dry, alkali soils. Polymeric-coated pipe is not damaged by spilled petroleum products or industrial wastes. Corrugated aluminum alloy pipe, fabricated in the same shapes and sizes as corrugated steel pipe, shows corrosion resistance in clear granular materials even when subjected to sea water. Corrugated aluminum pipe will not be installed in soils that are highly acid (pH less than 5), are alkaline (pH greater than 9), in metallic contact with other metals or metallic deposits, where known corrosive conditions are present, or where bacterial corrosion is known to exist. Also, it will not be installed in material classified as OH (organic clays of medium to high plasticity, organic silts) or OL (organic silts and organic silty clays of low plasticity) according to the USCS (see UFC 3-220-10N on Soil Mechanics). Although bituminous coatings can be applied to aluminum alloy pipe, these coatings do not provide adequate protection (bituminous adhesion is poor) under these corrosive conditions. Suitable protective coatings for aluminum alloy have been developed but are not economically feasible for culverts or storm drains. When considering a coating to use, performance data from users in the area can be helpful. Performance history indicates various successes or failures of coatings and their probable cause; these histories are available from local highway departments.

PERMANENT CULVERTS

8-8. Materials for permanent-type installations include nonreinforced concrete, reinforced concrete, corrugated steel, plastic, corrugated aluminum alloy, and structural plate steel pipe.

Corrugated-Metal-Pipe Culverts

8-9. CMP is commonly used in military construction because it is commercially available in numerous shapes, lengths, and diameters. For JOA construction, CMP is made of an aluminum and steel alloy. It comes in nested half sections that are 2-foot lengths when assembled. It is available in the diameters and gauges listed in table 8-1. The minimum diameter recommended is 18 inches for lengths up to 20 feet and 24 inches for the other lengths. Small diameters may become clogged with debris and are difficult to maintain.

Table 8-1. Sizes of CMP

Diameter (inches)	*Gauge*	*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

8-10. Specific construction techniques are employed when placing CMP. A retaining wall called a headwall is placed at the upstream end of the culvert. Headwalls are always used upstream; they are 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.

8-11. CMP joints must be overlapped so that water flowing through the culvert passes over the joint, not into it. Failure to properly overlap the pipes allows the water to force through the joints and into the fill. Joints must be sealed, preferably with caulk or bituminous material. If the joints are not sealed, voids may be generated around culverts, causing the fill to collapse. To increase the velocity for a greater quantity of flow, line the culvert with asphalt. Commercial CMP is available with asphalt linings.

8-12. CMP has flange-type fittings that are easily fastened together by nuts and bolts that come with the sections. Vise grips and a ratchet set make assembly faster and easier.

High-Density Polyethylene Pipe

8-13. High-density polyethylene (HDPE) pipe is commonly used in construction because it is readily available in numerous corrugation types, lengths, and diameters. Type C HDPE pipe comes with corrugation on the inside and outside. Type S HDPE pipe comes with a smooth interior and a corrugated exterior. Type

D HDPE pipe has a smooth inside with a spiral type of corrugation. Pipe sizes range from 4 inch inside diameter to 72 inch inside diameter. External diameters range from 5 to 80 inches. For detailed information on installation of HDPE culverts, contact the U.S. Army Geotechnical and Structures Laboratory at ERDC USACE, in Vicksburg, MS at (601) 636-3111 or by e-mail at ERDCinfo@usace.army.mil.

Concrete-Pipe Culverts

8-14. When available, precast concrete pipe should be used for culverts instead of CMP. 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 (roughness coefficient equals 0.013) than CMP (roughness coefficient equals 0.024). Because of these advantages, concrete pipe of the same diameter and slope as CMP will carry a higher flow.

8-15. 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 each end to prevent separation at the joints. Start assembly downstream and work upstream, ensuring that male ends point downstream.

Concrete-Box Culverts

8-16. 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.

8-17. Concrete-box design requires knowledge of construction techniques for reinforced concrete structures. Box culverts can be precast, but construction in the JOA may require that they be cast in place.

EXPEDIENT CULVERTS

8-18. Expedient field-type culverts are built of material available on-site (logs, oil drums, sandbags filled with a sand-cement mixture). Some examples of culverts built of these materials appear in figure 8-1; figure 8-2, page 8-4; and figure 8-3, page 8-4. Expedient culverts built and sized properly should serve until permanent structures can be built. For evaluating hydraulics, end areas equivalent to CMP can be used for similar slopes.

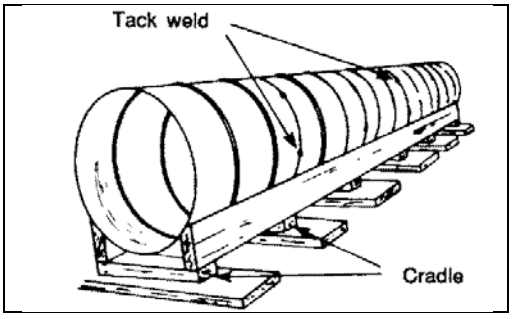


Figure 8-1. Oil-drum culvert

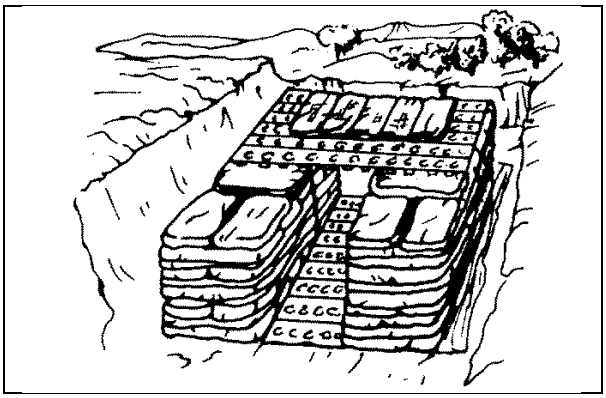


Figure 8-2. Landing-mat and sandbag culvert

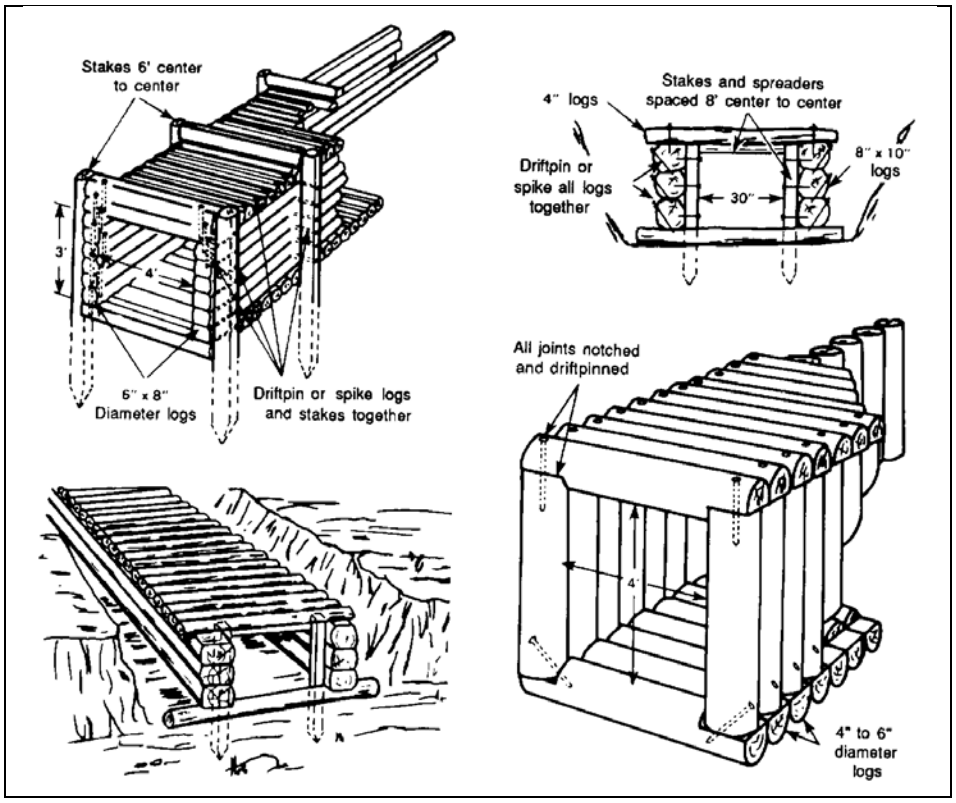


Figure 8-3. Log culverts

CULVERT CONSTRUCTION

8-19. 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 when placing culverts are alignment, slope, depth of fill, compaction, length, and erosion control.

Alignment

8-20. 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 8-4.

8-21. Place the culvert directly in the streambed to maintain an existing drainage pattern. (See view A of figure 8-4.) If the channel hydraulics are not changed, the stream will not change directions even though it may be diagonal to the fill.

8-22. Prevent the stream from shifting course at the culvert inlet or outlet. Sometimes a structure will cut across a stream meander, which leads to uncertainty when placing the culvert in the streambed. (See view B of figure 8-4.) In this case, it is best to cut a new channel to 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 similar material.

8-23. Provide a smooth transition into and out of the culvert. A structure may cut across a bend of the stream with a straight run of the stream through the structure. (See view C of figure 8-4.) 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 described above.

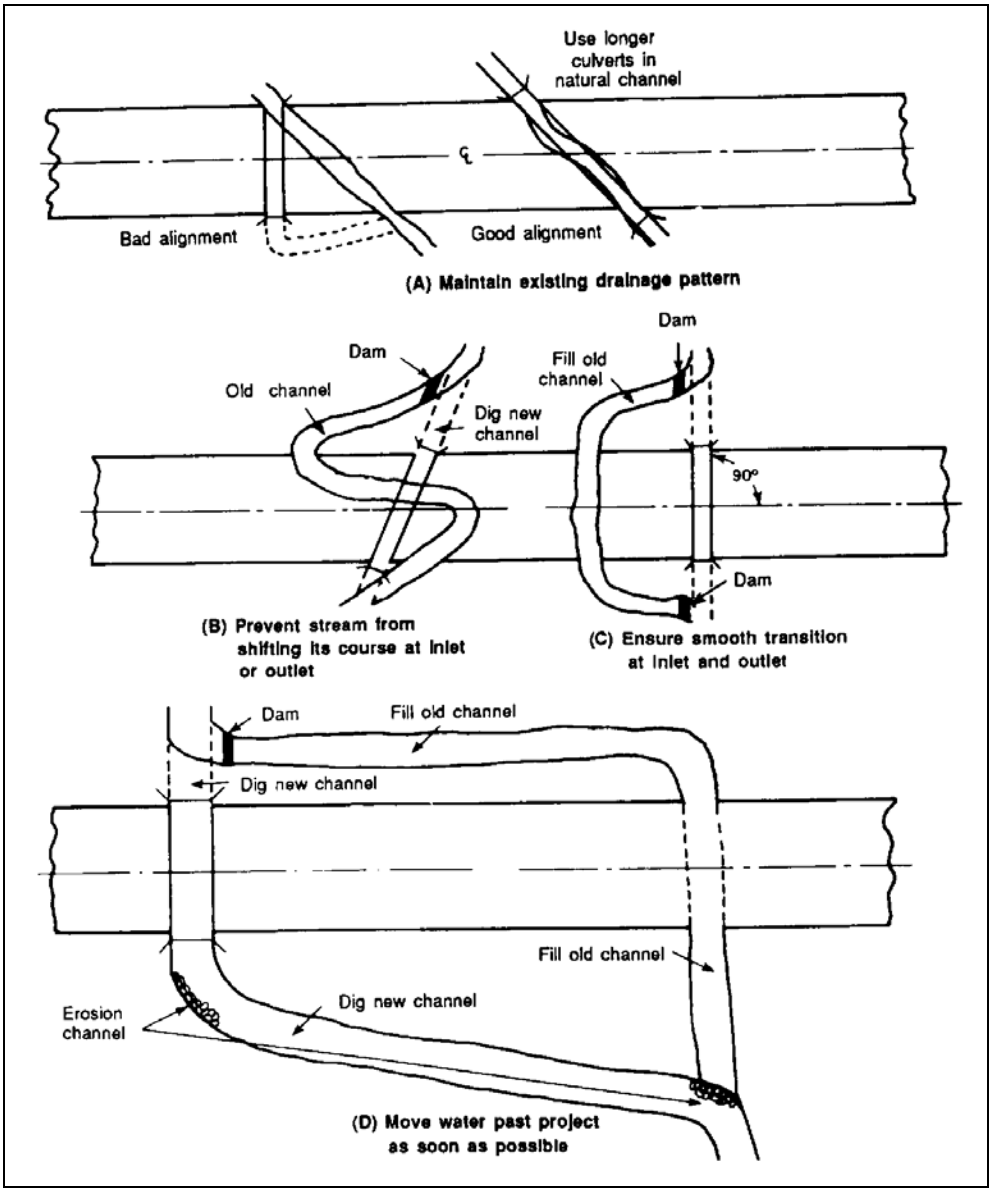


Figure 8-4. Culvert alignment

8-24. Move the water past the project as quickly as possible. When the channel flows parallel to the structure, erosion will eventually occur. To prevent erosion, dig a new channel that routes the flow through the culvert and away from the structure. (See view D of figure 8-4.) Again, be sure to fill and dam the old streambed at the junction point.

8-25. The alignment of ditch relief culverts is shown in figure 8-5, page 8-6. 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 apart. On an 8-percent grade, spacing should be reduced to 300 feet.

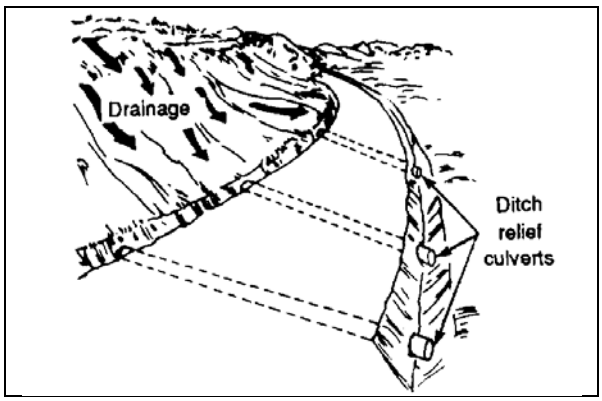


Figure 8-5. Ditch relief culverts

Slope

8-26. Culverts 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 internal cross section of an artificial channel or the bottom of the culvert. The normal grade of the culvert can be modified by the following two techniques:

- **Drop-inlet technique.** Drop inlets can be used to lower the culvert inlet. This will reduce the slope of the culvert. (See view A in figure 8-6.)
- **Outlet-above-grade technique.** The culvert outlet can be raised to reduce the slope of the culvert. (See view B in figure 8-6.)

8-27. For culverts to be self-cleaning, flow velocity should be at least 3 feet per second. To achieve this velocity, the culvert should not be set on less than a 0.5 percent grade, if practical. To prevent excessive outlet velocity, the culvert grade should not exceed 2 percent.

8-28. 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 JOA, it may be advisable to accept the erosion problem and install the culvert with a slope for an excess of 2 percent. This will prevent a blockage caused by debris passing through the pipe.

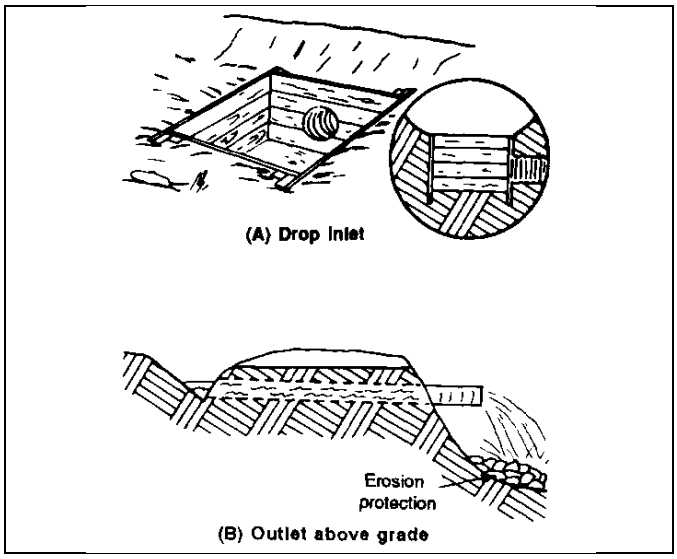
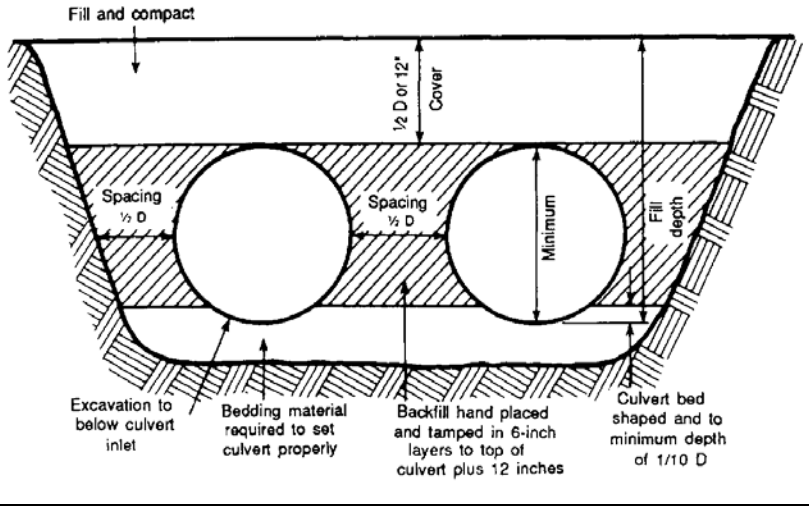


Figure 8-6. Changing the normal grade of a culvert

Depth of Fill

8-29. The distance measured from the culvert invert to the edge of the shoulder or top of the fill is the depth of fill. (See figure 8-7.) The depth of fill must be equal to or greater than the total of the cover and 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.



Legend:

D	depth of fill

Figure 8-7. Culvert specifications for CMP

Compaction

8-30. The depth of the 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, minimum protective cover and maximum permissible

depths of backfill must be included in design considerations for culverts. During construction, provide adequate additional cover to protect the culvert from damage in places where heavy construction equipment will be crossing frequently.

8-31. The minimum cover required to protect the culvert pipe against live loads will depend on 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 8-2 lists characteristics of United States aircraft. Use this table to determine the landing-gear configuration, wheel loads, and culvert weight type. This allows the selection of the proper fill requirements category in table 8-3, page 8-10, or table 8-4, page 8-11.

8-32. Table 8-3 and table 8-4 provide the cover requirements for culverts under airfields. These tables are a series of charts based on aircraft load and gear configurations. A culvert weight type and pipe diameter is identified in each chart. Use linear interpolation for the incremental diameters not listed. These charts are good for 5,000 pounds more than listed. For example, the chart 60,000-pounds single-wheel is good for single-wheel loads up to and including 65,000 pounds.

Table 8-2. Characteristics of United States aircraft

Aircraft	*Max Weight (kilopound)*	*Gear Type*	*Load on Main Gear (kilopound)*	*Culvert Weight Type*
A-7	42	S	18	2
A-10A	51	S	23	2
A-37B	15	S	7	1
F-111A/D/E	100	S	44	4
F-111F	100	S	48	4
F-4C/D/E/G	58	S	25	3
F-14C	72	S	34	4
F-15A/B	56	S	25	2
F-15C/D	68	S	30	2
F-15E	81	S	35	2
F-16A	33	S	14	2
F-18	20	S	9	1
AV-8B	28	DU-S(BIC)	10	2
AV-16A	30		11	2
B-1	477	T-T-TA	222	15
B-52G/H	488	T-T	265	16
FB-111A	114	Single	54	5
C-5B	840	T-DE-TA	400	15
C-9A	108	Twin	52	7
C-17	580	T-TA-TR1	267	15
C-130E	175	S-TA	84	13
HC-130H	175	S-TA	84	13
KC-135A	301	T-TA	142	14
C-141A/B	323	T-TA	153	14
VC-137B	334	T-TA	167	14
E-3A	323	T-TA	155	14
E-4B	800	DU-T-TA	372	16
SR-71C	170	Triple	77	9
OV-10A/B	14	DU-S(TA)	55	1
UV-18A	13	Single	6	1

Table 8-3. 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
Ga. 16	1.0	1.0					1.0	1.5					1.5	2.0					2.0	3.0				
Ga. 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				
Ga. 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		
Ga. 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
Ga. 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 assembly						40,000-lb Twin-wheel assembly						60,000-lb Twin-wheel assembly					
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
Ga. 16	2.5	4.0					1.0	1.5					N/A						N/A					
Ga. 14	2.0	3.0	4.0				1.0	1.0	1.5															
Ga. 12		2.5	3.5	4.0				1.0	1.5	1.5														
Ga. 10		2.0	3.0	3.5	4.0	4.5		1.0	1.0	1.0	1.5	2.0												
Ga. 8			2.5	3.0	3.5	4.0			1.0	1.0	1.0	1.5												
Weight type	**9**						**10**						**11**						**12**					
	80,000-lb Twin-wheel assembly						1000,000-lb Twin-wheel assembly						120,000-lb Twin-wheel assembly						40,000-lb Single-tandem assembly					
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
Ga. 16	2.5	4.5					N/A						N/A						N/A					
Ga. 14	2.5	3.5	4.5																					
Ga. 12		3.0	4.0	4.5																				
Ga. 10		2.5	3.5	4.5	4.5	5.0																		
Ga. 8			3.0	3.5	4.5	4.5																		
Weight type	**13**						**14**						**15**						**16**					
	60,000-lb Single-tandem assembly						150,000-lb Twin-tandem assembly						200,000-lb Twin-tandem assembly						265,000-lb Twin-tandem assembly					
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
Ga. 16	2.0	3.0					N/A						N/A						N/A					
Ga. 14	1.5	2.5	3.5																					
Ga. 12		1.5	2.5	3.0	3.0	3.5																		
Ga. 10		1.5	2.5	3.0	3.0	3.5																		
Ga. 8			2.0	2.5	3.0	3.0																		

Refer to UFC 3-230.01 and TM 5-820-3.

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. This table is 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 according to this table (exception note 4).
4. Pipe placed under airfield rigid pavements will have a minimum cover measured from the bottom of the slab of 0.5 foot 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 foot.
5. In seasonable frost areas, minimum pipe cover will meet the requirements in UFC 3-220-04FA for protection of storm drains against freezing conditions.

Legend:

AASHTO	American Association of State Highway and Transportation Officials
CMP	corrugated metal pipe
dia	diameter
Ga.	gauge
in	inches
lb	pounds
N/A	not applicable

Table 8-4. 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	1.5	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 Assembly						40,000-lb Twin-Wheel Assembly						60,000-lb Twin-Wheel Assembly					
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 Assembly						100,000-lb Twin-Wheel Assembly						120,000-lb Twin-Wheel Assembly						40,000-lb Single-Tandem Assembly					
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	5.9	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 Assembly						150,000-lb Twin-Tandem Assembly						200,000-lb Twin-Tandem Assembly						265,000-lb Twin-Tandem Assembly					
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

Refer to UFC 3-230.01 and TM 5-820-3.

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. This table is 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 according to this table (exception note 4).
4. Pipe placed under airfield rigid pavements will have a minimum cover measured from the bottom of the slab of 0.5 foot 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 foot.
5. Dashes (–) indicate that the allowable load is less than the load on the pipe. Blanks indicate that the pipe is not specified by applicable standards.
6. In seasonable frost areas, minimum pipe cover will meet the requirements in UFC 3-220-04FA for protection of storm drains against freezing conditions.

Legend:

AASHTO	American Association of State Highway and Transportation Officials
CMP	corrugated metal pipe
dia	diameter
in	inches
lb	pounds

8-33. To use these tables, select the chart that applies to the aircraft wheel or gear load and landing gear type. Find the appropriate culvert type using table 8-2, page 8-8. When the proper chart is located, read down the left column of the table to determine the appropriate row for the gauge concerned, and then read horizontally to the right to locate the minimum cover under the pipe diameter column.

8-34. For example, a C130E Hercules requires culvert weight type 13. (See table 8-2.) Select chart 13 (see table 8-3, page 8-9) for a 60,000-pound single-tandem to select 12-gauge pipe. For a 36-inch culvert, the minimum required cover would be 2.5 feet.

8-35. For the maximum permissible cover for CMP, see table 8-5. See table 8-6 for aluminum alloy. Reinforced concrete pipe made under standard specifications can be used in fill up to 50 feet.

Table 8-5. Maximum permissible cover for CMP

Diameter (inches)	*Maximum permissible cover (feet)*									
	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 gauge tables, CMP will conform to the requirements of UFC 3-230.01 and TM 5-820-3.
.
2. This 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 shop fabrication or field strutting and will generally be 5% of the pipe diameter.

Legend:

AASHTO	American Association of State Highway and Transportation Officials
CMP	corrugated metal pipe
lb	pounds

Table 8-6. Maximum permissible cover for corrugated aluminum-alloy pipe

Diameter (inches)	Maximum permissible cover (feet)									
	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 UFGS-33 46 16 and AASHTO A-A-60005.
2. This table will be used for normal installation conditions and 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 shop fabrication or field strutting and will generally be 5% of the pipe diameter.

Legend:
AASHTO American Association of State Highway and Transportation Officials

Bedding (Foundations)

8-36. The minimum bedding depth for pipe culverts is one-tenth the diameter of the pipe. (See figure 8-7, page 8-7.) The bedding is formed and shaped to fit the bottom of the culvert. In addition, the foundation is cambered (curved slightly upward) along the culvert centerline to allow for settlement and ensure tightness in the joints. (See figure 8-8.) The invert elevation should not increase as the flow proceeds downstream.

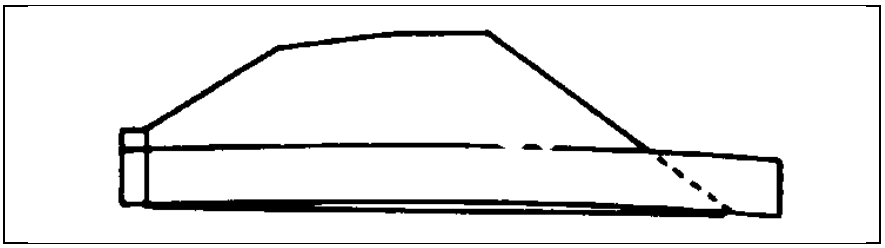
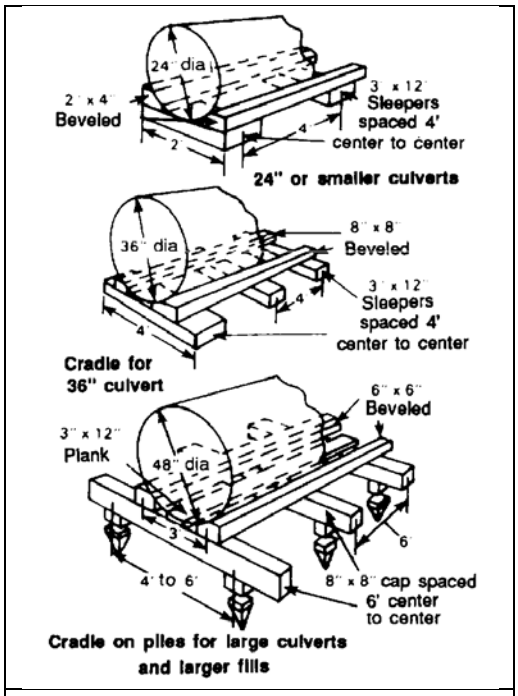


Figure 8-8. Elevation of center section of culvert

8-37. 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 (organic matter, muck, silt, large material that could puncture the CMP), remove and replace the material. The depth of material to be removed will depend on information obtained from borings. Provide cradle footings when the bearing strength of the soil is completely inadequate and uneven settlement is expected.

8-38. These footings may require piling. (See figure 8-9, page 8-14.) Footings used for timber culverts can also be adapted for various types of culverts and soils. (See figure 8-10, page 8-14.)



Legend:

dia	diameter

Figure 8-9. Cradle footings

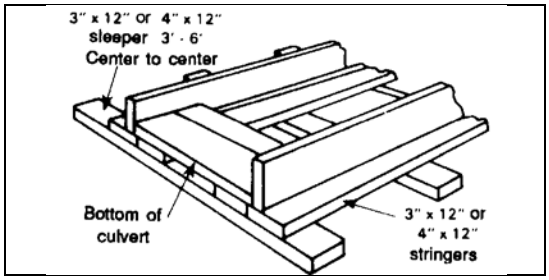
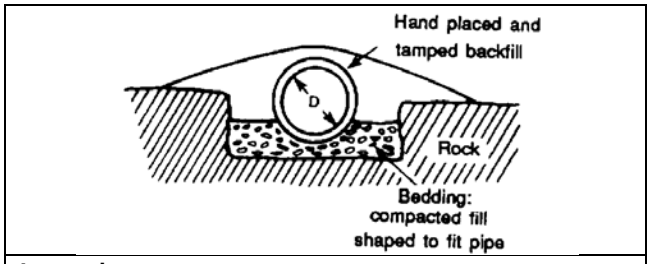


Figure 8-10. Footings for timber culverts

8-39. Culverts can be placed on properly prepared rock foundations. Trench the rock and backfill with firmly compacted soil for bedding the culvert. (See figure 8-11.)



Legend:

D	diameter

Figure 8-11. Concrete-culvert installation in rock

Backfill

8-40. Use caution when backfilling around the culvert. The backfill supports the culvert against soil pressure generated by surface loads. (See figure 8-6, page 8-7.) The soil selected should be placed in well-compacted layers kept at the same elevation on both sides of the culvert. If hand-operated, air-operated, or other mechanical tampers are used, compact in 6-inch layers. If logs are used, hand tamping or another expedient method is used. Compact the soil in 4-inch layers. Compact from the culvert bed material to 12 inches or one-half the diameter above the top of the culvert (whichever is greater).

Length

8-41. 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 to extend a minimum of 2 feet beyond the toe of the embankment to prevent erosion.

Headwalls, Wing Walls, and Aprons

8-42. Headwalls and aprons are constructed to guide water into the culvert, prevent or control erosion, reduce seepage, hold the soil in place, and support the culvert ends. Headwalls close to a road should not protrude above shoulder grade and should be at least 2 feet outside the shoulder so that it does not present a traffic hazard.

8-43. Use headwalls on the upstream end of culverts. If possible, use a headwall on the downstream end too. When concrete pipes are used, headwalls are mandatory at each end. Headwalls, upstream and downstream, should have wing walls or retaining walls set at an angle to the headwall. This supports the fill, and directs water flow to prevent erosion. The upstream wing wall guides the water into the culvert and improves culvert hydraulics. The downstream wing wall, combined with an apron, reduces the velocity of the stream and lessens outlet erosion. Headwalls and wing walls should be reinforced concrete or mortared stone. Refer to the TM 5-302 series of manuals for standard designs. These structures can be made of expedient material (lumber, logs, sandbags). (See figure 8-12, page 8-16.) For speedy construction in the JOA, sandbags filled with a soil and cement mixture may provide the best headwall possible.

8-44. When headwalls are not used on the downstream end, the culvert should project beyond the toe of the fill by at least 2 feet. Use riprap to protect the projecting area culvert riprap. (See figure 8-13, page 8-16.)

Erosion Control

8-45. 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 are gully scour and scour hole. (See figure 8-14, page 8-17.) 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.

8-46. Gully scour is expected when channel flow exceeds what is required for stability. It begins downstream where the channel is stable and progresses upstream. (See the top portion of figure 8-14.) 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 the vertical and downstream directions.

8-47. A scour hole or localized erosion is expected if the downstream channel is stable. (See the bottom portion of figure 8-14.) The severity of damage anticipated depends on existing conditions or those created at the outlet. Sometimes the extent of the scour hole may be insufficient to produce instability of the embankment or structural damage to the outlet. However, flow conditions can produce scour that erodes the embankment and causes structural damage to the apron, end wall, and culvert. (See figure 8-15, page 8-17.)

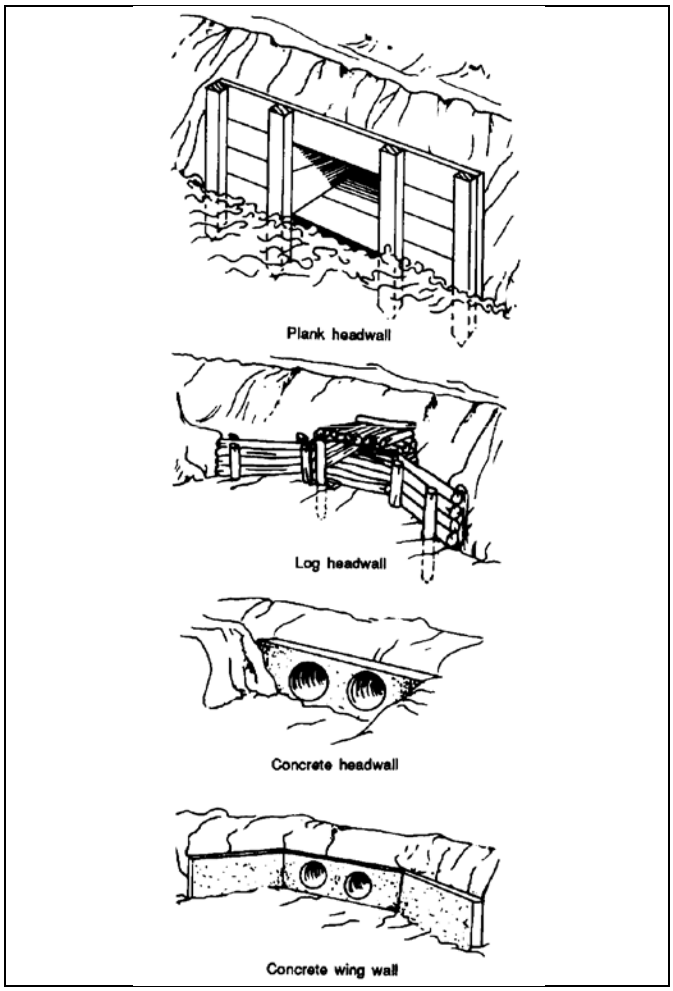
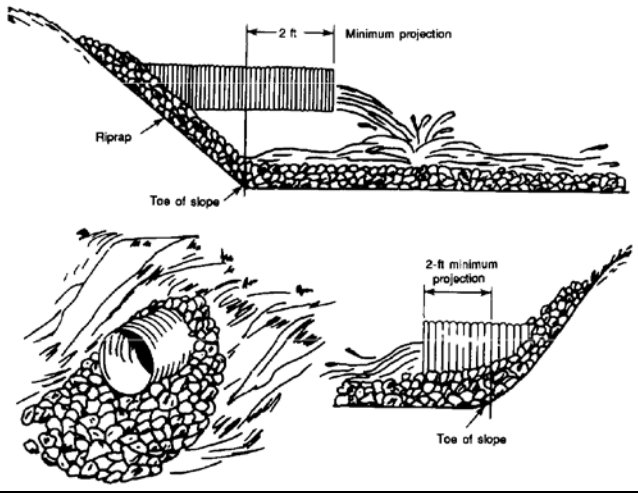


Figure 8-12. Headwalls and wing walls for culverts



Legend:
ft feet

Figure 8-13. Culvert outlets without headwalls

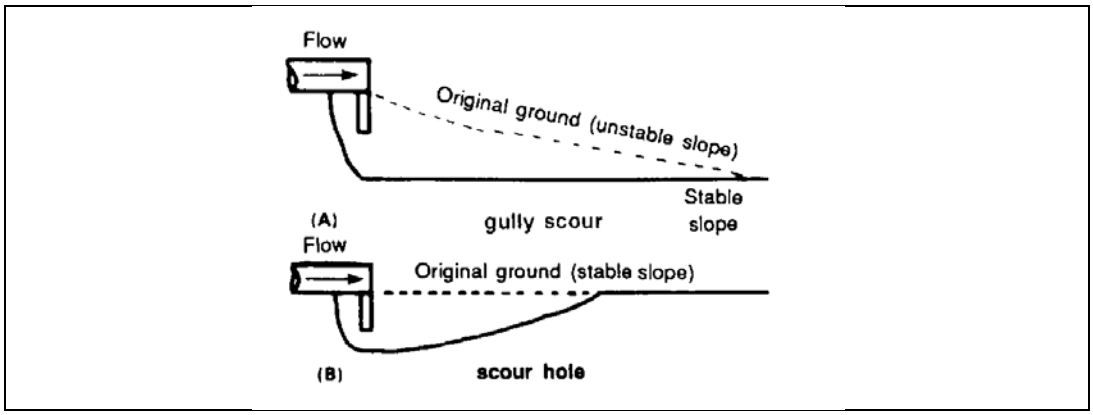


Figure 8-14. Types of scour at culvert outlets

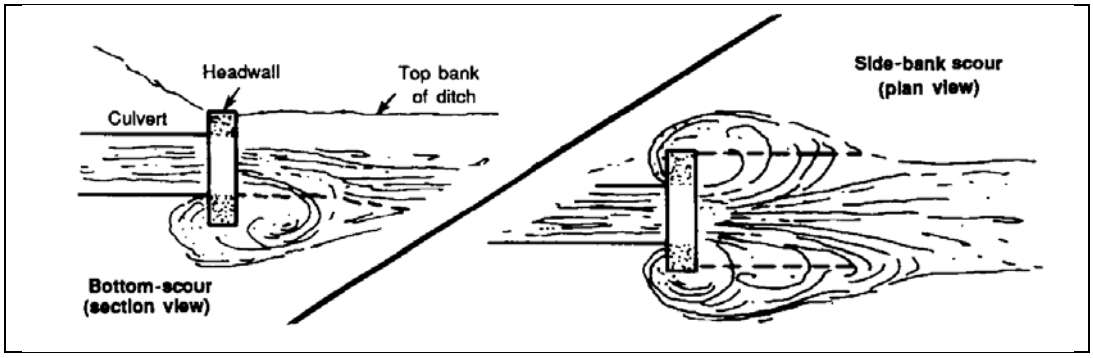


Figure 8-15. Outlet erosion

8-48. Erosion is best controlled by two methods: velocity reduction and channel protection. Reducing the slope of the culvert reduces the velocity. This solution may require a larger pipe since it changes the capacity of the culvert.

8-49. 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 discussed in chapter 7.

CULVERT DESIGN

8-50. The hydraulic load on a culvert is the amount of water that will flow to the culvert inlet, as direct-surface or channel-stream flow. Surface flow is determined by the rational method and the channel-flow method that was discussed earlier.

Hydraulics of Culverts

8-51. Culvert quantity of flow is the amount of water the culvert will carry in a unit of time. This capacity is expressed in cubic feet per second. For a particular culvert of known size, shape, and interior roughness, the discharge capacity is controlled by one or more of the following factors:

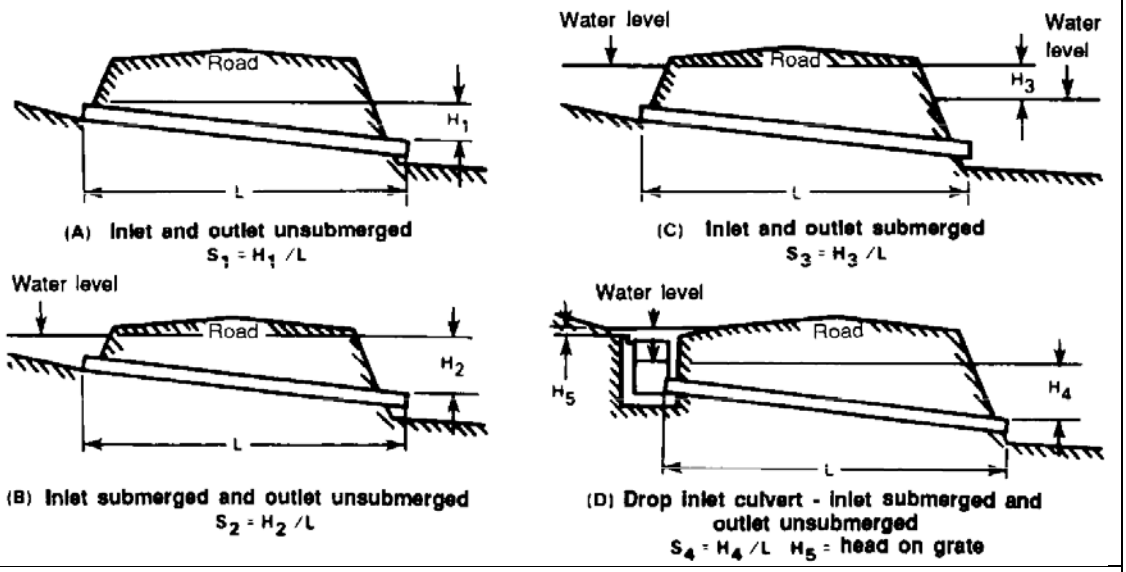
- Height of the water above the culvert inlet.
- Hydraulic gradient of the culvert.
- Length of the culvert.
- Elevation of the tailwater at the culvert outlet.

8-52. The type of inlet is generally not considered in military culvert design. However, it should be remembered that the discharge capacity of a culvert would be increased by a smooth, transition type of inlet

with headwalls and wing walls. For construction in the JOA, the culvert should have a design capacity sufficient to pass the peak runoff from the design storm.

8-53. The hydraulic gradient of a culvert is one of the culvert discharge capacity controls. It can be satisfactorily estimated as the slope in feet per foot. The gradient is calculated by dividing the head on a culvert by the culvert length. (See figure 8-16.) The head is the difference in elevation between the—

- Ends of a culvert if the inlet and the outlet are not submerged.
- Water surface directly above the inlet and the top of the outlet if the inlet is submerged and the outlet is not submerged.
- Water surface directly above the inlet and the outlet if the inlet and outlet are submerged.



Legend:

H	culvert head
L	culvert length
S	hydraulic gradient

Figure 8-16. Hydraulic gradient, S, and heads, H, for culverts

8-54. Use the following equation to calculate the hydraulic gradient:

$$S = H/L$$

where—

S = *hydraulic gradient*
H = *culvert head*
L = *culvert length*

8-55. The normal flow pattern for culverts is shown in view A of figure 8-16. In this case, the water is at crown elevation at the inlet, and the outlet is free flowing. Use table 8-7 to determine the quantity of runoff and velocity.

8-56. Ponding occurs when water at the culvert inlet accumulates. When ponding occurs, the outlet will normally be free flowing, but the water will be at some depth above the inlet. When the water depth at the inlet is 1.2 times the culvert diameter or less ($depth \leq 1.2 \times diameter$), use table 8-7 to determine the quantity of runoff and the velocity. When the water depth at the inlet is greater than 1.2 times the culvert diameter $depth \geq 1.2 \times diameter$) and the outlet is free flowing, use a nomograph (see figure 8-17, page 8-20) to P determine the quantity of runoff and the continuity equation, $Q = VA$, to determine velocity. The continuity equation is—

$$Q = VA$$

where—

A = *area*
Q = *quantity*
V = *velocity*

Table 8-7. 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																		

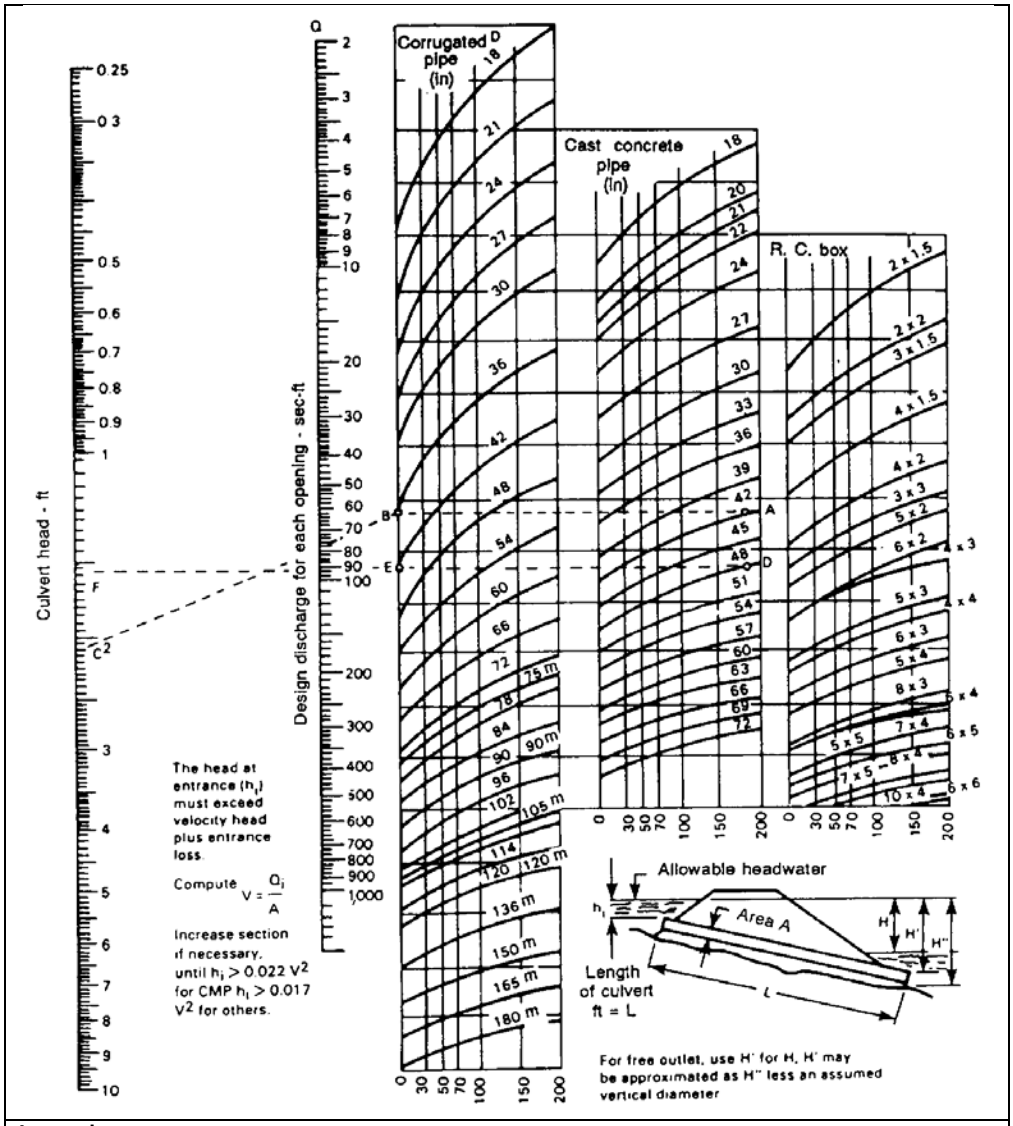
*Culverts that have a free outlet with water surface at the inlet at the same elevation as the top of the pipe and the outlet unsubmerged.

Notes.

1. Values are in cfs.
2. Heavy horizontal lines indicate critical slope.
3. Steeper slopes than *critical* do not result in an increased discharge.
4. Numbered *stepped* lines indicate approximate 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 the diameter.
7. Manning's n = 0.024 applies to CMP; standard unpaved.

Legend:

cfs	cubic feet per second
fps	feet per second
n	roughness coefficient



Legend:	
A	area
CMP	corrugated metal pipe
ft	feet
H	height
in	inches
L	length
m	meter
Q	quantity
sec	section
V	velocity

Figure 8-17. Nomograph for culverts flowing full with ponding inlets

Critical Slope

8-57. For a given culvert size and head, 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. As the hydraulic gradient is increased beyond the critical slope, culvert discharge remains constant.

8-58. The critical slope is the maximum discharge. The area decreases because the pipe does not flow full, and the velocity of flow in the culvert increases because—

$$V = Q/A$$

where—

A = *area*
Q = *quantity*
V = *velocity*

Roughness

8-59. For the selection of pipe roughness values for flow determination, refer to the notes in table 8-7, page 8-19.

Box-Culvert Flow Design

8-60. Flow characteristics of box culverts may be different from pipe culverts, even those with the same slope, lining, and inlet and outlet conditions. Flow quantity can be determined as previously noted for the conditions of inlet and outlet water elevations.

8-61. 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. Computing the cross-sectional area required for a pipe and designing a box of the same material, slope, and cross-sectional area can determine box-culvert sizes.

8-62. When the water elevation is above the top of the box inlet, use the nomograph in figure 8-17. In this case, make trial solutions until there is correlation between the box size and ponding depth.

Submerged-Inlet Pipe Culvert Design

8-63. Submerging the culvert inlet results in ponding. The elevation of the pond surface—which determines the depth of submergence—is a function of the extent of the pond, the structure safety requirement, and the time it takes to empty the pond. The runoff rate, the pond volume, and the culvert-flow rate determine these factors. This is derived using the following steps:

- **Step 1.** Determine the rate of runoff (quantity) the culvert must drain or, in the case of ponding, the inlet drain capacity.
- **Step 2.** Determine the length of the culvert.
- **Step 3.** Determine the head on the culvert.

  Step 4. Determine the size and type of pipe or box culvert required to handle the quantity of flow. (See figure 8-16, page 8-18.) To use table 8-7 enter the nomograph at the intersection of the length for the type and size of culvert (for example, Point A in the nomograph). (See figure 8-17.) Extend the line horizontally to the turning-point line (Point B in the nomograph), and extend the line to the culvert head in feet (Point C in the nomograph). Find the discharge quantity in cubic feet per second for the pipe selected at the intersection with the discharge line.
- **Step 5.** Compute the discharge velocity in feet per second. Use the following equation:

  $$V = Q/A$$

  where—

  A = *area*
  Q = *quantity*
  V = *velocity*

  Increase the pipe diameter or decrease the culvert slope if the discharge velocity is greater than the maximum permissible velocity for the outfall or if the water height above the culvert inlet is less than 0.022(V2) for CMP or 0.017(V2) for concrete pipe or boxes.

Example

(Submerged Inlet, Unsubmerged Outlet) Determine the most economical pipe size and number of pipes required for a culvert across an airfield. The outfall from the culvert is a natural drainage channel in a GP soil. The design flow is 210 cubic feet per second. There are 42-inch and 48-inch concrete pipes available.

- **Step 1.** Determine the rate of runoff (quantity) (Q = 210 cubic feet per second).
- **Step 2.** Determine the length of the culvert (L = 180 feet).
- **Step 3.** Determine the head on the culvert.

$$H\ (42\ inches) = 598.6\ feet - (593.0\ feet + 3.5\ feet) = 2.1\ feet$$

$$H(48\ inches) = 598.6\ feet - (593.0\ feet + 4.0\ feet) = 1.6\ feet$$

- **Step 4.** Determine the size and number of pipes required to handle the flow.

  (a) On the cast-concrete-pipe portion of the nomograph, draw a horizontal line from the intersection of the 180 foot length line and the 42 inch pipeline (Point A) to the turning-point line (Point B). From Point B, draw a line to a height of 2.1 feet (Point C). The intersection of this line with the *Q* portion of the nomograph shows the maximum discharge of one 42-inch pipe to be about 78 cubic feet per second. For 210 cubic feet per second, three 42-inch pipes are required.

  (b) For the 48-inch pipe, draw a horizontal line from the intersection of the 180 foot length line and the 48-inch pipe line (Point D) to the turning-point line (Point E). From Point E, draw a line to a height of 1.6 feet (Point F). One 48-inch pipe would have a capacity discharge of about 92 cubic feet per second. Three 48-inch pipes are required for 210 cubic feet per second.

- **Step 5.** Compute the discharge velocity and check for excessive outlet velocity. Check the smaller pipe.

  (a) Since three pipes are used, assume that each will carry one-third of the total quantity of flow or 70 cubic feet per second, whichever is greater.

  (b) If the pipe flows full, the exit velocity is—

$$\mathrm{V} = \frac{\mathrm{Q}}{\mathrm{A}} = \frac{78}{3.14\ (1.75)^2} = 8.1\ \text{feet per second}$$

  (c) If the exit velocity is based on a design flow of 70 cubic feet per second, the pipe would be flowing only partially full and the exit velocity would be 9.2 feet per second. 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 $H_1 > 0.017(V2)$. Use the following equation:

$$H_1 = 598.6\ feet - (594.0\ feet\ +\ 3.5\ feet) = 1.1\ feet\ and\ 0.017 \times 9.22 = 1.44$$

  (e) Since allowable outfall velocity exceeds that listed on Table 7-3 and H1 < 0.017(V2), use three 48-inch pipes as the most economical available size.

where—

Q	=	*design flow quantity*
L	=	*culver length*
H	=	*culvert head*

Unsubmerged-Inlet Pipe Culvert Design

8-64. The steps to be applied to the design of these culverts are determined as follows:

- **Step 1.** Determine the rate of runoff. Use the area that the culvert must drain. This will be the required capacity of the culvert.
- **Step 2.** Determine culvert use (road or airfield).
- **Step 3.** Calculate the critical dimensions from the cross sections. (See figure 8-18.) Determine the length in place and the fill critical.
- **Step 4.** Determine the largest pipe for the fill. Begin from the cross section at the outside edge of the shoulder. Consider available pipes and adequate cover. The maximum culvert diameter is equal to two-thirds of the minimum fill for roadway loadings. (See figure 8-18.) The cover required for culvert protection is equal to one-half the diameter of the culvert or 12 inches (whichever is greater). (See table 8-2, page 8-9, for runways and taxiways sustaining aircraft wheel loads, and see table 8-3, page 8-10, or table 8-4, page 8-11, for wheel loads and minimum cover required.)
- **Step 5.** Determine the culvert capacity and outlet velocity. Use values based on culvert material and slope. (See table 8-7, page 8-19.)
- **Step 6.** Determine the number of culvert pipes required. Divide the area runoff by the pipe capacity. Round up to the next whole number.
- **Step 7.** Determine the order length. The order length is calculated by multiplying the number of pipes times the length in place times a waste factor of 1.15. (See Step 3 for the length in place and Step 6 for the number of pipes.)
- **Step 8.** Determine the maximum permissible discharge velocity. Calculate the maximum velocity that the culvert outlet will discharge into the channel lining using table 7-3, page 7-7. Determine the correct pipe to use. Apply the following criteria to calculations:
  - Ensure that the outlet velocity is equal to or less than the maximum velocity of the discharge. If the outlet velocity exceeds the soils, the outlet must be protected against erosion.
  - Use the least number of culvert pipes possible to carry the flow and still be consistent with the above criteria.

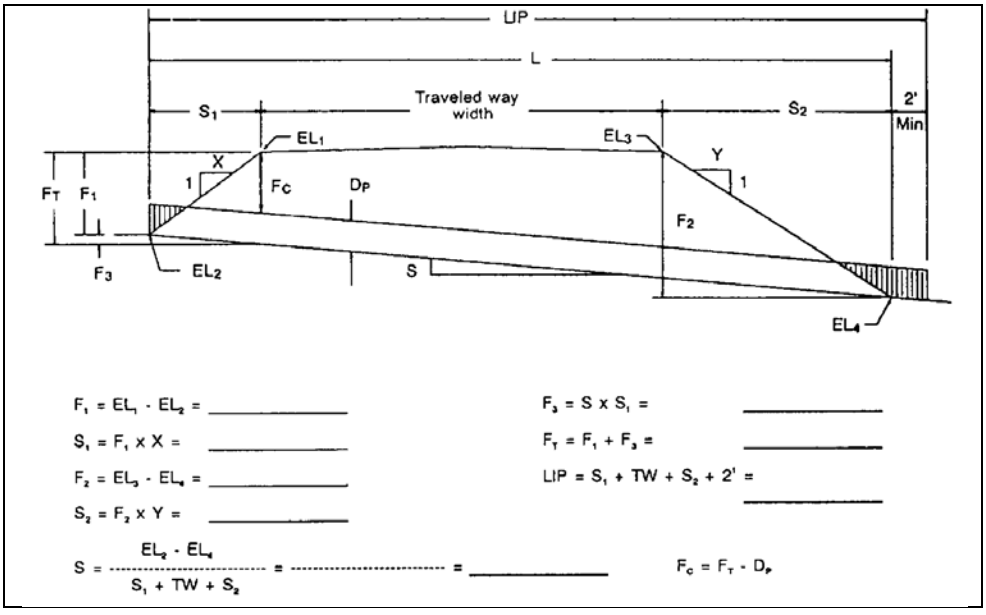


Figure 8-18. Calculating cover

- 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 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_3 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).

Legend:

D_P	pipe size
EL	elevation
ft	feet
L	horizontal length
LIP	length in place
min	minimum
TW	top width

Figure 8-18. Calculating cover (continued)

Example

(Unsubmerged Inlet) Determine the most economical pipe size and the quantity of pipe required for a culvert located under a runway. The pipe sizes available are 24-, 30-, 36-, and 42-inch CMP (10 gauge). The maximum aircraft weight class for this example will be an SR-71C aircraft. No headwall will be constructed downstream. (See table 8-2, page 8-8, and table 8-3, page 8-10.) Given—

Culvert weight type = 9

Flow rate to be handled by the culvert = 32 cubic feet per second

Soil type = bare SC

Maximum allowable outlet velocity = 3 to 4 feet per second

Before the design can be accomplished, determine the horizontal length of one culvert; the slope of the culvert pipe (to determine the flow characteristics of various pipe diameters), which is a portion of the critical fill depth; and the critical fill depth (to determine if the cover over the pipe meets the requirements of table 8-3). (See table 8-7, page 8-19, and figure 8-19, page 8-27.) Use the following steps:

- **Step 1.** Determine the runoff rate.
- **Step 2.** Determine the airfield culvert use.

Example (continued)

- **Step 3.** Calculate the critical dimensions. Computations should be made in the following sequence. (See figure 8-19, page 8-27, and figure 8-20, page 8-27.)

  (a) Determine the difference in elevation between the edge of the runway and the culvert upstream invert—

  $$F_1 = EL_1 - EL_2$$
  $$F_1 = 607.00 - 600.80 = 6.20\ feet$$

  (b) Determine the horizontal distance between the culvert invert and the shoulder edge—

  $$S_1 = F_1 \times X$$
  $$S_1 = 6.20 \times 10 = 62\ feet$$

  (c) Determine the difference in elevation between the edge of the runway and the culvert outlet and fill slope—

  $$F_2 = EL_3 - EL_4$$
  $$F_2 = 607.00 - 599.62 = 7.38\ feet$$

  (d) Determine the actual slope of the culvert—

  $$S = \frac{EL_2 - EL_4}{S_1 + TW + S_2} = \frac{600.8 - 599.62}{62 + 100 + 73.8} = \frac{1.18}{235.8} = 0.005\ or\ 0.5\%$$

  (e) Determine the incremental elevation difference between the elevation of the upstream invert and the invert of the culvert at the critical fill section—

  $$S = 0.005\ feet\ per\ foot = 0.5\%$$
  $$F_3 = S \times S_1$$
  $$F_3 = 62 \times 0.005 = 0.31\ feet$$

  (f) Determine the depth of critical fill section—

  $$F_T = F_1 + F_3$$
  $$F_T = 6.2 + 0.31 = 6.51\ feet$$

  (The length of the horizontal projection of the culvert.)

  (g) The upstream invert elevation to downstream invert elevation is—

  $$LIP = S_1 + TW + S_2 + 2\ feet$$
  $$LIP = 62 + 100 + 73.8 + 2 = 237.8\ feet\ (round\ up\ to\ 238\ feet)$$

Note. These computations depict the cross section and figure 8-20 summarizes the calculations. From these calculations, it has been determined that the actual length of the culvert is 235.8 feet, rounded up to 236 feet. A 2-foot projection is added because there will be no downstream headwall. This gives a culvert length of 238 feet for a single pipe. The culvert slope is 0.005 feet per foot or 0.5%. The depth of fill at the critical section is 6.5 feet (rounded to the nearest tenth).

- **Step 4.** Determine the largest pipe for the fill. Prepare a table as shown in table 8-8, page 8-28, and fill in known values. Start by entering the largest pipe available into the table. Subtract the pipe size from total fill to get the fill critical, which represents the actual cover over the pipe at the critical section. Compare total fill to cover required.

  To find the cover required for aircraft, refer to table 8-2, page 8-9, which indicates that an SR-71C aircraft has a culvert weight type of 9. The CMP available is 10 gauge. With this

Example (continued)

information, refer to table 8-3, page 8-10, and select chart 9, corresponding to culvert weight type 9. The diameters and cover are given in chart 9 under the 10-gauge line. Starting with the largest pipe available, 42-inch or 3.5 feet, is 4.0 feet (interpolated between pipe diameters.) Enter this value in table 8-8, page 8-28. 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 fill critical of 3.51 feet and a cover required of 3.5 feet. This pipe will work.

- **Step 5.** Determine the culvert capacity. Use table 8-7, page 8-19, to find the capacity and velocity for the 36-inch pipe. The slope of the culvert is 0.5 percent. Start at the top of the graph (pipe diameter), and move down the column to the slope intersection in the left-hand column. This will state 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 feet per second for the 0.5 percent slope. The velocity indicated is 4 feet per second.

- **Step 6.** Determine the number of pipes required to carry the flow by dividing the total flow rate 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 pipes required is 32/27.5 = 1.2 or two 36-inch diameter pipes.

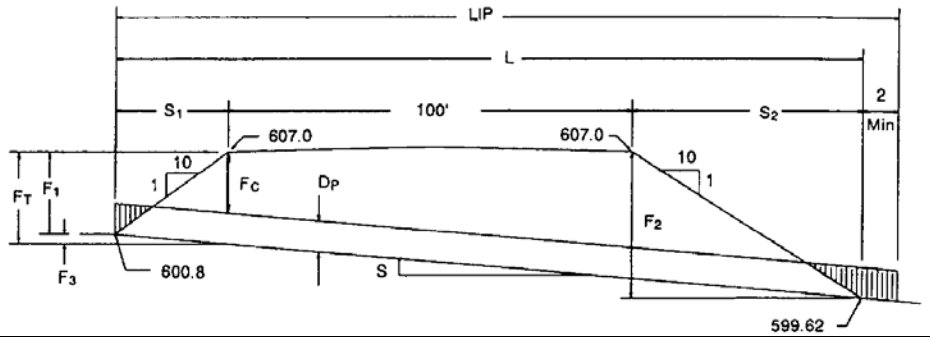
- **Step 7.** The next step is determining the length of pipe to be ordered. The order length is calculated by multiplying the number of pipes (Step 6) times the length in place (Step 3) times the waste factor. Since pieces of material will be damaged in manufacturing, handling, transporting, and assembling, an additional amount over the actual length in place 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—

$$NP \times LIP \times WF$$
$$2 \times 476 \times 1.15 = 1{,}094.8\ feet$$

The pipe comes in 2-foot increments; therefore, the value of 1,094.8 is rounded up to 1,096 feet of pipe.

where—

D_P	=	*pipe size*
EL	=	*elevation*
F	=	*fill*
F_C	=	*fill critical*
F_R	=	*fill required*
F_T	=	*total fill*
L	=	*horizontal length of one culvert*
LIP	=	*length in place*
NP	=	*number of pipes*
Q	=	*flow rate*
S	=	*slope*
TW	=	*top width*
WF	=	*waste factor*



Legend:

D_P	pipe diameter
F_C	fill critical
F_T	total fill
L	length
LIP	length in place
min	minimum
S	slope

Figure 8-19. Unsubmerged inlet culvert problem

$F_1 = EL_1 - EL_2 = 607.0' - 600.8' = 6.2'$

$S_1 = F_1 \times X = 6.2' \times 10 = 62'$

$F_2 = EL_3 - EL_4 = 607.0' - 599.62' = 7.38'$

$S_2 = F_2 \times Y = 7.38' \times 10 = 73.8'$

$F_3 = S \times S_1 = 0.005 \times 62' = .31'$

$F_T = F_1 + F_3 = 6.2' + .31' = 6.51'$

$LIP = S_1 + TW + S_2 + 2' =$
$62' + 100' + 73.8' + 2' = 237.8' \text{ or } 238'$

$$S = \frac{EL_3 - EL_4}{S_1 + TW + S_2} = \frac{600.8 - 599.62}{62 + 100 + 73.8} = \frac{1.18}{235.8} = \frac{0.005}{\text{OR}\ 0.5\%}$$

$F_C = F_T - D_P$

- 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_3 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).

Legend:

D_P	pipe diameter
EL	elevation
F	fill
F_C	fill critical
F_T	total fill
LIP	length in place
S	slope
TW	top width
X	horizontal component of the inlet side slope ratio
Y	horizontal component of the outlet slide slope ratio

Figure 8-20. Calculating cover (sample problem)

Table 8-8. Unsubmerged inlet sample data

$F_C = F_T - D_P$		
F_T	6.51	6.51
D_P	3.50	3.00
F_C	3.01	3.51
*C_R	4.00	3.50
*For aircraft, see table 8-2, page 8-8, and table 8-3, page 8-9; for roads, 2/3 F_T.		
Legend:		
C_R	cover required	
D_P	pipe diameter	
F_C	fill critical	
F_T	total fill	

PONDING AREAS

8-65. 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 JOA are designed to discharge the runoff based on the two-year design storm that, 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, areas around drainage structure inlets are designed to accommodate a certain amount of ponding.

8-66. In some cases, a system may not be able to take care of the runoff from the design storm due to limitations in culvert cover and space, terrain conditions, time, materials, or other conditions. To provide for this, include sufficient ponding areas in the original plan to prevent flooding of vital areas. The ponding areas store excess runoff until the intensity of the storm decreases and the structure can handle the flow.

8-67. When possible, military drainage systems are designed to pass the runoff from the design storm without ponding. However, some provision for ponding may be made in areas where flooding for a period of time will not affect the facility operational status.

8-68. The following specifications are generally adhered to in ponding area design 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.
- 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 a pond is used with road fills or embankments. When ponding 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 ponding is allowable will depend on the soil type and soil condition in the ponding area and the embankment. In general, this period will be no more than four hours from the start of the storm.

8-69. In designing ponding 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 due to ponding does not increase the discharge capacity of the culvert.

8-70. The determination of permissible volume, the preparation of runoff curves, and the soil analysis must be done before designing ponding areas to meet the above specifications and assumptions.

PERMISSIBLE VOLUME

8-71. The volume of permissible ponding is determined by the elevations of the area available for ponding and the surrounding areas. A contour map showing the final grading plan is required to compute the volume of permissible ponding. By inspection, a contour line may be selected to provide a ponding area that is located a safe distance or elevation from the pavement. Ponding volumes may be computed from the contour 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. Use the following equation:

$$V = \frac{A + B}{2} \times b$$

where—

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)*
V = *volume in cubic feet*

8-72. When computing the volume for ponding, consider the contours. (See figure 8-21.) 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 another 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. Use the following equation:

$$Volume\ 62 - 64 = \frac{(10{,}000 + 0)}{2} \times 2 = 10{,}000\ cubic\ feet$$

$$Volume\ 62 - 66 - \frac{(25{,}000 + 10{,}000)}{2} \times 35{,}000\ cubic\ feet$$

The total volume of the ponding area is—

$$10{,}000 + 35{,}000 = 45{,}000\ cubic\ feet$$

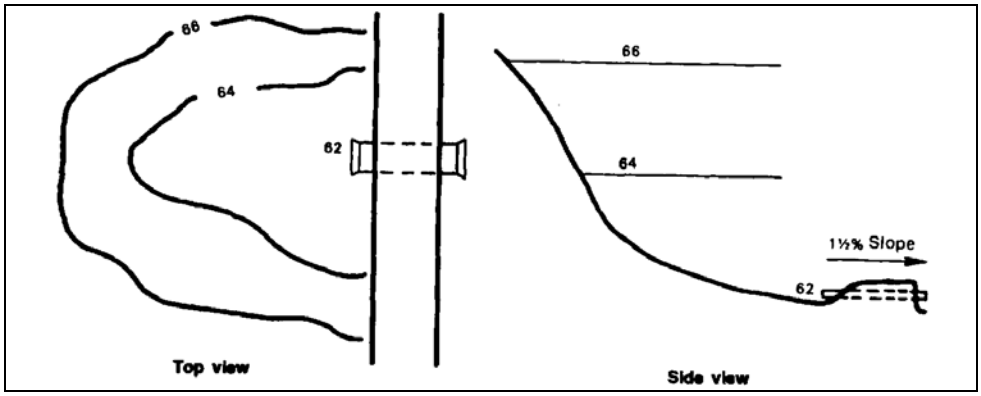


Figure 8-21. Ponding areas

8-73. A further example of the computation of ponding volumes by the average-end-area method is shown in figure 8-22, page 8-30. Assume that the ponding 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. Use the following equation:

$$Volume\ 62-64 = 10{,}000\ cubic\ feet$$
$$Volume\ 62-66 = 35{,}000\ cubic\ feet$$
$$Volume\ 66-68 = \frac{(30{,}000 + 25{,}000)}{2} \times 2 = 55{,}000\ cubic\ feet$$

If the ponding area extends to the 68-foot contour line, the total volume available for ponding will be—

$$10{,}000 + 35{,}000 + 55{,}000 = 100{,}000\ cubic\ feet$$

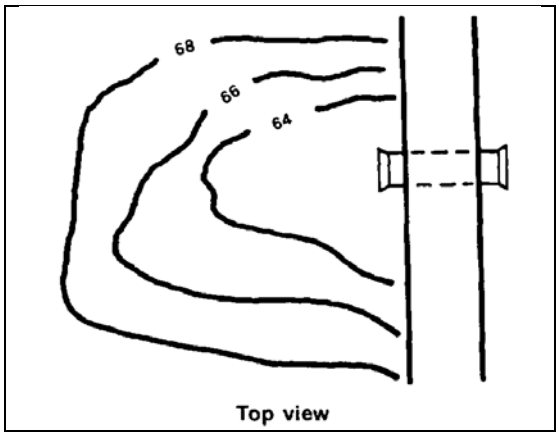


Figure 8-22. Ponding area enlarged

RUNOFF CURVES

8-74. To determine the amount of water that an area will contribute to a pond, a cumulative runoff curve must be plotted. For 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 two-year frequency. Prepare a cumulative runoff curve based on the data given. Ponding may be required to reduce the culvert size.

Tabulation of Data—Rational Method

8-75. The data required to plot a cumulative runoff curve using the rational method is shown in table 8-9. Each column is prepared as follows:

- Column 1 is a tabulation of time in minutes. A similar combination of time increments can be used if 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 (as determined by 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 6-10. The value of 2.2 is the location intensity, in inches per hour, of the 1-hour, two-year storm as specified in the given conditions.
- Column 4 is the rate of runoff in cubic feet per second for the entire interval of time shown in column 1. It is obtained by multiplying columns 2 and 3.
- 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.

Example

If water was not released from the pond after 20 minutes from the start of rainfall, the volume of water would be—

105 x 1,200 or 126,000 cubic feet

Table 8-9. Cumulative runoff data tabulation, rational method

1	*2*	*3*	*4*	*5*	*6*
Time (minutes)	***C x area (0.75 x area)***	***Intensity (inches/hour)***	***Q (cfs)***	***Time (seconds)***	***Volume (cu ft)***
5	—	6.5	163.0	300	48,900
10	—	5.5	138.0	600	82,800
15	—	4.6	115.0	900	103,500
20	—	4.2	105.0	1,200	126,000
30	—	3.4	85.0	1,800	153,000
40	—	2.9	72.5	2,400	174,000
50	—	2.5	62.5	3,000	187,000
60	—	2.2	55.0	3,600	198,000
120	—	1.3	32.5	7,200	234,000
180	—	1.0	25.0	10,800	270,000

Legend:

C	runoff coefficient
cfs	cubic feet per second
cu	cubic
ft	feet
Q	rate of runoff

Preparation of Cumulative Runoff Curve

8-76. The cumulative runoff curve is obtained by plotting the volume (column 6, table 8-9) obtained by the rational method on the vertical axis and the time in minutes (column 1, table 8-9) on the horizontal axis. The cumulative runoff curve for the data listed in table 8-9 is shown in figure 8-23, page 8-32.

Analysis of Cumulative Runoff Curve

8-77. Based on data given in the problem, the flow for the culvert design shown in figure 8-22, page 8-29, would be determined with the rational method equation. Given—

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, two-year storm*

A = *33.3 acres*

$$Q = CIA$$

$$Q = 0.75 \times 4.9 \times 33.3 = 122.4 \; cubic \; feet \; per \; second$$

where—

Q = *quantity of runoff (cubic feet per second)*

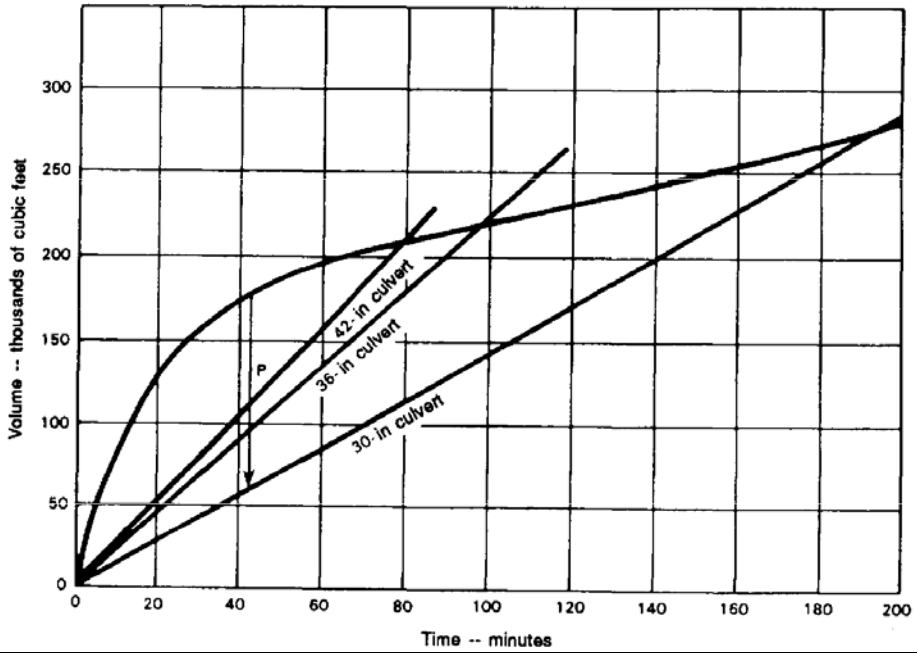
C = *runoff coefficient*

I = *rainfall intensity (inches per hour)*
A = *drainage area (acres)*

8-78. The number of culverts at a slope of 1.2 percent when the quantity of runoff equals 122.4 would be—

- A 30-inch pipe at 24 cubic feet per second = 6 pipes.
- A 36-inch pipe at 38 cubic feet per second = 4 pipes.
- A 42-inch pipe at 57 cubic feet per second = 3 pipes.
- A 48-inch pipe at 80 cubic feet per second = 2 pipes.
- A 60-inch pipe at 140 cubic feet per second = 1 pipe.

Pipes in these sizes and quantities will pass the flow without ponding.



Legend:
in inch

Figure 8-23. Cumulative runoff curve

ANALYSIS FOR PONDING

8-79. The safe volume for ponding is 100,000 cubic feet with a 4-hour limit on ponding time. Reduce the flow past the outlet point by using a single culvert and allowing ponding at the inlet.

8-80. A 30-inch CMP culvert (roughness coefficient = 0.024) on a 1.2-percent slope discharges 24 cubic feet per second. (See table 8-7, page 8-19.) Assuming that the pipe always discharges at the rated capacity, the cumulative discharge is a straight-line function for a time interval with a line passing through the origin.

8-81. Continue the straight lines so that it intersects the cumulative runoff curve. At the point of intersection, the cumulative runoff will equal the cumulative amount of water passed by the culvert. From then on, there will be no ponding. Determine if the ponding area is large enough, based on how long the pond will exist behind the inlet. The greatest vertical distance between the cumulative runoff curve and the 30-inch cumulative discharge volume is line P. (See figure 8-23.) Line P represents the maximum volume of water ponded behind the 30-inch culvert.

8-82. Measuring line P by the vertical scale indicates that the maximum ponding volume will be 115,000 cubic feet. At 42 minutes, the cumulative supply curve shows that 175,000 cubic feet of water was supplied to the pond by the rainstorm. At the end of 42 minutes, the 30-inch culvert has theoretically discharged 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.

8-83. Since the safe ponding volume is 100,000 cubic feet, a 30-inch CMP is unsatisfactory. Calculating for a 36-inch culvert, the pond volume will be 75,000 cubic feet with a pond time of 95 minutes. Since the safe ponding volume of 100,000 cubic feet and the 4-hour limit on the ponding time are 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

8-84. Although ponding is an added safeguard against the effects of storms more severe than the design storm, its primary use is as an economic measure. Ponding allows reductions in the culvert pipe size necessary to handle runoff. However, if additional pipe or other construction is required for ponding, 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 a longer TOC, ponding has little or no effect on pipe sizes. The area available for ponding often restricts the use of ponding as an economical option. 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 ponding should be coupled with initial grading operations, if possible, to secure the most efficient use of personnel and equipment.

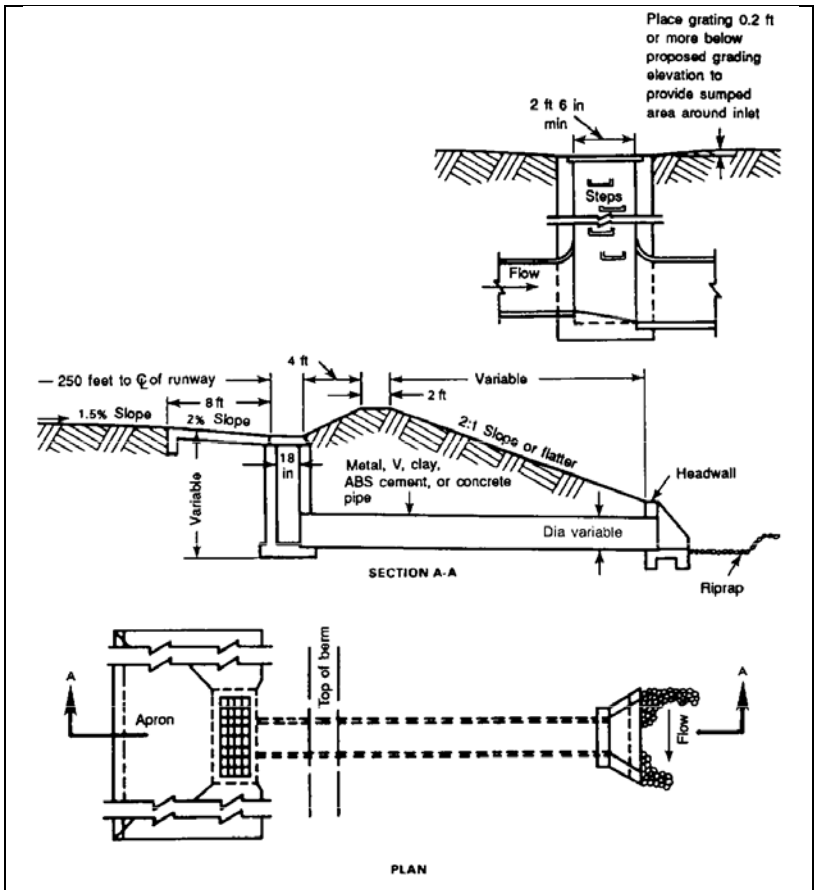
DROP INLETS AND GRATINGS

8-85. Drop inlets are vertical entrances to a culvert or a storm drain. They may be used to lower the elevation of the culvert inlet below ditch elevation. This is done when fill does not provide sufficient cover or when discharge velocity is erosive and can only be controlled 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. (See figure 8-24, page 8-34.)

8-86. A framework of bars, or a perforated plate called a grating, passes the storm runoff into a drop inlet. The grating serves as a filter and a cover for the inlet. Gratings are shown in figure 8-25, page 8-34.

CONSTRUCTION OF DROP INLETS AND GRATINGS

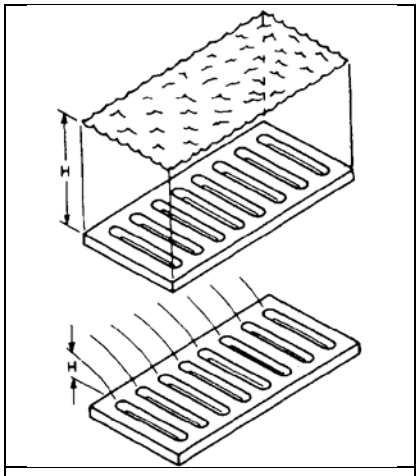
8-87. Drop inlets should always be protected with grating. An expedient grating can easily be fabricated using reinforcement bars welded together. These gratings should be spaced to readily admit debris that will pass unobstructed through the culvert. A drop inlet may be constructed of concrete, brick, 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.



Legend:

ABS	acrylonitrile-butadiene-styrene
℄	centerline
dia	diameter
ft	feet
in	inch
min	minimum

Figure 8-24. Drop inlets



Legend:

H	height

Figure 8-25. Grating heads

8-88. 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.

8-89. To determine the proper size of grating—

- Determine the peak rate of runoff from the area that drains into the drop inlet. This will be the required discharge capacity of the grating.
- 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 ponding area, the head will be the depth of water running in the ditch or gutter or the depth of the pond.
- Select the minimum size grating required with the required head and design discharge capacity values. (See table 8-10.)
- Multiply the required grating size by the appropriate safety factor to determine the actual grating size to use.

Table 8-10. 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 the left of heavy line were calculated from the weir formula:

$$Q = 3LH\frac{1}{2}$$

where—

L = *perimeter*
H = *head*
Q = *flow rate*

3. Values to the right of heavy line were calculated from the orifice formula:

$$Q = 5.37AH\frac{1}{2}$$

where—

A = *grate opening*
H = *head*
Q = *flow rate*

4. The clear opening between the grate bars was taken to be 50% of the total grate area.
5. The grate size should be increased 50% in paved areas.
6. The grate size should be increased 100% in turfed areas.

Legend:

ft	feet
in	inches
sq	square

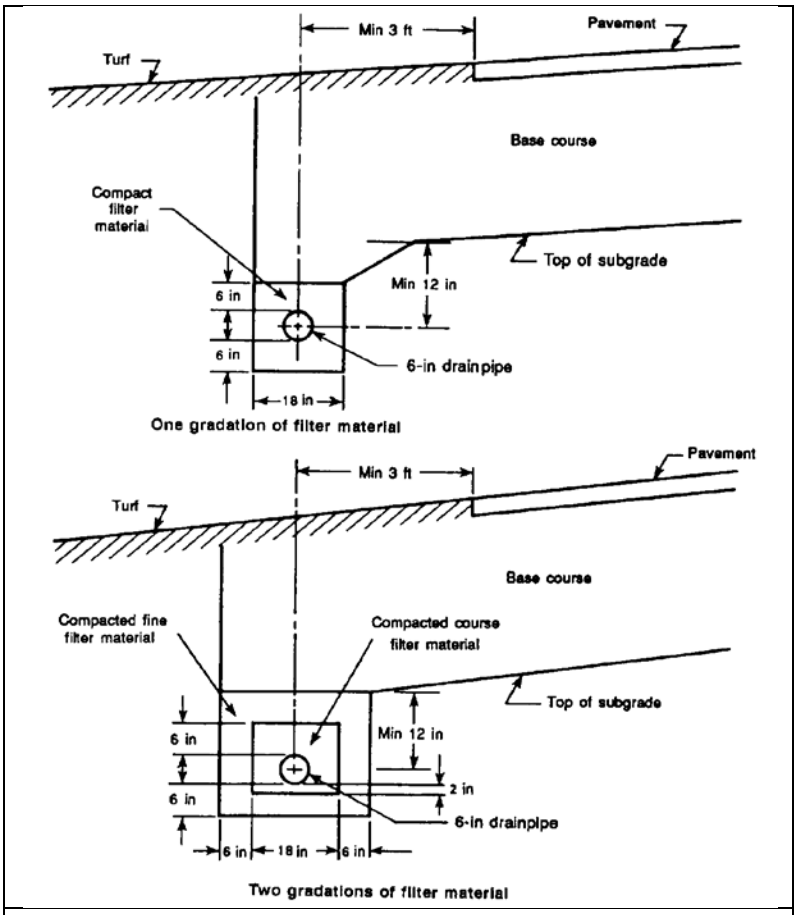
8-90. A safety factor of 50 percent (1.5 times total grating area) for paved areas and 100 percent (2 times total grating area) for turfed areas should be added to the grating-area measurement to compensate for debris caught in the openings.

8-91. A grating opening should be at least 18 inches long and placed parallel to the direction of flow. The grating opening is estimated at 50 percent of the total grating area. When using bars with large openings, determine the area of the grating opening.

MAINTENANCE OF DROP INLETS AND GRATINGS

8-92. 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 consists of subgrade saturation by the pond or direct flooding of adjacent areas. Periodic maintenance of drop inlets should include removing the cover and inspecting and cleaning the chamber.

8-93. Expedient drop inlets must be covered with bars. (See figure 8-26.) If the box is left open, it will fill with debris and make cleaning difficult, especially if a pond has developed.



Legend:	
ft	feet
in	inches
min	minimum

Figure 8-26. Typical base drainage installation

SUBSURFACE DRAINAGE

8-94. 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.

CRITERIA

8-95. Generally, subsurface drainage may be divided into three classes. These classes are base drainage, subgrade drainage, and intercepting drainage.

Base Drainage

8-96. Base drainage generally consists of subsurface drainpipes laid parallel and adjacent to pavement edges with pervious material joining the base and the drain. (See figure 8-26.) 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 ponding of surface runoff. If pavement becomes temporarily inundated and there is little possibility that the water will drain from the base into the subgrade, base drainage is required. Table 8-11 establishes the criteria for this situation.

Table 8-11. Base drainage criteria for K

Depth of ground water (feet)	***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 soil with $K = 10^{-8}$.

Legend:

fpm feet per minute

8-97. 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 times 10^{-3} feet per minute. The coefficient of permeability, a property of each soil type, is defined as the discharge velocity at a unit hydraulic gradient. The coefficient of permeability is determined experimentally with a laboratory test or an actual field test of the soil involved. The coefficient is expressed in units of velocity, such as feet per minute or centimeters per second. Base drainage is required if the subgrade coefficient of permeability is smaller than the coefficient of permeability. (See table 8-12.)

Table 8-12. Drainage characteristics of soil

	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}**
Drainage properties	Good drainage						Poor drainage			Practically impervious		
	Drains very rapidly				Drains rapidly		Drains slowly		Drains very slowly	Drainage imperceptible		
Soil classification	GW-SW				GP-SP		ML-OL GC-SC-CL GF-SF-MH			CL-CH-OH		
Types of soil	Clean gravel		Clean sand, clean sand and gravel mixtures		Very fine sands; organic and inorganic silts; mixtures of sand, silt, and clay; glacial till; stratified clay deposits					Impervious soils, homogeneous clays below the zone of weathering		
	Impervious soils which are modified by the effects of vegetation and weathering											

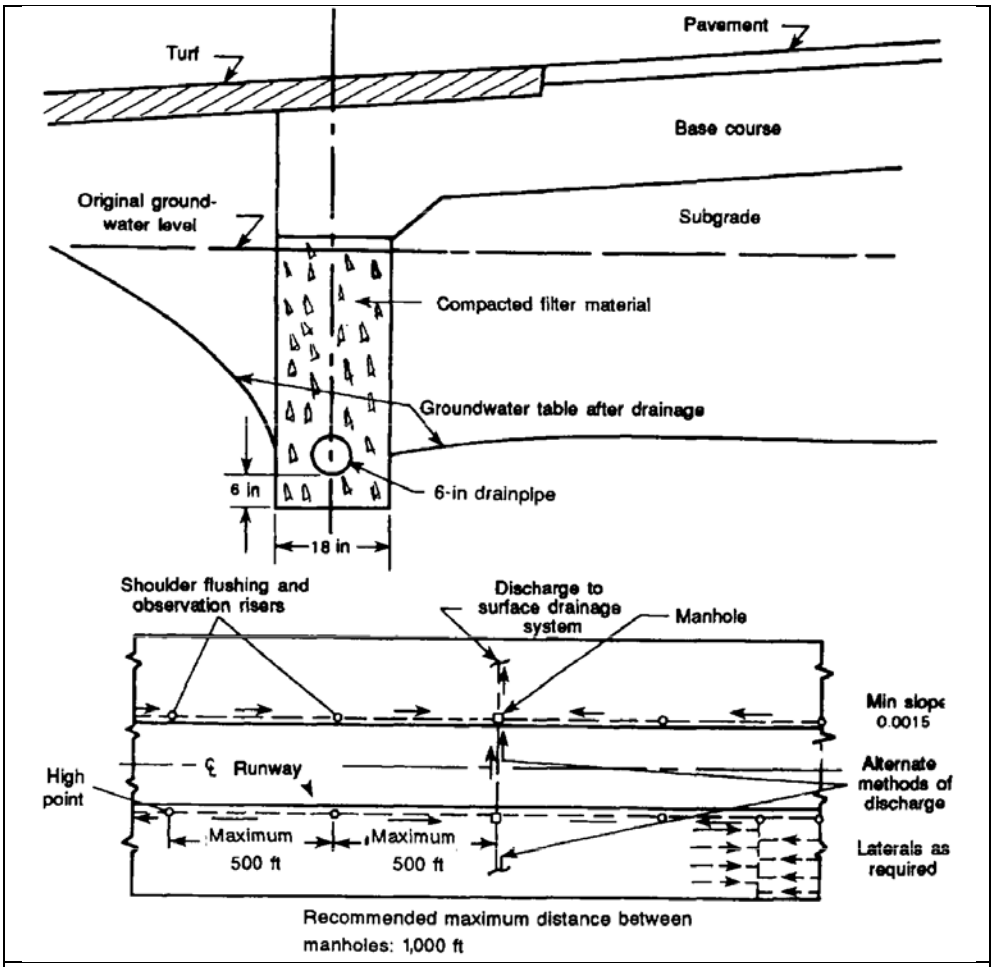
Legend:

cm centimeter
ft foot/feet
min minute
sec second

Subgrade Drainage

8-98. Subgrade drainage is required for permanent construction when seasonal fluctuations of groundwater may be expected to rise to less than 1 foot below the bottom of the course. See figure 8-27, age 8-38, for

typical subgrade drainage installation and figure 8-28 for types of subsurface drainage. These drains, although similar to base drains, have a larger area of filter material in contact with the subgrade.



Legend:	
℄	centerline
ft	feet
in	inches
min	minimum

Figure 8-27. Typical subgrade drainage installation

Intercepting Drainage

8-99. 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 and pervious soil layers, in exposed rock cuts, or in seepage from springs. A typical intercepting drainage section is shown in figure 8-29, page 8-40.

TECHNIQUES

8-100. 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.

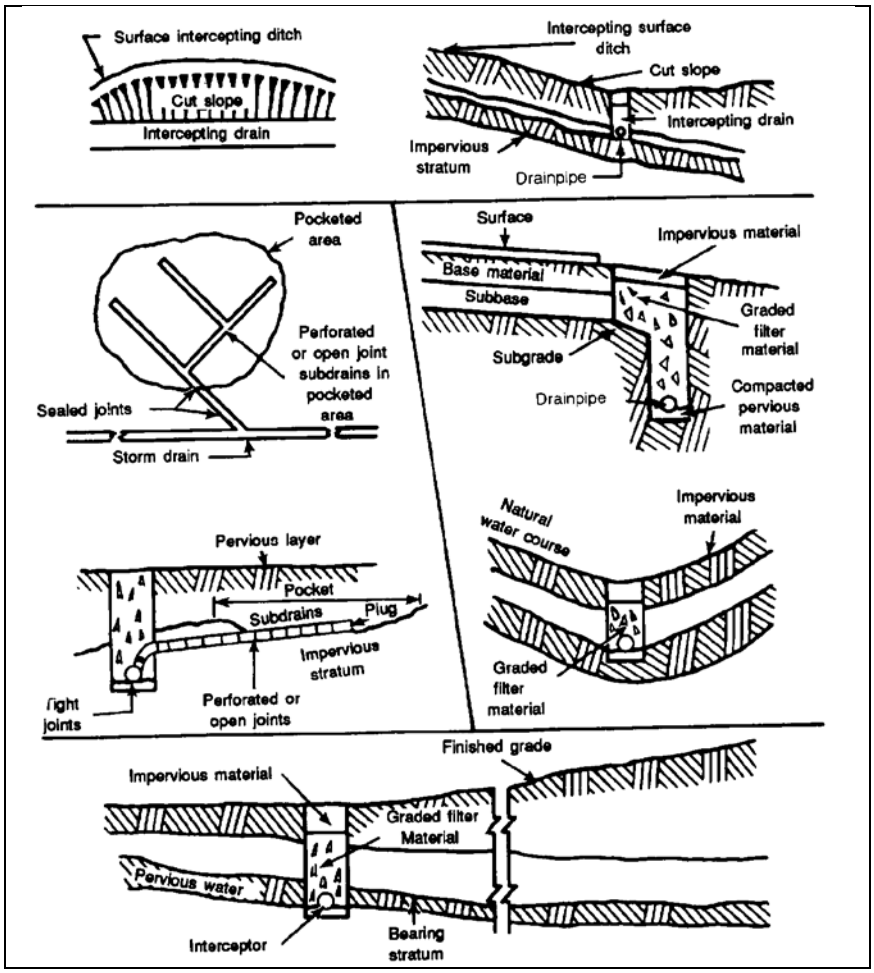


Figure 8-28. Types of subsurface drainage

Depth of Base Course

8-101. 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—

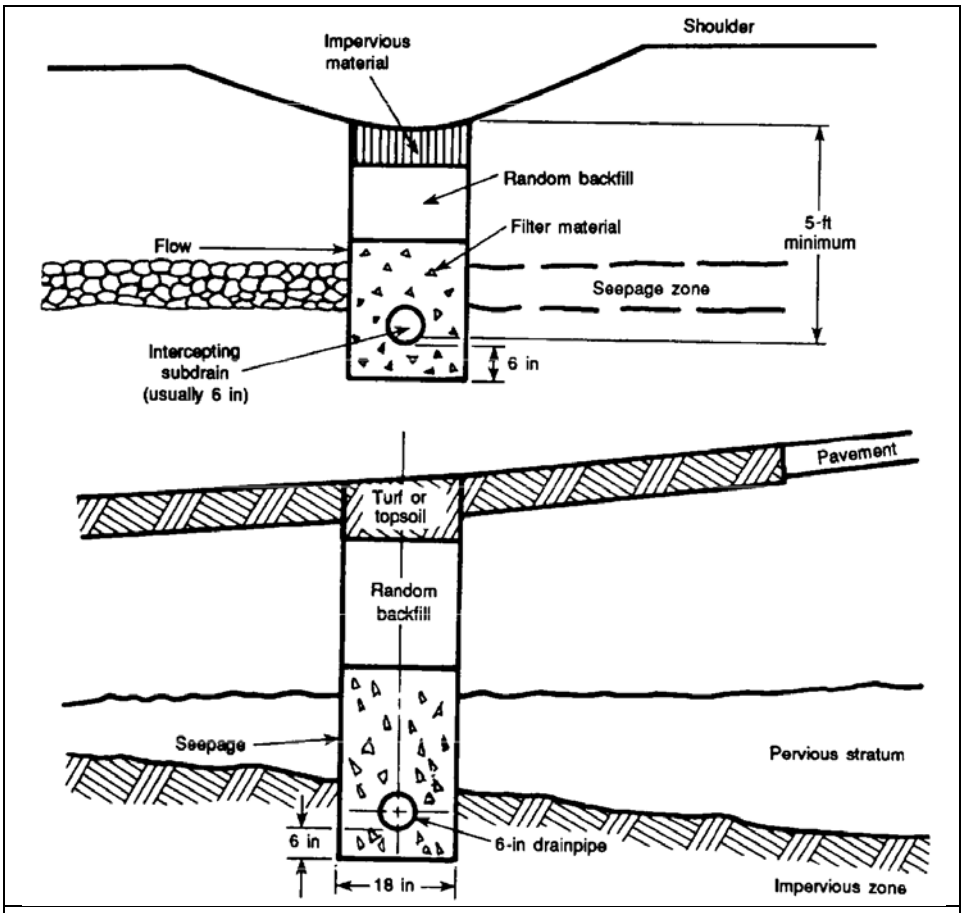
- A gravity drainage system is impractical.
- The condition to be controlled is limited to a small area, such as a narrow swamp crossing.
- Adequate base-course material is available.

Deep Ditch

8-102. 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 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

8-103. 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.



Legend:

ft	feet
in	inches

Figure 8-29. Typical intercepting drainage installations

Blind Drains

8-104. French (or blind) 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 acts 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. French drains are not recommended for permanent construction because they tend to silt up over time. In JOA 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

8-105. In cases where a V-type or other open-ditch type drainage system is not practical, construction of subsurface drainage may be necessary. Failure of the subsurface system stemming from improper control of the grade, bedding, pipe placement, placement of filter material, or other installation work gives no warning before failure. Such failure is extremely difficult to repair.

8-106. 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 are corrugated metal, cast iron, vitrified clay, nonreinforced concrete, bituminized fiber, and asbestos cement.

8-107. Bell and spigot pipes can be laid with open joints. 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.

8-108. Porous concrete pipe is laid with closed joints. It collects water by seepage through the wall of the pipe, and it should not be used where sulfated waters may cause disintegration of the concrete.

8-109. Farm tile is laid with butt joints slightly separated to permit water collection 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

8-110. Combination drains, which attempt to handle both surface runoff and subsurface water in the same pipe system, are not 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 generally sealed so that surface runoff cannot enter. The only drainage system that will satisfactorily handle surface runoff and subsurface water is the open channel or ditch.

PIPE-LAYING CRITERIA

8-111. There are essentially four different types of pipe available for subsurface drainage, as previously mentioned. They should be laid according to the following specifications:

- The minimum slope for a subdrainage pipe is a 0.15-foot drop in elevation per 100 feet of length. The pipe elevation at a particular location is generally specified by the invert elevation in which the invert is defined as the lowest point in the internal pipe cross section at the location.
- Manholes should be provided at intervals of not more than 1,000 feet with flushing risers between manholes and at dead ends. (See figure 8-27, page 8-38.)
- The pipe should be at least 6 inches in diameter with 6-inch pipe used for drains. Long intercepting lines or extremely severe groundwater conditions may require 8-inch or larger pipe.
- 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 a considerably larger pipe, it should discharge above the water level into the larger pipe. When the drain discharges into a pipe of equal or only slightly larger size, it is generally better to bring the drain in above the receiving drain and make a vertical connection between the two. This prevents the water from backing up in the drainage pipe since these pipes rarely flow full.
- When the impervious layer is at a reasonable depth, intercepting drains should be placed in the impervious layer below the intercepted seepage stratum. The quantity of water collected by an intercepting drain is difficult to determine, but 6-inch pipe is generally sufficient for lengths up to 1,000 feet.

VERTICAL WELLS

8-112. Vertical wells are 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 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.

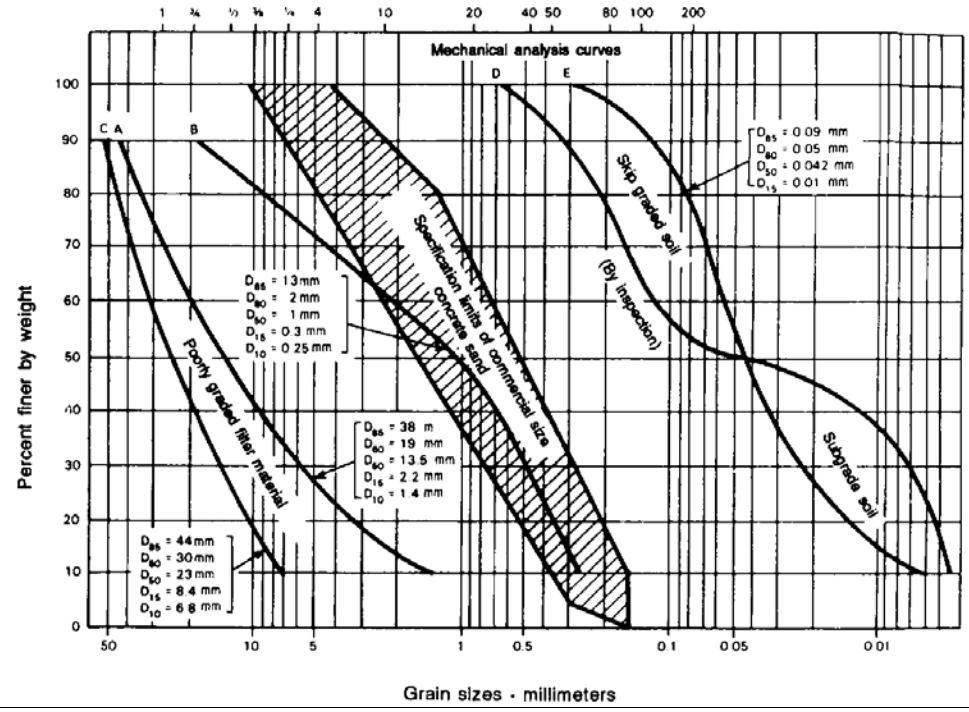
FILTER MATERIAL

8-113. A layer of filter material approximately 6 inches deep should be placed around 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 the following 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.

8-114. Criteria have been developed, based on the mechanical-analysis soil curve, to prevent these 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 8-30 show various gradation characteristics.



Legend:	
D	diameter
M	meters
mm	millimeters
U.S.	United States

Figure 8-30. Mechanical analysis curves for filter material

8-115. A coefficient of uniformity value 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.

8-116. Filter material can clog a pipe by moving through the perforations or openings. Prevent this by using the following specifications:

- For a slotted opening—

$$\frac{85\%\ size\ of\ filter\ material}{slot\ width} > 1.2$$

- For a circular hole—

$$\frac{85\%\ size\ of\ filter\ material}{hole\ diameter} > 1.0$$

8-117. Use the following equations to prevent particles from the protected soil from moving into or through the filter or filters:

$$\frac{15\%\ size\ of\ filter\ material}{85\%\ size\ of\ protected\ soil} \leq 5$$

$$\frac{50\%\ size\ of\ filter\ material}{50\%\ size\ of\ protected\ soil} \leq 25$$

8-118. 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—

$$\frac{15\%\ size\ of\ filter\ material}{15\%\ size\ of\ protected\ soil} \geq 5$$

8-119. If it is not possible to secure a mechanical analysis of available filter materials and protected soil, concrete sand with mechanical-analysis limits may be used. (See figure 8-30.) Well-graded concrete sand is satisfactory as a filter material in most sandy or silty soils.

STEPS IN FILTER DESIGN

8-120. 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 that will be in contact with the filter material. Plot the mechanical analysis of each soil on semilogarithm paper and draw a curve for each sample.

8-121. From the mechanical-analysis curve, determine the diameter (in millimeters) of the particle for the samples. The subscript *10* in D_{10} indicates that 10 percent of the particles are finer (by weight). Similarly, determine the D_{15}, D_{50}, D_{60}, and D_{85} particle size for each soil.

8-122. Determine the slot width or hole diameter, in millimeters, of the type of pipe to be used. Check the following:

- The coefficient of uniformity to ensure that—

$$C_U = \frac{D_{60}}{D_{10}} \leq 20$$

- The design criteria for clogging of pipe openings to ensure that—

$$\frac{D_{35}}{slot\ width} > 1.2\ or$$

$$\frac{D_{35}}{hole\ diameter} > 1.0$$

- The design criteria for movement of particles from the protected soil (subgrade) through the filter material to ensure that—

$$\frac{D_{15}\ of\ filter\ material}{D_{85}\ of\ protected\ soil} \leq 5\ or$$

$$\frac{D_{50}\ of\ filter\ material}{D_{50}\ of\ protected\ soil} \leq 25$$

- The design criteria for relative perviousness to ensure that—

$$\frac{D_{15}\ of\ filter\ material}{D_{15}\ of\ protected\ soil} \geq 5$$

where—

C_U = *coefficient of uniformity*

D = *diameter*

8-123. Select the filter material that best meets the above criteria. Install the pipe to grade, bedded and surrounded with the selected filter material as shown in figure 8-26, page 8-36; figure 8-27, page 8-38; figure 8-28, page 8-38; and figure 8-29, page 8-40.

Selecting Filter Materials

8-124. Filter material should be selected with a view toward the simplest construction and the lowest cost. To further this end, try to use 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. (See figure 8-26.) 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. (See figure 8-30, page 8-42.) The soils represented by curve A and curve B are readily available from local borrow pits.

Tabulate data from the mechanical-analysis curves. (See table 8-13.)

Check the coefficient of uniformity of the soils—

Soil A:

$$C_U = \frac{D_{60}}{D_{10}} = \frac{19.0}{1.4} = 13.6$$

Soil B:

$$C_U = \frac{D_{60}}{D_{10}} = \frac{2.0}{0.25} = 8.0$$

Soil A and soil B satisfy the requirement that the coefficient of uniformity be less than 20.
Apply design criteria to soil A—

$$\frac{D_{15}\ (filter)}{D_{85}\ (protected\ soil)} = \frac{2.2}{0.09} = 24.4$$

It should be ≤ 5 to prevent movement of subgrade soils through the filter. Soil A is unsuitable because movement of the subgrade soil through the filter material is possible.

Example (continued)

Apply design criteria to soil B—

$$\frac{D_{15}\ (filter)}{D_{85}\ (protected\ soil)} = \frac{0.30}{0.09} = 3.33$$

It should be ≤ 5 to prevent movement of subgrade soils through the filter.

$$\frac{D_{50}\ (filter)}{D_{50}\ (protected\ soil)} = \frac{1.0}{0.042} = 23.8$$

It should be ≤ 25 to prevent movement of subgrade soils through the filter.

$$\frac{D_{15}\ (filter)}{D_{15}\ (protected\ soil)} = \frac{0.30}{0.01} = 30$$

It should be ≥ 5 to permit water movement through the filter.

$$\frac{D_{85}\ (filter)}{hole\ diameter} = \frac{13.0\ millimeter}{1/4 \times 25.4} = \frac{13}{6.35} = 2.0$$

where—
C_U = *coefficient of uniformity*
D = *diameter*

It should be > 1.0 to prevent clogging of the pipe. The soil particle size is usually given in millimeters, and 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, soil B is > 1.0. Thus, soil B satisfies the criteria for a good filter material, while soil A does not.

Table 8-13. Tabulated data

Soil A	*Soil B*	*Subgrade (protected soil)*
D_{10} = 1.4 mm	D_{10} = 0.25 mm	
D_{15} = 2.2 mm	D_{15} = 0.3 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
Legend:		
mm	millimeter	

Geotextiles as a Filter

8-125. Geotextile fabrics may be used in place of a graded granular filter. To use a geotextile as a filter, line the trench with a geotextile filter fabric following the recommendations of the fabric manufacturer. The filter fabric should be placed to separate the permeable backfill of the trench from the subgrade or subbase materials, but it must not impede the flow of water from the drainage layer to the drainpipe. The filter fabric must also protect from the infiltration of fines from surface layers. This is particularly important for drains placed outside the pavement area where surface water can enter the drain through a soil surface. The filter fabric for the trench will be a nonwoven needle punch fabric. (See table 8-14, page 8-46.)

Table 8-14. Filter fabric criteria

Soil or Fabric Characteristic	*ASTM Method Used*	*Criteria*
Soil with 50% or less passing No. 200 Sieve	Standard test method for determining apparent opening size of a geotextile.	AOS < 0.6 mm (Sieve No. 30)
Soil with greater than 50% passing No. 200 Sieve		AOS < 0.297 mm (Sieve No. 50)
Minimum grab strength in kN (lb) at 50% elongation	Standard test method for grab breaking load and elongation of geotextiles.	0.6 (130)
Minimum puncture strength in kN (lb)	Standard test method for index puncture resistance of geomembranes and related products.	0.25 (55)

Legend:

AOS	apparent opening size
ASTM	American Society for Testing and Materials
kN	kilonewton
lb	pounds
mm	millimeters
No.	number

INSTALLATION OF A SUBDRAINAGE SYSTEM

8-126. The most efficient and practical type of subdrainage system is one that adequately performs the operations for which it was intended and is installed with the care and skill consistent with its purpose. Lowering the quality of construction or using a sketchy or inadequate subdrainage system can result in disastrous failures. Conversely, installing an elaborate system of underground piping when a simple V ditch would serve as well is inadvisable.

Chapter 9

Surface Drainage Design in Arctic and Subarctic Regions

This chapter discusses the problems involved in the design of drainage facilities in arctic and subarctic regions. While the design data presented has been developed primarily for Alaska, the methods used are generally applicable to other arctic and subarctic regions.

ARCTIC AND SUBARCTIC REGIONS

9-1. Arctic is defined as the northern region. The mean temperature for the warmest month is 50 degrees Fahrenheit or less, and the mean annual temperature is below 32 degrees Fahrenheit. In general, the arctic coincides with the tundra region north of the limits of trees.

9-2. Subarctic is defined as the region adjacent to the arctic. The mean temperature is 32 degrees Fahrenheit or below for the coldest month, 50 degrees Fahrenheit or above for the warmest month, and there are less than four months with a mean temperature above 50 degrees Fahrenheit. In general, the subarctic coincides with the circumpolar belt of dominant coniferous forests.

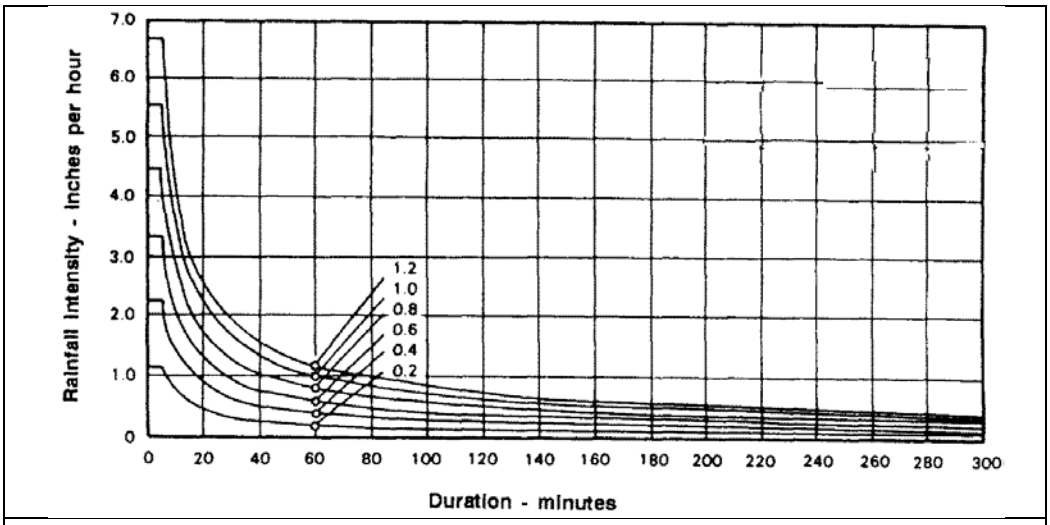
DESIGN CRITERIA FOR COLD CLIMATES

9-3. In the following section, hydraulic and hydrologic design criteria specific to cold climates are presented and discussed.

RAINFALL

9-4. 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 less than one hour. This is evident when compared with similar rainfall in the continental United States. Even within the area of Alaska, there was a noticeable difference between the rains at Juneau and Fairbanks. The higher values for rainfall intensity were used to develop design intensity-duration (supply) curves. (See figure 9-1, page 9-2.) For design purposes, a minimum rainfall rate of 0.2 inches per hour is recommended, even maps with an intensity-frequency design storm index. One-hour rainfall intensities having various average frequencies of occurrence in the arctic and subarctic regions of Alaska are shown on the maps in figure 9-2, page 9-2. 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 8-2, page 8-4. Figure 9-1 and figure 9-2 provides sufficiently accurate rainfall intensities so that runoff computations for any duration may be readily determined.

9-5. 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 a short record of rainfall at a minimum. 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 gauge might have a maximum hourly value of 0.60 inch one summer, and the same storm might produce the summer 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 two-year, 1-hour value of the long-record station to get the estimated two-year, 1-hour value for the short-record station. If the long-record station has a two-year, 1-hour value for the construction site, it would be 6/4 of 0.5 or 0.75. Usually, arrangements can be made to borrow a rain gauge for temporary use from the meteorological agency in the area in which the project is located.



Note. Curve numbers correspond to the 1-hour values of supply in the design storm index. (See figure 9-2.)

Figure 9-1. Intensity-duration curves, arctic and subarctic

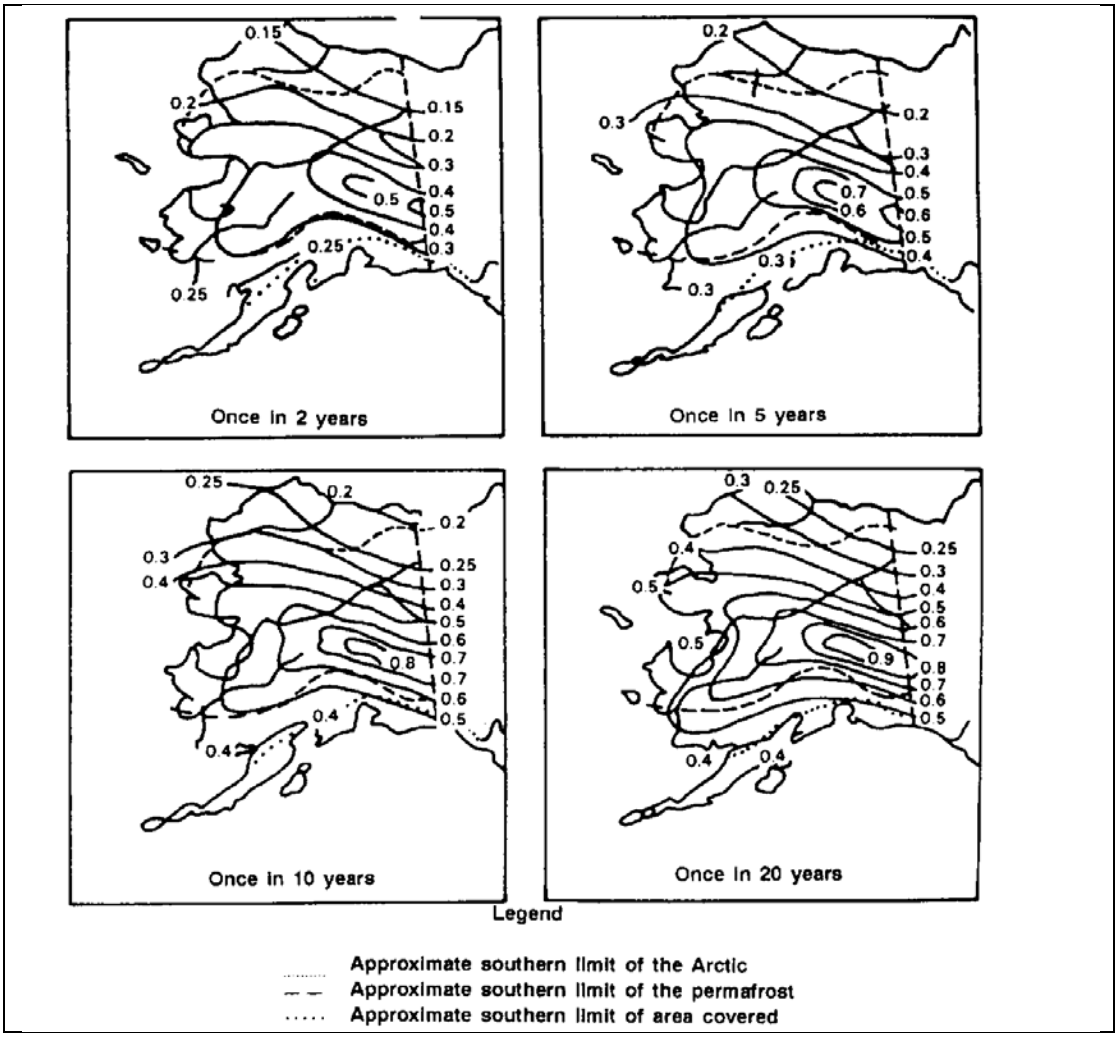


Figure 9-2. Design storm index for Alaska

Notes.

1. Rainfall observation at National Weather Service stations provides this basic data. 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 9-1.

4. The once-in-50-years amount can be estimated by multiplying the once-in-2-years amount by 2.2.

Figure 9-2. Design storm index for Alaska (continued)

INFILTRATION

9-6. 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. Normally, values would not exceed about 0.5 inches per hour for coarse sands and gravels and would be as low as 0.1 inches per hour for clayey soils with low permeability.

STORM DRAIN DESIGN

9-7. The type and capacity of the storm-drain facilities required are determined primarily by the promptness the design storm runoff must be removed to avoid serious interruption or hazard in the use of important operational areas and prevent serious damage to pavement subgrade. It is presumed that site reconnaissance phases have been carefully completed and that information that shows topography, natural drainage patterns, groundwater conditions, and seasonal frost and permafrost levels is available.

Basic Considerations

9-8. Even though rainfall magnitudes are small in arctic and subarctic regions, drainage is an important factor in selecting an airfield site and planning for construction. The planner should be aware of several factors related to drainage to ensure a successful design. These factors 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.
- Design features of a drainage system may be similar to those used in frost regions of the United States if the upper surface of the permafrost layer is deep and if provisions are made for lower temperatures.
- 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 that 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 away from runway and road shoulders as possible.
- Freeing drainage channels of drifted snow becomes a significant task before breakup each spring in many subarctic regions. In these areas, it is advantageous to have ditch shapes and slopes sufficiently wide and flat to accommodate heavy snow-moving equipment. In locations where flow continues year round, narrow, deep ditches are preferable to avoid icings.

- Large, cut sections should be avoided when planning the drainage layout. Thawed zones or water-bearing strata may be encountered and cause serious icings later. 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 before breakup each spring.
- Fine-grained soils located immediately above a receding frost zone are very unstable. Consequently, sliding and caving is to be expected on unprotected ditch side slopes.
- Ditches located 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 a heavy snow-removing apparatus and a steam boiler with accessories for steam thawing of structures that have become clogged with ice. Pipes for this purpose are often fastened inside the upper portions of culverts before their placement.

Grading

9-9. Proper grading is an 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

9-10. Trunk drains and laterals should have sufficient capacity to accommodate the project design runoff. Do not consider supplementary ponding above the drain inlets in airfield drainage designs for the arctic and subarctic. Formulate plans in sufficient detail to avoid flooding, even during construction.

ICING

9-11. An *icing* is an irregular sheet or field of ice with no uniformity as to its shape, thickness, or size. 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 the water supply source is depleted or warmer weather begins.

Types

9-12. For the purpose of analysis, icings may be divided into three groups, depending largely on the nature of the water supply source. River icing is when icing forms along rivers or streams and adjacent areas have a water source above or below the riverbed. Ground icing occurs when the source is from groundwater flow above permafrost. This term should not be confused with ground ice, which is often encountered in the arctic and subarctic as deposits in fine-grained soils. Spring icing occurs when the water source is from subpermafrost levels or subpermafrost water under hydrostatic pressure. Spring icings are commonly large and thick. Human activity can disturb the ground regime sufficiently enough to cause or accelerate the formation of icings.

River

9-13. Most arctic and subarctic streams carry large loads of sediment that 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 the thermal effects of flowing water, the soil below streambeds is unfrozen to greater depths than soil located elsewhere. Consequently, there is a large space for

groundwater storage and flow above the permafrost and below riverbeds. The head motivating ground water flow is ordinarily quite large and can result in large pressures above sections where the groundwater flow is retarded.

9-14. Groundwater-flow retardation is a natural process at many river sections because riverbeds are 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 further constriction of the channel cross-section area. The water then finds avenues of escape to the top of the ice through weak points, cracks, and fissures. Exposed to the cold atmosphere, the water quickly freezes into 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 warmer weather begins. A bridge may shade the streambed and prevent the deposition of snow. Freezing would occur more rapidly beneath the bridge than upstream or downstream. 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 on the valley topography, the depth of the snow, the intensity of cold, the water supply, and other factors.

Ground

9-15. Ground icings form mounds that have considerable thickness but cover a small area. They also form as crustations if groundwater flow is induced to the surface at points that 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 freeze. Icings from groundwater above the permafrost are not likely to occur in the arctic, as the permafrost there is too close to the surface to permit appreciable storage in the active layer. This occurrence is most severe in the southern zones of the subarctic and on slopes that 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 through which ground water must pass. The path of least resistance may lead to the ground surface through a frost crack, fissure, or holes that have been previously made by burrowing animals. Water coming to the surface in this way may flow considerable distances down slopes beneath a snow blanket without freezing.

9-16. Various methods have been tried to prevent the occurrence of such icings. Some of these have met with partial success. Russian investigators have advocated the Frost Belt or dam, 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

9-17. Icings that occur from artesian, subpermafrost water and springs are ordinarily quite thick and cover a considerable area. Reference is often made to the icing in the Momy River Valley of Siberia. The spring icing is about 15 miles long and 3 miles wide, with an average thickness of 12 feet. It does not melt and form each year. Spring icings can be easily controlled. 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

9-18. In the case of river icings, depths to permafrost are ordinarily too large to be blocked effectively by accelerated freezing, such as induced by a frost belt or dam. 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.

9-19. 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 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 provides a cold space where 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 to not 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.

9-20. If the source of water forming the icing is a spring, then drainage or diversion is necessary to control the occurrence. This sometimes requires insulated channels. In the case of springs, flows are ordinarily too large to permit a storage of ice at or upstream from the site.

Chapter 10

Soils Trafficability

Soils trafficability is the capacity of soils to support military vehicles. This chapter includes information on operating and maintaining the soil-trafficability test set, measuring trafficability with the results of the tests performed by the cone penetrometer and remolding equipment, and 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 percent 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.

BASIC TRAFFICABILITY FACTORS

10-1. Trafficability is more difficult to assess for fine-grained soils (silts and clays) and sands that contain enough fine-grained material to behave like fine-grained soils when wet than 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 effects under vehicle traffic determined by remolding test procedures.

10-2. The information discussed in this chapter is limited to problems associated with soils. It does not include problems associated with natural or manmade obstacles (forests, ditches) or information on vehicle characteristics (such as the maximum tilt or side angle a vehicle can climb without power stall or overturning). The basic principles 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 or passage of traffic.

10-3. Originally, this chapter was designed to permit calculations of trafficability by field personnel with a hand-held calculator. Performances were estimated for a minimum number of vehicle passes (one pass) or a maximum of 50 vehicles in the same ruts (50 passes). Today most relations are used for one pass, and the combined effects on vehicle performance of terrain features (soil, vegetation, slope) are determined through the use of the computerized Army mobility prediction system contained in the North Atlantic Treaty Organization Reference Mobility Model (NRMM). The engineering relationships that produce vehicle speed predictions or GO/NO-GO performance based on measured terrain and vehicle characteristics are contained in the NRMM. This chapter introduces fundamental relationships, terminologies, and illustrations of this computerized, comprehensive mobility evaluation tool. Most military units now have access to NRMM relationships through personal computer-based systems that contain the comprehensive NRMM.

SOIL STRENGTH

10-4. Bearing and traction capacities of soils are the most important soil strength factors effecting trafficability, and they are functions of soil shearing resistance. To evaluate the capacity of soils to support military vehicle traffic, shearing resistance is measured using a trafficability cone penetrometer. Cone penetrometer measurements are expressed in terms of cone index (CI).

10-5. Because the strength of fine-grained soils (silts and clays) may decrease when loaded or disturbed, remolding tests are used to characterize soil strength. Remolding tests measure the potential for a loss of soil strength to occur with vehicle traffic. The results of remolding tests are expressed in terms of the remolding index (RI).

10-6. The soil CI multiplied by the RI produces the rating cone index (RCI) used to denote soil strength corrected for remolding effects. The RCI is the principle measure of soil strength used to quantify the trafficability of soils.

SLIPPERINESS

10-7. The effects of soil slipperiness on vehicle performance can be drastic. Excess water or a layer of soft, plastic soil (especially clay soil) overlying a firm layer of soil can produce a slippery surface. This is especially evident on relatively firm soils after significant rainfall events. Slipperiness is troublesome, even when associated with soils that have very high bearing capacities.

10-8. Slippery soil conditions can cause extreme steering difficulty and degraded traction, especially for 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 or removal of rubber track pads will usually improve mobility in slippery conditions.

STICKINESS

10-9. Soil stickiness may pose a problem for vehicles operating in wet soils of low bearing capacity. Stickiness occurs in fine-grained soils when they are wet. The greater the plasticity of the soil, the more severe the effects of stickiness.

10-10. Under extreme conditions, sticky soil can accumulate in the vehicle running gear, making travel and steering difficult. Although it can adversely affect the speed and control of all vehicles, it usually will not cause immobilization except for the smallest tracked vehicles. The worst stickiness is just a nuisance to larger, more powerful military vehicles. Removing fenders will reduce stickiness effects on some vehicles.

INSTRUMENTS AND TESTS FOR TRAFFICABILITY

10-11. This section contains general information regarding the instruments and tests used to measure soils trafficability. The specific instruments, operating instructions, and test procedures are described in more detail in TM 3-34.48-2. Sieve-analysis tests, plasticity tests, and other field identification tests are described in TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034.

SOIL-TRAFFICABILITY TEST SET

10-12. Trafficability measurements are made with the soil-trafficability test set. This set consists of one canvas carrying case; one cone penetrometer with a 3/8-inch steel shaft, a 5/8-inch aluminum penetrometer shaft, a 0.5-square-inch cone, and a 0.2-square-inch cone; one soil sampler; remolding equipment (includes a 2 1/2-pound drop hammer and a remolding cylinder with base and pin); and a bag of hand tools. (See figure 10-1 for the items shown in their proper locations in the carrying case.) The set is carried on the back. (See figure 10-2.) The complete set weighs 19 pounds.

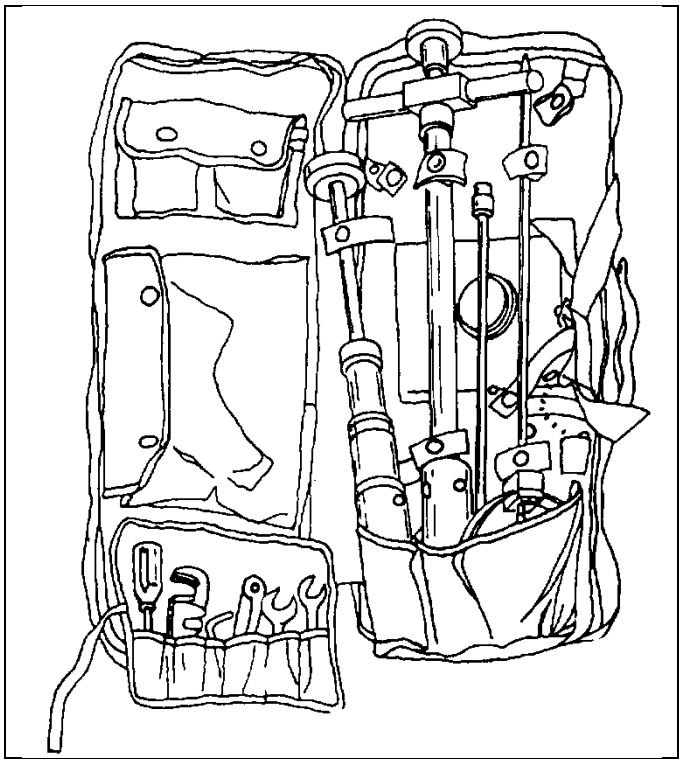


Figure 10-1. Soil-trafficability test set

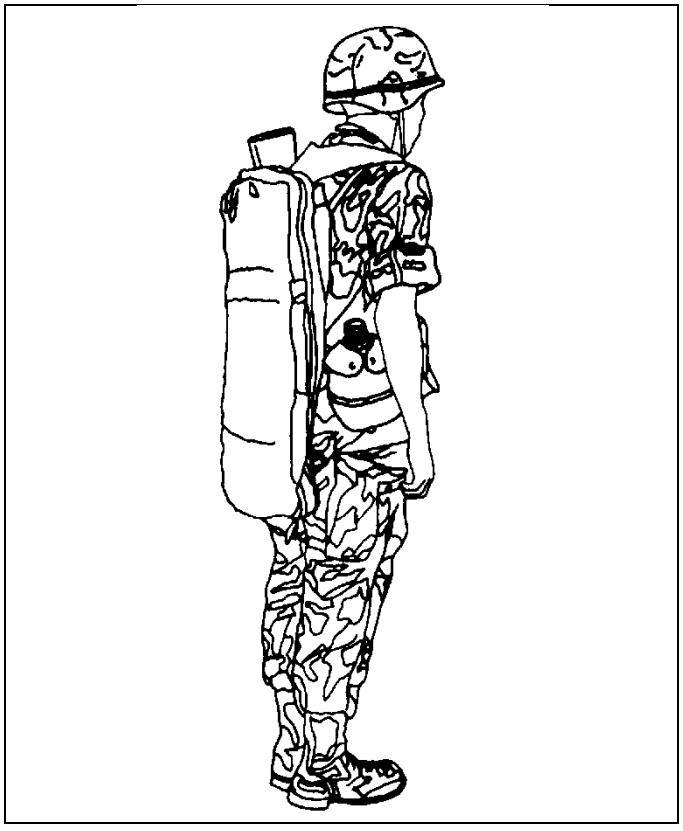


Figure 10-2. Carrying a soil-trafficability test set

CONE PENETROMETER

10-13. The primary instrument of the soil-trafficability test set is the cone penetrometer. (See figure 10-3, figure 10-4, and figure 10-5.) It is used to determine the shearing strength of low-strength soils for trafficability purposes. It is functionally similar to the dynamic cone penetrometer (described in TM 3-34.48-2). They are used to measure the shear strength of soils, but the dynamic cone penetrometer is used to determine shear strength for relatively high-strength soils (such as subgrade and base course materials for roads and airfields) and is not suitable for trafficability measurements.

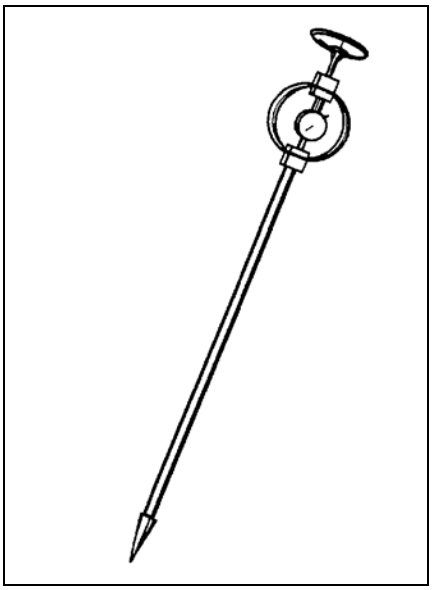


Figure 10-3. Cone penetrometer

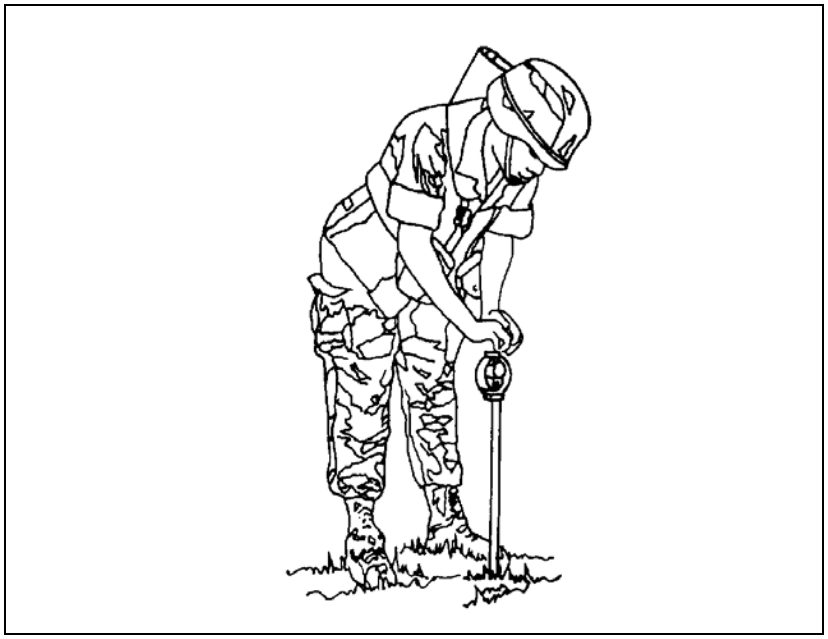


Figure 10-4. Using a cone penetrometer in the upright position

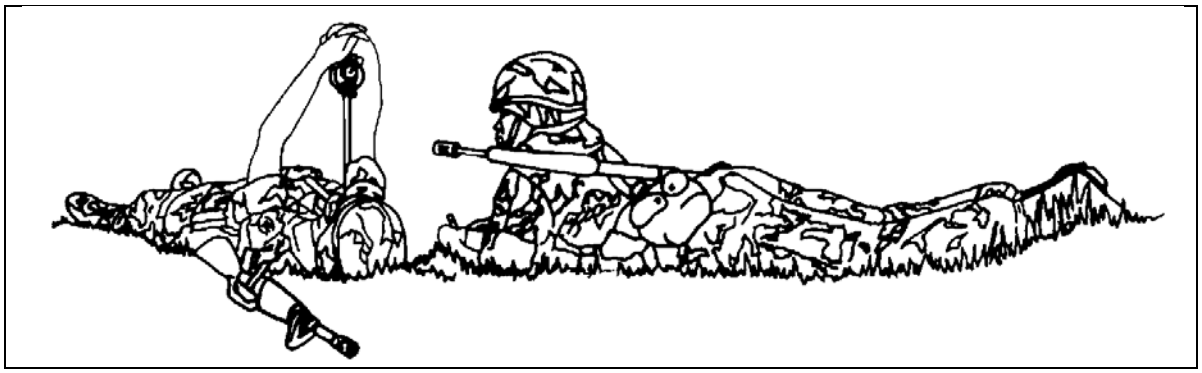


Figure 10-5. Using a cone penetrometer in the prone position

10-14. The cone penetrometer consists of a 30° cone, a 19-inch long penetrometer shaft, a proving ring with mounting blocks, a dial indicator, and a handle. When the cone is forced into the ground, the proving ring is deformed in proportion to the force applied. The amount of force required to move the cone slowly through a given plane per unit area of the base of the cone (0.5 or 0.2 square inch) is indicated on the dial indicator inside the proving ring. This force per unit base area is an index of the soils shearing resistance and is called the soil CI in that plane.

10-15. The dial range is 0 to 300 pounds per square inch. The range of loads applied to the cone penetrometer for 0 to 300 pounds per square inch on the 0.5 square-inch cone is 0 to 150 pounds. The 3/8-inch penetrometer shaft and the 0.5 square-inch cone are used for most measurement applications. The 0.2-square-inch cone is used with the 3/8-inch penetrometer shaft for sampling in relatively hard soils and for remolding tests in remoldable sands. The cone penetrometer cannot be used to measure CI in gravels. However, gravels are considered excellent for 50 passes and problems can be determined by visual observation. As previously stated, the operating instructions and test procedures for the cone penetrometer and the remainder of the soil-trafficability test set are described in greater detail in TM 3-34.48-2.

SOIL MEASUREMENTS FOR TRAFFICABILITY

10-16. When reconnaissance parties have time to make trafficability measurements, they should obtain data for determining the number and type of vehicles that can cross the area and the slopes that they can climb. The procedures for measuring soil trafficability data are described in this section.

Note. The measurements will only be valid for short time periods provided no weather changes occur.

CRITICAL LAYER

10-17. The critical layer is the layer of soil that contributes most significantly to the trafficability of the ground for a specific vehicle. The depth of the critical layer varies with the soil type, soil strength profile, vehicle type and weight, and number of passes required. Table 10-1, page 10-6, summarizes these variations for common types of military vehicles.

RANGE OF CONE INDEX VALUES

10-18. 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 most vehicles for 50 passes. These limits usually make it possible, while gathering data for a trafficability evaluation, to classify large areas as above or below the critical range without extensive testing.

Table 10-1. 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***
Wheeled vehicles with wheel loads under 2,000 lb	3 to 9	0 to 6	3 to 9	0 to 6
Wheeled vehicles with wheel loads of 2,000 to 15,000 lb	6 to 12	0 to 6	6 to 12	0 to 6
Wheeled vehicles with wheel loads over 15,000 lb	0 to 12	0 to 6	0 to 12	0 to 6
Tracked vehicles with ground contact pressure under 4 psi	3 to 9	0 to 6	3 to 9	0 to 6
Tracked vehicles up to 100,000 lb	6 to 12	0 to 6	6 to 12	0 to 6
Tracked vehicles more than 100,000 lb	0 to 12	0 to 6	0 to 12	0 to 6

* Fine-grained soils and remoldable sands
** Coarse-grained soils
Note. Vehicle weights are located in TM 3-34.48-2 with VCI_1 and VCI_{50} values for military vehicles.

Legend:

lb	pounds
psi	pounds per square inch
VCI	vehicle cone index

NUMBER OF MEASUREMENTS

10-19. 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 average accuracy for a 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 a given depth. The 15 readings should be distributed throughout a uniform area.

10-20. If time is not available to take a large number of measurements, use judgment to reduce the number according to the following instructions:

- If CI values 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 RI values. 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 CI values 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. For RI values 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 CI values 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.

10-21. Using the sample work sheet in figure 10-6, page 10-8, 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 and the 6-inch reading is 63. The readings are then averaged for each depth, as demonstrated in the equation below for the 0-inch depth. The numbers in the numerator are the readings from each test at that depth. The number in the denominator represents the number of tests conducted. The resulting quotient is the average CI for that depth. See the following equation:

$$Average\ CI = \frac{58 + 63 + 65 + 72 + 75}{5} = 66.6 = 66 \text{ (round CI values down)}$$

10-22. After calculating the average CI for each depth, the average CI for each 6-inch layer is calculated by averaging the CI for the top and bottom depths of each layer. If the 0-inch CI is 66 and the 6-inch CI is 71, the average CI for the 0- to 6-inch layer is 68. The CI is calculated for the 6- to 12-inch, 12- to 18-inch, and 18- to 24-inch layers the same way.

Note. Intermediate values for the 3-, 9-, and 15-inch depths (fine-grained soils and remoldable sands) can be interpolated when the vehicle types under consideration require them.

10-23. The critical layer for the M929 dump truck for one vehicle and for 50 vehicles is the 9- to 15-inch layer. 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 when necessary.

Example. To find the average CI for the 3- to 9-inch layer, the average CI for the 0- to 6-inch and 6- to 12-inch layers are interpolated as shown in the following equation:

$$\frac{68 + 73}{2} = 70.5 = 70 \text{ (always round CI values down)}$$

Likewise, for the 9- to 15-inch layer, the 6- to 12-inch and 12- to 18-inch layers are interpolated as follows:

$$\frac{73 + 76}{2} = 74.5 = 74$$

The resulting average CI for the 9- to 15-inch critical layer is 74.

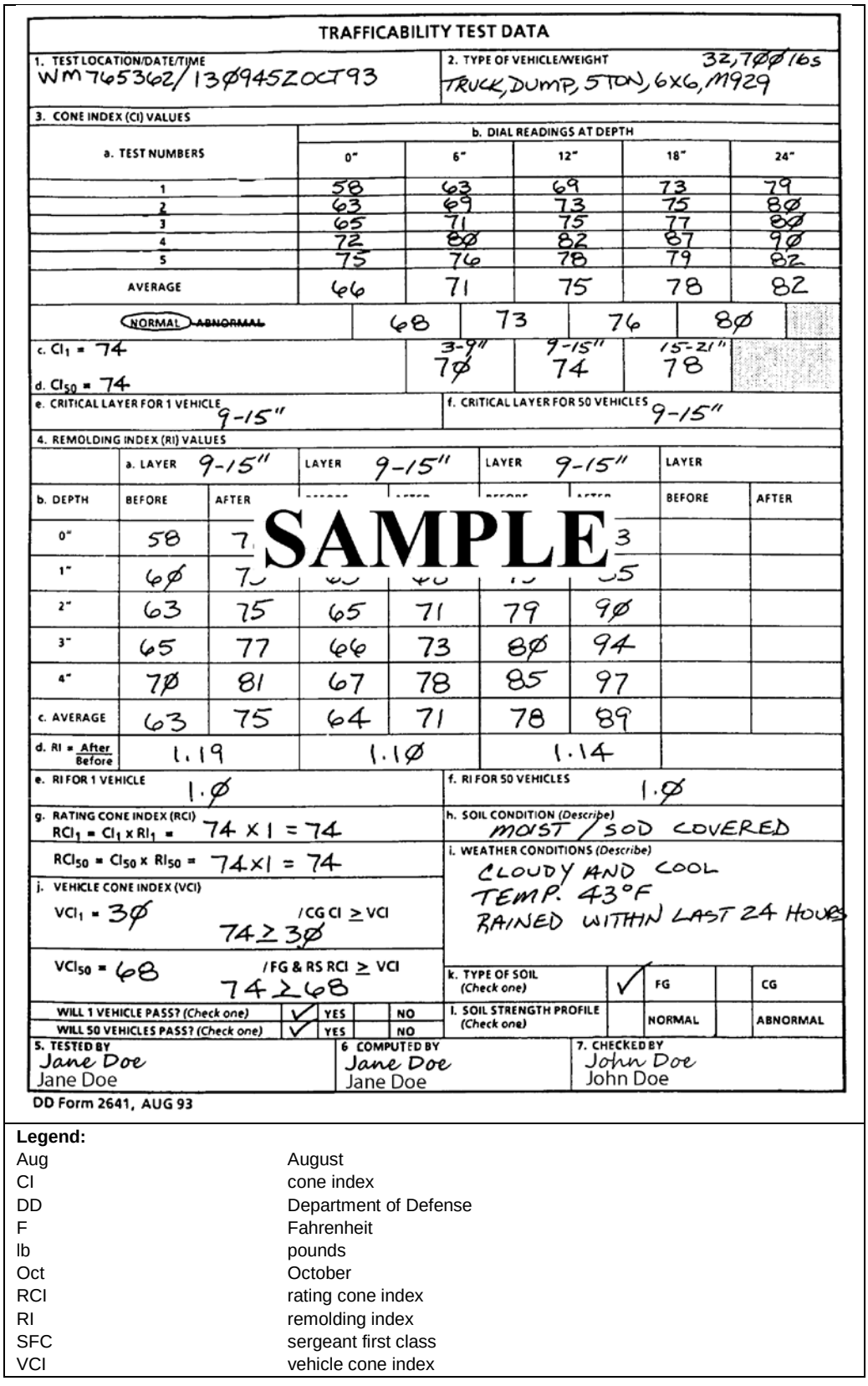
STRENGTH PROFILE

10-24. A soil profile is a group of soils with similar characteristics and arrangement of soil horizons in the soil profile. For most Army vehicles, in an area with a normal soil profile the critical layer is that between six and twelve inches.

Normal Strength Profile in Fine-Grained Soils and Remoldable Sands

10-25. In a soil with a normal strength profile, the CI readings increase or remain essentially constant with each increment of depth. An area with a normal strength profile is shown in table 10-2, page 10-9. CI values 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 normal critical layer need to be recorded. For a tracked vehicle weighing less than 100,000 pounds, such as the M113A3 armored personnel carrier, only readings for the 6- and 12-inch depths must be recorded since the critical layer is the 6- to 12-inch layer. (See table 10-1.)

10-26. Since a decrease in RI with increasing depth is not common, remolding tests are taken from the normal critical layer for the vehicle in question for a normal strength profile. The resulting average RI value for the critical layer is multiplied by the average CI value for that layer to determine the average RCI value for the critical layer. The RCI for this layer is used as the criterion of trafficability for the particular vehicle.



TRAFFICABILITY TEST DATA

1. TEST LOCATION/DATE/TIME: WM765362/13Ø945ZOCT93
2. TYPE OF VEHICLE/WEIGHT: TRUCK, DUMP, 5TON, 6X6, M929 — 32,7ØØ lbs

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	8Ø
3	65	71	75	77	~~8Ø~~
4	72	~~8Ø~~	82	87	9Ø
5	75	76	78	79	82
AVERAGE	66	71	75	78	82
~~ABNORMAL~~ NORMAL	68	73	76	8Ø	

c. CI_1 = 74 — 3-9": 7Ø; 9-15": 74; 15-21": 78

d. CI_{50} = 74

e. CRITICAL LAYER FOR 1 VEHICLE: 9-15"

f. CRITICAL LAYER FOR 50 VEHICLES: 9-15"

4. REMOLDING INDEX (RI) VALUES

	a. LAYER 9-15"		LAYER 9-15"		LAYER 9-15"		LAYER	
b. DEPTH	BEFORE	AFTER					BEFORE	AFTER
0"	58	7[illegible]				3		
1"	6Ø	7[illegible]				[illegible]5		
2"	63	75	65	71	79	9Ø		
3"	65	77	66	73	8Ø	94		
4"	7Ø	81	67	78	85	97		
c. AVERAGE	63	75	64	71	78	89		
d. RI = After / Before	1.19		1.1Ø		1.14			

SAMPLE

e. RI FOR 1 VEHICLE: 1.Ø

f. RI FOR 50 VEHICLES: 1.Ø

g. RATING CONE INDEX (RCI): $RCI_1 = CI_1 \times RI_1$ = 74 X 1 = 74

$RCI_{50} = CI_{50} \times RI_{50}$ = 74X1 = 74

h. SOIL CONDITION (Describe): MOIST / SOD COVERED

i. WEATHER CONDITIONS (Describe): CLOUDY AND COOL; TEMP. 43°F; RAINED WITHIN LAST 24 HOURS

j. VEHICLE CONE INDEX (VCI)

VCI_1 = 3Ø — /CG CI ≥ VCI: 74 ≥ 3Ø

VCI_{50} = 68 — /FG & RS RCI ≥ VCI: 74 ≥ 68

k. TYPE OF SOIL (Check one): ✓ FG; CG

l. SOIL STRENGTH PROFILE (Check one): NORMAL; ABNORMAL

WILL 1 VEHICLE PASS? (Check one): ✓ YES; NO

WILL 50 VEHICLES PASS? (Check one): ✓ YES; NO

5. TESTED BY: Jane Doe / Jane Doe

6. COMPUTED BY: Jane Doe / Jane Doe

7. CHECKED BY: John Doe / John Doe

DD Form 2641, AUG 93

Legend:

Aug	August
CI	cone index
DD	Department of Defense
F	Fahrenheit
lb	pounds
Oct	October
RCI	rating cone index
RI	remolding index
SFC	sergeant first class
VCI	vehicle cone index

Figure 10-6. Trafficability test data form

Table 10-2. Examples of soil strength profiles

Depth (inches)	*Cone Index Penetration Test Number*															
	1	*2*	*3*	*4*	*5*	*6*	*7*	*8*	*9*	*10*	*11*	*12*	*13*	*14*	*15*	*AVG*
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 three remolding tests made in the 6- to 12-inch 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 three remolding tests made in the 6- to 12-inch layer are: 0.87, 0.90, and 0.93. The average remolding index for the 6- to 12-inch layer is 0.90. The results from three remolding tests in the 12- to 18-inch layer are: 0.88, 0.91, and 0.91. The average remolding index for the 12- to 18-inch 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						69
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						286+
Note. The + is normally added to the average to indicate several of the readings exceeded 300. Do not add a + if the number of readings exceeding 300 is less than 20%.																
Legend:																
avg average																

Abnormal Strength Profile in Fine-Grained Soils and Remoldable Sands

10-27. An abnormal strength profile has at least one CI reading that is lower than the reading immediately above it. An area with an abnormal strength profile is shown in table 10-2. 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 armored personnel carrier) to 6 inches below the bottom of the normal critical layer (18 inches for the M113A3 armored personnel carrier). (See table 10-1, page 10-6.)

10-28. Similarly, remolding tests must be run on samples from the normal critical layer and 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 used as the critical layer for these vehicles.

Strength Profile in Coarse-Grained Soils

10-29. The critical layer for most vehicles in coarse-grained soils is the 0- to 6-inch layer. (See table 10-1, page 10-6.) Most coarse-grained soils have a normal strength profile with a large increase in strength and depth when compared to fine-grained soils. For this reason, CI measurements should be taken at 3- to 18-inch increments or until the maximum capacity (750 CI) of the penetrometer has been reached.

10-30. Usually, fewer penetrations are required to establish an average because coarse-grained soil areas are generally more uniform than fine-grained soils and remoldable sands. The RI tests are not required. Example strength measurements in a coarse-grained soil area are shown in table 10-2, page 10-9.

RATING CONE INDEX

10-31. As previously defined, the RCI is the CI that will result under traffic. This value can be compared to the vehicle cone index (VCI) to determine the trafficability of an area for a specific vehicle.

Example

Fine-grained soil areas are investigated for vehicle trafficability with a normal critical layer of 6 to 12 inches. Because area A has a normal strength profile, a remolding test was run for the 6- to 12-inch layer. (See table 10-2.) The RCI for area A is 60 (the average of 50 and 70).

$$60 \times 0.90 = 54$$

In area B, an abnormal strength profile was evident. Therefore, 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).

$$60 \times 0.90 = 54$$

The RCI of the 12- to 18-inch layer is 40.

$$40 \times 0.90 = 36$$

Since the RCI of the 12- to 18-inch layer is less than the RCI for the 6- to 12-inch layer, an RCI of 36 is the governing value for the trafficability of the vehicles in area B.

Example

The critical layer for one or 50 M929 dump trucks is 9 to 15 inches. (See figure 10-6, page 10-8.) Three remolding samples were collected from the 9- to 15-inch critical layer, yielding different RI results. The average of these three remolding results is calculated and this number becomes the RI for that layer.

$$\frac{1.19 + 1.10 + 1.14}{3} = 1.14 = 1.0 \ (always\ use\ 1.0\ for\ RI\ results\ greater\ than\ 1.0)$$

10-32. The remaining calculations shown at the bottom of figure 10-6, demonstrate how RCI measurements can be used to evaluate trafficability for a specific vehicle, to include—

- The RCI values in the critical layer for one vehicle pass (RCI_1) and for 50 vehicle passes (RCI_{50}) are 74.
- The VCI for one vehicle pass (VCI_1) is 30, and the VCI for 50 vehicle passes (VCI_{50}) is 68.
- Comparing the RCI_1 to VCI_1 and RCI_{50} to VCI_{50}, it is determined that one M929 dump truck can cross the area as well as 50 M929 dump trucks because the RCI values are greater than or equal to the VCI values.

10-33. In other words, the VCI represents the minimum RCI soil strength that is required for a specific vehicle to make a specified number of passes through an area without becoming immobilized. The VCI concepts are explained in greater detail later in this chapter.

10-34. A mixture of vehicles will usually pass through an area, not a column of one vehicle type. Therefore, the VCI and the critical layer RCI values should be determined for the worst case vehicle (the one with the highest VCI).

VARIATION OF TRAFFICABILITY WITH WEATHER

10-35. Changes in weather produce changes in soil trafficability. Trafficability characteristics measured on a given date cannot be applied later, unless full allowance is made for potential changes in the state of the soil caused by weather effects.

10-36. Fine-grained soils increase in moisture during rainy periods. This results in slipperiness, stickiness, and decreased strength. Dry periods produce the opposite effects. Trafficability on loose sands improves during rainy periods due to an increase in cohesion, and sands return to the loose, less trafficable state during dry periods.

10-37. Freezing and thawing conditions 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 most vehicles. Snow cover can have a significant effect on the depth of freezing. The absence of snow allows frost to penetrate deeper into the soil.

10-38. Techniques have been developed for predicting the effects of weather on soil trafficability. These techniques are part of the comprehensive NRMM, but are beyond the scope of this manual.

OTHER TRAFFICABILITY EVALUATION FACTORS

10-39. Other trafficability evaluation factors include slope, vegetation, and organic-soil areas. Refer to the NRMM for procedures beyond the scope of this manual.

10-40. 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 above the requirements of level terrain.

10-41. The specific effects of vegetation on trafficability are beyond the scope of this manual, but some points are worthy of mention. Dense grass, especially if wet, may produce slippery conditions. Additionally, soil strength requirements will be greater than normal if small trees or thick brush must be pushed down by the vehicle.

10-42. 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, can travel 50 passes over organic materials that are more than 6 inches thick.

10-43. Usually, high-ground-pressure vehicles can travel a few passes before they break through and become immobilized. Military wheeled vehicles usually cannot travel on most of these organic-soil areas. CI measurements 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.

10-44. A complete assessment of the trafficability for a given area must include an evaluation of obstacles (such as forests, rivers, boulder fields, ditches, and hedgerows). Obstacle effects on the performance of vehicles can be determined using the comprehensive NRMM.

APPLICATION OF TRAFFICABILITY PROCEDURES IN FINE-GRAINED SOILS AND REMOLDABLE SANDS

10-45. The procedures presented earlier in this chapter are intended for use in tactical operations. Criteria is established so that when the RCI for a given area is equal to or greater than the VCI for one or 50 passes of a selected vehicle, sufficient soil strength will be available for the ground to withstand one or 50 passes of the same vehicle (or vehicles with lower VCI_1 or VCI_{50}) operating at a slow speed in the same ruts (in the case of multiple passes) and to permit stopping and resumption of movement, if necessary. The following text gives examples for application of trafficability performance criteria for both one and 50 vehicle passes.

10-46. As previously described, vehicle performance can be significantly degraded when the soil is in a slippery state (for example, a recent rainfall event on soil with high bearing capacity). Performance curves that can be used for estimating vehicle performance on slippery soil conditions have been developed, but using these curves involves complex computation procedures that are beyond the scope of this manual.

Note. The trafficability techniques in the following text is applicable on slippery surface conditions, but the vehicle performance relations provided in this manual are only applicable to nonslippery soils.

VEHICLE CONE INDEX

10-47. The following paragraphs provide information on VCI data, a means of calculating VCI for new or unlisted vehicles, and a means to approximate the VCI for less than 50 passes.

Vehicle Cone Index for Typical, Representative Vehicles

10-48. TM 3-34.48-2 contains a representative listing of typical tracked and wheeled self-propelled military vehicles where each vehicle is identified by its standard nomenclature. The purpose of the listing is to provide the nominal gross vehicle weights and the associated VCI performance values for one ($VCI_{1)}$ and 50 (VCI_{50}) vehicle passes. The listing is segregated based on the type of running gear (tracked or wheeled) and the intended vehicle mission. The listing includes other useful information, such as general vehicle characteristics that is required for estimating vehicle trafficability performance.

Vehicle Cone Index for New or Unlisted Vehicles

10-49. For conventional-type vehicles not shown in TM 3-34.48-2, the following procedure can be used to calculate the VCI.

10-50. Calculate VCI by using the NRMM and an axle-by-axle consideration of vehicle characteristics and their influence on performance. This is the most accurate method; however, the VCI can also be estimated by using assumed equal wheel or track load distributions and nominal highway tire pressures. The NRMM accounts for uneven load distributions and differences in tire pressures. The VCI for wheeled vehicles may vary over 20 percent due to tire pressure changes alone.

Note. A reasonable estimate of the improved performance for wheeled vehicles that are equipped with central tire inflation systems can be determined by using the formulas below to estimate VCI and reduce those estimates by 15 percent to reflect that drivers of vehicles with central tire inflation systems will reduce the tire pressures as required.

Self-Propelled, Tracked Vehicles

10-51. Use the following steps for self-propelled, tracked vehicles:

- **Step 1.** Determine the mobility index (MI).

$$MI = \left[\frac{\begin{matrix} contact \\ pressure \\ factor \end{matrix} \times \begin{matrix} weight \\ factor \end{matrix}}{\begin{matrix} track \\ factor \end{matrix} \times \begin{matrix} grouser \\ factor \end{matrix}} + \begin{matrix} bogie \\ factor \end{matrix} - \begin{matrix} clearance \\ factor \end{matrix} \right] \times \begin{matrix} engine \\ factor \end{matrix} \times \begin{matrix} transmission \\ factor \end{matrix}$$

The eight MI factors are calculated using the following formulas. The term *weight* is used to represent gross vehicle weight, and the units are in pounds and inches unless otherwise specified.

$$\begin{matrix} contact \\ pressure \\ factor \end{matrix} = \frac{weight}{area\ of\ tracks\ in\ contact\ with\ the\ ground}$$

$$weight\ factor = \left\{ \begin{array}{ll} 1.0 & for\ weights\ less\ than\ 40{,}000\ pounds \\ \dfrac{weight/1000}{24.84 + 0.379 \times (weight/1000)} & for\ weights\ of\ 40{,}000\ pounds\ or\ more \end{array} \right\}$$

$$track\ factor = \frac{track\ width}{100}$$

$$grouser\ factor = \left\{ \begin{array}{l} 1.0\ \ for\ grouser\ heights\ less\ than\ 1.5\ inches \\ 1.1\ \ for\ grouser\ heights\ of\ 1.5\ inches\ or\ more \end{array} \right\}$$

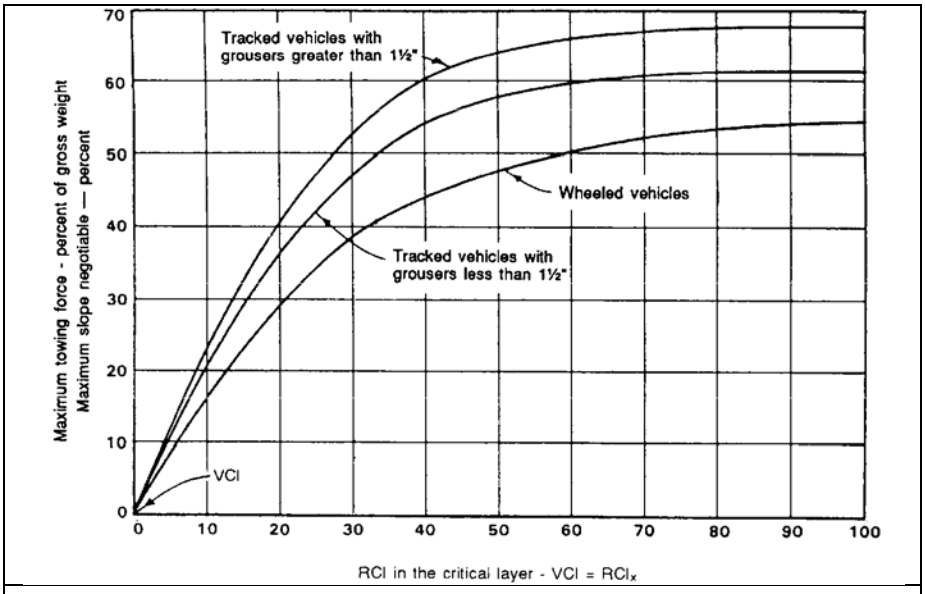
$$bogie\ factor = \frac{weight/10}{\begin{pmatrix} total\ bogies\ on\ tracks \\ in\ contact\ with\ the\ ground \end{pmatrix} \times \begin{pmatrix} area\ of\ one\ track\ shoe \\ in\ square\ inches \end{pmatrix}}$$

$$clearance\ factor = \frac{ground\ clearance\ height}{10}$$

$$engine\ factor = \left\{ \begin{array}{l} 1.00\ \ for\ horsepower\ per\ ton\ of\ vehicle\ weight\ equal\ to\ 10\ or\ more \\ 1.05\ \ for\ horsepower\ per\ ton\ of\ vehicle\ weight\ less\ than\ 10 \end{array} \right\}$$

$$transmission\ factor = \left\{ \begin{array}{l} 1.00\ \ for\ automatic\ transmissions \\ 1.05\ \ for\ manual\ transmissions \end{array} \right\}$$

- **Step 2.** Use figure 10-7 or table 10-3, page 10-14, to convert MI to VCI.



Legend:

RCI	rating cone index
VCI	vehicle cone index

Figure 10-7. Relationship between MI and VCI

Table 10-3. VCI for tracked vehicles (MI versus VCI for one and 50 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	60.7	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	62.9	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	13.3	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	28.1	64.2	140	34.7	78.6
9	6.1	15.3	42	14.6	34.8	75	21.5	50.0	108	28.3	64.6	141	34.9	79.1
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	67.3	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	36.5	82.5
18	8.9	22.0	51	16.5	39.0	84	23.4	54.0	117	30.1	68.6	150	36.7	83.0
19	9.2	22.6	52	16.7	39.5	85	23.6	54.5	118	30.3	69.0	151	36.9	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	43.7	94	25.4	58.4	127	32.1	72.9	160	38.8	87.3
29	11.7	28.3	62	18.8	44.1	95	25.6	58.9	128	32.3	73.4	161	39.0	87.8
30	11.9	28.8	63	19.0	44.6	96	25.8	59.3	129	32.5	73.8	162	39.2	88.2
31	12.1	29.3	64	19.2	45.0	97	26.0	59.8	130	32.7	74.3	163	39.4	88.6
32	12.4	29.8	65	19.4	45.5	98	26.2	60.2	131	32.9	74.7	164	39.6	98.1

Note. For MI values, the VCI is obtained from the following equations:

$$1\ Pass: VCI_1 = 7.0 + 0.2\ MI - [39.2/(MI + 5.6)]$$

$$50\ Passes: VCI_{50} = 19.27 + 0.43\ MI - [125.79/(MI + 7.08)]$$

Legend:

MI	mobility index
VCI	vehicle cone index

Self-Propelled, All-Wheel-Drive Wheeled Vehicles

10-52. Use the following steps for self-propelled, all-wheel-drive wheeled vehicles:

- **Step 1.** Determine the MI.

$$MI = \left[\frac{\begin{matrix} contact \\ pressure \\ factor \end{matrix} \times \begin{matrix} weight \\ factor \end{matrix}}{\begin{matrix} tire \\ factor \end{matrix} \times \begin{matrix} grouser \\ factor \end{matrix}} + \begin{matrix} wheel \\ load \\ factor \end{matrix} - \begin{matrix} clearance \\ factor \end{matrix} \right] \times \begin{matrix} engine \\ factor \end{matrix} \times \begin{matrix} transmission \\ factor \end{matrix}$$

The eight MI factors are calculated using the following formulas. The term *weight* is used to represent gross vehicle weight, and the units are in pounds and inches unless otherwise specified.

$$\begin{matrix} contact \\ pressure \\ factor \end{matrix} = \frac{weight}{\begin{matrix} tire \\ width \end{matrix} \times \frac{outside\ diameter\ of\ tires}{2} \times \begin{matrix} total\ number \\ of\ tires \end{matrix}}$$

$$\begin{matrix} weight \\ factor \end{matrix} = \begin{cases} 0.553 \times (axle\ weight/1000) & for\ axle\ weights\ less\ than\ 2{,}000\ pounds \\ 0.033 \times (axle\ weight/1000) + 1.050 & for\ axle\ weights\ of\ 2{,}000\ to\ 13{,}499\ pounds \\ 0.142 \times (axle\ weight/1000) - 0.420 & for\ axle\ weights\ of\ 13{,}500\ to\ 19{,}999\ pounds \\ 0.278 \times (axle\ weight/1000) - 3.115 & for\ axle\ weights\ of\ 20{,}000\ to\ 31{,}499\ pounds \\ 0.836 \times (axle\ weight/1000) - 20.686 & for\ axle\ weights\ of\ 31{,}500\ pounds\ or\ more \end{cases}$$

$$\begin{matrix} tire \\ factor \end{matrix} = \frac{10 + tire\ width}{100}$$

$$\begin{matrix} grouser \\ factor \end{matrix} = \begin{cases} 1.00 & for\ tires\ without\ chains \\ 1.05 & for\ tires\ with\ chains \end{cases}$$

$$\begin{matrix} wheel \\ load \\ factor \end{matrix} = \frac{weight/1000}{total\ number\ of\ wheels}\ (dual\ wheels\ count\ as\ singles)$$

$$\begin{matrix} clearance \\ factor \end{matrix} = \frac{ground\ clearance\ height}{10}$$

$$\begin{matrix} engine \\ factor \end{matrix} = \begin{cases} 1.00 & for\ horsepower\ per\ ton\ of\ vehicle\ weight\ equal\ to\ 10\ or\ more \\ 1.05 & for\ horsepower\ per\ ton\ of\ vehicle\ weight\ less\ than\ 10 \end{cases}$$

$$\begin{matrix} transmission \\ factor \end{matrix} = \begin{cases} 1.00 & for\ automatic\ transmissions \\ 1.05 & for\ manual\ transmissions \end{cases}$$

- **Step 2.** Use figure 10-7, page 10-13, or table 10-4, page 10-16, to convert MI to VCI.

Table 10-4. VCI for wheeled vehicles (MI versus VCI for one and 50 passes)

MI	*VCI₁*	*VCI₅₀*	*MI*	*VCI₁*	*VCI₅₀*	*MI*	*VCI₁*	*VCI₅₀*	*MI*	*VCI₁*	*VCI₅₀*	*MI*	*VCI₁*	*VCI₅₀*
0	1.0	3.0	33	17.0	39.9	66	24.1	55.3	99	30.9	69.9	132	36.2	79.4
1	3.4	8.8	34	17.2	40.4	67	24.3	55.7	100	31.1	70.3	133	36.3	79.7
2	5.1	12.7	35	17.5	40.9	68	24.5	56.2	101	31.3	70.5	134	36.4	80.0
3	6.3	15.6	36	17.7	41.4	69	24.7	56.6	102	31.5	70.8	135	36.6	80.2
4	7.2	17.9	37	17.9	41.9	70	24.9	57.1	103	31.7	71.1	136	36.7	80.5
5	8.0	19.7	38	18.1	42.3	71	25.2	57.5	104	31.9	71.4	137	36.8	80.8
6	8.7	21.2	39	18.4	42.8	72	25.4	58.0	105	32.1	71.7	138	36.9	81.0
7	9.2	22.6	40	18.6	43.3	73	25.6	58.4	106	32.3	72.0	139	37.0	81.3
8	9.7	23.7	41	18.8	43.8	74	25.8	58.9	107	32.5	72.3	140	37.1	81.5
9	10.2	24.8	42	19.0	44.3	75	26.0	59.3	108	32.7	72.6	141	37.3	81.8
10	10.6	25.8	43	19.2	44.7	76	26.2	59.7	109	32.9	72.9	142	37.4	82.1
11	11.0	26.6	44	19.5	45.2	77	26.4	60.2	110	33.1	73.2	143	37.5	82.3
12	11.4	27.5	45	19.7	45.7	78	26.6	60.6	111	33.3	73.5	144	37.6	82.6
13	11.7	28.3	46	19.9	46.1	79	26.8	61.1	112	33.5	73.8	145	37.7	82.8
14	12.1	29.0	47	20.1	46.6	80	27.0	61.5	113	33.7	74.1	146	37.9	83.1
15	12.4	29.7	48	20.3	47.1	81	27.2	62.0	114	33.9	74.4	147	38.0	83.3
16	12.7	30.4	49	20.5	47.5	82	27.4	62.4	115	34.1	74.7	148	38.1	83.6
17	13.0	31.1	50	20.8	48.0	83	24.6	62.9	116	34.2	75.0	149	38.2	83.8
18	13.3	31.7	51	21.0	48.5	84	27.8	63.3	117	34.3	75.3	150	38.3	84.1
19	13.6	32.3	52	21.2	48.9	85	28.0	63.7	118	34.4	75.6	151	38.4	84.3
20	13.8	32.9	53	21.4	49.4	86	28.2	64.2	119	34.6	75.8	152	38.5	84.6
21	14.1	33.5	54	21.6	49.8	87	28.4	64.6	120	34.7	76.1	153	38.7	84.8
22	14.4	34.1	55	21.8	50.3	88	28.7	65.1	121	34.8	76.4	154	38.8	85.1
23	14.6	34.6	56	22.0	50.8	89	28.9	65.5	122	34.9	76.7	155	38.9	85.3
24	14.9	35.2	57	22.2	51.2	90	29.1	65.9	123	35.1	77.0	156	39.0	85.6
25	15.1	35.7	58	22.4	51.7	91	29.3	66.4	124	35.2	77.3	157	39.1	85.8
26	15.4	36.3	59	22.7	52.1	92	29.5	66.8	125	35.3	77.5	158	39.2	86.1
27	15.6	36.8	60	22.9	52.6	93	29.7	67.3	126	35.4	77.8	159	39.3	86.3
28	15.8	37.3	61	23.1	53.0	94	29.9	67.7	127	35.6	78.1	160	39.4	86.6
29	16.1	37.9	62	23.3	53.5	95	30.1	68.1	128	35.7	78.4	161	39.5	86.8
30	16.3	38.4	63	23.5	53.9	96	30.3	68.6	129	35.8	78.6	162	39.6	87.0
31	16.6	38.9	64	23.7	54.4	97	30.5	69.0	130	35.9	78.9	163	39.8	87.3
32	16.8	39.4	65	23.9	54.8	98	30.7	69.5	131	36.1	79.2	164	39.9	87.5

Note. For MI values, the VCI is obtained from the following equations:

$$1\ Pass: VCI_1 = 11.48 + 0.2\ mi - [39.2/((MI + 3.74)\ )]\ for\ MI \leq 115$$

$$VCI_1 = 4.1\ MI^{0.446}\ for\ MI > 115$$

$$50\ Passes: VCI_{50} = 28.23 + 0.43\ MI - [92.67/((MI + 3.67)\ )]\ for\ MI \leq 100$$

$$VCI_{50} = 9.0 MI^{0.446}\ for\ MI > 100$$

Legend:

MI	mobility index
VCI	vehicle cone index

Self-Propelled, Rear-Wheel-Drive Wheeled Vehicles

10-53. For rear-wheel-drive vehicles where the first axle is a nondriven axle, a rough approximation for VCI can be estimated by using the all-wheel-drive formulas. Convert MI to VCI using figure 10-7, page 10-13, or table 10-4 and multiply the resulting VCI by 1.4 to account for the effect of the nondriven front axle.

Self-Propelled, Half-Tracked Vehicles

10-54. For half-tracked vehicles, a rough approximation for VCI can be estimated by using the all-wheel-drive formulas with the assumption that the vehicle has wheels instead of tracks on the rear end. The notional rear wheels are assumed to be 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. Use figure 10-7 or table 10-4 to convert MI to VCI.

Approximate Vehicle Cone Index for Passes Between One and Fifty

10-55. To estimate the number of vehicles that can cross an area when the RCI is greater than VCI_1, but less than VCI_{50}, or to estimate the VCI for less than 50 vehicles, use the following formula:

$$(VCI_{50} - VCI_1) \times \frac{n}{50} + VCI_1 = VCI_n \ (round\ VCI\ estimates\ up)$$

where—

n = *number of vehicles*

Note. VCI_n provides a rough estimate for the required minimum RCI for 2 to 49 vehicles. This approach involves a linear interpolation simplification. The change in the required RCI soil strength is larger for the smaller number of vehicles than it is for the larger number of vehicles, but this approach is easier to estimate by hand and produces fair approximations in many cases.

Example

Estimate the number of M1A1 tanks that can cross a level area with fine-grained soil. (See TM 3-34.48-2.) Given—

CI = *65 (critical layer)*
RI = *0.80 (critical layer)*
VCI_{50} = *58*
VCI_1 = *25*

$$RCI = 65 \times 0.80 = 52$$
$$VCI_{50} - VCI_1 = 58 - 25 = 33$$
$$33 \times \frac{n}{50} + 25 = VCI_n$$

where—

n = *number of vehicles*

To estimate the number of tanks that can cross the area, the number of vehicles is adjusted using trial and error until VCI_n is equal to the RCI or an increase in the number of vehicles will cause VCI_n to exceed the RCI.

If $n = 40$, then $VCI_n = 51.4$; or if $n = 41$, then $VCI_n = 52.1$

Since VCI estimates are rounded up, the maximum number of M1A1 tanks that can cross the area is approximately 40 (a rough estimate).

Performance of Vehicles on Level Terrain

10-56. As previously described, a comparison of the RCI with the VCI indicates that a specific vehicle can negotiate a given soil condition for a specified 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 120×0.60, or an RCI of 72 under traffic. Therefore, this soil is not trafficable for vehicles with a VCI greater than 72.

One-Pass Performance

10-57. The ability of a vehicle to make one pass when operating at a slow speed in a straight line on level terrain is assured if the RCI of the area is greater than the VCI for one pass (VCI_1).

Example

Estimate if an M1A1 tank can complete one pass on a level, fine-grained soil. (Use TM 3-34.48-2, to determine VCI_1.) Given—

CI = 50 (critical layer)
RI = 0.70 (critical layer)
VCI_1 = 25

$$RCI = 50 \times 0.70 = 35$$

Because the RCI is greater than VCI_1 (35 is greater than 25), the M1A1 tank can complete one pass. Immobilization of a vehicle will likely occur the moment a vehicle enters an area where the RCI is less than VCI_1. Immobilization may occur even when the RCI is slightly greater than VCI_1 if water on the soil surface causes excessive sinkage or slipperiness.

Fifty-Pass Performance

10-58. The ability of a given vehicle to make 50 passes when operating at a slow speed in a straight line through the same ruts (or 50 of the same vehicle type following in straight-line formation) on level terrain is assured if the RCI of the area is greater than the VCI for 50 passes (VCI_{50}).

Example

Estimate if an M923 5-ton cargo truck can complete 50 passes on a level, fine-grained soil. (Use TM 3-34.48-2 to determine VCI_{50}.) Given—

CI = 80 (critical layer)
RI = 0.82 (critical layer)
VCI_{50} = 68

$$RCI = 80 \times 0.82 = 65.6 = 65\ (always\ round\ RCI\ values\ down)$$

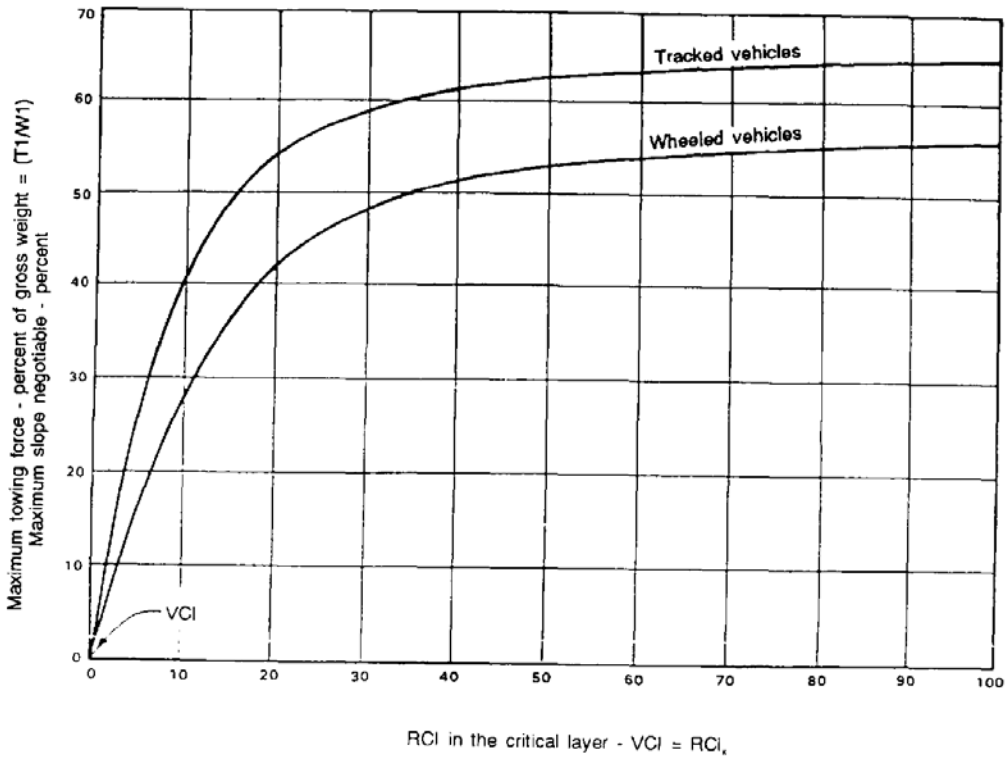
Because the RCI is less than VCI_{50} (65 is less than 68), the M923 truck cannot complete 50 passes. Immobilization of a vehicle will likely occur before 50 passes when the RCI is less than VCI_{50}. Immobilization may occur before 50 passes even when the RCI is slightly greater than VCI_{50} if water on the soil surface causes excessive sinkage or slipperiness.

PERFORMANCE OF VEHICLES NEGOTIATING SLOPES

10-59. The performance relations presented herein are applicable for self-propelled tracked and all-wheel-drive vehicles.

One-Pass Performance

10-60. The maximum slope negotiable (expressed in percent grade) and the maximum towing force divided by the gross vehicle weight for a given RCI 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 estimated from figure 10-8. 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 the specific soil type. NRMM accounts for the effects of slipperiness on slopes, but those procedures are beyond the scope of this manual.



Legend:

RCI	rating cone index
VCI	vehicle cone index

Figure 10-8. One-pass performance curves for self-propelled vehicles operating in fine-grained soils or remoldable sands

Example

Determine the maximum slope that an M1A1 tank can climb on one pass where the slope consists of fine-grained soil. (See TM 3-34.48-2.) Given—

CI = *100 (critical layer)*
RI = *0.85 (critical layer)*
VCI_1 = *25*

$$RCI = 100 \times 0.85 = 85$$
$$RCI_X = RCI - VCI_1 = 85 - 25 = 60$$

Using figure 10-8, page 10-19, at $RCI_x = 60$, the maximum slope equals 63 percent. Therefore, under the stated conditions, the maximum slope the M1Al tank can negotiate for one pass is 63 percent.

Example

Determine the maximum slope that an M923 truck can climb in one pass where the slope consists of remoldable sand. (See TM 3-34.48-2.) Given—

CI = *93 (critical layer)*
RI = *0.40 (critical layer)*
VCI_1 = *30*

$$RCI = 93 \times 0.40 = 37$$
$$RCI_X = RCI - VCI_1 = 37 - 30 = 7$$

Using figure 10-8 at $RCI_x = 7$, the maximum slope equals 21 percent. Therefore, the M923 truck can climb a slope less than or equal to 21 percent for one pass under the stated conditions.

Fifty-Pass Performance

10-61. The maximum slope negotiable (expressed in percent grade) and the maximum towing force divided by the gross vehicle weight for a given RCI are essentially equal. Therefore, when the RCI is known, the maximum slope negotiable by a specific vehicle for 50 passes (or 50 of the same vehicle type following in straight-line formation) can be estimated from figure 10-9. The differences in the properties of various soils produce some differences in vehicle performance. NRMM predicts performance in fine-grained soils based on the specific soil type. NRMM accounts for the effects of slipperiness on slopes, but those procedures are beyond the scope of this manual.

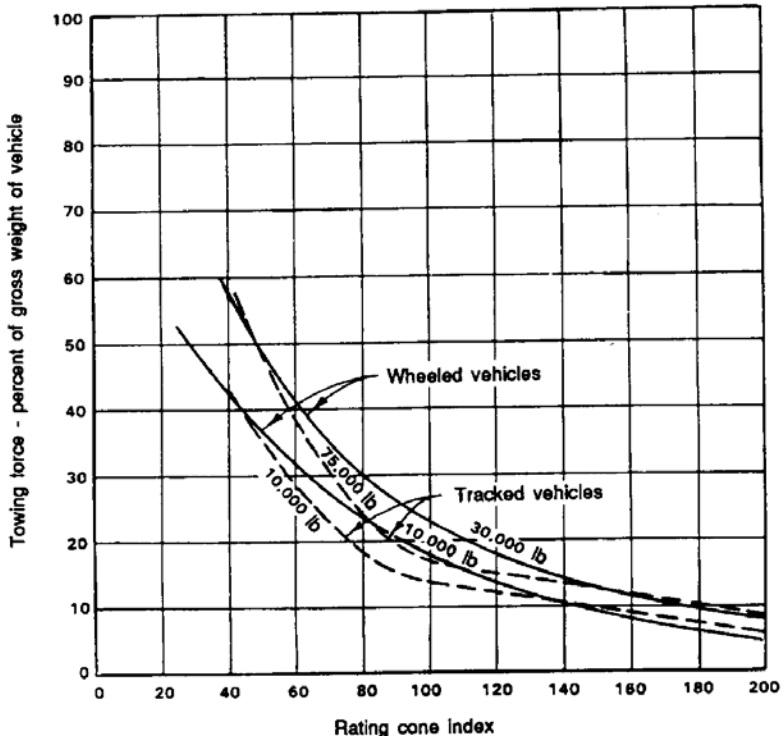
Example

Estimate the maximum slope that an M1A1 tank can climb for 50 passes where the slope consists of fine-grained soil. Use TM 3-34.48-2 to determine VCI_{50}. Given—

CI = 100 (critical layer)
RI = 0.85 (critical layer)

$$RCI = 100 \times 0.85 = 85$$
$$RCI_X = RCI - VCI_{50} = 85 - 58 = 27$$

Using figure 10-9 the maximum slope ($RCI_x = 27$) is 50 percent. Therefore, the maximum slope the M1A1 can negotiate for 50 passes under the given conditions is 50 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.

Legend:

lb pounds

Figure 10-9. Fifty-pass performance curves for self-propelled vehicles operating in fine-grained soils or remoldable sands

Example

Estimate the maximum slope that an M923 5-ton cargo truck can climb for 50 passes where the slope consists of remoldable sand. Given—

CI = 93 (critical layer)
RI = 1.00 (critical layer)

$$RCI = 93 \times 1.00 = 93$$
$$RCI_X = RCI - VCI_{50} = 93 - 68 = 25$$

Using figure 10-9 at RCI_x = 25, the maximum slope is 35 percent. Therefore, the maximum slope that the M923 truck can climb for 50 passes under the stated conditions is 35 percent.

PERFORMANCE OF VEHICLES TOWING OTHER VEHICLES ON LEVEL TERRAIN

10-62. The performance relations presented are applicable for self-propelled tracked and all-wheel-drive vehicles. The required towing force in soft areas where vehicles are bogged down may equal or exceed the weight of the towed vehicle, especially if the towed vehicle has substantial hull or chassis dragging (or bulldozing). This may occur when the VCI of the towed vehicle is much higher than the towing vehicle.

One-Pass Performance

10-63. When the RCI is equal to VCI_1, the soil has just enough shear strength for a given vehicle to overcome its motion resistance for one pass. If the vehicle is required to tow another vehicle, additional shear

strength is required to produce the necessary thrust to overcome motion resistance (required towing force) of the towed vehicle. Thus—

$$RCI - VCI1 = RCI_X$$

10-64. The equation above provides a measure of the additional shear strength that is available for a vehicle to develop a towing force for one pass. Therefore, when the RCI and VCI_1 are known, the ability of one vehicle to tow another for one pass can be determined.

10-65. Two one-pass performance curves—one for self-propelled tracked vehicles and one for self-propelled wheeled vehicles—are shown in figure 10-8, page 10-19. The maximum towing force (expressed as a percentage of the vehicle gross weight) that can be developed by a vehicle on level terrain is related to RCI_X. A performance curve for vehicles when towed for one pass on level terrain is shown in figure 10-10 where the required towing force (expressed as a percentage of the vehicle gross weight) is related to RCI_X.

Example

Estimate if an M1A1 tank can tow an M923 5-ton cargo truck for one pass on level fine-grained soil where the shear strength is such that—

CI	=	100 (critical layer for the tank) 50 (critical layer for the truck)
RI	=	0.70 (critical layer for the tank) 0.70 (critical layer for the truck)

(See TM 3-34.48-2.)

Given for the tank—

VCI_1	=	25
Gross weight	=	125,000 pounds
Grousers	<	1 1/2 inches

$$RCI = 100 \times 0.70 = 70$$
$$RCI_X = 70 - 25 = 45$$

Using figure 10-8 at $RCI_X = 45$, the maximum towing force = 63% of 125,000 pounds.

$$0.63 \times 125{,}000 = 78{,}750\ pounds$$

Given for the truck—

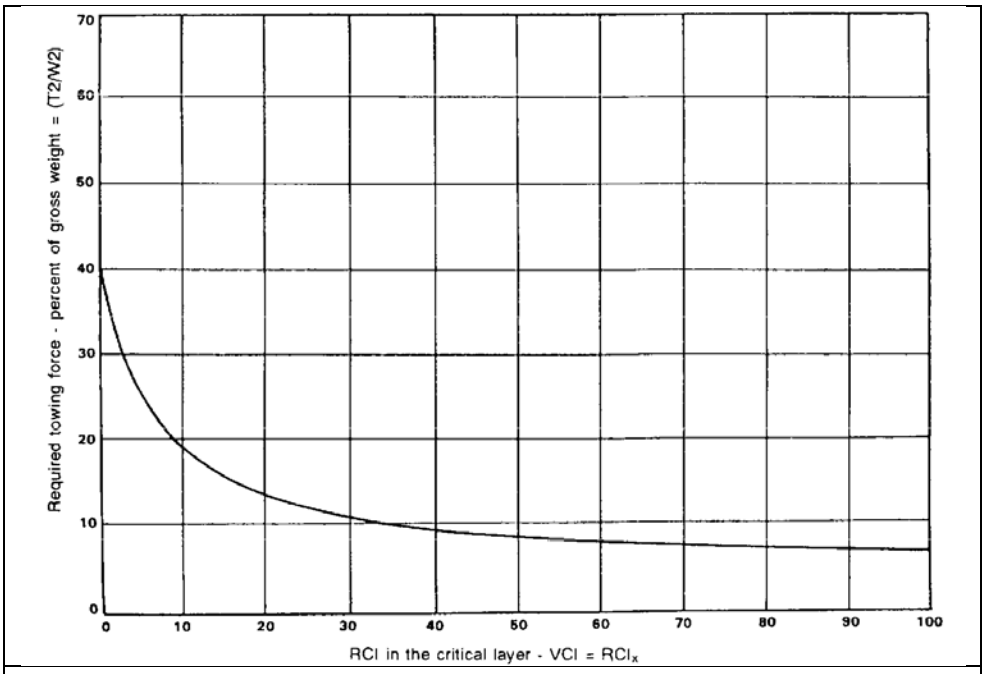
VCI_1	=	30
Gross weight	=	32,500 pounds

$$RCI = 50 \times 0.70 = 35$$
$$RCI_X = 35 - 30 = 5$$

Using figure 10-10, page 10-23, at $RCI_X = 5$, the required towing force = 25% of 32,500 pounds.

$$0.25 \times 32{,}500 = 8{,}125\ pounds$$

Because the available towing force (78,750 pounds) of the tank exceeds the required towing force (8,125 pounds) for the truck, the tank can tow the truck for one pass under the stated conditions.



Legend:

RCI	rating cone index
VCI	vehicle cone index

Figure 10-10. One-pass performance curves for vehicles towed in level, fine-grained soils or remoldable sands

Example

Estimate if an M923 5-ton cargo truck can tow an M1A1 tank for one pass on level fine-grained soil. Given—

CI = 95 (critical layer for the truck and the tank)
RI = 1.00 (critical layer for the truck and the tank)

$$RCI = 95 \times 1.00 = 95$$

For the M923 truck—

$$RCI_X = RCI - VCI_1 = 95 - 30 = 65$$

Using figure 10-8, page 10-19, at $RCI_X = 65$, the towing force equals 54.6% of 32,500 pounds.

$$0.546 \times 32{,}500 \; = \; 17{,}745 \; pounds$$

For the M1A1 tank—

$$RCI_X = RCI - VCI_1 = 95 - 25 = 70$$

Using figure 10-10 at $RCI_X = 70$, the required towing force equals 8% of 125,000 pounds.

$$0.08 \times 125{,}000 \; = \; 10{,}000 \; pounds$$

Because the available towing force of the truck (17,745 pounds) exceeds the force required to tow the tank (10,000 pounds), the truck can tow the tank for one pass under the stated conditions.

Fifty-Pass Performance

10-66. When the RCI is equal to VCI_{50}, the soil has just enough shear strength for a given vehicle to overcome its motion resistance for 50 passes. If the vehicle must tow another vehicle, additional shear

strength is required to produce the thrust needed to overcome the motion resistance (required towing force) of the towed vehicle. Thus—

$$RCI - VCI_{50} = RCI_X$$

10-67. This equation provides a measure of additional shear strength available for a vehicle to develop a towing force for 50 passes. Therefore, when the RCI and VCI_{50} are known, the ability of one vehicle to tow another for 50 passes can be determined.

10-68. Three fifty-pass performance curves—two for self-propelled tracked vehicles (segregated based on grouser height) and one for self-propelled wheeled vehicles—are shown in figure 10-9, page 10-21. The maximum towing force (expressed as a percentage of the gross vehicle weight) that can be developed by a vehicle on level terrain is related to RCI_X. Four performance curves—two for tracked vehicles and two for wheeled vehicles (both segregated based on vehicle weight) when towed for 50 passes on level terrain where the required towing force (expressed as a percentage of the gross vehicle weight) is related to RCI are shown in figure 10-11.

Example

Estimate if an M1A1 tank can tow an M923, 5-ton cargo truck for 50 passes on a level, fine-grained soil. Given—

CI	=	100 (critical layer for the tank)
		60 (critical layer for the truck)
RI	=	0.80 (critical layer for the tank)
		0.80 (critical layer for the truck)

For the M1A1 tank—

VCI_{50}	=	58
Gross weight	=	125,000 pounds
Grousers	<	1 1/2 inches

For the M923 truck—

VCI_{50}	=	68
Gross weight	=	32,500 pounds

$$RCI(tank) = 100 \times 0.80 = 80$$
$$RCI(truck) = 60 \times 0.80 = 48$$

The maximum towing force (T1) of the tank is read from the curve in figure 10-9, labeled *tracked vehicles* with grousers less than 1 1/2 inches. Using this curve, where—

$$RCI_X = 80 - 58 = 22$$

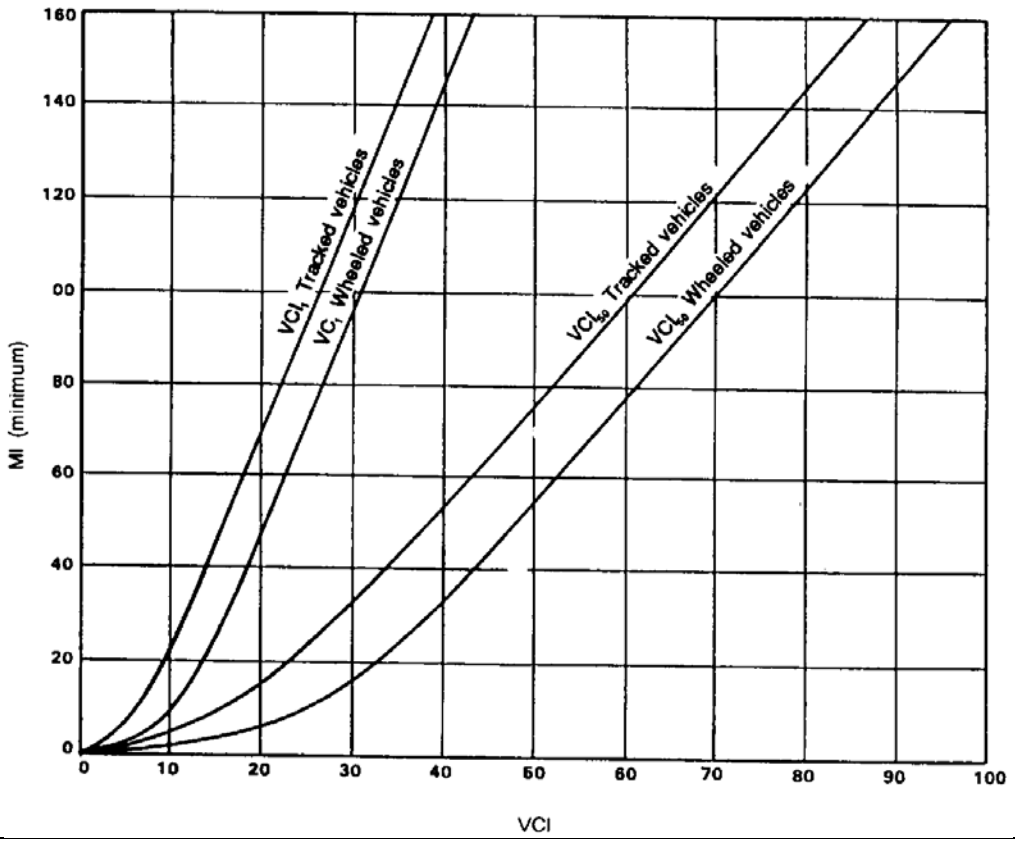
—it is estimated that the tank can tow 25% of its weight. Thus—

$$25\%\ of\ 125{,}000 = 0.25 \times 125{,}000 = 31{,}250\ pounds$$

The required towing force (T2) for the M923 truck is read from the curve in figure 10-11 for 30,000 pounds for wheeled vehicles. On this curve (at RCI = 48)—

$$T2 = 49\%\ of\ 32{,}000 = 0.49 \times 32{,}500 = 15{,}925\ pounds$$

Because the available towing force (31,250 pounds) of the M1A1 tank is greater than the required towing force (15,925 pounds) for the M923 truck, the tank can tow the truck for 50 passes under the stated conditions.



Legend:

MI	mobility index
VCI	vehicle cone index

Figure 10-11. Fifty-pass performance curves for vehicles towed in level, fine-grained soils or remoldable sands

Example

Estimate if an M923 5-ton cargo truck can tow an M1A1 tank for 50 passes on a level, fine-grained soil where shear strength for the critical layers are the same for both vehicles. Given—

CI = *95*
RI = *1.00*

$$RCI = 95 \times 1.00 = 95$$
$$RCI_X = 95 - 68 = 27\ (M923\ truck)$$

The maximum towing force (T1) of the M923 truck is read from the curve labeled *Wheeled vehicles* in figure 10-9, page 10-21. On this curve—

$$RCI_X = 27, T1 = 37\%\ of\ 32{,}500 = 0.37 \times 32{,}500 = 12{,}025\ pounds$$

Example (continued)

The required towing force (T2) for the M1A1 tank is read from the curve in figure 10-11, page 10-25, that is labeled *75,000 pounds* for tracked vehicles. On this curve—

$$RCI = 95, T2 = 18\%\ of\ 125{,}000\ pounds = 0.18 \times 125{,}000 = 22{,}500\ pounds$$

Because the available towing force (12,025 pounds) of the M923 truck is less than the required towing force (22,500 pounds) of the M1A1 tank, the truck cannot tow the tank for 50 passes under the stated conditions.

PERFORMANCE OF VEHICLES TOWING OTHER VEHICLES UP SLOPES

10-69. The performance relations presented are applicable for self-propelled tracked and all-wheel-drive wheeled vehicles. The maximum slope negotiable for one vehicle towing another vehicle can be estimated using the following formula:

$$G_{Max} = \frac{T1 - T2}{W1 + W2}$$

where—

G_{Max} = *maximum slope negotiable (percent grade)*
T1 = *maximum towing force of the towing vehicle (pounds)*
T2 = *force required to tow the towed vehicle on level terrain (pounds)*
W1 = *weight of the towing vehicle (pounds)*
W2 = *weight of the towed vehicle (pounds)*

One-Pass Performance

10-70. An example for the maximum slope negotiable for one-pass performance follows.

Example

Estimate the maximum slope that can be negotiated by an M1A1 tank towing an M923 truck for one pass where the slope consists of fine-grained soil. Given—

CI = 100 (critical layer for the tank and the truck)
RI = 0.85 (critical layer for the tank and the truck)

$$RCI = 100 \times 0.85 = 85$$

For the M1A1 tank—

$$RCI_X = RCI - \text{-}VCI_1 = 85 - 25 = 60$$

In figure 10-8, page 10-19, at $RCI_X = 60$, the maximum towing force (T1) = 63% of 125,000.

$$0.63 \times 125{,}000 = 78{,}750\ pounds$$

For the M923 truck—

$$RCI_X = RCI - \text{-}VCI_1 = 85 - 30 = 55$$

Example (continued)

In figure 10-10, page 10-23, at RCI_X = 55, the required towing force (T2) = 8.3% of 32,500.

$$0.083 \times 32{,}500 \ = \ 2{,}698 \, pounds$$

$$G_{Max} = \frac{T1 - T2}{W1 + W2} = \frac{78{,}750 - 2{,}698}{125{,}000 + 32{,}500} = 0.483 = 48\% \, grade$$

(round slope estimates down)

Thus, the maximum slope negotiable by the M1A1 tank towing the M923 truck for one pass under the stated conditions is 48 percent.

Fifty-Pass Performance

10-71. Two examples for the maximum slope negotiable for fifty-pass performance follow.

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—

CI = 100 (critical layer for the tank)
80 (critical layer for the truck)
RI = 0.85 (critical layer for the tank)
0.80 (critical layer for the truck)

$$RCI = 100 \times 0.85 = 85 \, for \, the \, tank$$
$$RCI_X = RCI - VCI_{50} = 85 - 58 = 27 \, for \, the \, tank$$
$$RCI = 80 \times 0.80 = 64 \, for \, the \, truck$$

Using figure 10-9, page 10-21, the maximum towing force (T1) of the MIA1 tank at—

$$RCI_X = 27 = 45\% \, of \, 125{,}000 = 0.45 \times 125{,}000 = 56{,}250 \, pounds$$

Using figure 10-11, page 10-25, the required towing force (T2) for the M923 truck at—

$$RCI = 64 = 38\% \, of \, 32{,}500 = 0.38 \, H \, 32{,}500 = 12{,}350 \, pounds$$

$$G_{Max} = \frac{T1 - T2}{W1 + W2} = \frac{56{,}250 - 12{,}350}{125{,}000 + 32{,}500} = 0.279 = 27\% \, grade$$

(round slope estimates down)

Thus, the maximum slope negotiable by the M1A1 tank towing the M923 truck for 50 passes under the given conditions is 27%.

Example

Estimate the maximum slope negotiable by an M923, 5-ton cargo truck towing an M998 high mobility, multipurpose wheeled vehicle for 50 passes where the slope consists of fine-grained soil. Given—

CI = 120 (critical layer for both vehicles)
RI = 1.00 (critical layer for both vehicles)

$$RCI = 120 \times 1.00 = 120\ for\ both\ vehicles$$
$$RCI_X = RCI - VCI_{50} = 120 - 68 = 52\ for\ the\ M923\ truck$$

Using figure 10-9, page 10-21, where the RCI_X = 52, the maximum towing force (T1) of the M923 truck is—

$$47\%\ of\ 32{,}500 = 0.47 \times 32{,}500 = 15{,}275\ pounds$$

Using figure 10-11, page 10-25, the required towing force (T2) for the M998 multipurpose-wheeled vehicle at RCI = 120 is 13% of *7,500 = 0.13 X 7,500 = 975 pounds*.

$$G_{Max} = \frac{T1 - T2}{W1 + W2} = \frac{15{,}275 - 975}{32{,}500 + 7{,}500} = 0.358 = 35\%\ grade$$

(round slope estimates down)

Thus, the maximum slope negotiable by the M923 truck towing an M998 multipurpose wheeled vehicle for 50 passes under the given conditions is 35 percent.

PERFORMANCE OF VEHICLES TOWING TRAILERS ON LEVEL TERRAIN AND UP SLOPES

10-72. The performance of vehicles towing trailers and of tractor-trailer combinations can be predicted using the comprehensive NRMM. However, these computations can be reliably made only through the integration of complex considerations requiring calculations on an axle-by-axle basis. Therefore, these computations are beyond the scope of this manual. Procedures used in the examples for vehicles towing other vehicles should not be extended for estimating a single VCI for vehicles towing trailers or for tractor-trailer combination vehicles.

OPERATION IN COARSE GRAINED SOILS

10-73. Coarse-grained soils present trafficability problems different from those encountered in fine-grained soils. Some important differences are that—

- 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 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 first-pass ruts.

10-74. 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 that RI equals 1.0).

10-75. Use the following procedures to ensure that 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.

10-76. The presence of vegetation in coarse-grained-soil areas indicates that the soil is stabilized and is of high trafficability. This effect is reflected in high CI readings. Testing to date has not permitted the development of CI performance relations for tracked vehicles because tracked vehicles are able to travel on level, coarse-grained soils. Furthermore, the effects of soil strength on the performance of a given tracked vehicle in coarse-grained soils are minimal.

10-77. 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

10-78. 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, number of tires, tire construction, vehicle characteristics, and the soil condition (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, dry-to-moist, poorly-graded sands are evaluated using the coarse-grained vehicle performance relationships contained in the NRMM or other NRMM-based mobility models, such as the Condensed Army Mobility Model System.

10-79. Other sandy soils with appreciable quantities of fine material 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 entered 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 for these soils). Speed predictions can then be made for these areas.

10-80. In coarse-grained soils, the performance of wheeled vehicles is affected mostly by tire pressure. To optimize vehicle performance, tire pressures should be reduced to mud, sand, and snow pressures 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 floatation and traction are maximized for the terrain conditions encountered.

TRACKED VEHICLES ON LEVEL TERRAIN

10-81. As a general rule, tracked vehicles are able to travel on level, coarse-grained soils regardless of soil strength. Performance predictions in the NRMM are made for two categories of tracked vehicles: flexible tracks (found on most military vehicles) or girderized tracks (found on most bulldozer-type vehicles). Vehicles with girderized tracks generally produce higher tractive forces on SP soils than do flexible-tracked vehicles.

VEHICLE CONE INDEX CALCULATIONS

10-82. 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. Immobilization can easily occur in these soils, especially if the soils are dry and loose. Prepare for such emergencies. Tracked vehicles generally do not suffer immobilization on level SP soils.

TRAFFICABILITY DATA

10-83. The objective of mapping trafficability data is to provide commanding officers with an estimate of an area trafficability before 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 10-12. The maps produced by the techniques described in the following paragraphs are elementary when compared with the complicated and comprehensive maps now in production for use with the NRMM.

ESTIMATING

10-84. Trafficability can be estimated if weather conditions, soils, and area topography are generally known. Weather and climatic information are usually available, even for remote areas, from meteorological records, climatology textbooks, or interaction with the local population. Soils and topography data may be obtained from topographic, soils, and geologic maps; aerial photographs; or interaction with the local population.

10-85. 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 photographs.

Weather Conditions

10-86. For estimating trafficability, consider the general weather conditions: the dry season and the wet season.

Dry Season

10-87. A dry season is defined as a time when climatic and vegetation factors combine to produce 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 soil is high because of long days, high temperatures, and few clouds. Water is rapidly extracted from the soil and transpired into 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. Arid climates may be considered to be constantly in a dry season.

10-88. During the dry season, fine-grained soils and remoldable sands are usually 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 wet, coarse-grained soils except quicksand. Even in the dry season, a high water table that results from underground springs, low-lying and poorly drained soils, or other causes effects trafficability of soil types.

Wet Season

10-89. A wet season is defined as a time when 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 keep soil moisture near a maximum value. Melting snow and ice may produce wet soil conditions. Wet seasons may occur at any time as a result of prolonged rains, floods, or irrigation. Adding moisture to soil affects the strength of that soil; the effect differs with soil types.

Topography and Classification of Soils

10-90. The configurations of the soil surface and soil types in a given area are determined from aerial photographs, geologic maps, soil maps, and topographic maps.



Legend:

N not slippery

Figure 10-12. Photomap with trafficability data

Aerial Photographs

10-91. Elevations and slopes can be estimated by personnel who are properly instructed to read (with the aid of stereo pairs) and interpret information in aerial photographs. Accurate elevations and slopes can be obtained with mechanical equipment by operators trained to use the equipment. Locations for rivers, forests, escarpments, and embankments can also be obtained from aerial photographs.

10-92. The techniques for identifying soils from aerial photographs are so complex that only well-trained personnel can fully use aerial photographs for this purpose. However, personnel with minimal training can obtain general information. 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 (silts and clays).

10-93. In a photograph, 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 photograph and may signify an entirely different condition in another aerial photograph. 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

10-94. 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

10-95. 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. Information 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

10-96. Physical features (rivers, streams, cultivation, forests, roads) can usually be identified from a topographic map. Surface slope estimates can be made from the contour lines (lines passing through points of equal elevation).

10-97. 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 the 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 during the year.

MANUALLY MAPPING SOIL CONDITIONS AND TRAFFICABILITY

10-98. 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 available should the need arise. The following paragraphs describe the proper procedure in detail.

10-99. VCI values vary widely, and it is desirable to present basic terrain data that can be compared directly with those values. The four basic terms that describe trafficability are: soil type, RCI, slope, and slipperiness. Stickiness effects are not considered significant enough to include on maps.

10-100. The soil-type condition is shown as A, B, C, or D (defined in table 10-5). The RCI is shown as a single number; the slope (in percent grade) is shown as a single number; and slipperiness is shown as N (not

slippery), P (slippery when wet), or S (slippery). The four factors may be presented in fractional form with two items in the numerator and two in the denominator as shown in figure 10-12, page 10-31.

Table 10-5. 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

10-101. 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 vehicles with a VCI of 49 or less. The following are the recommended techniques of mapping trafficability:

- 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 10-5. See previous weather conditions.
- Obstacles should be indicated by added numbers in red and 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 important data on soils, topography, and obstacles that cannot readily be shown on the face of the map.

Example

Fifty M60 tanks (102,000 pounds) and 50 M923 trucks (32,500 pounds) are to be moved from Point X to Point Y in the area shown in figure 10-12, page 10-31. Movement must be cross-country because the road net is heavily mined.

- **Step 1.** From TM 3-34.48-2—

$Vehicle$	VCI_1	VCI_{50}
$Tank$	20	48
$Truck$	30	68

- **Step 2.** Examine the possibility of single-file travel through flat terrain. Vehicles can negotiate areas 1 and 6. The RCI of 50 for area 3 will allow passage of 50 tanks but not all 50 trucks. The tanks travel single file from X through area 1, area 3, area 6, to Y. However, 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 = 30, RCI = 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 area 1, area 5, and area 6, provided the slope of area 5 could be negotiated. For this example, it is assumed that the combination of 60% slope and other terrain obstacles in area 2 and area 4 would not allow travel through them.
- **Step 3.** Check the slope-climbing ability (using figure 10-8, page 10-19, and figure 10-9, page 10-21) of both vehicle types in area 5:

$1\ Pass$

$Vehicle$	$Slope$	RCI_X	$RCI_X + VCI_1 = RCI_{Req'd}$
$Tank$	30%	6	$6 + 20 = 26$
$Truck$	30%	11	$11 + 30 = 41$

$50\ Passes$

$Vehicle$	$Slope$	RCI_X	$RCI_X + VCI_{50} = RCI_{Req'd}$
$Tank$	30%	15	$15 + 48 = 63$
$Truck$	30%	20	$20 + 68 = 88$

The tanks can negotiate the 30% 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 with arrows. This example indicates the usefulness of mapped trafficability data in planning operational exercises.

SOIL-TRAFFICABILITY CLASSIFICATION

10-102. 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 the critical vehicles has been determined using TM 3-34.48-2 or computed, when necessary, as previously described.

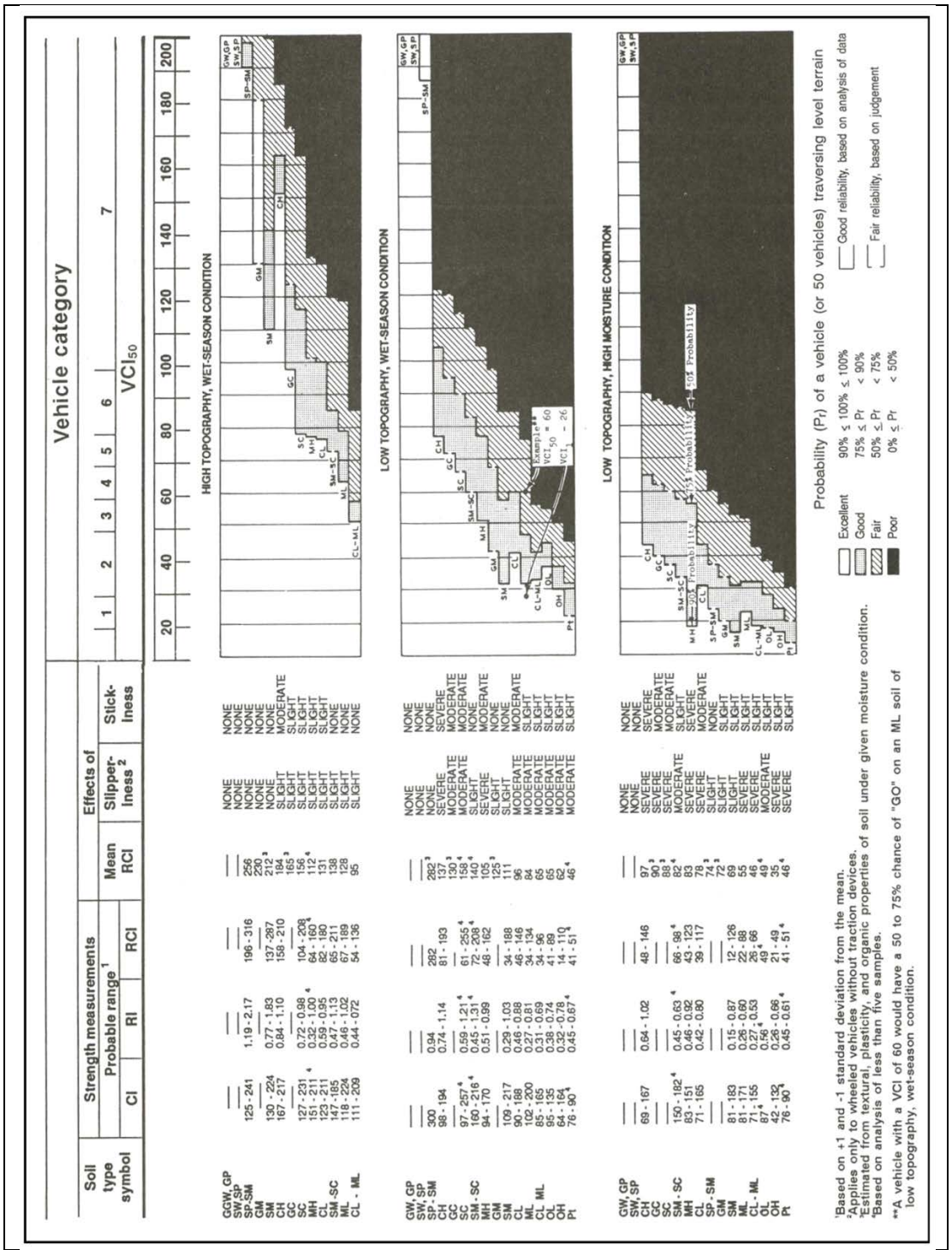
FINE-GRAINED SOILS

10-103. The trafficability classification of fine-grained soils is shown in figure 10-13, page 10-36. The interpretation of the example shown in the *Low Topography, High Moisture Condition* graph in the table for a level area of MH soil is as 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 75 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

10-104. The trafficability classification of coarse-grained soils can be obtained from figure 10-14, page 10-37. The classification interpretation is the same as for the trafficability of fine-grained soils from figure 10-13. To use figure 10-14, identify only the coarse-grained soils (location and origin) and determine the VCI values (must use NRMM or other similar mobility models). Figure 10-14 applies to wheeled vehicles. The effect of the strength of coarse-grained soils on tracked-vehicle performance is negligible.



Soil type symbol	Strength measurements: Probable range[1] CI	RI	RCI	Mean RCI	Effects of: Slipper-iness[2]	Stick-iness
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[3]	SLIGHT	MODERATE
GC	—	—	—	165[3]	SLIGHT	SLIGHT
SC	127 - 231[4]	0.72 - 0.98[4]	104 - 208[4]	156[4]	SLIGHT	SLIGHT
MH	123 - 211	0.59 - 0.95	82 - 180	131	SLIGHT	SLIGHT
CL	147 - 185	0.47 - 1.13	65 - 211	138	SLIGHT	NONE
SM-SC	118 - 224	0.46 - 1.02	67 - 189	128	SLIGHT	NONE
ML	111 - 209	0.44 - 072	54 - 136	95	SLIGHT	NONE
CL - ML						
GW, GP	—	—	—	—	NONE	NONE
SW, SP	—	—	—	—	NONE	NONE
SP, SM	300	0.94	282	282[3]	SEVERE	SEVERE
CH	98 - 194	0.74 - 1.14	81 - 193	137	MODERATE	MODERATE
GC	—	—	—	130[3]	MODERATE	MODERATE
SC	97 - 257[4]	0.59 - 1.21[4]	61 - 255[4]	158[4]	SLIGHT	NONE
SM - SC	160 - 216[4]	0.45 - 1.31[4]	72 - 208[4]	140[4]	SEVERE	MODERATE
MH	94 - 170	0.51 - 0.99	48 - 162	105	SLIGHT	NONE
GM	—	—	—	125[3]	SLIGHT	NONE
SM	109 - 217	0.29 - 1.03	34 - 188	111	SLIGHT	SLIGHT
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]	SLIGHT	SLIGHT
GW, GP	—	—	—	—	NONE	NONE
SW, SP	—	—	—	—	NONE	NONE
CH	69 - 167	0.64 - 1.02	48 - 146	97[3]	SEVERE	SEVERE
GC	—	—	—	90[3]	SEVERE	MODERATE
SC	—	—	—	88[3]	MODERATE	MODERATE
SM - SC	150 - 182[4]	0.45 - 0.63[4]	66 - 98[4]	82[4]	SEVERE	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
ML	81 - 183	0.15 - 0.87	12 - 126	69	SLIGHT	SLIGHT
CL - ML	81 - 171	0.26 - 0.60	22 - 88	55	SLIGHT	SLIGHT
OL	71 - 155	0.27 - 0.53	26 - 66	46	SEVERE	SLIGHT
OH	87[4]	0.56[4]	49[4]	49[4]	SEVERE	SLIGHT
Pt	42 - 132	0.26 - 0.66	21 - 49	35	MODERATE	SLIGHT
	76 - 90[4]	0.45 - 0.61[4]	41 - 51[4]	46[4]	SEVERE	SLIGHT

Legend:

CI	cone index
RCI	rating cone index
RI	remolding index
VCI	vehicle cone index

Figure 10-13. Soil-trafficability classification in USCS terms

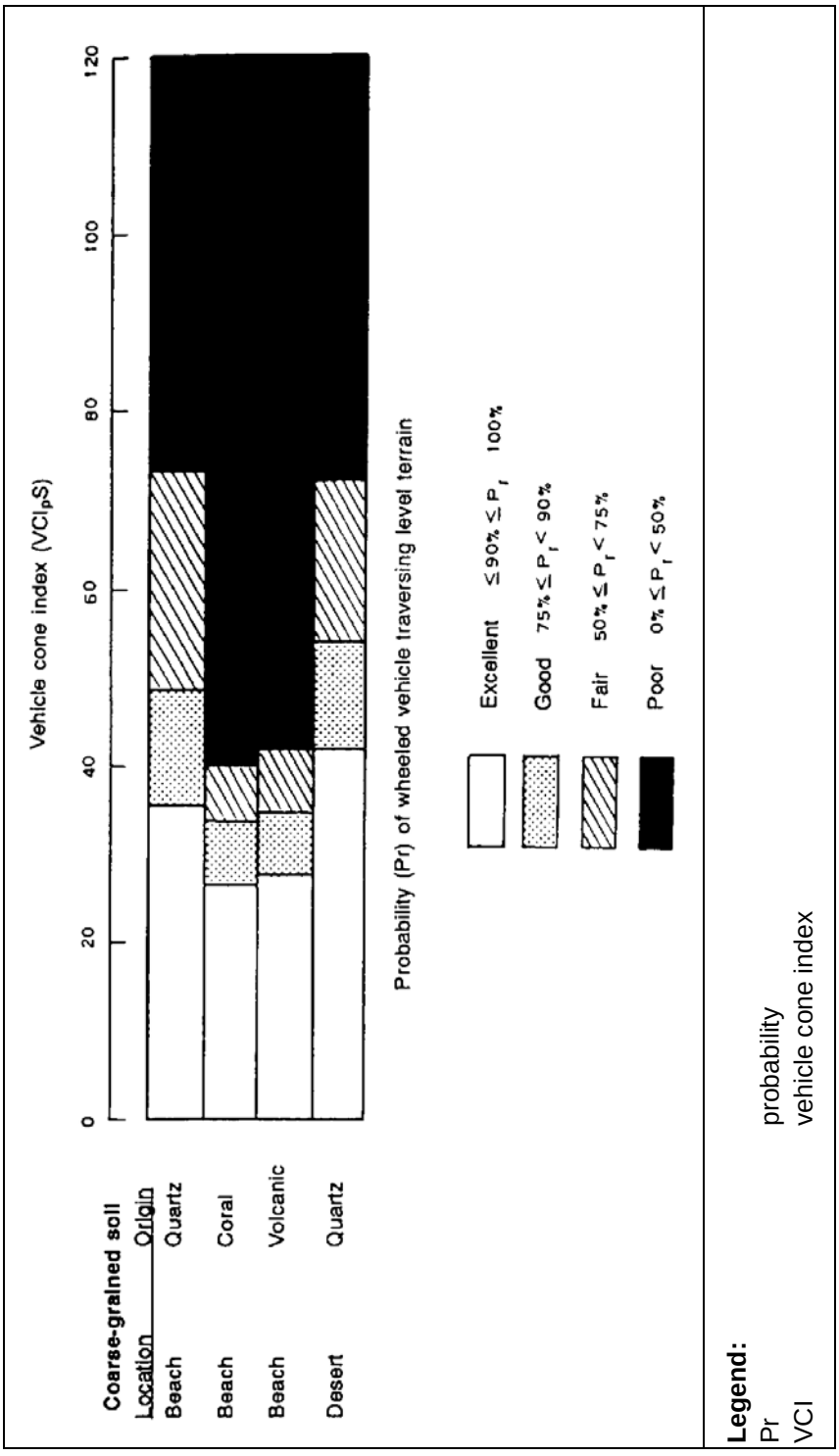


Figure 10-14. Trafficability classification of dry-to-moist, course-grained soils for wheeled vehicles

This page intentionally left blank.

Chapter 11

Maintenance, Repair, and Rehabilitation of Roads, Airfields, and Heliports

Maintenance is the routine prevention and correction of normal operational damage and general deterioration. It is needed to keep road and airfield surfaces and facilities in usable condition. Repair is the work necessary 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

11-1. The purpose of maintenance and repair activities is to keep roads, airfields, and 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.

11-2. 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. When 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 traffic, select alternative facilities or perform repair work at night or during periods of reduced activity. Reopen the facility as soon as practical.
- Remedy the cause before repairing the problem. For example, surface repairs made on a defective subgrade are wasted. Maintenance and repair jobs should include an investigation to find the cause of the damage or deterioration. To ignore the cause invites the reappearance of the 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 tactical requirements, 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

11-3. Maintenance and repair operations include many tasks besides improving the pavement condition. To ensure a comprehensive maintenance and repair operation, incorporate the following tasks:

- Perform routine inspections.
- Stockpile material.
- Perform maintenance and repair of related drainage systems.
- Perform maintenance and repair of the pavement, to include dust and mud control and snow and ice removal.
- Perform miscellaneous tasks, to include maintenance and repair of necessary buildings, structures, and utilities and the operation of necessary utilities.

MAINTENANCE INSPECTIONS

11-4. 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, during spring thaws, and after heavy storms.

Surface Inspection

11-5. 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. Surface defects can be evaluated using UFC 3-270-05 and UFC 3-270-06 for airfield pavements and UFC 3-270-08 for roads and streets.

Drainage Inspection

11-6. Ensure that drainage channels and structures are 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

11-7. Generally, materials required in the maintenance and repair of roads and airfields are 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 emergency maintenance and repair. Arrange stockpiles for quick loading and transporting to the road or runway. Premixed, cold patching material may be prepared as explained in UFC 3-270-01. Maintain small quantities of aggregate in dry storage for concrete patching.

DRAINAGE MAINTENANCE

11-8. Defective or inadequate drainage causes most pavement failures and deterioration. Ponding or delayed runoff of surface water allows seepage into the pavement structure unless the surface is tightly sealed.

Surface Drainage

11-9. Mark areas where ponding occurs on surfaced areas. Correct by filling or raising depressions and by providing outlets for water blocked by high shoulders. Control penetration of storm water through pavement by sealing 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

11-10. Keep shoulders smooth and graded so that water drains 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 to 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

11-11. Keep drainage ditches clear of weeds, brush, sediment, and other debris that obstruct water flow. Maintain line and grade of ditches. Correct sags and minor washouts as they occur. Side ditches can usually be maintained with graders.

11-12. 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, geosynthetic erosion control materials, or concrete.

11-13. Inspect check dams in side ditches and clean them regularly. The weir notch of a check dam must be kept clean or water will cut into the surfaced area at the edge of the dam. (See figure 11-1.) 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.

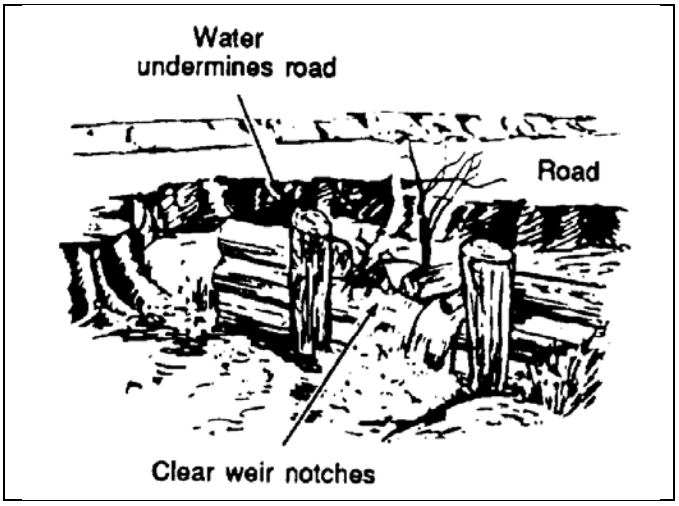


Figure 11-1. Check dam

11-14. Cut-slope interceptor ditches must be considered for 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. 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 11-2, page 11-4, illustrates proper terraced side-hill-cut drainage. For proper design considerations of cut slopes, refer to TM 3-34.64/MCRP 3-17.7G.

Culverts

11-15. Keep culverts clear of debris and sediment. (See figure 11-3, page 11-4.) This prevents water from cutting around or undermining the culverts. Inspect culverts frequently to determine if they are functioning properly. Cleaning by hand is usually necessary after heavy rains.

NONPAVED SURFACES

11-16. Basic maintenance of nonpaved surfaces includes shaping the cross section to maintain adequate drainage and a smooth, compacted surface.

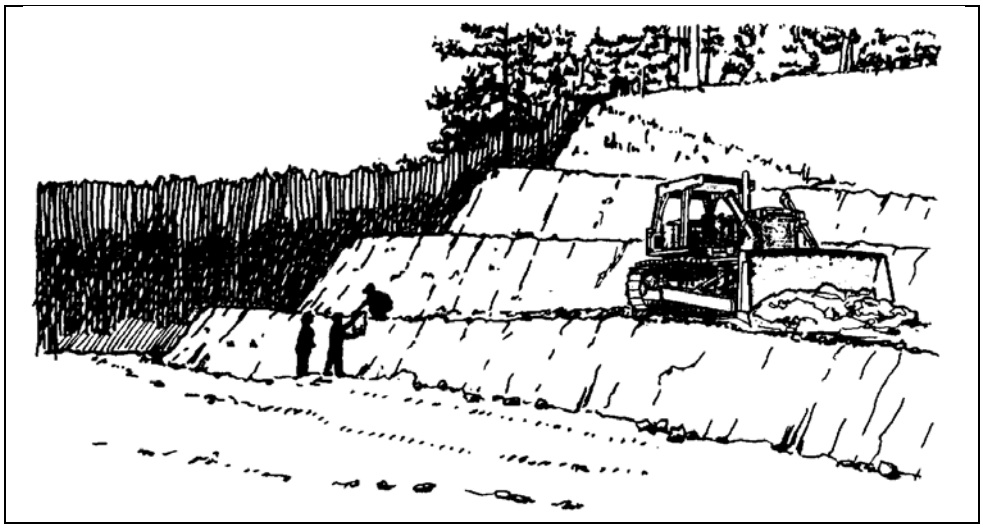


Figure 11-2. Side-hill terracing

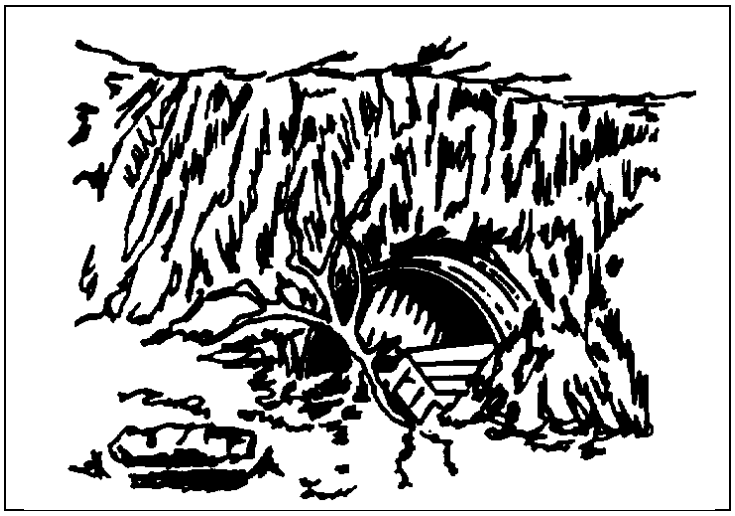


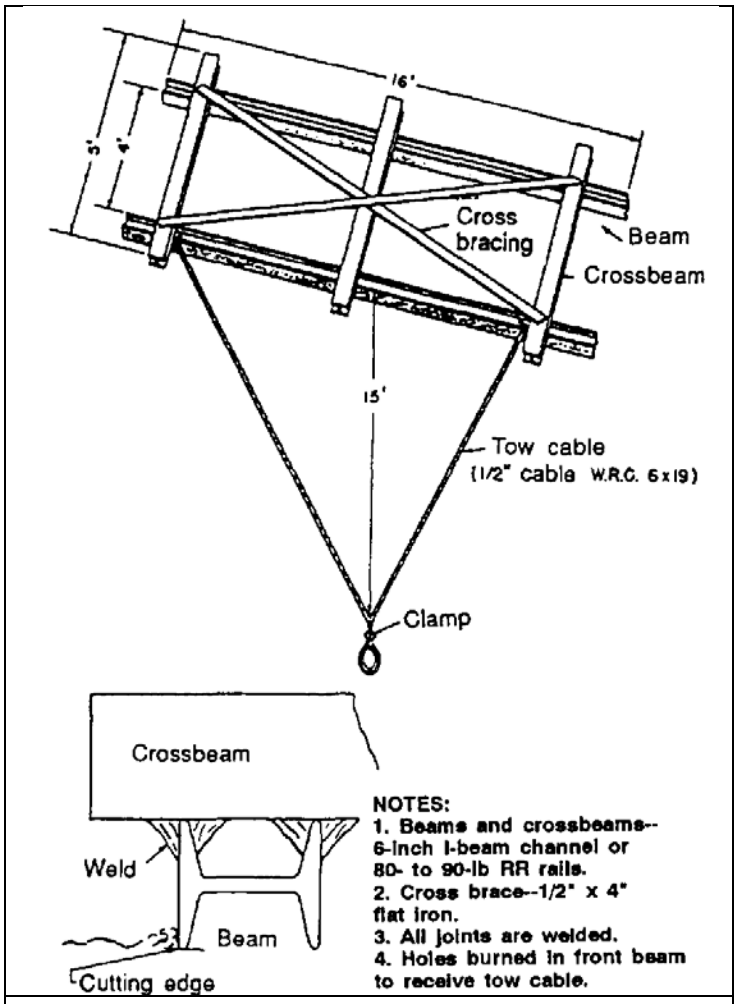
Figure 11-3. Culvert entrance

Equipment

11-17. 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 11-4.

Materials

11-18. Materials removed from ditches (other than silt, some clays, and 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.



Legend:

lb	pounds
RR	railroad

Figure 11-4. Improvised road drag

Procedures

11-19. 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 with a rototiller, plow, or scarifier before compaction.

11-20. Compaction of the graded surface material will reduce maintenance and repair for nonpaved surfaces. The type of roller used will vary with the material compacted. The correct moisture content will result in the most economical compaction.

11-21. 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 pockets of mulch or peat cause surface failures, it may be necessary to remove the objectionable material and replace it with a more stable material. When adequate repairs cannot be made, adding crushed rock or clean gravel can temporarily reinforce soft spots.

11-22. Keep nonpaved surfaces crowned to prevent water from remaining on the surface and saturating the soil. Maintain the crown and superelevation with drags or graders.

11-23. 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 TM 3-34.48-2. Examples of road repairs are shown in figure 11-5.

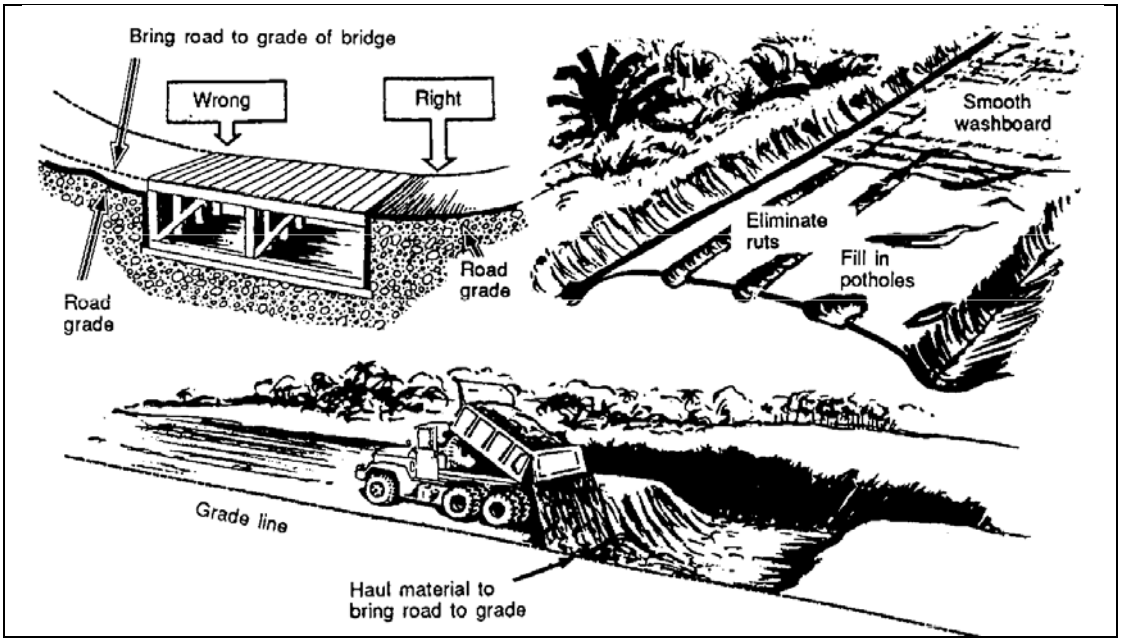


Figure 11-5. Road maintenance problems and proper corrective action

OILED SURFACES

11-24. 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 are necessary before oiling each year.

GRAVEL SURFACES

11-25. Maintenance procedures for gravel surfaces are similar to those 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

11-26. 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 ruts and holes are filled. Do not work on a dry surface.

11-27. 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.

11-28. 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 before fall and winter seasons and prolonged wet periods. Material added or spread on the surface during warm, dry weather is of little value.

Repair of Potholes

11-29. Most potholes are caused by material that was displaced by traffic. Initially, potholes are shallow and 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

11-30. 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.

11-31. Some blade equipment inherently chatters and causes 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 soft subgrade, poor grading of the gravel, poor binder, and an insufficient amount of binder.

11-32. Corrugations can be prevented to a considerable extent by frequent maintenance and the careful use of maintenance equipment. Once corrugations form, only thoroughly scarifying, reshaping, and compacting can remove them. This is called *regrading* and should begin with a thorough cleaning and reshaping of the shoulders and ditches, continuing across the entire roadway.

Use of Calcium Chloride

11-33. 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 one pound of calcium chloride per square yard in the late spring and one-half 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.

11-34. 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

11-35. Traffic areas composed of processed materials (crushed and screened rock, gravel, 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

11-36. The maintenance and repair of bituminous surfaces are discussed in UFC 3-270-01. Some considerations applicable to bituminous-surfaced traffic areas follow.

Inspection

11-37. 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

11-38. Patches should be trimmed square or oblong with straight, vertical sides running parallel and perpendicular to the centerline of the traffic area. (Refer to UFC 3-270-01 for additional information.)

Temporary Repairs

11-39. 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. These patches must be thoroughly compacted and constantly maintained with replacement material. More permanent patching should be accomplished as soon as possible.

Maintenance of Shoulders

11-40. Blade shoulders so that water drains from the surface and ruts and washouts are filled. Grade shoulder material flush against or slightly below pavement edges to restrict water seepage to the subgrade and 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

11-41. 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 are 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

11-42. 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

11-43. Edge *raveling* is caused by water softening the foundation material. Before proceeding with patching operations, reconstruct the shoulder or lower the subdrainage so that this condition will not recur. Then, build up the foundation. The patching mixture should conform to the surrounding area.

CORAL SURFACES

11-44. The maintenance of well-built coral traffic areas is relatively simple. Use fresh, raw coral with the proper moisture content for the repair material. The best time to perform maintenance is during or after a rain storm, while the coral is wet. Fill low spots, ruts, and potholes by shoveling or dumping coral directly from a truck onto the low spots. If rolled while wet, these patches will bond onto the original material almost without a mark. Salt water is usually available where coral is present, and salt water promotes better bonding than fresh water.

11-45. 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. 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 the drainage system is essential.

RIGID PAVEMENTS

11-46. Maintenance of rigid pavements is covered in UFC 3-270-03 and UFC 3-270-04. Frequently inspect concrete pavement to detect early signs of failure, and make prompt repairs to prevent minor defects from spreading.

CRATER REPAIR

11-47. Bombs, shells, land mines, and cratering charges can produce extensive craters in traffic areas. Surface damage does not present an 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.

11-48. Satisfactory repairs require the restoration of subgrade stability to support traffic and prevent undue surface settling after repairs have been completed. Use the following steps for crater repairs:

- **Step 1.** Remove surfacing from around the crater edge that is damaged or not firmly bonded to the base course.
- **Step 2.** Trim the surface and base course to a sound, vertical edge.
- **Step 3.** Remove water, mud, and debris from the crater.
- **Step 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.
- **Step 5.** Repair the base course and wearing surface.

11-49. Gravel; rock; masonry debris; sandy soil; or other suitable, stable materials can be used to fill craters. (See figure 11-6.) 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 UFC 3-270-07.)

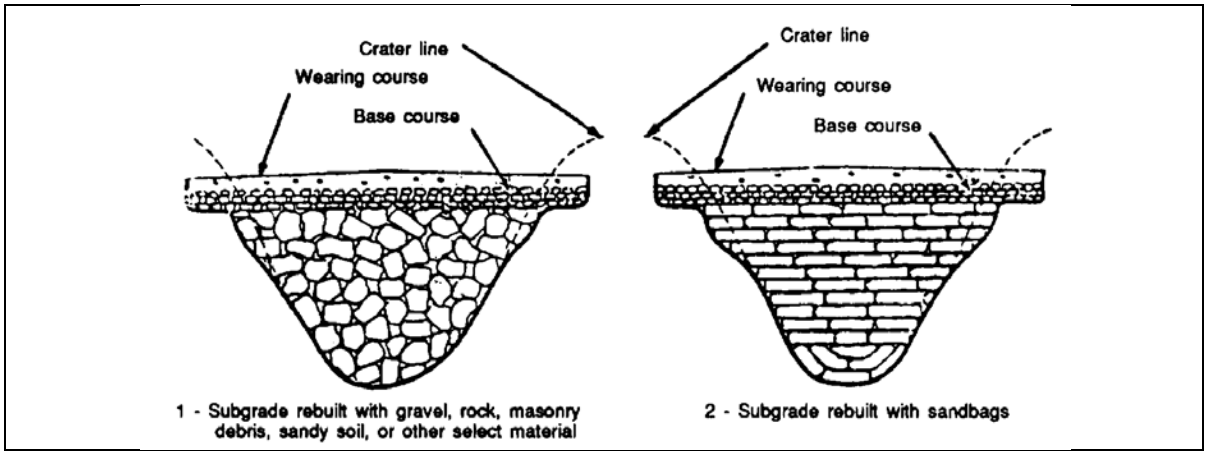


Figure 11-6. Crater backfill materials

ROAD MAINTENANCE

11-50. The importance of preventive maintenance and the necessity for prompt maintenance cannot be overemphasized. Neglect and delay permit traffic and weather to turn minor defects into major problems. Progressive road failure is serious. The more serious a failure is, the more quickly it deteriorates.

11-51. In forward areas, extensive repairs are often necessary before roads can be used. Combat engineer units usually do expedient work. 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 take over the work of additional repair and maintenance. Expedient repairs that were previously made are supplemented or replaced by more permanent work. Surfaces are brought to a standard that will withstand the required use and maintenance becomes routine.

MAINTENANCE PATROLS

11-52. Adequate maintenance requires a workable maintenance organization. Usually, engineer units establish a patrol system that covers the road net they are responsible for. 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 tables of organization and equipment. The platoon furnishes reinforcements (personnel or equipment) as needed.

11-53. Assign each patrol to a specific area. Organize as many patrols as necessary to adequately cover the total area of operations. 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

11-54. 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.

11-55. One patrol consists of a normal squad that is equipped with a dump truck, grader, and hand tools. This patrol can handle maintenance and minor repairs normally encountered 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.

11-56. The other plan calls for the assignment of a one to three person patrol, a grader, and hand tools. This crew can handle 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 when material must be hauled or surface patching is needed.

11-57. Winter weather; severe storms; heavy, destructive enemy action; and other conditions demand that 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

11-58. 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 (eliminate ruts, potholes, washboards).
- Maintain road shoulders and ditches.
- Make frequent, thorough inspections of road conditions.
- Report the need for repair work more significant than the patrol is equipped or manned to handle to higher headquarters.

Inspection and Supervision

11-59. An officer or senior noncommissioned officer 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

11-60. The engineer sending out maintenance patrols should keep sufficient equipment and personnel available to send out repair crews to handle 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 area must not be neglected.

MAINTENANCE WITH TRAFFIC

11-61. 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 temporary and emergency repairs can be made to maintain traffic flow.

Traffic Control

11-62. Repairs should be made on one-half of 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.

11-63. When single-lane traffic must be used, control traffic using 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, 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 the 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.

11-64. 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 labor, material, and equipment should be on hand to complete the work as rapidly as possible.

Bypasses and Detours

11-65. Bridge or pavement failures or the destruction of part of a roadway by floods or combat action may make part or the entire 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.

11-66. Conduct 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 conditioned, to handle traffic for the period when it will be used. Use existing roads when possible. Construct short sections of connecting roads, if necessary.

11-67. 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.

11-68. Direct traffic by placing signs at detour entrances, road intersections, and turns. 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 FM 3-34.170/MCWP 3-17.4 for the types and posting of road signs.

Safety of Maintenance Personnel

11-69. 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 crewmembers 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

11-70. Winter weather may present special problems in JOA maintenance. Regions of heavy snowfall require special equipment and material to keep pavement and traffic areas in usable condition. Low temperatures cause icing on pavement and frost on subgrade structures. Alternate freezing and thawing may damage surfaces and block drainage systems with ice. Spring thaws may result in surface and subgrade failure. Winter maintenance mainly consists of removing snow and ice, sanding icy surfaces, erecting and maintaining snow fences, and keeping drainage systems free from obstruction.

Preparation for Winter

11-71. 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.

11-72. Keep earth and gravel surfaces smooth and shaped to prevent moisture from entering the subgrade. Smoothing and shaping 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

11-73. 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 windborne snow is picked up in open spaces, loses velocity, and is deposited in sheltered places. Danger spots are roads at ground level or in cuts adjacent to large, open areas. Drifts form in the lee (downwind 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.

11-74. Place snow fences on the windward side of roads according to prevailing winds. (See figure 11-7.) The height of the fence determines the distance it is 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.

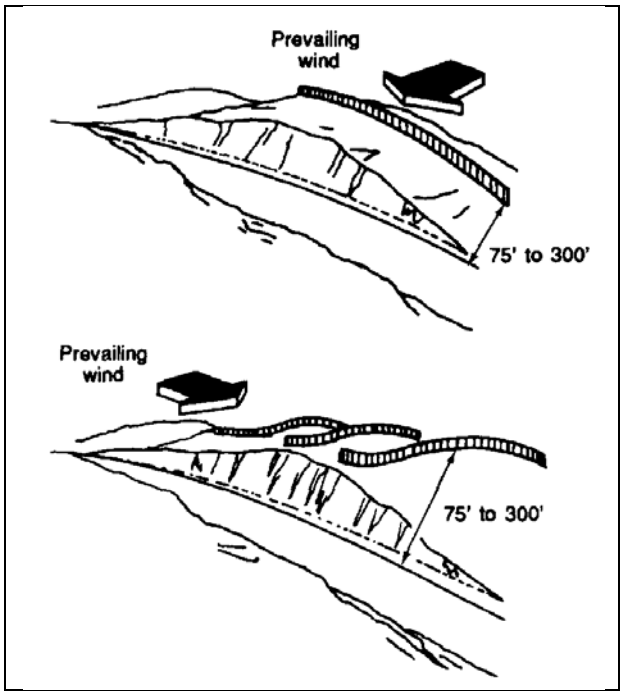


Figure 11-7. Location of snow fences

11-75. Fences should be as long as possible without any holes or openings. Openings provide for dispersion of snow on the backside of the fencing. The effect of a snow fence in controlling drifts caused by a road cut is shown in figure 11-8. Two or more parallel fences may be required, but one tall fence is generally better than two short fences. If fences are set too far away, they have little or no effect in reducing drifts. If they are placed too close to the road, drifts to the leeward side of the fence fall on the road.

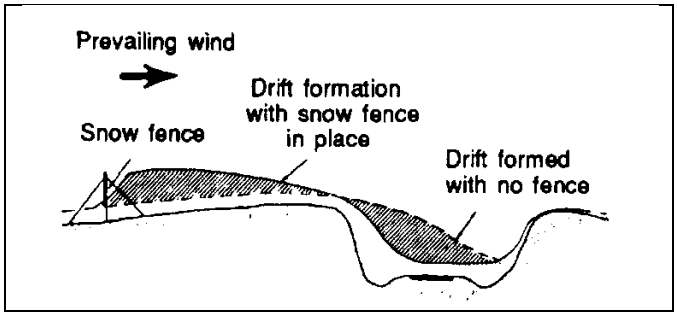


Figure 11-8. Snow fence control of road cut drifts

11-76. 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 mesh snow fencing is lighter and more efficient than wooden fencing.

11-77. 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 11-9, page 11-14, shows three types of snow fences.

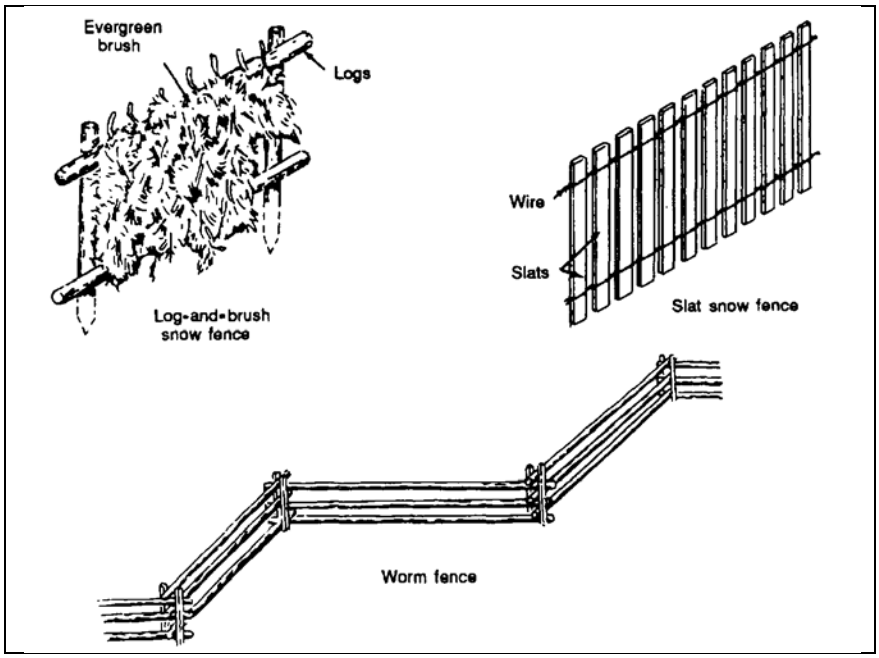


Figure 11-9. Types of snow fences

11-78. Erect a snow fence 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.

11-79. 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 fences may be required after midseason thaws or long periods of settling.

11-80. 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

11-81. 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 road nets that are to receive early attention, and operators should be ready to move promptly when a snowstorm arrives.

11-82. Since snow-removal equipment must frequently be operated on a 24-hour basis during periods of heavy storms, provide relief crews. 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 underbody blades; V-type plows; and rotary snowplows. These items are described in TM 5-624/NAVFAC DM MO-102/AFJMAN 32-1040. Graders, dozers, and loaders are also useful in snow removal. Improvised equipment, such as drags, may be used in expedient situations.

11-83. 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.

11-84. 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, removes snow from the road before it is compacted, and provides an open track for traffic.

11-85. Use 5- to 10-ton trucks equipped with straight or V-shaped blades for heavier snowfalls and to widen traffic lanes. This equipment operates at about 15 miles per hour. Use V-shaped plows to break through heavy drifts. Use 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.

11-86. Trucks used for plowing generally carry abrasives as 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

11-87. The formation of road ice that results 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.

11-88. 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.

11-89. 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.

Salts

11-90. 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.

Abrasives

11-91. Treat icy pavements with abrasives and salts to reduce slipping and skidding. Sand, (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.

11-92. 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 degrees Fahrenheit.

11-93. 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 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

11-94. Graders can sometimes remove ice that is not tightly bonded to the road surface. Using scarifiers or rotary tillers equipped with special teeth can reduce extremely icy conditions. Exercise extreme care so that the road surface is not damaged.

Correction of Spring Breakup Problems

11-95. 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 have a tendency to saturate permeable surfaces. Drainage obstructions may raise the water table and make subbases unstable.

11-96. 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.

11-97. 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.

11-98. 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. 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.

11-99. 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 rocks. It is better to remove soft, water-soaked material beforehand, although more time is required.

11-100. For better results, provide adequate drainage while performing repairs 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

11-101. The best maintenance is preventive maintenance. Prohibit traffic during the critical 2- or 3-day period on roads prone to frost heaves and boils. Methods to eliminate or minimize the damaging effects of frost action are discussed in TM 3-34.48-2.

FORDS AND BRIDGES

11-102. 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.

11-103. 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

11-104. Airfield and heliport maintenance is the responsibility of the primary user. At Air Force facilities, such maintenance is normally performed by an Air Force civil engineering squadron or similar unit. Army engineer units normally maintain Army airfields and heliports. When the repair and rehabilitation requirements of Air Force bases exceed the immediate, emergency-damage recovery capability of the air base, Army engineer units will be assigned to perform the work.

MAINTENANCE

11-105. Routine maintenance of hot-mixed asphalt and concrete airfield and heliport pavements can be performed during scheduled closures using the methods and materials presented in UFC 3-270-01 (hot-mixed asphalt) or UFC 3-270-03 and UFC 3-270-04 (concrete).

11-106. At expeditionary locations or when scheduled closures are limited, partial-depth spall repairs of Portland cement concrete pavements can be accomplished using rapid-set proprietary materials and the construction methods of UFC 3-270-03. Acceptable materials are listed in table 11-1.

Table 11-1. Proprietary materials for patching partial-depth spalls in concrete

Material	*Temperature Range (°F)*	*Working Time (minutes)*	*Extend with (Y/N)*	*Self-Leveling (Y/N)*	*NSN*
Pavement 15	40–140	7–9	N	Y	6850-01-500-6996
Pavement SLQ	0–95	3–5	Y	Y	6850-01-532-9958
Pavement 5.0	0–95	2–3	N	Y	None
DOT Rapid Set	32–100	10–15	Y	N	None
Dayton-Superior HD50	10–100	15	Y	N	None
Flex Patch	40–100	20	N	N	6850-01-549-0548
Legend:					
N	no				
NSN	national stock number				
Y	yes				
^{0}F	degrees Fahrenheit				

AIR BASE DAMAGE REPAIR

11-107. The immediate, emergency-damage recovery of air bases 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 paved surfaces, which is called rapid runway repair. This is accomplished through the employment of Air Force base civil engineering troop assets, prime base engineer emergency forces, and rapid engineering deployable heavy operational repair squadrons engineering units. The Army is responsible for semipermanent construction, the beyond-emergency repair of the air base and, emergency repairs that exceed Air Force capability upon request.

Army Responsibilities for Air Base Damage Repair

11-108. 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 with the emergency repair of war-damaged air bases where requirements exceed the Air Force organic repair capability.
- Repairing and restoring damaged air bases with beyond-emergency repairs.
- Developing engineer designs, plans, and materials to meet the needs of the Air Force as agreed upon. Where practicable, designs are based on the Army Facilities Component System.
- Supplying construction materials and equipment, except for that provided by the Air Force.
- Assisting within their capabilities with the removal of unexploded ordnance declared safe by explosive ordnance disposal personnel and limited damage assessment operations upon request.
- Managing and supervising the repair and restoration of war damage performed by Army personnel. The Air Force base commander sets the priorities for air base repair.

Air Force Responsibilities for Air Base Damage Repair

11-109. The Air Force provides military troop engineering support from its resources. The Air Force ensures that units are equipped, manned, and trained adequately to support its needs. This support includes—

- Providing emergency repair of war damage to air bases.
- Organizing host nation support (overseas).
- Managing force beddown of units and weapon systems, excluding Army base development responsibilities.
- Providing operation and maintenance of facilities and installations.
- Providing crash rescue 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 supply except classes II, V, VII, and IX.
- Conducting damage assessment and removal of unexploded ordnance.
- Providing chemical, biological, radiological, and nuclear collective shelters. 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.)

11-110. Air base support agreements may be established in some theaters between the Air Force and the host nation where air base damage repair support capability exists. These host nation support agreements may include equipment, materials, and manpower assets.

11-111. 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 UFC 3-270-07. For a further, detailed discussion of general air base damage repair, refer to UFC 3-270-07.

TURF SURFACES

11-112. Plant grass to provide a turf surface on shoulders and graded areas. Turf helps with camouflage, reduces dust, and minimizes erosion. Turf surfaces are limited to areas where the climate and soil are favorable. (See table 11-2 for the different characteristics of grass.)

MUD CONTROL

11-113. 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.

Table 11-2. Characteristics of grass

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								
Kentucky bluegrass (Poapratensis)	Good	Loam, clayey loam	Good	Fair	Slow	Seed, sod	Fall or spring	Number 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
Smooth brome (Bromus inermis)	Good	Various	Good	NA	Medium, fast	Seed	Spring	Mix with native grass
WARM, HUMID REGION								
Bermuda grass (Cynodon dactylon)	Excellent	Sandy to clayey loam	Excellent	NA	Fast	Sprigs, sod, seed	Spring or summer	Seed along or with carpet grasses
Common carpet grass (Axonopus affinis)	Good	Moist clays, clayey loam	Fair	NA	Medium, fast	Seed, sod	Spring or summer	Seed along or with carpet grasses
St. Augustine grass (Stenotaphrum secundatum)	Fair	Moist, various	Fair	NA	Fast	Sprigs	Spring or summer	NA
DRY REGION								
Buffalo grass (Buchloe dactyloides)	Excellent	Clayey loam to loam	Excellent	NA	Medium, fast	Block, sod	Spring	Seed blue grama between buffalo-grass-sod blocks
ROUGH TURF AND BUNCH GRASSES								
COOL, HUMID REGION								
Common ryegrass (*Lolium multi-*florum and L. perenne)	Good	Various	Good	Good	Fast	Seed	Fall or spring	Number 9
Orchard grass (*Dactylis glomerata*)	Fair	Loam to clay	Good	Excellent	Fast	Seed	Fall or spring	Number 8
WARM, HUMID REGION								
Hairy crab-grass (*Digitaria sanguinalis*)	Excellent	Loam to clay	Excellent	NA	Fast	Hayseeding	Summer	NA
Bluestem (broomsedge) (*Andropogan*)	Fair	Loam to clay	Excellent	Excellent	Medium	Hayseeding	Fall	NA
Korean lespedeza (*Lespedeza stipulacea*)	Excellent	Loam to clay	Excellent	NA	Medium	Seed	Spring	NA

Table 11-2. Characteristics of grass (continued)

DRY REGION								
Little bluestem (*Andropogon scoporius*)	NA	Moist, sandy to loamy	NA	NA	NA	Seed, hayseeding	Spring	Other grasses suited to soil
Blue grama (*Bouteloug gracilis*)	NA	Loam to clay	NA	NA	NA	Seed, hayseeding	Spring	Seed between blocks of buffalo grass
Crested wheatgrass (*Agropyron cristatum*)	NA	Clayey loam to loam	NA	NA	Slow	Seed	Spring	Other native grasses
Note. Consult for further information, and consult with local agricultural agents or major command agronomist for information on grasses best suited for a particular area or purpose.								
Legend:								
NA	not applicable							
TM	technical manual							

11-114. Mud on airfield and heliport surfaces is deposited by traffic from adjacent muddy areas or caused by subgrade failure when excess moisture and mud is pumped to the surface by traffic. Enforce mud discipline by limiting taxiway access to service vehicles. Remove mud from vehicle wheels and undercarriages before they enter the taxiway. The most satisfactory solution is to provide surfaced service roads to hardstands.

Repairs

11-115. 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 traffic until the surface dries. In extreme conditions, resurfacing may be necessary.

Mud Removal

11-116. 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.

11-117. When a grader is used on a landing-mat surface, prevent the blade from tearing the surfacing. One method is to bolt a 4- by 12-inch hardwood moldboard over the cutting edge, 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 center of the runway and proceed progressively to the edge, overlapping several feet on each pass.

SNOW REMOVAL AND ICE CONTROL

11-118. Snow removal methods, the order of operations, and the assignment of equipment are established before the winter season. Factors to consider when planning the snow-handling program include climatic conditions, average snowfall, aircraft to be accommodated, equipment available, and camouflage requirements. Aircraft may be equipped with 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 camouflage, it is undesirable to remove all snow from a runway when the surrounding terrain is blanketed with snow.

11-119. 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

11-120. In regions of heavy snowfalls with prolonged cold weather relatively free from sudden thaws, snow may be handled by packing. The runway, shoulders, and the adjacent area are packed. Rolling begins when 3 inches of snow has fallen and continues during the snowfall. Rollers drawn behind a tractor with snow treads pack the snow.

11-121. 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 the drag and the rollers with the drag ahead of the rollers.

11-122. Rollers are made to a desirable diameter and length with a shell of 10-gauge 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 the ends. One plate has a hole to permit filling the roller with sand to increase the weight.

Clearing and Removing

11-123. Snow clearing and removal are required where climatic conditions prevent packing or snowfalls exceed what 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.

11-124. Equip trucks with tire chains and carry ballast for traction while plowing. Keep blades about 1/2 inch above the runway surface, especially if the surface is a landing mat. Runners mounted at each end of the blade accomplish this clearance.

11-125. The assignment of plows varies with the condition at each airfield and the type of equipment available. Arranging the plows into units simplifies coordination of snow removal with the control tower. Ordinarily, one-way, truck-mounted snowplows operate in formation to expedite snow removal. Remove snow near landing lights and other obstructions with a blower, if available, or by hand.

11-126. Do not permit the snow plow moldboard to touch the airfield pavement surface. Use castor wheels or shoes to maintain a minimum 1/2-inch clearance. Avoid straddling the centerline with the plow blade, since the change in transverse grade across the centerline will reduce or eliminate the clearance between the moldboard and the pavement surface, resulting in spalling of the pavement surface.

11-127. The rapid removal of snow requires a rotary blower, snow loader, or other special equipment. Trucks used for hauling snow are equipped with high sideboards. Tractor-drawn sleighs built of lumber may be used as an expedient hauling device or to supplement snow-handling trucks.

Abating Ice Conditions

11-128. Sprinkle ice coatings on runways, taxiways, and hardstands with urea, coarse sand, or cinders. Spread by hand or with mechanical spreaders. If practical, heat abrasives before spreading. Remove accumulated abrasives in the spring by brooming. Ice conditions on airfields used by jet aircraft are 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

11-129. Coordinate maintenance and repair work during flying operations, and plan the work for minimum interference with air and ground traffic. Maintenance may need to be performed at night or during inclement weather so that flying operations are not interrupted.

11-130. 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

11-131. 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. A combat-heavy engineer battalion usually accomplishes the work. Their mission is to convert existing facilities, which are usually damaged, to the standard decided on 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.

11-132. An appraisal of the damage 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 includes war damage by our forces in the battle for the airfield and deliberate damage by the enemy before yielding the field to our forces. Complete destruction of an airfield is a major undertaking; therefore, the enemy will likely resort to one 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.

11-133. When conducting such reconnaissance, assume these damages were inflicted. Use the following criteria to prioritize rehabilitation operations for a captured airfield:

- **Establish minimum facilities and utilities.** Establish a minimum operating strip for immediate operation of friendly aircraft, to include removing unexploded ordnance, delayed-action bombs, mines, and booby traps; clearing debris; and repairing craters on the runway and taxiway surfaces. Promptly repair the drainage system. Concentrate runway work on the minimum operating strip, then on an access route, and finally on other traffic areas. Give early attention to suitable sanitary and water facilities. See TC 5-340 for additional information.
- **Improve 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.
- **Improve other operational facilities.** Repair the control tower, operational buildings, crew shelters, communication centers, and other maintenance facilities.
- **Final phase of rehabilitation.** Camouflage the installation; restore utilities (make use of utility maps and available citizen labor familiar with installation utilities); repair or establish bathing, dining, and recreational facilities; clean up the grounds, to include a complete removal of debris; and seed and sod the installation, as needed.

Chapter 12

Road Design and Airfield Layout

Road selection and design depend on the nature of the subgrade; 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 meet the requirements for a 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.

GEOMETRIC DESIGN

12-1. For safe and speedy traffic movement, the geometric design requirement for 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 support 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. (See figure 12-1 and figure 12-2, page 12-2.) UFC 3-250-01FA provides details on the design of bituminous and concrete-surfaced roads, UFC 3-250-09FA provides details on the design of aggregate surfaced roads, and UFC 3-250-09FA provides details on the geometric design for roads. UFC manuals can be found in the Whole Building and Design Guide under Department of Defense criteria.

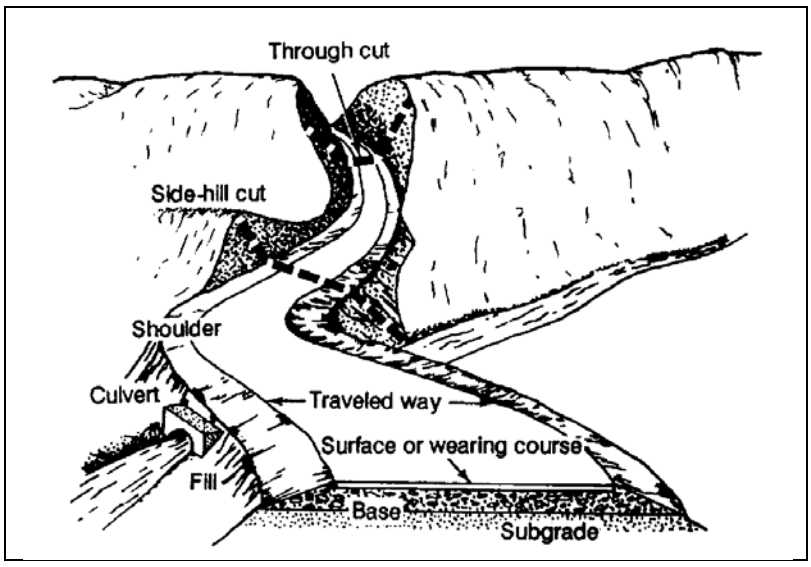
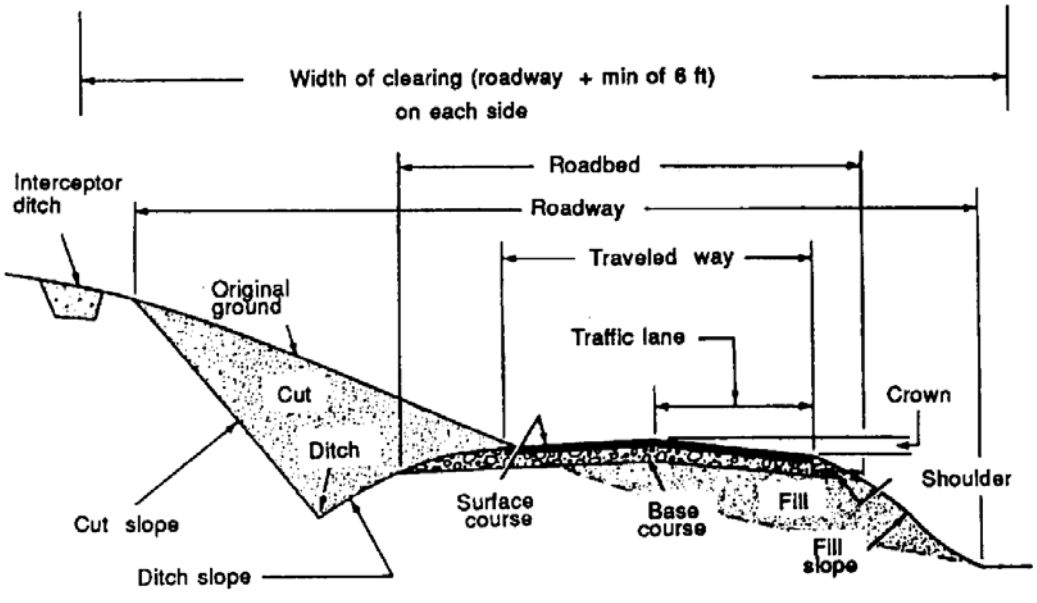


Figure 12-1. Road nomenclature



Legend:

ft	feet
min	minimum

Figure 12-2. Road cross section and nomenclature

12-2. 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. EM 1110-1-1005 has information on topographic surveys. Perform the following steps to complete the design process:

- **Step 1.** Draw the proposed centerline on the topographic survey.
- **Step 2.** Plot the centerline on plan and profile paper or with computer-aided design software.
- **Step 3.** Calculate grades, the degree of curvature of horizontal curves, and curve lengths of vertical curves.
- **Step 4.** Compare these values with the military road specifications stated in table 12-1.
- **Step 5.** Adjust the centerline, if possible, to reduce any calculated grades and limit horizontal and vertical curves that exceed the specifications.
- **Step 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.
- **Step 7.** Design horizontal and vertical curves for tangent intersections.
- **Step 8.** Plot newly designed curves on the plan and profile.
- **Step 9.** Develop a mass diagram for the project. Balance the cuts and fills and optimize ruling grade and earthwork volumes.
- **Step 10.** Design superelevations (curve banking) and widening for horizontal curves.
- **Step 11.** Draw typical cross sections.
- **Step 12.** Design the required drainage structures and bridges.

Table 12-1. 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)*	*Remarks*
Design Controls					
Traffic composition					The DHV shown for roads is in total vehicles per hour for lanes in both directions. The DHV is approximately 15% of the ADT.
ADT (45% trucks)	3,400-6,700	935-3,400	200-935	Under 200	None.
DHV	510-1,000	140-510	30-140	Under 30	None.
Sight distance restriction, %	40-0	80-0	80-40	100	The values shown for this term indicate the combined effects of horizontal (curves) and vertical (grade) alignment on capacity. A value of 0% indicates an absolute straight, flat alignment with no restriction on sight distance. A value of 100% indicates a road with numerous sharp curves and grade changes where sight distance is less than 1,500 ft.
Design speed (V), mph	60	60	40	30	None.
Average running speed, mph	45	45	35	25	None.
Cross-Section Elements					
Pavements					If the anticipated traffic includes a significant number of vehicles having widths in excess of 8.5 ft, traffic lanes should be widened in the amount that the vehicle width exceeds 8.5 ft.
Minimum width of traffic lane with barrier curb, ft	12	12	10	10	None.
Minimum width of traffic lane without barrier curb, ft	12	12	10	10	None.
Minimum distance between curb faces, ft	53	29	25	15	None.
Lateral clearance from edge of traffic lane to obstructions, ft	6	6	6	4	None.
Normal cross slope (crown slope), rate	0.0104-0.0108	0.0104-0.0208	0.0208-0.0417	0.0208-0.0417	None.
Shoulders					There should be a color or texture contrast between traffic lane and shoulder surfaces.
Minimum width without barrier curbs, ft	10	10	6	4	None.
Normal cross slope, rate	0.0417-0.0625	0.0417-0.0625	0.0417-0.0625	0.0417-0.0625	None.
Type (permanent road)	Dustless	Stable	Compacted soil	Compacted soil	None.
Bridge clearance (permanent)					
Curb offset for barrier curb, ft	2.5	2.5	2.0	2.0	None.

Table 12-1. Geometric design data for military roads (continued)

Design Controls and Elements	*Class A (4-lane)*	*Class B (2-lane)*	*Class C (2-lane)*	*Class D (1-lane)*	*Remarks*
Alignment Elements					
Sight distance					
Minimum stop sight distance, ft	475	475	275	200	None.
Minimum pass sight distance, ft	NA	2,100	1,500	NA	None.
Horizontal alignment					
Maximum horizontal curvature	5.5°	5.5°	14.5°	26.7°	Values shown are calculated based on a maximum rate of 0.100 superelevations.
Pavement widening, ft	None	None	2-4	2-5.5	Pavement widening for a Class C or Class D road varies 2 to 5.5 ft as the curvature varies from 2 to 26.7°. Values obtained may be rounded off to the nearest 0.5 ft.
Vertical alignment					
Grade	NA	NA	NA	NA	None.
Maximum grade, %	6	6	10	15	None.
Critical length, ft	700	700	450	250	Critical length indicates the maximum length of a designated upgrade a loaded truck can operate without an unreasonable reduction in speed. Critical lengths may be increased at an approximate rate of 50 ft per % decrease in grade from the values shown.
Minimum grade, %	0.3	0.3	0.3	0.3	None.
Vertical curves	NA	NA	NA	NA	The minimum lengths of vertical curves are determined by multiplying k by the algebraic differences in grades (in percent).
Overt (crest) vertical curve k, ft	160	160	55	35	None.
Invert (sag) vertical curve k, ft	105	105	55	28	None.
Absolute minimum length, ft	180	180	120	80	None.

Notes.
1. Bridge clearance (permanent) width of the traveled way should equal the width of the lanes plus 5 ft (2.5 ft on each side) and 14.75 ft vertical clearance.
2. 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.
3. Turnouts should be provided at 1/4-mile intervals on Class D roads.
4. Curbs will generally not be provided in open areas.

Legend:

ADT	average daily traffic
DHV	design hourly
ft	feet
k	rate of vertical curvature
mph	miles per hour
NA	not applicable

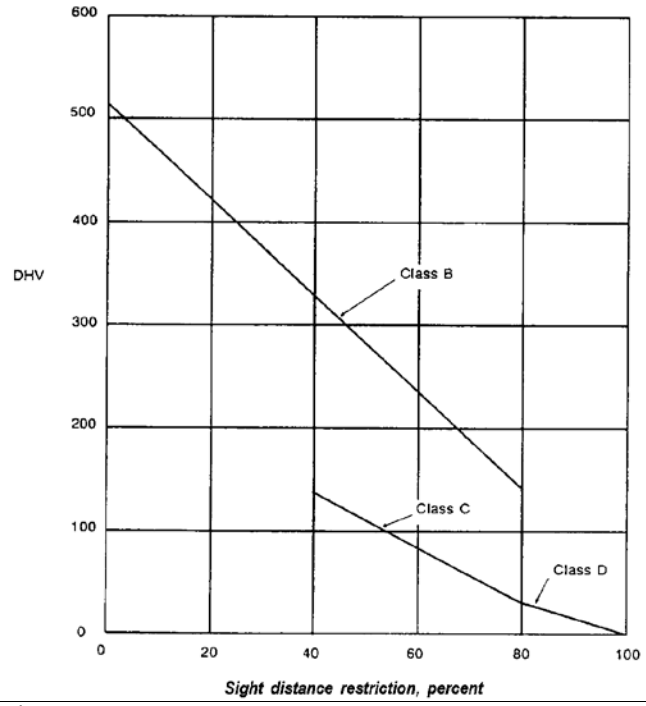
SELECTION OF ROAD TYPE

12-3. Structural characteristics should accommodate traffic volumes throughout the road design life. Table 12-1, page 12-3, shows four possible road classes. They are based on expected traffic volumes and show the values for the design control elements for each road class. The capacities are shown as a range of values. Only road Classes B, C, and D apply to JOA 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

12-4. The values for each geometric feature must be attained to ensure that the desired road will have a capacity equal to or greater than the average daily traffic or design hourly volume shown in table 12-1, page 12-3. The first step in the design of a road is to estimate the daily or hourly number of vehicles in a military organization. If this cannot be done, a conservative estimate is to multiply the number of vehicles organic to the units that will use the road by a factor of two. This assumes that each vehicle will use the road twice a day (one round-trip).

12-5. If the anticipated design hourly volume or average daily traffic is known and the sight distance restriction can be estimated from preliminary plans, the necessary road type can be determined. (See figure 12-3 for the relationship between design hourly volume and sight distance restriction.) If average daily traffic or design hourly volume and the road type desired are known, sight distance restriction requirements can also be determined.



Note. Not to scale.

Legend:

DHV	design hourly volume

Figure 12-3. Interpolation of design hourly volume for selection of road class

12-6. A range of possible design hourly volume values is given for each road classification in table 12-1, page 12-3. The actual design hourly volume for a road is a function of the sight distance restriction factor, which also has an allowable range for each type of road. The design hourly volume varies directly with a change in the sight distance restriction factor. Figure 12-3 shows this straight-line relationship. After the sight

distance restriction factor is determined from the design plans of an assumed road class, the actual design hourly volume is determined to ensure that the capacity is adequate.

Example

A road needs to be designed for a military organization that has approximately 250 vehicles.

$$Average\ daily\ traffic = 250 \times 2 = 500$$
$$Design\ hourly\ volume = 0.15 \times 500 = 75$$

The 0.15 factor clusters traffic into rush hours. Otherwise—

$$Hourly\ volume = 500\ vehicles\ per\ day/24\ hours\ per\ day = 21\ vehicles\ per\ hour$$

The calculated design hourly volume of 75 could be met by a Class C road. Therefore, assuming that a Class C road is used, plan and profile designs could be drawn and a sight restriction factor can be determined from the design.

12-7. From figure 12-3, page 12-5, a sight distance restriction factor of 62 percent is determined (based on a Class C road and a design hourly volume of 75). The maximum sight distance restriction factor for a Class C road is 80 percent. This provides a design hourly volume 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 design hourly volume being greater than or equal to 75. Therefore, the Class C road assumption is adequate. If a design hourly volume of 75 could not be handled by the Class C road, it would be necessary to construct a Class B road.

ESTIMATING CAPACITY

12-8. The information in figure 12-3 and table 12-1, page 12-3, is adequate for the geometric design of military roads. The information can be used to evaluate the capacity of existing roads by obtaining pertinent characteristics and comparing them to the values in table 12-1. If the data does not conform to a given road type, estimate the road type and the average daily traffic or design hourly volume that the data best conforms to. The volume and capacity values in table 12-1 are for roads that 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

12-9. Before building a road, the engineer must determine the best vertical and horizontal alignment of the facility. Design 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 alignments are interrelated and must be considered concurrently. However, the principles on each are best studied separately. Horizontal and vertical curves are discussed in TM 3-34.55.

HORIZONTAL ALIGNMENT AND HORIZONTAL CURVES

12-10. 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, to improve route capacity make each tangent as long as possible, limit the number of curves, and provide long, straight stretches.

12-11. 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, and decrease the tangent length. However, this reduction in tangent length is minor compared to the benefits gained by reducing the total number of curves.

12-12. Tangents should intersect other roads and railroads at right angles. Military roads supplement existing road nets and have intersections at one or both ends of the military road. Operating efficiency usually is improved when these intersections approach right angles.

12-13. The most common horizontal curves (see figure 12-4) are—

- **Simple 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 JOA 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.
- **Reverse curve.** A *reverse curve* uses two simple curves that are 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 not be equal in length.
- **Compound curve.** A *compound curve* has two simple curves that are 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.
- **Spiral curve.** 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 (Class A and Class B). Detailed steps for the design and layout of spiral transition curves are in TM 3-34.55. Low design speeds of Class C and Class D roads do not require spiral transition sections.

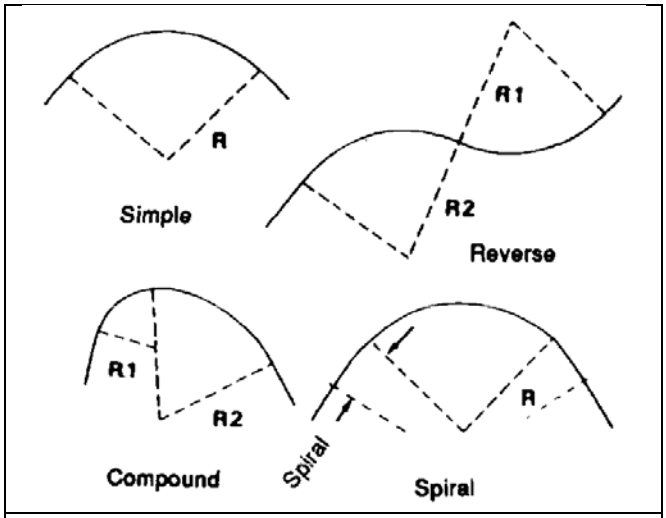


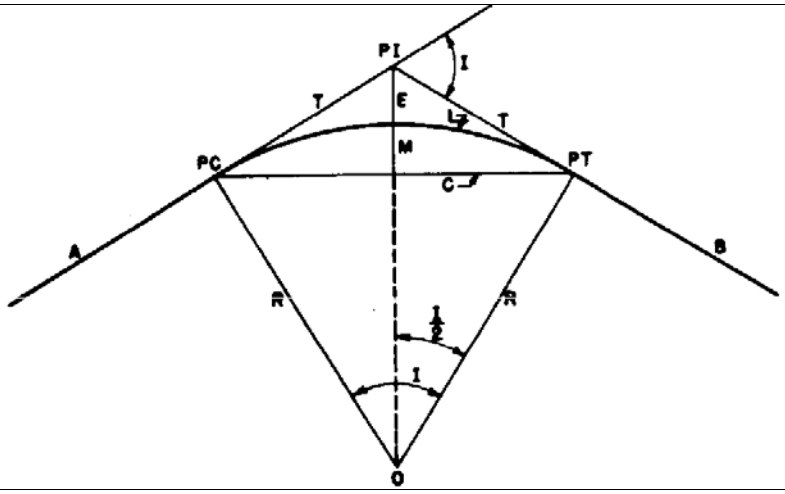
Figure 12-4. Types of horizontal curves

ELEMENTS OF A HORIZONTAL CURVE

12-14. The elements of a simple, horizontal curve (see figure 12-5, page 12-8) are the—

- Point where the curve begins (point of curvature) or leaves tangent A—the tangent nearest the origin of stationing (Station 0 + 00) or start of the project.
- Point where the curve ends (point of tangency) or joins tangent B.
- Intersecting point of two tangents that must be connected by a horizontal curve.
- Tangent distance is the distance from the point of intersection to the point of curvature or from the point of intersection to the point of tangency.
- Radius of curvature is the radius of the circle that forms the curve from point of curvature to point of tangency.

- Length of the curve is the distance from point of curvature to point of tangency along the curve, measured as an arc or a series of 100-foot arcs. Railroad engineers measure the length of the curve as a series of 100-foot chords.
- Angle of intersection is the exterior angle at the point of intersection formed by tangents A and B. The central angle, between the radius points at O, is equal to the exterior angle.
- External distance is the distance from the point of intersection to the midpoint of the curve.
- Long chord is the straight-line distance from the point of curvature to the point of tangency.
- Middle ordinate is the distance from the midpoint of the curve to the midpoint of the long chord.



Legend:	
C	long chord
E	external distance
I	angle of intersection
L	length of the curve
M	middle ordinate
PC	point of curvature
PI	point of intersection
PT	point of tangency
R	radius
T	tangent distance

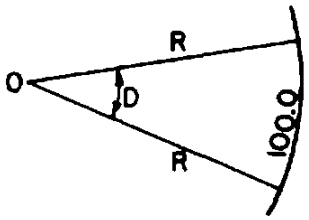
Figure 12-5. Elements of a simple, horizontal curve

DEGREE OF CURVATURE

12-15. 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. The degree of curvature is established as a whole or half degree. The degree of curvature may be stated in terms of the arc or the chord.

Arc Definition

12-16. The degree of curvature is the angle that subtends a 100-foot arc along the curve. (See figure 12-6.) This definition is used by state highway departments and USACE in road design.

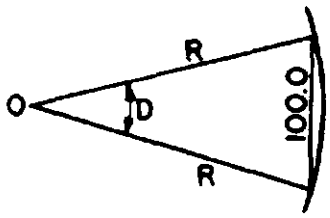


Legend:	
D	degree of curvature
R	radius

Figure 12-6. Arc definition for degree of curvature

Chord Definition

12-17. The degree of curvature is the angle that subtends a 100-foot chord on the curve. (See figure 12-7.) This definition results in a slightly larger angle than the arc method, and it is used by the railroad industry and USACE in railroad design.



Legend:	
D	degree of curvature
R	radius

Figure 12-7. Chord definition for degree of curvature

12-18. The difference between the arc and chord definitions is very slight and nearly insignificant (frequently below 1 percent) for JOA construction. However, because the arc definition is the most widely used procedure in road design, its definition will be used throughout the rest of the chapter.

EQUATIONS FOR THE SIMPLE, HORIZONTAL-CURVE DESIGN

12-19. The two methods commonly used to solve horizontal curve problems are the 1° curve method and the trigonometric method. These methods may be used with the same degree of accuracy. The 1° curve method requires the functions of a 1° curve table (refer to TM 3-34.48-2).

12-20. Curves of different degrees of curvature can be readily designed because of the proportionality between curves and the l° curve. For example, a curve of D = 15° has one-fifteenth the length of the curve, external distance, tangent distance, and middle ordinate values as for a 1° curve (D = 1°). The only information needed to obtain these values for a 1° curve is the angle of intersection, and the angle of intersection is always known at the onset of the design process. The trigonometric method requires a calculator with trigonometric functions or trigonometric tables.

Radius of Curvature

12-21. As previously described in the arc definition, the degree of curvature 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 the radius is developed in terms of degree of curvature. See the following equation:

$$\frac{D_{arc}}{100^1} = \frac{360°}{2\pi\, R}$$

Solving for R—

$$R = \frac{(100)(360)}{2\pi\, D} = \frac{5{,}729.58}{D}$$

where—

D = *degree of curvature*
R = *radius*

Tangent Distance

12-22. In the right triangle shown in figure 12-5, page 12-8, the vertices are at the point of curvature (or the point of tangency), point of intersection, and D. The tangent distance is found using the 1° curve method, as follows:

$$T = \frac{T_{1°}}{D}\ (arc\ definition)$$

where—

D = *degree of curvature*
T = *tangent distance*

12-23. Refer to table B-1, page B-9, for additional information on T1° for a given angle of intersection. Use the table to determine the chord correction.

External Distance

12-24. Using the 1° curve method (see figure 12-8), the external distance is found as follows:

$$E = \frac{E_{1°}}{D}(arc\ definition)$$

where—

D = *degree of curvature*
E = *external distance*

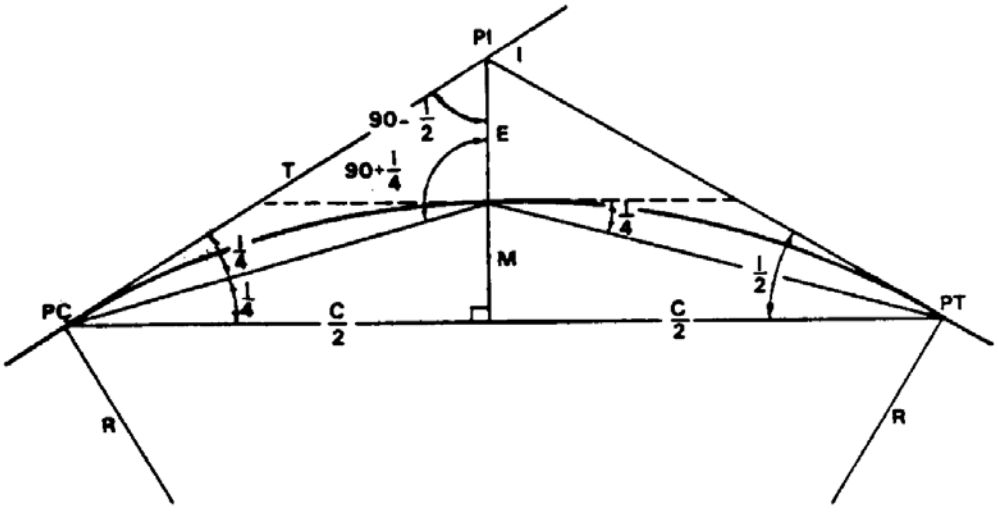
Middle Ordinate

12-25. Using the 1° curve method (see figure 12-8), the middle ordinate is found as follows:

$$M = \frac{M_{1°}}{D}(arc\ definition)$$

where—

D = *degree of curvature*
M = *middle ordinate*



Legend:	
C	long chord
E	external distance
I	angle of intersection
M	middle ordinate
PC	point of curvature
PI	point of intersection
PT	point of tangency
R	radius
T	tangent distance

Figure 12-8. Derivation of external distance

Length of Curve

12-26. Measure the length of the curve in 100-foot arcs. Because the degree of curvature subtends a 100-foot arc, the total number of such arcs in a horizontal curve must be the number of times that the degree of curvature can be included in the central angle of interception. See the following equation:

$$L = \frac{I}{D} \times 100$$

where—

D = *degree of curvature*
I = *angle of intersection*
L = *length of the curve*

12-27. The central angle subtended by the entire horizontal curve has sides that are radii to the point of curvature and point of tangency. Both of these radii are perpendicular to the tangents that form the angle of intersection. The quadrilateral formed by four points of intersection (180° – I), point of curvature (900), angle of intersection, and point of tangency (90°) must total 360°—

$$(180 - I) + 90 + I + 90 = 360$$

where—

I = *angle of intersection*

Hence, the central angle is equal to the angle of intersection.

DESIGNING HORIZONTAL CURVES

12-28. The engineer designing horizontal curves must know two facts about the curve from the preliminary survey: the location and station of the point of intersection and the angle between intersecting tangent lines. The curves can be designed after this information is obtained.

12-29. 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.

12-30. 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 12-1, page 12-3, specifies the maximum degree of curvature for each class of road as stated in the row titled *Maximum Horizontal Curvature*.

12-31. After the degree of curvature is selected, determine the stations of the point of curvature and the point of tangency. Next, design the curve except for the calculations needed to locate stationing points of the curve between the point of curvature and point of tangency. The following example shows the design of horizontal curves using the 1° curve method.

Example

Find the degree of curvature by using one of the following three equations. (See TM 3-34.48-2.)

If the curve is unrestricted—

$$D = \frac{5{,}729.58}{R}$$

If the curve is restricted by the tangent distance—

$$D = \frac{T_{1^\circ}}{T_{restricted}}$$

If the curve is restricted by the external distance—

$$D = \frac{E_{1^\circ}}{E_{restricted}}$$

Round up the degree of curvature to the next half degree when possible, and determine the length of the tangent.

$$T = \frac{T_{1^\circ}}{D}$$

Find the stationing of the point of curvature.

$$PC = PI - T$$

Example (continued)

Calculate the length of the curve.

$$L = \left(\frac{I}{D}\right) 100$$

Find the stationing of the point of tangency.

$$PT = PC + L$$

where—

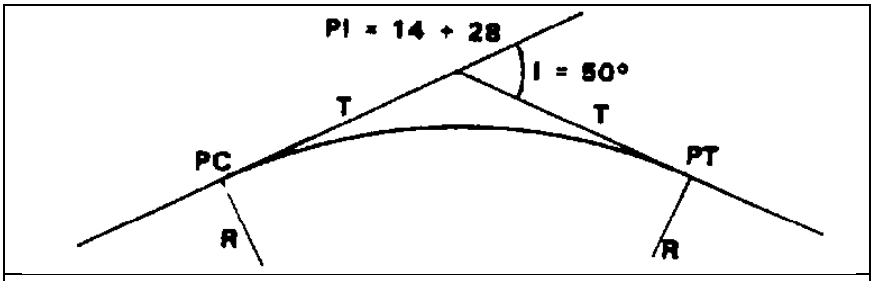
D	=	*degree of curvature*
$E_{1°}$	=	*external distance for 1° curve*
$E_{restricted}$	=	*restricted external distance for a horizontal curve*
I	=	*angle of intersection*
L	=	*length of curve*
PC	=	*point of curvature*
PI	=	*point of intersection*
PT	=	*point of tangency*
R	=	*radius of the curve*
T	=	*tangent distance*
$T_{1°}$	=	*tangent distance for a 1° curve*
$T_{restricted}$	=	*restricted tangent distance for a horizontal curve*

HORIZONTAL-CURVE DESIGN EXAMPLES

12-32. This section describes the horizontal-curve design procedures for the following common situations:

- Terrain restriction does not limit the tangent distance or the external distance.
- Terrain restriction limits the tangent distance.
- Terrain restriction limits the external distance.

12-33. Figure 12-9 illustrates the degree of curvature without terrain restriction.



Legend:

I	angle of intersection
PC	point of curvature
PI	point of intersection
PT	point of tangency
R	radius
T	tangent distance

Figure 12-9. Horizontal curve without sharpness restriction

Example

Find the station and location of point of curvature and point of tangency for a Class C road. Given—

D = *6°*
I = *50°*
PI = *14 + 28*

A degree of curvature 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 the following equations:

$$R = \frac{5{,}729.58}{D} = \frac{5{,}729.58}{6} = 954.93'$$

$$T = \frac{T_{1^\circ}}{D} = \frac{2{,}671.58}{6} = 445.30'(arc\ definition)$$

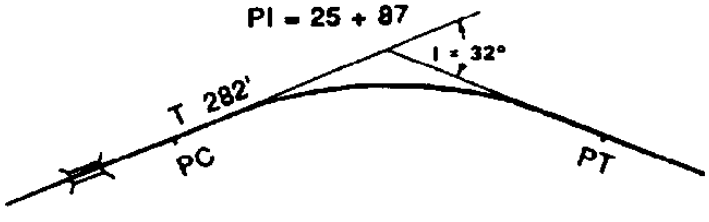
$$L = \frac{I}{D} \times 100 = \frac{50}{6} = 833.33'$$

$$PC = PI = T = (14 + 28) - (445.29') = (9 + 82.71)$$

$$PT = PC + L = (9 + 82.71) + (833.33') = (18 + 16.04)$$

where—
D = *degree of curvature*
I = *angle of intersection*
L = *length of curve*
PC = *point of curvature*
PI = *point of intersection*
PT = *point of tangency*
R = *radius of the curve*
T = *tangent distance*
T_{1° = *tangent distance for a 1° curve (see* TM 3-34.48-2/MCRP 3-17.7A-2/NTRP 4-04.2.10-2/AFMAN 32-8013-2*)*

12-34. The station of the point of tangency is determined by adding the curve length to the station of the point of curvature, not by adding the tangent distance to the station of the point of intersection. However, the actual point of tangency is found by measuring the tangent distance (at the angle of intersection) from the point of intersection. The station is the distance from the point of origin at Station 0 + 00, as measured along the centerline. (Figure 12-10 illustrates the terrain restriction of the tangent distance.)



Legend:	
I	angle of intersection
PC	point of curvature
PI	point of intersection
PT	point of tangency
T	tangent distance

Figure 12-10. Horizontal curve with restriction on tangent

Example

Find the station and location of PC and PT. Given—

I = *32°*
PI = *25 + 87*
$T_{(restricted)}$ = *282° (due to bridge)*

If T must not exceed 282 feet and D will give this value, it is necessary to find which D gives 282 feet for $T_{(restricted)}$ and then round it off, as shown below:

$$D = \frac{T_{1^\circ}}{T_{(restricted)}}$$

$$D = \frac{1{,}643}{282} = 5.83^\circ = 5^\circ 50'$$

where—
D = *degree of curvature*
I = *angle of intersection*
PC = *point of curvature*
PI = *point of intersection*
PT = *point of tangency*
T = *tangent distance*
T_{1° = *tangent distance for a 1° curve*
$T_{(restricted)}$ = *restricted tangent distance for a horizontal curve*

12-35. If the tangent distance was specified as exactly 282 feet (as opposed to a maximum or restricted value), the degree of curvature of 5°50’ must be used. Rounding the degree of curvature up to the next half degree (from 5°50’ to 6°) will slightly sharpen the curve and reduce the tangent distance slightly below the 282-feet maximum.

Example

Find the station and location of PC and PT. Given—

D = 6°

$$R = \frac{5{,}729.58}{D} = \frac{5{,}729.58}{6} = 954.93'$$

$$T = \frac{T_{1^\circ}}{D}\ (arc\ definition)$$

$$T = \frac{1{,}642.9300}{6} = 273.82'$$

$$L = \frac{I}{D} \times 100 = \frac{32}{6} \times 100 = 533.33'$$

$$PC = PI - T = (25 + 87) - (273.82') = (23 + 13.18)$$

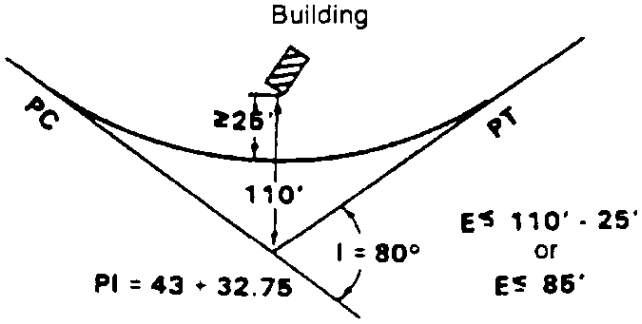
$$PT = PC + L = (23 + 13.18) + (533.33') = (28 + 46.51)$$

where—
D = degree of curvature
I = angle of intersection
L = length of curve
PC = point of curvature
PI = point of intersection
PT = point of tangency

Example (continued)

R	=	*radius*
T	=	*tangent distance*
T_{1°	=	*tangent distance for a 1° curve*

12-36. 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. Therefore, if the maximum value of the tangent distance is used to determine a trial value of the degree of curvature, 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. (Figure 12-11 illustrates the terrain restriction on the external distance.)



Legend:	
E	external distance
I	angle of intersection
PC	point of curvature
PI	point of intersection
PT	point of tangency

Figure 12-11. Horizontal curve with restriction on the external distance

Example

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. Given—

E	<	85 feet
I	=	80°

Station PI at 43 + 32.75

$$D = \frac{E_{1^\circ}}{E_{(restricted)}}$$

$$D = \frac{1{,}749.9}{85} = 20.59^\circ = 20^\circ 35'$$

Because of the limiting value for E and the value used to get the trial value of D are maximums, it is necessary to round to the next higher half degree, thereby decreasing T and E. Given—

D = *21°*

Example (continued)

$$R = \frac{5{,}729.58}{21} = 272.83'$$
$$T = \frac{T_{1^\circ}}{D} = \frac{4{,}807.6867}{21} = 228.94'$$
$$E = \frac{E_{1^\circ}}{D} = \frac{1{,}749.8548}{21} = 83.33 \leq 85'$$
$$L = \frac{I}{D} \times 100 = \frac{80}{21} \times 100 = 380.95'$$
$$PC = PI - T = (43 + 32.75) - (228.94') = (41 + 03.81)$$
$$PT = PC + L = (41 + 03.81) + (380.95') = (44 + 84.76)$$

where—

D	=	*degree of curvature*
E	=	*external distance*
E_{1°	=	*external distance for 1° curve*
$E_{(restricted)}$	=	*restricted external distance for a horizontal curve*
I	=	*angle of intersection*
L	=	*length of curve*
PC	=	*point of curvature*
PI	=	*point of intersection*
PT	=	*point of tangency*
R	=	*radius*
T	=	*tangent distance*
T_{1°	=	*tangent distance for a 1° curve*

Station Adjustments Due to Curve Installation

12-37. Horizontal curves are occasionally 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 the station adjustments required by the shortening of the overall centerline length by a distance of—

$$2T - L$$

where—

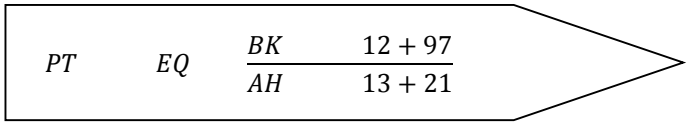
L	=	*length of curve*
T	=	*tangent distance*

12-38. However, when horizontal curves are designed in an office with the data supplied by the preliminary survey, the adjustments must be calculated.

12-39. When the preliminary tangent alignment of a route is first determined and stationing along the tangent lines is accomplished, the station of a 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 point of curvature to the point of tangency is shortened by $2T - L$ for each horizontal curve.

12-40. In figure 12-5, page 12-8, the initial centerline distance from the point of curvature to the point of tangency 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 point of curvature to the point of tangency becomes the length of the curve. At this point, the centerline stationed ahead would need to be restationed or adjusted. To prevent restaking the rest of the project centerline, an adjustment is made to the

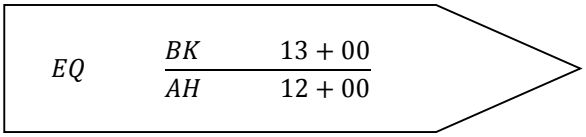
construction stake at the point of tangency. The method of adjustment will produce a stationing equation at the point of adjustment that will satisfy the stationing back and the stationing ahead. The equation will have a station that corresponds with the correct station back and the correct station ahead. The equation will be written on the construction stake as follows:



where—

PT = *point of tangency*
EQ = *equation*
BK = *correct station back*
AH = *correct station ahead*

12-41. Equations are most often used at the point of tangency, but may be used anywhere an adjustment to centerline stationing is required. The equation indicates that an adjustment to the centerline stationing has occurred for some reason. If a survey crew accidently placed two centerline stakes with the same station number (for example, at 13 + 00), the equation stake would look like this:



12-42. 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 are normally 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

12-43. The location and station of the point of curvature and point of tangency of a horizontal curve constitute two points on the curve. They do not adequately define the necessary construction. The following methods are used in 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 locate the center of the curve. This method is not recommended for precise construction or long curves. It is impractical when the terrain on the curve side of the point of intersection is difficult to negotiate and measure.
- **Deflection-angle method.** This method is usually 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 12-12.)

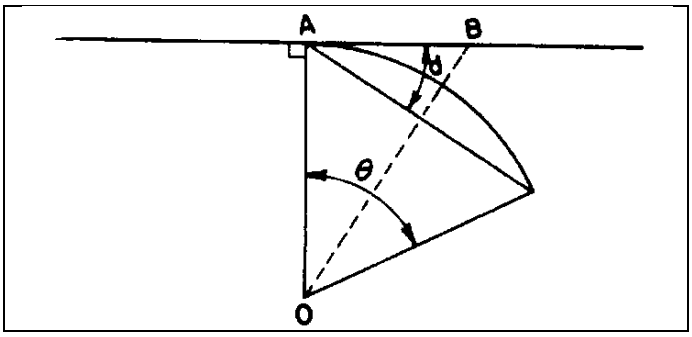


Figure 12-12. Deflection angle

Example

Find the deflection angle in the triangle in figure 12-12—

$$Angle\ ABO\ (at\ B) = 90 - \frac{0}{2}$$

$$d = 180 - (90 + B)$$

$$d = 180 - 90 - 90 + \frac{0}{2}$$

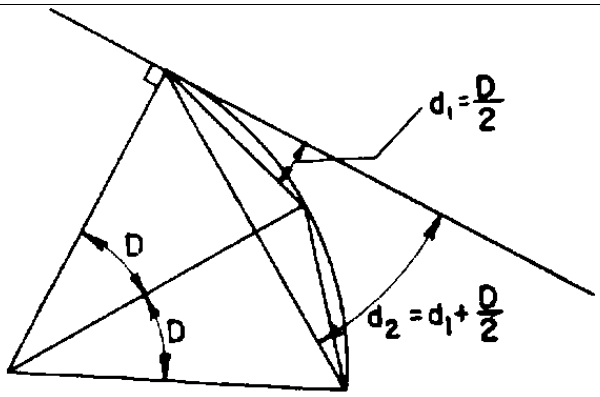
$$d = \frac{0}{2}$$

where—

d = deflection angle

Note. The deflection angle is always one-half the intercepted central angle.

12-44. If the initial arc is 100 feet long, the central angle is the degree of curvature, and the deflection angle is half the degree of curvature. (See figure 12-13.) With the addition of each 100-foot arc, the total central angle increases by the degree of curvature and the total deflection angle increases by half.



Legend:

d	deflection angle
D	degree of curvature

Figure 12-13. Deflection angles for 100-foot arcs

12-45. When laying out a curve, it is common practice to locate stakes at every full station. Because the point of curvature a curve rarely falls on an even station, the first arc will be less than 100 feet in length

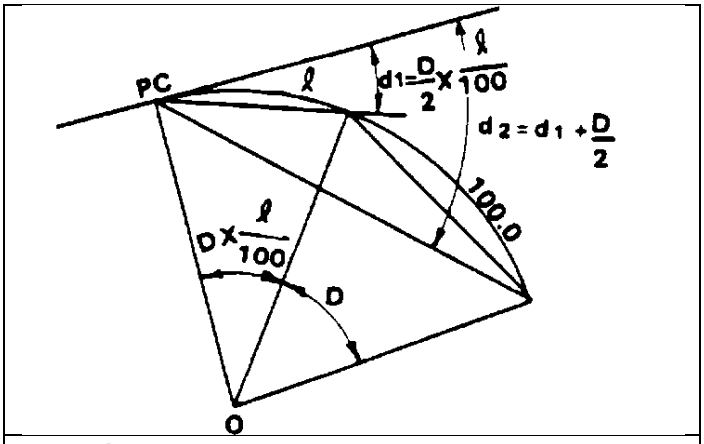
(called a subarc). The deflection angle for the subarc to the first full station is the same proportion of half the degree of curvature that the subarc is at 100 feet. (See figure 12-14.)

12-46. Once located at a full station, the curve continues by 100-foot arcs. Therefore, the central angle increases by the degree of curvature and the deflection angle increases by half the degree of curvature. (See figure 12-15.) The length of the chord for a 100-foot arc on the curve is equal to—

$$2R\ Sin\ (D/2)$$

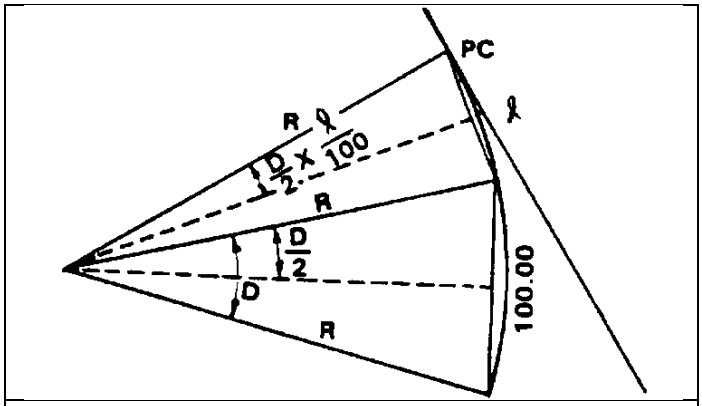
where—

D = *degree of curvature*
R = *radius*
Sin = *sine*



Legend:	
d	deflection angle
D	degree of curvature
PC	point of curvature

Figure 12-14. Subarc deflection angles



Legend:	
D	degree of curvature
PC	point of curvature
R	radius

Figure 12-15. Calculation of chord lengths

12-47. 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—

D = *degree of curvature*
L = *distance from the point of curvature or point of tangency to the closest full station*
R = *radius*
Sin = *sine*

(See figure 12-16 for a summary of the deflection-angle and chord-length calculations.)

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	$d_f = \left(\frac{D}{2} \times \frac{\ell}{100}\right) + d_{f-1}$	$C_f = 2RSin\left(\frac{D}{2} \times \frac{\ell}{100}\right)$

Legend:

C	chord length
d	deflection angle
D	degree of curvature
PC	point of curvature
PT	point of tangency
Sin	sine

Figure 12-16. Deflection-angle and chord-length determinations

12-48. In the example of a horizontal curve with no terrain restrictions, the station of point of curvature is 9 + 82.71, the station of point of tangency is 18 + 16.04. (See table 12-2, page 12-22.) A useful check for long series of computations is that the final deflection angle from the point of curvature to the point of tangency must always equal to the angle of intersection divided by 2. This is based on the previously stated principle that the deflection angle (from the point of curvature to the point of tangency) is half the total angle subtended. This check is illustrated in table 12-2.

Given—

D = *25°*

$$PT = \frac{I}{2} = \frac{500}{2} for\ Station\ 18 + 16.04$$

where—

I = *angle of intersection*
PT = *point of tangency*

Table 12-2. Deflection angles and chord distances

Station	*Deflection Computation*	*d*	*Chord Computation*	*Chord Length*
PC 9 + 82.71				
10 + 00	$d_1 = \left(\frac{D}{2} \times \frac{\varphi}{100}\right) = \frac{6}{2} \times \frac{17.29}{100} = 0.519°$ $0.519 \times 60 = 31.1$	0° 31' 6"	$C = 2R\,Sin\left(\frac{D}{2}\frac{\varphi}{100}\right)$ $= 2(954.93)(0.52336)\left(\frac{17.20}{100}\right)$	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	$d_{10} = d_9\left(\frac{D}{2} \times \frac{\varphi}{100}\right) = 24.519 + \frac{6}{2} \times \frac{16.04}{100}$ $= 24.519 + 481 = 25.00$	25° 00' 00" check $\frac{I}{2} = 25° 00'$	$C = 2R\,Sin\left(\frac{D}{2}\frac{\varphi}{100}\right) = 99.954\left(\frac{16.04}{100}\right)$	16.032'

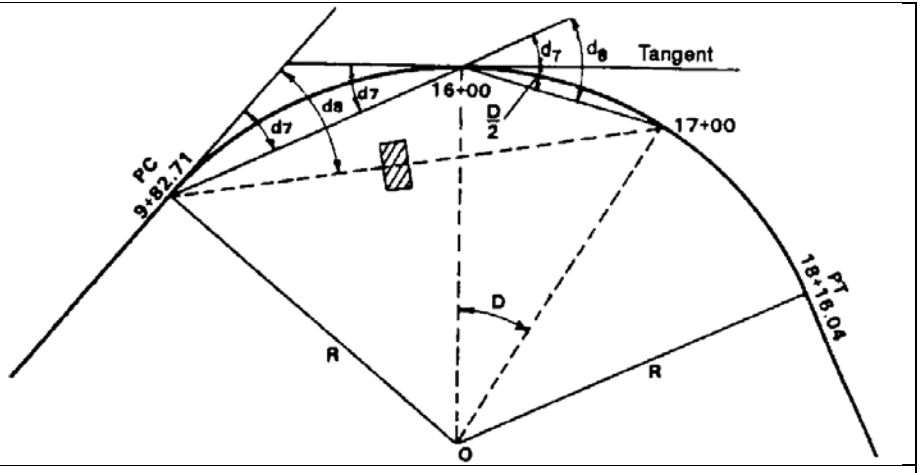
Legend:

C	chord length
d	deflection angle
D	degree of curvature
PC	point of curvature
PT	point of tangency
Sin	sine

Layout Techniques

12-49. When using the deflection-angle method, set the transit up at the point of curvature. Set zero on the vernier, sight the point of intersection (or take a backsight down the centerline), and turn the first deflection angle. Measure the subarc distance along the instrument line of sight. To locate the second point on the curve (Station 11 + 00), turn the second deflection angle (the angle is measured turning from the point of intersection to Station 11 + 00) with the transit still at the point of curvature. 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.

12-50. 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 12-17.)



Legend:	
d	deflection angle
D	degree of curvature
PC	point of curvature
PT	point of tangency
R	radius

Figure 12-17. Obstruction on a curve

12-51. If Station 17 + 00 cannot be sighted, move the transit to Station 16 + 00. Set zero on the vernier, backsight the point of curvature, and turn the angle 21° 31'6" to sight Station 17 + 00. Figure 12-17 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 d7 + D/2, which is as originally calculated.

Frequency of Placing Survey Stakes (In Feet)

12-52. 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. (See table 12-3.)

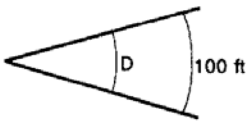
Table 12-3. Staking intervals (in feet)

Degree of Curvature (100')	*Radius (feet)*	*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

12-53. The design of horizontal curves using metric units is essentially the same as in English units. The only difference is in the relationships of arc length to the degree of curvature. (See figure 12-18, page 12-24.)

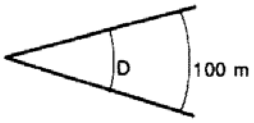
Note. The functions of a 1° curve table are applicable to metric design based on the relationship shown in figure 12-18. However, if designing using metric units, the lengths of the tangent distance, external distance, middle ordinate, and radius are in meters.



$$\frac{D_{100\,ft}}{100\,ft} = \frac{360}{2\pi R}$$

$$D_{100\,ft} = \frac{5729.58}{R(ft)}$$

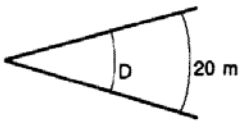
(a) D based on a 100-foot arc



$$\frac{D_{100\,m}}{100\,m} = \frac{360}{2\pi R}$$

$$D_{100\,m} = \frac{5729.58}{R(m)}$$

(b) D based on a 100-meter arc



$$\frac{D_{20\,m}}{20\,m} = \frac{360}{2\pi R}$$

$$D_{20\,m} = \frac{5729.58}{R(m)}$$

(c) D based on a 20-meter arc

LOCATION ON ARC	DEFLECTION ANGLE	CHORD LENGTH
PC to first full station	$D_1 = \frac{D_{20\,m}}{2} \times \frac{\ell}{20\,m}$	$C_1 = 2RSin\left(\frac{D_{20\,m}}{2} \times \frac{\ell}{100}\right)$
Full station to full station	$d_n = d_{n-1} + \frac{D_{20\,m}}{2}$	$C_n = 2RSin\left(\frac{D_{20\,m}}{2}\right)$
Last full station to PT	$d_f = d_{f-1} + \left(\frac{D_{20\,m}}{2} \times \frac{\ell}{20\,m}\right)$	$C_f = 2RSin\left(\frac{D_{20\,m}}{2} \times \frac{\ell}{100}\right)$

(d) Deflection-angle and chord length selection based on $D_{20\ meters}$

Note. ℓ is the distance from the point of curvature or the point of tangency to the closest full station.

Legend:

D	degree of curvature
ft	feet
m	meters
R	radius
Sin	sine

Figure 12-18. Horizontal curve design

12-54. If the curve is designed based on $D_{100\ meters}$ but staking intervals are every 20 meters, determine the degree of curvature based on 20 meters ($D_{20\ meters}$) to determine the correct deflection angles. A summary of the deflection-angle and chord-length calculations based on $D_{20\ meters}$ is shown in figure 12-18.

Frequency of Placing Survey Stakes (In Meters)

12-55. Horizontal curves should be staked at a maximum interval of 20 meters. The staking internal on horizontal curves should be based on the degree of curvature. (See table 12-4.)

Table 12-4. Staking intervals (in meters)

Degree of Curvature (100')	*Radius (meters)*	*Staking Interval*
0 to 3°	>585	20
>3° to 8°	585 to 221	10
>8° to 16°	220 to 110	5
>16°	<110	5

VERTICAL ALIGNMENT

12-56. The capabilities of vehicles or aircraft using a 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 12-1, page 12-3.

12-57. Within limitations imposed by 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 earthwork operations between cut and fill in an 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 balance a volume of cut with an equivalent volume of fill at a distance beyond the hauling capabilities of available equipment.

12-58. 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.

12-59. 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 an intersection point, the permissible depth of cut or fill, and the maximum gradients in approaching bridges or points of intersection.

PLOTTING A PROFILE VIEW

12-60. 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 half and full stations, point of vertical curvature, point of vertical intersection, and high- and low-point elevations.

12-61. 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 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 for preliminary planning and initial location. The centerline profile may not represent the typical or prevailing condition across the entire section at a 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

12-62. 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

12-63. 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).

Example

The equation for determining grades is—

$$G = \frac{V}{H}(100)$$

where—

G = *percentage of grade*
H = *horizontal distance between the two points*
V = *rise or fall between the two points*

Note. *V and H must be in the same units.*

Note. 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 to denote falling grades.

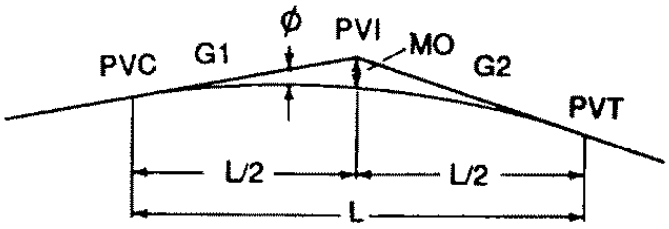
VERTICAL CURVES

12-64. 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

12-65. Two types of vertical curves must be considered: sags and crest. Figure 12-19 shows a typical vertical curve installed between two grade lines. The parts of a vertical curve include the following:

- **Point of vertical intersection.** The point of vertical intersection is the intersection of two grade lines. This station is always read from the profile view.
- **Point of vertical curvature.** The point of vertical curvature 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 point of vertical curvature is always one-half the length of the vertical curve from the point of vertical intersection, measured horizontally.
- **Point of vertical tangency.** The point of vertical tangency is the point along the second grade line at which the vertical curve ends. It has the same properties as the point of vertical curvature.
- **Percent of grade.** The percentage of grade 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 point of vertical curvature and point of vertical tangency, intersect at the point of vertical intersection.
- **Length of vertical curve.** The length of vertical curve is the horizontal distance from the point of vertical curvature to the point of vertical tangency. The point of vertical intersection is horizontally midway between the point of vertical curvature and point of vertical tangency. Therefore, the distance from the point of vertical curvature to the point of vertical intersection is the length of the vertical curve divided by 2, and the distance from the point of vertical intersection to the point of vertical tangency is also the length of the vertical curve divided by 2.
- **Offsets.** Offsets 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.
- **Maximum offset.** The maximum offset is the offset at the point of vertical intersection. It is always the greatest offset along the vertical curve.



Legend:	
G	percent slope of grade
L	length of vertical curve
MO	maximum offsets
PVC	point of vertical curvature
PVI	point of vertical intersection
PVT	point of vertical tangency

Figure 12-19. Elements of a vertical curve

Design of Vertical Curves

12-66. The design of vertical curves includes the following tasks:

- Determine the curve length.
- Calculate the heights of a sufficient number of offsets to adequately define or locate the final grade line.

12-67. 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 are usually specified.

12-68. The difference in grade between two grade lines is called the change in grade (ΔG). This difference (symbolized as *G*) is computed as |G1 – G2|, which represents the absolute value of G1 – G2. The curve installed between two grade lines with a large difference in grade, which might occur at the top of a steep hill, is longer than the curve required between two grades with a smaller difference in grade.

12-69. 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. Allowable rate of change (symbolized as *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 difference in grade of 1.0 percent requires a curve length of 1,000 feet. However, the allowable rate of change is usually expressed as the allowable gradient change in 100 feet of length. The allowable rate of change of grade is frequently used for airfield vertical curve design.

12-70. When a crest 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 (symbolized as *S*) depends on the design speed permitted.

12-71. The vertical-curve-length factor (symbolized as *k*) 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.

12-72. Determine the vertical curve length by using the vertical-curve-length factor for the given class of road (A, B, C, or D) and for the type of curve. (See table 12-1, page 12-3.) The vertical-curve-length factor is used in the following equation:

$$L = \frac{k\Delta G}{100}$$

where—

ΔG = *change in grade*
k = *vertical-curve-length factor*
L = *length determination of a vertical curve in 100-foot stations*

12-73. If length, 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 12-1.

12-74. Knowing the curve length and the station of the point of vertical intersection, compute the station of the point of vertical curvature and the point of vertical tangency by using the following equations:

$$PVC = PVI - L/2$$

$$PVT = PVI + L/2$$

where—

L = *length determination of a vertical curve*
PVC = *point of vertical curvature*
PVI = *point of vertical intersection*
PVT = *point of vertical tangency*

12-75. For the curve to be defined, the engineer must determine elevations at various locations along the curve. To do so, the engineer must determine the offset, which is the vertical distance from the original grade line to the designed curve.

12-76. As previously stated, the maximum offset will always be located at the point of vertical intersection. The following equation is used to calculate maximum offset:

$$MO = \frac{L\Delta G}{8}$$

where—

MO = *vertical height of the maximum offset in feet*
L = *length determination of a vertical curve in sections*
G = *change of grade in percent*

12-77. Offsets at locations along the curve other than the point of vertical intersection 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 equation to calculate the intermediate offsets:

$$\emptyset = \frac{\Delta G}{2L} d^2$$

where—

∅ = *offset at a distance, d, from the point of vertical curvature or point of vertical tangency in feet*
ΔG = *change in grade (in percent)*
d = *selected distance (in stations) from the point of vertical curvature or point of vertical tangency where an offset distance is calculated*
L = *curve length in stations*

12-78. Since the percent of grade and the length of the curve have been determined before using this equation, ΔG/2L is a constant. Finding the offsets is a simple matter of varying the value of the selected distance.

Note. The offsets at equal distances from the point of vertical curvature or point of vertical tangency are equal. In other words, the offsets are symmetric about the point of vertical intersection. This does not mean that the resulting curve is symmetric (unless G1 = G2). 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.

12-79. Vertical curves have the shape of a parabola. Use the following equations to calculate elevations along the vertical curve for a—

- Sag curve:

$$y = elev\ PVC - change\ in\ elevation + \emptyset$$

$$y = elev\ PVC - G1d + \frac{\Delta G}{2L}d^2$$

$$y = elev\ PVT\ \pm change\ in\ elevation + \emptyset$$

$$y = elev\ PVT \pm G2d + \frac{\Delta G}{2L}d^2$$

- Crest curve:

$$y = elev\ PVC + change\ in\ elevation - \emptyset$$

$$y = elev\ PVC + G1 - \frac{\Delta G}{2L}d^2$$

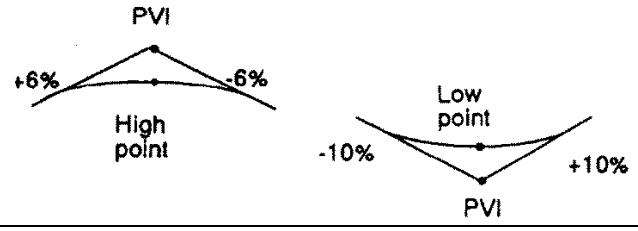
$$y = elev\ PVT \pm change\ in\ elevation - \emptyset$$

$$y = elev\ PVT \pm G2d - \frac{\Delta G}{2L}d^2$$

where—

Ø	=	*offset*
ΔG	=	*change in grade (in percent)*
d	=	*horizontal distance to a point on the curve from PVC or PVT in stations*
elev PVC	=	*elevation at point of vertical curvature (in feet)*
elev PVT	=	*elevation at the point of vertical tangency (in feet)*
G1	=	*percent slope of first grade line*
G2	=	*percent slope of second grade line*
y	=	*elevation of point on curve (in feet)*

12-80. When the tangent grades (G1 and G2) are equal, the high point or low point of the curve occurs at the point of vertical intersection. (See figure 12-20.)

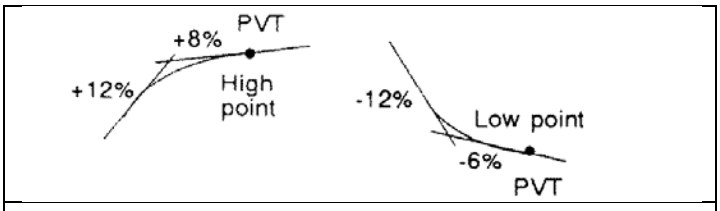


Legend:

PVI — point of vertical intersection

Figure 12-20. Equal tangent grades

12-81. When the tangent grades are the same sign (positive or negative), the high points and low points correspond with the point of vertical curvature or point of vertical tangency. (See figure 12-21.)



Legend:

PVT	point of vertical tangency

Figure 12-21. Same sign tangent grades

12-82. When the tangent grades are unequal, the high point or low point of the curve always falls on the flatter of the two grades. (See figure 12-22.)

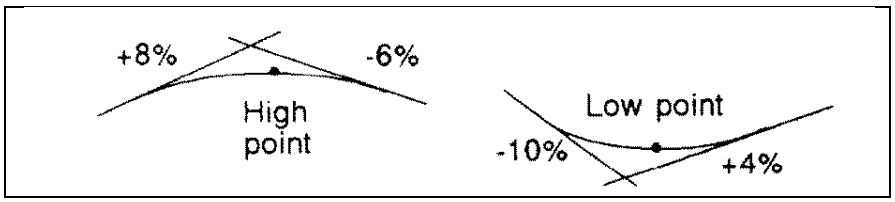


Figure 12-22. Unequal tangent grades

12-83. To determine the location along the curve of the maximum (or minimum) elevation, use the following equation:

$$d = \frac{GL}{\Delta G}$$

where—

ΔG = *change in grade (in percent)*
d = *horizontal distance along curve from point of vertical curvature (or point of vertical tangency)*
in stations
G = *percent slope of flattest grade*
L = *length of vertical curve in stations*

12-84. 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 minus 82 feet).

12-85. The following steps show the design procedure for a vertical curve:

- **Step 1.** Compute the change of grade.

$$\Delta G = |G1 - G2|$$

- **Step 2.** Compute the vertical curve length. Round up to the next higher full station (if possible).

$$L = \frac{k\Delta G}{100}$$

- **Step 3.** Determine the point of vertical curvature.

$$PVC = PVI - (L/2)$$

- **Step 4.** Determine the point of vertical tangency.

$$PVT = PVI + (L/2)$$

- **Step 5.** Determine the elevation of point of vertical curvature.

$$elev\ PVC = elev\ PVI + |G1|\ x\ (L/2)$$

- **Step 6.** Determine the elevation of point of vertical tangency.

$$elev\ PVT = elev\ PVI + |G2|\ x\ (L/2)$$

- **Step 7.** Determine the maximum offset.

$$MO = \frac{L\Delta G}{8}$$

- **Step 8.** Determine final curve elevations. Compute at every half and full station along the curve.
  - Determine grade-line elevations.

$$GLE = elev\ PVC + |G1| \times d\ from\ PVC$$
$$GLE = elev\ PVT + |G2| \times d\ from\ PVT$$

  - Determine intermediate offsets.

$$\emptyset = MO \times \left(\frac{d}{L/2}\right)^2 or \left(\frac{\Delta G}{2L}\right) d^2$$

  - Determine curve elevations.

$$curve\ elevations = GLE + \emptyset$$

- **Step 9.** Determine the location of maximum (or minimum) elevation if required.

$$d = \frac{GL}{\Delta G}$$

- **Step 10.** Determine the highest (lowest) elevation, if required.

$$Y = \frac{-\Delta G x^2}{2L} + G_1 x + elev\ PVC$$

where—

Ø	=	*offset*
ΔG	=	*change in grade (in percent)*
d	=	*horizontal distance*
elev PVC	=	*elevation at the point of vertical curvature*
elev PVI	=	*elevation at the point of vertical intersection*
elev PVT	=	*elevation at the point of vertical tangency*
G	=	*percent slope of grade*
k	=	*vertical-curve-length factor*
GLE	=	*grade-line elevations*
L	=	*length of vertical curve in stations*
MO	=	*maximum offset*
PVC	=	*point of vertical curvature*
PVI	=	*point of vertical intersection*
PVT	=	*point of vertical tangency*
Y	=	*elevation of point on curve*

Example

Complete the design of a vertical curve (to include PVC, PVT, and ∅) with cuts and fills determined every 50 feet (one-half station). The unit survey section has completed a centerline survey for a proposed vertical curve. The road is Class D.
Given—

G1 = +3.1%
G2 = -5.75%
k = 35 (see table 12-1, page 12-3, crest vertical curve, Class D road)
L = 4
PVI = 5 + 00
Stations = 400'

- **Step 1.** Determine ΔG.

$$\Delta G = |G_1 - G_2| = |(+3.1) - (-5.75)| = 8.85\%$$

- **Step 2.** Determine *L*.

$$L = \frac{k\Delta G}{100} = \frac{35(8.85)}{100} = 3.10\ stations$$

- **Step 3.** Determine *PVC*.

$$PVC = PVI - L/2 = (5 + 00) - (2 + 00) = (3 + 00)$$

- **Step 4.** Determine *PVT*.

$$PVT = PVI + L/2 = (5 + 00) - (2 + 00) = (7 + 00)$$

- **Step 5.** Determine the elevation of *PVC*.

$$\begin{aligned} elev\ PVC &= elev\ PVI \pm |G_1|(L/2) \\ &= 73.00 - |0.031|\left({}^{400}/_{2}\right)\ or\ 73 - (3.1)(4\ /\ 2) \\ &= 66.80' \end{aligned}$$

- **Step 6.** Determine the elevation of *PVT*.

$$\begin{aligned} elev\ PVT &= elev\ PVI \pm |G_1|(L/2) \\ &= 73.00 - |0.031|(400/2) or\ 73 - 5.75(4/2) \\ &= 61.50' \end{aligned}$$

- **Step 7.** Determine *MO*.

$$MO = \frac{L\Delta G}{8} = \frac{4(8.85)}{8} = 4.43'$$

- **Step 8.** Determine final curve elevations.

  - Determine *GLEs*. (See table 12-5, page 12-34.)

$$\begin{aligned} GLE &= elev\ PVC\ \pm |G_1| \times d\ from\ PVC \\ GLE &= elev\ PVT\ \pm |G_2| \times d\ from\ PVT \end{aligned}$$

  - Determine intermediate offsets.

$$\emptyset = \left(\frac{\Delta G}{2L}\right)d^2 = \left(\frac{8.85}{2(4)}\right)d^2 = 1.106d^2$$

$$\emptyset_{(zero\ stations)} = 1.106(0)^2 = 0\ (no\ offset\ at\ PVC\ and\ PVT)$$
$$\emptyset_{(0.5\ station)} = 1.106(0.5)^2 = 0.28'(at\ stations\ 3 + 50\ and\ 6 + 50)$$
$$\emptyset_{(1.0\ station)} = 1.106(1.0)^2 = 1.11'(at\ stations\ 4 + 00\ and\ 6 + 00)$$
$$\emptyset_{(1.5\ station)} = 1.106(1.5)^2 = 2.49'(at\ stations\ 4 + 50\ and\ 5 + 50)$$
$$\emptyset_{(2.0\ station)} = 1.106(2.0)^2 = 4.43'(at\ station\ 5 + 00)$$

Example (continued)

Note. The term ΔG/2L becomes a constant (1.106).

- **Step 9.** Determine the location of the maximum or minimum elevation if required.

$$D = \frac{GL}{\Delta G} = \frac{(3.1)(4)}{8.85} = 1.40\ stations\ or\ 140'$$

Note. The maximum or minimum elevation is always on the flattest grade when grades have opposite signs.

$$Station\ maximum\ elevation\ point = PVC + d\ (or\ PVT - d)$$
$$= (3 + 00) + (1 + 40) = 4 + 40$$

- **Step 10.** Determine the highest elevation if required.

$$y = elev\ PVC\ (or\ PVT) + Gd \pm \emptyset$$
$$y = PVC_{elev} + Gd - \left(\frac{\Delta G}{2L}\right) d^2$$
$$y = 66.80 + 3.1(1.4) - \frac{8.85}{8}(1.4)^2$$
$$y = 66.80 + 4.34 - 2.17 = 68.97$$

Note. The + or – is determined by inspection.

where—

Ø	=	*offset*
ΔG	=	*change in grade (in percent)*
d	=	*horizontal distance*
elev PVC	=	*elevation at the point of vertical curvature*
elev PVI	=	*elevation at the point of vertical intersection*
elev PVT	=	*elevation at the point of vertical tangency*
G	=	*percent slope of grade*
GLE	=	*grade-line elevations*
K	=	*vertical-curve-length factor*
L	=	*length of vertical curve in stations*
MO	=	*maximum offset*
PVC	=	*point of vertical curvature*
PVI	=	*point of vertical intersection*
PVT	=	*point of vertical tangency*

12-86. When the vertical curve must go through a known point and the known point is at the point of vertical intersection, the length of the curve can be determined using the following formula:

$$L = \frac{MO(8)}{\Delta G}$$

where—

ΔG	=	*change in grade (in percent)*
L	=	*length of curve*
MO	=	*maximum offsets*

Table 12-5. Compiling vertical curve design data

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

Legend:

Ø	offset
d	horizontal distance
GLE	grade-line elevations

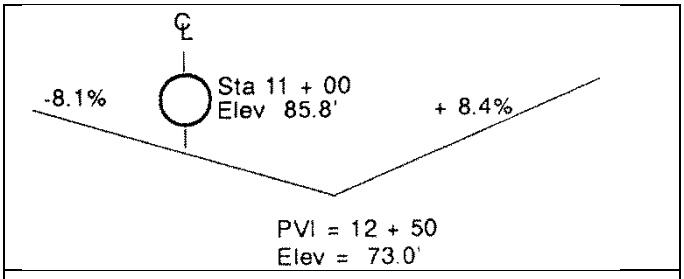
12-87. When the known point is at a location other than the point of vertical intersection, the length of the curve can be determined using the following formula:

$$L = 2\left[A + \frac{2(\emptyset)}{\Delta G}\right] + 4\sqrt{\left(\frac{A(\emptyset)}{\Delta G}\right) + \left(\frac{\emptyset}{\Delta G}\right)^2}$$

where—

$\varnothing$ = *offset between elevation of known point and GLE of known point*
ΔG = *change in grade (in percent) (must be entered as a decimal)*
A = *horizontal distance from point of vertical intersection to known point*
L = *length of curve*
MO = *maximum offsets*

12-88. 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. (See figure 12-23 for the next example.)



Legend:

elev	elevation
PVI	point of vertical intersection
Sta	station

Figure 12-23. Vertical curve design procedures

Example

A new road with a point of vertical intersection 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. Determine the length of the vertical curve required to clear this culvert with 1 foot of cover.

- **Step 1.** Determine horizontal distance (A) from the PVI to the known point.

$$A = 1{,}250' - 1{,}100' = 150'$$

- **Step 2.** Determine ΔG (as a decimal).

$$\Delta G = |G1 - G2| = |-0.081 - 0.084|$$
$$\Delta G = 0.165$$

- **Step 3.** Determine the elevation of the vertical curve required to clear the culvert with 1 foot of cover (at the known point).

$$elevated\ curve = 85.8' + 2.0'\ (pipe\ diameter) + 1.0'(cover\ required)$$
$$elevated\ curve = 88.8'$$

- **Step 4.** Determine *GLE* at the known point (directly below the culvert).

$$GLE = ELPVI + |G1| \times d\ (from\ PVI\ to\ known\ point)$$
$$GLE = 73.0' + |0.081| \times 150'$$
$$GLE = 85.15'$$

- **Step 5.** Determine $\varnothing$ between the elevation of a known point and the *GLE* of a known point.

$$\emptyset = 88.8' - 85.15' = 3.65'$$

- **Step 6.** Determine the length of the vertical curve.

$$L = 2\left[A + \frac{2(\emptyset)}{\Delta G}\right] + 4\sqrt{\left(\frac{A(\emptyset)}{\Delta G}\right) + \left(\frac{\emptyset}{\Delta G}\right)^2}$$

$$L = 2\left[150' + \frac{2(3.65')}{0.165}\right] + 4\sqrt{\left(\frac{150(3.65)}{0.165}\right) + \left(\frac{3.65}{0.165}\right)^2}$$
$$= 388.48' + 246.82'$$
$$= 635.30'$$

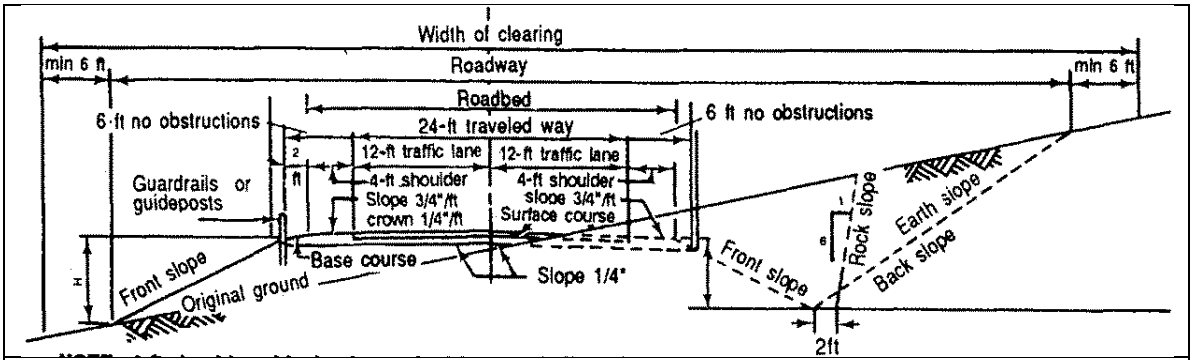
where—

Ø	=	*offset*
ΔG	=	*change in grade (in percent)*
A	=	*horizontal distance*
d	=	*horizontal distance*
elev PVI	=	*elevation at the point of vertical intersection*
G	=	*percent slope of grade*
GLE	=	*grade-line elevations*
L	=	*length of curve*
PVI	=	*point of vertical intersection*

12-89. The design of vertical curves using metric units is the same as English units. The only difference is the use of meters as units of measurement for elevations and distances. 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.

CROWNED SECTION

12-90. 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 2 percent for asphalt and 4 percent for aggregate surfaced roads. Figure 12-24 shows a typical cross section for a Class A road. If the road is to be surfaced, the subgrade and the finished surface will have the same crown.



Note. A 3 foot shoulder widening is required for guardrails and guideposts.

Legend:

ft	feet
min	minimum

Figure 12-24. Normal crown cross section for a Class A road

12-91. Design slopes to be as flat as is reasonable. Generally, low cuts and fills are economical to construct on relatively flat slopes and will enhance aesthetics, safety, and maintenance. Where practical, embankment front slopes should be 1V:4H or flatter. Slopes 1V:3H are generally traversable by an errant vehicle that has run off the road but do not provide for vehicle recovery. Front slopes steeper than 1V:3H will cause most vehicles to overturn. (See table 12-6 for desirable and maximum slope ratios.)

Table 12-6. Desirable and maximum slopes

Cut and Fill Slope Ratios for Soil Materials							
Height (feet)	***Slope Type***	***Flat***		***Rolling***		***Mountainous***	
		Desirable	***Maximum***	***Desirable***	***Maximum***	***Desirable***	***Maximum***
0-3	Cut	1V:6H	1V:4H	1V:6H	1V:4H	1V:6H	1V:3H
	Fill	1V:6H	1V:4H	1V:6H	1V:4H	1V:6H	1V:4H
3-10	Cut	1V:4H	1V:3H	1V:3H	1V:2H	1V:3H	1V:2H
	Fill	1V:4H	1V:4H	1V:4H	1V:4H	1V:3H	1V:3H
10-15	Cut	1V:3H	1V:2H	1V:3H	1V:2H	1V:3H	1V:2H
	Fill	1V:4H	1V:3H	1V:4H	1V:3H	1V:3H	1V:2H
15-20	Cut	1V:3H	1V:2H	1V:2.5H	1V:2H	1V:2H	1V:1.5H
	Fill	1V:3H	1V:2H	1V:3H	1V:2H	1V:2H	1V:1.5H
>20	Cut	1V:3H	1V:2H	1V:2H	1V:1.75H	1V:2H	1V:1.5H
	Fill	1V:3H	1V:2H	1V:3H	1V:1.75H	1V:2H	1V:1.5H

Legend:

H	horizontal
V	vertical

12-92. Variable slope ratios for cut and fill slopes are desirable to minimize earthwork in high fill areas and enhance safety in low fill areas. Avoid using constant slope ratios. When varying slope ratios, a rule-of-thumb is at minimum to vary one unit of the horizontal slope ratio over 50 feet of linear roadway distance,

such as a transition from a 1V:2H slope to a 1V:4H slope should be over a minimum distance of 100 feet, for constructability.

12-93. Cut and fill slopes steeper than 1V:2H should be avoided in clay and silty soils subject to erosion. Loessal soils require special design for cut slopes. Embankment height and side slope are the basic factors considered in determining whether a guardrail is needed. Embankment side slopes 1V:3H and flatter do not require guardrail. Embankments with a side slope of 1V:2H and a height greater than 6 feet require a guardrail.

SUPERELEVATION

12-94. 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 curves is in TM 3-34.55. Table 12-7 lists superelevation rates and appropriate transition lengths to develop the superelevated section as a function of the design speed and degree of curvature. Figure 12-25, page 12-38, shows a Class A road section with a superelevated curve.

12-95. As a safety 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.

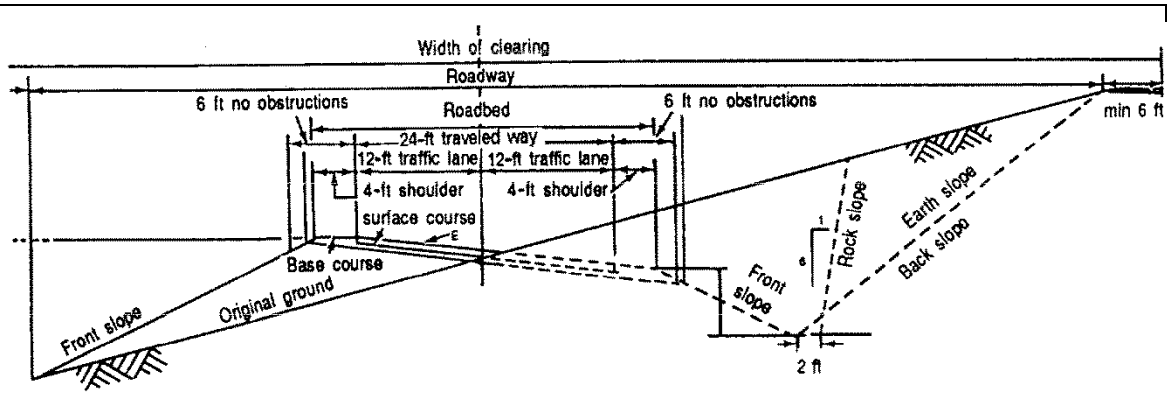
Table 12-7. Superelevation lengths and transition lengths

D	*R*		*V = 30 mph (48 kph)* *L*			*V = 40 mph (64 kph)* *L*			*V = 60 mph (97 kph)* *L*		
	feet	meters	E	feet	meters	E	feet	meters	E	feet	meters
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	150	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 = 28.7°								

Note. Transitions are desirable, but not as essential above the heavy line.

Table 12-7. Superelevation lengths and transition lengths (continued)

Legend:	
D	degree of curvature
E	rate of superelevation
kph	kilometers per hour
L	minimum length of transition
max	maximum
mph	miles per hour
NC	normal crown section
R	radius of curvature
RC	remove adverse crown, superelevate at normal crown slope
V	design speed



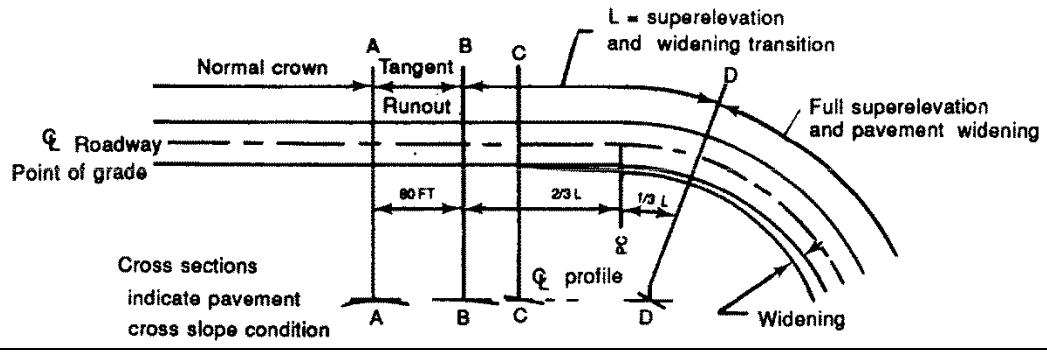
Legend:	
ft	feet
min	minimum

Figure 12-25. Superelevated cross section

12-96. Table 12-8 lists pavement widening requirements. 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. (See figure 12-26.) The length of highway needed to accomplish this transition is given in table 12-7, page 12-37. Two-thirds of the specified transition length is affected on the tangent, and one-third is affected on the curve.

Table 12-8. Pavement widening

	Widening for 2-lane pavements on curves for width of pavement with a tangent of 20 feet		
	Design Speed		
Degree of curve	*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'	—	—
Legend:			
mph	miles per hour		



Legend:	
℄	centerline
ft	feet
L	minimum length of transition (superelevation and widening transition)
PC	point of curvature

Figure 12-26. Method for transition (not to scale)

SPECIAL CONSIDERATIONS FOR ROADS IN MOUNTAINOUS TERRAIN

12-97. There are a number of design and construction elements that should be considered in mountainous terrain. The following paragraphs provide guidance on vertical alignments, road prisms or bench cuts, and embankments.

VERTICAL ALIGNMENT

12-98. Divide long grades with relief sections (level or adversely sloped) where possible to prevent heavy truck brake overheating and failure. Switchbacks may be necessary in mountainous areas with steep grades. Switchbacks are designed with a minimum centerline radius of 60 feet. The roadway width through switchback horizontal curves will have to be widened to allow vehicles to negotiate the curve. Where practical, reduce the gradient through sharp switchback curves to facilitate braking and vehicle control in the vicinity of the switchback. For switchbacks with a curve speed of 20 miles per hour, a maximum gradient of 4 percent is recommended. For switchbacks with a curve speed of 25 miles per hour, a maximum gradient of 5 percent is recommended.

ROAD PRISM SELECTION

12-99. The proper design of the roadway prism can significantly reduce the amount of sediment and debris that enters adjacent streams. Often the basic cause of a particular mass failure can be traced to overroading or overdesign. Cuts facilitate stability within the roadway by reflecting stability downward.

Sliver Bench Cut

12-100. Sliver bench construction involves excavating a bench in the existing embankment and using the material to construct a sliver fill. A sliver fill is defined as a wedge-shaped fill used to form the road embankment.

12-101. Where the existing sideslopes are 1V:3H or less, the sliver fill may be constructed by sidecasting excavated material and compacting in lifts. Existing slopes 1V:4H to 1V:3H should be plowed, stepped, or otherwise broken up so that fill material will bond with the existing material.

12-102. Where the existing sideslopes exceed 1V:3H, the sliver fill should be keyed into the existing embankment slope using 2- to 8-foot high benches to allow for a stable joining of new and existing embankment material as indicated in figure 12-27. The minimum bench width should be 12 feet to accommodate placement and compaction operations and equipment. Construction of sliver fills by simply sidecasting excavated soil over the edge of the existing embankment is not recommended. This technique does not allow for adequate compaction of the sliver fill.

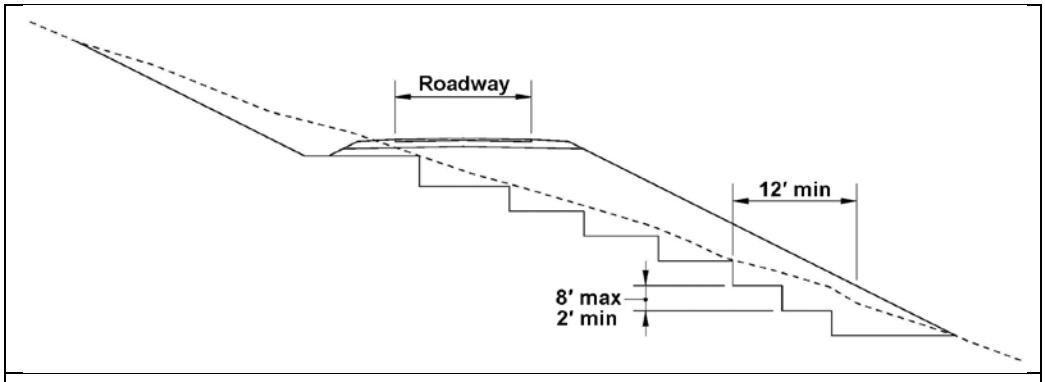


Figure 12-27. Sliver bench cut

Full Bench Cut

12-103. Full bench construction typically involves excavation of the roadbed using a hydraulic excavator. (See figure 12-28.) A bench is cut into the rock or soil equal to the width of the road plus the width of the ditch, if required. Normally, no material is sidecast, and excavated waste is used to fill low areas along the road alignment. Full bench road construction is typically used on steeper sideslopes over 1V:2H. Endhauling of excavated material can be expensive, and full bench construction can cost four to seven times more than balanced cut and fill methods. Also, full bench roads often result in tall cut slopes. Several rock and soil types may not support these large cut slopes.

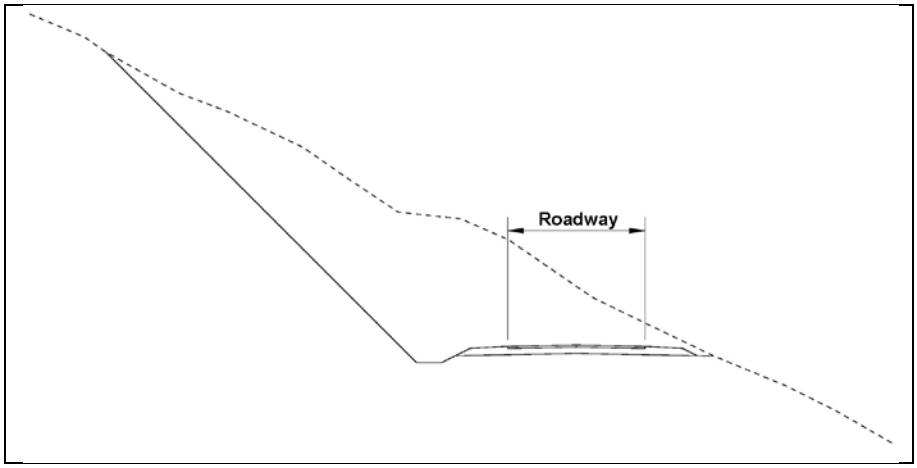


Figure 12-28. Full bench cut

Fill-Side Retaining Wall

12-104. Where the existing sideslopes are steeper than 1V:1.5H, the sidefill may have to be contained within a suitable retaining wall to avoid long fill slopes. (See figure 12-29.) Suitable retaining wall types include reinforced concrete walls, mechanically stabilized earth walls, soil nail walls, and bin walls.

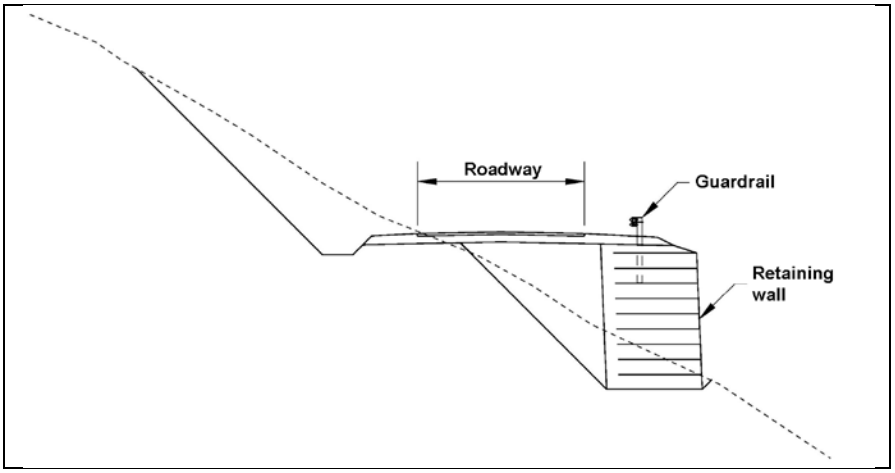
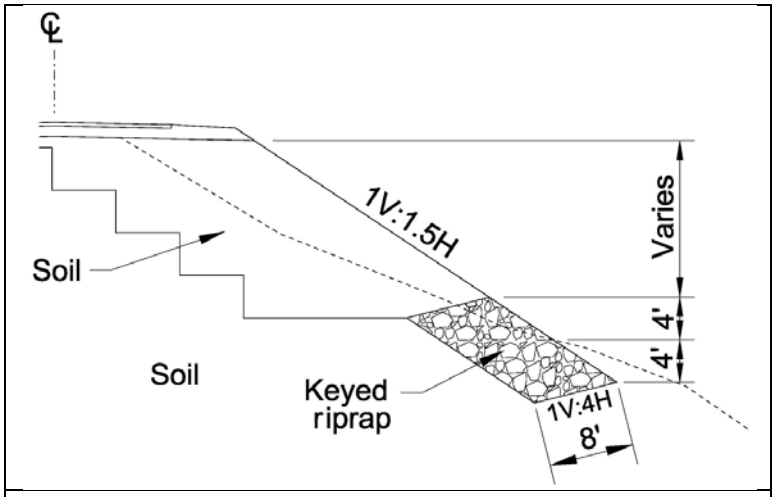


Figure 12-29. Fill-side retaining wall

EARTHEN EMBANKMENTS

12-105. Embankment benching is designed for construction of embankments on existing ground that is sloping by 1V:3H or steeper and for narrow embankments. Cut horizontal benches in the existing slope to a sufficient width to accommodate placement and compaction operations and equipment. Bench the slope as

the embankment is placed and compacted in layers. Begin each bench at the intersection of the original ground and the vertical cut of the previous bench. A riprap keyway should be constructed at the base of earthen embankments with front slopes of 1V:1.5H as indicated in figure 12-30.



Legend:

℄	centerline
H	horizontal
V	vertical

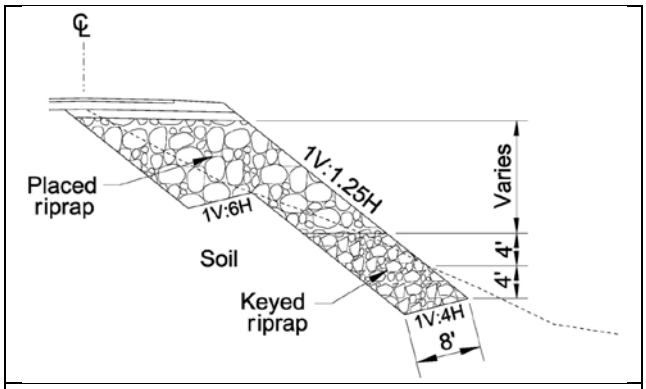
Figure 12-30. Earth embankment with riprap keyway

12-106. Place embankment material in horizontal layers not exceeding 12 inches in compacted thickness. Incorporate oversize boulders or rock fragments into the 12-inch layers by reducing them in size or placing them individually. Material composed primarily of boulders or rock fragments too large for 12-inch layers may be placed in layers up to 24 inches thick. Incorporate oversize boulders or rock fragments into the 24-inch layer by reducing them in size or placing them individually. Place sufficient earth and smaller rocks to fill the voids, and compact.

12-107. Place rock fragments and boulders greater than 24 inches in diameter individually. Reduce rock to less than 48 inches in the largest dimension. Distribute rock within the embankment to prevent nesting. Place layers of embankment material around each rock. Fill the voids between rocks. Compact each layer before placing the next layer.

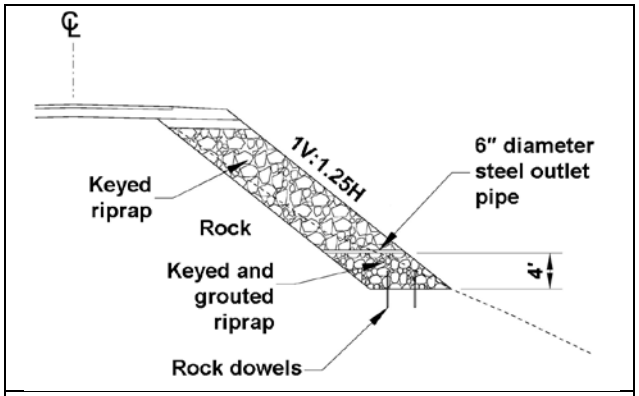
ROCK EMBANKMENTS

12-108. Rock embankments may be used where steeper front slopes are required in mountainous terrain. Embankment benching is constructed when required for earthen embankments. Embankment benching is required for soil and rock foundations. Where rock embankments are being constructed on a soil foundation, the bottom layer of riprap should be keyed into the soil to provide a good interlock between the rock and the ground. (See figure 12-31.) Where rock embankments are being constructed on a rock foundation, dowels should be installed in the existing rock, and the bottom layer of riprap should be grouted. (See figure 12-32.) Dowels should be grouted into the existing rock and spaced at 3- to 6-foot intervals.



Legend:	
℄	centerline
H	horizontal
V	vertical

Figure 12-31. Placed riprap with riprap keyway



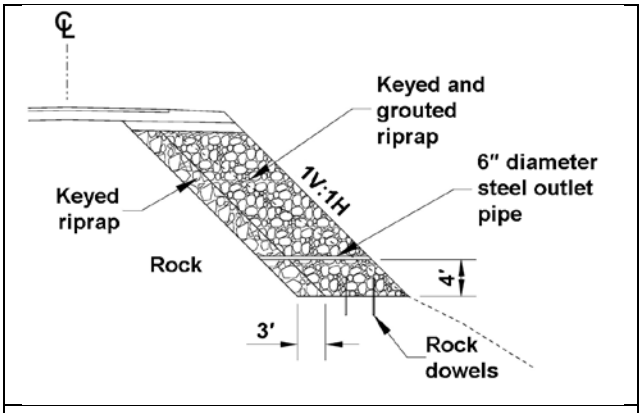
Legend:	
℄	centerline
H	horizontal
V	vertical

Figure 12-32. Keyed riprap with rock dowels

12-109. The maximum slope of rock embankments constructed with riprap is 1V:1.25H. The rock embankment can be constructed of placed riprap or keyed riprap. Generally, rock embankments are a combination of placed and keyed riprap. Placed riprap is rock placed by hand or with a machine to obtain a dense uniform blanket. Keyed riprap is rock placed on a prepared surface and set into place by impact pressure. Set the riprap into place by exerting impact pressure with a hydraulic-powered bucket or an approximate 5,000-pound, 4-foot by 5-foot flat faced mass. Repeated impacts are made until the rock is firmly seated and forms a reasonably uniform surface without reducing the effective sizes of the rock. The top portion of rock embankments below the pavement should be covered with geotextile or choked with smaller rock to prevent pavement subgrade materials from migrating into voids between the riprap.

12-110. The maximum slope of rock embankments constructed with grouted riprap is 1V:1H. Grouted riprap is rock placed or keyed on a prepared surface, and the voids are filled with grout. Each layer of grouted riprap should not exceed 5-foot in thickness. (See figure 12-33, page 12-44, and figure 12-34, page 12-44.) After placing each riprap layer, thoroughly moisten the rocks and wash excess fines from the riprap. Place grout when the air temperature is no less than 35 degrees Fahrenheit within the near-surface voids of the riprap. Place the grout in a manner to prevent segregation. Begin placing grout at the lowest elevation of the riprap.

Fill voids without unseating the rocks. Allow 3 days curing time before adding the next layer of riprap and grout. Keep grouted riprap moist for 3 days after the work is completed. Protect the area from freezing for a minimum of 7 days after grouting is completed. Keyed riprap may be used behind the grouted riprap in some circumstances to reduce the amount of grout that is required.



Legend:

℄	centerline
H	horizontal
V	vertical

Figure 12-33. Grouted riprap with rock dowels

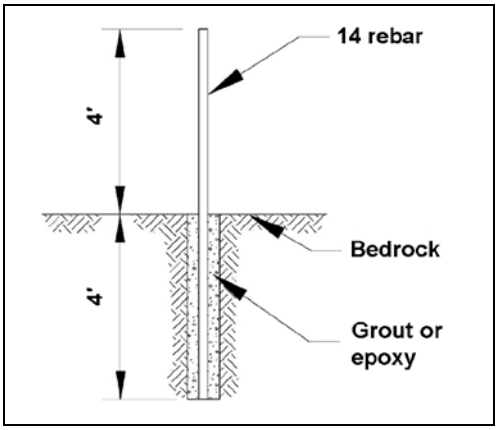


Figure 12-34. Rock dowel

ROCK CUT SLOPES

12-111. Discontinuities (joints, foliations, shears, faults) are important factors in the stability of rock slopes. The orientation, frequency, persistence, and strength of rock discontinuities are obtained from existing cuts, outcrops, or rock cores. The shear strength of rock along the discontinuities that separate the rock mass into discrete blocks is a more critical rock slope stability parameter than the strength of the intact rock. Groundwater pressure acting with discontinuities can cause significant destabilization by decreasing the shear strength due to uplift and increasing the driving forces acting on the block. Typically, the groundwater level within a slope can be estimated by observing seepages from and around the rock slope.

12-112. Some slope construction methods damage the rock so that the finished cut slope has an increased likelihood of long-term rockfall. Uncontrolled blasting can cause fracturing and open existing fractures tens of feet into the slope. A finished cut slope can be constructed by excavating rock using heavy equipment ripping or production blasting techniques, or it can be augmented with controlled blasting methods. The use

of controlled blasting, presplitting (preshear) or trim (cushion) blasting, produces a cut slope with significantly less potential for rockfall. Lightly loaded, aligned, and closely spaced blast holes are used to form the final cut slope face to minimize the effects of intense detonation gas pressures caused by production blasting. Controlled blasting is performed before main production blasting is detonated (presplit blasting) or after production blasting (cushion blasting). In presplit blasting, a row of control blast holes is detonated to form a break in the slope along the final cut slope, which serves to vent production gas pressure and keep it from penetrating and damaging the rock that will form the final cut face. In cushion blasting, a row of control blast holes is detonated last to trim off the rock outside the cut slope. The cushion blasting technique is normally used in weaker rock conditions or when the rock thickness to be excavated is less than 15 feet. Controlled blasting is routinely used for rock cuts that are 1V:0.75H or steeper.

12-113. Avoid cut slopes in bedrock dipping down toward the road (downwards out of the slope) because this type of slope may cave boulders onto the road later. This type of geologic alignment s prone to direct subsurface drainage toward the road or fill slope. Consider stabilizing uphill slopes by removing perched boulders and cracked rock faces.

12-114. Rock slope and rockfall issues can often be mitigated by several different options, including fall out ditches, barriers, catch fences, draped mesh, rock bolting, dowels, shotcrete facing, cable lashing, and buttresses. Cuts less than 20 feet in height generally do not require a fallout ditch. Rock slopes higher than 30 feet from shoulder grade may require wider fallout ditches.

12-115. When soil or highly weathered rock overlays solid rock, consider designing overburden benches at the top of the solid rock. The overburden slope should range from 1V:3H to 1V:2H, depending on the type and depth of overburden and the steepness of the topography. (See table 12-9.)

Table 12-9. Maximum rock cut slopes

Rock Type	*Maximum Cut Slope Ratio*	
	Massive	*Fractured*
Igneous (granite, trap, basalt, volcanic tuff)	1V:0.25H	1V:0.50H
Sedimentary (massive sandstone, limestone)	1V:0.25H	1V:0.50H
Interbedded (sandstone, shale, limestone)	1V:0.50H	1V:0.75H
Massive (claystone, siltstone)	1V:0.75H	1V:1H
Metamorphic (gneiss, schist, marble)	1V:0.25H	1V:0.50H
Slate	1V:0.50H	1V:0.75H
Legend:		
H horizontal		
V vertical		

SPRAY APPLICATIONS AND SURFACE TREATMENTS

12-116. 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.

12-117. Spray applications, when used as a prime coat or for dustproofing, provide soil or aggregate surfaces with the following benefits:

- Waterproofing.
- Dust control.
- Stabilization of the surface.

12-118. Spray applications are also used for tack coats and fog seals. The tack coat is used when an asphalt mixture is placed over an existing paved surface. In this case, a tack coat is applied to the existing surface to

bond the two layers together. A tack coat should be used each time an additional layer is placed. The fog seal is applied to an existing asphalt surface to help seal surface voids and rejuvenate the surface.

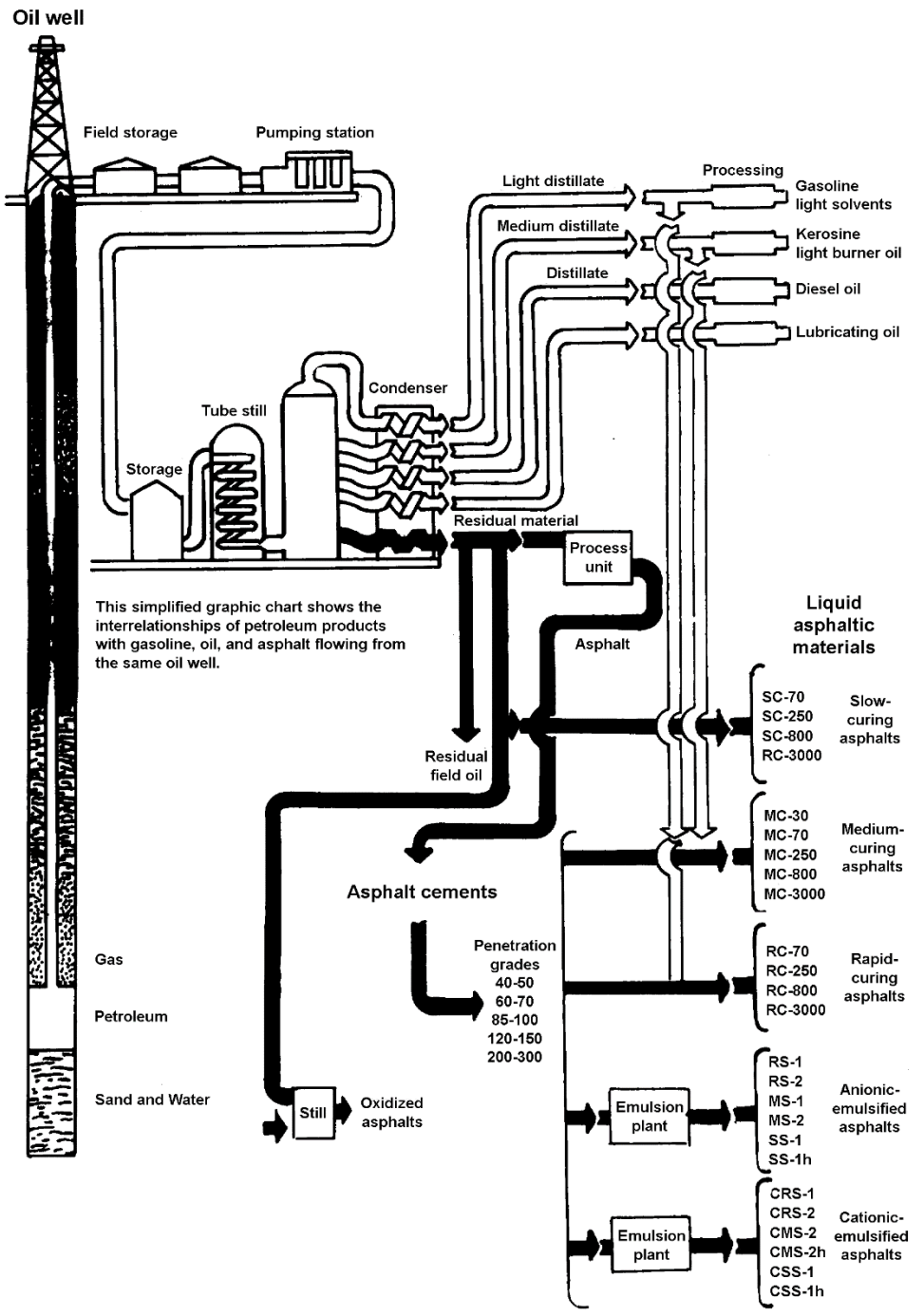
12-119. Sprayed bitumen with an aggregate surface provides a waterproof, abrasive, and wear-resistant surface. The surface treatment provides little additional structural strength over the underlying base course. Bitumen with aggregate surface treatments will be single-surface or multiple-surface treatment.

12-120. Bituminous materials are emulsified coal tar, asphalt cement, asphalt cutbacks, or asphalt emulsions. (See table 12-10 for uses of bituminous materials.) 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 other products. (See figure 12-35.) Asphalt cement may then be further modified by cutting back the asphalt with a petroleum product (such as naphtha, gasoline, kerosene, or diesel) to form an asphalt cutback. Asphalt can also be converted into an emulsion by mixing asphalt cement, water, and an emulsifying agent together with variable-speed pumps to form an asphalt water suspension. Emulsions are anionic (carrying a negative charge) or cationic (carrying a positive charge). The emulsifying agent determines whether the emulsion is anionic or cationic and whether it is slow set, medium set, or rapid set.

Table 12-10. Use of bituminous materials

Type of Construction	*Asphalt Cements*				*Cutback Asphalts*			*Emulsified Asphalts*		*Coal Tar Emulsion*
	Viscosity Graded Original	*Viscosity Graded Residue*	*Penetration Graded*	*Performance Graded*	*Rapid Curing*	*Medium Curing*	*Slow Curing*	*Anionic*	*Cationic*	
Hot Mix Asphalt	AC-10 AC-20	AR-4000 AR-8000	60-70 85-100	PG 64-22 PG 76-22 PG 58-28	—	—	—	—	—	—
Cold Placed Asphalt Mix	—	—	—	—	RC-250 RC-800	MC-250 MC-800	SC-250 SC-800	SS-1h SS-1	CSS-1 CSS-1h	—
Mixed in Place Material	—	—	—	—	RC-250 RC-800	MC-250 MC-800	SC-250 SC-800	SS-1 SS-1h	CSS-1 CSS-1h	—
Bituminous Surface Treatment	—	—	—	—	RC-250 RC-800 RC-3000	—	—	RS-1 RS-2	CRS-1 CRS-2	—
Slurry Seal	—	—	—	—	—	—	—	SS-1 SS-1h	CSS-1 CSS-1h	—
Fog Seal	—	—	—	—	—	—	—	SS-1 SS-1h	CSS-1 CSS-1h	—
Prime Coat	—	—	—	—	—	MC-70 MC-250	—	SS-1 SS-1h	—	—
Tack Coat	—	—	—	—	RC-250 RC-800	—	—	RS-1 SS-1 SS-1h	CRS-1 CSS-1 CSS-1h	—
Dust Control	—	—	—	—	RC-70	MC-70 MC-250	SC-70 SC-250	SS-1 SS-1h	CSS-1 CSS-1h	—

Notes.
1. The asphalt grades shown are commonly used and are not meant to be a complete list. The stiffer grades of asphalt cement are typically used in hotter climates and in high-traffic areas.
2. Asphalt cement can be used as a tack coat if heated sufficiently to spray over the surface being overlaid.
3. In many places, cutback asphalt is not available and cannot be used for environmental reasons. In these areas, emulsified asphalt should generally be used.
4. Emulsified tar should be used in areas subjected to petroleum products to help protect asphalt pavement from the adverse effects of petroleum.

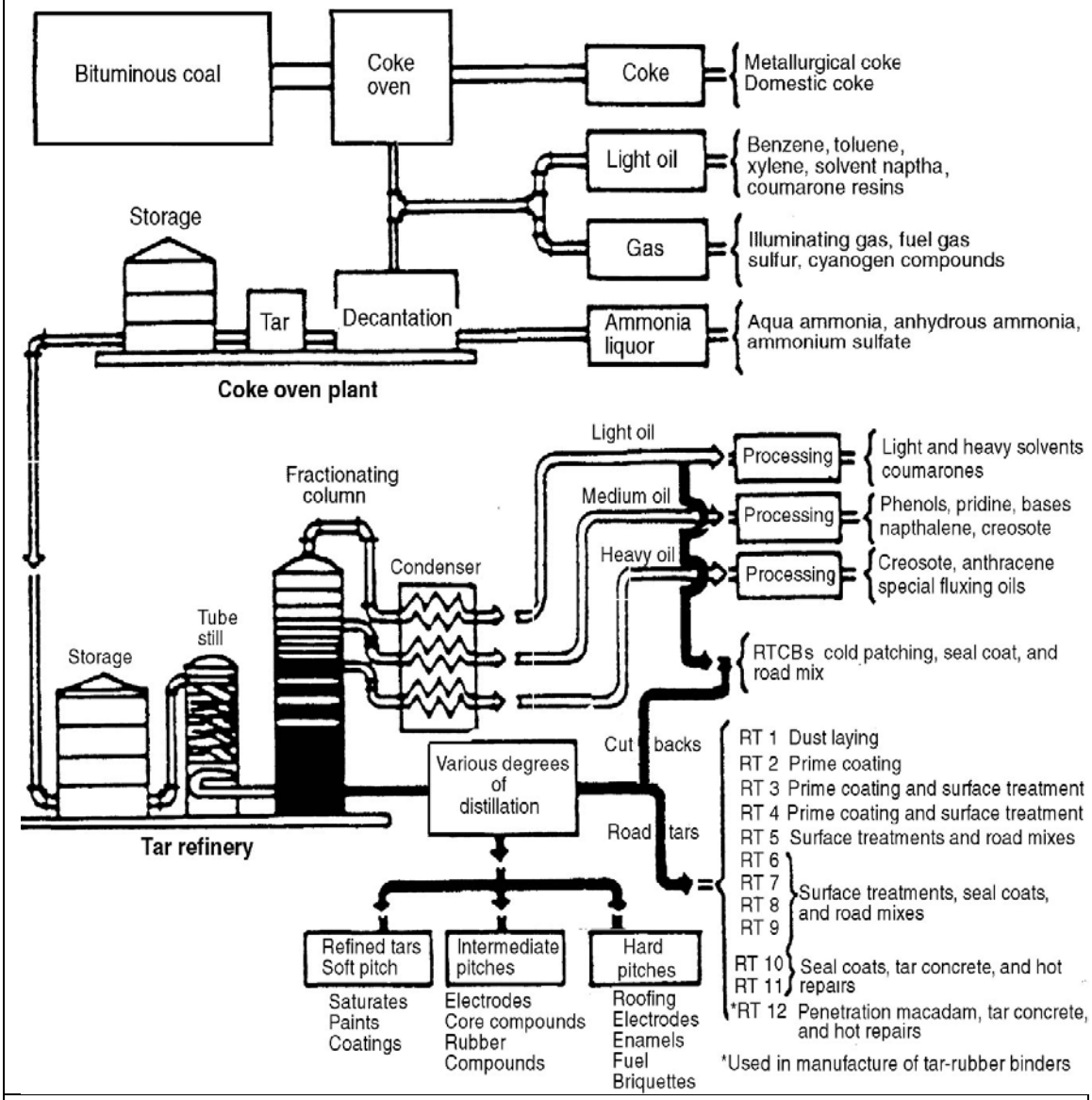


Legend:

CMS	cationic medium setting	MS	medium setting
CRS	cationic rapid setting	RC	rapid curing
CSS	cationic slow setting	RS	rapid setting
h	hard	SC	slow curing
MC	medium curing	SS	slow setting

Figure 12-35. Petroleum-asphalt flowchart

12-121. Tars are the residue from high-temperature conversion of coal to coke. (See figure 12-36.) While there are tars, cutback tars, and emulsified tars, it is recommended that only emulsified tars be considered for use since tar has more environmental issues than asphalt. These tar materials should be used in areas that expect to be exposed to petroleum products (gasoline, oil, grease). (See the ASTM on standard specification for emulsified refined coal-tar (ready to use, commercial grade) for additional information.) Tars must be handled with care when heated because the emissions may irritate the skin. For this reason, the only tar discussed in this manual is emulsified coal tar, which can be placed at relatively low temperatures. Tars do not dissolve in petroleum products. Tars should be used to seal existing asphalt pavements to provide waterproofing and fuel resistant qualities to the pavement.



Legend:

RT road tar
RTCB road-tar cutback

Figure 12-36. Tar simplified flow chart

FIELD IDENTIFICATION

12-122. 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 use of a bituminous material and how to use it safely, not the exact specifications that it conforms to. Field-identification procedures are applicable to tars and asphalts.

12-123. Bituminous materials are often stockpiled in unmarked or incorrectly marked containers. This leads to confusion and delay during construction because 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 can be determined. The safety procedures to be followed also depend on the material identification.
- Once the surface type is known, the construction procedure can be outlined and scheduled for a specific target date.
- Once established, the procedure will determine the proper equipment for the job.

12-124. Field tests may be performed to identify the bituminous paving material as asphalt cement, asphalt cutback, asphalt emulsion, or emulsified coal tar. In addition, it is necessary to identify the viscosity grade of the bitumen as close as possible. To distinguish among several asphalt and tar products, it is necessary to know something of their origin, physical properties, and the way they are normally used. The identification procedure outlined is based on the physical properties of these materials. (See figure 12-37, page 12-50.)

Asphalts and Tars

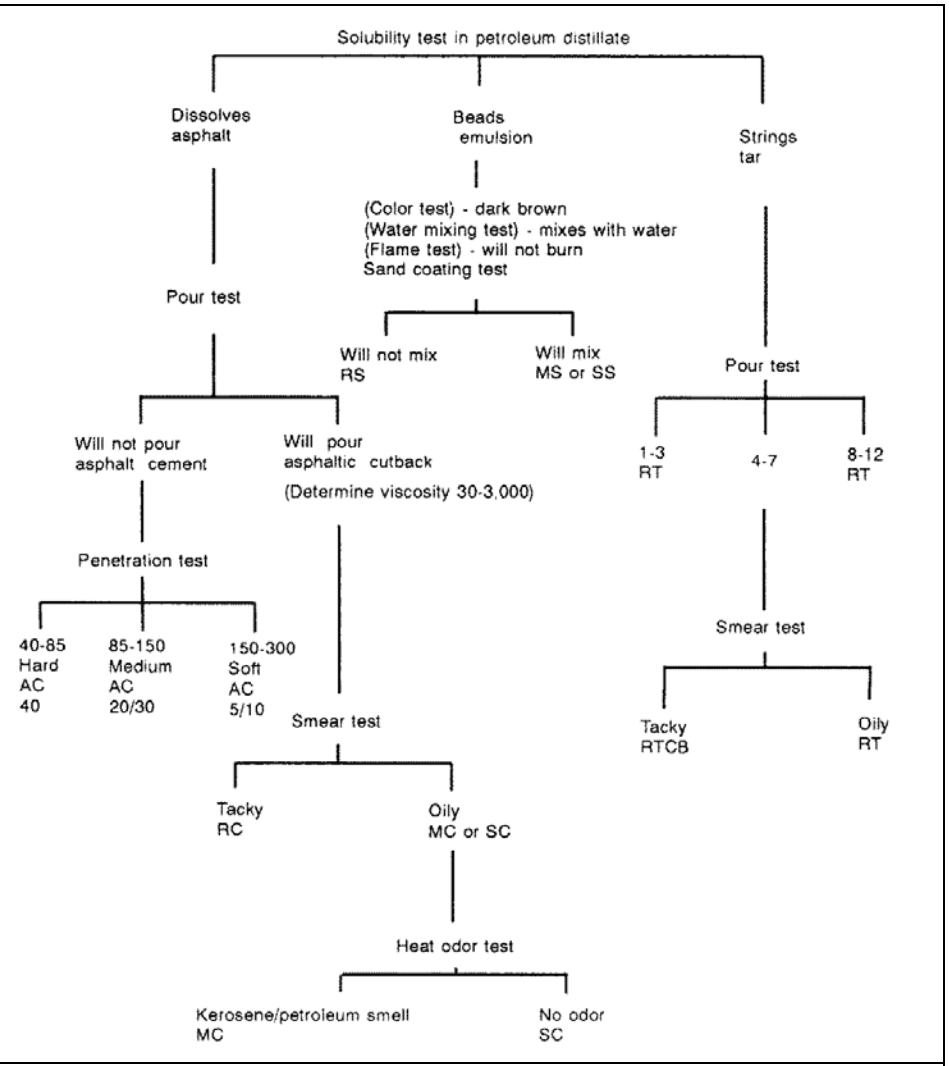
12-125. The first procedure in the identification of an unknown bituminous material is to determine if it is asphalt or tar. Asphalts and tars may be differentiated by a simple solubility test. To perform the test, attempt to dissolve an unknown sample (a few drops if it is liquid or enough to cover the head of a nail if it is 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 asphalt, the distillate mix will consist of a dark, uniform liquid. If the sample is road tar, the sample will be a dark, stringy, undissolved mass in the distillate mix. A check can be made by spotting a piece of paper or cloth with the mix. The solubility test provides a positive method of identification.

Grades of Asphalt Cement

12-126. The various grades of asphalt cement are distinguished by their hardness at designated temperatures. Methods for identifying classification of asphalts include penetration, viscosity, and performance graded. Most of the United States use performance-graded asphalts, but in other areas of the world the grades might be based on viscosity or penetration. A stiffer grade of asphalt resists rutting better, but may result in more thermal and fatigue cracking. The selected grade of asphalt will depend on the climate and traffic level. For hotter climates or pavements subjected to higher traffic levels, a stiffer grade of asphalt is normally selected. When in doubt, use local experience when selecting a grade of asphalt cement to use.

Asphalt Cutbacks

12-127. The pouring or nonpouring quality is one way to distinguish between asphalt cement and a cutback or emulsion. Asphalt cement is cutback with a petroleum distillate to make it more fluid; this is called cutback asphalt. If the material does not pour, it is asphalt cement. At 77 degrees Fahrenheit, 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. Also, cutback asphalts have a distinctive gasoline, diesel, or other solvent smell.



Legend:

AC	asphalt cutback
MC	medium curing
MS	medium setting
RC	rapid curing
RS	rapid setting
RT	road tar
RTCB	road-tar cutback
SS	slow setting

Figure 12-37. Identification of unknown bituminous materials

12-128. To determine if a cutback is a rapid cure or not, a smear test is performed. Smear the substance uniformly on a piece of glazed paper or another nonabsorbent surface. If volatile materials are present, they will evaporate. Since rapid cures 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 lighter grades of medium- and slow-curing asphalt cutbacks, which remain fluid and smooth for some time. A medium cure will not result in a tacky surface for hours, and slow cure will require several days.

12-129. To identify an 800- or 3,000-grade medium-cure or slow-cure cutback, a prolonged smear test is used. This process is necessary because these grades of medium cure and slow cure contain such small quantities of cutterstock that they may also 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 a rapid cure, the smear will have cured and will be hard or slightly sticky. However, if the material being tested is a medium or slow cure, the smear will not be cured and will still be quite sticky or fluid-like. If the material is a rapid cure 3,000, it will cure completely in 3 hours, but a rapid cure 800 will take about 6 hours. Even after 24 hours, a medium or slow cure will still be sticky.

12-130. Since medium cures are cutback with kerosene and slow cures are cutback with oil, this may be used to differentiate between them. If kerosene is present, heat will produce an odor. It is best to heat the unknown sample in a closed container to capture the escaping vapors. Do not apply too much heat. If the sample is a medium cure, it will have a strong petroleum or kerosene odor. But if the sample is a slow cure, no kerosene or petroleum odor will be detected. It may smell like hot motor oil. The ability to distinguish them from each other is perhaps as important as any other part of field identification.

Asphalt Emulsions

12-131. Another asphalt 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 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 if the emulsion is a mixing grade. The best way to determine if the emulsion is a mixing grade (slow setting or medium setting) is to try to mix a small amount (6 to 8 percent by weight) with damp sand, using a metal spoon. A rapid-setting emulsion cannot be mixed; it breaks immediately, gumming up the spoon with the relatively hard original asphalt cement. A slow-setting or medium-setting emulsion mixes nicely, coating the sand. Do not add too much emulsion to the sand. This will saturate the sand and give inconclusive results. No further identification is necessary since medium-setting and slow-setting grades are largely used for the same jobs.

Emulsified Coal Tar

12-132. If an unknown bituminous material is liquid at 77 degrees Fahrenheit, it is an emulsion or a cutback. Add water to the mixture. If the water mixes with the material, it is an emulsion. Add heat to remove the water in the emulsified coal tar, and place the material in a petroleum distillate. If the unknown bituminous material did not dissolve in the solubility test but formed a stringy mass, the material is tar. If it dissolved, it is asphalt.

AGGREGATE IDENTIFICATION

12-133. 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.

12-134. Available aggregate may not always have desirable characteristics. An aggregate meeting most of the requirements is usually selected, unless rejected for other reasons (availability, length of haul, difficulty in conducting borrow-pit or quarry operations).

Angular and Rough

12-135. 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 are best for this purpose since they do not tend to slide past each other. More binder may be required since the angular shaped

aggregate does not pack as closely together as a more rounded aggregate, thus requiring more asphalt to fill the voids to the proper level.

Tough, Hard, and Durable

12-136. 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 equipment is available. Equipment and procedures are detailed in the American Association of State Highway and Transportation Officials method T96. Equipment for the test is not usually available for field testing. The Mohs' hardness scale may be used to determine the hardness of the aggregate. The Mohs' 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

12-137. 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 and dry. If the aggregate is not clean, it should be washed 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

12-138. 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, resulting in stripping. Stripping is the loss of the bituminous coating from the aggregate particles due to the action of water, leaving exposed aggregate surfaces. The stripping test or the swell test can be used to determine the detrimental effect of water on a bituminous mix.

Stripping Test

12-139. 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 (no more than one-half of the jar 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 that is reported as percent stripping.

Swell Test

12-140. Asphalt 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 soaks for a specified period (usually 24 hours) or until there is no further swelling. Another reading of the dial is taken. The difference 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

12-141. The following paragraphs describe proper construction methods for three different types of bituminous spray applications: prime coat, tack coat, and dust proofing.

PRIME COAT

12-142. 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 from penetrating the wearing surface and 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. Typically, one should wait at least 48 hours after priming before material is placed above the primed base.

Base Preparation

12-143. The base should be well-graded, shaped to the desired cross section, compacted to the specified density, well-drained, not too wet but not completely dry, and swept clean. The surface of the base should be broomed if it contains excessive loose material (fine or coarse) or dust. Exercise caution when brooming to ensure that it does not excessively loosen and remove aggregate particles. 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—the moisture may come out, particularly in hot weather, and strip the prime from the base during construction. Rain may strip the prime from a base that was too wet when primed. Heavy rain may strip a properly primed base to some extent, but less than an improperly cured base. However, if a base dries out completely, cracks may develop and a heavy rain may cause swelling and loss of density.

Materials

12-144. Bituminous materials used for prime coats will depend on the condition of the soil base and the climate. In moderate and warm climates, MC-30, MC-70, SS-1, SS-1h, CSS-1 and CSS-1h are satisfactory. In cold climates, rapid-setting asphalt cutbacks, such as RC-70 and RC-250, are sometimes more satisfactory. If the climate is damp and cold, the prime coat is sometimes eliminated because it is likely to be extremely slow in curing. The following formula is used to determine the quantity of prime coat material required:

$$\frac{L \times (W + 2) \times AR \times LF}{9(ft^2/yd^2)} = Q_p$$

where—

AR = *application rate of prime coat (in gallons per square yard)*
L = *length of untreated surface (in feet)*
LF = *handing loss factor for prime coat (usually 1.05 - 1.10)*
Q_p = *quantity of prime-coat material (in gallons)*
W = *width of untreated surface (in feet)*

Notes.

1. The +2 in the formula allows for over-spray of shoulders 1 foot on each side of the road.

2. The application rate for dense soils should be approximately 0.2 gallon per square yard; application rate for soils with a lot of cracks can be as high as 0.5 gallon per square yard.

Example

Compute the quantity (in gallons) of prime coat material required to prime an untreated surface with dense soil. The surface is 1,000 feet long and 12 feet wide. Given—

Loss factor = 1.05.

$$Q_p = \frac{L \times (W + 2) \times AR \times LF}{9(^{ft^2}/_{yd^2})}$$

$$= \frac{1{,}000 \times (12 + 2) \times 0.2 \times 1.05}{9(^{ft^2}/_{yd^2})}$$

$$= 326.67 \; or \; 327 \; gallons$$

where—

AR	=	*application rate of prime coat (in gallons per square yard)*
L	=	*length of untreated surface (in feet)*
LF	=	*handing loss factor for prime coat (usually 1.05 - 1.10)*
Q_p	=	*quantity of prime-coat material (in gallons)*
W	=	*width of untreated surface (in feet)*

TACK COAT

12-145. 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 mixture is placed over the existing surface. The purpose of the tack coat is to provide a bond between the existing surface 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 12-38 shows the sequence of operations for the application of a tack coat.

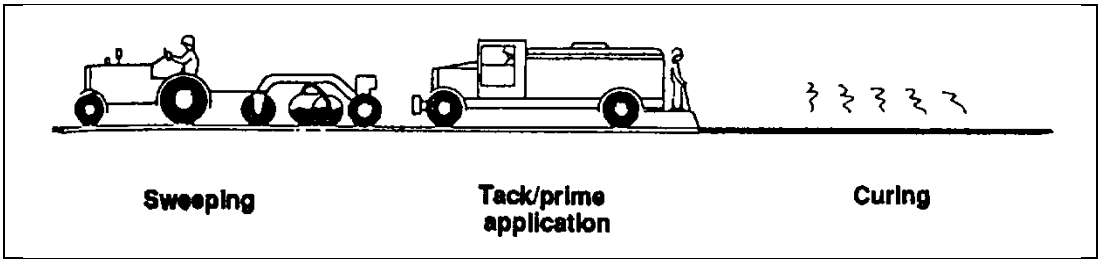


Figure 12-38. Tack-coat sequence of operations

12-146. 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 over the proposed width of the pavement. The following formula is used to determine the quantity of tack coat required:

$$\frac{L \times W \times AR \times LF}{9(ft^2/yd^2)} = Q_t$$

where—

AR	=	*rate of application of bitumen (in gallons per square yard)*
L	=	*length of treated section (in feet)*
LF	=	*handling loss factor for bitumen (usually 1.05)*
Q_t	=	*quantity of tack-coat material (in gallons)*
W	=	*width of treated section (in feet)*

12-147. 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 varies between

0.05 and 0.10 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 0.10 gallon per square yard or slightly higher may be required. An excessive application of tack coat may be absorbed into the surface mixture, resulting in bleeding and flushing of the mixture and loss of stability. Areas at the beginning of a tack coat application and areas sprayed by hand tend to have heavier application rates; ensure that too much tack coat is not applied.

Example

Compute the quantity (in gallons) of tack-coat material required to cover a worn, rough, and cracked surface. The surface is 1,000 feet long and 12 feet wide. Given—Loss factor = 1.05

$$Q_t = \frac{L \times W \times AR \times LF}{9\left(ft^2/yd^2\right)}$$
$$= \frac{100 \times 12 \times 0.10 \times 1.05}{9\left(ft^2/yd^2\right)}$$
$$= 14\ gallons$$

where—

AR	=	rate of application of bitumen in gallons per square yard
L	=	length of treated section in feet
LF	=	handling loss factor for bitumen (usually 1.05)
Q_t	=	quantity of tack-coat material in gallons
W	=	width of treated section in feet

DUSTPROOFING

12-148. Dustproofing consists of spraying an untreated surface with a diluted, slow-setting asphalt emulsion or a low-viscosity cutback. The asphalt and diluent penetrate the fine soil particles and adhere to the dust particles.

12-149. An asphalt cutback is usually sprayed at a rate of 0.1 to 0.5 gallons per square yard. When using an emulsion, dilute it with up to five parts of water by volume. Diluted-emulsion dustproofing treatments usually require several treatments. The dust stirred by traffic between applications eventually conglomerates and is no longer a problem. This is an effective treatment in very dusty areas where one application of cutback asphalt is insufficient. Lay a test strip to determine what application rate will be the most effective. Apply with an asphalt distributor or something as simple as a common watering can. Dustproofing is usually a temporary solution and project plans should include regular inspections and maintenance, as required.

SURFACE TREATMENTS

12-150. The following paragraphs provide guidance on the proper design and construction of surface treatments, which are generally described as a sprayed binder with cover aggregates and commonly referred to as chip seals.

SPRAYING ASPHALT WITH COVER AGGREGATE AND SINGLE- AND MULTIPLE-SURFACE TREATMENTS

12-151. 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 are referred to as multiple-surface treatments (double, triple, quadruple), depending on the number of applications.

12-152. Apply these surface treatments on a primed, unbound base; an asphalt base course; or any type of existing pavement. This type of surface treatment, with a good prime coat, 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.

12-153. This 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. There are limitations when this application is used. Consider using plant-mix when—

- Weather conditions are unfavorable.
- The surface that the asphalt is going to be sprayed onto is not hard, clean, and dry. (The surface must be hard, clean, and dry for the treatment to adhere properly.)
- The amount and viscosity of the asphalt cannot be balanced with the condition of the underlying material and with the size and amount of cover aggregate to assure proper aggregate retention.
- Heavy, high-speed traffic is likely. (Heavy, high speed traffic may dislodge aggregate from the asphalt. This can be a significant problem, causing broken windshields from flying debris.)

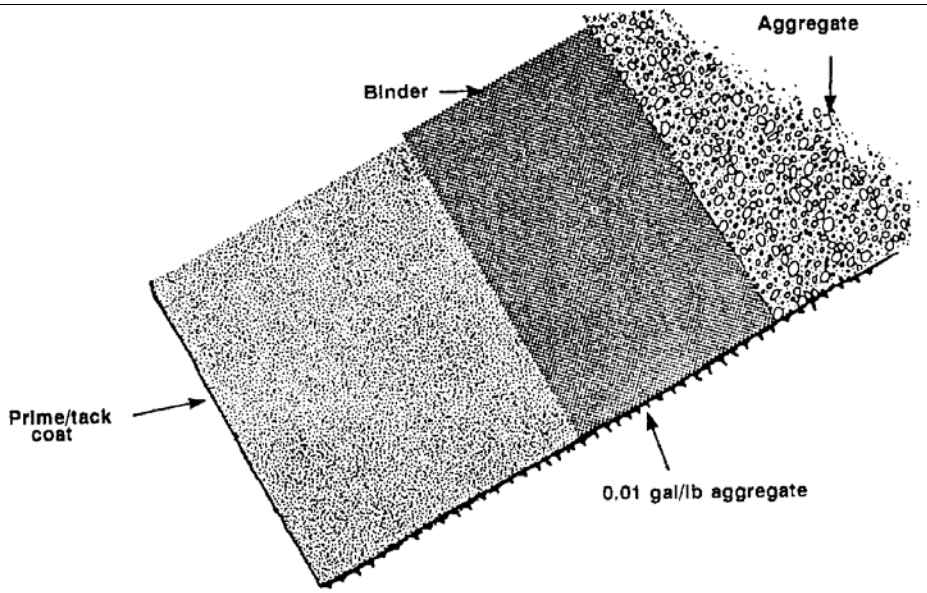
Single-Surface Treatment

12-154. A single-surface treatment usually consists of a sprayed application of bitumen and an aggregate cover one stone deep. Surface treatment may be referred to as a seal coat, armor coat, or carpet coat. A single-surface treatment is usually less than 1-inch thick. (See figure 12-39.) Surface treatments serve as an abrasive and weather-resistant medium that waterproofs the base. They are not as durable as hot mix asphalt and may require frequent maintenance. Although not recommended for airfields (foreign object debris), they may be used as an expedient measure. They are particularly suitable for JOA 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.

12-155. Surface treatment will not withstand the action of metal wheels on vehicles, tracked vehicles, or nonskid chains on vehicle wheels. Do not attempt construction of surface treatments when the temperature is below 50 degrees Fahrenheit. The requirements for a surface treatment are as follows:

- The quality of the bitumen must be sufficient enough to hold the stone in place without submerging it in asphalt binder.
- Sufficient aggregate must be used to cover the bitumen.
- The base course that the surface treatment is laid onto must be strong enough to support the anticipated traffic load.

12-156. 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 sealing a smooth, existing surface. For a badly broken surface, the maximum size of the aggregate should be about 3/4 inch. Surface treatments include the following: dust palliatives, prime coats, and sprayed asphalt with a cover aggregate (single- and multiple-surface treatments). Rounded aggregates do not provide satisfactory performance with surface treatments. Hence, care must be exercised when considering using natural sands or poorly crushed gravels.



Legend:

gal	gallon
lb	pounds

Figure 12-39. Single-surface treatment

Material

12-157. Rapid-cure and medium-cure cutbacks, emulsified tars, rapid-setting emulsions, and asphalt cements may be used for surface treatment. Rapid-setting emulsions are the most material. In many places, cutback asphalt is not used due to environmental issues. Viscosity grades of the cutbacks or asphalt cements 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.

12-158. To assure uniform distribution, the bitumen should be applied with a bituminous distributor. The quantity of the bitumen required is based on the average particle size of the cover stone. The bitumen must be sufficient enough to hold the aggregate in place without leaving a sticky surface. The aggregate must not be completely submerged in the bitumen. Submerge 1/4-inch aggregate approximately 30 percent, 3/8-inch aggregate approximately 32 percent, 1/2-inch aggregate approximately 35 percent, and 3/4-inch aggregate approximately 43 percent. Approximately 1 gallon of bitumen is used for 100 pounds of aggregate. The recommended rate of bitumen application is given by the following formula:

$$\frac{weight\ of\ aggregate\ (pounds)}{area(yard^2)} \times \frac{1\ gallon\ bitumen}{100\ pounds\ aggregate} = quantity\ of\ bitume\ in\ gallon/yard^2$$

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 by using the following equation:

$$\frac{300\ pounds\ of\ aggregate}{1\ (yard^2)} \times \frac{1\ gallon\ bitumen}{100\ pounds\ aggregate} = 0.30\ gallon/yard^2$$

Requirements for a Surface Treatment

12-159. In bituminous surface treatments, the unit quantities of bitumen and aggregate can be determined by a test strip, the specifications for the project, or adding approximately 1 gallon of bitumen for every 100 pounds of aggregate. The weight of the aggregate, one stone deep, required to cover 1 square yard is determined by spreading the aggregate a depth of one stone over a measured surface, weighing it, and computing the amount in pounds per square yard. The equation used to determine the quantity of binder material required for a surface treatment is—

$$\frac{L \times W \times ARB \times ARA \times LF}{9\left(ft^2/yd^2\right)} = Q_b$$

where—

9 = *square feet per square yard (ft^2/yd^2) conversion factor*
ARA = *application of aggregate in pounds per square yard*
ARB = *application of bitumen in gallons per pound of aggregate*
L = *length of treated surface in feet*
LF = *handling loss factor for bitumen*
Q_b = *quantity of binder material required in gallons*
W = *width of treated surface in feet*

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 1 gallon per 100 pounds of aggregate and a loss factor of 1.05. The aggregate is 3/8-inch crushed stone with a unit weight of 100 pounds per foot[3].

Determine the application rate of the aggregate in pounds per square yard—

$$ARA = 100\ pounds/foot^3 \times (3/8\ inch\ \times (1\ foot/12\ inches\ \times (9\ foot^2/1\ yard^2+)$$
$$= 28.125\ pounds/yard^2$$

Determine the binder quantity—

$$Q_b = \frac{L \times W \times ARB \times ARA \times LF}{9\ ft^2/yd^2}$$
$$= \frac{1{,}000\ ft\ \times 12\ ft \times 0.01\ gal/lb \times 28.125\ lb/yd^2 \times 1.05}{9\ ft^2/yd^2}$$
$$= 393.75\ or\ 394\ gallons$$

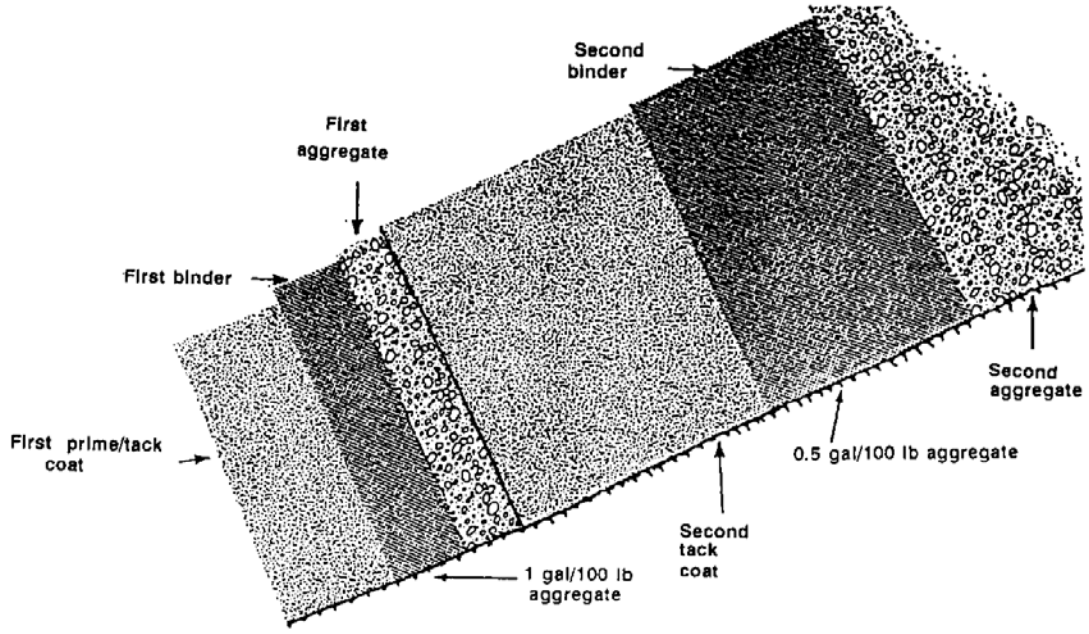
Example (continued)

where—

9	=	*square feet per square yard (ft^2/yd^2) conversion factor*
ARA	=	*application of aggregate in pounds per square yard*
ARB	=	*application of bitumen in gallons per pound of aggregate*
L	=	*length of treated surface in feet*
LF	=	*handling loss factor for bitumen*
Q_b	=	*quantity of binder material required in gallons*
W	=	*width of treated surface in feet*

Multiple-Surface Treatment

12-160. When a tougher, more resistant surface is desired more 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. (See figure 12-40.) 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.



Legend:

gal	gallon
lb	pounds

Figure 12-40. Multiple-surface treatment

12-161. The first layer of a multiple-surface treatment is laid according to the instructions previously given for a single-surface treatment. Loose aggregate remaining on the first layer must be swept from the surface so that the layers may bond together. As previously stated, 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 to minimize the potential for foreign object debris damage to aircraft. Final sweeping is also recommended for roads.

CONSTRUCTION OF SURFACE TREATMENTS USING SPRAYED ASPHALT WITH COVER AGGREGATE

12-162. Surface treatments are widely used because of their low cost and usefulness in light to medium traffic roadways and parking areas. These treatments are normally used to seal or waterproof the pavement, provide wear resistance, and increase skid resistance. All surface treatments are relatively thin coatings of material and do not add structurally to the pavement.

Weather

12-163. Weather conditions are an important factor for success in the construction of sprayed asphalt with cover-aggregate surface treatments and seal coats. For best results in aggregate retention, the pavement temperature should be relatively high during the application of the seal coat and some amount of curing should be allowed before fast moving and turning traffic is allowed to use the new seal coat. A certain amount of curing, or setting, is required even with the heaviest liquid-asphalt materials. This curing takes place best when the air temperature is well above 50 degrees Fahrenheit and the relative humidity is low. 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

12-164. Once an aggregate has been selected for use based on 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 equation:

$$\frac{L \times W \times ARA \times LF}{9 \times 2{,}000} = QA$$

where—

ARA = *application rate for aggregate*
L = *length of treated surface in feet*
LF = *handling loss factor for aggregate (10 percent or 1.10)*
QA = *quantity of aggregate in tons*
W = *width of treated surface in feet*

12-165. 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

12-166. Before the application of asphalt begins, an adequate aggregate spreader should be available and properly adjusted for the aggregate 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.

12-167. Aggregate spreaders vary from a simple, controllable gate box attached to the dump truck, to very efficient, self-propelled units that apply the larger-size aggregate on the bottom and the finer-size on the top. The more efficient, self-propelled units are most desirable.

Standard, Hopper-Type Aggregate Spreaders

12-168. The standard aggregate spreader can handle aggregate, which ranges from sand to 1 1/2-inch gravel. (See figure 12-41.) The rate and depth of application depend on the gate opening. The width of the spread may be varied from 4 to 8 feet in 1-foot increments. Depending on the manufacturer, the spreader has 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, no one should be allowed to stand on the aggregate, in the truck, or on the spreader at any time.

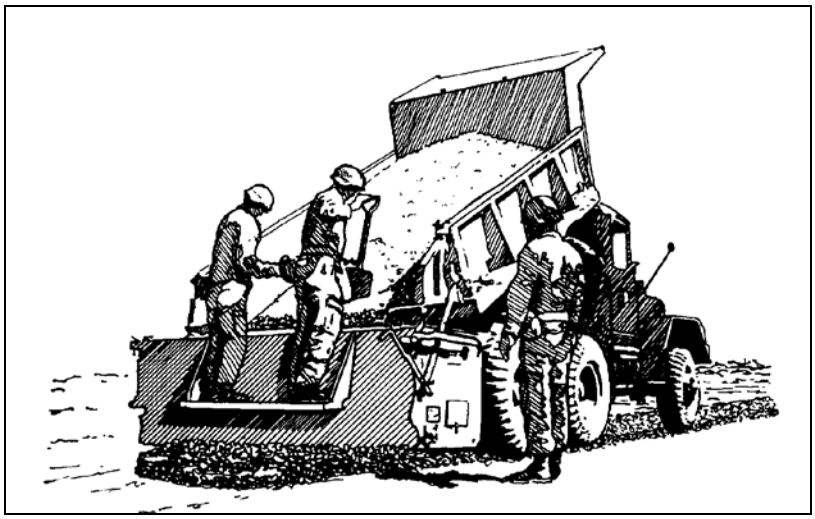


Figure 12-41. Typical hopper-type aggregate spreader

Rolling

12-169. Pneumatic-tire rollers should be used for surface-treatment construction. Steel-wheeled rollers are not recommended for rolling, unless that is the only equipment available. The rollers should not be heavy enough to crush the aggregate particles. Vibratory rollers should not be used.

12-170. Pneumatic-tire rollers are essential to firmly embed the aggregate into low areas or graded deformities that would normally be bridged over by using steel-wheel rollers. Pneumatic-tire rollers 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 complete.

Traffic Control

12-171. It is 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.

Asphalt Distributor

12-172. 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 that is usually oil-burning, with direct heat from the flue passing through the tank. It is supplied with a power-driven pump that is 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 provide 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. (See figure 12-42.) Normally a double or triple lap application is desired. A suitable 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 that range from 800 to 4,000 gallons. Some maintenance distributors as small as 400 gallons are available.

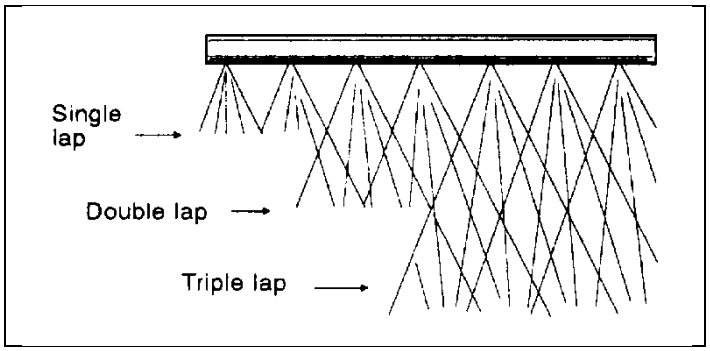


Figure 12-42. Spray bar coverage

12-173. It is essential that the distributor be capable of distributing the asphalt uniformly over the surface being treated. For best results in surface treatments—

- Maintain uniform pressure and temperature on the 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 manufacturer 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 manufacturer instructions) to provide complete and uniform overlap of the spray fans.
- Ensure that the distributor road speed is uniform.
- Check the spread of the distributor spray bar before beginning. The valve action should be instantaneous, 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

12-174. 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; it may result in interference with traffic or, in extreme cases, require construction of detours to avoid a complete stoppage.

12-175. Patches should be trimmed square or oblong with straight, vertical sides running parallel and perpendicular to the centerline of the traffic area. Stable material can 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. These patches must be thoroughly compacted and constantly maintained with replacement material. More permanent patching should be accomplished as soon as possible.

12-176. 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 the 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.

This page intentionally left blank.

Appendix A

METRIC CONVERSION CHART

This appendix complies with AR 25-30 which states that weights, distances, quantities, and measures contained in Army publications will be expressed in both United States standard and metric units. Table A-1 is a metric conversion chart.

Table A-1. Metric conversion chart

United States Units	*Multiplied By*	*Equals Metric Units*
Acres	4,047	Square meters
Cubic feet	00.02830	Cubic meters
Cubic feet per second	00.28316	Cubic meters per second
Cubic yards	00.76460	Cubic meters
Feet	00.30480	Meters
Feet per minute	00.50800	Centimeters per square inch
Feet per second	00.30480	Meters per second
Gallons	03.78540	Liters
Inches	02.54000	Centimeters
Inches	00.02540	Meters
Inches	25.40010	Millimeters
Knots	01.85300	Kilometers per hour
Miles (statute)	01.60930	Kilometers
Miles (nautical)	01.85320	Kilometers
Miles per hour	01.60930	Kilometers per hour
Pounds	00.45359	Kilograms
Pounds per cubic yard	16.01846	Kilograms per cubic meter
Pounds per square inch	06.89476	Kilopascals
Square feet	00.09290	Square meters
Square inches	06.45160	Square centimeters
Square yards	00.83610	Square meters
Tons (short)	00.90700	Metric tons
Yards	00.91440	Meters

Table A-1. Metric conversion chart (continued)

Metric Units	*Multiplied By*	*Equals United States Units*
Centimeters	00.39370	Inches
Centimeters per square minute	0.032808399	Feet per minute
Cubic meters	35.31440	Cubic feet
Cubic meters	01.30790	Cubic yards
Cubic meters per second	35.3146667	Cubic feet per second
Kilograms	02.20460	Pounds
Kilograms per cubic meter	00.06243	Pounds per cubic yard
Kilometers	00.53960	Miles (nautical)
Kilometers	00.62137	Miles (statute)
Kilometers per hour	00.53960	Knots per hour
Kilonewton	00.22481	Kilopound
Kilopound	4,448.22160	Newton
Kilopascals	00.14493	Pounds per square inch
Liters	00.26420	Gallons
Meters	03.28080	Feet
Meters	39.37000	Inches
Meters	01.09361	Yards
Meters per second	3.28084	Feet per second
Metric tons	01.10200	Tons (short)
Millimeters	00.03937	Inches
Square centimeters	00.15500	Square inches
Square meters	02.47100×10^{-4}	Acres
Square meters	10.76400	Square feet
Square meters	01.19600	Square yards

Appendix B

Hydrologic and Hydraulic Tables and Curves

HYDROLOGIC SUPPLY CURVES FOR OVERLAND FLOW

B-1. Figures B-1 through B-7, pages B-2 through B-8, represent the peak runoff rates from individual storm events of various durations, all of which have the same average frequency of occurrence.

B-2. These curves are computed from Horton's equation using a retardance coefficient n = 0.04 and hydraulic gradient S = 1 percent.

HYDRAULIC CHARACTERISTICS OF SELECTED DRAINAGE CHANNELS

B-3. Tables B-1 through B-9, pages B-9 through B-17, present commonly used ditch sections, with cross-section areas and hydraulic radius, to facilitate selection of ditch size and shape.

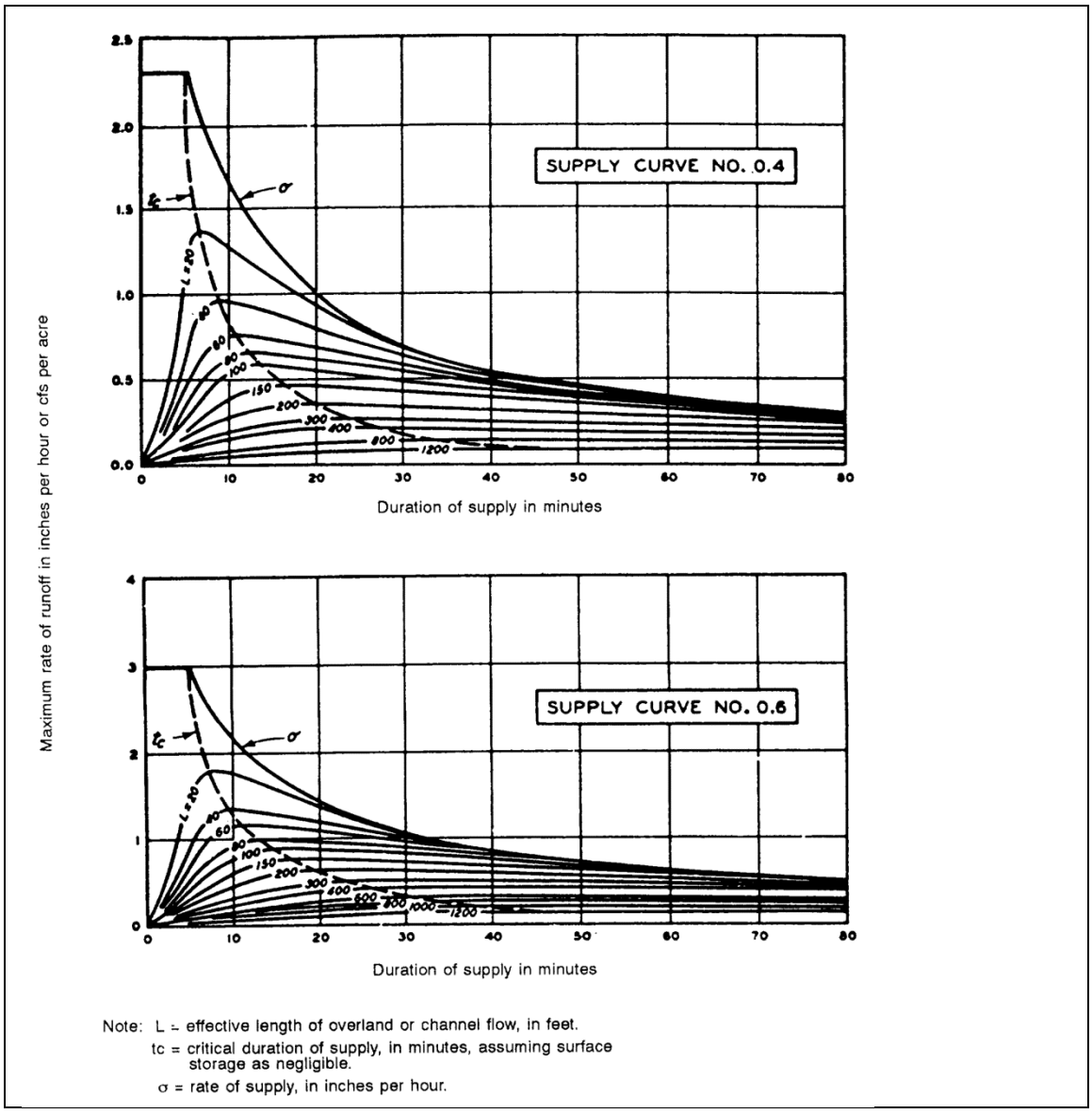
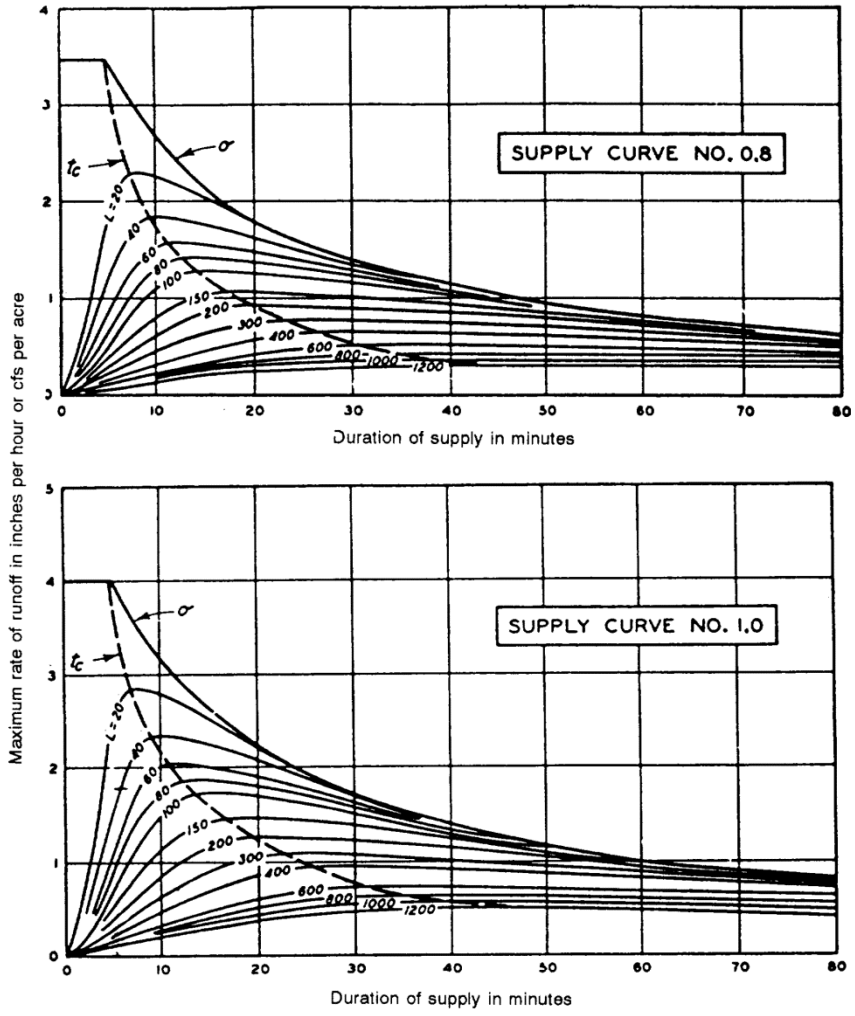
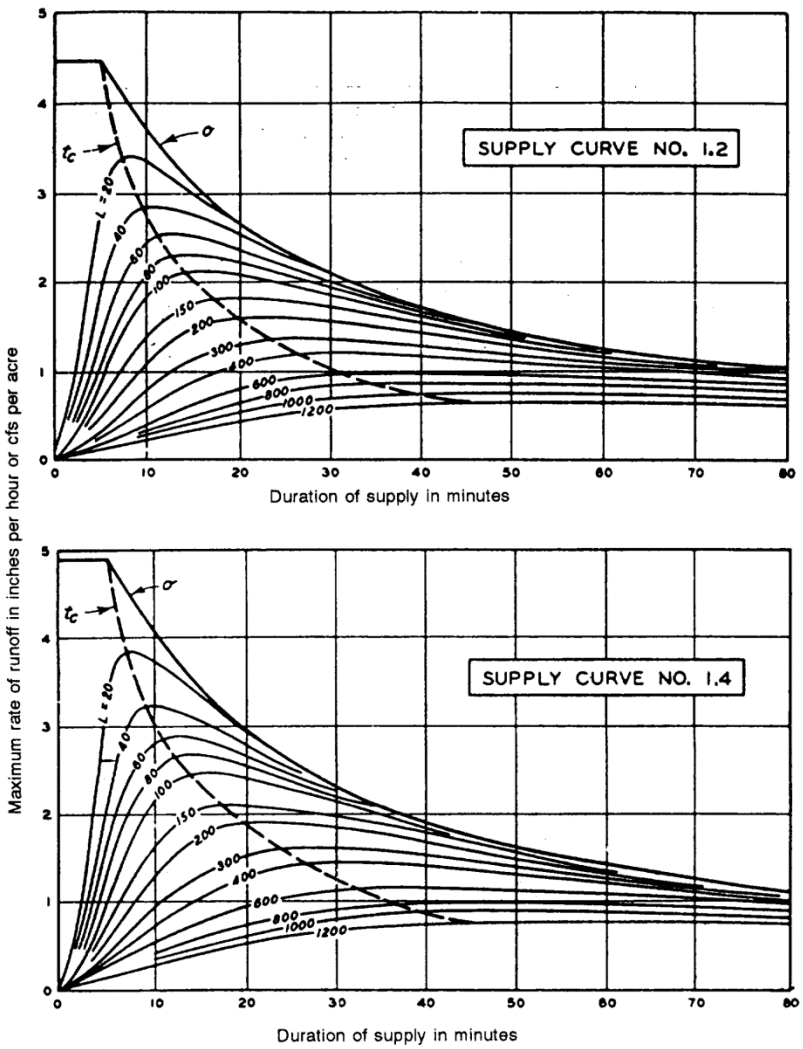


Figure B-1. Rates of overland flow corresponding to standard supply curves, supply curve number 0.4 and 0.6; n=0.40, S=1 percent



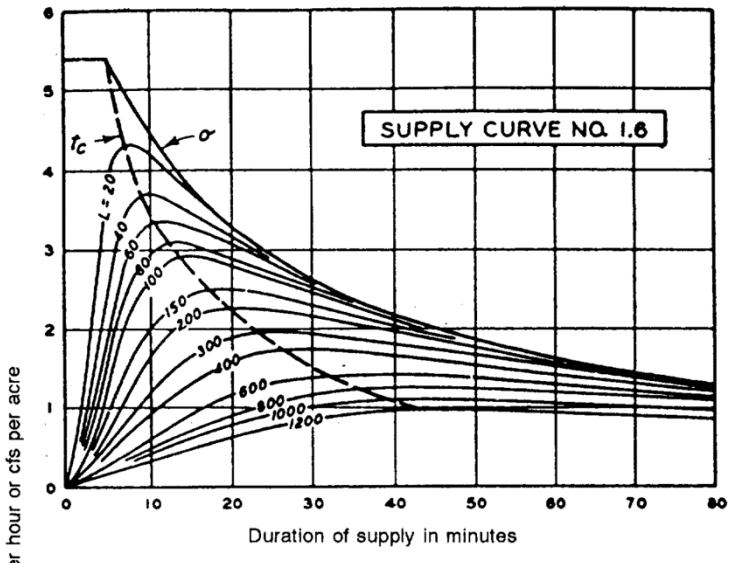
Note: L = effective length of overland or channel flow, in feet.
tc = critical duration of supply, in minutes, assuming surface storage as negligible.
σ = rate of supply, in inches per hour.

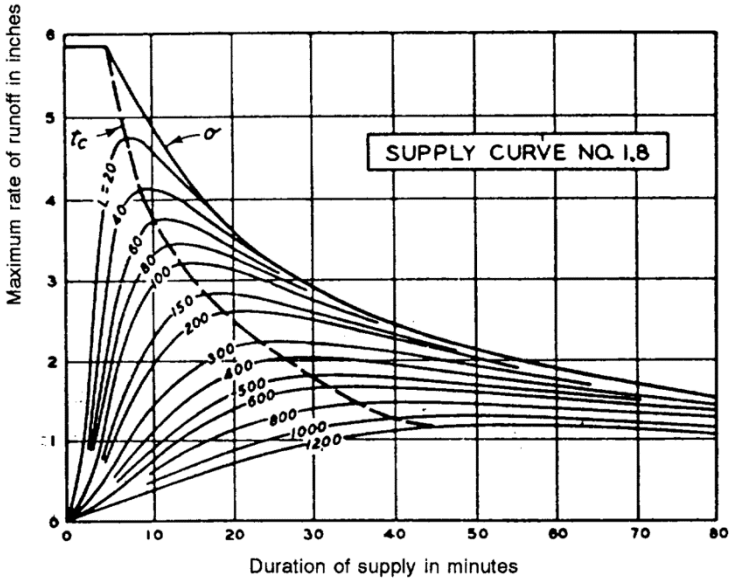
Figure B-2. Rates of overland flow corresponding to standard supply curves, supply curve numbers 0.8 and 1.0; n = 0.40, S = 1 percent



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 B-3. Rates of overland corresponding to standard supply curves, supply curve numbers 1.2 and 1.4; n = 0.40, S = 1 percent





Maximum rate of runoff in inches per hour or cfs per acre

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 B-4. Rates of overland flow corresponding to standard supply curves, supply curve numbers 1.6 and 1.8; n = 0.40, S = 1 percent

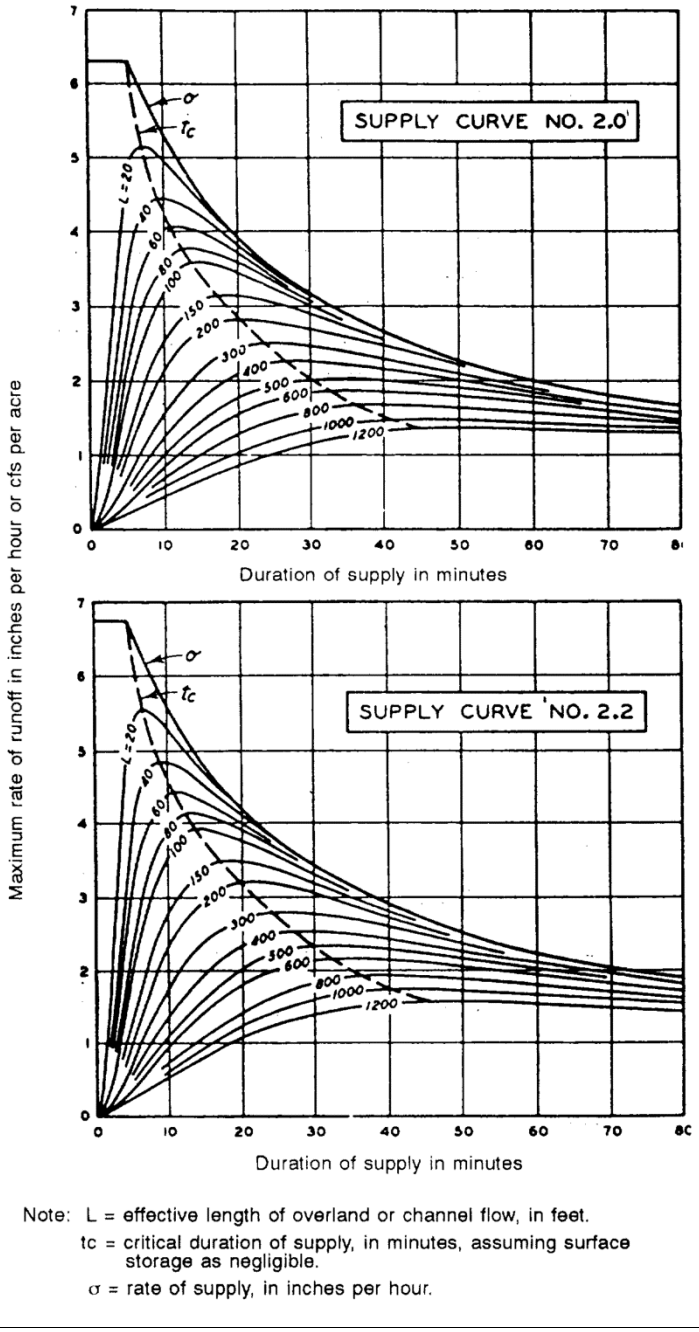
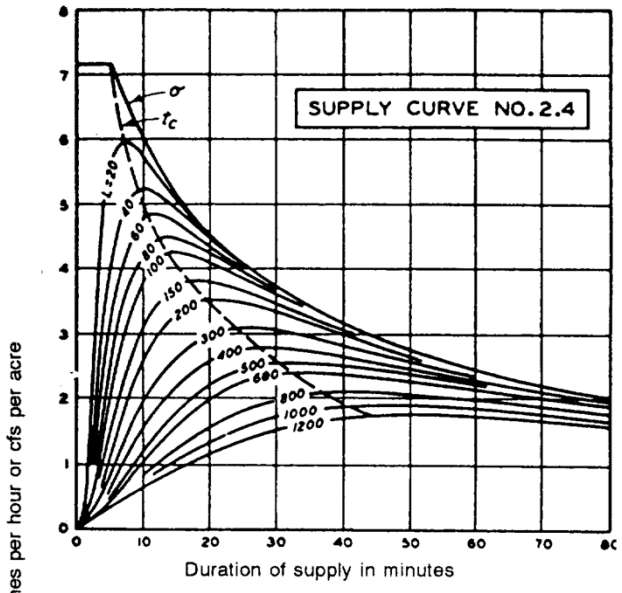
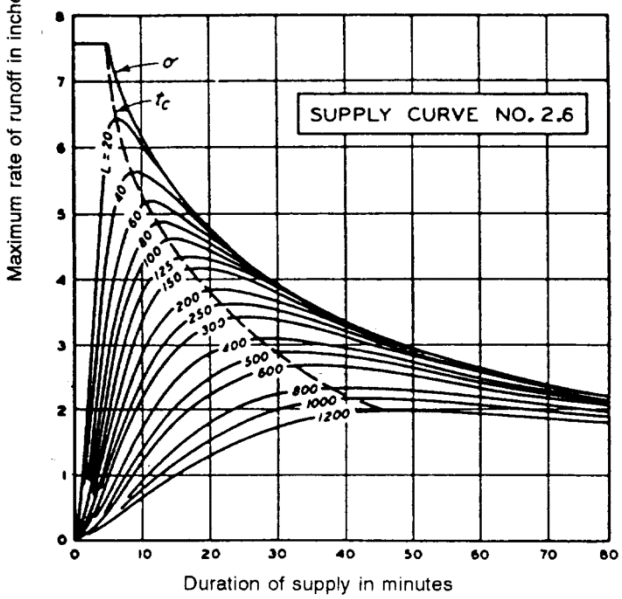


Figure B-5. Rates of overland flow corresponding to standard supply curves, supply curve numbers 2.0 and 2.2; n = 2.0 and 2.2; n = 0.40; S = 1 percent





Maximum rate of runoff in inches per hour or cfs per acre

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 B-6. Rates of overland flow corresponding to standard supply curves, supply curve number 2.4 and 2.6; n = 0.40, S = 1 percent

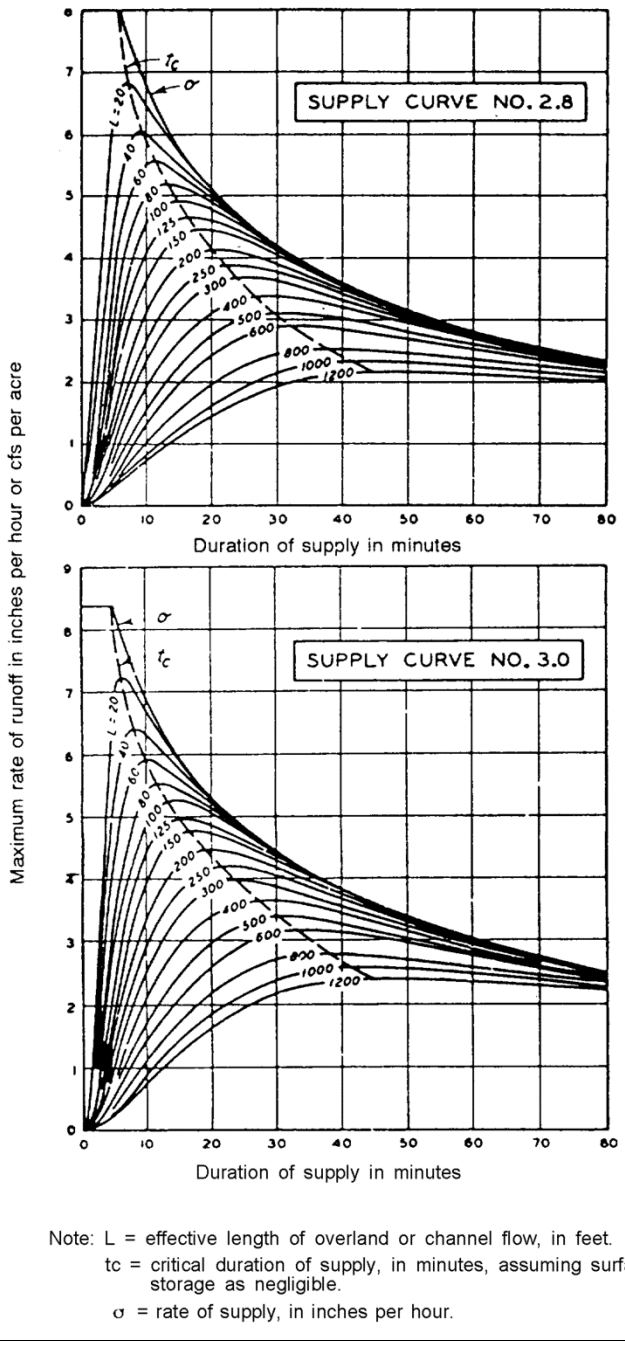
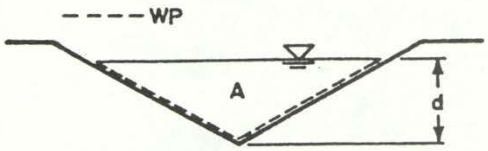


Figure B-7. Rates of overland flow corresponding to standard supply curves, supply curve numbers 2.8 and 3.0; n = 0.40, S = 1 percent

Table B-1. Hydraulic radius and area of symmetrical triangular channels

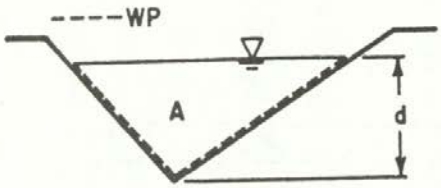


R= A/WP

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	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		6:1		7:1		8:1		9:1		10:1	
	A	R	A	R	A	R	A	R	A	R	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

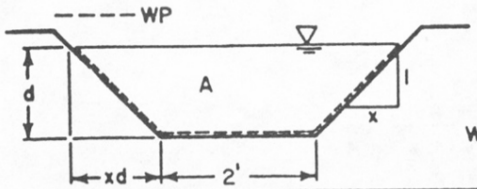
Table B-2. Hydraulic radius and area of nonsymmetrical triangular channels



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 B-3. Hydraulic radius and area of symmetrical trapezoidal channels (2-foot bottom width)



$A = xd^2 + 2d$

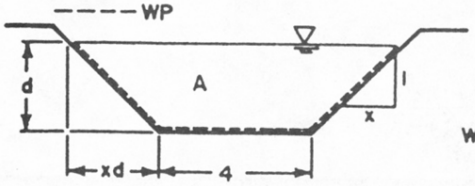
$WP = 2d\sqrt{1 + x^2} + 2$

$R = A/WP$

Depth, d (feet)	Slope ratio 1:1 A	1:1 R	1½:1 A	1½:1 R	2:1 A	2:1 R	2½:1 A	2½:1 R	3:1 A	3:1 R	4:1 A	4:1 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 A	5:1 R	6:1 A	6:1 R	7:1 A	7:1 R	8:1 A	8:1 R	9:1 A	9:1 R	10:1 A	10:1 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 B-4. Hydraulic radius and area of symmetrical trapezoidal channels (4-foot bottom width)



$A = xd^2 + 4d$

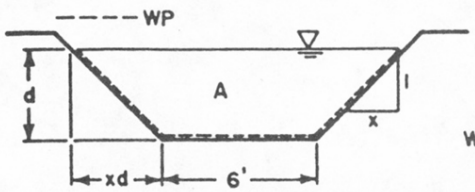
$WP = 2d\sqrt{1 + x^2} + 4$

$R = A/WP$

Depth, d (feet)	Slope ratio 1:1 A	1:1 R	1½:1 A	1½:1 R	2:1 A	2:1 R	2½:1 A	2½:1 R	3:1 A	3:1 R	4:1 A	4:1 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 A	5:1 R	6:1 A	6:1 R	7:1 A	7:1 R	8:1 A	8:1 R	9:1 A	9:1 R	10:1 A	10:1 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 B-5. Hydraulic radius and area of symmetrical trapezoidal channels (6-foot bottom width)



$A = xd^2 + 6d$

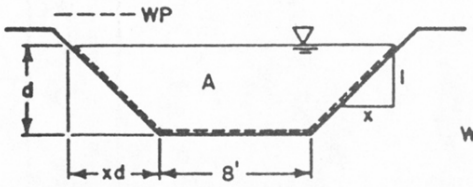
$WP = 2d\sqrt{1 + x^2} + 6$

$R = A/WP$

Depth, d (feet)	Slope ratio 1:1 A	1:1 R	1½:1 A	1½:1 R	2:1 A	2:1 R	2½:1 A	2½:1 R	3:1 A	3:1 R	4:1 A	4:1 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	5:1 R	6:1 A	6:1 R	7:1 A	7:1 R	8:1 A	8:1 R	9:1 A	9:1 R	10:1 A	10:1 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	1.33
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 B-6. Hydraulic radius and area of symmetrical trapezoidal channels (8-foot bottom width)



$A = xd^2 + 8d$

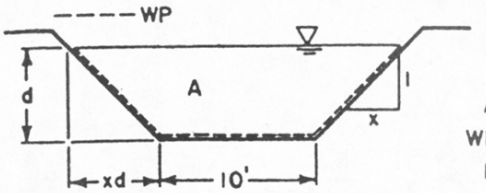
$WP = 2d\sqrt{1 + x^2} + 8$

$R = A/WP$

Depth, d (feet)	Slope ratio 1:1 A	1:1 R	1½:1 A	1½:1 R	2:1 A	2:1 R	2½:1 A	2½:1 R	3:1 A	3:1 R	4:1 A	4:1 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 A	5:1 R	6:1 A	6:1 R	7:1 A	7:1 R	8:1 A	8:1 R	9:1 A	9:1 R	10:1 A	10:1 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	96.00	1.70	105.00	1.70	114.00	1.69

Table B-7. Hydraulic radius and area of symmetrical trapezoidal channels (10-foot bottom width)



$A = xd^2 + 10d$

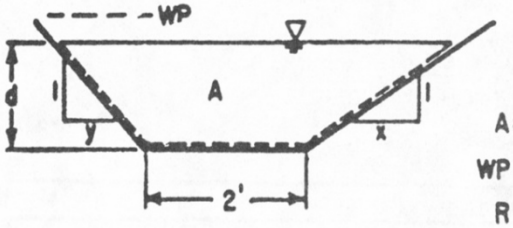
$WP = 2d\sqrt{1 + x^2} + 10$

$R = A/WP$

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 B-8. Hydraulic radius and area of nonsymmetrical trapezoidal channels (2-foot bottom width)



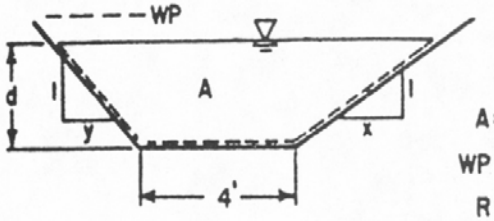
$$A = \frac{1}{2} d^2(x+y) + 2d$$

$$WP = d(\sqrt{1+y^2} + \sqrt{1+x^2}) + 2$$

$$R = A/WP$$

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 B-9. Hydraulic radius and area 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

This page intentionally left blank.

Glossary

SECTION I – ACRONYMS AND ABBREVIATIONS

AASHTO	Amercian Association of State Highway and Transportation Officials
ADRP	Army doctrine reference publication
AFH	Air force handbook
AFI	Air Force instruction
AFJMAN	Air Force Joint manual
AFM	Air Force manual
AFMAN	Air Force manual
AFPD	Air Force policy directive
AR	Army regulation
ASTM	American Society of Testing and Materials
attn	attention
CA	California
CBR	California bearing ratio
CDID	Capabilities Development and Integration Directorate
CFR	Code of Federal Regulations
CI	cone index
CMP	corrugated metal pipe
CODDD	Concepts, Organizations, and Doctrine Development Division
CRD	capstone requirements document
DA	Department of the Army
DC	District of Columbia
DD	Department of Defense
DM	design manual
DSN	defense switched network
EIAP	Environmental Impact Analysis Process
EM	Engineer manual
ERDCC/GSL	Engineer Research and Development Center, Geotechnical and Structure Laboratory
FL	Florida
FM	field manual
GTM	Gyratory Testing Machine
HDPE	high-density polyethylene
http	hypertext transfer protocol
https	hypertext transfer protocol secure
JOA	joint area of operations
JP	joint publication
LA	Louisana

LCF	lime-cement-fly ash
LL	liquid limit
MCRP	Marine Corps reference publication
MI	mobility index
MO	Missouri
MSCoE	Maneuver Support Center of Excellence
NAVFAC	Naval Facilities Engineering Command
No.	number
NRMM	North Atlantic Treaty Organization Reference Mobility Model
NTRP	Navy tactical reference publication
OR	Oregon
pH	hydrogen
PI	plasticity index
RCI	rating cone index
RI	remolding index
TC	training circular
TM	technical manual
TOC	time of concentration
TR	technical release
UFC	unified facilities criteria
USACE	United States Army Corps of Engineers
USCS	Unified Soil Classification System
USMC	United States Marine Corp
VA	Virginia
VCI	vehicle cone index
www	world wide web

References

REQUIRED PUBLICATIONS

These documents must be available to intended users of this publication.

ADRP 1-02. *Terms and Military Symbols.* 7 December 2015.

JP 1-02. *Department of Defense Dictionary of Military and Associated Terms.* 8 November 2010.

RELATED PUBLICATIONS

These documents contain relevant supplemental information.

AIR FORCE PUBLICATIONS

Most Air Force publications are available online: <http://www.e-publishing.af.mil/index.asp>.

AFI 32-7061. *The Environmental Impact Analysis Process.* 12 March 2003.

AFPD 32-70. *Environmental Quality*. 20 July 1994.

ARMY PUBLICATIONS

Most Army doctrinal publications are available online: www.apd.army.mil.

AR 200-1. *Environmental Protection and Enhancement.* 13 December 2007.

FM 27-10. *The Law of Land Warfare*. 18 July 1956.

TC 5-340. *Air Base Damage Repair (Pavement Repair).* 27 December 1988.

TM 3-34.48-2. *Theater of Operations: Roads, Airfields, and Heliports—Airfield and Heliport Design.* 29 February 2016.

TM 3-34.55. *Construction Surveying.* 3 August 2012.

TM 5-302-1. *Army Facilities Components System: Design.* 28 September 1973.

TM 5-302-2. *Army Facilities Components System: Design.* 28 September 1973.

TM 5-302-3. *Army Facilities Components System: Design.* 28 September 1973.

TM 5-302-4. *Army Facilities Components System: Design.* 28 September 1973.

TM 5-302-5. *Army Facilities Components System: Design.* 28 September 1973.

TM 5-349. *Arctic Construction*. 19 February 1962.

MULTI-SERVICE PUBLICATIONS

ATP 3-34.5/MCRP 4-11B. *Environmental Considerations.* 10 August 2015.

FM 3-34.170/MCWP 3-17.4. *Engineer Reconnaissance.* 25 March 2008.

FM 3-34.214/MCRP 3-17.7L. *Explosives and Demolitions.* 11 July 2007.

TM 3-34.43/MCRP 3-17-7H/NAVFAC MO 330/AFH 32-1034. *Materials Testing.* 3 April 2015.

TM 3-34.62/MCRP 3-17.7I. *Earthmoving Operations.* 29 June 2012.

TM 3-34.64/MCRP 3-17.7G. *Military Soils Engineering.* 25 September 2012.

TM 3-34.65/NTRP 4-04.2.12/AFMAN 10-903. *Quarry Operations.* 30 October 2013.

TM 3-34.85/MCRP 3-17A. *Engineer Field Data.* 17 October 2013.

TM 5-624/NAVFAC DM MO-102/AFJMAN 32-1040. *Maintenance and Repair of Surface Areas.* 27 October 1995.

TM 5-822-14/AFJMAN 32-1019. *Soil Stabilization for Pavements.* 25 October 1994.

TM 5-852-7/AFM 88-19. *Surface Drainage Design for Airfields and Heliports in Arctic and Subarctic Regions.* 15 April 1981.

OTHER PUBLICATIONS

Most UFC publications are available online at <http://www.wbdg.org/ccb/browse_cat.php?o=29&c=4>.

32 CFR 989. *Environmental Impact Analysis Process (EIAP).* 14 October 2015.

AASHTO A-A-60005. *Frames, Covers, Gratings, Steps, Sump and Catch Basin, Manhole*. 2 March 1998. Web site <https://www.wbdg.org/ccb/FEDMIL/aa60005.pdf>, accessed 15 October 2015.

AR 25-30. *Army Publishing Program.* 3 June 2015.

CRD-C 649-95. *Standard Test Method for Unit Weight, Marshall Stability, and Flow of Bituminous Mixtures.* 1 December 1995. <http://www.wbdg.org/ccb/ARMYCOE/COESTDS/crd_c649.pdf>, accessed on 15 October 2015.

CRD-C 651-95. *Standard Gyratory Testing Machine Method for Design of Hot-Mix Bituminous Pavement Mixtures.* 1 December 1995. <http://www.wbdg.org/ccb/ARMYCOE/COESTDS/crd_c651.pdf>, accessed on 15 October 2015.

EM 1110-1-1005. *Engineering and Design: Control and Topographic Surveying*. 1 January 2007. <http://www.publications.usace.army.mil/USACEPublications/EngineerManuals.aspx?udt_43544_param_page=2>, accessed on 15 October 2015.

ERDC/GSL TR-08-3. *A Rapid Soils Analysis Kit.* March 2008. <http://dsearch.dtic.mil/search?site=default_collection&q=ERDC%2FGSL+TR-08-3&client=dticol_frontend&proxystylesheet=dticol_frontend&proxyreload=1&filter=0&tlen=200&getfields=*&btnG.x=0&btnG.y=0>, accessed 15 October 2015.

Federal Standard 376B. *Preferred Metric Units for General Use By the Federal Government*. 5 May 1983. <http://www.nist.gov/pml/wmd/metric/upload/fs376-b.pdf>, accessed 15 October 2015.

Public Law 91-190. *National Environmental Policy Act of 1969.* 1 January 1970. <http://www.gpo.gov/fdsys/pkg/STATUTE-83/pdf/STATUTE-83-Pg852.pdf>, accessed 15 October 2015.

TM 5-820-3/AFM 88-5. *Drainage and Erosion-Control Structures for Airfields and Heliports*. 3 June 1991.

UFC 3-220-04FA. *Backfill for Subsurface Structures.* 16 January 2004.

UFC 3-220-10N. *Soil Mechanics.* 8 June 2005. Web site <http://www.wbdg.org/ccb/DOD/UFC/ufc_3_220_10n.pdf>, 19 January 2016.

UFC 3-230-01. *Water Storage, Distribution, and Transmission.* 1 November 2012. Web site <http://www.wbdg.org/ccb/DOD/UFC/ufc_3_230_01.pdf>, accessed 19 January 2016.

UFC 3-250-01FA. *Pavement Design for Roads, Streets, Walks, and Open Storage Areas*. 16 January 2004.

UFC 3-250-03. *Standard Practice Manual for Flexible Pavements.* 15 May 2001.

UFC 3-250-09FA. *Aggregate Surfaced Roads and Airfields Areas.* 16 January 2004.

UFC 3-250-11. *Soil Stabilization for Pavements.* 16 January 2004.

UFC 3-260-01. *Airfield and Heliport Planning and Design.* 17 November 2008.

UFC 3-260-02. *Pavement Design for Airfields.* 30 June 2001.

UFC 3-260-03. *Airfield Pavement Evaluation,* 15 April 2001.

UFC 3-270-01. *Asphalt Maintenance and Repair.* 15 March 2001.

UFC 3-270-03. *Concrete Crack and Partial-Depth Spall Repair.* 15 March 2001.

UFC 3-270-04. *Concrete Repair.* 15 March 2001.

UFC 3-270-05. *Paver Concrete Surfaced Airfields Pavement Condition Index (PCI).* 15 March 2001.

UFC 3-270-06. *Paver Asphalt Surfaced Airfields Pavement Condition Index (PCI).* 15 March 2001.

UFC 3-270-07. *O&M: Airfield Damage Repair.* 12 August 2002.

UFC 3-270-08. *Pavement Maintenance Management.* 16 January 2004.

UFGS-32 11 16.16. *Unified Facilities Guide Specifications Exterior Improvements.* November 2011. Web site <http://www.wbdg.org/ccb/DOD/UFGS/UFGS%2032%2011%2016.16.pdf>, accessed 19 January 2016.

UFGS-33 46 16. *Unified Facilities Guide Specifications Utilities.* April 2008. Web site <http://www.wbdg.org/ccb/DOD/UFGS/UFGS%2033%2046%2016.pdf>, accessed 19 January 2016.

Whole Building and Design Guide. <www.wbdg.org/ccb/ccb.php>, accessed 26 January 2016.

WEB SITES

Natural Resources Conservation Service, National Water and Climate Center Web site <http://www.wcc.nrcs.usda.gov/>, accessed on 15 October 2015.

USACE Hydrologic Engineering Center Web site < http://www.hec.usace.army.mil/>, accessed on 15 October 2015.

PRESCRIBED FORMS

None.

REFERENCED FORMS

Unless otherwise indicated, DA Forms are available on the Army Publishing Directorate (APD) Web site: www.apd.army.mil. DD Forms are available on the Office of the Secretary of Defense (OSD) Web site: http://www.dtic.mil/whs/directives/forms/index.htm.

DA Form 1248. *Road Reconnaissance Report.*

DA Form 1711. *Engineer Reconnaissance Report.*

DA Form 2028. *Recommended Changes to Publications and Blank Forms.*

DD Form 2641. *Trafficability Test Data.*

RECOMMENDED READINGS

TM 5-803-5/NAVFAC P-960/AFM 88-43. *Installation Design.* 1 March 1981.

TM 5-822-5/AFM 88-7, *Pavement Design for Roads, Streets, Walks, and Open Storage Areas.* 12 June 1992.

UFC 3-230-02. *Operation and Maintenance: Water Supply Systems.* 10 July 2001.

Whole Building Design Guide. Web site <http://www.wbdg.org>, accessed on 15 October 2015.

Index

Entries are by page number.

A

abnormal strength profile, 10-9, 10-10

aggregate, 2-3, 2-4, 5-3, 5-9, 5-13, 5-14, 5-15, 5-16, 5-17, 5-19, 5-22, 5-28, 7-5, 11-2, 11-7, 11-8, 12-1, 12-36, 12-45, 12-46, 12-51, 12-52, 12-53, 12-55, 12-56, 12-57, 12-58, 12-59, 12-60, 12-61

air base damage repair, 11-17, 11-18

airfield and heliport maintenance, 11-17

airfield construction, 1-1, 1-2, 2-2, 2-4, 3-1, 3-13, 4-1, 4-5, 4-15, 5-1, 6-1, 7-1, 7-20

alignment, 2-3, 3-2, 3-3, 6-4, 6-14, 7-25, 7-27, 8-4, 8-6, 12-6, 12-17, 12-25, 12-40, 12-41, 12-45

alluvial terraces, 2-2

asphalt distributor, 5-16, 12-55, 12-61

asphalt pavement, 6-23, 12-48

asphalts and tars, 12-49

average depth of cut or fill, 3-13

average end area, 3-13

average haul distance, 3-20, 3-22, 3-23, 3-25

B

balance lines, 3-22, 3-23, 3-24, 3-26

base course, 2-1, 2-2, 5-1, 5-3, 5-7, 5-10, 5-12, 5-13, 5-14, 5-15, 5-16, 5-17, 5-18, 5-19, 6-4, 8-37, 8-38, 8-39, 8-41, 10-4, 11-1, 11-9, 12-1, 12-46, 12-55, 12-56

base flow, 2-13, 6-11, 6-13

baton, 11-11

baton method, 11-11

berm, 6-4, 7-16, 7-17, 11-3

best management practices (BMPs), 6-1

bituminous, 5-14, 5-16, 5-22, 5-27, 5-28, 8-1, 8-2, 11-6, 11-7, 11-8, 11-11, 11-15, 11-16, 12-1, 12-45, 12-46, 12-49, 12-51, 12-52, 12-53, 12-54, 12-57, 12-58, 12-60, 12-62

bituminous construction, 12-51

bituminous materials, 5-9

bituminous surface, 11-7, 11-8, 11-15, 11-16, 12-58, 12-62

blind drain, 8-40

box-culvert flow, 8-21

bridge, 11-11, 11-12, 11-16, 12-2, 12-25

bridge approach, 2-1, 2-4

C

calcium chloride, 5-21, 8-41, 11-7, 11-15, 11-21

camouflage, 2-5, 2-8, 4-4, 4-5, 4-8, 4-15, 11-18, 11-20, 11-22

causeway, 2-12, 2-16, 2-17, 2-18

channel, 1-1, 6-1, 6-4, 6-5, 6-11, 6-13, 6-14, 6-22, 6-29, 7-4, 7-5, 7-15, 7-16, 7-17, 7-18, 7-19, 7-27, 7-28, 7-30, 8-5, 8-15, 8-39, 9-3, 9-4, 9-6, 11-2

channel flow, 7-4, 8-6, 8-15

check dam, 7-28, 7-30, 7-31, 11-2, 11-3

climate classifications of forest, 4-1

coarse-grained, 5-9, 5-21, 5-22, 10-1, 10-10, 10-11, 10-28, 10-29, 10-30, 10-35

coarse-grained soil, 10-28

compacted, 2-2, 2-15, 3-18, 4-16, 5-3, 5-4, 5-5, 5-6, 5-9, 5-12, 5-14, 5-15, 5-16, 5-17, 5-20, 6-3, 7-21, 11-3, 11-5, 11-6, 11-7, 11-8, 11-15, 12-42, 12-52, 12-53, 12-62

compacted foundations, 8-13

compacted gravel, 6-27, 6-30, 6-37

compacted layers, 8-15, 12-42

compacted material, 3-19, 7-31, 11-5

compacted soil, 5-6, 8-7, 8-14

compaction, 3-18, 4-13, 4-15, 5-1, 5-3, 5-4, 5-5, 5-6, 5-9, 5-12, 5-14, 5-15, 5-16, 5-17, 5-18, 5-19, 5-20, 5-21, 5-27, 5-28, 8-4, 8-7, 11-5, 11-9, 12-40, 12-41

computer programs
- Hydrologic Modeling System, 6-40
- TR-20, 6-40
- TR-55, 6-40

cone index, 10-1, 10-2, 10-5, 10-10, 10-12, 10-17, 10-29

cone penetrometer, 2-23, 10-1, 10-2, 10-4, 10-5, 10-7, 10-29

construction stake, 3-1, 3-2, 3-3, 12-18, 12-30

construction survey, 3-1

continuity equation, 7-5, 8-19

coral, 5-9, 5-12, 5-13, 11-8

corduroy, 11-16

corrugation, 11-4, 11-5, 11-7

critical layer, 10-5, 10-6, 10-7, 10-9, 10-10, 10-11, 10-17, 10-18, 10-20, 10-21, 10-22, 10-23, 10-24, 10-25, 10-26, 10-27, 10-28

critical layer depth variation, 10-5

critical slope, 2-21, 8-20

cross section, 3-1, 3-8, 3-9, 3-10, 3-12, 3-14, 3-15, 5-15, 6-17, 6-40, 7-1, 7-7, 7-10, 7-12, 8-3, 8-23, 8-25, 8-41, 12-2, 12-38, 12-53

cross section at a station, 3-7

cross section at bridge site, 2-17

cross section for Class A road, 12-36

cross section of a ford, 2-14

cross section of a stream, 2-17

cross section of a typical runway gutter, 7-15

cross section of a waterway, 2-18
cross section of an artificial channel, 8-6
cross section of an outlet channel, 7-29
cross section of dip section, 2-15
cross section to maintain drainage, 11-3
cross section type, 7-9
crowned section, 12-36
culvert, 6-5, 6-11, 6-12, 6-14, 6-15, 6-22, 6-36, 7-1, 7-4, 7-9, 7-18, 7-28, 7-29, 8-1, 8-2, 8-3, 8-4, 8-6, 8-7, 8-8, 8-13, 8-14, 8-15, 8-17, 8-18, 8-21, 8-23, 8-32, 8-33, 9-4, 11-2, 11-3, 11-11, 11-12, 11-16
culvert alignment, 8-4
culvert outlet, 7-29, 8-6, 8-17, 8-23, 8-25
curve table, 12-9, 12-23
cut operation, 3-22
cutback, 5-22, 5-27, 12-46, 12-48, 12-49, 12-50, 12-51, 12-53, 12-55, 12-57

D

defilade position, 2-5
degree of protection, 6-6
design capacity, 6-6
design storm frequencies higher than minimum, 6-6
ditching, 6-4, 6-5
drags, 11-2, 11-4, 11-6, 11-14
drainage pipe definition, 8-1
drop inlet and grating maintenance, 8-36
drop inlets and gratings, 8-33
drop inlets and gratings construction, 8-33
dust control, 4-15, 5-19, 5-27, 11-6, 12-45

E

earthwork volume sheet, 3-19, 3-20
edge raveling, 11-8
emulsion, 5-22, 5-23, 5-24, 5-27, 12-46, 12-49, 12-51
entrance, 8-33, 11-12
environmental conditions, 4-1, 5-24
erosion control, 4-15, 7-18, 7-19, 7-30, 8-4, 8-15, 11-3
estimating runoff, 6-11, 6-22, 6-36
existing facilities, 1-1, 2-1, 2-23, 11-22

F

fabric, 5-17, 8-45, 8-46
field identification, 10-2, 12-49, 12-51
fill operation, 3-22
fine-grained soil, 5-9, 5-20, 5-21, 5-23, 9-4, 10-1, 10-2, 10-6, 10-7, 10-9, 10-10, 10-11, 10-17, 10-18, 10-19, 10-20, 10-21, 10-22, 10-23, 10-24, 10-25, 10-26, 10-27, 10-28, 10-29, 10-30, 10-35
flexible-pavement structure, 5-1, 5-2, 5-5, 5-6, 5-10
flight-way obstruction, 2-1, 2-4
ford, 2-12, 2-13, 2-14, 2-15, 2-16, 2-17, 2-18
fords and bridges, 11-16
forest types, 4-1
French drain, 8-40
frost boils, 11-16
frost heaves, 11-16
frost susceptibility, 5-6, 5-7
future expansion, 2-1, 2-5, 2-6, 5-6

G

gabions, 2-13, 2-17, 7-19, 7-24, 7-25, 7-27, 7-28
geologic and permafrost conditions, 4-3
geology, 2-1, 2-2
geometric design process, 12-2
geotextile, 2-4, 5-7, 8-45, 8-46
glide angle, 2-4, 2-19, 2-23, 4-6, 4-15
gradation, 2-10, 2-19, 5-1, 5-6, 5-7, 5-9, 5-10, 5-12, 5-13, 5-14, 5-15, 5-19, 5-20, 5-21, 5-22, 5-23, 5-24, 7-23, 8-42, 11-7
gradation requirement, 5-9, 5-10, 5-13
grade determination, 12-25
grade line, 3-4, 3-9, 3-18, 3-21, 3-22, 5-4, 12-25, 12-26, 12-27, 12-28, 12-29
grader, 4-5, 4-6, 4-13, 5-15, 5-16, 5-18, 5-19, 5-20, 5-27, 7-1, 7-2, 7-17, 11-2, 11-4, 11-6, 11-10, 11-14, 11-15, 11-16, 11-20
grating, 7-15, 8-33, 8-35, 8-36, 12-43
grid, 3-15, 3-17, 5-6, 5-7, 6-19
ground cover, 2-1, 2-4, 4-4, 6-6, 6-13, 6-24, 11-3
ground icing, 9-4, 9-5
gunite lining, 7-19, 7-20
gutters, 7-15, 7-20, 8-33

H

hardpan or rock, 4-3
hot-mixed asphalt, 5-1
hot-mixed asphalt and concrete, 11-17
hydraulic gradient, 8-17, 8-18, 8-20, 8-37
hydraulic radius, 7-5, 7-7, 7-9, 7-11, 7-12, 7-13, 7-19
hydraulic radius calculation, 7-7
Hydrologic Engineering Center-River Analysis System, 6-40
hydrophobic, 6-13, 12-51, 12-52

I

ice control, 11-15, 11-20, 11-21
icing types, 9-4
interception ditch, 6-5
inundated, marshy, and boggy areas, 4-3
isohyetal map, 6-8, 6-9, 6-10, 6-11, 6-24, 6-30, 6-32

L

lag time, 6-11, 6-12
land clearing, 4-1, 4-4, 4-13
landing mats, 11-16
layout techniques, 12-22
length of curve, 12-11, 12-17, 12-33, 12-34
lines of communication, 2-2, 2-9, 2-10
lines of communication (LOC), 2-1, 2-3
liquid limit, 5-10
liquid limit (LL), 5-1, 5-9, 5-10, 5-14

load distribution, 5-2, 10-12, 12-1
location and design, 2-1, 2-2
location factors, 2-1

M

macadam, 2-15, 2-16, 5-13, 5-17, 11-10
maintenance during flying operations, 11-21
Manning's velocity of flow, 7-5, 7-6, 7-7, 7-8, 7-9, 7-11, 7-12, 7-27
marginal material, 2-1, 2-4
mass diagram, 3-1, 3-19, 3-20, 3-21, 3-22, 3-23, 3-24, 3-26, 3-27, 3-29
mass diagram construction, 3-20
mass diagram limitations, 3-29
mass diagram properties, 3-21
measures against icing, 9-5
median, 7-15
middle ordinate, 12-8, 12-9, 12-10, 12-12, 12-23
minimum rehabiltation, 2-2
mobility index (MI), 10-12, 10-13, 10-15, 10-17
mud control, 11-2, 11-18

N

natural material, 5-12, 5-15, 7-23, 7-29
net volume, 3-18

O

obstacle crossing, 2-1, 2-3, 2-4
obstacles, 2-3, 2-4, 2-19, 2-23, 6-6, 10-1, 10-11, 10-33
Office of the Chief of Engineers, 2-7
one-pass performance, 10-18, 10-19, 10-21, 10-22, 10-26, 10-29
organic-soil area, 10-11

P

patches, 4-12, 11-8, 12-62
pavement, 1-1, 2-2, 2-4, 2-15, 3-6, 5-1, 5-2, 5-3, 5-5, 5-6, 5-7, 5-10, 5-12, 5-13, 5-14, 5-16, 5-17, 5-18, 5-19, 5-20, 5-26, 5-27, 5-28, 6-1, 6-4, 6-8, 6-23, 6-26, 7-15, 7-20, 8-1, 8-28, 8-29, 8-37, 8-41, 8-45, 9-3, 9-5, 11-2, 11-3, 11-8, 11-9, 11-10, 11-11, 11-12, 11-15, 11-16, 11-17, 11-21, 11-22, 12-37, 12-38, 12-43, 12-48, 12-49, 12-51, 12-52, 12-53, 12-54, 12-55, 12-60, 12-62, 12-63
paving, 5-16, 7-19, 7-20, 8-1, 11-3, 12-49, 12-51
peak flow, 6-11, 6-12, 6-13, 6-17, 6-22, 6-35, 6-40, 7-7, 7-9, 7-17
pending, 12-56
permafrost, 4-3, 4-4, 5-7, 5-15, 6-3, 9-3, 9-4, 9-5, 9-6
permissible volume, 8-29
pipe
  selection, 8-1, 8-2
    bituminous-coated XE "bituminous" pipe restrictions, 8-1
    life cycle cost factors, 8-1
plain outlet, 7-29
planking, 2-14, 11-11
planks, 11-16
planning considerations, 1-1
plasticity index (PI), 5-1, 5-10, 5-13, 5-14, 5-15
pneumatic roller, 5-18, 12-60
pneumatic tamping tool, 11-9
pneumatic-tire, 5-18
pneumatic-tire roller, 5-19, 12-61
preconstruction phase, 2-1
preswelling, 5-6
prime, 12-53
prime coat, 12-45, 12-53, 12-54, 12-55, 12-56
prismoidal formula, 3-13
processed material, 5-9, 5-13, 11-7

R

radius of curvature, 12-7, 12-10
rating, 10-2, 10-7, 10-10, 10-11, 10-17, 10-19, 10-20, 10-21, 10-22, 10-23, 10-24, 10-28, 10-32, 10-34
reconnaissance, 1-1, 1-2, 2-1, 2-5, 2-6, 2-7, 2-8, 2-9, 2-10, 2-11, 2-12, 2-15, 2-18, 2-19, 2-20, 2-21, 2-23, 2-26, 3-1, 3-2, 3-13, 4-1, 4-3, 6-3, 6-13, 9-3, 10-5, 10-7, 11-11, 11-12, 11-22
referencing point, 3-15
rehabilitation of captured airfields, 11-22
remoldable sand, 10-5, 10-6, 10-7, 10-9, 10-10, 10-11, 10-30
required area, 2-1, 2-4
revetment, 4-5, 7-28
riprap, 12-42, 12-43, 12-44
river icing, 9-4, 9-5
road, 5-5, 12-1, 12-5, 12-8, 12-9
road maintenance, 11-9
road tar, 12-49
roughness coefficient, 2-14, 7-5, 7-7, 7-9, 7-11, 7-15, 7-16, 7-20, 7-21, 8-3, 8-32
rubble, 5-13, 5-14, 7-21, 7-23, 11-8, 12-62

S

safe-slope ratio, 2-3
sand grid, 5-7
scour, 6-22, 7-17, 7-21, 7-22, 7-23, 7-28
sediment control, 7-17
sedimentary rock, 2-3
select material, 5-7, 5-8, 5-9, 5-19
shrinkage, 3-18
site selection, 1-1, 2-1
site selection phases, 1-1
slipperiness, 10-2, 10-11, 10-18, 10-19, 10-20, 10-32, 10-34
snow and ice, 10-30, 11-12
snow and ice removal, 11-2
snow removal, 11-14, 11-15, 11-20, 11-21
soil, 5-5, 5-7, 5-19, 5-21, 10-1, 10-2, 10-5, 10-6, 10-10, 10-11, 10-17, 10-18, 10-29, 10-32, 10-35
soil characteristics, 2-1, 2-2, 6-13, 8-41
sprayed, 12-45
spring icing, 9-4, 9-5
standard, 6-10
standards of trafficability, 2-11
stilling basin, 7-20, 7-29
stream velocity, 2-14
strength profile, 10-7, 10-9, 10-10
stripping, 12-52

stripping binder, 12-52
stripping test, 12-52
structural design, 2-17
subbase course, 5-7, 5-9, 5-19
subbase material, 5-7, 5-9, 5-10, 8-45
subgrade, 2-1, 2-2, 2-5, 2-15, 3-3, 3-7, 4-15, 5-1, 5-2, 5-3, 5-4, 5-5, 5-6, 5-7, 5-9, 5-10, 5-15, 5-17, 5-18, 5-19, 6-4, 6-5, 8-28, 8-36, 8-37, 8-43, 8-45, 9-3, 10-4, 11-1, 11-2, 11-3, 11-7, 11-8, 11-9, 11-12, 11-16, 12-1, 12-36, 12-63
subgrade drainage, 8-37
subgrade drains, 8-41
subgrade failure, 4-4, 11-12, 11-20
subgrade material, 11-20, 12-43
subgrade preparation, 5-17
subgrade soil, 8-43, 8-44, 8-45, 11-16
subgrade soil characteristics, 8-41
subgrade structures, 11-12
subgrade-compaction operation, 5-16
subsurface drainage, 6-3, 6-5, 8-1, 8-36, 8-37, 8-38, 8-40, 8-41, 11-5, 12-45
successive area, 6-29, 6-35, 6-36
successive area runoff estimation, 6-36
superelevation, 7-29, 11-2, 11-6, 12-2, 12-37
surface runoff, 6-8
surface treatment, 12-45, 12-46, 12-53, 12-55
survey, 1-2, 2-2, 2-6, 2-11, 2-21, 2-23, 3-1, 3-2, 3-5, 3-6, 3-7, 4-15, 5-3, 6-13, 12-2, 12-12, 12-17, 12-18, 12-23, 12-24, 12-25, 12-30, 12-32
swell test, 12-52
swelling soils, 5-6

T

tack coat, 12-45, 12-53, 12-54, 12-55
tactical considerations, 2-1, 2-5
tangents, 2-3, 3-4, 12-2, 12-6, 12-7, 12-8, 12-11, 12-17
temporary repair, 11-1, 11-8, 11-10, 12-62
terracing, 7-19, 11-3
timber cruising, 4-4
traffic control, 11-11, 12-61
transition point, 3-22
tuff, 5-14, 12-45
turbulent flow, 6-27, 7-5
turf surfaces, 11-18
turfing, 7-19, 7-20

U

unsubmerged inlet, 8-24
unsurfaced, 7-15
unsurfaced road construction, 5-7

V

vegetation, 2-2, 2-4, 2-7, 2-8, 2-13, 2-14, 3-6, 4-1, 4-3, 4-12, 4-13, 6-3, 6-4, 6-6, 6-11, 6-12, 6-30, 8-2, 10-1, 10-11, 10-29, 10-30, 10-32, 11-3, 11-12
vehicle cone index, 10-12
vehicle cone index (VCI), 10-10, 10-12, 10-17, 10-29
velocity relationship, 7-7
vertical alignment, 12-6, 12-25, 12-40
volume, 2-12, 2-15, 3-6, 3-7, 3-13, 3-15, 3-16, 3-17, 3-18, 3-19, 4-5, 5-6, 5-15, 6-11, 6-26, 6-40, 7-17, 8-1, 8-21, 8-28, 8-29, 8-30, 8-31, 8-32, 8-33, 11-1, 12-2, 12-5, 12-6, 12-25, 12-55
volume calculation, 3-19
volume compacted, 3-18, 3-19
volume determination, 3-7
volume determination methods, 3-12
volume for ponding, 8-32
volume in-place, 3-18
volume loose, 3-18
volume of cut, 3-18
volume of cut, 3-19
volume of fill, 3-19, 12-25
volume of flow, 2-15, 2-16, 6-12
volume of material, 3-13, 3-19
volume of ponding, 8-29
volume of runoff, 7-15
volume of storm water, 7-9, 7-10
volume of traffic, 6-6, 11-7, 12-25, 12-56
volume of water, 7-7
volume sheet, 3-19

W

washboarding, 11-4, 11-5
watershed, 6-15, 6-17, 6-19, 6-20, 6-22, 6-26, 6-29, 6-31, 6-40
weather conditions, 4-1, 5-14, 5-28, 10-1, 10-30, 10-33, 12-56, 12-60
weir notch, 7-30, 7-31, 7-32, 11-3
wet season, 5-6, 10-30, 11-8
wet weather, 11-6
windrowing, 4-10, 4-12
wire-mesh baskets, 7-24
witnessing point, 3-6

ATP 3-18.2
29 February 2016

By Order of the Secretary of the Army:

MARK A. MILLEY
General, United States Army
Chief of Staff

Official:



GERALD B. O'KEEFE
Administrative Assistant to
the Secretary of the Army
1603506

DISTRIBUTION:
Active Army, Army National Guard, and United States Army Reserve: Distributed in electronic media only (EMO).